EWRA

Proceedings

Water Resources Conservancy and Risk Reduction Under Climatic Instability

European Water Resources Association
Seventh International Conference

Limassol, Cyprus
25 – 27 June 2009

Editor:
Dr. S.E. Christodoulou



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Organized by:
The University of Cyprus
Department of Civil and Environmental Engineering

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"Water Resources Conservancy and Risk Reduction Under Climatic Instability"

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Proceedings, EWRA Seventh International Conference on Water Resources Conservancy and Risk Reduction Under Climatic Instability

Edited by Dr. S.E. Christodoulou

This book contains the full-length papers presented at the European Water Resources Association's Seventh International Conference, held in Limassol, Cyprus (25 - 27 June 2009). The Conference Proceedings is accompanied by a CD containing digital copies of the aforementioned full-length papers.

The conference, which was hosted by the University of Cyprus, Department of Civil and Environmental Engineering, focused on the following thematic areas:

Water Related Natural Hazards: Proactive Planning and Crisis Management

- Droughts / Floods
- Water Shortage
- Water Pollution
- Landslides
- Dam Break
- Sea Water Intrusion into Coastal Aquifers

Water Conservation: Management & Technology – Public Learning & Participation

- Urban Water Supply Systems
- Irrigation
- Water Reuse
- Brackish Water – Water Treatment
- Desalination
- Energy Conservation
- Groundwater Resources
- Decentralized Wastewater Treatment
- Integrated Water Resources

Macro-engineering and Environmental Governance

- Water Conveyance
- Transboundary River Basins
- Coastal Areas
- Energy Production and Distribution
- Mega-cities and Population Movement
- Transportation Systems
- Climate change impacts and Adaptation
- Water and Society

The book also contains contributions by the SMART Research Consortium whose scientific coordination meeting on Sustainable Management of Available Water Resources with Innovative Technologies (SMART) was held within and in parallel with EWRA's conference.



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Disclaimer:
Although this Conference Proceedings Book and the accompanying CD with the full-length manuscripts have been compiled with the outmost care, the Editor cannot be held responsible for any misprints, omissions or typographical problems during the conversion and CD creation process.

Prolegomena

G. Tsakiris
President of the European Water Resources Association (EWRA)

It is true that water is the most abundant natural resource on earth. It is also true that for hundreds of years in the past it was commonly taken for granted as nature's gift. The value of water has been regarded by philosophers and scholars in ancient times as the "most precious resource" and "the driver of nature".

Long after the industrial revolution, water problems gradually emerged related to water scarcity, water pollution from natural hazards caused by extreme phenomena of the hydrological cycle. During the last century an increasing abuse of water resources was practiced together with expanding population, industrialisation and urbanisation, which increased the burden on governments and professionals to safeguard water quantity and quality for the domestic, agricultural and other economic sectors. The booming irrigation development in most countries created more stress in the water sector since the "requirements" were multiplied. Big water projects together with the expansion of economic activities, created a higher but also more vulnerable equilibrium between availability and demand. The consequence of this "ever"-continuing tendency seemed to reach a deadlock due to the new problems of increased pollution, overexploitation of aquifers and stagnation of water sources.

The approaching global water crisis was predicted by scientists in conferences and symposia from the early seventies. However, governments and international organisations seemed to be reluctant to realise and appreciate the seriousness of the global water situation.

Even in the International conferences organised by the United Nations in Dublin and Rio de Janeiro in the early nineties, water was a rather minor issue and was discussed indirectly. The primary interest of the participating leaders of state was focused on issues such as climate change, biodiversity and deforestation.

Recently, an increasing number of states started to recognise that water problems are multiplying and that several anthropogenic causes lead to deterioration in the water availability and use. Climate change is the most recent threat to water conservation, worldwide. According to the IPCC projections during the next decades, the magnitudes and the patterns of precipitation and air temperature are expected to change in most parts of the world. Expected sea level rise also threatens most of the coastal aquifers and the water security of most coastal zones.

It is fully understood that the "climate change" hazard is pressing the world to curb emissions of CO2 and other greenhouse gases which are blamed for the climate change. We all hope to reach a consensus during the Copenhagen World Summit and a brave new agreement will be signed replacing Kyoto protocol.

It is also fully understood that water and the environment will be affected mostly by the anticipated climate change. However, whatever will be decided in Copenhagen on energy and emissions, it seems indirect and inadequate for solving the water problems which exist and will increase with or without the various trends in gas emissions.

In short we believe that "water problems" are affected by climate change but can be also dealt with by more direct and efficient methods. In this context, although EWRA participates actively in

the ongoing dialog on climate change, it focuses its attention on direct actions on water problems (availability and demand) at the national, regional and international level. Our activities (meetings, symposia, conferences, journals and publications) reflect exactly this priority.

The EWRA Conference in Cyprus is expected to enrich the knowledge among academics and professionals as to how to improve the planning and management of water resources in an integrated, equitable, sustainable and adaptive way to face present and future water problems. In this respect and focusing on reducing the risk from water related hazards, EWRA systematically promotes proactive and preparedness planning with the most democratic procedures through the involvement of all stakeholders, aiming at tackling water problems efficiently and comprehensively, while protecting the environment.

Looking at the topics, the papers submitted, and the list of participants, I am sure that the conference will be fruitful and useful for the participants, guiding our actions directly against "problems and causes" with the objective of securing water availability and quality for future generations in our countries.

Welcome Note

S.E. Christodoulou
Conference Host and Chair

Dear Friends and Colleagues,

Nowadays, amidst global climatic change, extended periods of drought, heightened demand of water and increased urbanization, the development and application of scientific knowledge and integrated water resources management policies for sustainable management of aquatic resources is of paramount importance not only to national governments but also to transnational networks. Such scientific knowledge and policies should cover a myriad of issues related to water, such as water scarcity and conservancy, climate change and their effects on water, drought and floods, water quality, water recycling, water loss, wastewater treatment, commodification of water, water pricing, etc.

The following sections discuss most of the aforementioned important components of sustainable management of aquatic resources, as presented at **EWRA's Seventh International Conference** titled "**Water Resources Conservancy and Risk Reduction Under Climatic Instability**".

The conference aimed to provide an international forum for the exchange of scientific information and knowledge-sharing on state-of-the-art research efforts and on contemporary water-related issues, on a multitude of thematic areas such as water management and technologies, macro-engineering and environmental governance, proactive planning and crisis management, sustainable development, urban water supply systems, water losses, water shortage and water reuse, water pollution, climate change, and IWRM.

The work presented and included in the conference proceedings constitutes an excellent blending of cutting-edge research, of scientific and applied knowledge, and of case-studies across the globe which should be of great interest to both researchers and practitioners since it offers the European and the international community of water professionals at large a great opportunity to experience the latest achievements in water-related research, science, practice and management.

This great experience starts with your participation in a state-of-the-art technical programme and culminates in a unique opportunity to, not only immerse yourself in cutting-edge water-related science, but also to engage other professionals in discussions, possible collaborations and future scientific endeavors for the advancement of knowledge in water resources.

Finally, a great venue and an even greater host-city can only add value to the conference and guarantee the success and fun of it. We are definite that the city and the people of Limassol will make this a memorable experience to all participants and we invite you all to take full advantage of country's historical, cultural, urban and tourist assets. Conference delegates, family and friends alike should take advantage of the great opportunities the city provides and the ideal weather conditions to not only enjoy the sun and the sandy beaches of Limassol, but to also experience the full range of activities and excursions in Cyprus.

For all the above reasons and many more, we warmly invite you to Limassol, Cyprus, for EWRA's 7th International Conference and wish you the best and most memorable stay.

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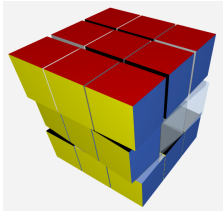
The Organizers wish to extend their gratitude to the following public and private institutions for financially contributing to the success of the conference.

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MATHEMATICAL APPROACH TO MANAGEMENT OF WATER AND ENERGY RESOURCES IN CENTRAL ASIA

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Abstract: The problem of complex use of water resources in Central Asia by taking into consideration the sovereignty of the states and increasing demand on use of water for economic are considered. Complex program with appropriate mathematical software intended for calculation of possible variants of using the Amudarya up-stream water resources according to satisfaction of incompatible requirements of the national economics in irrigation and energy generation is proposed.

Key words: water resources, national economics, irrigation, transboundry, energy generation, optimal solution, program complex, up-stream, down-stream, compensating services.

1. INTRODUCTION

Problem of joint management of transboundry water resources and energy generation for Central-Asian republics are main factors, which defined practically all aspects of the national and regional economics.

Up to the recent time the Central-Asian republics have exploited their water resource systems within the distributive scheme. This scheme considered the region as a space controlled and guided by the center. Former mechanism of centralized management of the water recourses provided reliable work of all hydro technical constructions coordinating solutions of problems on regional water using within multiyear regulation of river flow. This mechanism has been guaranteed for stable function of agricultural sector based on irrigation and safety flow of water to river downstream during winter-spring period. Each republic supported the rational balance between energetic and other resources, which contribute for regulation of delivery the energy supply and appropriate distribution of water recourses between republics.

However, presently the political and economic situation in the region has radically been changed. After proclamation of independence, each of the sovereign state tends first of all to maximize the use of all natural resources, taking into account the own economic and political interests.

Thus, the main water potential of Central Asia is concentrated in the basin of two big rivers the Syrdarya and the Amudarya which provide needs of population of five countries (Kazakhstan, Kyrgyzstan, Tajikistan, Turkmenistan and Uzbekistan). The interrelation of the countries in the region depends on true choosing of water resources management strategies.

There are certain disagreements between the states of the Amudarya and the Syrdarya transboundary upstream and dounstream river basin according to energy potentials in up-stream rivers. As an example, Kyrgyzstan completely determines the policy of use the Syrdarya basin water resources being formed in its territory. Other Central-Asian republics, using water resources of the Syrdarya river, are compelled to be satisfied with it. Such the situation has initiated the introduction of compensating services between the republics of the region.

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However, the certain interstate arrangements are not undertaken than the finalizing construction of the big hydropower stations which started in 80th years (Cambaratin in Kyrghyzstan, Sangtuda and Rogun hydroelectric power station in Tadjikistan) can change essentially a regim of Syrdarya and Amudarya river which they had been earlier.

Situation in the Amudarya river basin can become more aggravated, if we take into account Afghanistan intentions in the near future to act with requirements on participation in water distribution in Pyandj river basin. As a result the volume of water will be reduced for Turkmenistan and Uzbekistan especially in the Amudarya basin downstream.

The main problems existing between Central-Asian republics in the field of water and energy regulation are:

- contradictions between interests of the states which are in upstream and downstream of the river;
- absence of interstate structures in sphere of a joint management of the water-energy resources, allocated corresponding authorities;
- recommendatory character of making decisions and absence of any responsibility for their execution at the level of the created regional management structures;
- inconsistency of actions at regional and national levels between management structures of water economic and power engineering;
- absence of arbitration practice between countries which have conflict concerning distribution of water resources with opportunities of real influence to the solution of this problem.

2. PROBLEM SOLUTION

For decision of these problems it is important to take into consideration of the realizations of the following main positions:

- formation of the coordinated regional policy of fair and reasonable development and use of water-energy resources based on international law with participation of all countries of the region;
- complex assimilation of hydropower energy potential and regulation of water and energy resources and maintenance synchronism of their decisions;
- production of effective economical and legal mechanisms of co-operation which are guaranteed caring out accepted decision;
- working out the complex of mathematical models of the accepted decisions, allowing modeling various scenario of water using in the basin of transboundry river.

Proceeding from this in 1992 Institute of Mathematics of Tajik Academy of Sciences brought in Tajik Government the proposal on developing a substantiated strategy to using the Vakhsh river water resources. That proposal was taken and a series of governmental institutions was associated to realizing the project suggested. Institute of Mathematics prepared the computing complex intended for calculation of possible variants to using the Vakhsh river water resources with regard for satisfaction of incompatible requirements of the national economics in irrigation and power engineering. The first computing experiences were completed (Navruzov 1986). But the civil war that enveloped Tajikistan for a long time interrupted the elaboration of the water strategy for our Republic.

The terminal aim presupposed to be achieved at the process of developing the problem is contained in forming the state strategy to a rational use of the Amudary basin up-stream Central-Asian republics. Principal aspects of the strategy should be brought to the information of other Central-Asian republics in order to conclude an interstate agreement on using the Amudary river water resources. In this connection the problem of a complex use of the Central Asia water resources will be completely decided by taking into consideration the sovereignty of the states and their market interrelations.

The project suggested by author is directed to achieving the purpose mentioned. While realizing the project the following results will be obtained:

- the computer instrumentality for conducting the calculation such as to get acceptable variants of utilizing the Amudarya basin up-stream water resources by managing the activity of the reservoirs cascade existing;
- the computer instrumentality for conducting an analogous multi-variant calculation in case the Rogun's reservoir is constructed and joined to the reservoirs cascade for a combined regulation.

The instrumentality will allow:

- to carry out an expert selection of reasonable variants to using the water resources;
- to develop recommendations for the Tajik Government to realizing the new water use policy.

To get the mentioned results the following actions are planned:

- gathering, systematizing and integrating the hydrologic, hydro-geological and morphometric data on the Amudary basin up-stream and existing modes of the hydrounits ;
- creating a database of the information;
- qualitative and quantitative estimations of the Amudarya up-stream water resources, i.e. the water resources of the Vakhsh river;
- developing the program complex for management of the Vakhsh cascade with regard to priority of the Tajik Republic requirement in electricity (in two variants: both for the existing cascade of reservoirs and for the cascade extended by the Rogun's reservoir);
- realization of computing experiments for a qualitative estimation of efficiency of different managing decisions to functioning the Vakhsh cascade of reservoirs;
- creating information system of the Amudary transboundry river with using of the Geographical Information System (GIS)

3. MATHEMATICAL SOFTWARE

Three levels of mathematical models are offered: *analytical, optimization and imitational*. Within the framework of analytical models (Navruzov 1990, 1991) the theoretical-game models of distribution of water resources between the states of the Amudary basin are considered. A number of characteristic examples of games with not opposite interests concerning to problem of rational use of water resources of the transboundry basin are discussed (Navruzov 2008).

Water-division problem for the transboundry basin is spent from the point of view of the system analysis and a model of substantiations of cooperation of countries-users of water resources of the transboundry river is offered. Suggested approach is focused on research of efficiency of cooperation at management of water resources on a qualitative level.

For a finding effective (optimum on Pareto) ways of water distribution the following procedure is offered. On all managements of all countries and the prices p^k are a maximum of expression

$$\sum_{k=1}^{n} \max_{A_u^k, A_d^k} \mathrm{M} \max_{w_b^k, w_w^k, w_a^k, c_u^k, c_d^k} g^k$$

where M - operator of calculation of a mathematical expectation designates; A_u^k, A_d^k - appropriately area borrowed under agricultural crops and located above and below a reservoir in country *k*; w_b^k, w_w^k, and w_a^k - appropriately outflow from reservoir of country *k* before, during and after irrigation period; c_u^k, c_d^k - appropriately volume of water, used for irrigation from source of country *k* in the up-stream and down-steam of reservoir.

Let's consider, that the purposes of the countries k are described by the following auxiliary criteria: agricultural production $C^k = C_u^k + C_d^k$, $C_u^k = \Phi^k(A_u^k, c_u^k)$ and $C_d^k = \mathrm{y}^k(A_d^k, c_d^k)$ - the production functions (during vegetation water collects, then it is used by optimum ways for agricultural productions); $e_b^k = \mathrm{j}^k(w_b^k)$, $e_w^k = \mathrm{y}^k(w_w^k)$, $e_a^k = \mathrm{j}^k(w_a^k)$ - volumes of the electric power generations is determined by production functions; $W_a^k = W_b^k + w^k + w_o^k - c_u^k - w_b^k - w_w^k - w_a^k$ - probability of state of emergency as a result of a drought in the future which monotonously depends on a stock of water; W_b^k - stock of water in reservoir by the beginning of a year; $f^k = p_e^k(e_b^k + e_w^k + e_a^k) + p_c^k C^k$ - financial expenses; p_e^k, p_c^k - appropriately of monetary expenses for production of unit of the electric power and production of unit of agricultural products in country k; $m^{k+1} - m^k$ - benefit from sale of water.

Let's accept restrictions on a choice of managements in the form of likelihood restrictions that provides a level of guaranteed feedback of water object: $Bep(c_u \le w_u + w_o, c_d \le w_w) \ge b$, where the level b characterizes a guarantee for which the managing body of the country expects. We fix the following mechanism of water calculations. We shall consider that for volume of water w^k, corresponding to a point status quo, the country pays nothing, and water over this quantity gets under the price p^k. Decision-making is based on use of convolution of criteria $g^k(C^k, e_b^k, e_w^k, e_a^k, W_a^k, f^k, m^{k+1} - m^k)$. It is proved

Theorem 1. Let in a point status quo inequalities

$$\frac{\dfrac{\partial g^{k+1}}{\partial C^{k+1}} \cdot \dfrac{\partial \Psi^{k+1}}{\partial c_d^{k+1}} + \dfrac{\partial g^{k+1}}{\partial e_w^{k+1}} \cdot \dfrac{\partial \mathrm{y}^{k+1}}{\partial c_d^{k+1}}}{\dfrac{\partial g^{k+1}}{\partial m^{k+1}}} > \frac{\dfrac{\partial g^k}{\partial C^k} \cdot \dfrac{\partial \Psi^k}{\partial c_d^k}}{\dfrac{\partial g^k}{\partial m^k}}$$

are execute for all k. Then the point status quo is not effective.

On the basis of the theorem, on an induction, since $t=T$ and finishing $t=1$, it is checked, that on each step of the algorithms described in a dynamic case solve the problems completely similar to problems, considered at the analysis of static models. Therefore all qualitative conclusions received earlier concerning efficiency of association of the countries in a coalition and opportunities of improvement of position of the countries in comparison with a condition status quo, are kept and in a dynamic case. A determinative is the opportunity of use by down-stream countries "benevolent" of outflows of up-stream countries, and also an opportunity of direct exchanges of the goods and resources without monetary calculations. Certainly, the effect from cooperation will collect year by year.

For a case when the considered river has inflows and in each country there are some power stations, all received qualitative conclusions concerning effect of association in a coalition and expediency of cooperation are kept. As well as in the linear cascade is a condition status quo. For this purpose it is necessary to solve some problems of optimization consistently. We shall record on the considered river one point in a month and one point in a source. We shall allocate on the unique simple way connecting these points on one power station in territory of each country. The managements concerning to all other power stations and adjoining it agricultural land, we shall put equal to corresponding values in a point status quo and we shall record. By virtue of the established properties, there will be managements which give all players prizes big, than in a point status quo. If we now "shall release" earlier fixed managements prizes of all players can increase only.

Optimization models [S. Navruzov, 1990] of the reservoir management of the Amudary up-stream water resources are offered which put as a base the "block-hierarchical" principle which provide at the initial stage development of different national models and their further coordination within the framework of regional models. According to it we can divide territory of the Amudarya basin into two zones accordingly: *formation zones of demand and offer of water*. Hierarchical tasks

of management of reservoirs are considered which provide the aggregated needs of the states in the demand zone on irrigation when the hydroelectric power limitation is carried out.

Let the transboundry basin is divided into two zones - a zone of formation of water resources and a zone of their consumption. In a zone of formation reservoir of long-term regulation, and in a zone of consumption - reservoir of seasonal regulation functions. Hierarchical problems of management of reservoir which provide the aggregated requirements of the states of a zone of consumption on irrigated agriculture, under condition of performance of restrictions on production of the hydroelectric power are considered.

Management is function $u^k(t)$ actual outflow from reservoir at the moment of a season t a year k for long-term rows $\{n \times T\}$ where n - set of the years numbered by an index $k = 1,2,...,n$. Every year consist of T intervals with the current index t, $t = \overline{1,T}$. Let $w^k(t)$ - required outflow water at the moment of t a year k, necessary for satisfaction of requirements of irrigated agriculture. Production of the electric power at the moment of t a year k, is designated through $E^k(t)$.

The problem consists in an optimum choice of management $u^k(t)$ which delivers the minimal or maximal value of criteria function:

$$I_{irr} = \sum_{k=1}^{n} \sum_{t=1}^{T} (u^k(t) - w^k(t))^2 \tag{1}$$

$$I_{elec} = \sum_{k=1}^{n} \sum_{t=1}^{T} E^k(t) \tag{2}$$

Dynamics of a condition of reservoir is described by the ordinary differential equation

$$\frac{dx^k(t)}{dt} = v^k(t) - u^k(t) \tag{3}$$

in area $v^k(t) \geq 0, u^k(t) \geq 0$. (4)

Where $x^k(t), v^k(t)$ - appropriately a condition of reservoir and inflow of water to reservoir at the moment t a year k.

It is considered set initial volume of water in reservoir

$$x^1(0) = x^0,\ x^{k+1}(0) = x^k(T),\ k = \overline{1,n-1} \tag{5}$$

and also restrictions are carried out

$$x^- \leq x^k(t) \leq x^+, \tag{6}$$

where x^- and x^+ - accordingly minimally and as much as possible allowable volumes of water in reservoir, and

$$\sum_{k=1}^{n} \sum_{t=1}^{T} H(\bar{x}^k(t), u^k(t) \cdot u^k(t) \cdot \eta \geq E^k(t), \tag{7}$$

where $H(\circ,\circ)$ - a pressure of the hydroelectric power station, monotonously growing function on $\bar{x}^k(t) = \frac{x^k(t+1) + x^k(t)}{2}$ and monotonously decreasing function on $u^k(t); \eta$ - the set factor determining efficiency of hydroelectric power station.

Thus, we have the problem of mathematical programming consisting in minimization (1) and (or) maximization of criterion function (2) at restrictions (3) - (7).

The technique of finding a compromise solution among needs of the states in volumes of consuming water at the level of the coordination of management between zones of consumption and formation is offered. It is shown, that for countries which are in different geographical zones it is

different both schemes of constructions of models and their relations to each other. For example, between countries of the offer and demand zones the scheme of compensartion is offered: Uzbekistan and Turkmenistan (demand zone), getting necessary water during the vegetative period simultaneously accepts the electric generated in Tajikistan (offer zone). During winter period Uzbekistan and Turkmenistan return received electricity or volumes of other its equivalents power to the offers zone as well.

Construction of imitational models is carried out on example the Vakhsh-Amudarya cascade of reservoirs [S. Navruzov, 1986, 1991]. The cascade includes three large reservoirs, such as Rogun, Nurek and Tuymuyn. Mean while, these reservoirs are located in the territory of different Central-Asian republics, namely: Rogun and Nurek belong to Tajikistan and Tuymuyn to Uzbekistan.

Tajikistan being in the zone of drain formation of the Amudarya river basin can recustomize the main operating rules of reservoir from irrigational to energy regime. We know that Kyrgyzstan carries out such policy concerning to Toktogul reservoir for the last years.

However the compromise variant is offered when satisfaction Uzbekistan requirement on irrigation calculates beforehand based on imitational models. Production of hydroelectric power for the Vakhsh cascade to calculates in parallel, which will be getting according to irrigation regime. Recalculation of the not received generation of the electric energy is carried out. This volume of unworking out energy should be compensated by the countries of the Amudarya down-stream water resources.

Within the framework of **imitating model**, are offered a technique of the automated construction of dispatching rules of management by the linear cascade of reservoir of complex purpose.

The problem of management by the linear cascade from three reservoirs located on transboundry basin is considered. It is supposed, that the cascade is intended for maintenance with water of the agricultural production located between the second and third reservoir and is lower than the third reservoir, and also for development of the electric power in two top hydrounits.

Criterion of management for irrigated agriculture is the satisfaction of requirements on number of uninterrupted intervals, i.e. at calculation on a line of supervision for an estimation of security the share of intervals in which corresponding requirements were satisfied is accepted. As a criterion of management on production of the electric power to accept a level of guaranteed monthly average total capacity which it is necessary to support hydroelectric power station of the cascade within a year. Managing parameters are volumes outflow from the up-stream reservoir (outflow from the third reservoir it is unequivocally determined by requirements of irrigation).

Lexicographic ordering problems is considered, i.e. needs of an agriculture first of all are satisfied and only at performance of this condition the problem about hobby of production of the electric power is solved.

At the decision of an irrigational problem joint functioning first two reservoirs of the cascade is considered. Dispatching schedules are under construction counting upon total volume of water in these reservoirs. It is proved by that, first, there is no top restriction on outflow from the first reservoir; second, in model time going is not taken into account, i.e. dynamic components of capacities of water basins and, thirdly, in down-stream the first reservoir are absent irrigational requirements.

Let's designate through $\overset{=k}{u}_l(t)$, $l = 1,2$, accordingly full and reduced irrigational norm of feedback in k year for an interval of time t in down-stream of the incorporated reservoir. Then dynamics of a condition of the incorporated reservoir is described by the following balance ratios:

$$x^k(t+1) = x^k(t) + v^k(t) - u^k(t) \qquad (8)$$

$$\begin{cases} x^k(t) = x_1^k(t) + x_2^k(t) \\ v^k(t) = v_1^k(t) + v_2^k(t) \\ u^k(t) = u_2^k(t), \quad t = 1,2,...,T, \quad k = 1,2,...,n \end{cases} \tag{9}$$

$$\begin{cases} x^{k+1}(1) = x^k(T+1), \quad k = 1,2,...,n-1 \\ x^1(1) = x^+ \\ v^k(t) \geq 0, \quad u^k(t) \geq 0, \quad t = 1,2,...,T, \quad k = 1,2,...,n \end{cases} \tag{10}$$

$$x^- \leq x^k(t) \leq x^+ \tag{11}$$

Where $x^k(t)$ - filling of i reservoir to the beginning of an interval t year k; $v^k(t)$ - inflow of water to i reservoir for an interval t year k; $u^k(t)$ - outflow (feedback) of water from i reservoir for an interval t in year k;

Formal procedure of finding of dispatching lines $d_1(t)$ and $d_2(t)$ for the incorporated reservoir described by balance ratio (8) - (11) is offered. As the first dispatcher it is considered top bending around so-called curves of sufficient filling of reservoir, constructed for each year from rows. Curves of sufficient filling are under construction (a course back) with feedback $u^k(t) = \overline{\overline{u}}_1^k(t)$ at for the set rows $\left\{v^k(t),\ t = \overline{1,T},\ k = \overline{1,n}\right\}$ according to balance ratio (8) - (11) under the formula

$$\widetilde{x}^k(t) = \max\left(x^-,\ \widetilde{x}^k(t+1) - v^k(t) + \overline{u}_1^k(t)\right). \tag{12}$$

Further at infringement restriction $\widetilde{x}^k(t) \leq x^+$ the trajectory is modified and $d_1(t)$ defined as

$$d_1(t) = \max_{k \in \{1,2,...,n\}} \widetilde{x}^k(t), \quad t = \overline{1,T} \tag{13}$$

The second dispatching line is defined as top bending around lines of switching from full feedback on reduced that is as follows. Proceeding from already constructed curves of sufficient filling, the trajectory $\overline{x}^k(t)$ (a course back) with feedback $u^k(t) \geq \overline{\overline{u}}_2^k(t)$ is under construction at $\overline{x}^n(T+1) = x^-$ for rows $v^k(t),\ t = \overline{1,T},\ k = \overline{1,n}$ under the formula

$$\overline{\overline{x}}^k(t) = \max\left(x^-,\quad \overline{\overline{x}}^k(t+1) - v^k(t) + \overline{\overline{u}}_2^k(t)\right)$$

Than $d_2(t)$ defined as

$$d_2(t) = \max_{k \in \{1,2,...,n\}} \widetilde{x}_*^k(t), \quad t = 1,2,...,T \tag{14}$$

where

$$\widetilde{x}_*^k(t) = \begin{cases} \overline{\overline{x}}^k(t), & \text{если} \quad \overline{\overline{x}}^k(t) > \widetilde{x}^k(t) \\ x^-, & \text{если} \quad \overline{\overline{x}}^k(t) \leq \widetilde{x}^k(t) \end{cases}$$

The case when for some interval t value $d_1(t)$ appears less, than value $d_2(t)$ is theoretically possible. In this case it is necessary to increase corresponding values $d_1(t)$ up to $d_2(t)$. It is proved

Theorem 2. *For resolvability of an irrigational problem (problem A) it is necessary and enough that at anyone t and k the inequality was carried out*

$$\overline{\overline{x}}^k(t) \leq x^+, \quad \forall t,k$$

Theorem 3. *The offered technique of construction of dispatching lines* $d_1(t)$ *and* $d_2(t)$*, at performance of conditions of the theorem 2, guarantees performance of an inequality*

$$u^k(t) \geq \overset{=k}{u}_2(t)$$

for all hydrological rows.

Results of the carried out calculations show, that the offered technique of construction of dispatching rules allows to find quickly enough practically comprehensible estimations of an opportunity of satisfaction of incompatible requirements of the national economics in irrigation and energy generation.

3. DECISION-MAKING

Program realization of mathematical models, data base, ways of reception and transfer information, method of organization dialogue with users, visualization means based on GIS are decision-making support system (DMSS). Methodology of creation of the DMSS includes questions: *computer modeling of managing decision-making*; solving *problem of multi-purpose management of water resources; carrying out decision on results of imitation modeling.*

In the structure of methodology of decision problem of multi-purpose management of water resources the original approach proposed to solution complex problem of planning and using water resources of the transboundry basin. The first aspect approach related to spatial division territory of considering basin on levels of management. The second determined by multi-purpose problems related with identifying appropriate compromise solutions between states on using water resources by taking into consideration of economic interests. Combination of multilevel and multidisciplinary goals represents the key moment of methodology offered approach.

The specialized applied program of DMSS uses of program software MS Office operation system Windows XP & GIS Arc Info/Arc View. Information database is constructed based on MS Access and mathematical models are used for finding of optimal solution. Analysis and representation of spatial data is carried out based on GIS technology.

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INFILTRATION OF MIXTURES OF TREATED WASTEWATER AND SPRING WATER INTO COARSE ALLUVIAL SEDIMENTS: HYDRAULIC AND HYDROCHEMICAL OBSERVATIONS AT A TEST SITE FOR ARTIFICIAL RECHARGE IN THE WADI KAFREIN, JORDAN

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Abstract: The paper describes hydraulical and hydrocemical observations during long term infiltrations tests at a basin for artificial recharge in the Wadi Kafrein. Due to water scarcity in the Lower Jordan Valley, artificial recharge is one possible solution for sustainable water management in the area. Therefore evaluation of potential sites, with respect to negative effects like soil clogging and water quality, is fundamental. The measurements conducted within this study showed frequent occurrence of pharmaceuticals as well as a diurnal cycle in the electric conductivity and consequently the influence of treated wastewater on the wadi baseflow. The infiltration rates, mainly controlled by a build up clogging layer, significantly exceeded those reported by other authors in a similar context. They remained high, even after several weeks of operation. At last, technical feasibility and environmental aspects are critically discussed.

Key words: clogging, groundwater, iodinated x-ray contrast media, Lower Jordan Valley, pharmaceuticals

1. INTRODUCTION

The Lower Jordan Rift Valley is a region of extreme water scarcity. The high evaporation losses of the reservoirs along the valley slopes where most of the (winter seasons) runoff is collected, lead to a non-optimal utilization of resources. The farmers of this area extract their irrigation water either from already over-abstracted groundwater resources or from the reservoirs. The latter water is often mixed with (treated) wastewater before use. Therefore, farmers do face marketing problems as Jordan`s neighboring countries like Saudi Arabia stopped the import of crops irrigated with treated wastewater/ reclaimed water. Furthermore, groundwater quality deteriorates due to overpumping and soil salinization and, thus, this practice leads to further water issues. Hence, water quality also becomes a major concern.

2. STATE OF THE ART

2.1 Managed aquifer recharge

Although several studies took place in Jordan focusing on potential areas for artificial recharge (e.g. RAPP, 2007, Fedhi, 2001 or El-Hassan, 1999), only few related implementation projects have been carried out. One study in the area of the Jordan Valley located high recharge potential within the Jordan Valley especially on the alluvial fans of the eastern side wadies. Up to 104 – 119 MCM in total are estimated to be stored in underground which equals roughly the capacity of all reservoirs in Jordan. According to Salameh (2001) the alluvial fan of the Wadi Kafrein bears a high potential

and therefore is considered suitable for further investigations. Additionally to the studies mentioned, several projects have been launched to recharge groundwater with different techniques (described in Wolf et al., 2006).

Recently eight dams were built in the stream bed of Wadi Madoneh and Wadi Butum to infiltrate flood waters (Abu Taleb, 1999). However the infiltration rates and capacities are only based on short term infiltration tests with small water amounts. Thus the difficulty of clogging could not occur and there are no experiences concerning long term infiltration tests.

Beside those studies, only two sites are known to use treated wastewater for recharge, one of them being the Tal el Mantah wastewater treatment plant (WWTP) close to Deir Alla. Here, wetlands are used for the infiltration of treated wastewater. The other recharge project is located near Aqaba and uses a recharge pond for the direct infiltration of treated wastewater with an estimated recharge rate of ca. 1.9 MCM/yr (Bataineh et al, 2002).

However, there are no known monitored applications of intentional managed aquifer recharge with treated or blended wastewater in Jordan.

2.2 Water quality

After treatment almost all wastewater is used for irrigation (about 96.5%) and thereby contributes 14 % of the irrigation water. Most of the water is used “unrestricted” which means it is used only for irrigation of cooked vegetables and has to be mixed with fresh water prior irrigation (MWI, 2004).

Continuous monitoring of treated wastewater parameters like E. coli, anions, cations and BOD/COD is performed at selected spots by the Water Authority of Jordan (WAJ). Regarding pharmaceutical residues and most xenobiotics, there are no governmental analyses. So far, only few screenings were conducted by foreign projects/institutions (e.g. TZW within the SMART framework)

As most of the WWTP effluents are used for irrigation in Jordan, the pharmaceuticals might accumulate in plants or crops and remain present in food. Furthermore, these substances might infiltrate through river banks and reach the groundwater (Kümmerer, 2001).

With respect to pharmaceuticals the risk of consuming fruits irrigated with treated wastewater or drinking it seems to be negligible, as the maximum consumption in a lifetime is far below the dosage of a therapy. There is not much information about a long term low dosage medication. Additionally, most single substances are checked for their health risks only, but not for risks arising due to mixtures with other substances (Kümmerer, 2001).

3. METHODOLOGY & CONCEPTS

3.1 Study area

The Wadi Kafrein catchment is located southwest of Amman. The Shallow Alluvium Aquifer consists of quaternary alluvial sediments, fans and soils. It is mainly composed of uncemented gravels and sands derived from limestone, dolomite and chert, which originate from exposed formations at the rift side (Salameh, 2001). The quaternary sediments are the ones which build the underground for the infiltration experiments of!!.

The water in the Wadi stream consists of the discharge of 22 perennial springs (Lenz, 1999) which build the main flow. The average annual base flow for Wadi Kafrein was determined to be 7.91 MCM per year and the annual surface runoff about 2.84 MCM, which results in a total average stream flow of 10.75 MCM (Salzgitter GmBH & JCE, 1992). Hence the average stream flow was calculated to 250 l/s, but might be much lower in the summer months. A measurement carried out in October 2007 led to a flow of about 140 l/s (Al Khoury, verbal information). Additionally the

effluent of the Wadi el Sir WWTP contributes about 4000 m³/day (Department of Statistics homepage, 2008).

3.2 Test site

The site consists of a square basin (Figure 1) with an average depth of 80 cm and approx. 35 m² floor area. The basin walls are built with concrete bricks. To prevent horizontal infiltration the basin walls were plastered and subsequently covered by a plastic foil. The infiltrating water entered through a 100 m long, 3“-diameter polypropylene pipe and originated from the wadi stream. A valve near the basin was installed to regulate the water volume. A filter and intake structure inside the stream prevented the pipe from heavy clogging by algae.



Figure 1: Infiltration basin



Figure 2: Broken up clogging layer

3.3 Operation and measurements

The different measurements took place between September and November 2007. Most of the time the site was operated allowing the maximum water flow into the basin. The water level was measured by an online probe placed in the basin. The inflowing water volume was manually measured on a frequent basis. Since the inflow showed a great temporal variance, it was stopped several times when the basin was full, in order to monitor the decreasing water level under known conditions. After six weeks of operation, the clogging layer (Figure 2), which built up at the bottom was removed manually.

Two soil samples were taken, one from the unaffected alluvial fan beside the basin and the other from the clogging layer. The grain size distribution was determined by sieving and sedimentation.

Water samples were taken regularly over the whole period. Additionally, a 24-h sampling campaign was conducted with a 3 hour sample interval. The samples were analysed for major cations and anions by using AAS and photometer at the Institute for Applied Geology (AGK). Measurements of pharmaceutical residues and iodinated x-ray contrast media was performed at the Water Technology Centre (TZW) in Karlsruhe using solid phase extraction and LC-MS-MS as described in Sacher *et al.* (2001).

The continuous measurement of the electric conductivity was carried out with an online probe placed at the inflow to the basin.

4. RESULTS AND DISSCUSSION

4.1 Chemical analysis

The results for the major cations and anions (Table 1) all fit the national requirements for artificial aquifer recharge. Only the magnesium concentrations were in parts slightly increased. The concentrations did not show a wide variation over the sampling period. However, some substance concentrations like sodium, chloride, potassium and boron are directly following the diurnal cycle of the electric conductivity and therefore are likely to originate mainly from the WWTP (Figure 3). The ion concentrations that are not following the electric conductivity or that show opposite behaviour are likely to originate from the wadi stream.

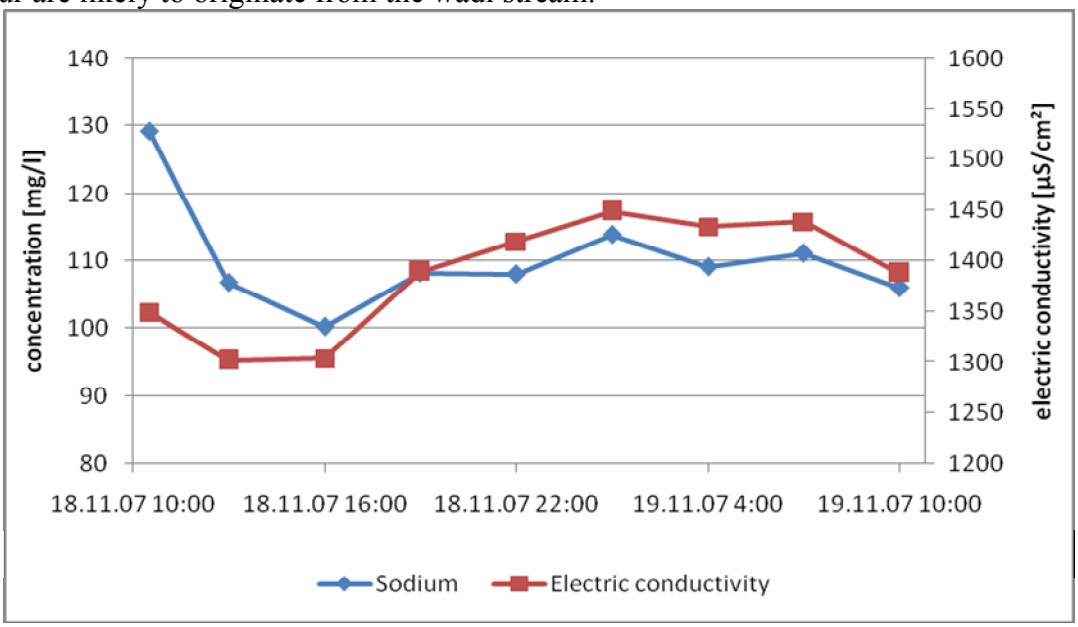


Figure 3: Diurnal cycle of the electric conductivity compared to sodium concentrations in the wadi stream

Table 1: Mean ion concentrations in the wadi stream and the related maximum values for artificial recharge

Parameter	Unit	Jordan limit for artificial aquifer recharge (MWI, 2004)	Infiltration basin mean[1] values from AGK
Nitrate	mg/l	30	11.25
Chloride	mg/l	350	223.9
Phosphate	mg/l	15	0.97
Sulfate	mg/l	300	155.99
Sodium	mg/l	200	116.15
Magnesium	mg/l	60	60.54
Calcium	mg/l	200	103.8
Iron	mg/l	5	0.3
Manganese	mg/l	0.2	0.07
Lead	mg/l	0.2	< LOD
Boron	mg/l	1	0.36

1 = from 20 samples

The analysis for pharmaceutical residues showed ten detectable substances (Table 1). Most of them are among the class of iodinated x-ray contrast media which are known to have high persistence and wide distribution in the environment due to their good solubility in water and the poor biodegradability (DVGW - TZW Karlsruhe, 2006). They can originate either from the upstream El Sir WWTP or from direct discharge without treatment. The spectrum of substances is similar to a TZW screening conducted in March 2007 at the discharge of El Sir WWTP, however some concentrations are much higher, e.g. the concentration of iohexol is around 65 times increased.

Table 1: Mean pharmaceutical concentrations in the wadi stream

Pharmaceutical	LOD	Unit	Infiltration basin mean[1] values
Diclofenac	10	ng/l	28
Gemfibrozil	10	ng/l	172
Bezafibrate	10	ng/l	26
Carbamazepine	10	ng/l	242
Atenolol	10	ng/l	26
Amidotrizoe Acid	10	ng/l	116
Iohexol	10	ng/l	24800
Iomeprol	10	ng/l	74
Iopamidol	10	ng/l	714
Iopromide	10	ng/l	1960

1 = from 4 samples

4.2 Electrical conductivity

The values for electrical conductivity in the river water range between 1150 µS/cm and 1450 µS/cm. Thereby, the EC follows a diurnal cycle which could be traced back to a diurnal cycle of the effluent volume of Wadi el Sir WWTP. Hence, the WWTP is assumed to affect the water quality

4.3 Soil Analysis

The grain size analysis of the soils resulted in sandy gravel for the alluvial fan. The k_f-value, determined after Nahrgang, is $1.2 * 10^{-4}$ m/s (Hölting et al. 2005).

The analysis of the clogging layer points to clayey silt with a k^f-value of $1.3 * 10^{-8}$ m/s after Hazen (Hölting et al. 2005).

4.4 Infiltration rates

The k_f values for the basin floor derived from the infiltration tests range between $1.2 *10^{-6}$ m/s and $2.4 *10^{-6}$ m/s. Therefore, the infiltration rates at a water level of 70 cm are calculated in the range of 4.5 m³/h for the whole basin under typical conditions. The maximum infiltration rate quantified at 70 cm water level was 8.6 m³/h.

The interpretation of the infiltration tests appears rather difficult. Neither the plaster nor the plastic foil prevented the water from horizontal infiltration. For expedient predictions, the basin floor was assumed to be the only permeable surface with the clogging layer being the key element controlling the infiltration. Therefore, the calculated rates are maximum values, not considering the possible infiltration through the walls.

However, it must be clarified, that the basin infiltrated water amounts from 4 m³/h to 8.6 m³/h, depending on the available water volume, until the very end of operation time.

The calculations of the possible per square meter infiltration rates result at a water table of one meter are between 3.48 m/day and 4.36 m/day. Comparing the measured infiltration rates with other experiments is not straightforward (Table 3), as the details about water level, hydrogeological conditions and infiltration surface are partly not existent or are at least imprecise.

Table 2: Overview on Infiltration rates found in literature

Author (year)	Technique	Infiltration rate per m²	Water level	Surface
Zemann (2008)	basin	3.48 - 4.36 m/day	1 m	gravel, sand, silt
Kalantari (2005)	infiltrometer and basin	0.74 -84 m/day	no information	sandy gravel
Grützmacher (2005)		2.54 - 0.59 m/day		sand
Hida (2005)	basin	2.88 - 3.84 m/day	no information	alluvial gravel
Ting (2005)	basin	15.6 m/day	1.7 m	gravel
	basin	5.25 m/day	1.7 m	gravel and sand
Abu Taleb (1999)	Wadi Madoneh pond	0.408 m/day	no information	gravel, sand, silt
	Wadi Madoneh off-channel	0.468 m/day	no information	
	Wadi Butum in-channel	0.162 m/day	no information	fine sand, silt gravel
	Wadi Butum pond	0.147 - 0.24 m/day	no information	

5. CONCLUSIONS

Summing up, from a hydraulic point of view, the alluvial gravels in the Wadi Kafrein are very suitable for artificial recharge via pond infiltration. A mixture of treated and and natural spring water in the wadi was infiltrated and led to the development of a clogging layer which had only minor impact on infiltration rates. The observed infiltration rates are, in comparison to similar experiments, high enough to secure functionality of a recharge pond over longer terms. The water quality is within the respective Jordanian Standards. Pharmaceutical residues were found in low doses and should be monitored further, especially with respect to their accumulation potential vs. geo-purification processes. Due to reduced evaporation in the aquifer compared to the reservoir, more water would remain in the regional water cycle when using the artificial recharge concept. In this respect it could help to meet the actual and future requirements of the Jordanian population.

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RAINFALL-RUNOFF RELATIONSHIP IN MICROSCALE WADIES IN A SEMI ARID ENVIRONMENT – A CASE STUDY FROM WADI KAFREIN IN JORDAN

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Abstract: This study aims to investigate the rainfall-runoff relationships and the main controlling factors affecting it using microscale wadies in Wadi Kafrein/ Jordan. Micro scale wadies are monitored using automatic pressure transducers and tipping bucket rain gauges. This study showed that runoff generated in these micro scale wadies is affected mainly by the rainfall intensity and duration, additionally; the land use play a significant role in this process, as cultivation and vegetation captures most of the precipitated rain. The altitude doesn't seem to have a direct impact on runoff generation. The annual area rainfall average over Kafrein catchment is 411 mm/year, but the precipitation during the study period was found to be under the annual average by 40-45% in the winter year 2007/2008 and 67% until 18.02.2009 for the winter year 2008/2009. Consequently no runoff was generated in most of the micro scale wadies.

Key words: arid-semi arid regions, micro scale wadies, rainfall-runoff relationships, Wadi Kafrein Jordan.

1. INTRODUCTION

Jordan is considered to be one of the 10 water poorest countries in the world and has a population growth rate of about 2.9% (1998-2002), the 9th highest in the world (National Water Master Plan, 2004). The country total water demand in the year 2001 was in the range of 1,350 MCM/ year, of which only 755 MCM/ year was supplied due to water shortages (Ministry of Water and Irrigation, 2005).

Meeting the country's water demand is a big challenge. The available ground water and surface water doesn't satisfy the needs of the population. Searching for alternative resources including non renewable groundwater, brackish water and reclaimed wastewater has become very important.

Studying the rainfall- runoff relationships in Kafrain area will help to better understand the water balance in the proposed area and will offer a good scientific knowledge for the decision makers in order to enhance the usage of the limited available water resources by maximizing water collection and storage and minimizing losses due to evaporation.

Wadi Kafrein stream flow measurements are very limited and incomplete; the only available data are collected on a monthly basis and belong to the period during the 1960ies, where a gauging station was installed. Unfortunately this gauging station does not work any more. Even these records are incomplete and only part of the total flow was recorded and storm runoff was estimated and not measured.

1.1 Kafrein area water shed

Wadi Kafrain drains from the eastern highlands near Wadi Es-Sir city (south west of Amman) to the Jordan Valley. The catchment has a total extent of 189 Km^2. The elevation ranges from 1,100 m asl in its eastern side, down to areas lying below sea level (100m bsl) near the Kafrain dam. The

catchment area lies between 212,000 to 231,000 Easting and 135,000 to 160,000 Northing in Palestine Grid (Figure 4).

The average precipitation in the eastern part is around 550 mm/ year while it decreases to 150 mm/ year in the western parts, along the foothills of the mountains in the Jordan valley area.

The total discharge of the Wadi is 6.4 MCM/ year, consisting of 1.6 MCM/ year flood flow and 4.8 MCM/year base flow (Salameh and Bannayan, 1993).

A dam has been constructed in 1968 at the entrance of the Wadi into the Jordan Valley with initial capacity of 3.8 MCM. The dam wall was raised by seven meters in 1996/97 and has a capacity of 7.1 MCM (Ministry of public works and housing, 1993).

The geology of Wadi Kafrein has been studied by Mc Donald and Partners in 1965, Bender 1974, and Bandel and Khoury 1981.

The major rocks outcropping in the area consist of limestone, dolomite, marl and sandstone. Limestone, dolomites and marls predominate over the upper slopes of the wadies and the upper highlands, and in the northern and the lower part of the catchment area, while the sandstone predominate over the middle and lower slopes of the main stream channels, particularly in the southern part of the catchment area.

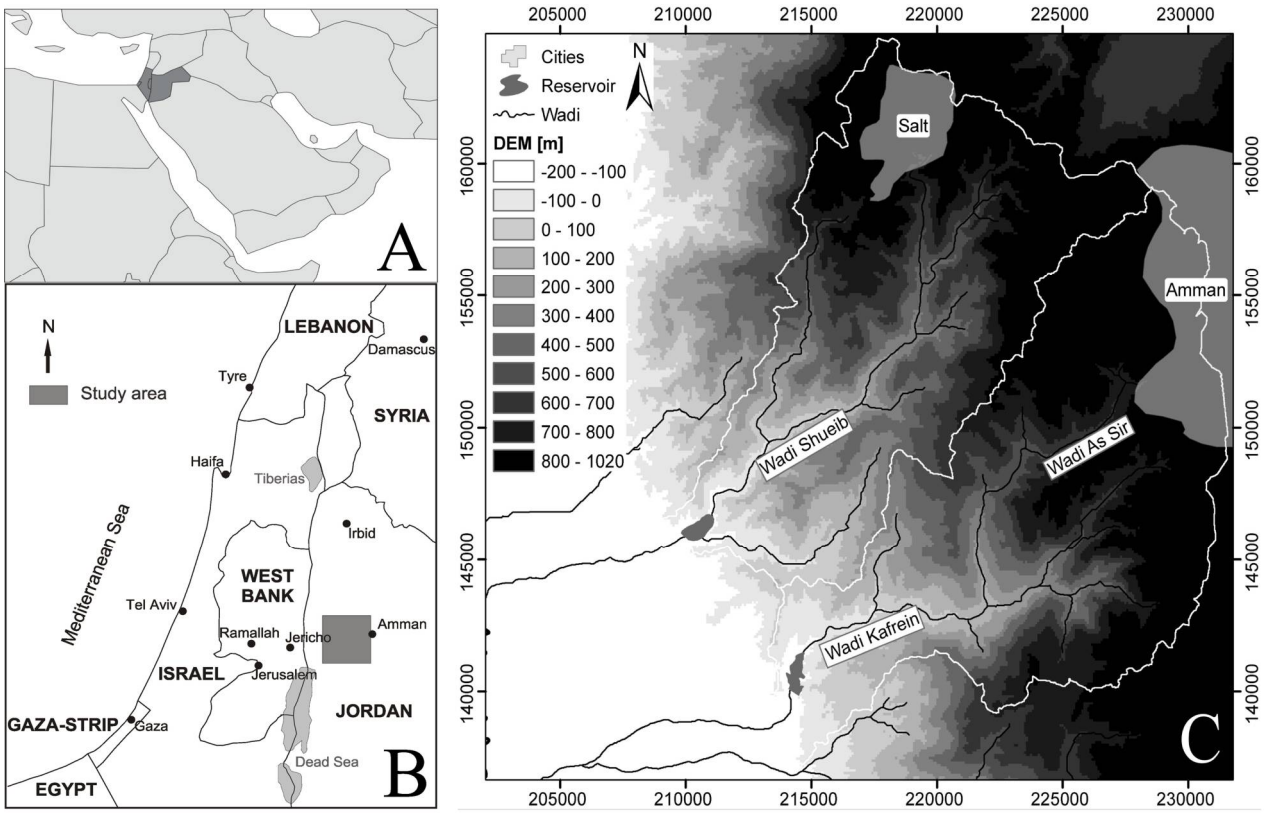


*Figure 4: Location of the study area. **A**: Middle East; **B**: Study area in the regional context; **C**: Study area (the surface water catchment boundaries of Wadi Kafrein and Wadi Shueib are highlighted by a white line).*

2. MATERIAL AND METHODS

Four small subwadies in Wadi Kafrein catchment in addition to a subwadi in Wadi Shueib catchment, the neighbouring catchment of Kafrein, were selected for the study of flood runoff. Both wadies have similar geologic and hydrogeologic properties. This subwadi was selected because it consists of sandstone (Kurnub Group).No representative subwadi was found in the area of Kafrein for monitoring and surface discharge instruments installation.

The subwadies have different characteristics. They differ in altitude, geology, slope, rainfall intensity, land use, and size. The altitudes of the subwadies vary from 760 m asl in subwadi S1

down to 283 m asl in subwadi S4. The catchments of the subwadies S1 to S5 consists of sedimentary rocks. The catchment areas of S1 and S2 consist of nodular limestone, marl, marly limestone, dolomitic limestone, coquina, fossiliferous limestone and nodular marly limestone (Wadi Shueib Formation). The catchment area of S1 is covered with natural forests mainly in its lower slopes while it is cultivated and urbanized in most of its upper slopes. S2 doesn't have any natural forests but most of it is cultivated and precipitation is collected for irrigating the planted trees and farms. The catchment area of S3 consists of grey-green marl, calcareous siltstone, thinly bedded nodular limestone and secondary gypsum (Fuheis Formation), this subwadi is urbanized and part of it is planted with fruits and trees. The catchment area of S4 subwadi consists in its basel parts of fine-to-medium-grained sandstone interbedded with siltstone, mudstone and marl, the upper part of the formation consists of shelly limestone, marl, dolomitic limestone and dolomite with chert nodules at the top (Naur Formation), it has no vegetation cover and its not cultivated. No urbanization is affecting its surface characteristics. The S5 subwadi consists of white, medium-to coarse-grained, pebbly, cross-bedded sandstone (Kurnub Group). It has no vegetation cover and no cultivation or urbanization is taking place within its catchment. The rainfall distribution is decreasing from North-East to South-West.

These different characteristics and variations between the subwadies give the opportunity to investigate the effect of these factors on the generation of surface runoff in a more precise way. Table 1 illustrates the main characteristics of the monitored subwadies.

Automatic pressure transducers with 0.1cm accuracy were installed in concrete structures and culverts at the outlet of every subwadi in order to measure the runoff elevation in the structure (Figure 2). The runoff elevation will be calibrated to runoff amount by using either the salt dilution method (Juraschek 2002, Okunishi et al., 1992, and Hongve 1987), and/or a dipping bar and/or a current meter. The use of the applied calibration methods depends on the subwadi characteristic and the runoff volume.

For rainfall measurements tipping buckets with a resolution of 0.2 mm (per tip) along with supportive rain gauges were installed within the catchment area or close to the monitored subwadis. Climatic parameters including temperature, relative humidity, wind speed and direction in addition to solar radiation is measured using two automatic weather stations. These weather stations were installed in the upper north eastern part of the study area close to R1 and in the lower south- eastern part, close to R4 (Figure 3).

Table 3. The main characteristics of the monitored subwadies.

	ID	Altitude [m asl]		Slope	Area [km^2]	Geology	Land-Use
Wadi Shueib Formation 1	S1	760	0°-20° 31°-45°	91.79% 1.24%	6.86	Nodular Limestone	Natural forest
Wadi Shueib Formation 2	S2	630	0°-20° 31°-47°	91.30% 1.42%	2.82	Nodular Limestone	Olive and Fruit trees
Fuheis Formation	S3	510	0°-20° 21°-35°	98,40% 1.61%	3.66	Marl, calcareous siltstone	Olive Trees
Na'ur Formation	S4	280	0°-20° 21°-32°	96.35% 3.65%	3.02	Limestone, marl, dolomitic limestone s	Shrubs
Kurnub Group	S5	520	0°-20° 21°-30°	81.70% 18.29%	0.18	Medium to coarse sandstone s	Shrubs



*Figure 2: **Left**: Installation of pressure transducers in cement tube. **Right:** channelling the flow to the cement tube.*

3. FIRST RESULTS

3.1 Rainfall

Precipitation in arid and semi arid regions is characterized by high spatial and temporal variability (Simmers 2003). During the rainy seasons of 2007/08 and 2008/09 precipitation was lower than the annual average herewith negatively affecting the stored amounts of surface water in the dams. The total stored amounts in the dams by the end of 2006 were 90.64 MCM while it decreased to 77.983 MCM by the end of 2007 (Ministry of Water and Irrigation 2007).

Most of precipitation occurs between middle of January and middle of April with highest precipitation amounts in January which accounts for about 24% of the average annual rainfall amounts, followed by February with almost 23%. Figure 3 shows the rainfall distribution in the study area in January 2008. The amount of rain in the rainy season of 2007/2008 in the study area was 64% of its long term annual average while until 18.02.09 during the present season 2008/2009 the rain stations within the Kafrein catchment registered only 33% of the annual average (Meteorological Department of Jordan, 2009). The total accumulated rainfall in January 2009 was very low in all stations ranging from 7.2mm in R4 down to 1.2mm in R2, while the highest rainfall occurred in February 2009 mainly during two rain storms in the second half of the month. The total accumulated rainfall in this month ranged from 106.8mm in R2 up to 228.6mm in R1 representing around 73% of the total precipitation until the end of February 2009. Table 2 illustrates the monthly precipitation amounts in every station and the precipitation amount percentages in January and February 2009.

Table 2. Monthly rainfall (mm) in the winter season 2008/2009 including the contribution of January and February in percent.

	Oct.08	Nov.08	Dec.08	Jan.09	Feb.09	TOTAL	Jan. to Total	Feb. to Total
Shueib R6	11.05	3.40	31.80	4.60	163.80	214.65	0.03	0.76
Hussein Gardens R1	18.20	7.20	35.40	5.00	228.60	314.40	0.02	0.73
Rabaheya R2	9.20	3.20	26.80	2.80	106.80	148.80	0.02	0.72
Al Fahas R3	13.40	0.40	31.20	1.40	121.80	168.20	0.01	0.72
WSWWTP R4	12.80	4.20	19.80	7.20	122.40	166.40	0.04	0.74
Bahath R5	13.80	1.60	31.40	3.40	132.80	183.00	0.02	0.73

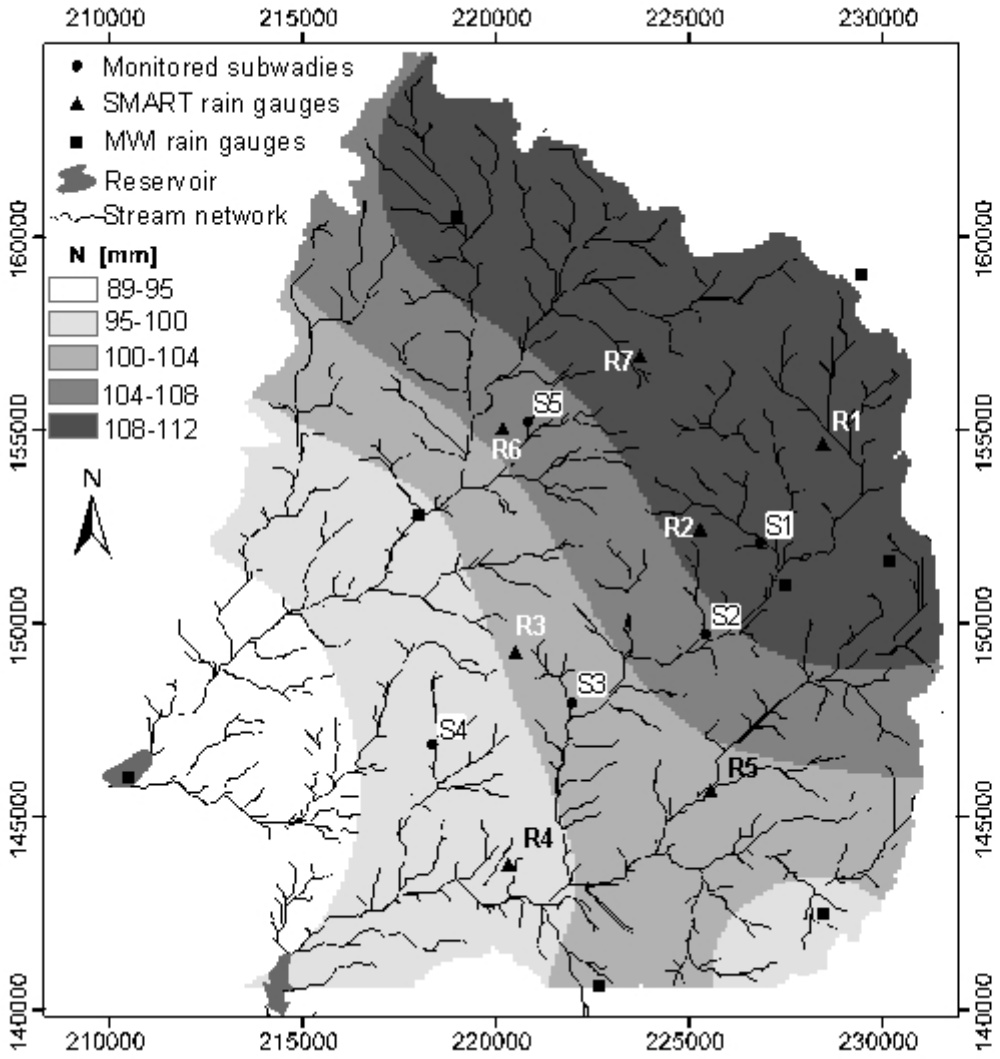


Figure 3: Rainfall distribution over the catchment area during January 2008.

3.2 Runoff

Kafrein water shed has a semi arid climate and flood runoff is generated as a direct response to high rainfall intensity. This runoff is of flashy type, which means it lasts only for few minutes or up to a maximum of several hours. These high magnitude floods are known to destroy gauging stations and change the geometry of the Wadi and they can reach a quantity of more than 6 m^3/s. The overland flow process, which is also called the Horton overland flow, occurs only when the precipitation rates exceed the infiltration capacity and it appears to be more common in arid regions (Fetter 1988). Direct velocity measurements during flood events have to be continuous since the runoff amounts change rapidly. Either it increases or decreases depending on the rain storm event. Additionally, the sediment load makes the measurement of velocity by using a current meter or a dipping bar difficult and dangerous. Therefore, small subwadis were studied on a microscale. This gives more reliable and accurate results because the measuring environment can be better controlled and involve less personal risk. Precipitation during the study period where less than the annual average resulting in only a few runoff events along most of the subwadies.

The first runoff event recorded has occurred in subwadi S5 on the 29th of October 2008. Before this rain storm only one rain storm occurred on the 25th of October 2009, with rainfall depth of 10.8 mm and total duration of 5 hours as measured in R7. On the 28th of October (second rain storm) a

9.4mm rainfall depth was measured by R7. Table 3 illustrates the whole rain storms and its characteristics occurred in S5.

During January 2009 precipitation was very low; the rain gauge R6 recorded only 4.6 mm in the whole month. The fourth runoff event in S5 occurred in the 10th of February 2009. Prior to 10th of February a rainstorm occurred during 31st January to 1st of February with total amount of 5.6 mm in 3.5 hours, no runoff was generated. The rainfall intensity during the 10th of February storm was 6.6 mm in its first hour causing a direct runoff which was measured to 29.5 l/s in its peak. The generated runoff of this rain storm was measured in 5 minutes interval using a current meter. Figure 4 shows the hydrograph of the storm from the 10th of February 2009.

Table 3. Rain storms characteristics for S5 runoff events.

Rain storms	Rainfall [mm/d]	Duration [h]	Station ID	Runoff
25.10.08	10.8	5	R7	Not generated
28-29.10.08	9.4	24	R7	Generated
24-25.12.08	14,2	16	R6	Generated
30-31.12.08	12,4	23	R6	Generated
31.01.09	5.6	3,5	R6	Not generated
10-11.02.09	24.6	24	R6	Generated
21-22.02.09	48,4	38	R6	Generated
27-28.02.09	77,2	40	R6	Generated

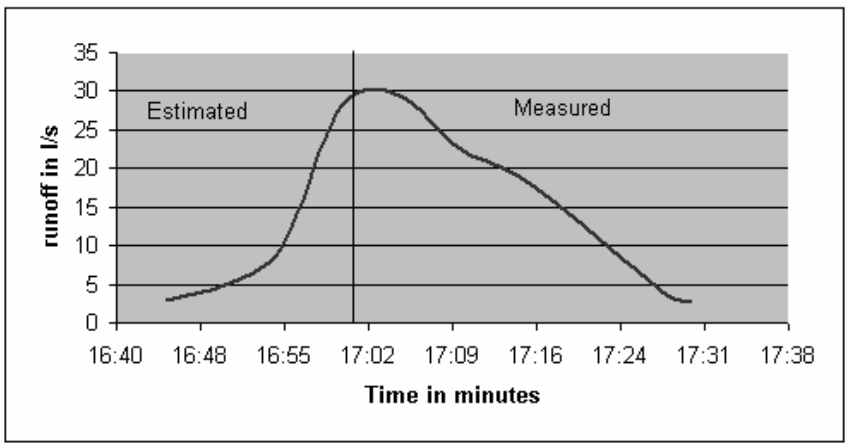


Figure 4. Storm hydrograph in subwadi S5 during the rainstorm of 10.02.2009.

No runoff was generated in any of the other wadies since the beginning of the winter season until the 21st of February where runoff was recorded for the first time in S4. Another runoff event was recorded on the 27th of February 2009 in the same subwadi (Figure 5).

Due to the high intensity of precipitation with relatively short lag time between rain storms in February 2009, the precipitation rate has exceeded the infiltration capacity in S4 and S5 causing a direct runoff shortly after or during the rainstorms. This was noticed and measured in S4 where runoff was generated 4 times during the rainstorm of 27th-28th of February. The water level above the logging device ranged from 43 cm down to 12-15 cm.

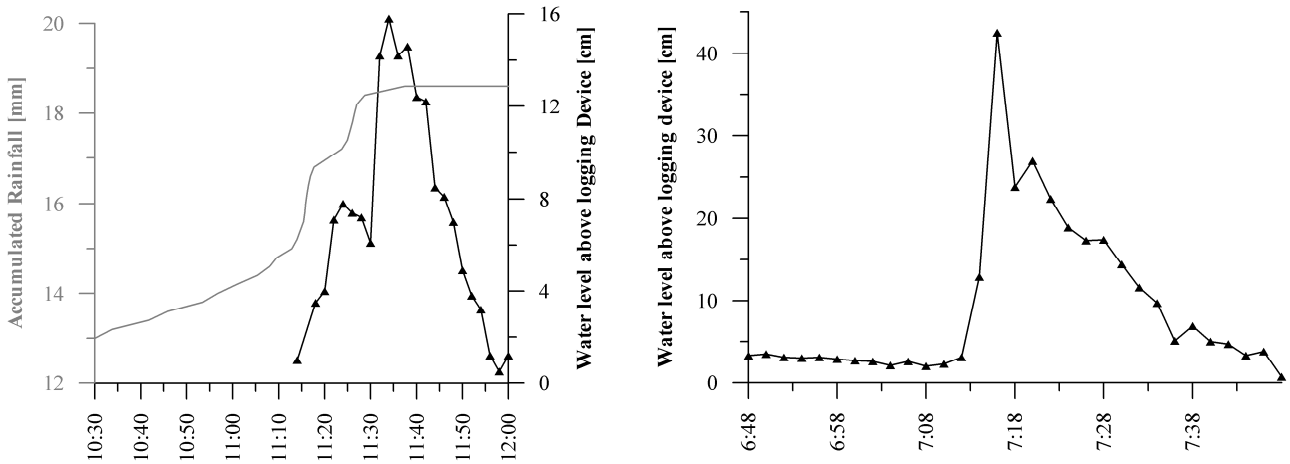


*Figure 5: **Left**: Runoff event recorded in S4 subwadi on the 21.02.2009. Accumulated rainfall of the nearby rainfall station can also be seen. **Right:** Runoff event recorded in the S4 subwadi on the 27.02.2009.*

4. SUMMARY AND CONCLUSIONS

The annual area rainfall average over Kafrein catchment is 411 mm/year, but the rainfall amounts during the study period was less than its seasonal mean in 40-45% for the winter year 2007/2008 and 67% until the 18.02.2009 for the winter year 2008/2009, this was reflected negatively in the stored amounts in Jordan Valley dams and the generated runoff in the monitored subwadies. The long time span between the rainstorms and its relatively low intensity caused only a few runoff events in some of the observed subwadies.

The outcropped rocks in S4 are limestone, marl, dolomitic limestone and in S5 medium to coarse sandstone. Both subwadies have no vegetation cover and it's not cultivated or planted by any trees or fruits. Runoff was observed and measured only in these subwadies indicating the role of land use and cultivation in runoff processes. S4 has the smallest area and its slopes are steeper than the other wadies, the number of runoff events recorded in this subwadi was higher and started earlier compared with S5. This could be attributed to the steepness of this subwadi. The subwadies S1 and S2 have the highest altitudes and they receive higher rainfall intensities over their catchments. However, no runoff events where recorded. This might be attributed to rainfall interception by vegetation covering their watersheds. This indicates the minor role of altitude in runoff generation.

The subwadies are still under monitoring and further runoff measurements are taking place using different methods, including current meter, dipping bar and the salt dilution method.

5. ACKNOWLEDGMENTS

The authors would like to express their thanks to the German Ministry of Education and Research (BMBF) for supporting and funding the project: Sustainable and Integral Management of Available Water Resources using Innovative Technologies (SMART). Project Number 02WM0802, also to the Jordanian Ministry of Water and irrigation, Jordan Valley Authority, and the Dam management unit.

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A SEARCH FOR NEW WATER RESOURCES IN THE LOWER JORDAN RIFT VALLEY

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Abstract: The study area is an arid zone, located at the southernmost tip of the Lower Jordan Rift Valley where the main population of the area is located as well as the agricultural activity. The area has a great potential for agricultural development being the natural greenhouse for a widespread region. The scarcity of water due to arid conditions is the limiting factor for profitable growth. The lower Jordan Rift Valley is a segment of a large geotectonic rift feature bordered by a deep-seated fault system that forms a NS, elongated, narrow basin. The rift is a topographic low, acting as the drainage basin for groundwater originating in the mountain aquifers bordering the rift depression to the west. The most important aquifer is the Cenomanian-Turonian Judea Group aquifer which consists mainly on carbonates. The replenishment of the aquifer occurs essentially on the outcrops of the formations located on the mountainous backbone. Fresh water is exploited from the aquifer by net of water wells located at the foothills of the mountains along the major rift fault system. According to new researches the local aquifers formations contain fresh water bodies along the margins of the Judea Mountains in the west and saline water bodies along the rift basin in the east without any hydrological dividing barrier. The Judea Group aquifer in the valley itself is deeply-seated and overlain by thick sequence of basin-fill young formations. Therefore the area of the basin-fill was considered over the years as unsuitable for groundwater exploitation. The classical, conventional prevailed hydrological thought was, that drilling in the east, in the valley itself, is useless because of the depth of the aquifer and the danger to obtain saline water (due to the occurrence of deep brines and salt bodies). New geological and geophysical investigations show that in several places across the valley the Judea Group aquifer strata are located in rather shallow depths (- 500 meters) and can be drilled economically. The water might be brackish or saline but still suitable for irrigation of certain agricultural growths. The present study suggests new sites for water wells which are located rather close to the distribution network supply system.

Key words: Jordan Rift Valley, aquifer, water resources, boreholes, fresh, saline and brackish water, irrigation.

1. INTRODUCTION

The search for new ground water resources is one of the prime targets of the Integrated Water Resources Management (IWRM) SMART project. The study is focused in the Lower Jordan Rift Valley (LJRV) and over its two shoulders and is funded by the German Ministry of Science (BMBF).

The LJRV includes the rift and its margins from the Judean Mountains in the west through the rift basin to the Jordanian highlands in the east and from the southernmost tip of the Sea of Galilee in the north to the Dead Sea in the south (Fig. 1). The study area is located at the southernmost tip of the LJRV from the Fazael-Uja area in the north to the Jericho area and the northernmost shores of the Dead-Sea in the south (Fig. 1). The area is an arid zone with precipitations of 50 to 150 mm/year and potential evaporation of up to 2,600 mm/year. The main sources of fresh water are the local deep aquifers which are naturally replenished by the precipitation in the surrounding mountains in the east (Jordan) and West (Israel and the Palestine).

The main economic activity of the area is agriculture and its development depends significantly on the availability of water for irrigation. The water supply is coming from several sources. The major and most important source is fresh groundwater from the deep aquifers which are used primary for domestic needs. The secondary potential resources of the area are brackish and treated

waste water as well as desalinated water mainly for irrigation use. The area is rather remote geographically separated from areas with more favorable hydrologic conditions or large desalination centers (such as the areas along the Mediterranean coast) and therefore any future development should be base on local resources.

According to new researches (Hötzl et al 2008), the local aquifers formations contain fresh water bodies along the margins of the Judea Mountains in the west and saline water bodies along the rift basin in the east without any hydrological dividing barrier.

During the agriculture seasons (two seasons in a year), the daily demand increase dramatically and the pumping from the fresh water body rises. In some wells the increasing in the pumping rate is accompanied with parallel significant rise in the chloride content (Guttman 1997; 2000; 2008; Guttman et al 2008).

Since the main pumping occurs from the fresh water body, its potential water level is decreasing below the potential water level of the saline body. Subsequently, during the high pumping period, saline water migrate towards the radius of influence of the pumping wells, raising the salinity of the supplied water and vice versa during low pumping periods.

In order to protect the fresh water body, it is recommended to pump simultaneously from the saline water body and to reduce its potential water level. The saline water can be supplied and used directly to the palm plantations or indirectly by mixing with treated waste water which makes it suitable for other agricultural purposes.

According to a detailed seismic study (Anker 2007; Anker et al 2008) there are some places within the LJRV where the aquifer is rather shallow and enables drilling of new wells for water exploration and, hopefully exploitation.

Applying this hydrogeological conceptual model to the study area (Fig. 1) where the main population of the area is located as well as the agricultural activity and taking into consideration the strategic water planning program of the region, two new sites were located for drilling within the valley. The wells are planned to pump saline water (2000-2500 mg/lit Cl-) for additional supply of irrigation of the palm plantations. The new drilling program is combined with connection program to the distribution network in the area by Mekorot (The Israel National Water Company).

2. GEOLOGY AND HYDROGEOLOGY

The LJRV is the base level for the surrounding surface and groundwater. In the western side of the valley, the regional aquifer is the Judea Group aquifer. The total thickness of the Judea Group is about 800-850 m. The aquifer layers are the Cenomanian-Turonian limestone and dolomite interbedded by marls and clays (Table 1). The natural replenishment is mainly from direct precipitation on the Hebron and Ramallah Mountains outcrops (Fig. 1). The current research explores the nature of the groundwater that flows eastward from the structural water divide. The flow regime evaluation is assisted by the geological map of Sneh et al (1998) as well as the study of Anker (2007; Anker et al 2008) which presents the structural and geolological anatomy of surface and subsurface in the study area (Fig. 1).

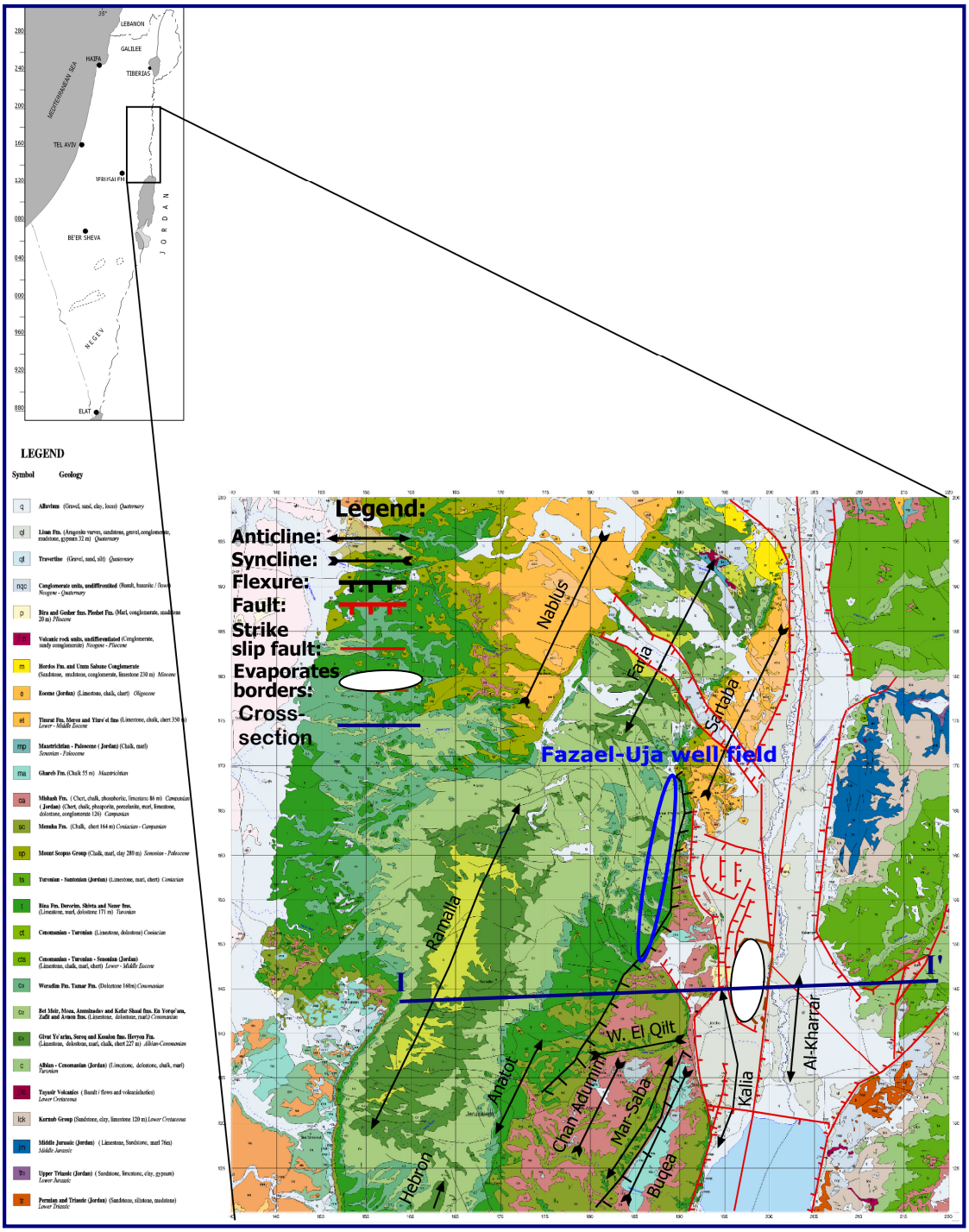


Figure 1: Location map of the study area (after Anker 2007)

In order to capture the fresh water before it flows to the deep-seated rock units of the rift basin and salinized a net of wells were drilled along the foothills (Fig. 1). During the commencement of production from the wells the salinity was low. Concomitantly to the decline of water levels the salinity started to rise. The increase in salinity was accompanied by seasonal salinity fluctuations.

The salinity values in the natural replenishment zones range from 30 to 50 mg/l Cl- and remain constant throughout the years. However, in wells located along the foothill (Fig. 1), a wide range of salinity values were observed ranging from 30 to 600 mg/l Cl. Two types of salinity behavior were encountered in these wells (Fig. 2): a constant salinity over time (Fazael-2, for location see Fig. 4) and a seasonally salinity fluctuation of tens to hundreds mg/l Cl- (Fazael-6, for location see Fig. 4).

During high demand periods in which the monthly pumping rate is very high, there is a significant increase in the salinity and during low demand periods the salinity content decreases. Such behaviour indicates that very close to the radius of influence of the well there is saline water body and that the wells are probably pumping mixture of fresh and saline waters. The chemical composition of these wells supports this assumption and is graphically presented on the middle part of the mixing line between the fresh water end-member and the saline water end-member (for more detail the reader is referred to the book "The water of the Jordan Valley" by Hötzl et al 2008).

The Judea Group aquifer in the valley itself is deeply-seated and overlain by thick sequence of Basin-fill young formations. Therefore the area of the basin-fill was considered over the years as unsuitable for groundwater exploitation. A set of seismic lines were re-interpreted in order to study the geological structure in the deep formations within the Jordan Valley (Anker 2007; Anker et al 2008) as well as the continuation of the Judea Group aquifer from the foothill to the rift valley itself.

The detailed seismic study resulted in a major three main new findings related to water exploitation in the study area (Fig. 3): (a) the Judea Group aquifer was encountered at a relatively shallow depth, which varies from a few hundred meters in the west (east of the foothill) to more then 1000 m close to the Jordan River in the east (b) several east stepping fault blocks were encountered in the area (c) the discovery of a lenticular shaped sedimentary body located within a structural depression in the east near Auja which highly resembles the typical appearances of salt bodies in seismic lines (Anker 2007; Anker et al 2008).

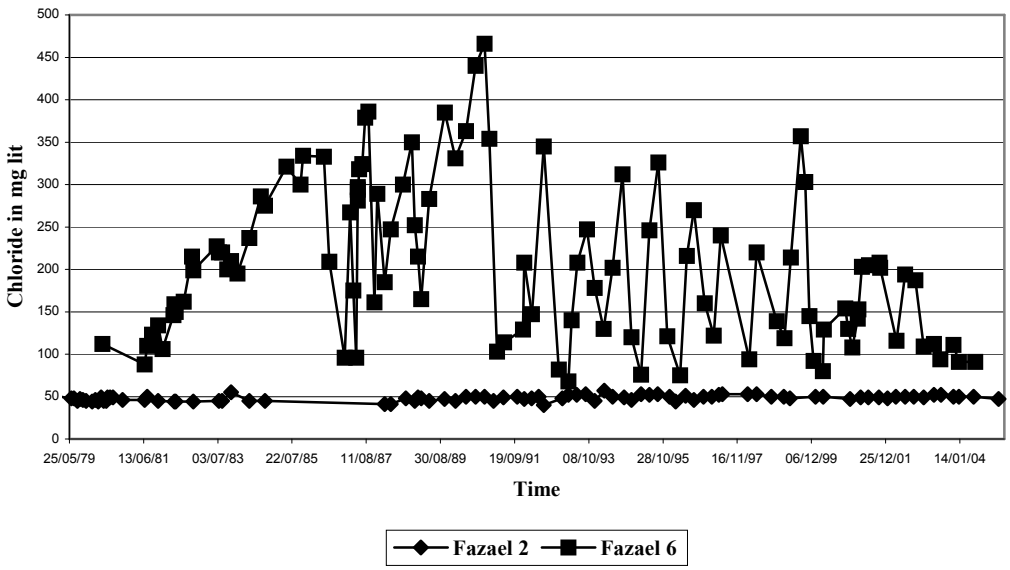


Figure 2: Two types of salinity behaviours in the Fazael well field.

Table 1:Stratigraphy, thicknesses and hydrological properties of formations in the study area (Guttman 2004)

Age	Group	Formation	Lithological Composition	Thickness (m)	Hydrological Properties	Remarks
Plio-Pleistocene	Dead Sea	Lisan	Clays, silt, conglomerate, gypsum, chalk	Base unexposed		Pebble horizons serving as local aquifers
Paleocene	Mt. Scopus	Taqiye	Marl, chalk, clay, limonite concentrations	100-400	Aquiclude	Thickness of Mt. Scopus varies from anticlinal margins to synclinal center
Maastrichtian		Ghareb	Yellowish chalk		Aquiclude	
Campanian		Mishash	Chalk with black chert layers		Aquiclude	
Santonian-Campanian		Menuha	White chalk, some calcareous chalk		Aquiclude	
Turonian	Judea Group	Bina [1]	Fine grained limestone and dolomite	90-100	Aquifer	
Upper Cenomanian		Veradim	Hard gray dolomite, porous	90-100	Aquifer	
		Kefar Shaul	Chalky limestone, chalk, marl	30-40	Aquitard	
Lower Cenomanian		Aminadav	Hard gray dolomite, porous	110-140	Aquifer	
		Moza [2]	Marl, clay	10	Aquiclude	Not always present
		Bet Meir [2]	Chalky limestone, bluish clay, chert, dolomite in Bet Meir Tongue	120-140	Aquitard	Bet Meir Tongue constitutes an aquiferous layer
Albian		Kesalon	Limestone and dolomite rich in fossils	30	Aquifer	Poor aquifer due to small thickness
		Soreq	Limestone and marly dolomite, marl and some chert	110-170	Aquitard	Limestone and dolomite horizons constitute an aquifer
		Givat Yearim	Hard dolomite, at time also limestone	20-40	Aquifer	
		Kefira	Limestone alternating with marl	160-180	Aquifer	Profile at the base is marly
Aptian	Kurnub Group	Qatana	Marl with some limestone	50	Aquiclude	
		Ein Qinya	Limestone	60-70	Aquifer	
		Tammun	Limestone, marl and some sand	Base not penetrated	Aquitard	

[1] The Bina Fm. divides into three formations in the eastern part of the study area, namely: Netzer, Shivta and Derorim Fms.
[2] The Moza and Bet Meir Fms. are referred to as the Ein Yorqe'am Fm. in the eastern part of the study area (Begin, 1976).

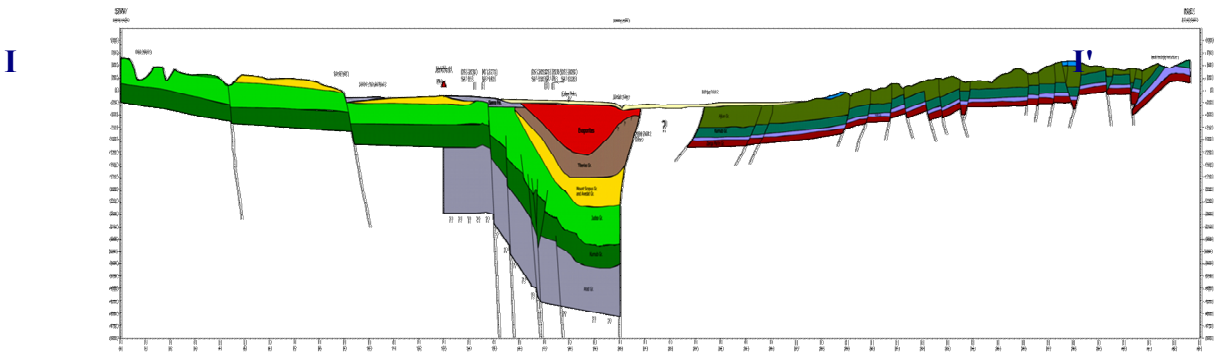


Figure 3: East-West Geological cross section (for location see Fig-1)which presents the relatively shallow the Judea Group Cenomanian-Turonian aquifer(green), the east stepping fault blocks and the presumably newly discovered salt body(Pink) (Anker, 2007).

3. IMPLEMENTATION OF DRILLING SITES

The goal of the current study is to exploit areas for drilling that will pump partly fresh or saline water for agricultural water source in this area. Palm plantations can be benefited directly, as well as other agricultural activities after mixing the water with treated effluents or desalination. In addition the activity of pumping saline water will prevent migration to the fresh water body and reduce its salinity.

The study included review of previous geological and hydrological investigations (Begin 1975; Sneh 1998; Anker 2007; Anker et al 2008), and delineation of the existing water supply systems (Fig. 4). The idea is to locate the newly suggested sites (wells) close to the supply systems in order to reduce the investments and field survey with the local authorities for locating the proposed wells.

The result of the newly seismic interpretation reveals several conspicuous structural features, which are of considerable hydrological importance for this area. As mentioned above it was found that the Judea Group water bearing aquifer layers within the Valley are probably encountered at a relatively shallow depth (Fig. 3) and since there is a continuity of the aquifer layers from the foothill (where the existing well fields are located) towards the east where the saline water body is located, preliminary, three propose sites had been chosen (Fig. 4).

The hydraulic connection between the fresh water body and the saline water body as was mentioned before means that for protecting the fresh water body from salination, it is recommended to pump in parallel the fresh water as well as the saline water from the saline water body. This action will reduce the pressure of the saline water body and reduce the flow from the eastern part of the aquifer towards the well fields while the pumped saline water will be used for agricultural purposes in the region.

The depth of the wells will be between 300-550 m (depend on the outcrop). The target is the Upper Judea Group and the expected discharge is around 200 cum/hour with salinity between 2000-2500 mg/lit Cl-.

4. DISCUSSION

The water supply for the livelihood in the study area is base on local resources. The most important resource is the fresh water, exploited from the existing well fields and used primary for domestic needs. Saline water is an additional resource that can be used directly for agricultural irrigation of the palm plantations as well as mixing with treated waste water for other agricultural purposes or as a source for local inland desalination plants.

The saline water is in a coexisting relationship with the fresh water. Over pumping in the fresh water body may cause a sharp drop of the water level and migration of saline water into the fresh water body. The result is a potential rise in the water salinity and deterioration in the water quality of the fresh water body. Pumping from the saline water body will reduce its pressure and may prevent the migration of the saline water towards the fresh water body.

From the IWRM concept, the co-operational of the two water bodies is the only way to protect the fresh water body from salination as well as protecting its quality for long periods.

Drilling of the new proposed wells in the saline water body (Fig. 4) is integrated in the sustainable development concept of the study area.

Mekorot- Israel National Water Co. is willing to drill these wells in order to increase the available water resources to the local customers.

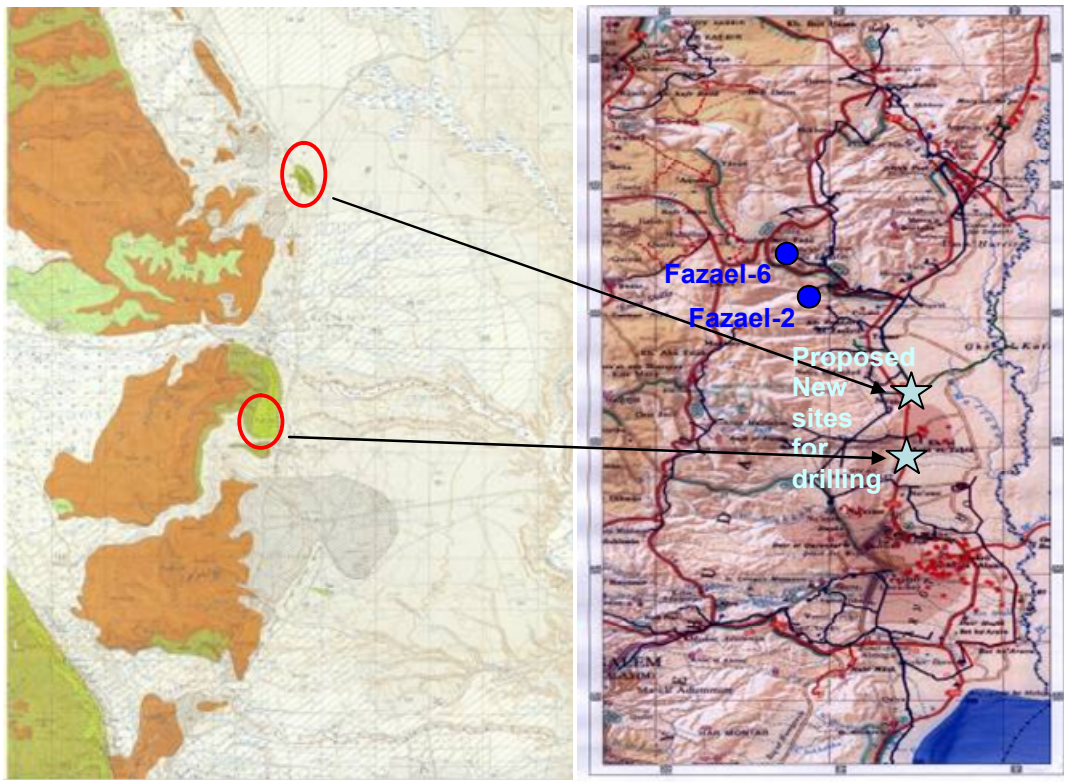


Figure 4: Location of two proposed well on a geological map and on a water supply network

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PARAMETERIZATION OF POTENTIAL EVAPORATION AND LONG-TERM WATER BALANCE TRENDS

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Abstract: The water balance model underlying calculations of the Palmer Drought Severity index (PDSI) has been recently used on a worldwide long–term basis leading to the conclusion of continental drying during recent decades. In its traditional formulation potential evaporation (Ep) is estimated from Thornthwaite's empirical formula, based solely on air temperature (Tair). Consequently, as Tair steadily increases with global warming, the calculated value of Ep in the model also steadily increases. In contrast, calculations of reference evapotranspiration from Penman-Monteith method have been reported as declining in some regions of the world, in general agreement with declining pan evaporation records. Thus, assessing the hydrologic impacts of climate change, the conflicting estimates of declining evaporative demand must be taken into account. In this paper, the effects of three different parameterisations of potential evaporation on long-term trends in actual evaporative flux (AET) and soil moisture (SM) are examined by running the PDSI water balance model across Greece, three times. In the first and second run, Ep is calculated from Thornthwaite's or Penman-Monteith's method, respectively, at 29 stations during the period 1974-2001. The third run uses measurements of evaporation from class-A pans available for 15 sites during the period 1979-1999, for more accurate outputs of the water balance model and hence modelling of drought. The sign of trends in the modelled soil moisture and actual evaporative flux is examined, according to the parameterisation used and the prevailing climatic regime. The analysis indicates that trends in soil moisture and actual evaporative flux do not depend on Ep. Trends in AET and SM, are largely determined by trends in precipitation in Greece.

Key words: Actual evapotranspiration , PDSI, soil moisture , water balance trends

1. INTRODUCTION

Assessing the impact of global warming on the hydrological cycle is not only important from the point of view of the exploitation of water resources, but also with regard to the management of forests as well as agricultural ecosystems. Simple water balance models are often used to assess land surface moisture conditions. The water balance model underlying calculations of the Palmer Drought Severity index (PDSI) (Palmer 1965) has been recently used on a worldwide long–term basis leading to the conclusion that many regions have become drier and more drought-affected during recent decades (Dai et al., 2004). In the traditional formulation of the PDSI bucket model, potential evaporation (Ep) is estimated from Thornthwaite's empirical formula (1948) and is based solely on air temperature (Tair). Consequently, as Tair steadily increases with global warming, the calculated value of Ep in the model also steadily increases. In contrast, calculations of reference evapotranspiration from Penman-Monteith method (Monteith, 1981), being better physical representations of Ep, by using observations of solar radiation, air temperature, humidity and wind speed have been reported as declining in some regions of the world (Thomas 2000; Cohen et al. 2002; Chen et al. 2005; Wu et al., 2006; Shenbin et al., 2006; Zhang et al., 2007) in general agreement with declining pan evaporation records (Epan) (Peterson et al., 1995; Chattopadhyay and

Hulme, 1997; Brutsaert and Parlange, 1998; Quintana –Gomez, 1998; Lawrimore and Peterson, 2000; Golubev et al., 2001; Asanuma, 2004; Hobbins et al., 2004; Liu et al., 2004; Liu and Zeng, 2004; Groisman et al., 2004; Xu et al., 2005; Roderick and Farquhar, 2004, 2005; Chen et al., 2005; Tebakari et al., 2005; Shenbin et al., 2006; Zhang et al., 2007; Papaioannou et al., 2007; Roderick et al., 2007; Burn and Hesch, 2007; Hobbins et al., 2008; Jovanovic et al., 2008). The poor performance of temperature-based Ep in predicting observed evaporative demand is further pointed out in China, where rising Tair but declining Epan are found across eight of the 10 river basins (Chen et al., 2005), being in general agreement with declining Ep calculations according to Penman- Monteith in China (Thomas 2000; Chen et al. 2005; Shenbin et al. 2006; Wu et al. 2006). This can be explained by the fact that Ep is less affected by changes in air temperature than by changes in surface radiation, wind speed, and vapor pressure deficit (Roderick et al. 2007), factors taken into account in the Penman-Monteith equation or integrated, as well, in the measurements of evaporation from a pan (Rotstayn et al., 2006). It must be noticed though, that actual evapotranspiration, is more complicated and driven essentially by climatic factors, mediated by the vegetation and soil characteristics, and constrained by the amount of available water with the climate change having the potential to affect all of these factors in a combined way.

The contradiction between the declining trend in physically based measures of Ep or calculations based on radiation, air temperature, humidity, and wind speed and the increasing trend implied by calculations based only on air temperature (Thornthwaite), might lead to different conclusions about trends in drying (Moonen et al., 2002). Thus, in investigating the hydrologic impacts of climate change, the referred continental drying must be considered taking into account, the conflicting estimates of declining evaporative demand. While other researchers have examined the sensitivity of long-term PDSI to various model characteristics (e.g., Karl, 1986), Hobbins et al. (2008) have examined the effects of two different parameterizations of Ep (based on pan measurements or Thornthwaite's method) on long-term trends in soil moisture, evaporative flux and runoff, simulated by the water balance model underlying PDSI. The trends in water-limited regions were found as driven by precipitation trends but the choice of parameterization for potential evaporation was shown to be critical in energy-limited regions. Their work was the first to examine the effects of using a physically based Ep. It must be noticed that pan measurements have been widely used as a physical measure of Ep in agricultural and engineering applications. In addition, Ep from Penman-Monteith, being a fine representation of reference evapotranspiration (Allen et al., 1998) has been shown as a good estimator of pan evaporation measurements (Epan), in cases where they are not available. Thus, Gao et al. (2007), using Ep either from Thornthwaite or Penman- Monteith method had analyzed the trends of actual evapotranpiration from the water balance methodology introduced by Thornthwaite and Mather (1955). Trends in soil water deficit had been also reported for Greece, from the latest water balance methodology, considering Ep from Thornthwaite's or Penman's method estimates (Kerkides et al., 1996).

In this paper, the direction of trends in soil moisture and actual evapotranspiration is examined, according to different parameterizations of Ep, in order to investigate the hydrologic impacts of climate change. Trends in drying are examined by running the PDSI water balance model at 29 stations across Greece, three times. In the first run, Ep is calculated from Thornthwaite's method using Tair measurements only and in the second run, Ep is based on Penman-Monteith's method. The period of analysis is 1974-2001. A third run uses pan measurements available for 15 sites during the period 1979-1999. Precipitation and model parameters are not changed, for isolating the effect of differed formulations of evaporative demand on the outputs of the water balance model and hence on the modelling of drought. The sign of trends in the modelled soil moisture (SM) and actual evaporative flux (AET) is examined, according to the parameterisation used and the prevailing climatic regime. The analysis indicates that trends in soil moisture and actual evaporative flux do not depend on potential evaporation parameterisation. Ep trends show conflicting signs in some sites. Trends in AET and SM in Greece, are found largely determined by trends in precipitation.

2. MATERIALS AND METHODS

In this study, daily measurements of dry and wet bulb-temperature, humidity, wind velocity and sunshine hours or monthly values of air temperature, obtained from the National Meteorological Service for 29 stations in Greece during the period 1974-2001, are used for estimating potential evapotranspiration by Penman-Monteith's equation (Ep-m) or by Thornthwaite's method (Eth), respectively. The distribution of the 29 stations over the study area is presented in Figure 1. Moreover for 15 out of the 29 stations (marked with $*$ in Table 1) daily pan evaporation records, available for the period 1979-1999, are also employed in estimating potential evaporation (Epan). As the soil moisture-accounting in the PDSI model requires continuous monthly input time-series, missing data of one or two months were scaled from their climatological monthly means, according to the proportion in the rest of the year. When over two months data were missing from a given year, all months of that year were replaced by their average climatological values. In the worst case, less than 4% of data were filled.

Monthly values of Eth, Ep-m, Epan and precipitation (Pr) are used in the calculation of the water balance underlying the Palmer Drought Severity Index (PDSI), as proposed by Palmer (1965). According to the model, soil moisture storage is handled by dividing the soil into two layers. It is assumed, that 25 mm of water can be stored in the surface layer, while the underlying layer has an available capacity that depends on the soil characteristics of the site being considered. Therefore, for every station of the study, each of the three runs of the PDSI algorithm (Eth^{PDSI} or $Ep\text{-}m^{PDSI}$ or $Epan^{PDSI}$, based on Eth or Ep-m or Epan, respectively) provides different soil moisture (SMth or SMp-m or SMpan, respectively) and actual evaporative flux (AETth or AETp-m or AETpan, respectively) estimates. Figure 2 presents the time evolution of the average annual SMth, SMp-m, SMpan (a) and AETth, AETp-m, AETpan (b), for three selected stations in Greece. It is obvious, that the three estimates of soil moisture or actual evaporative flux are not quite similar in some stations, with the discrepancies being quite greater between the Thornthwaite- based values and the ones obtained from the other two methods (especially in the more southern stations).

Trends of Eth, Ep-m or AETth, AETp-m or SMth, SMp-m are calculated for two time periods (1974-2001 or 1979-1999). Trends are defined as the slope of an ordinary Least Squares Regression through annual time-series, with the significance of annual trend determined by t-tests. The slopes of the trends (during the period 1974-2001), of potential evapotranspiration (bEth or bEp-m), actual evapotranspiration (bAETth or bAETp-m) and soil moisture (bSMth or bSMp-m), according to Eth^{PDSI} or $Ep\text{-}m^{PDSI}$ run, respectively, are shown in Table 1. Trends with 95% level of significance are marked as bold. It must be noticed that the slopes of precipitation trends are negative for 27 stations and positive for 2 stations (Rhodes and Lamia), during this period. Increasing Eth (statistically important) is obvious in all stations (except Sitia), while Ep-m is increasing in 18 stations and decreasing in 11 stations. Actual evaporative flux has negative trend for both Thornthwaite and Penman Monteith Ep^{PDSI} run in all stations, except Rhodes. The latest shows positive trend, in coincidence with the positive trend of precipitation for this period. Soil moisture is mainly decreasing, either from Eth^{PDSI} run in 27 stations (except of Lamia and Rhodes), or from $Ep\text{-}m^{PDSI}$ run in 26 stations (except of Serres, Florina and Rhodes). Table 2 shows the number of stations with negative or positive trends of the considered parameters, with the significant trends at the 95% level of significance shown in parenthesis. Mean and standard deviations and maximum and minimum values of the considered trends are also shown in Table 2. Thus, it is observed that AET-trends are negative in Greece, generally following the negative precipitation-trends. This, results in the conclusion, that Greece seems to be rather largely water-limited and hence the trends in moisture supply and not the trends in evaporative demand, determine largely the trend in the actual evapotranspiration.

Since pan evaporation data were available only for 15 (out of 29) stations during the period (1979-1999), trends for actual evaporative flux or soil moisture obtained from Eth^{PDSI}, $Ep\text{-}m^{PDSI}$ or $Epan^{PDSI}$ runs are estimated for this period. Table 3 shows the slopes of the trends of actual

evaporation and soil moisture from Eth^{PDSI} or $Epan^{PDSI}$ runs ($bAET_{th}$, bSM_{th} or $bAET_{pan}$, bSM_{pan} respectively). During this time period, precipitation decreases in 14 stations and increases only in the station of Serres. As a result, this station has positive bAETth and bAETpan, while the trend of both soil moisture estimates is negative (Table 3). Trends of AET are mostly negative (13 stations for the Thornthwaite-based parameterization and 14 for the pan evaporation-based parameterization) in agreement with the negative trends of soil moisture. Alexandroupoli is the only station presenting positive bAET and bSM, in disagreement with its corresponding negative precipitation-trend.

Figure 3 presents the relations between the slopes of the soil moisture trends obtained from the two parameterizations of potential evaporative demand based either on Thornthwaite and Penman-Monteith methods (a), during the period 1974-2001 or on Thornthwaite method and pan measurements (b) during the period 1979-1999, at all available stations. During the period 1974-2001 both parameterizations generate generally declining SM trends in almost all the stations, with increases in the station of Lamia (Figure 3a). There are opposite SM-trends for both parameterizations in Serres and Florina. During the period 1979-1999 the two parameterizations also generate negative SM trends (Figure 3b) with the exception of positive trend in Alexandroupoli.

3. DISCUSSION AND CONCLUSIONS

The water balance model underlying calculations of the PDSI, routinely used in many countries to assess developing drought conditions, has been recently extended on a worldwide, long-term basis leading to the conclusion that many regions have become drier and more drought-affected during recent decades (Dai et al.,2004). The primary attraction of the traditional formulation of the PDSI bucket model, based on Thornthwaite approach for potential evaporation, is simplicity, since only air temperature data are required, and such data are widely available in time and space, including historical global grids at many spatio-temporal resolutions. In the other hand, PDSI is based on a hydrological model that predicts evaporative demand from the direct effect of surface warming only and neglects the effects of other accompanying changes, such as those in surface radiation, wind speed and humidity referred to play an important role over many land areas (Roderick et al., 2007). The fact that historical Tair trends may be more certain than trends in solar radiation, humidity and wind speed, variables necessary for estimating Penman-Monteith estimates, but physically integrated into pan evaporation measurements (Rotstayn et al., 2006), demonstrates the value of the Epan records when they are available for a long time. It is hence, worthy to evaluate, either the oversimplistic or the based on appropriate dynamics, treatment of evaporative demand in water balance models, for a more rigorous approach to assessing long –term changes in the terrestrial water balance.

In this study, it has been shown that Tair-based Eth predicts 38% (or 50%) of trend directions opposite to those calculated from Ep-m (or Epan, respectively). These results for Epan are similar with the ones reported by Hobbins et al. (2008), although pan measurements are available in Greece, for a shorter time period (21 years). The parameterization of the PDSI model with potential evapotranspiration, based on either temperature-based Thornthwaite method or Penman-Monteith approach and pan evaporation measurements, (both responding to the appropriate dynamics: air temperature, humidity, wind speed, and net available energy) provides three different actual evapotranspiration and soil moisture time series in Greece. It is evident from the analysis that all parameterizations generate spatial patterns of soil moisture-trends more closely resembling to the trends of precipitation than the ones of potential evaporation in Greece. This conclusion is in agreement with similar findings reported for Australia and in disagreement with the ones concerning New Zealand (Hobbins et al., 2008). Trends of actual evapotranspiration in Greece are found generally following the trends of precipitation and not of potential evaporative demand and this has been also reported for AET-trends in Australia (Hobbins et al., 2008) or most parts of China (when the Thornthwaite and Mather water balance methodology was used) (Gao et al., 2007). Results from a multi-model ensemble-mean (Meehl et al., 2007) indicate also patterns of projected trends in AET broadly similar to those in precipitation over much of the global land surface. In the contrary AET



trends for New Zealand (Hobbins et al., 2008) or southeast China (Gao et al., 2007) were found increasing or decreasing depended on the potential evaporative demand parameterization.

Figure 1. Locations of the Meteorological stations.

Table 1. Slopes of the trends of potential and actual evapotranpiration and soil moisture according to Thornthwaite's or Penman-Monteith's method (bEth, bAETth, bSMth or bEp-m, bAETp-m, bSMp-m, respectively) for the period 1974-2001.

Station	bEth (mm/y^2)	bAETth (mm/y^2)	bSMth (mm/y)	bEp-m (mm/y^2)	bAETp-m (mm/y^2)	bSMp-m (mm/y)
Serres*	**0.1054**	-1.5962	-0.1898	-0.0110	-1.5769	0.0238
Florina	**0.0545**	-1.4279	-0.2789	**-0.0956**	-3.1116	0.0263
Mikra*	**0.1175**	-2.5382	-0.4656	**0.0947**	-3.1074	-0.3264
Kavala	**0.1034**	**-5.9836**	**-0.8111**	**0.2399**	**-5.7882**	**-1.2201**
Alex/poli*	**0.0744**	**-3.2599**	-0.4554	0.0673	-3.4142	-0.2514
Kozani	**0.0941**	-2.4692	-0.1146	**0.1454**	-3.3598	-0.4450
Kerkira	**0.0979**	-0.6936	-0.1876	0.0624	-1.5162	-0.3361
Ioannina	**0.0444**	-2.8018	**-0.4338**	**0.2428**	-2.4277	**-0.8384**
Larisa*	**0.0826**	-2.1864	-0.6831	0.0488	-2.2627	-0.5315
Aghialos*	**0.0809**	-2.4206	-0.5182	**0.2074**	-3.8826	-0.9505
Mitilini*	**0.1010**	-1.3848	-0.2984	**-0.1759**	-3.3156	-0.3192
Agrinio*	**0.0800**	-1.8280	-0.4374	-0.0365	-3.0545	-0.4770
Lamia*	0.0574	-0.5676	0.1017	0.0606	-0.5089	0.1143
Adravida*	**0.0655**	-2.1569	-0.2869	0.0252	**-3.5528**	-0.5120
Argostoli	**0.0688**	-3.0065	-0.3820	0.0703	**-4.4874**	-0.4697
Chios	**0.0976**	-0.8882	-0.4411	-0.1992	-2.0218	-0.5270
Pyrgos*	0.2960	-0.9204	-0.4538	0.0042	-1.8324	-0.6296
Tripoli*	0.0170	-0.7933	**-0.4019**	-0.0023	-1.7280	-0.4921
Elliniko	**0.1295**	-3.3960	-0.4306	0.0383	-4.0322	-0.0762
Kalamata*	**0.0559**	-1.2692	-0.3407	-0.0682	-2.9402	-0.3766
Naxos	**0.0607**	-3.1098	-0.8325	**0.1027**	-4.5635	-0.6027
Methoni	**0.0575**	-1.6238	-0.1182	-0.0145	**-2.9236**	-0.0809
Milos	**0.1046**	-1.1622	-0.1407	**0.2239**	-1.6973	-0.4924
Kythira	**0.0917**	**-1.7645**	-0.4544	0.0130	**-3.6998**	-0.5322
Rhodes	**0.0836**	0.8898	0.0094	**-0.1273**	0.1698	-0.0047
Iraklio*	**0.0661**	**-3.1177**	-0.4748	0.0494	-3.0452	-0.2869
Ierapetra*	**0.1184**	-3.1044	-0.6096	-0.0498	**-6.2980**	**-1.0075**
Sitia	-0.0312	-1.5502	-0.0936	**0.1562**	-3.0071	-0.8007
Tymbaki*	**0.0941**	**-2.1588**	**-0.7305**	**-0.1160**	**-4.9784**	-0.6183

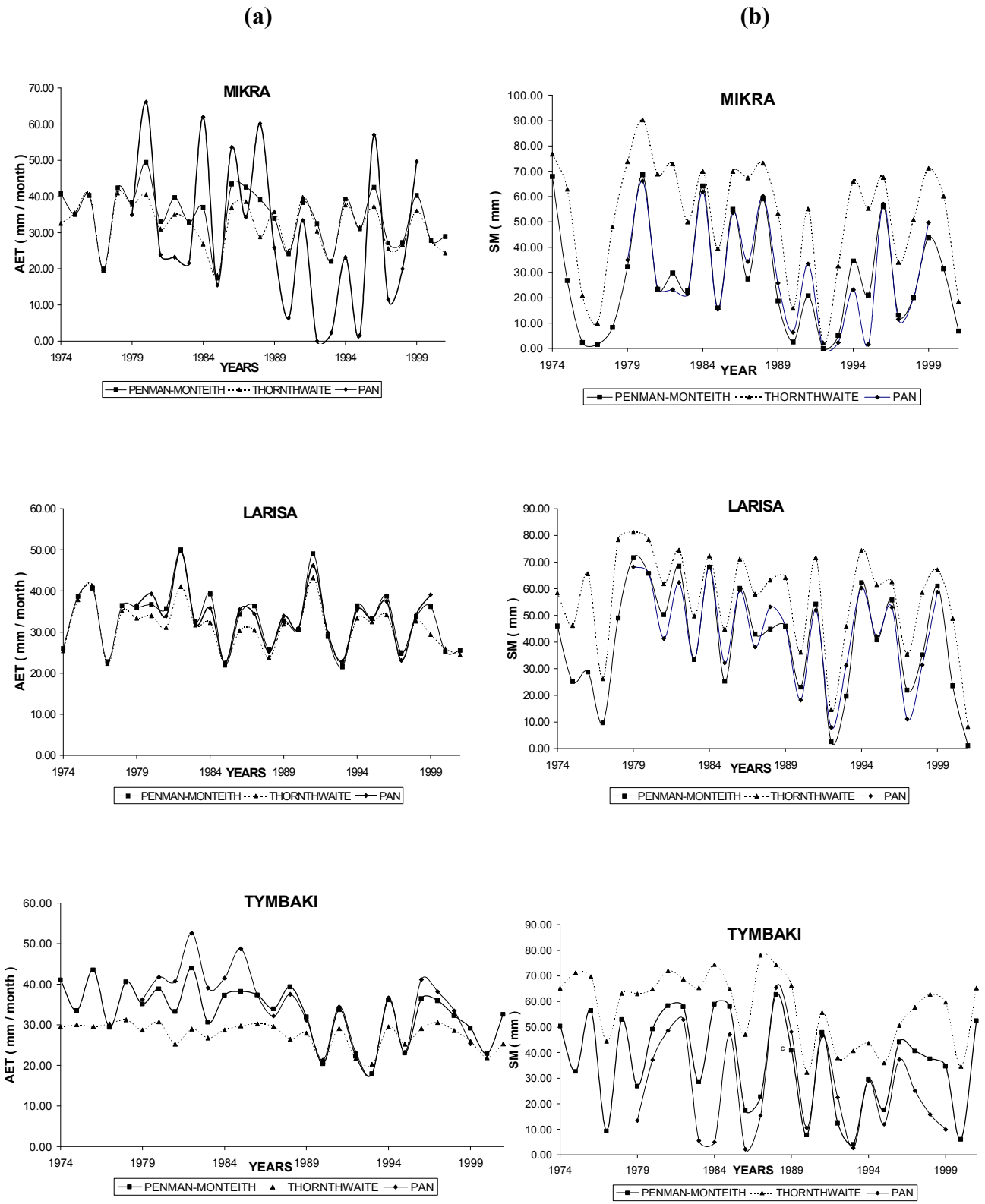


Figure 2. Actual evapotranspiration (a) and soil moisture (b) according to Thornthwaite and Penman – Monteith method or pan measurements as a function of time, for three selected stations.

Table 2. Magnitude, range, direction and significance of the slopes of the trends of potential and actual evapotranpiration and soil moisture according to Thornthwaite's or Penman-Monteith's method (bEth, bAETth, bSMth or bSMp-m, respectively) for the period 1974-2001.

						trend direction	
		mean	**stdev**	**min**	**max**	+	-
bEth	(mm/y^2)	0.085	0.052	-0.031	0.129	28(25)	1.000
bAETth	(mm/y^2)	-2.010	1.254	-5.984	0.890	1	28(5)
bSMth	(mm/y)	-0.378	0.230	-0.833	0.102	2	27(4)
bEp-m	(mm/y^2)	0.033	0.117	-0.176	0.242	18(8)	11(4)
bAETp-m	(mm/y^2)	-3.033	1.430	-6.298	0.170	1	28(7)
bSMp-m	(mm/y)	-0.450	0.322	-1.220	0.114	3	26(3)

Table 3.Slopes of the trends of actual evapotranspiration and soil moisture, according to Thornwaite's method or pan measurements (bAETth, bAETpan, bSMth, bSMpan, respectively) for the period 1979-1999

Station	**bAETth (mm/y^2)**	**bSMth (mm/y)**	**bAETpan (mm/y^2)**	**bSMpan (mm/y)**
Serres	1.2670	-0.0255	0.7607	-0.05897
Mikra	-2.0446	-1.2963	-3.7924	-0.92597
Alex/poli	0.6444	0.1165	0.0108	0.3283
Larisa	-1.7322	-0.9066	-2.6989	-1.12144
Aghialos	-1.9316	-0.8575	-5.4697	**-1.83553**
Mitilini	1.4414	-0.0628	0.2828	-0.04818
Agrinio	-2.8990	-0.5516	-3.8119	-0.62079
Lamia	-0.6001	-0.2247	0.2110	-0.0921
Adravida	-1.7761	-0.4856	-2.0912	-1.20056
Pyrgos	-2.2074	**-0.9237**	-5.2841	-0.96545
Tripoli	-1.8319	-0.7150	**-4.9914**	-0.07074
Kalamata	-2.0864	-0.6339	-1.1900	**-1.61432**
Iraklio	**-3.2870**	-0.6105	-4.0019	-0.22043
Ierapetra	-2.2477	-0.4656	-5.7152	-0.01527
Tymbaki	-1.2079	**-1.0827**	**-9.0348**	-0.61814

(a) (b)

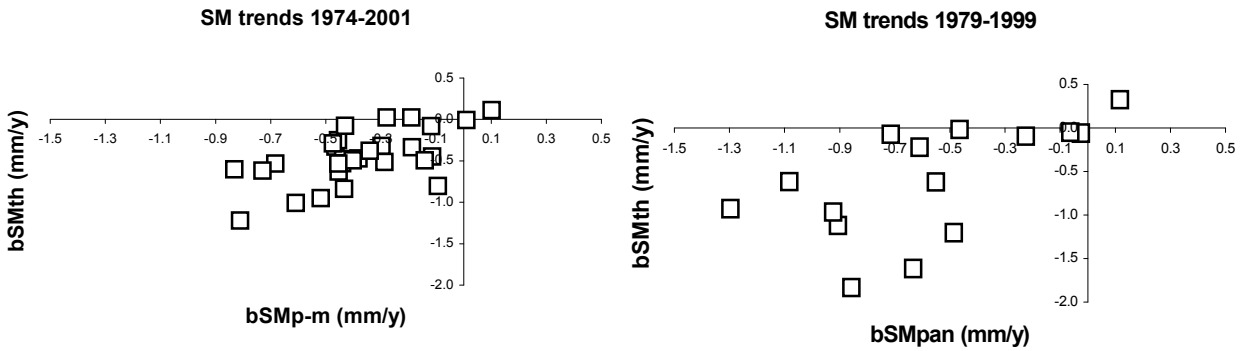


Figure 3. Slopes of the trends of soil moisture according to Thorthwaite's method, as a function of slopes of the trends of soil moisture according to Penman-Monteith (a) or pan measurements (b) for the available stations.

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EFFECTS OF CLIMATIC CHANGES ON SURFACE AND GROUNDWATER RESOURCES IN THE NORTHWESTERN PART OF JORDAN

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Abstract: During the last 4 decades precipitation records in Jordan have shown a general decreasing trend. Such decreases have certainly their impacts on the availability of surface and groundwater, on soil moisture contents (green water) and on the surface and groundwater qualities. In this article the impacts of decreasing precipitation on the availability of surface and groundwater will be analyzed. The results show that a decrease in precipitation of 10% will result in the reduction of flood runoff by about 39%, and a reduction in groundwater recharge of 16% in rain rich areas receiving more than 500mm/yr increasing to 59% in areas receiving moderate precipitation of around 300mm/yr.

Key words: climatic changes, evaporation calculation, surface and groundwater.

1. INTRODUCTION

Precipitation (henceforth ppt) records in Jordan during the last 4-5 decades show a general decrease in the amount of ppt with the passage of time. The study area in the northwestern part of the country (Fig. 1) is a very important part of the country in what concerns surface and groundwater resources. The precipitation trends for that area and for Jordan as a whole are given in Figures (2-5). The main consequences of such decreases are: (1) Reduced flood flows, (2) Reduced groundwater recharge, (3) Reduction in soil moisture content (green water) and shorter periods of soils being wetted, (4) Increases in both surface and groundwater salinities.

These consequences have themselves further implications on discharge of springs, water supply, air relative humidity soil temperature etc.

As a result of all these effects rain fed agriculture will reduce, water supply will decline, existing dams will become over sized and generally investments in many projects will be lost or will reduce in value, with all the socioeconomic ramifications of all the above mentioned impacts.

In this article, the decreases in ppt in the northwestern parts of the country, which receive the highest amounts of ppt and hence the water richest areas in Jordan will be illustrated in the form of time series of representative rain gauging stations. After that, the impacts of the reduction in ppt rates on the quantities of surface and groundwater resource will be elaborated.

2. REDUCTION IN FLOOD FLOWS

Decreasing ppt in an area reflects on flood flows as a result of two factors:

1- Decreasing flood flow amounts due to decreasing ppt according to the equation:

$$\text{Flood flow} = \text{ppt} \times \text{runoff ratio}$$

If ppt decreases, the flood flow decreases also.

2- Flood flow decreases as a result of the fact that the higher the ppt in an event or in the sum over a certain time period the higher the runoff / ppt ratio (Fig. 6).

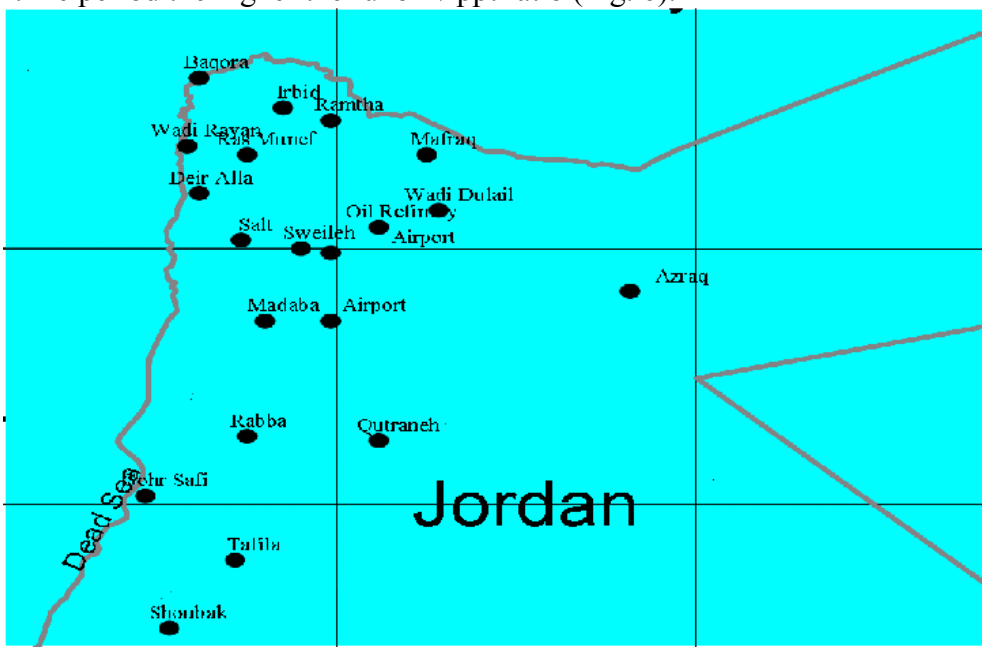


Figure !. Location map of the study area and some of the rainfall stations.

The reduction in the flood runoff / ppt ratio in the northwestern part of Jordan can be deduced from Figure 6. In average it equals 0.8% (based on the NWMP 1977). Precipitation over the area ranges from 312 mm/yr to 542 mm/yr with an overall weighted average of 456 mm/yr (DoM and WaJ open files). Decreases in ppt by 10% result in direct reduction in flood flow by the same rate of 10%. This is a direct result of flood flow/ppt ratio.

The flood flow of the wadis in an average year sums up to 16.3 MCM, a 10% reduction equals 1.63 MCM /yr of that average flood flow. The flood flow / ppt ratio decreases due to 10% decrease in ppt rates by 0.8% (Fig 6). That means, instead of a flood flow/ ppt ratio ranging from 0.0 to 5.22% with an average of 2.77%, the ratio, when ppt decreases by 10% will range from 0.8 to 2.87% with an average of 1.97%.

Accordingly, the second factor, which is the decrease in the flood flow/ ppt ratio itself will cause a reduction in the flood flow of (0.8 / 2.77) x 16.3 = 4.7 MCM/yr. The total reduction equals then (1.63 + 4.7) 6.33 MCM/yr out of 16.3 MCM/yr of the flood flow amounts of the 30s 40s and 50s of the last century. This is an average reduction of around 39% in the flood flow amounts.

3. REDUCTION IN GROUNDWATER RECHARGE

Natural groundwater recharge is normally calculated from the water balance equation:

$$P = E + Sr + I$$

where: P: Precipitation, Sr: Surface runoff, E: Evaporation, I: infiltration to the groundwater or natural recharge. The calculation or measurement of both evaporation and infiltration give only approximations of quantities. Because evaporation in semi arid areas makes a high percentage of ppt amounts (60 - 99%), a small error in its measurement or calculation reflects strongly on the other water balance parameters especially infiltration and groundwater recharge.

Therefore, the adapted approach for the calculation of actual evaporation will be based on Dalton's (1502) equation:

$$E_D = c(e_s - e_a)$$

Where, E_D : evaporation according to Dalton (1502), e_s: water pressure at the surface of water, e_a : water pressure of the air, c: factor.
Haude in 1952 developed that equation to

$$E_H = f \, . \, s$$

Where, E_H: Evaporation after Haude, f: a factor depending on the month of the year according to Haude (1952), s: Saturation deficit of the air in mbar.

3.1 Background for the calculation of evaporation

1. After the dry season, evaporation can only start when rainfall begins and soil moisture becomes available for evaporation.
2. Evaporation can take place only as long as soil moisture is available in the capillary fringe of rocks and soils and that period in Jordan extends from November to April.
3. Moisture in soils and rocks exceeding the capillary fringe capacity either remains in the soil and/or rock or move downwards by gravity and capillary forces of the unsaturated rocks lying further beneath the top layer.
4. Soil and rocks are considered to contain water available for evaporation during the whole rainy season.
5. Water retention needs of soils and rocks down to the depth of the capillary fringe are considered as a part of the evaporation water amounts.
6. The recharge effective rainy season is November to March of each year and the evaporation effective season from November to April.
7. Monthly evaporation is calculated according to Haude (1952):

$$E_H = f.s$$

Considering the average monthly maximum daily temperatures and the relative humidity or saturation deficit of the air, the evaporation for the period of the year, when soil moisture is available for evaporation can be calculated (Tables 1-7). The calculated evaporation amounts resemble those of potential evaporation for the studied stations.

Stanhill (1969) gives for the open water surface of Lake Tiberias an evaporation rate of mm 283mm for the months November through March (Table 8). Worth mentioning is that Lake Tiberias site is the only site in the area where actual evaporation rates have been calculated and calibrated. In the present calculation, evaporation during the same time period from the land surface in Baqura station, a few kilometers to the south of Lake Tiberias sums up to 234.5 mm. Class A- Pan Evaporation for Baqura area for the same period is given in the NWMP (1977) of Jordan as 443mm. Evaporation after Stanhill is 64% and evaporation in the present study 55% of the Class A-Pan Evaporation. The discrepancy can be explained by the nature of the two evaporation surfaces, Baqura, which is a land surface and Lake Tiberias, which is an open water surface. By applying the ratios calculated above to the Class A-Pan evaporation figures given in the NWMP (1977) shows that the discrepancies between the calculated evaporation amounts in this study and those calculated using the ratios do not exceed a few percentages. This justifies the approach using Haude's equation for the calculation of evaporation rates.

Using Wundt (1939) equation and Keller (1961) formulation of that equation (given below) for the calculation of evaporation and using the same runoff ratios given in Tables 1-7 results in a reduction in the rate of groundwater recharge of around 25% for areas receiving more than 450mm/yr of precipitation if precipitation rates decrease by 10%.

$$E_w = ppt / \{0.95 + ppt / f(t)\}^3$$

Where, E_w : Evaporation after Wundt, ppt: Precipitation amount per year (mm), f(t) : Temperature factor according to Wundt. This result confirms the reduction rates calculated according to Haude's equation (Tables 1-7).

The soil retention capability for water is very essential for the calculation of infiltration and groundwater recharge. To measure it 27 samples collected in cylinders were dried at 105oC. After

that they were saturated with water by merging the cylinders containing the dry soil in water containers allowing water to enter them from the bottom to the top for 12 hours. After that they were allowed to drain by gravity after being covered with plastic sheets to minimize evaporation. Drainage stopped in most samples after less than two hours.

Table 1. Water balance for Salt area

	Month/ parameter	**Nov.**	**Dec.**	**Jan.**	**Feb.**	**Mar.**	**Sum**
1	Temp. C^0	18.6	13.2	11.1	11.7	15.1	
2	ppt (mm)	51.9	105.9	140.2	119.6	94.6	
3	Rh. at noon %	61	70	74	73	65	
4	Sat.vap. p. (Torr)	16.1	11.4	9.9	10	12.9	
5	Total evaporation	50.6	27.4	20.7	19.9	46.5	
6	Flood runoff	1.3	2.6	3.5	2.3.	2.4	
7	Evap. + flood runoff	51.9	30	24.2	22.2	48.9	
8	Net infilt. into soil	0	75.9	116	96.9	45.7	335
9	Soil retention						48
10	Net recharge to gr. w.						287
1	10% less ppt	46.7	95.3	126.2	107.6	85.1	
2	Flood runoff	1.2	2.4	3.2	2.7	2.1	
3	Evap. + flood runoff	51.8	29.8	23.9	22.6	48.6	
4	Net infilt. into soil	0.0	65.5	102.3	85.0	36.5	289.3
5	Soil retention						48
6	Net recharge to gr. w.						241.3
7	Reduction in net recharge						16%

Table 2. Water balance for Baqura area

	Month/parameter	**Nov.**	**Dec.**	**Jan.**	**Feb.**	**March**	**Sum**
1	Temp. C^0	26.5	20.2	18.4	19.7	23.2	
2	ppt	45.1	80	93.5	74.8	56	
3	Rh. at noon %	62	71	74	74	70	
4	Sat.vap.p. (Torr)	26	17.7	15.9	17.2	21.3	
5	Total evaporation	77.2	41.1	33	32.8	82.6	
6	Flood runoff	0.5	0.8	0.9	0.8	0.6	
7	Evap. + flood runoff	77.7	41.9	33.9	33.6	83.2	
8	Net infilt. into soil	0	38.1	59.6	41.2	0	139
9	Soil retention						48
10	Net recharge to gr. w.						90.9
1	10% less ppt	45.2	72	84.2	67.3	50.4	
2	Flood runoff	0.4	0.7	0.8	0.7	0.6	
3	Evap. + flood runoff	77.6	41.8	33.8	33.7	83.1	
4	Net infilt. into soil	0.0	30.2	50.4	33.6	0.0	114.2
5	Soil retention						48
6	Net recharge to gr. w.						66.2
7	Reduction in net recharge						27.1%

Table 3. Water balance for the University of Jordan area

	Month/parameter	**Nov.**	**Dec.**	**Jan.**	**Feb.**	**March**	**Sum**
1	Temp. C^0	18.1	12.3	10.2	11.7	15.2	

2	ppt	48.5	94.8	116.9	105.1	85.1	
3	Rh. at noon %	58.1	67.4	70	65.7	59	
4	Sat.vap.p. (Torr)	15.6	12.1	9.4	10	13	
5	Total evaporation	49.1	31.4	22.6	24.8	54.7	
6	Flood runoff	0.9	1.7	2.1	1.9	1.5	
7	Evap. + flood runoff	50	33.1	24.7	20.7	56.2	
8	Net infilt. into soil	0	61.7	92.2	78.4	28.9	261
9	Soil retention						48
10	Net recharge to gr. w.						213
1	10% less ppt	43.7	85.3	105.2	94.6	76.6	
2	Flood runoff	0.7	1.3	1.6	1.4	1.1	
3	Evap. + flood runoff	49.8	32.7	24.2	26.2	55.8	
4	Net infilt. into soil	0.0	52.6	81.0	68.4	20.8	222.8
5	Soil retention						48
6	Net recharge to gr. w.						174.8
7	Reduction in net recharge						23%

Table 4. Water balance for Wadi Rayyan area

	Month/parameter	**Nov.**	**Dec.**	**Jan.**	**Feb.**	**March**	**Sum**
1	Temp. C^0	26.7	20.5	18.8	20.3	23.8	
2	ppt	35.4	62.8	71.6	58	44.4	
3	Rh. at noon %	68.5	77	78.9	77.1	72.8	
4	Sat.vap.p. (Torr)	26.3	18.1	16.3	17.8	22.1	
5	Total evaporation	64.7	33.9	27.4	29.8	61.4	
6	Flood runoff	0.9	1.6	1.8	1.5	1.1	
7	Evap. + flood runoff	65.6	35.5	29.2	31.3	62.5	
8	Net infilt. into soil	0	27.3	42.4	26.7	0	96.4
9	Soil retention						36
10	Net recharge to gr. w.						60.4
1	10% less ppt	31.9	56.5	64.5	52.2	40	
2	Flood runoff	0.6	1.0	1.2	1.0	0.7	
3	Evap. + flood runoff	65.3	34.9	28.5	30.8	62.1	
4	Net infilt. into soil	0.0	21.6	35.9	21.4	0.0	78.9
5	Soil retention						36
6	Net recharge to gr. w.						42.9
7	Reduction in net recharge						59%

Table 5. Water balance for Irbid area

	Month/parameter	**Nov.**	**Dec.**	**Jan.**	**Feb.**	**March**	**Sum**
1	Temp. C^0	20.6	15	13	14.8	17.1	
2	ppt	50.4	82.1	109.1	97.1	85.9	
3	Rh. at noon %	60.4	71.3	73.6	70.8	65.9	
4	Sat.vap.p. (Torr)	18.2	12.8	11.2	12.6	14.6	
5	Total evaporation	56.2	24.2	24.2	26.8	51.7	
6	Flood runoff	2.7	4.3	5.7	5.1	4.7	
7	Evap. + flood runoff	58.9	28.5	29.9	37.9	56.4	
8	Net infilt. into soil	0	54.5	79.3	66.2	29.5	230
9	Soil retention						48

10	Net recharge to gr.w.						182
1	10% less ppt	45	73.9	98.2	87.4	77.3	
2	Flood runoff	2.5	4.1	5.4	4.8	4.3	
3	Evap. + flood runoff	58.7	28.3	29.6	31.6	56.0	
4	Net infilt. into soil	0.0	45.6	68.6	55.8	21.3	191.3
5	Soil retention						48
6	Net recharge to gr. w.						143.3
7	Reduction in net recharge						21%

Table 6. Water balance for Madaba area

	Month/parameter	**Nov.**	**Dec.**	**Jan.**	**Feb.**	**March**	**Sum**
1	Temp. C^0	20.2	14.1	12.7	14.1	14.5	
2	ppt	37.4	64.1	84.1	74.4	61.4	
3	Rh. at noon %	61.5	71.3	74.3	70.4	63.9	
4	Sat.vap.p. (Torr)	17.7	12.1	11.1	12.1	15	
5	Total evaporation	53	28.2	22.6	26.2	56.8	
6	Flood runoff	1	1.8	2.4	2.1	1.7	
7	Evap. + flood runoff	54	30	25	28.3	58.5	
8	Net infilt. into soil	0	34.1	69.1	46.1	2.9	152
9	Soil retention						48
10	Net recharge to gr. w.						104
1	10% less ppt	33.7	57.7	75.7	67.0	55.3	
2	Flood runoff	0.8	1.4	2.1	1.9	1.5	
3	Evap. + flood runoff	53.8	29.6	24.7	28.1	58.3	
4	Net infilt. into soil	0.0	28.1	51.0	38.9	0.0	118
5	Soil retention						48
6	Net recharge to gr. w.						70
7	Reduction in net recharge						33%

Table 7. Water balance for Deir Alla area

	Month/parameter	**Nov.**	**Dec.**	**Jan.**	**Feb.**	**March**	**Sum**
1	Temp. C^0	27.2	21.2	19	20.3	23.7	
2	ppt	35.8	54.7	66.7	54.1	44.5	
3	Rh. at noon %	50.1	60.3	62.9	63.9	60.4	
4	Sat.vap.p. (Torr)	27.1	18.9	16.5	17.9	22	
5	Total evaporation	105	60.5	49.2	47.3	89.9	
6	Flood runoff	0	0	0	0	0	
7	Evap. + flood runoff						
8	Net infilt. into soil	0	0	17.5	6.8	0	24.3
9	Soil retention						24
10	Net recharge to gr. w.						0.3

The water retention ranged from 16% to 28% with an average of 24% of the total soil volume. Soil thicknesses of 10, 15, and 20cm can hence hold 24, 36, and 48 mm of rain water respectively. If the amount of rain exceeds these figures soil water will move further down by gravity to reach areas beyond the reach of capillary forces. Therefore the amounts of 24, 36, and 48 mm of rain water are considered recharge ineffective rain. They are lost by evaporation or evapo-transpiration at a certain stage.

Soils in hydrologic terms serve as a water storage reservoir for evaporation. Therefore, decreasing precipitation will affect the saturation degree of the soil during the rainy season allowing herewith less water to move down from the soil horizon into the groundwater regimes.

A 10% decrease in precipitation will result in the study area in decreasing groundwater recharge rates as shown in Table (9).

Table 8. Long term average monthly evaporation from Lake Tiberias, mm/month (Stanhill 1969)

Nov.	Dec.	Jan.	Feb.	March	Total
71	59	52	43	58	283

Table 9. Decreases in groundwater recharge rates as a result of 10% decrease in precipitation rates

Station/	Salt	Madaba	Jordan University	Irbid	Deir Alla	Baqura	Rayyan
Present recharge mm/yr	286.5	104.2	213.2	181.5	0.3	90.9	60.4
Decrease in groundwater recharge							
in mm/yr	46.7	34	38.2	38.7	0.0	24.7	17.1
in % of total recharge	16	33	23	21	0.0	27.1	59

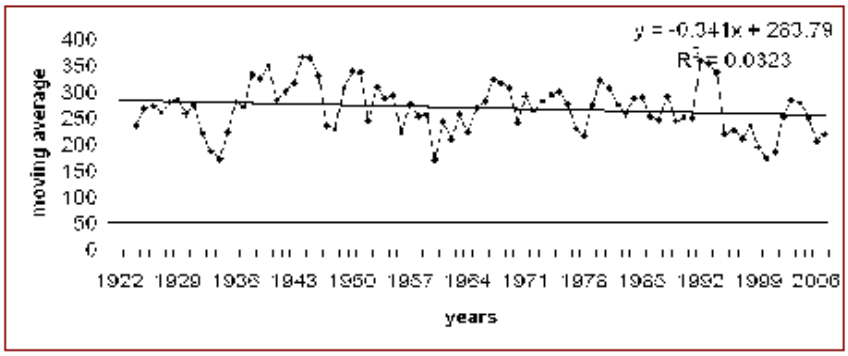


Figure 2. Annual rainfall in Amman Station

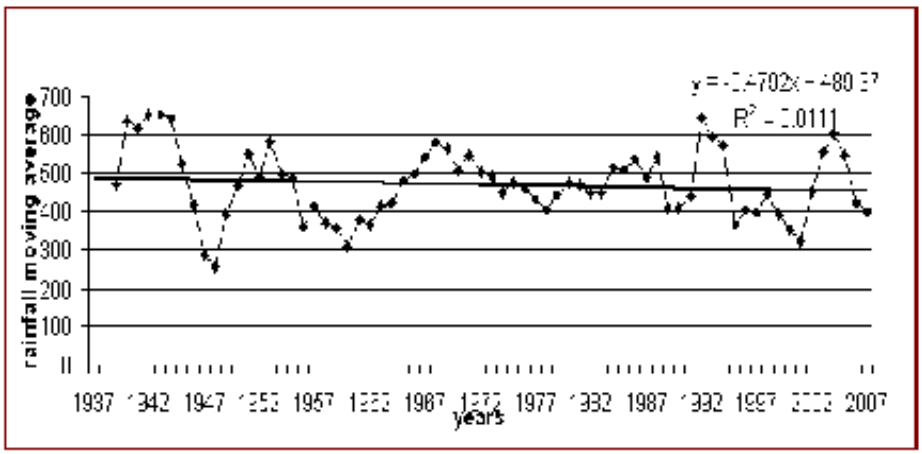


Figure 3. Annual rainfall in Irbid station

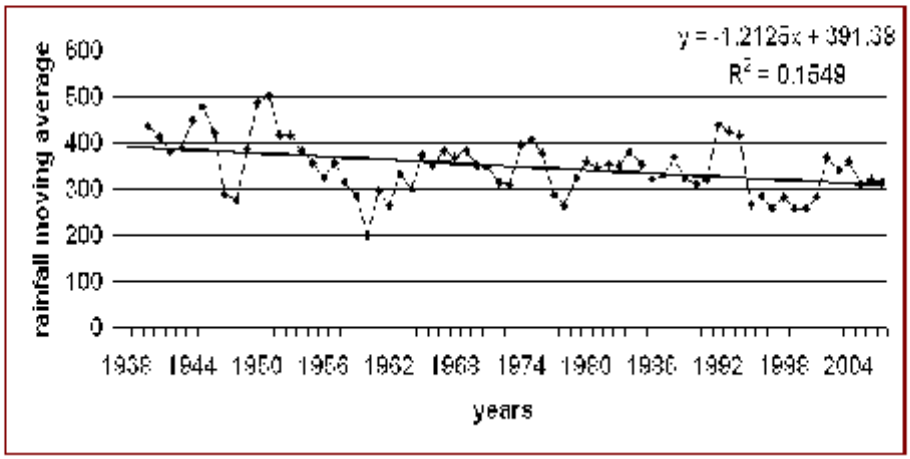


Figure 4. Annual rainfall in Madaba station

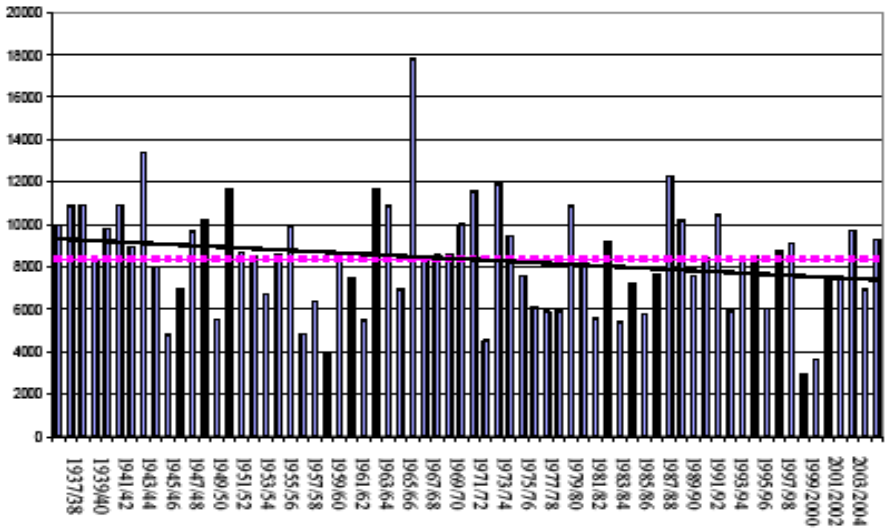


Figure 5. Total volume of precipitation over Jordan from 1937/38 to 2005/2006

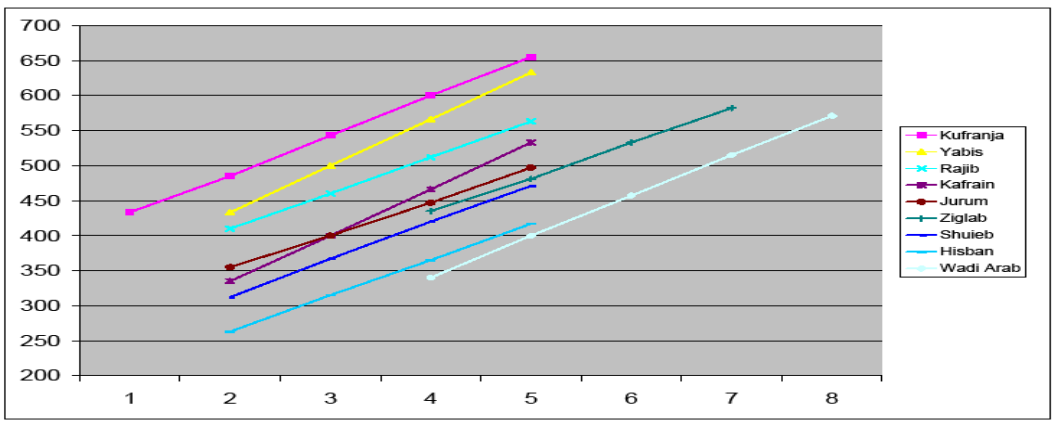


Figure 6. Correlation of flood run off/precipitation ratio (in %) and rainfall amounts (mm/yr) for different catchments in the study area

4. CONCLUSION

Climatic changes are expected to have their rigorous effects on precipitation amount and distribution, air temperature and humidity, which in turn will have appropriate impacts on the surface and ground water resources, soil moisture, water quality and socio-economics.

There are strong indications that climatic changes are affecting the amount of precipitation falling over the northwestern parts of Jordan, which have been showing decreasing trends of precipitation during the last few decades.

Flood runoffs of Wadis will be affected by two mechanisms, namely: first, as a result of runoff amounts being directly proportional to precipitation amounts and second as an indirect result of lower flood runoff/ precipitation ratios inherent in lower precipitation amounts. Such mechanisms will result in a flood runoff reduction in northwest Jordan of 39%, if the precipitation amount decreases by 10%.

The results show also that a decrease in precipitation of 10% will result in a reduction in groundwater recharge of 16% in rain rich areas receiving more than 500mm/yr increasing to 59% in areas receiving moderate precipitation of around 300mm/yr.

Countries in the MENA region should gradually become aware of the climatic processes affecting their surface and ground water resources with all the socio-economics and stability implications on these countries. That may trigger taking the necessary measures to alleviate the negative impacts.

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EVALUATION OF POTENTIAL SITES FOR MANAGED AQUIFER RECHARGE (MAR) USING SURFACE INFILTRATION IN NW-JORDAN

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Abstract: The purpose of this work was to evaluate the potential of Managed Aquifer Recharge (MAR) via surface infiltration in NW Jordan for an area of about 9800 km². A MAR potential map scaled 1:200.000 was prepared based on several parameters such as hydraulic characteristics of the near surface lithology, topography, land use and availability of water to be infiltrated. The map was prepared in a GIS-based thematic layer overlay approach. The resulting areas of the combined thematic layers were delineated to four different MAR potential classes (low, medium, high very high). 30 % of the delineated MAR potential classes show high potential, 14 % very high potential. The results show that 1388 km² in NW Jordan have been evaluated to fulfil the high potential criteria for MAR via surface infiltration. This means that at these locations an aquifer crop out at the surface, or are covered by permeable sediment layers, with hydrogeological characteristics allowing artificial recharge (mainly limestone and alluvial aquifers with sufficient hydraulic conductivity). In addition the unsaturated zone is thick, the slope at the surface is < 5 %, urban areas are absent, and finally the proximity to at least one water source exists. The water sources comprise dams, waste water treatment plant and surface runoff. In that context all water sources suitable for MAR in the study area were listed and described regarding their capacity, catchments and purpose. MAR is considered as a valuable alternative to water storage at the surface and as a valuable countermeasure to declining groundwater levels. The results of this overview MAR map can be understood as basis that identifies potential areas for further studies that aim for a detailed planning of surface infiltration. In Jordan MAR has not only the benefit of reducing evaporation losses it can also play an important role in water reuse, by improving the water quality (e.g. when soil-aquifer treatment is applied) and storage opportunities to balance seasonal differences between supply and demand for reclaimed sewage effluent. Furthermore, it can be valuable to store the strong episodic rainfall runoff from the highlands towards the Jordan Valley and the Dead Sea during the rainy season.

Key words: SMART, artificial recharge, Jordan, thematic layer approach, water resources, water management

1. INTRODUCTION

1.1 Background

The natural water resources in Jordan are limited due to the prevailing semiarid climate which is typified by high evaporation (evapotranspiration) rates and low seasonal precipitation, which in turn results in a deficit of groundwater recharge. The increasing water demand due to high population growth, prevalent irrigated agriculture and management deficits, results in overexploitation of the water sources. Low water prize, interrelation with surrounding countries by sharing groundwater basins and surface catchments constitutes additionally to the water scarcity in Jordan. Deteriorated surface water quality due to reduction in natural flow volumes and sewage disposal is observed as well as increasing salinity of the groundwater. Northwest Jordan shows the highest groundwater abstraction of entire Jordan. The abstraction rates exceed the natural recharge rate of the aquifers, resulting in continually declining groundwater levels. The increasing water demand due to high population growth, prevalent irrigated agriculture and management deficits, results in

overexploitation of the water sources. The aim of this study is to evaluate potential areas for Managed Aquifer Recharge (MAR) using
surface infiltration in NW-Jordan considering basic parameters applied in different MAR concepts. The delineation of potential areas is based on the hydraulic character of near surface lithologies, topography, land use (urban areas) and availability of water. The study was conducted in the framework of the SMART project (Sustainable Management of Available Water Resources with Innovative Technologies). This project is part of the "Integrated Water Resources Managements" initiative (IWRM) for regions with water shortages funded by the German Federal Ministry for Research and Technology (BMBF) from 2006-2009.

1.2 Managed Aquifer Recharge (MAR)

Artificial recharge or managed aquifer recharge (MAR) encompasses the process of infiltrating water in the underground into existing voids of permeable rocks with the overall intention to increase the volume of water in aquifers by storage and treatment of water. The term Managed Aquifer Recharge was introduced by Bouwer (2002) to replace the term Artificial Recharge, with the intention to replace the "artificial", which is today often negatively related. Furthermore the term "artificial recharge" describes al kind of aquifer recharge taking place due to human interaction no matter if it is the purpose or not, while the term "managed aquifer recharge" refers only to intended recharge.

Three basic principles of managed aquifer recharge can be distinguished: Point infiltration, surface infiltration and vadose zone infiltration. Surface infiltration comprises the MAR techniques were the vertical movement of water into the aquifer prevails. The water is impounded, by natural or implemented structures, over a permeable ground with connection to-, or directly on the aquifer. Surface infiltration is the favoured MAR method in regions with abundant land availability, highly permeable surface over unconfined aquifers (Ghayoumian, 2004) because of best clogging control and soil/aquifer treatment and low installation and maintenance costs (Bouwer, 2002). Point infiltration on the contrary means that the conduction of water into the aquifer is concentrated to a point. This "point" is represented by an injection well. The recharge of the aquifer is dominated by the lateral movement of the infiltrating water. Because the concentration of the recharge to a "point" recharge of the aquifer is restricted to a relatively small area in comparison to surface infiltration by taking the whole installation structure into account. Point infiltration is a valuable method where permeable soils and/or sufficient land area for surface infiltration are not available, vadose zones not suitable or aquifers deep or confined (Bouwer, 2002). The vadose zone infiltration can be considered as a combination of surface and point infiltration, as an alternative to surface infiltration if the soil cover on the aquifer is not permeable enough. This can be obtained by the installation sand or gravel backfilled trenches or other structures penetrating into the underground with restricted surface extension.

In Jordan the prevailing effects and intentions of MAR can be: the long term recovery of the overexploited groundwater basins, especially the Upper Limestone Aquifer in North Jordan; the annual storage of surplus water for times of water shortage (surface runoff & surface water); the improvement of water quality through soil and underground passage, e.g. to extend the reuse of reclaimed waste water which is nowadays in Jordan mainly discharged in the Rivers and Wadis; the reduction of salt water intrusion from saline aquifers above or below the aquifer exploited; and the retardation of land subsidence caused by declining groundwater levels hence declining base flow as it is e.g. the case on the Lisan Peninsula of the Dead Sea where sink holes developed.

2. STUDY AREA

The study area covers NW Jordan comprising around 9800 km² with a maximum NS extension of 143 km and a maximum EW extension of around 110 km (Figure 1). To the N and NW the study

area extend to the border to Syria, to the W to the borders to Israel and the West Bank. The eastern border is defined by the groundwater divide to the Azraq Basin which is in the northern part adjoined to the Amman-Zarqa- and in the southern part to the Dead Sea Groundwater Basin as defined by the National Water Master Plan (NWMP, 2004). The site selection was driven by the intention to cover that part of Jordan with the highest groundwater abstraction rates, due to the highest population density of Jordan and main area for irrigated agriculture (evaluated by Hobler et al, 2001, presented and modified after NWMP, 2004). In Jordan a dry Mediterranean climate is characteristic for the Northern and Central Highlands, while in the east dry continental desert conditions prevails. Precipitation amounts decreases rapidly from west to east and more gradually from north to south ranging between < 100 mm/y and > 700 mm/y. December to March is the major period of precipitation, with 80 % of the total average rainfall. The average temperatures show a reverse pattern increasing rapidly from the Highlands to the Jordan-Araba Valley and decreasing gradually from north to south in line with increasing altitude. Precipitation is exceeding the evapotranspiration only in the Highlands, during the winter months from December to February/March.

The study area comprises several land regions that are widely differing in their characteristics namely the Jordan Valley, the Jordan Valley escarpment, Northern and Central Jordan Highlands and North Jordan Basalt Plateau. The regions were defined on the base of climate, vegetation and land use, geology and geomorphology, topography, hydrology and Landsat imagery characteristics (MWI, 1993). The dominating tectonic element of the Jordan River Valley is the Dead Sea Transform (DST), an active plate boundary and segment of the East African – Red Sea Rift System. The Jordan Valley escarpment, which is formed by the uplifted graben shoulders, borders the Jordan Valley. Altitude reaches maximum levels to the east of about 1200 m a.m.s.l. descending to a minimum at the Dead Sea of > 400 m b.m.s.l. On the Northern and Central Jordan Highlands west of the valley floor Middle Cretaceous carbonates und Upper Cretaceous clays, marls, chalks and cherts are exposed. The northern part of the Highlands is bordered by the Basalt Plateau that covers the majority of the eastern part of the study area.

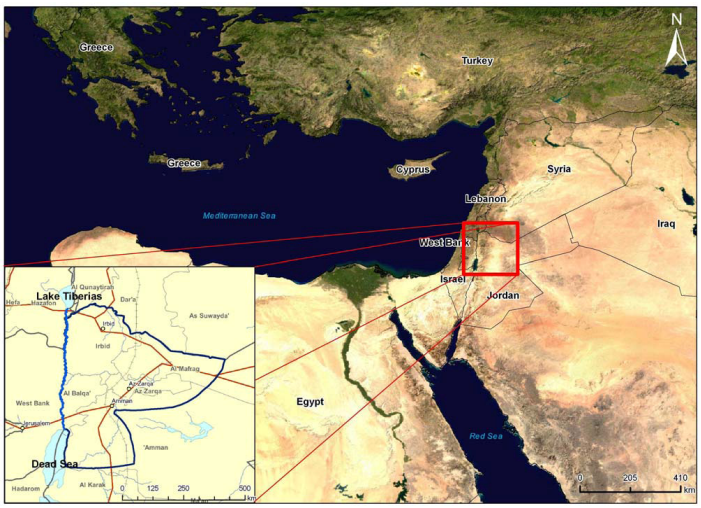


Figure 1. Location of the study area (blue marked area on the enlarged map).

In the study area, three aquifer types can be distinguished: fractured, unconsolidated and karst aquifers. They build three multi-layered aquifer systems: the Ram-Zarqa- Kurnub, the Upper Cretaceous Limestone and the Tertiary-Quaternary Shallow NWMP (2004) (Figure 2). The Ram-Zarqa- Kurnub Aquifer System consists of the fractured Ram aquifer (Palaeozoic sedimentary sequences), the Azab aquifer (Jurassic sediments), the Khreim aquitard (Middle Ordovician-Upper Silurian sediments) and the fractured Kurnub aquifer (Lower Cretaceous sediments). In the south eastern part of the study area the Ram aquifer is directly overlain by the Kurnub aquifer, while in the central and north western part by the Zarqa aquifer (Jurassic sediments). Above this aquifer

system the Upper Cretaceous Limestone Aquifer System follows. This complex consists of an alternating sequence of limestones, dolomites, marl stones and chert beds. It includes in stratigraphic order the A1/A2 karst aquifer, the A3 aquitard, the A4 karst aquifer, the A5/A6 aquitard and the A7/B2 karst aquifer. The Tertiary-Quaternary Shallow Aquifer system includes the B3 aquitard, the B4/B5 karst aquifer, the Basalt aquifer and alluvial deposits.

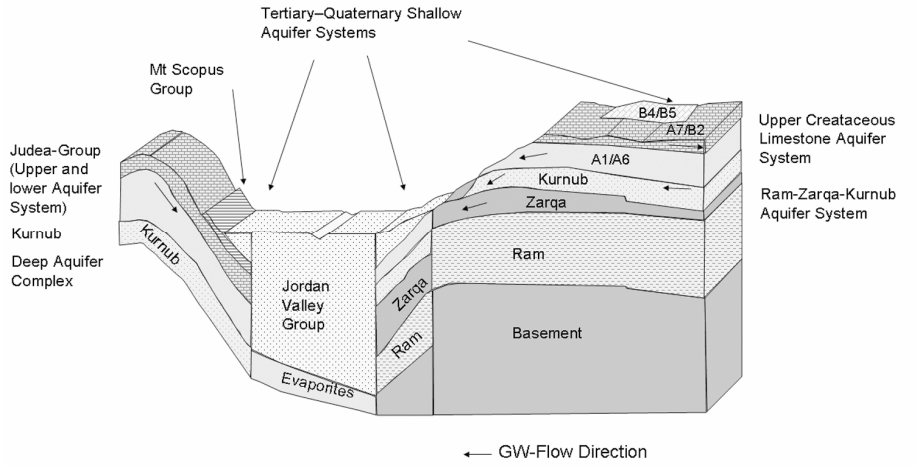


Figure 2: Schematic hydrogeologic profile of the Lower Jordan Valley (Wolf et al. 2007)

3. METHODOLOGY

3.1 Parameters for MAR

In the study area in Jordan many areas with abundant land availability and highly permeable surface over an unconfined aquifer exist, thus surface infiltration is regarded to be the preferential method of MAR because of best clogging control and soil/aquifer treatment (Bouwer, 2002). The potential of MAR using surface infiltration was mapped based on the parameters aquifer potential of the bedrock, topography and the proximity to water sources. These parameters are considered as the basis of MAR site selection in case of surface infiltration via ponds or dams is planned. Based on these parameters the following thematic maps are prepared using GIS: hydrogeological classes, slope, land cover (urban areas), proximity to water sources. The resulting potential map of the combined thematic maps gives an overview of favoured areas for MAR in NW Jordan.

3.2 Thematic MAR layers processing

3.2.1 Hydrogeological classes

The hydrogeological classes for MAR are based on the lithology and hydrogeological characteristics of the aquifers/aquitards. The geological units were divided in three different classes due to their hydraulic characteristics: aquitard, aquitard/aquifer, aquifer. The aquifers are further subdivided in aquifer types: fractured rock aquifer, unconsolidated aquifer, karst aquifer (Table 1). Due to the wide range in hydraulic conductivities (especially for the limestones, which show different degrees of karstification) found in the literature for the hydrogeological units, this subdivision in aquifer types does not affect the later potential shown on the map. Nevertheless, the subdivision was retained to give an idea about the general hydraulic characteristics of the diffenrent bedrocks, even if the hydraulic conductivity was not taken into account as MAR parameter. Besides, a high hydraulic conductivity does not necessarily lead to a high MAR potential, since the water cannot be stored effectively in the underground if the retention time is to short due to high hydraulic conductivities.

In the thematic map the hydrogeological classification follows the New Water Master Plan classification (NWMP, 2004), with the exception of the Zarqa unit which was classified as aquifer/aquitard instead of aquifer due to its really high variability in lithology (Figure 2a). The distribution of the lithologies is based on the geological maps prepared by the National Resources Authority in Amman (NRA) (NGMP 1991). Thin soil or sedimentary units above aquifers where not taken into account and thus stripped and replaced by the simplified hydrogeological map of North Jordan (Hobler et al., 1994). Thick Holocene alluvial unconsolidated deposits are widely distributed in the study area. They facilitate recharge via surface infiltration in respect to high infiltration rates and to the construction of pond an trenches for surface infiltration. Zeeman (2008) carried out a long term observation of hydraulic characteristics of the alluvial fan in Wadi Kafrein which is regarded as a representative alluvial wadi fill. An infiltration rate in an infiltration pond between 3.48 m/day to 4.36 m/day was calculated by taking the developing clogging layer into account. Thick Holocene superficial alluvial sediments are marked on the potential map with a special signature to indicate additional potential of surface infiltration (Figure 2d).

Table 1. Hydrogeological classification of the lithology.

Hydrogeological Class	Class ID	Hydrogeological unit	Unit ID
Aquitard	0	Lisan Marl	LM
		Muwaqqar	B3
		Shueib	S
		Fuheis	F
Aquifer/Aquitard	1	Jordan Valley 1	JV1
		Jordan Valley 2	JV2
		Lower Ajlun Series	A1-A6
		Zarqa	Z
Fractured rock	2	Basalts	B
		Kurnub	K
		Ram	R
Unconsolidated	3	Neogene alluvials of the JV	AL
		Lisan gravel/beachrock	Lmg/Lmb
Karst	4	Umm Rijam/Shallala	B4/B5
		Wadi as Sir/Amman	A7-B2
		Hummar	A4
		Naur	A1/A2
Shallow Unconsolidated	5	Superficial Alluvials	SAL

3.2.2 Slope

Gradient is an important factor for infiltration processes. The potential for surface infiltration of water decreases, whereas surface runoff velocity and erosion increases if the slope of a surface exceeds 5 % (Saraf & Choudhury 1998; Jasrotia et al., 2007; Shahid et al. 2000). The most effective range of slope is considered to be 1 % - 5 % and therefore classified as suitable regarding MAR surface infiltration potential. Although, slopes < 1% are considered to be less effective they are still classified suitable. Areas with a slope > 5 % are delineated to have a low potential for MAR via surface infiltration (Figure 2b).

The thematic map of the slope is based on two different digital elevation models (DEM). The first one has a spatial resolution of approximately 25 m (source: John Hall, GSI). Since it does not cover the whole study area, the missing part in the east was covered using SRTM data (source: http://srtm.csi.cgiar.org/SELECTION/inputCoord.asp). The SRTM data used has a spatial resolution of 3 arc seconds or accordingly approximately 90 m. After converting and combining the two different elevation models the slope (in percentage) was calculated and classified in two classes: slope 0 % - 5 %, slope > 5% . Areas, where the bedrocks are delineated as an aquitard in the hydrogeological classes layer, maintain their low potential in the map and are not upgraded by the slope- and water sources layer.

3.2.2 Land cover

Surface infiltration, in comparison to point infiltration, requires an adequate area that is big enough for excavation or construction of an infiltration basin or a dam without interfering in existing land use. The most suitable and economic way for site construction would be to choose a region without urban or agriculture areas. Whereas agriculturally used areas does generally not restrict surface infiltration, and proximity of water for recharge and potential users (demand) of the water is an important factor. For the potential map only urban areas and smaller settlements are considered as a restricting factor and delineated in the map as low potential areas. The areas were digitised based on topographical maps 1:50 000 of the Royal Jordanian Topographic Centre (1997).

3.2.2 Proximity to water sources

The basic requirement for recharging groundwater using MAR is the source water availability. For the potential map all existing and planned dams and waste water treatment plants (WWTP) and the drainage pattern of the surface water runoff were taken into account (Figure 2c). The location of infiltration basins should be determined to optimize a reasonable proximity to a water source because then the maximum gradient can be developed and construction and maintenance costs can be saved. The distance of around 5 km was considered to be an economical distance for the transport of water via a gravity pipeline or canal. For the potential map a buffer was used around the WWTP's and dams covering all areas in a linear distance of 5 km with an altitude lower than the altitude of the dam side. The effective distance for the transport will in most cases exceed the 5 km, therefore this relatively short distance was chosen. The natural flow direction along the steepest path is illustrated in the map by the drainage pattern. If a suitable site for MAR is located nearby, this is obviously the most economic choice for site selection. In addition to the drainage pattern a buffer was built. This buffer envelopes the calculated drainage of 250 m on each side to indicate the vicinity to a potential water source for MAR due to surface water runoff, comprising at least an catchment area of 3 km². By way of example, 3 km² chosen in a region with an average annual rainfall of 300 mm would provide around 900 000 m^3/year of water to be infiltrated as it is e.g. the situation in Wadi ed Dardur in Jordan where actually a dam site for MAR is in planning process.

The drainage pattern of the study area was generated using the Arc Hydro tool for the digital elevation models which enables the delineation of the catchment area (larger than 3 km²).

3.3 Combination of thematic layers

In a first step the thematic hydrogeological layer was combined with the thematic slope layer resulting in low, medium and high potential MAR areas. Low and medium potential have one signature each, while the high potential areas retain three different signatures depending on the aquifer type (fractured, unconsolidated, karst). Areas with low and medium classes were neglected for the combination with the water sources layer since they cannot be upgraded by the existence of a water source nearby. The remaining areas were combined with the proximity to water sources layer. Areas that provide a high MAR potential based on hydrogeology and slope and are located within the defined distance to a water source are classified to have very high MAR potential. In a last step alluvial deposits and urban areas were overlayed. Alluvial deposits, characterised by high values of hydraulic conductivity, indicate an enhanced potential for surface infiltration, whereas urban areas classify every area to low potential, independently from the output of the thematic layer (Figure 2d).

4. RESULTS

The downsized final MAR surface infiltration map is shown in Figure 2d, the original mapping scale was 1:200.000. Areas delineated to have a high potential inherit three different signatures (colours) in respect to retain their hydrogeological aquifer classification.

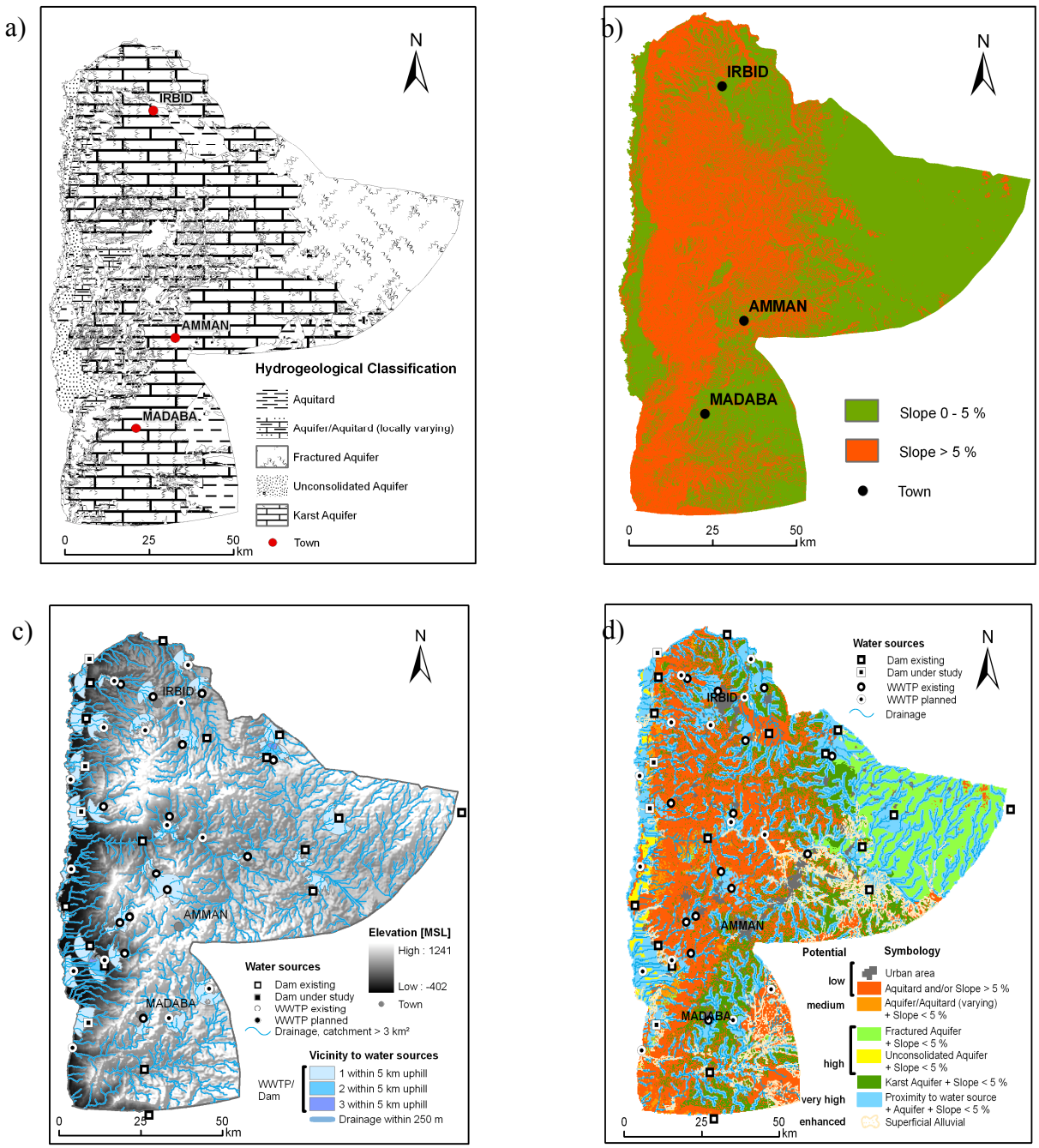


Figure 2. Thematic layers and resulting potential MAR map using surface infiltration.
a): hydrogeological classes, b): slope classification, c): proximity to water source, d): MAR potenial map.

1388 km² (~14%) of the study area are delineated to have a very high potential for the aquifer recharge via surface infiltration. The high potential areas cover about 3003 km² (~30 %). Around 55 % of the area have a low potential, in 45% of the whole study area due to an unsuitable slope (> 5 %). Restrictions resulting from areas classified as aquitard are with 16 % negligible, as well as the urban areas with 4 % of the whole study area. The small dissected pattern of areas which are delineated to different potential classes in some regions is due mainly to different slope classes.

5. CONCLUSIONS

The results of the MAR potential map using surface infiltration shows that there is a good potential in NW Jordan for MAR. Aquifers with high transmisivities are available close to the surface and

surface runoff collected for example in dams or water from wastewater treatment plants exist that can serve as water supplier. Due to the large mapping area the potential MAR evaluation was restricted to few basic parameters namely lithology, topography, settlements and availability of water. By choosing a distinct area for planning and constructing a MAR-site much more parameters such as aquifer confinement, permeability, thickness, salinity, lateral hydraulic gradient, aquifer mineralogy (Dillon and Jiminez, in press) and recovery of the injected water have to be considered. The result of potential MAR sites map in NW Jordan was compared with existing MAR sites for surface infiltration and MAR studies (Rapp, 2008). All sites are located in areas that were classified to have a very high potential.

6. ACKNOWLEDGEMENTS

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CONSTRUCTION OF ISOERODENT CURVES FOR A TRANSBOUNDARY BASIN

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Abstract: The study area is the area between Guadiana River, flowing through the south-eastern part of Portugal, and the Portugal – Spain border. In the framework of a research project concerning the drought study in the transboundary basin mentioned above, the soil erosion due to rainfall was also computed for the basin. The computational results of soil erosion over the basin are represented by a map of isoerodent curves. The basin area is about 2853 km^2. The pilot basin was divided into 136 sub-basins by means of a quadrangular grid (5 km x 5 km). For the computation of soil erosion, the classical "Universal Soil Loss Equation" (USLE) was used. Monthly rainfall data of 14 meteorological stations located in Portugal were available. The longest time period, for which rainfall data are available, is 74 years, while the shortest one is 25 years. Apart from that, a contours map was constructed. For the construction of the isoerodent curves, the software "SURFER 8" was used. The computed mean annual soil erosion amount per unit area in the above basin is lower than the corresponding measured amount in basins located in Cyprus.

Key words: isoerodent curves, soil erosion, transboundary basin

1. INTRODUCTION

One of the unfavourable consequences of the drought conditions in a basin is the lack of vegetation, which again increases the influence of soil erosion by rainfall and runoff on the basin. On the other side, the low values of rainfall amount in a basin, where the drought dominates, decrease the influence of soil erosion on the basin.

In the framework of a research project (SADMO – INTERREG IIIB - MEDOCC) concerning the drought study in a transboundary area located between Guadiana River, flowing through the south-eastern part of Portugal, and the Portugal – Spain border, the soil erosion due to rainfall was also computed. The basin area studied is about 2853 km^2. For the computation of soil erosion, the classical "Universal Soil Loss Equation" (USLE) was used (Wischmeier and Smith 1978). The USLE was initially developed for the computation of soil loss in farms, but its use was later extended to small basins (e.g. Hrissanthou 1985, Hrissanthou and Pyliotis 1995).

Some years after the presentation of USLE, a revised version of USLE referred to as "Revised Universal Soil Loss Equation" (RUSLE) was presented (Renard et al. 1996). The RUSLE could not be applied to the basin considered because of the plethora of data required for its application. In the following sections, the differences between USLE and RUSLE will be summarized (Renard et al. 1994).

2. SHORT DESCRIPTION OF USLE

The classical USLE has the following form:

$$A = RK(LS)CP \tag{1}$$

where
A : mean annual soil loss per unit area (t/ha)
R : rainfall erosivity factor (N/h)

K : soil erodibility factor [(t/ha)/(N/h)]
LS : topographical factor
C : soil cover factor (cropping management factor)
P : erosion control practice factor

3. APPLICATION OF USLE TO THE PILOT BASIN

The pilot basin was divided into 136 sub-basins by means of a quadrangular grid (5 km x 5 km) (Figures 1, 2 and 3). Monthly rainfall data of 14 meteorological stations located in Portugal were available. The longest time period , for which rainfall data are available, is 74 years, while the shortest one is 25 years. The mean annual rainfall amount from the above stations is 512 mm.

3.1 Estimate of the rainfall erosivity factor

The rainfall erosivity factor (R) for a rainfall event is the product of two rainstorm characteristics, kinetic energy and the maximum 30 minutes intensity. It means that a continuous record of rainfall intensity is required for the computation of R. In this study, however, an empirical regression equation (Schwertmann et al. 1990) was used for the estimate of R because continuous rainfall records were unavailable:

$$R = 0.083N - 1.77 \quad (2)$$

where
R : rainfall erosivity factor (N/h)
N : mean annual rainfall amount (mm)

Equation (2) was applied to each sub-basin (grid cell) separately. The value of N for each sub-basin was determined by means of the constructed Thiessen polygons map (Figure 1; Iliopoulos and Charitopoulos 2007).

The disadvantage by using this equation is that it was developed for bavarian climatic conditions (Germany).

RUSLE computes a correction to R to reflect the effect of raindrop impact for flat slopes striking water ponded on the surface.

3.2 Estimate of the soil erodibility factor

The value of the soil erodibility factor (K) depends on the soil characteristics: percentage of sand, percentage of silt and very fine sand, percentage of organic matter, structure and permeability.

In this study, an average value for the whole basin was estimated approximately ($K = 0.28$) taking into account that most soils of the basin considered belong to the category of “lithosols” (Mitchell and Bubenzer 1980).

According to RUSLE, the factor K is adjusted to account for seasonal changes such as freezing and thawing, soil moisture and soil consolidation.

3.3 Estimate of the topographical factor

The topographical factor (LS) was computed by the following equation (in Mitchell and Bubenzer 1980):

$$LS = (l / 22.13)^m (0.065 + 0.045s + 0.0065s^2) \quad (3)$$

where

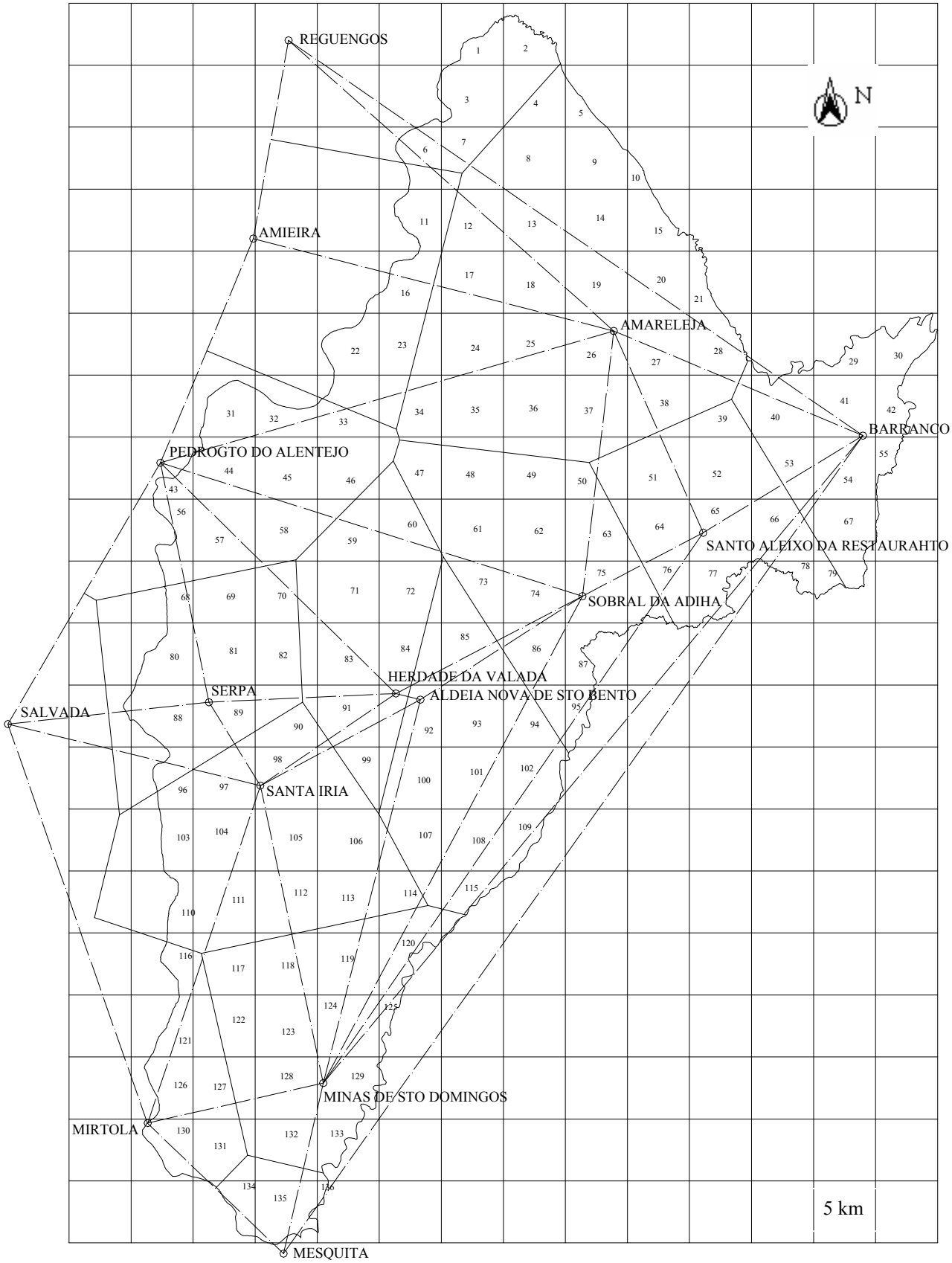


Figure 1. Thiessen polygons map

LS : topographical factor
l : slope length (m)
s : slope gradient (%)
m : exponent depending on the slope gradient

For slope gradients greater than 30%, the following relationship was used (van Vuuren 1982):

$$LS = (l / 22.13)^m 16[\sin 2(\arctan(s / 100))]^{1.6} \quad (4)$$

Equation (3) was applied to the most sub-basins, while Equation (4) only to four sub-basins. The slope length was taken as equal to 150 m (Huggins and Burney 1982). The slope gradient (s) was computed by means of the following equations (Williams and Berndt 1972):

$$s_i = [H(L_j + L_{j+1}) / 2F_i]100 \quad (5)$$

where
s_i : slope gradient (%) of the area i between the contours j and $j+1$
H : altitude difference between the contours j and $j+1$
L_j, L_{j+1} : length of the contours j and $j+1$, respectively
F_i : size of the area i

$$s = (\sum_{i=1}^{n} s_i F_i) / F \quad (6)$$

where
s : mean slope gradient of a sub-basin
F : sub-basin area
n : number of the areas between contours in a sub-basin

For the application of Equations (5) and (6), a countours map was used (Figure 2).

RUSLE refines USLE by assigning new equations based on the ratio of rill to interrill erosion, and accommodates complex slopes.

3.4 Estimate of the soil cover factor

The soil cover factor (C) was computed for each sub-basin separately on the basis of the soil cover percentage of the sub-basin considered. The soil cover percentage for the whole basin has as follows: forest 15.96%, bush 19.46%, urban area 3.07%, no vegetation 19.0%, fruit-trees 32.24%, irrigated land 1.45%, meadow 2.98%.

On the basis of the literature (Wischmeier and Smith 1978; Schwertmann et al. 1990), the following C-values were assumed: $C_{forest} = 0.004$, $C_{bush} = 0.03$, $C_{urban} = 0.001$, $C_{no-veget.} = 0.20$, $C_{fruit-tree} = 0.09$, $C_{irrig.land} = 0.18$, $C_{meadow} = 0.10$.

The soil cover factor for a sub-basin ($C_{sub-basin}$) was determined from the following relationship:

$$C_{sub-basin} = C_{forest}F_{forest} + C_{bush}F_{bush} + C_{urban}F_{urban} + C_{no-veget.}F_{no-veget.} +$$

$$+ C_{fruit-tree} F_{fruit-tree} + C_{irrig.land} F_{irrig.land} + C_{meadow} F_{meadow} \quad (7)$$

where F_{forest}, F_{bush}, F_{urban}, $F_{no-veget.}$, $F_{fruit-tree}$, $F_{irrig.land}$ and F_{meadow} the percentage of forest, bush, urban area, no vegetation, fruit-trees, irrigated land and meadow, respectively. These percentages were calculated by means of a soil cover map of the basin considered.

RUSLE uses for the computation of C-factor the subfactors: prior land use, canopy cover, surface cover, surface roughness, and soil moisture.

3.5 Estimate of the erosion control practice factor

For the estimate of the erosion control practice factor (P), the most unfavourable case was taken into account that, namely, no erosion control practices were applied to the basin ($P = 1.0$).

According to RUSLE, P-factor values are based on hydrologic soil groups, slope, row grade, ridge height, and the 10-year single storm erosion index value, while P-factor values for USLE change according to slope ranges with some distinction for various ridge heights.

4. ARITHMETIC RESULTS FOR SOIL EROSION

Equation (1) was applied to each sub-basin separately on a mean annual time basis. So, a mean annual value of soil erosion was computed for each sub-basin, and this value was assigned to the center of gravity of the sub-basin. The coordinates of the 136 centers of gravity of the sub-basins and the corresponding values of the mean annual soil erosion were given as input data to the software "SURFER 8". By means of this software, the isoerodent curves map for the whole basin resulted (Figure 3).

The mean annual soil loss in the sub-basins varies from 0.50 to 18 t/ha or from 50 to 1800 t/km^2. The mean annual soil loss in the whole basin (2853 km^2) amounts to approximately one million tons or 3.78 t/ha or 378 t/km^2. The latter mean annual value is equivalent to the mean annual rate of 0.13 mm for the whole basin.

5. CONCLUSIONS

The computed mean annual rate of soil erosion (0.13 mm) for the basin considered in this study (Portugal – Spain border) is quite lower than the corresponding measured value (0.50 mm to 0.70 mm) for basins located in Cyprus (Water Development Department, Nicosia, Cyprus). This fact is due, among others, to the relatively lower mean annual rainfall height and to the relatively lower mean slope gradient of the soil surface in the basin considered in the present study.

The computation of soil erosion in the sub-basins of the large basin considered was enabled on the basis of the classical USLE because the application of the newer RUSLE to a sub-basin of about 25 km^2 requires the availability of numerous and detailed soil data.

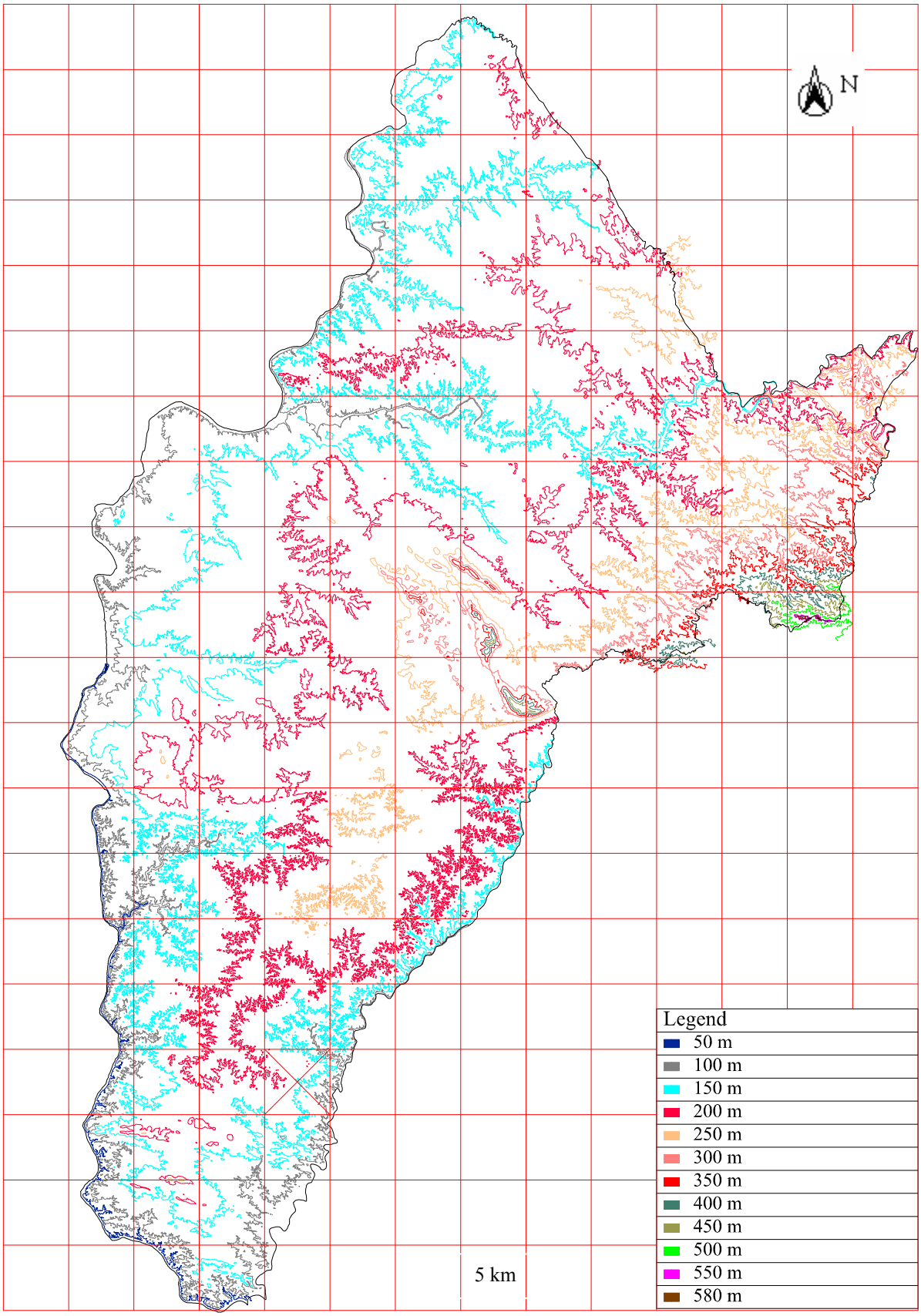


Figure 2. Contours map

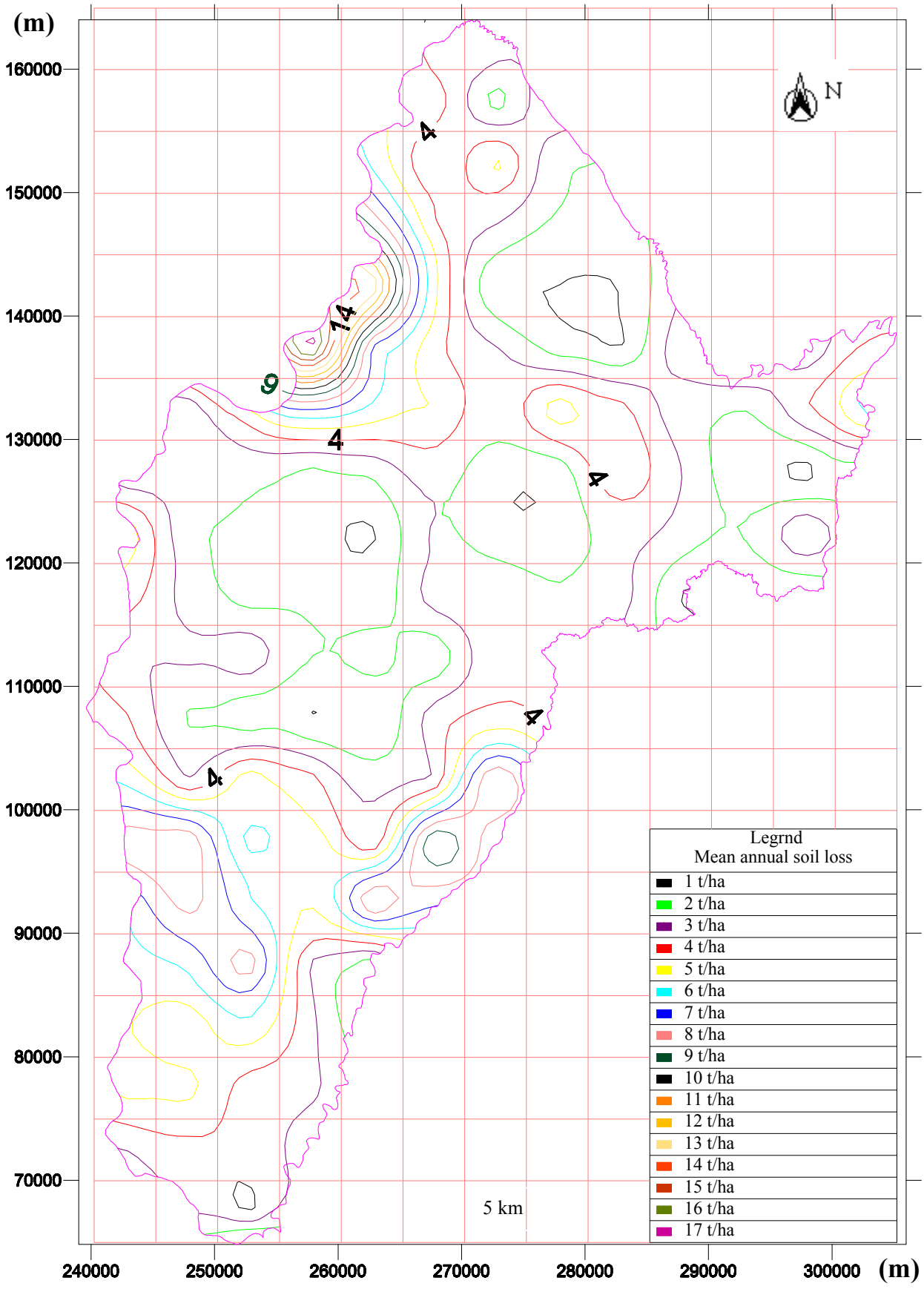


Figure 3. Isoerodent curves map

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SIMULATION OF WATER SUPPLY AND DEMAND IN THE JORDAN VALLEY REGION USING WEAP21: INITIAL RESULTS

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Abstract: Water in the Hashemite Kingdom of Jordan is extremely scarce, yet, key in the country's economic development and peace negotiations with neighbouring states. In the Jordan Valley, one of the most productive agricultural areas in the Middle East, water comes primarily from fresh (surface and ground water) and treated wastewater (brackish and blended water) sources. Due to a fast growing population and expanding agricultural sector additional demands for new water resources are expected to increase soon. Treated wastewater could be employed as a suitable alternative for agricultural irrigation, thereby relieving pressure for fresh water resources that satisfies the demand in urban areas. The SMART project (Sustainable Management of Available Water Resources with Innovative Technologies), jointly financed by the German government with the Ministry of Water and Irrigation in Jordan (MWI), recognises that water markets are increasingly relying on instruments that reallocate the available water between competing users in an efficient and equitable way so as to minimise the socio-economic impact of water scarcity. As treated waste water could play a prominent role in these future water markets there is a clear need for continued development of a policy tool that ensures that this reallocation process includes treated waste water sources. This also motivates the present study that presents the initial calibration of the Water Evaluation And Planning (WEAP) model that will be used to evaluate alternative water supply options in the Jordan Valley. WEAP is particularly suitable for the intended purpose as it easily accommodates extensive primary and secondary spatial data sets for our empirical analysis and allows the simulation of various water supply and demand scenarios, specifically for agricultural production. This paper reports on an important first step towards evaluating the use of treated wastewater in complementing existing fresh water resources: the implementation and calibration of dam operating rules within the WEAP21 application. It is shown that with simple model operating rules it is possible to reproduce historical dam volumes sufficiently accurate to be used for scenario evaluation with some confidence.

Key words: Integrated water resource management – Jordan Valley – simulation of water resources – WEAP21 – SMART

INTRODUCTION

The Hashemite Kingdom of Jordan has extremely scarce water resources. As shown in Table 1 Jordan faced in 2006 a deficit of nearly 600 million m^3 of water, amounting to 39% of the total demand. Water plays an important role in the country's economic development, which makes it of crucial strategic importance. Water, therefore, features prominently in peace negotiations with neighbouring states.

The fertile Jordan Valley, in particular, is an extensive water user as one of the most productive agricultural areas in the Middle East, and the agricultural sector can be expected to be most strongly affected by water scarcity. At present, 63% of Jordan's water resources are used for irrigation. Treated wastewater is thereby an important additional sources as it constitutes 25% of the surface water, about 90 MCM, that is used to meet irrigation demand,. In the future the demand for new unconventional water resources can be expected to rise considerably to mitigate the impact of water scarcity on the socio-economic well being of Jordan.

Table 4: Water Supply and demand requirements by sector for Jordan in 2006, (JVA 2007)

Demand Requirements	Water Demand MCM
Domestic	290
Industry & Remote Areas	46
Agriculture	589
Total Demand	1512
Water Supply	925
Deficit	-587

Despite the clear need for unconventional water supplies, the government does not employ appropriate tools to evaluate the ramifications of wastewater development in relation to the prevailing cropping patterns and rainfall regimes in the Jordan Valley (JV). Therefore, a methodology is required that explicitly evaluates the use of treated wastewater resources as a potentially viable source of water that is available for crop production in the JV. To address this concern, this study makes a first attempt to simulate water supply and demand in the Jordan Valley Region. The model described in this paper evaluates the use of treated wastewater as a source for agricultural production.

It is necessary to develop a better understanding how the water supply and distribution system behaves in the JV. Indeed, there is considerable scope for policy and investment that will affect the future water availability for agricultural, industrial, and domestic use. However, there are no systematic studies of the future implications concerning changes in demand and supply that take into account the spatial dimensions of water resources and their uses. Yet, an understanding of the spatial features of the water supply system in the JV is essential for evaluating the impact of changing water demands in different parts of the JV, changes in distribution rules, shifting agricultural production patterns, and the introduction of demand-side initiatives. This also motivates the present study that presents the initial steps in the development of a model of water supply and demand that can help decision makers form their plan for water allocation, through exploring the effect of different assumptions on water allocation and availability, in a spatially explicit manner.

For our study we selected the Water Evaluation and Planning (WEAP21) software (Yates, Sieber et al. 2005). WEAP is particularly suitable for the intended research objective as it incorporates a demand priority and supply preference approach to describe water resource operating rules that function as system demands to drive the allocation of water from surface and groundwater supplies to the demand centres (Yates, Sieber et al. 2005). WEAP can be integrated with groundwater models and water quality data and is easily extendable to other subcatchments and larger areas. Furthermore, WEAP's data structure maps the information in its appropriate spatial and temporal dimensions, the development of its structural equations is amenable to a statistical evaluation while its visual mode provides an attractive interface for decision making processes of policy makers and stakeholders. Concerning output WEAP can simulate various water management scenarios to evaluate the impact on water availability and water quality for different client groups in a spatially explicit manner..

The Jordan River has been well studied providing a rich source of primary and secondary data sets for the analysis described in this paper. Using these data sets, WEAP reproduces geographically-specific agricultural production along the north-south flow of the Jordan River. Furthermore, WEAP21 allows the user to develop supply and demand scenarios, allocating water to different demands based on user-supplied demand and supply priority weights. The design and calibration of the WEAP21 model is presented in this paper. In subsequent work it will be used for scenario analysis to evaluate different water allocation regimes and supply options for their effectiveness in meeting agricultural demand.

The focus of this paper is on calibrating dam volumes against data recorded by the Jordan Valley Authority. The reason for studying dam levels is the crucial role of dams in regulating the supply of water in the JV. The high rainfall variability in the JV is smoothed over by storing the water in

dams, and decisions on water allocation are based on dam levels at the end of the wet season. Attribution of dam levels—that is, the recorded level of stored water—and flows therefore contributes to the larger objective of this study, namely, analyzing the use of non-conventional water sources for agricultural irrigation. Additionally, it will be necessary to model the current water allocation decision-making process. As these are currently based on dam levels, the attribution of dam flows also contributes to this goal. Modelling water allocation rules will allow for alternative rules to be simulated, which can take re-used wastewater and other non-conventional water sources into account.

STUDY AREA

The Jordan Valley Authority (JVA) has jurisdiction over the 380-kilometer rift valley that runs from the Yarmuk River in the north to Al Aqabah in the south. The northern part, from the Yarmuk River to the Dead Sea, is known as the Jordan Valley (JV). It is divided into eastern and western parts by the Jordan River. Bordered by a steep escarpment on both the eastern and the western side, the valley reaches a maximum width of twenty-two kilometres at some points (JVA 2008).

The rift valley on the southern side of the Dead Sea is known as the Southern Ghor[1] and the Wadi[2] al Arabah. The Southern Ghor runs from Wadi al Hammah, on the south side of the Dead Sea, to Ghor Fifa, about twenty-five kilometres south of the Dead Sea. Wadi al Arabah is 180 kilometres long, from the southern shore of the Dead Sea to Al Aqabah in the south (Metz Dec,1989).

The Jordan Valley Development Law No. 19 of 1988 (amended in 2001) identifies the area of JVA responsibility. It extends from the Yarmuk River in the north to the Red Sea in the south. The eastern extension of the area is limited by the contour line at 300 m above mean sea level (a.m.s.l.) north of the Dead Sea and the contour line at 500 m a.m.s.l. south of the Dead Sea. The JVA service area comprises JV North, JV South, Southern Ghors and Wadi al Arabah. The total geographical area of JVA's responsibilities is about 4800 Km^2 (Figure 5), distributed through eight governorates: Irbid (494.6 Km^2), Ajloun (85.8 Km^2), Jerash (32.8 Km^2), Salt (669.8 Km^2), Madaba (403 Km^2), Karak (851.1 Km^2), Tafeeleh (654.2 Km^2), and Aqaba (1566.2 Km^2).

The variations in temperature,

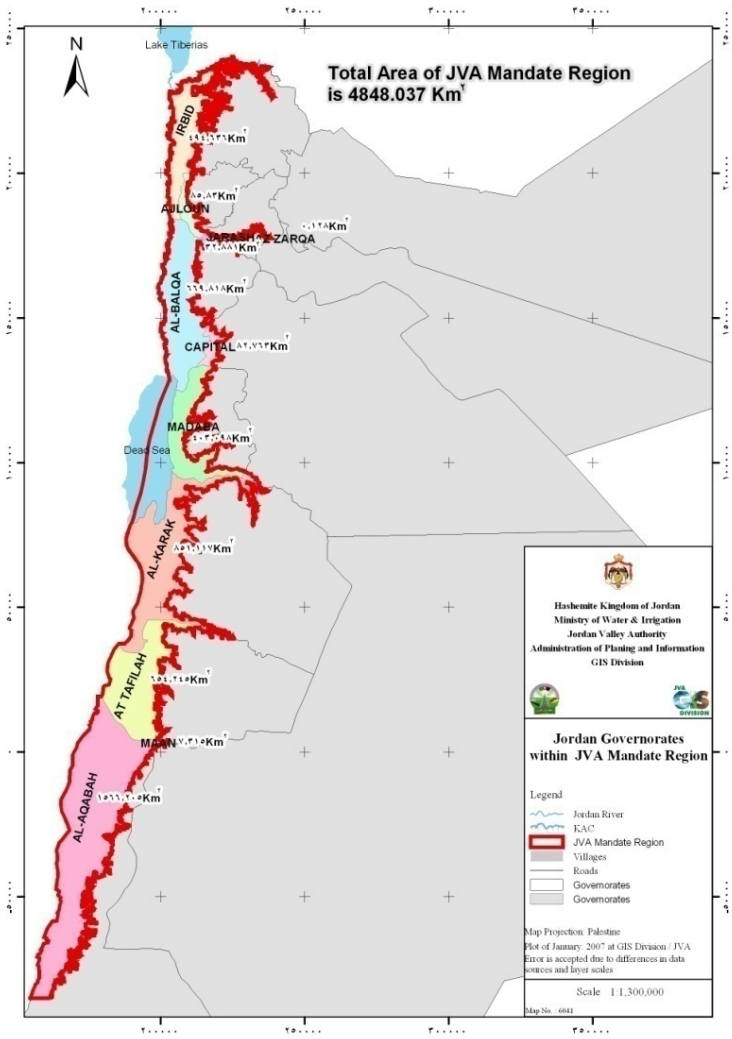


Figure 5: The Jordan Valley boundaries by Jordan law

[1] The northern part of the valley is known as the Ghor, and it includes the Jordan River. Several degrees warmer than adjacent areas, its year-round agricultural climate, fertile soils and water supply have made the Ghor a key agricultural area.

[2] **Wadi** (Arabic: وادي) is a valley. The term "wadi" refers to a dry riverbed that contains water only during times of heavy rain.

humidity, and rainfall across the Jordan Valley produce distinct agro-climatic zones. The rainy season starts in October and ends in May. Precipitation reaches 350- 400 mm/year in the north and drops down to 50 mm/year in the south. The warm winter of the valley allows the production of off-season crops and can be considered as a large natural green house. The annual available water in the valley is approximately 250-300 MCM, while the annual demand for irrigation exceeds 500 MCM. Approximately 60 MCM of water is pumped to the city of Amman and 20 MCM to Irbid for domestic uses (THKJ 2004).

The part of the Jordan Valley considered in this study is the area between the Yarmuk River and north of the Dead Sea. It has been divided into three main sub-parts, corresponding to the divisions used by JVA and the Ministries of Statistics and Agriculture (Figure 2). The three main sub-regions are identified by the following terms: Northern JV, Middle JV and Southern JV. Each of these regions has its own distinct climatic and agro-ecological characteristics, constituting a base for dividing the agricultural land. The total land area and irrigated area of the three zones is summarized in Table 2.

Table 5: Geographical & irrigated areas in the JRV, (DOS 2008)

Zone	Total Area Dunum*	Irrigable area Dunum*	% Irrigable
North JV	97,7	82,8	84,7
Middle JV	127,4	91,1	71,5
South JV	124,0	114,3	92,2
Total	349,1	288,2	82,6

*1 Dunum = 1,000 m^2 = 0.1 ha.

CURRENT WATER SUPPLY AND DEMAND

To provide a background for the analysis of the simulation of the water distribution system in the JV, this section describes the current water demand and supply

Water Supply

Flow from the Yarmuk River, upstream of the confluence with the Jordan River at the northern end of the valley, is fed into a concrete canal called King Abdallah Canal (KAC). Constructed initially as the East Ghor Canal by Jordan in 1950, the KAC runs parallel to the river on the eastern bank for 69 Km. The Jordan Valley irrigation scheme introduces the distribution of connection points from and to King Abdallah Canal (KAC), which is the main carrier for the valley. The canal receives water from different tributaries – in particular, all flows from side wadis are rechanneled to feed the canal – that is subsequently distributed to farms, as shown schematically in (*Figure 6*).

In 2006 the Yarmuk River supplied the KAC with 55 MCM /year while a further 55 MCM/year was provided by the Tiberias carrier according to the peace treaty on October 26, 1994 (Treaty 1994). In addition to these surface flows, 25 MCM/year are pumped from the Mukhyba wells to the KAC. Additional inflows come from several wadis that cut through the mountain ranges bordering the valley providing another 6 MCM/year (JVA 2007). The North Jordan Valley until the conveyance to Der Alla receives fresh water from KAC for agricultural purpose, while the Middle Jordan Valley receives blended water (treated wastewater mixed with fresh water) mainly from the King Talal Dam (KTD) via KAC, Zarqa River and Zarqa Carrier (ZC1 & ZC2). In the North and Middle JV the agriculture water requirement is 110 MCM/year each.

The major water source for farmers in the Jordan Valley is supplied via the JVA stage offices—offices that interact with farmers. Stage offices receive and process daily water requests, manage and regulate the supplies to farms, process billing and accounting, and register the cropping areas for a group of development areas. There are ten stage offices in the Jordan Valley from the north to the Dead Sea, and two stage offices in the Southern Ghors.

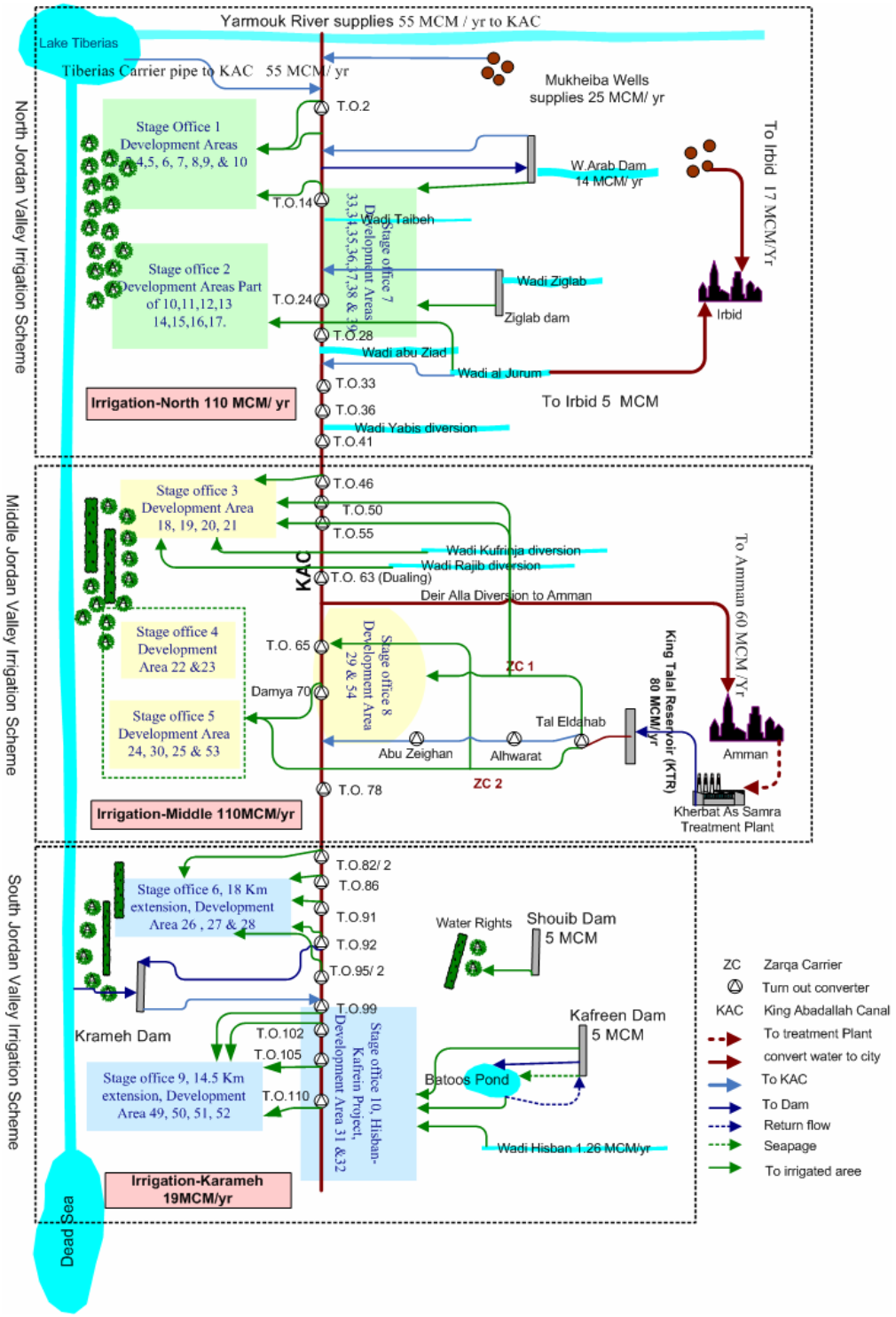


Figure 6: Jordan Valley Water allocation 2007, developed by researcher Amani Alfarra for the purpose of this research

The JVA develops an annual plan at the beginning of every irrigation water supply season. To determine the availability of water resources and to estimate the upcoming supplies for the season, JVA calculates the available resources in the reservoirs at the end of the wet period (i.e. end of March). To develop the Annual Water Plan the JVA predicts the resources, then estimates the expected demands for water, and finally computes the minimum target levels in the reservoirs. JVA considers April as the start of the irrigation water supply period. The JVA indicates two seasons of supply and demand. The summer season is considered to start at the beginning of April to September 30th, while the winter covers the period from the beginning of October to December 31st, and the period from January 1st to the end of March.

Water Demand

Monthly water accounts have been created for the years 1990-2006, using proprietary data from the Dams Control Department of the Jordanian Ministry of Water and Irrigation (MWI). The annual crop areas and water requirements for 1990 are shown in Tables 3 and 4. Water requirements are calculated by the MWI based on the records kept at the JVA stage offices.

Table 6: Agricultural Area in 1990, area in dunum (MoA 1990).

Zone	Veg. GH	Summer Veg.	Winter Veg.	Winter Seeds	Citrus Trees	Banana	Palm Trees	Other Trees	Total
North JV	125	15994	10539	19254	37261	461	1	2289	85924
Middle JV	4122	26149	14669	6498	9289	552	6	1210	62495
South JV	666	10538	27164	1755	2713	750	96	1978	45660

*1 Dunum = 1,000 m^2 = 0.1 ha.

In Jordan, agriculture consumes around 600 MCM of water per year. One-third of this amount (200 MCM) is consumed by the Jordan Valley. Almost 50% of this 200 MCM is reclaimed water. All in all, agriculture consumes less than 20 % of the freshwater resources available from the JV.
In 2006, the total municipal water consumption was around 290 MCM (Table 5). Of this, almost 42.6% was pumped into Amman Governorate while Ajloun received the smallest allocation, around 1.27 %. Out of this total, only around 111 MCM was treated in wastewater treatment plants, as only 61% of households have wastewater connections this means that around 40% of all household dwellings are not connected yet to the sewer network system. In other words, there is considerable amount of the influent lost without recycling or reuse at all since many households depend on cesspools. Aside from the lost opportunity to reuse the wastewater, the cesspools are a likely source of groundwater contamination.

Table 7: Annual crop water requirements (JVA 2006) in MCM

Zone	Veg. GH	Summer Veg.	Winter Veg.	Winter Seeds	Citrus Trees	Banana	Palm Trees	Other Trees	Total
North JV	360	444	314	622	1177	1752	688	1177	6534
Middle JV	359	447	327	626	1187	1790	688	1187	6611
South JV	439	454	344	676	1243	1854	688	1243	6940

Within Amman city, the population according to Department of Statistic (DOS) in Jordan was 1.6 million in 1994, and 1.9 million in 2004, corresponding to an average annual growth rate of 2.0%. The population growth prior to 1994 (between 1979 and 1994) was 4.4 % per year, while since 2004 it has been growing at 3.7% per year. An official estimate of the annual water demand is 51 m^3 per person per year. However, using this figure reveals significant discrepancies between estimated demand and supply, suggesting that it does not capture all of the water supplied. Within the WEAP21 simulation it is assumed that 15% of the delivered water is not captured in this figure. Accordingly, within WEAP21 an annual rate of 60 m^3 per year is assumed.

REPRESENTATION IN WEAP21

WEAP21, the Water Evaluation and Planning software is designed to be an effective tool for integrated water resource management (IWRM). The design goals were that it be useful to planners, easy to use, affordable, and readily available to the broad water resource community. WEAP21 is designed around a water accounting and allocation framework that balances demand and installed infrastructure. It also allows for hydrologic processes to be incorporated in models, using a lumped-parameter hydrologic model. As a planning tool, WEAP21 supports scenario analysis as part of its

core features. Examples of possible scenario variations include alternate water supply and demand options, climate scenarios, and changing land use. WEAP21's strength is addressing water planning and resource allocation problems and issues.(Yates, Sieber et al. 2005). WEAP21 has been extended so that it is relatively easy to link MODFLOW groundwater models and QUAL2K water quality models to a WEAP21 model. As discussed below, most of the calculations in WEAP21 are carried out automatically within a water allocation framework. In addition, WEAP21 offers spreadsheet-like capabilities for implementing algorithms. Finally, WEAP21 models are extensible in other ways as well (e.g., by linking to dynamic link libraries, or DLLs), and with more or less effort can be combined with other models.

WEAP21 is an appropriate tool for the present study for several reasons. First, it is available at no charge for institutions in developing countries and for an affordable price for developed countries and companies. Second, the scenario features of WEAP21 support the exploration of how non-traditional water sources can change water availability and use in the JV. Finally, because WEAP21 models are easily extendible, the model that is built within this research project can be used as the basis for a larger model that includes the whole of Jordan. Finally, the MWI of Jordan can later integrate groundwater models and water quality into the WEAP21 model.

At the same time, there are limitations to WEAP21 that should be kept in mind. First, WEAP21 represents spatial relations through the length of river reaches. The built-in hydrologic model is a lumped-parameter model that does not represent spatial variation across a catchment.

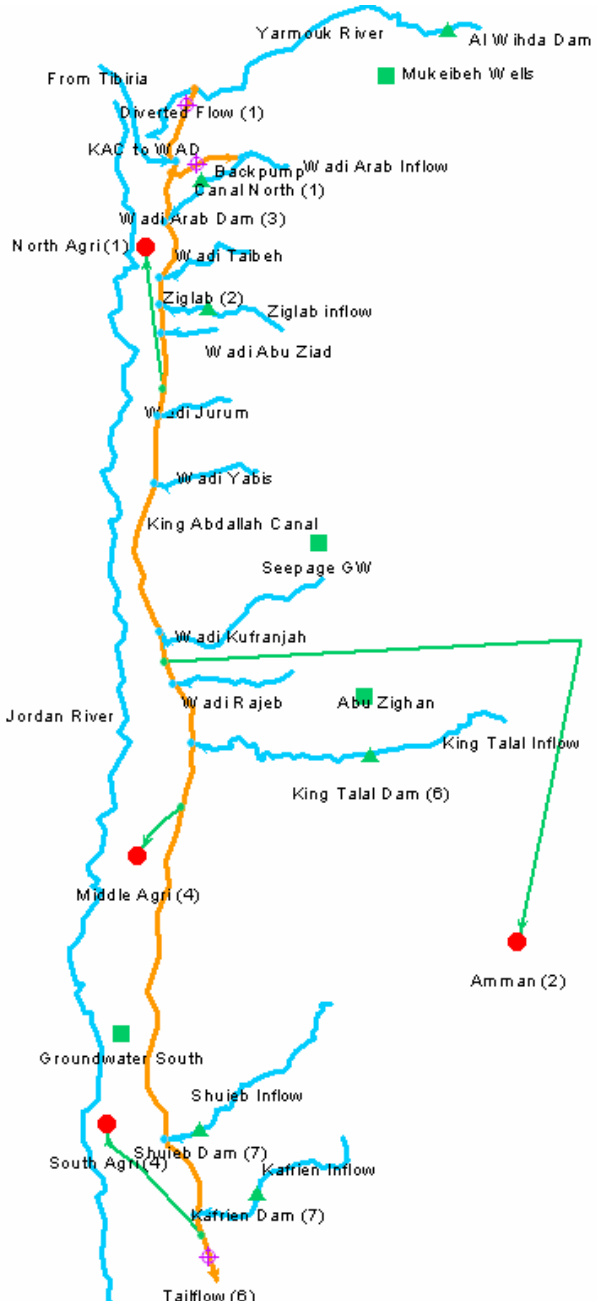


Figure 7:Study area represented in WEAP

Second, some aspects of the water distribution system in the JV were challenging to represent. Specifically, there is a two-way flow between the King Abdallah Canal and the Wadi Arab Dam. There are no built-in rules within WEAP21 for representing such a two-way flow, and so the calculation was implemented using WEAP21's modelling capabilities.

Data within the MWI are located in different department, for enhancing the model lots of cooperation between departments is required to include groundwater and water quality.

The major components of the water delivery system shown in Figure 2 have been represented in the WEAP water allocation and planning software. The elements of the model system are shown in Figure 3. In the application described in this paper, demands and supplies are represented on a monthly basis for the years 1990-2006 for purposes of calibration. The calibrated model will be used in the future to evaluate scenarios of alternative water supply.

In designing the schematic representation of the study area in WEAP21, the objective was to include as much detail as was needed to properly characterize both demand and supply sources, subject to the availability of field data. The representations consist of the following main elements:

Distribution Systems: A distribution system represents water users in a common geographic area with shared water sources. In the current representation, distribution systems are identified either with irrigation systems or municipal demands (Amman city) – the same categories used by the MWI for allocating water in the Jordan Valley. The water demand in each distribution system for Amman city is aggregate, while irrigation demand is partitioned by crop type, cultivated area and crop demand. Within WEAP21, distribution systems are represented by demand sites.
Municipal water demands are estimated as described in the previous section. Irrigation demands are estimated by multiplying the area under different crops by an assumed irrigation rate.
King Abdallah Canal (KAC) and Tributaries: These are the primary water conduits in the region. Stream flows are estimated along every tributary on a monthly basis. There are 13 wadis and tributaries flowing to the KAC.
Water flows both from the Wadi Arab to the KAC and from the KAC to Wadi Arab. The KAC-to-Wadi Arab backpump is represented in WEAP21 as a diversion with a minimum flow requirement that is set to the historical flow.
Dams: Five dams are represented within WEAP21 (Al Wihda dam, which is shown in Figure 3, is not currently operating, and is included only for scenario development). From north to south, the active dams are Wadi Arab, Ziglab (also called Sharhabiel Dam), King Talal, Shuieb, and Kafrien. Account is taken of inflows, outflows, releases, evaporative losses, and groundwater interactions.
The gross storage capacity of the dams is shown in Figure 5. As can be seen in the figure, the storage capacity of Kafrien dam was increased during the 1990-2006 period. In WEAP21, this was represented by a step increase between 1995 and 1996. The most important dams by volume are King Talal (86 MCM gross storage, 75 MCM live storage) and Wadi Arab (20 MCM gross storage, 17 MCM live storage).

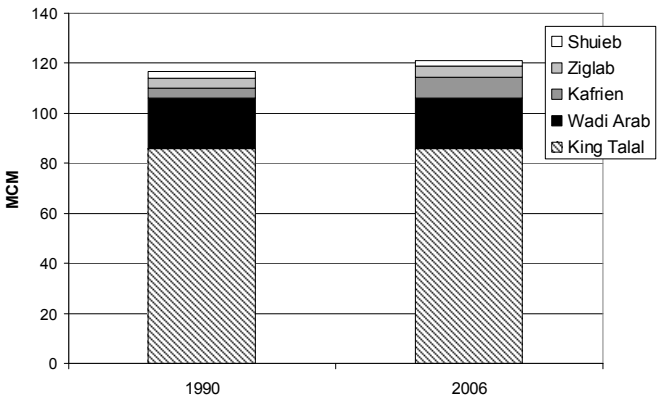


Figure 8: Gross storage capacity of JV dams (MCM)

SIMULATION AND CALIBRATION

The main focus of the work described in this paper is to represent dam operating rules in the JV. Rather than making an attempt to capture the decision processes carried out by the MWI, which are somewhat ad hoc, some simple rules were introduced that to a large extent captured the apparent water allocation.
Note that two allocation decisions are taken by the MWI:

- How much potential irrigation and municipal water demand will actually be supplied,
- How much water will be released from each dam to meet the required demand.

The focus of this paper is on the second decision, while the first decision is left to future work. For this reason, the water actually supplied to each distribution system (the "coverage" for the system) was set to its historical value, and then the dam operating rules were simulated to meet that supply. The question of how to effectively simulate coverages will be taken up in future work.

WEAP21 provides a constrained distribution of the total available water. Water allocation within a time step is carried out by using user-specified priorities for different demand sites and sources. At each time step, the coverage of highest-priority demands is set to as high a value as possible given constraints on water availability and other constraints specified in the model. Then those coverages are frozen, and the coverages for the next highest-priority demands are set. This process is repeated until all coverages are calculated, consistent with the demands and available volumes of water. Water is then supplied to each demand site, with the volume supplied being equal to the coverage multiplied by the demand.

The WEAP21 algorithm is implemented as a series of linear programming (LP) problems, iterated over demand and supply priorities. The algorithm can be written in the following way. Suppose that there are N demand sites and M sources. Denote the demand at demand site i, with priority p, by $D_i^{(p)}$. The amount of water actually supplied to the demand site from source r is $x_{i,r}^{(p)}$, while the total amount of water available from source r is $S_{r,t}$. (Sources are given a time label, t, because they can represent storage as well as transient flows. For other variables, the time label is suppressed.) Note that a source can also have a demand, for example, a reservoir accepts inflows and has targets for storage. Then, starting with priority $p = 1$, and looping over supply preferences to the demand sites at that priority, the following linear program is solved:

Maximize $C^{(p)}$, the coverage at priority p, subject to

$$S_{r,t} - \left(\sum_{i=1}^{N} x_{i,r}^{(p)} - \sum_{s=1}^{M} x_{r,s}^{(p)} \right) = S_{r,t-1} \qquad \text{Mass balance constraint for storage}$$

and

$$\sum_{r=1}^{M} x_{i,r}^{(p)} = c_i^{(p)} D_i^{(p)} \qquad \text{Coverage constraint for demands}$$

where either

$$c_i^{(p)} = C^{(p)} \qquad \text{Equity constraint for demand sites}$$

or

$$c_i^{(p)} \geq C^{(p)} \qquad \text{Equity constraint for reservoirs and instream flow}$$

Additionally,

$0 \leq C^{(p)} \leq 1$ Bound on coverage

$x_{i,r}^{(p)} \geq 0$ for priority p and supply r at specified supply priority

$x_{i,r}^{(>p)} = 0$ for lower priorities (that is, with values greater than p)

also, $x_{i,r}^{(p)} = 0$ if the supply priority is higher than the one currently being evaluated

The LP is solved, and the shadow prices for each equity constraint are evaluated. If the shadow prices are positive, then the $x_{i,r}^{(p)}$ are set to their optimal values. The routine is then repeated for the next lowest supply priority for the demands at priority p. The routine is then repeated at p+1, until all demand priorities have been accounted for.

After observing the patterns of dam releases and volumes over time, the following priorities were specified within the JV WEAP21 application, where a priority of 1 is the highest priority:

Priority 1: KAC headflow, Wadi Arab backpump, North Agriculture
Priority 2: Ziglab dam, Amman city
Priority 3: Wadi Arab dam
Priority 4: Middle Agriculture, South Agriculture
Priority 5: King Talal dam, KAC tailflow
Priority 6: Shuieb dam, Kafrien dam

The flow in the King Abdallah Canal as it exits in the study area (the tailflow), is modeled as an instream flow requirement that is tied to the volume of water within the King Talal dam. It is given the same priority as the filling priority for King Talal dam. The flow requirement is set in the

following way: when live storage in the dam is less than 25% of the capacity, the tailflow requirement is set to zero. When live storage in the dam is 100% of capacity, the tailflow requirement is set to 1.5 m^3/second. Between those two limits, the tailflow requirement increases linearly with the volume in the dam.

In addition to the priorities listed above, the Wadi Arab, Ziglab, Shuieb, and Kafrein dams have a "buffer" that slows down releases as the dam gets more empty. The rate of release from the buffer zone is set by a buffer coefficient. The levels of the buffers and the coefficients were used as calibration parameters. The calibration parameters were constrained to lie between minimum and maximum values, as shown in Table 8. Otherwise, WEAP21 imposes constraints that reflect water availability.

Table 8: Calibration parameters

Parameter	Minimum	Maximum	Initial Value	Run 1 Calib	Run 2 Calib
Top of Buffer (million m^3)					
Wadi Arab	3.1	20.0	9.1	16.2	16.4
Ziglab	0.4	4.3	2.4	3.2	3.2
Shuieb	1.43	2.30	2.12	2.30	1.93
Top of Buffer as fraction of storage capacity (dimensionless)					
Kafrein	0.00	1.00	0.25	1.00	0.58
Buffer Coefficient (dimensionless)					
Wadi Arab	0.10	1.00	0.20	0.10	0.13
Ziglab	0.10	1.00	0.75	0.10	0.10
Kafrein	0.10	1.00	1.00	0.67	0.10
Shuieb	0.10	1.00	0.50	0.10	0.10

Calibration was carried out in a two-step process using the PEST parameter estimation software version 1.1(Watermark Numerical Computing 2004). In both cases, calculated dam volumes at monthly time step were compared to historical data. PEST uses a Levenberg-Marquardt method to numerically estimate optimal parameter values, by minimizing the sum of squared difference between model outputs and observations.

In the first calibration run, observed dam levels for all five dams were compared to their modeled values. In the second run, the calibrated values from the first run were set as the initial values, and observed dam levels for all dams except for King Talal dam were compared to their modeled values. The reason for this two-step process is that the volume in King Talal dam is sufficiently large that, when it is included, it dominates the total volume. By carrying out the second calibration run, a better fit was obtained for the smaller dams. The results are shown in Table 8.

The historical and estimated dam volumes for the three largest dams (King Talal, Wadi Arab, and Kafrien) are shown in Figures 5, 6, and 7. As can be seen on the figures, the relatively simple simulation operating rules and priorities reproduce the historical dam levels quite well.

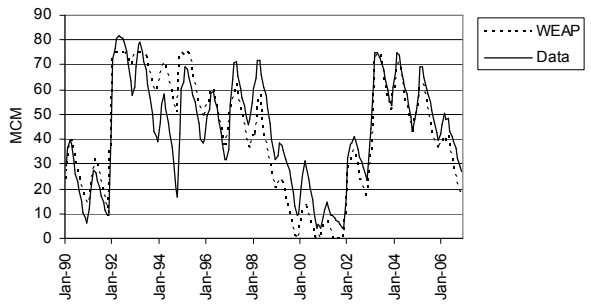


Figure 9: King Talal Storage, historical data and WEAP21 estimate

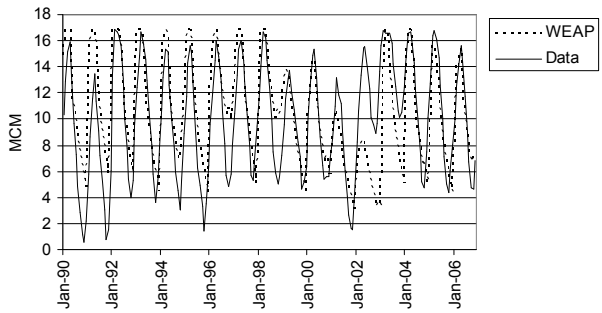


Figure 10: Wadi Arab Dam storage, historical data and WEAP21 estimate

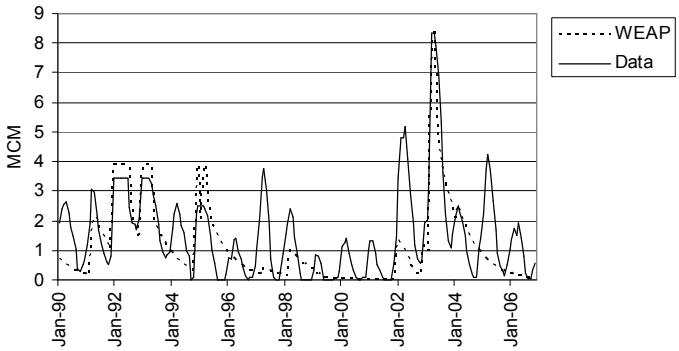


Figure 11: Kafrien Dam storage, historical data and WEAP21 estimate

DISCUSSION AND CONCLUSION

The reasonable agreement between the observed and modeled dam storage shown in Figures 5-7 suggest that the WEAP21 model can be used to create scenarios of alternative water sources... Before the scenarios can be developed, it is also necessary to simulate coverages for the distribution networks. This work is underway. The full calibrated model will then be applied to MWI data for 2007 to evaluate the effectiveness of the model when applied to a year that was not used for calibration.

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ASSESSMENT OF ARTIFICIAL RECHARGE TEST IN JEFTLIK – FARIA AREA, WEST BANK

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Abstract: An artificial recharge test site was selected in the Jeftlik area of the foot prints of the Faria basin, in the Upper Jordan Rift Valley. The artificial pond infiltration type of 26 meters diameter and 2.85 meters depth was used it is a hyperbolic form of 500 cubic meters volume. The integration parameters of hydrogeological settings, hydrochemical characteristics of allocated water bodies, geophyisal investigations and the potentiality of the aquifer system in the area were used for the test area selection. The test lies within the shallow Pleistocene aquifer of 30 – 50 meters saturated thickness. One groundwater well of 19 m distance and two groundwater wells of 1 km far distance were used for the system monitoring. The filled water in the test system comes from the wadi water floods brought by 1.5 km water open canal. The recharge rate within the Pleistocene aquifer should be determined and compared to the calculated transmissivity value that was extracted from the conducted pumping test campaign of pre test investigations. This experiment is planned to be completed in three phases water quality type fillings one of the first flushing water floods from the adjacent wadi, second of the water flood in the middle of the winter season, third with pumped water from other wells far away from the experiment.

Key words: Artificial recharge, Jeftlik area, Faria Area, Jordan Rift Valley

1. INTRODUCTION

Water resources in the Jordan Rift valley are scarce in nature. The increasing population growth in the Jordan Rift Valley has put more pressure on the existing limited resources in the region. The steep gradient of the Jordan Valley produces a shadow effect, which reduces the quantity of the rainfall in the Jordan Rift valley (Arad and Michaeli, 1967). This basin is of great tectonic complexity and the major movement is eastwards with a southerly component near the river Jordan (Guttman, 1995). The aquifer system in the study area includes the sub-aquifers of Quaternary and Neogene formations (Ghanem, 1999). This covers the formations of Jenin sub series (Eocene), Beida & Lisan (Neogene & Pleistocene) as well as alluvium. The Pleistocene sub-aquifer consists of unconsolidated sand, gravel, cobbles, and boulders of different sizes separated by impermeable layers of saline lacustrine marl deposits (Rofe and Raffety, 1965). These deposits are composed of limestone, dolomite, chert, gravel with sand and clay filling the intersect ices and forming alluvial fans. The groundwater occurring in the alluvial fans differs quantitatively and qualitatively according to its location within the fan. Fresh water occurs around the apex of the fan, whereas saline water occurs at the fringes. Very steep deep faults in the Jordan Rift valley may cause deep circulation of the groundwater bringing it into contact with salty formations which appears as brackish springs near the river. The study area includes the footprints of the Faria basin in the Jeftlik area. The experiment consists of the construction of the infiltration pond, pumping test of the adjacent wells and hydrochemical characteristics of the adjacent wells.

2. THE CONSTRUCTION OF THE PILOT OF THE ARTIFICIAL RECHARGE TEST

The first phase of the experiment was carried out in the area in construction of the infiltration pond in the pilot area (Fig. 1). Table 1 illustrates the coordinates as well as the volume needed measurements of the infiltration pond. The volume of the pond is calculated according to the hyperbolic formulas according to the equation of the Fig.2 .



Fig.1:- The artificail recharge infiltration pond

Table 1: Coordinates and measurements of the Artificial recharge pond (in meters).

Day	Date	Time	H (in bond)	D (in bond)	H (in well)	r1	r2	Vw	ΔV	Δh
Monday	29/12/2008	9:00	102	13.9	27.96	3.65	6.95	92.8732		
Thursday	1/1/2009	13:40	75	11.9	27.96	3.65	5.95	55.2974	37.5758	0
Thursday	1/1/2009	18:00	72	11.87	27.96	3.65	5.935	52.9099	2.3875	0
Friday	2/1/2009	8:00	69	11.8	27.93	3.65	5.9	50.3139	2.59599	0.03
Friday	2/1/2009	12:00	67	11.78	27.93	3.65	5.89	48.7472	1.56665	0
Friday	2/1/2009	18:00	62	11.6	27.91	3.65	5.8	44.2135	4.53377	0.02
Saturday	*3/1/2009*	7:00	57	11.5	27.9	3.65	5.75	40.1944	4.01902	0.01

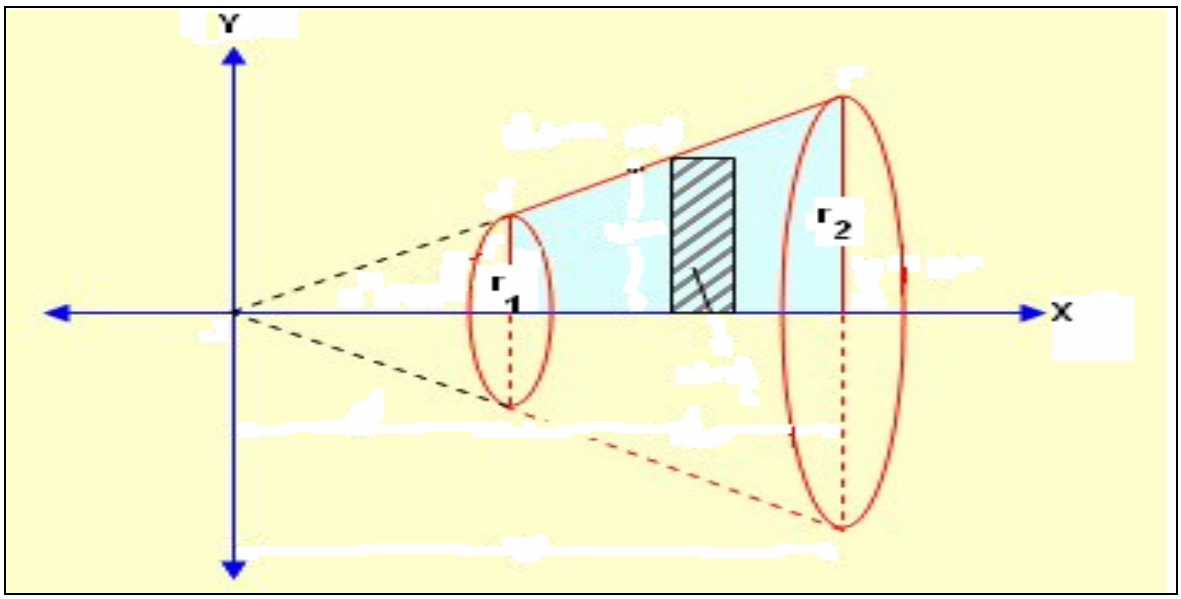


Fig. 2: The schematic shape of the artificial recharge pond and its equation

V= (1/3) π h (r12+r1r2+r22); where

V volume of the pond
h dynamic elevation of the water in the pond
r1, r2 lower and the upper radius of the pond

3. GROUNDWATER HYDROCHEMISTRY OF JEFTLIK – FARIA AREA, ARTIFICIAL RECHARGE TEST

Nineteen groundwater wells were investigated hydrochemically in the well field of the artificial recharge test in Jeftlik – Faria area (Fig. 3). The major ions of Ca^{+2}, Mg^{+2}, Na^{+}, K^{+}, Cl^{-}, NO_3^{-}, SO_4^{-2} as well as the trace elements of Fe, Zn, Li, Cr, Co, Ni, Mn, Al, Cd, Ba, Pb, Bi were analysed.

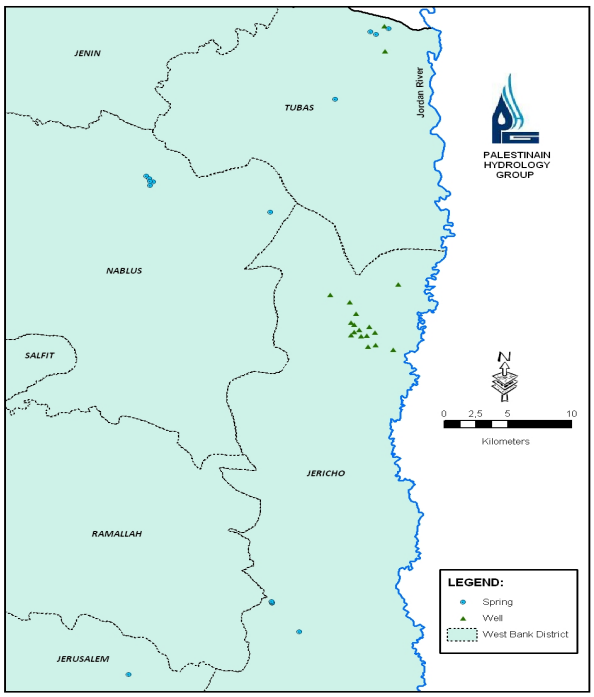


Figure 3: Sampled sites

Graphical and statistical methodologies were used to classify the water samples according to their main constituents. Water type of these wells are in the field of Na – Cl type. Piper diagrams were plotted for all the water samples analyzed in terms of major ions using AquaChem 5.1 software. The samples collected from the wells show that the majority falls into the earth alkaline water with increased portions of alkalis (prevailing sulfate and chloride) with the exception of one well (19-17/008) which is classified as alkaline water with sulfate-chloride characterized by its high TDS (Na^{+} and is the major cation and Cl^{-} is the major anion) and originates by leaching salt bearing rocks (Fig. 4 and Fig. 5).

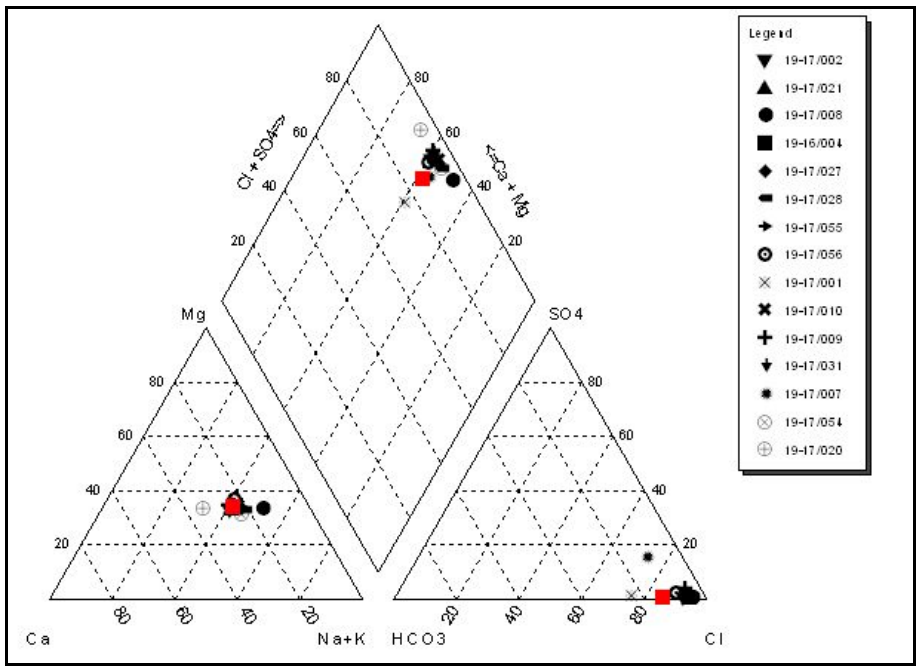


Figure 4: General Piper Diagram for the wells .

The well samples have the following ionic strength for the major ions: $Na^+ > Ca^{2+} > Mg^{2+} > K^+$ and $Cl^- > HCO_3^- > SO_4^{2-} > NO_3^-$. The analyzed parameters were also subjected to product linear correlation analysis. A correlation matrix is a symmetric square matrix whose entries are called correlation coefficients and main diagonal is all one unit.

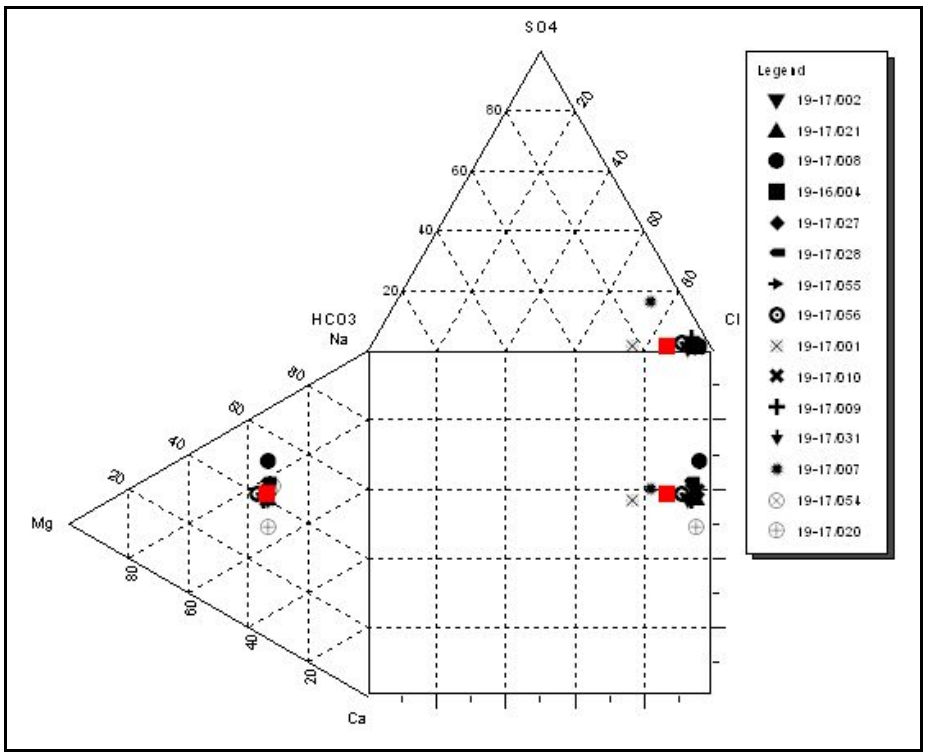


Figure 5: Durov diagram for the well samples

Correlation coefficients were calculated for all possible pairs of variables. The results of the saturation index show that all the groundwater samples were found to be undersaturated with respect to all the minerals. These relationships may be comparatively simple and uncomplicated as in the case of an aquifer that receives direct recharge by rainfall and from which water is discharged without contacting any other aquifer or other water. On the other hand, the situation may be

rendered complex by influences such as mixing of unlike water, interconnected aquifers of different composition, chemical reactions such as cation exchange and adsorption of dissolved ions and biological influences.

4. PUMPING TEST ANALYSES

To identify aquifer characteristics, such as Transmissivity (T) and Specific Storage (storativity), a pumping test was carried out in 9 selected Groundwater wells in Jordan Valley (Table 2). Pumping test were implemented for 9 groundwater wells in Al Fara'- Jeftlik area, in order to identify the characteristics of the aquifer; such as Transmissivity(T) and Storativity (S), to check the ability of water to be transmitted. Pumping test was implemented by using M-Scope, but the duration of each one was less than one day.

Table 2: The coordinates of the Pumping groundwater wells.

No	Name	Id.	X	Y	Z
	Wells				
1	Waheed Al Masri	19-17/002	196520	171240	-253
2	′Allan Al Damen & Partners	19-17/008	196250	170250	-251
3	Burhan Al Damen	19-16/008	196780	169670	-260
4	Fareed Abu Shamat	19-17/028	198150	170500	-268
5	′Inad Al Masri	19-17/001	196900	170740	-255
6	Husain Drai′i	19-17/010	197060	170150	-262
7	Rafeeq Qamhawi	19-17/009	197470	170230	-264
7	′Abed Al Lateef Haydar	19-17/031	197680	171060	-265
8	Fathalla Al Masri	19-17/007	196640	172290	-244
9	Mar′uf Abu Samrah	19-17/054	197600	169150	-273

The calculated transmissivity (T) of these wells are ranging between 86- 4763 m^2/day. This variability of T in the area is due to the existing of faults in the area, during the formation of the Jordan Rift Valley. The groundwater well 19-17/28 has a static head equal to 26.23m and dynamic head 26.5m. The discharge of groundwater well in m^3/ hr is ranging between 80-100 m^3/hr, the values of draw down were obtained from M-scope, and the pumping test was continued until the water level became constant. The T for the well 19-17/28 is calculated to be 86 m^2 per day.

5. CONCLUSION

An artificial recharge test site was selected in the Jeftlik area, in the Upper Jordan Rift Valley according to the hydrogeological data for the Pleistocene sub-aquifer and the hydrological characteristics of the ground surface. The hydrochemical data for the groundwater there was determined to integrate the qualitative behavior of water quality with its related quantitative. The artificial recharge pond of 500 cubic meters volume was used. The recharge rate within the Pleistocene aquifer was determined and compared to the calculated transmissivity value that was extracted from the conducted pumping test campaign of pre test investigations. This experiment is planned to be completed in three phases water quality type fillings one of the first flushing water floods from the adjacent wadi, second of the water flood in the middle of the winter season, third with pumped water from other wells far away from the experiment.

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PRECESSION WATER USE IN IRRIGATION OF TOMATO IN JERICHO JORDAN VALLEY / PALESTINE

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Abstract: The main objective of this study is to identify and investigate the precession uses of water in term of quantity for growing tomato in Jericho area. This objective was satisfied through optimizing the irrigation volume of water needed for growing the crop at different stages, saving water without decline the production, and increasing water productivity. This study was conducted over 2500 m^2 green house. Four control irrigation treatments (T1-T4), and one farmer treatment (T5) were carried out during the growing season. The result of this study shows that the required water for irrigation is lower than the applied one, where 12.4 to 33.6% of the water can be saved by referring to the T5 (farmer irrigation after PAPA technical manual), and this percent rise between 51.5% and 62.5% when by comparing with the average water used by normal farmers in Jericho district. This result can help to sustain the agricultural sector by using precession irrigation method with positive impact on the quality of shallow aquifer. The study also indicate an average irrigation water productivity of 15kg/CM. Using the study data for better irrigation management, the irrigated land could be increased from 14.2% to 33.7% % with reference to T5 (farmer treatment). However, while referring to the average irrigated volume by normal farmer, the irrigated area could be enlarged to 167.6 % .The treatment conditions in T2, which based on 90%ETc, and field efficiency 75% provides the best results, according to economic value, and then followed by T1, T4, T5, and T3. The provided conditions in T2 can save more than 100,000CM of irrigated water per one season where 500,000 square meters of tomato green houses related to T5 in Jericho area.

Key words: Precession irrigation, tomato cropping, water productivity.

1. INTRODUCTION

Jericho district is located in the eastern side of the West Bank with an area of approximately 353,300 dunums (one dunum is 1000 m^2). Agriculture is playing vital role in Palestine economy, and it is contributing between 11-33 % to (GNP). In Jericho district the agriculture is the main economical activity, mainly irrigation agriculture, with area about (45,194) dunums, where the vegetables is the dominant irrigated crop which occupy (33,807) dunums which forms (75%) of the total irrigated area in Jericho district, and production was about (79,354) ton (MOA 2006). The average rainfall for the period 1980-2006 was 151.84 mm, and the average evaporation rate in the same period was 2,085.3 mm (PCBS 2007). There are 20 irrigation wells (50 -180 m depth) in the Jericho district abstract water from the Plio-Plistocene shallow aquifer system with annual abstraction of less than 1 MCM/yr (MOA 2005). There are also three main springs groups in Jericho area (Ein Al-Sultan, Dyouk and Uja Spring with total annual discharge of 20 MCM (Martin B. & Joseph G.2004, MOA 2005).

Based on Ministry of Agriculture (MOA 2006) and Palestinian Central Bureau of Statistics (PCBS 2006), tomato production was the highest in Palestine during 2004-2005. It was 212,148 tons planted in 27,763 dunum. Tomatoes are produced under different cropping patterns; under open field, plastic houses, low and high plastic tunnels. The Jordan valley is the leader in production tomato fruit in open fields (PCBS 2006), where 29.8% of production and 26.4% of total planted areas by tomato in open field in West Bank are located in the Jordan Valley. The production in open field divides into rain fed and irrigated. Due to the lower rainfall events, the production of

vegetables in open field of Jericho depends only on irrigation (MOA 2005). In 2007, tomatoes growing under green houses were increased to about (200 dunums) more than 2006, (MOA 2007). The average production of cherry tomato is 13 ton/dunnum (PAPA Engineers 2008). Actual amount of water consumed by Tomatoes crops is about 600-700 CM/ dunum per crop season in open fields and about 1200 CM of water/ dunum under plastic houses, (MOA 2008). Previous related studies were conducting in several parts in the world (Steve, E. 2006; Brouwer, C., Heirloom, M. 1986; Dehayr, R., and Gordon, I. 2006; Reagan, M., Waskom 1994), less were adopted in the study area. A study prepared by Applied Research Institute about water resources and irrigated agriculture in the West Bank; shows that the total water demand for tomato irrigation was about 608 CM/dunum in open fields and in green houses is about 1023 CM/dunum (Isaac, Walid 1998) .

The main objective of this study is to identify and investigate the precession volume of water in term of quantity for growing tomato in Jericho area. This can be satisfied through Optimizing the irrigation volume of water needed for growing tomato during different growing stages, saving water without decline the production and increasing water productivity.

2. METHODOLOGY

A comparative study was conducted for the growth of tomato under different conditions in term of water consumption. The field location was a plastic green house with 2.5 dunum in Nueama, Jericho where the drip irrigation is applied. A cherry tomato variety 1335 (cluster) was planted in the green house in 28 lines with double rows, 15 lines of them under study control. Experimental design was divided into three blocks B1, B2, B3, each block includes five lines with five treatments TI, T2, T3, T4, T5, where the latest is connected with other 13 lines irrigated by farmer, irrigation amount of T5 depend on experience of farmer (25 years of experience) and controlled by engineers of PAPA project. The treatments were randomly selected without replacement. Each treatment value refers to irrigation percentage of ETc, relationship between treatments ETc is shown in table (1). On this design, the only variable item is the irrigated water amount, while other parameters are fixed such as pesticides, fertilizers for each trial, and recorded using global gab supervised by engineers of PAPA project. After approved, land was prepared before planting by added 6 ton of cow manure compost, cultivation, grading, and pre plant irrigated for about three hours, with 56 CM water.

Table 1: Treatments with %Etc

TREATMET	TI	T2	T3	T4	T5
%Etc	85%Etc	90%Etc	100%Etc	110%ETc	Farmer Irrigation

Crop water requirements (ETc) were calculated by multiplying reference evapo-transpiration (ET0) by a crop coefficient (Kc) (FAO 1998). ET0 was calculated with reference to NCARTT. The value of November to December 2007 was calculated as an average of ET0 for the period between2002 – 2006. For the growing months which in 2008, ETo was calculated as an average value for period between 2002 – 2007. Irrigation efficiency depends on field efficiency, and can be expressed as the ratio between the amount of water consumption by the plant and the total quantities delivered. The gross irrigation is the irrigation water includes losses that are occurring in the process.

$$\text{Gross irrigation} = \frac{\text{Net irrigation}}{\text{Field efficiency}} \quad (1)$$

Net irrigation requirements is the amount of irrigation water needed in actual irrigated area effected by factor of management practices such as effects of surface mulches which reduce loss by evaporation. The Net Irrigation Requirement (IRn) does not include losses that are occurring in the process of applying the water. Volume irrigation affected by leaching requirements, mulch factor and wetted area.

Net irrigation = (ETc * mulch factor * wetted area) /1000 (2)

Mulch factor = 0.9

Wetted area = No. of plants per line * space per plant (3)
= 500 * 0.4 = 200 m^2

The optimum space per plant is generally agreed to be 0.35-0.40 m^2 (A. Papadopoulos 1991).

Net irrigation= (ETc *0.9 *200) / 1000 = 0.18 Etc (4)

Leaching Fraction (Lr) is the amount of additional irrigation water required to move salts out of the root zone

Lr = (ECw/ 2*ECe) * 100% (5)

ECe max: is the maximum soil EC wanted in the root zone.

ECw: is electrical conductivity of the irrigation water (ds/m)

Lr= (0.680/4) * 100% = 17%

Preparation field with shallow cultivation and some difference in elevation estimated up to 8% that causes the Field efficiency = 75%
Gross irrigation = (0.18 Etc)/0.75 = 0.24 *ETc

The gross irrigation need per each treatment is calculated by multiplying gross irrigation by a treatment factor.
Gross irrigation for T1 = 0.85* 0.24 *ETc = 0.204 * ETc
Gross irrigation for T2 = 0.9 *0.24 *ETc = 0.216 * ETc
Gross irrigation for T3 = 1.0 *0.24 *ETc = 0.240 * ETc
Gross irrigation for T4 = 1.1 *0.24 *Etc = 0.264 * ETc

3. RESULT AND DISCUSSIONS

3.1 Water Consumption

The calculations show that the field efficiency with irrigation amount and scheduling for TI, T2, T3, T4, T5 and interval days for tomato crop was 75%. The study shows that there are relatively increasing in irrigation volume from initial stage to mid growth stage, then relatively decreasing in the end stage. In controlled trials (T1, T2, T3 and T4) the largest amount of water was used in14th March , while farmer used maximum amount of 3 CM water on 21th March. The Accumulation of irrigation scheduling per each treatment is shown in Table (2). T1 treatment used 57.63 CM of water, T2 used 61.01 CM, T3 used 74.55 CM and the T5 used more water than any controlled trial 86.92CM.

Table 2: Accumulation of irrigation scheduling for TI, T2, T3, T4 and T5

Date	TI (CM)	T2 (CM)	T3 (CM)	T4 (CM)	T5 (CM)	Stage
15 Nov.	0.2584	0.2736	0.3040	0.3344	0.666	A
18Nov.	0.7993	0.8501	0.9445	1.039	1.332	A
21Nov.	1.2645	1.3427	1.4918	1.641	1.998	A
24Nov.	1.9470	2.0653	2.2947	2.5242	2.664	B
27Nov.	2.6187	2.7765	3.085	3.3935	3.330	B
1 Dec.	3.3132	3.5110	3.9021	4.2923	4. 330	B
5 Dec.	3.8073	4.0342	4.4834	4.9317	5.330	B
8 Dec.	4.4137	4.6763	5.1968	5.7164	6.930	B
11 Dec.	5.1777	5.4852	6.0955	6.7052	8.1233	B
15 Dec.	5.7566	6.0981	6.7766	7.4544	9.1233	B
18Dec.	6.5808	6.9708	7.7462	8.521	10.1233	B
22Dec.	7.1680	7.5926	8.4370	9.2809	11.1233	B
25Dec.	7.9556	8.4266	9.3636	10.3002	12.9233	B
29 Dec.	8.8010	9.3217	10.2678	11.2948	13.9233	B
1 Jan.	9.5622	10.1276	11.1633	12.2799	16.5233	B
4 Jan	10.6135	11.2412	12.4006	13.6409	18.1233	B
8 Jan	11.0631	11.7168	12.9291	14.2222	19.1233	C
10Jan	11.8246	12.5231	13.8250	15.2077	20.1233	C
13Jan	12.8691	13.6290	15.0538	16.5594	21.1233	C
17 Jan	13.7096	14.5189	16.0426	17.6471	22.1233	C
20Jan	14.3442	15.1908	16.7892	18.4684	23.5233	C
23Jan	15.4144	16.324	18.0483	19.8533	24.9233	C
26Jan	16.4359	17.4056	19.2501	21.1752	25.9233	C
29Jan	17.3391	18.3620	20.3128	22.3442	26.9233	C
1Feb.	18.3826	19.4669	21.5405	23.6947	28.7233	C
4 Feb	19.4812	20.6301	22.8329	25.1164	30.0233	C
7 Feb	20.4732	21.6804	23.9999	26.4002	32.0233	C
10Feb	21.3988	22.6604	25.0889	27.5982	33.5233	C
13Feb	22.8148	24.1598	26.7549	29.4308	35.5233	C
17Feb	24.0391	25.4561	28.1952	31.0151	37.0233	C
20Feb	25.7540	27.2719	30.2128	33.2344	38.7233	C
24Feb	26.6271	28.1964	31.3005	34.4308	40.3233	C
26Feb	28.4031	30.0770	33.3901	36.7298	42.7233	C
1Mar	29.3563	31.0863	34.5115	37.9634	43.7233	C
3Mar	31.0125	32.8322	36.4600	40.1067	46.2233	C
6Mar	32.6614	34.5781	38.3999	42.2406	48.7233	C
9Mar	33.6648	35.6405	39.5803	43.5391	51.2233	C
11Mar	35.2814	37.3522	41.4822	45.6311	52.9233	C
14Mar	37.2943	39.4834	43.8502	48.236	54.9233	C
18Mar	38.940	41.2263	45.7867	50.366	57.4233	C
21Mar	40.0079	42.3571	47.0431	51.7480	59.4233	D
23Mar	41.7825	44.2361	49.1309	54.0445	62.4233	D
26Mar	42.9645	45.4877	50.5215	55.5742	63.9233	D
28Mar	44.1456	46.7383	51.9111	57.1027	65.9233	D
30Mar	45.3767	48.0419	53.3595	58.6959	67.9233	D
1April	46.3852	49.1098	54.5460	60.0011	69.9233	D
3April	47.4656	50.2538	55.8171	61.3991	71.4233	D
5April	48.6511	51.5090	57.2118	62.9333	72.9233	D
7April	50.1864	53.1346	59.0180	64.9202	74.9233	D
9April	51.4523	54.4749	60.5073	66.5585	76.9233	D
11April	52.7449	55.8436	62.0281	68.2313	78.9233	D
13April	54.0936	57.2717	63.6149	69.9767	80.9233	D
15April	55.2466	58.4925	64.9714	71.4688	82.9233	D
17April	56.4586	59.7758	66.3973	73.0373	84.9233	D
19April	57.6280	61.0140	67.7731	74.5507	86.9233	D

The accumulation of irrigation scheduling started from planting date to the end of November including initial stage (A). The difference between four trials (T1, T2, T3, T4) and T5 (farmer) reached maximum in 21th November at the end of initial stage. Starting from 27th Nov., the difference between four trails and T5 (farmer) decreased and accumulation of irrigation of T4 exceeded T5 (farmer), but T1, T2, T3 still lower than T5.

The accumulation of irrigation scheduling from beginning of December 2007 to 20th of March 2008 is known as the development (B) and mid season (C). The difference between four trials T1, T2, T3, T4 and T5 continue to increasing with irrigation time, which indicates that the farmer used more water in irrigation than any controlled trial. The difference between T2, T3, and T4 are relatively constant, while difference between T1 and T2 was the lowest due to the factor which used in multiple with ETc.

3.2 Production of Tomato

The first harvested day was after 130 days of planting. The production yields of tomato per each treatment were presented in Table (3).

Table 3: Production of Tomato

Date	TI (kg)	T2 (kg)	T3 (kg)	T4 (kg)	T5 (kg)
17/3/2008	48.730	64.200	43.790	53.280	40.629
21/3/2008	224.595	255.780	214.785	257.43	174.728
28/3/2008	292.125	286.980	295.368	218.975	279.300
5/4/2008	355.990	354.880	349.820	383.475	377.000
12/4/2008	339.060	377.300	294.222	262.400	307.600
20/4/2008	246.500	180.900	163.015	277.550	240.800
Total (kg)	1,507	1,520.04	1,361	1,453.11	1,420.056

Accumulation of the production as Figure (1) shows, T2 with total production of 1520 kg is the best productive trial, followed by T1 with 1507 kg , then T4 with 1453 kg , T5 (farmer) production is 1420 kg , and the T3 is the lowest productive line with 1361 kg.

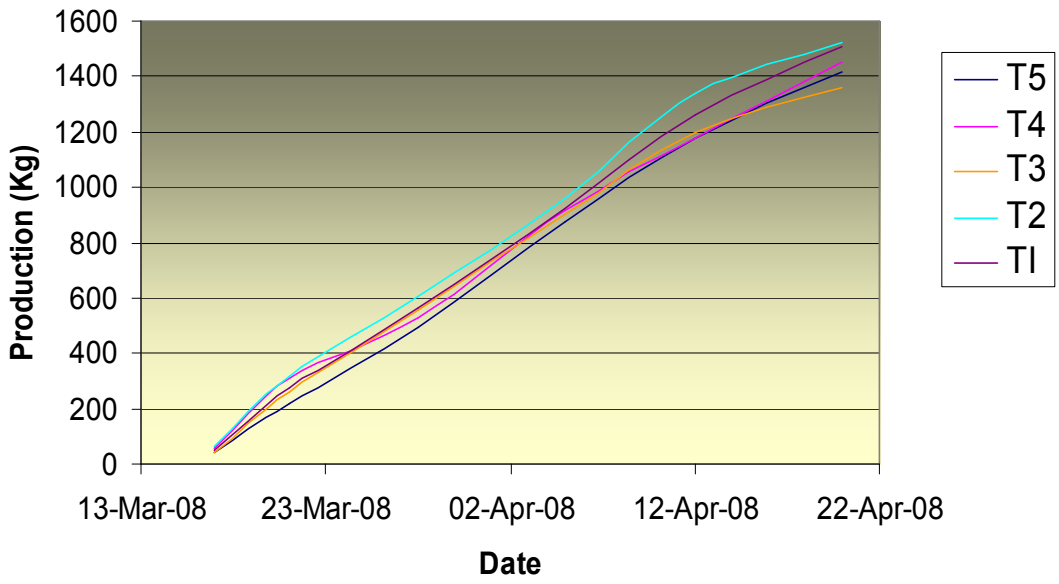


Figure 1: Accumulation of Production Tomato

While first line of T3 located closer to the door of the green house, it is affected by the frost during January month.

3.3 Relation between production and water consumption

Water productivity or water use efficiency is the amount of the production of harvested crop in kg per the volume of irrigation water in cubic meter used in irrigation. Table (4) summarizes the relation between production and water consumption.

Table 4: Relationship between production and water consumption

Treatment	TI	T2	T3	T4	T5
∑Production (kg)	1507.00	1520.04	1361.00	1453.11	1420.06
%Prod.	20.75	20.94	18.74	20.01	19.56
∑Water Consumed(CM)	57.63	61.02	67.77	74.5507	86.923
%Water Consumed	16.57	17.54	19.48	21.43	24.98
Water productivity Kg/CM	26.15	24.91	20.08	19.49	16.33

T2 treatment is more productive than other treatments, but less than T1 in term of water productivity. To evaluate which of these trials was the preferable, the economic value should be taken into consideration. Figure 5 shows the percentage relationship between production and water consumption. It represents that T2 with 17.54% of total irrigation volume produced 20.94% of total production, while T5 used 24.98% of total water but produced only19.56% of the production, that indicates that T2 produced more from less water as shown in Figure (2).

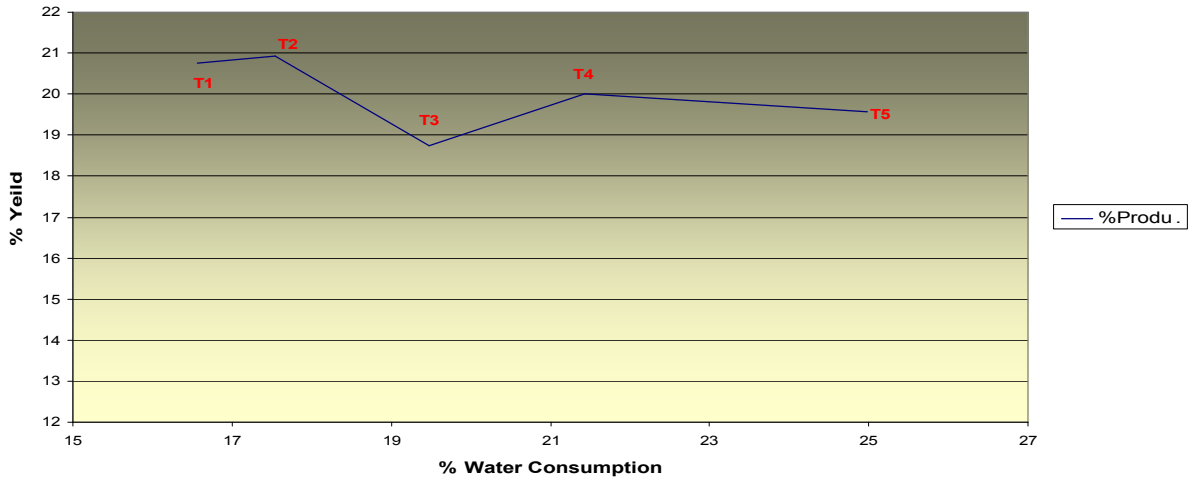


Figure 2: Percentage relationship between production and water consumption

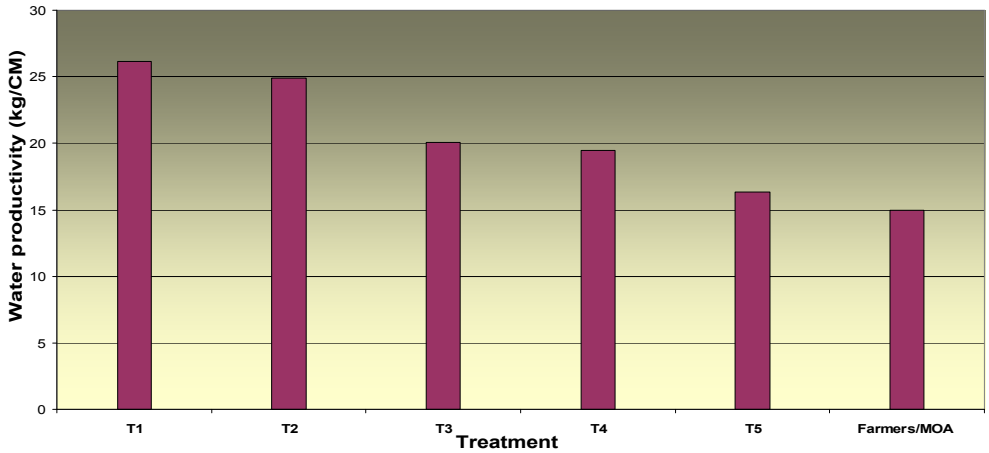


Figure 3: Water productivity

The required irrigation water is much lower than current water applied by farmer. Water productivity illustrate in Figure (3) where the productivity treatment of T1 is the highest with 26.15 kg/CM, followed by T2 with 24.91 kg/CM, T3 and T4, while T5 is only 16.33 kg/CM. Based on the Ministry of Agriculture data base, the average water productivity of tomato in Jericho district is 15 kg/CM, our study shows that the productivity of 1CM can reach more than this number.
Figure 3, present the benefit of water unit in T1 can reach 160% of T5, and 174% of average obtained by MOA. Comparison the results obtained from trials with farmer irrigation (T5) are presented in Table (5). All treatments gave more production with saving water, except T3 due to the frost weather. By applying precision irrigation method about 12.4% - 33.6% % of the current used water can be saved. The saved water can be used to expand the irrigated agricultural area also better irrigation practises can provide the maximum tomato production per unit land base.

Table 5: Percentage increasing of production and saving water related to T5

Treatment	T1	T2	T3	T4
%Increasing Production (related to T5)	6.12	7.04	-4.16	2.33
% Saving Water (related to T5)	33.66	29.81	22.03	12.37
%increase of agriculture area	33.70	29.80	22.00	14.20

Comparing the results of water consumed in irrigation which obtained from the experiment (Table 4) with the average water used by farmers in irrigation of tomato under plastic house in Jericho district, which obtained by MOA, 1,200 CM/dunum was calculated for full growing season of about 250 days, extends from mid of September to end of May, so the results of trial were converted to 250 days per dunum. Table 6 present the related average water used in tomato irrigation in Jericho distract, through precision irrigation, T1 treatment can give the maximum results in both water saving and ability to increasing agriculture area, 62.5% of water can be saved whereas 166.7% of agriculture area can be expanded. The production range between 10.63 ton in T3 to 11.88 ton in T2. The water amount, which is recommended by Technical Manual of PAPA project for cherry tomato is 700CM/season, which relatively meet the water which used by farmer (T5).

Table 6: Percentage of saving water and increasing of agriculture area related to average water used (MOA)

Treatment	T1	T2	T3	T4	T5
Water volume(CM)	450.2	476.7	529.5	582.4	679.1
Production (ton)	11.77	11.88	10.63	11.35	11.09
% Saving Water (related to average water used)	62.5	60.3	55.9	51.5	43.4
%increase of agriculture area	166.7	151.7	126.6	106.0	76%

4. CONCLUSIONS

The study shows that T2 of total production of 1520 kg is the best productive trial, followed by T1 with 1507 kg, then T4 with 1453 kg, T5 (farmer) production is 1420 kg, and the T3 is the lowest productive line with 1361 kg. T3 is affected by the frost, and the not smooth elevation.
The efficient irrigation management will leads to about 12.37% - 33.6% % of water saving with respect to T5 in experiment, a bit more related to PAPA technical manual, while 51.5%-62.5% of water can be saved with respect to average water used by farmers in Jericho district. This will cause a decrease in the water expense used by farmers, and increase in water quality and availability.
The result of water productivity for T1 was the best with 26.15 kg/CM , followed by T2 with 24.91 kg/CM, T3 with 20.08 kg/CM and T4 with 19.49 kg/CM, while T5 (farmer) is only 16.33 kg per cubic meter. The average water productivity in the district is 15kg/CM.

Applying this irrigation management will increase the agricultural land area from 14.2% to 33.7% %. However, while referring to the average of what farmers used in Jericho district, the agriculture land area could be enlarged from 76%-166.7%
The T2 treatment provide the best results in term of economic value, then followed by T1, T4, T5, and T3. T2 which show more than 100,000CM of saved water per season in 500 dunum of tomato green houses.

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OPTIMIZATION OF WATER RESOURCES SYSTEMS UNDER CLIMATIC UNCERTAINTY USING SCENARIO ANALYSIS

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Abstract: In water resources management optimization problems under climatic variability, uncertainty is mainly associated with the values attribution for hydrological exogenous inflows and demand pattern trends. Deterministic optimization models are inadequate to deal with these problems and traditional stochastic optimization models frequently cannot be used if there is insufficient statistical information to support the optimization model. In this paper the uncertainty is modelled by a scenario approach in a multistage environment analysis which can include different possible system configurations in a wide time horizon. A robust chance optimization model is then developed in order to obtain a so-called "barycentric value" with respect to decision variables.

The paper proposed a subsequent validation phase defined as "re-optimization". The re-optimization step, based on the previously obtained barycentric value solution, allows reducing the consequences deriving from wrong decisions. Implementation of the scenario-optimization procedure has been developed and coded in an improved version of WARGI DSS (Sechi et al. 2000, 2003, 2007). WARGI performs the scenario analysis by identifying trends and essential features of water system on which to base a robust decision policy for systems management. The improved version of WARGI can be linked to commercial solvers as well as to some free solvers for the mathematical problem coded using the MPS standard.

The improved WARGI DSS can be used for large dimension problems based on the open source philosophy, that exploits the speed of network simplex methods in order to obtain very efficient solutions to the scenario optimization problem. The application to a real water resource system in Sardinia - Italy, shows the usefulness of the proposed approach for water resources systems affected by a high level of uncertainty in data related to hydrological inputs. It appears that the used scenario optimization approach can be a promising alternative tool for large size optimization problems coming for complex real resource systems under climatic uncertainty.

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ACCESS TO WATER RESOURCES IN DIFFERENT PARTS OF ALLUVIAL MEGAFANS, EXAMPLES FROM ARID LANDS OF CENTRAL IRAN

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Abstract: The increasing demand for freshwater supplies as the result of growing populations and industrialization is in contrast with the limited water supplies in arid and semi-arid countries worldwide. Water scarcity has been of concern in arid and semi-arid lands of Central Iran since ancient people invented Qanats as a potential technology to harvest the underground water. However, the challenge of sustainable water management has been magnified by the climate, geomorphology and inherited social issues. The arid lands of Central Iran have only one main sources of renewable freshwater that is limited rainfall, occasionally transformed into flash floods. The best renewable water resources in these arid lands are the natural groundwater reservoirs developed in alluvial megafans, where the ancient people well recognized to dig the long underground tunnels of Qanats. Climatic fluctuations accentuated by periodic drought, affect water availability in cities built upon these alluvial megafans while this resulted quite different situations in wells and Qanats drilled in different parts of these megafans. The sedimentary facies association of alluvial megafans represents the ideal water reservoirs in semiarid to arid lands. Megafan has been defined as a large (10^3-10^5 km^2), fan shaped mass of sediment deposited by the migration of a permanent or intermittent channelized stream with a point source from the outlet of a large mountainous drainage network. These fans are usually characterized by the large size of the catchment, which is capable to conduct water to the apex of the fan and sedimentary basins adjacent to mountain belts. The morphology, facies distribution and tectonics control the distribution of recharge, lateral flow, and discharge zones in the fan aquifers. This study highlights the importance of alluvial megafans as the potential water reservoirs in arid land and with examples from central Iran demonstrates how the high quantity of groundwater pumping in proximal to distal areas of the megafans, associated with rural development in these areas, could bring along serious environmental problems. One of the key points to focus is the past and future of Qanats, the greatest effort and achievement in ground-water utilization by ancient people, who well recognized the importance of alluvial-fan aquifers in arid lands.

Key words: Arid land, alluvial megafans, Qanats, Central Iran, Water resources.

1. INTRODUCTION

It is a well known fact that the scarcity of water in arid lands and its misuse pose a serious threat to sustainable development and protection of the environment (e.g. Zektser & Everette, 2004; Kahlown & Aslam, 2005). The hydrological resources in arid lands are commonly linked to alluvial fans and where the size of the fan is in the range of megafans they comprise volumetrically significant depositional and hydrofacies elements of sedimentary basins adjacent to mountain belts (e.g. Gohain, & Parkash, 1990; Stanistreet & McCarthy, 1993; Houston, 2002; Arzani, 2003). The depositional system of alluvial fans is complex and it controls the fan aquifer systems where multiple scale of lithofacies heterogeneity may results in unpredictable hydrofacies distribution (Weissmann & Fogg, 1999; Weissmann, et al., 2005). The alluvial megafans are those fans developed over a huge surface area, and in the case of this study have fan radius (fan apex to fan toe) of at least 30 km and a fan surface area of 10^3-10^5 km^2 (Arzani, 2005; 2007a). Groundwater flow is an important process in alluvial megafans and groundwater recharge occurs by infiltration of periodic floods in the upper fan and by direct precipitation on the fan surface. The gradient and the depositional fan surface controls the downslope groundwater flow, which results in discharge zone along the lower fan and at the fan toe (Belitz & Heimes, 1990). The geomorphology and the hydrological system of alluvial fans well recognized by ancient people (the history comes back to >2000 year in Iran), who tried to use underground water by mean of Qanats (Wulff, 1986; Papoli &

Labaf, 2000; Bouayad, 2002). They were devised to tap groundwater through gravity flow, includes a mother well and a series of vertical shafts connected by a gently sloping tunnel. The vertical shafts, which are up to 100 m deep, decrease in depth toward the appearance of the underground tunnel to the surface. A Qanat can measure from several hundred meters to more than 50 kilometers (Arzani, 2003; 2007b).

This study highlights the importance of water resources in alluvial megafans, developed in arid lands. Examples from the Quaternary fluvial megafans of Central Iran (NE of Esfahan to E of Abadeh), which are located along tectonically active mountain fronts in an arid-semiarid climatic setting, are presented. It stresses the importance of understanding the hydrogeologic framework of the aquifer system in megafans and the environmental sensitivity controlled by underground water budget and consumption in these areas.

2. GEOMORPHOLOGY AND HYDROLOGY

The study areas are the alluvial fans and deserts around the Kohrud Mountain Belt (KMB) and part of Gavkhoni-Abarkoh Basin (GAB) in Central Iran (Fig. 1A & B). The GAB occupies the north-western to central portions of the larger Gavkhoni-Abarkoh-Sirjan depression (Tarez, et al. 1980; Berberian & King, 1981, Arzani, 2005).

The KMB megafans include those located in Soh, Zefreh and Ardestan areas. They have drainage basins that are carved into this mountain range and can be separated into two broad terranes divided by the Qom-Zefreh fault (Fig. 1C & D).

The GAB megafan includes the fan located in Abarkoh and its drainage basin comprises the neighboring Abadeh Basin (Fig. 2A & B). The width of the GAB is about 100 km in the studied area and is bounded by the Dehshir highlands and fault in the east and the Hambast Mountains and faults in the west (Fig. 1A & B & Fig. 2A). The drainage basin of GAB megafan is to the west and comprises the Abadeh Basin.

The long profiles for each of the fans are an important element of understanding catchment and fan area evolution in that, together with the channel networks, they fix the boundary conditions for slope processes and as the result provide the different conditions for underground water. In cross-section all of the studied fans are wedge-shaped and the later phase of fan development is also incised.

2.1 Kohrud Megafans

Along the southern side of the KMB, between Soh and southeast of Varzaneh, there are 35 alluvial fans that build out from the edge of the mountain front towards the Gavkhoni-Abarkoh-Sirjan depression and often cited against normal active faults (Fig. 1 E & F). Among these, three megafans were selected for this study. They are (from northeast to southwest): Soh; Zefreh; and Feshark. The Soh fan is the largest with an area of 537 km^2 (Table 1). It also illustrates some of the best exposures of the fanglomerates. The Soh fan is comparable to other similar sized fluvial fans along the margin of the basin (e.g. Arzani, 2005). The Zefreh and Feshark fans are slightly smaller is size with fan areas of 309 km^2 and 179 km^2 respectively (Table 1).

Table 1. The parameters of the three megafans of the Kohrud Mountain belt

Parameters	**Soh**	**Zefreh**	**Feshark**
Fan area (km^2)	537	309	179
Catchment area (km^2)	178	84.5	23.5
Average fan slope	0.01-0.15	0.04-0.2	0.06-0.23
Average catchment slope	0.48	0.39	0.39
Catchment length (km)	23.6	15.7	8.7
Fan progradation distance (km)	32.4	35.6	28.3
Fan radius (km)	8.1	6.3	3.7

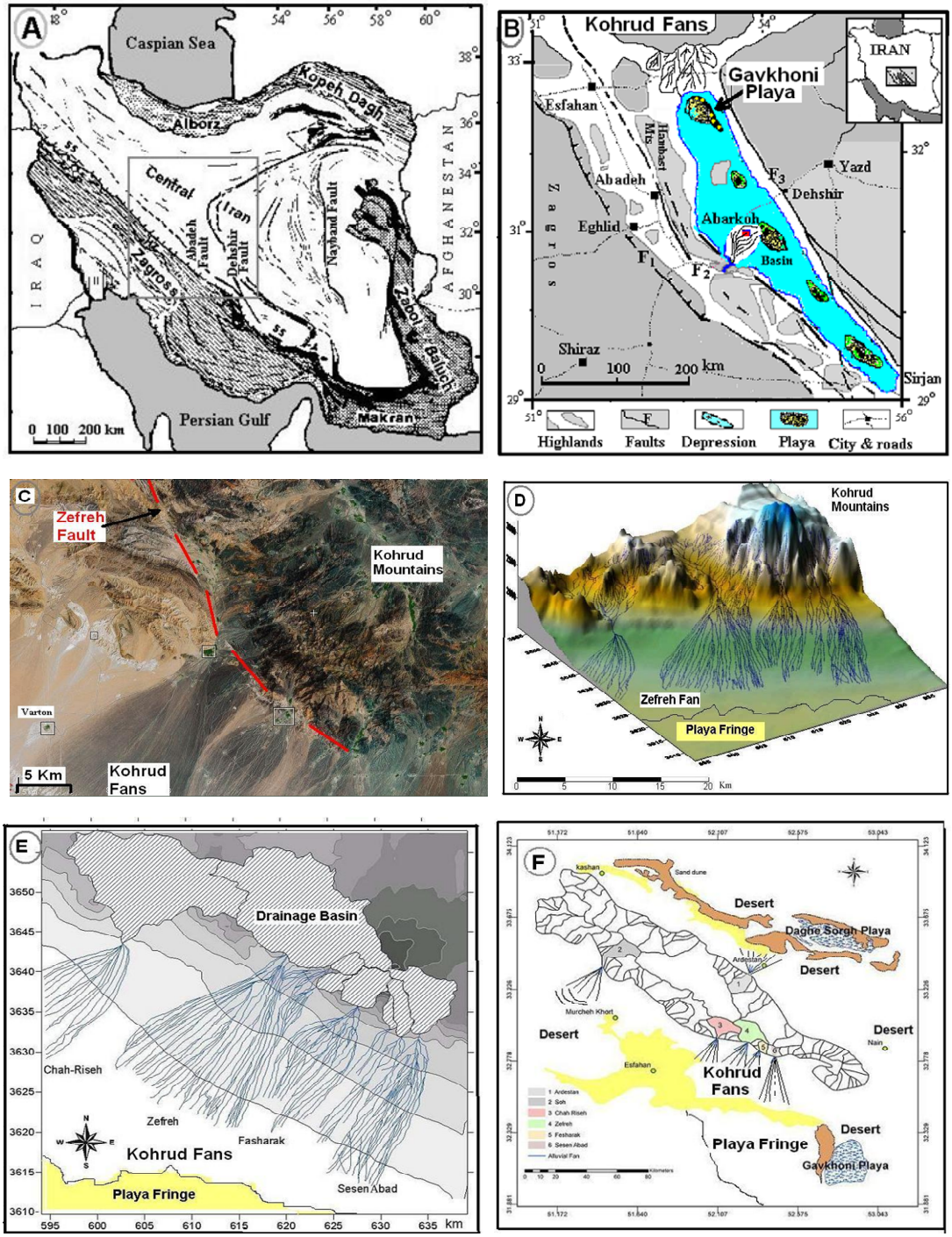


Figure 1. Alluvial megafans and deserts of KMB in Central Iran. A & B. Tectonostructural and sketch map of Iran showing the fault-controlled depression in Central Iran. C. Satellite images of fans around KMB and Qom-Zefreh fault. D, E & F. Drainage basins, fans and deserts around KMB.

The rivers draining the northern and southern sides of the MEB have predominate transverse patterns that cut across several basin bounding faults and build large alluvial fans. The fans on the northern side of the basin form the subject of this study. Along the basin axis flows the Zayandeh Rud River that in many places cut the toes of the alluvial fans or captures any associated drainage (Fig. 2).

The present climate of the area is arid to semi-arid. The average rainfall in KMB ranging between 50 and 200 mm in the basin, and up to 350 mm in the headwaters of many rivers of the Because of this, all rivers except the Zayandeh Rud are ephemeral, and most flows occur during the

late winter months and Spring of December to April. A rainfall gradient from the headwaters to the middle/lower reaches of many streams exists and there is the likelihood of flow more commonly in the upper reaches of the rivers.

2.2 Abarkoh Megafan

The Abarkoh megafan has an area of 940 km^2 with a radial length of 45 km, a maximum width of 40 km and a gentle slope of 25′ (0.006). It typifies one of the large Quaternary fans in Central Iran. This fan originates from a valley on the western highlands and terminates into a playa lake in the centre to eastern part of the Abarkoh Basin (Fig. 2).

At the apex of the studied megafan, the feeder channel is ~700-m wide, 2.5 km long and crosscuts the Hambast Mountain range. It connects the higher level (2500-1850 m above sea level) drainage basin of the Abadeh to the lower level (1850-1450 m above sea level) Abarkoh Basin to the east (Figs. 1 & 2).

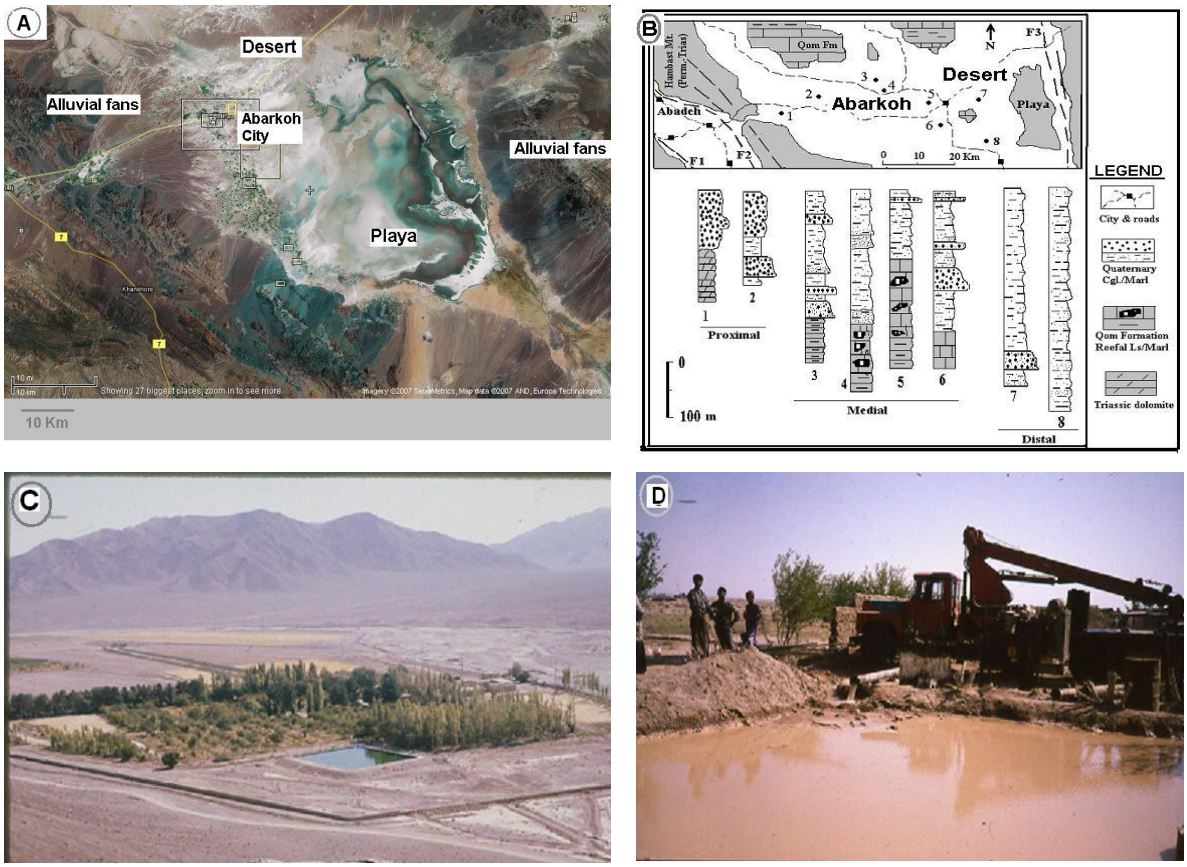


Figure 2. Abarkoh megafan and access to water resources in different parts of this fan. A. Satellite image showing the apex to toe of the fan and desert in playa fringe. B. Sketch map of the Abarkoh Basin with water wells from proximal (well no.1) to distal parts (well no. 7 & 8) of this fan. C. Access to good of water resources in proximal parts of the fan. D. Poor water tables in distal parts of the fan.

The climate of the Abarkoh is arid with average rainfall ranging 29-132 mm and the catchment area of the studied megafan is arid to semiarid with average rainfall ranging between 60 mm in the Abadeh Basin to 300 mm in the headwaters of some rivers in the Zagros highlands (Bagheri 1995). The precipitation is mostly storm rainfall (in late autumn and early spring) and less frequently with snow on the Zagros hills. The summer maximum daily temperature (for >15 days) is up to 42 ºC in the Abadeh plain and up to 50 ºC in the Abarkoh plain and near the playa borders. Other than the cultivated lands and mainly near the villages and cities, the vegetation cover is barren to very poor in the rest (~ 80%) of the basin floor and restricted to scattered small bushes and sparse grasses. There are many farm lands and villages located in the proximal to distal part of the

studied fan. The old city of Abarkoh, with about 40000 populations and 80 km^2 in area, is located in the medial part and 35 km down the apex (Fig. 2). Catastrophic floods have destroyed roads and part of this city and several villages in the last 100 years.

3. ACSESS TO GROUND-WATER RESOURCES

Periodic rainfall and low frequency floods and groundwater are the two major components of freshwater resources in arid and semi-arid lands of the studied area. The system of water supply is limited by the amount of underground water which can be extracted by Qanats and wells. Qanat is a type of underground irrigation canal between an aquifer on the piedmont to a garden on an arid plain (Wulff, 1986; Papoli & Labaf, 2000; Bouayad, 2002; Arzani, 2003). The word is Arabic, but the system is best known from Iran. The Ardestan and Abarkoh Qanats are famous in Iran, especially the double story moon Qanat in Ardestan which is a sign of intelligence and precision of the Qanat practitioners and well diggers at that time.

A Qanat is constructed based on mainly considering the logical relationship between (a) the gradient, (b) the depth of main well, (c) the length and (d) the discharge of the area. To make a Qanat, one needs a source of water, which may be a real well, but can also be an underground reservoir which can be best recognized in heterogeneous sedimentary facies of the alluvial fans and as a damp area in an otherwise arid region (see Arzani, 2005 for details). Although, sometimes recharge processes are not well understood in alluvial fans, when one has identified this source, a tunnel is cut to the farm or village that needs the water. Shafts are added for three reasons: as air supply, to remove sand and dirt, and to prevent the tunnels from becoming dangerously long. The shafts are not very far apart, and as a result, a Qanat seen from the air gives the impression of a long, straight line of holes in the ground - as if the land has been subjected to a bombing run.

There are two main types of Qanat based on the topographic situations: the mountain front with normally short-length Qanats, constructed at the base of foothills and the alluvial plain, long-length Qanats, with deep (up to 100 m) mother wells. The mother well of latter are normally located at the middle parts of the alluvial plains of megafans (Fig. 2 B). The discharge of the mountain front type is highly dependent on the local annual precipitation, where as the second type of the Qanats are collecting water over a wide area of the fan and their discharge fluctuate over a longer time in arid zones.

With the invention of modern technology and drilling deep water wells in different parts (from proximal and near the fan apex to distal near the toe) of the studies megafans, access to water was mainly controlled by the location of wells in the surface of the fan and the depth of the wells. For example, more than 850 water wells (100 m to 300 m deep) were drilled in Abarkoh megafan during 1968-1980. At the same time, 130 Qanats, some of them up to 18 km long and with a total discharge of about 800 lit/sec, were in this area. The gradient of the underground water table and the proximal to medial hydrofacies (sheets and channel belt conglomerates) of the megafan in contrast to their distal counterparts provided different water discharges and the best wells (discharge up to 20 lit/sec) were those located in proximal to medial parts of the fan. As the result this competition in ground-water pumping and recent climatic drought, more than 100 Qanats dried down during 1980-2003 (see Arzani, 2003, for details). Recently, more stressed and lowered ground-water discharges are those wells located at the distal parts of the studied fans, not only because of the limited fresh-water supply but also because of the invasion of the underground salty water of the playa lake (Arzani, 2007a). This brought along serious social and environmental problems with not only desert advance but also immigration of farmers. The crisis led to the realization that a national master plan for policy, institutional and technological changes is required to stabilize the situation and to manage its water sector more efficiently with a long-term perspective.

4. CONCLUSIONS

Although it has been widely recognised that alluvial fans are important water resources in different climatic settings, the megafans could provide potential water resources in arid lands. They comprise sensitive geomorphic and sedimentary hydrofacies system in different parts, from mountain front, proximal to desert and distal parts of megafans. Rate of water recharge and discharge in alluvial megafans controls the social and environmental issues associated with underground water budget, quality and consumptions in arid lands.

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A 3D CONCEPTUAL HYDROGEOLOGICAL MODEL FOR THE LOWER JORDAN RIFT VALLEY

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Abstract: In the frame of the Integrated Water Resources Management (IWRM) SMART project, funded by the German Ministry of Science (BMBF) a trans-boundary model (Jordan, Palestine and Israel) was constructed for the simulation of groundwater flow in major water bearing layers, in order to provide rational estimation of the water resources, their availability, to design groundwater exploitation schemes and to quantify the expected changes subsequent to the global climate change. The study area is the Lower Jordan Rift Valley (LJRV). Its geology and topography is rather complex and displays notable differences between the eastern part (Jordan) and the western one (Palestinian Authority and Israel). Uplifted rift shoulders bordering deep depressions filled with young sediments are common along the area. On the Highlands west of the basin floor, Middle Cretaceous carbonates and Upper Cretaceous clays, marls, chalks and cherts are exposed. The eastern highlands are composed mostly of Paleozoic, Lower Cretaceous Nubian sandstone and Middle-Upper Cretaceous carbonates. The different rock units on the two opposite sides of the basin reflect both relative motion and differential uplift. The basin-fill holds within it a few km thick succession of Neogene to Holocene marls, clays and conglomerates as well as salt bodies and layers. The main aquifers that supply most of the water demand are the deep water bearing layers of the Judea Group mainly the Cenomanian-Turonian carbonates in the west and the Lower Cretaceous Kurnub Group and the Cenomanian-Turonian Ajlun Group in the east. As first step a conceptual model of the LJRV geology and hydrogeology was constructed. It included the definition of the model borders, of the major geological layers to be taken into account for the purpose of water resources evaluations and the major faults (those capable of having an impact on the groundwater flow and transport processes). To this end a digital elevation model (DEM) with a resolution of 1X1 km was constructed for the elevation of the Top Turonian layer. Then a similar DEM was constructed for the thicknesses of the relevant layers above and below the Top Turonian. Additionally the major faults were mapped, together with their dip. All this information was organized within the frame of a GIS based application. The second step was to create a model made of triangles in plan view and of triangular prisms in 3D. The model was created by spreading point following a predefined density distribution, suited to match the geological characteristics of the subsurface. Fault paths were included as constraints. The resulting set of points was triangulated using a constrained Delaunay triangulation (Vigo). The vertical topography was constructed by applying an interpolation procedure based on Compact Support Radial Basis Functions. Regular geological cross-sections and outcrop maps were constructed from previous studies, borehole data, geological maps and seismic data and then compared to the Geological cross-sections and exposed layers created from the resulting model. The output show good agreement with the existing cross-sections and outcrops map. The model will further be used to the evaluation of the groundwater resources in the LJRV.

Key words: Lower Jordan Rift Valley, water resources, conceptual hydrogeological model.

1. INTRODUCTION

average yearly precipitation of approximately 100 mm. The LJRV suffers from substantial and steadily increasing hydrologic deficits. Water demand is also increasing as the most realistic prospects for economic development are based on agriculture and tourism, which are large consumers of water. The vast majority of the available water resources are aquifers, which are usually deep. The LJRV is physically isolated from other water bearing areas or from the large desalination centers, which are being built along the Mediterranean coast. Therefore, the LJRV will have to count on its own available sources of water in the foreseeable future. The LJRV encompasses an area of 5,000 Km2 and is shared by three political entities, Jordan, Palestine and

Israel. The German ministry of science and education (BMBF) is actively present in area, also promoting and funding multilateral cooperation projects. In 2005 it launched the SMART project,

aimed at the development of the LJRV water resources following an Integrated Water Resources Management approach. In this context, it was decided to construct a three dimensional computational hydro-geological model of the major water bearing layers in the LJRV so it could be used a major tool for the evaluation of water exploitation alternatives, for the estimation of the impact of global climate change that is contemplated worldwide, though various degrees of acuteness and to serve as a rational basis of discussion between decision makers that often have critical problems of communication.

This paper is structured as follows: section 2 presents a short description of the state of the art characterization of the LJRV geology and hydrogeology; Section 3 describes in detail the process of the conceptual model construction; section 4 describes the preparation of the computational model and section 5 outlines how the model will be used in the subsequent stages of the project execution.

2. GEOLOGY AND HYDROGEOLOGY

The study area (Fig.1) covers a surface of approximately 5,000 Km2 and extends on both sides of the LJRV from the ground water divide line of the Jordanian highland flanks in the east through the deep rift basin and to the ground water divide line of the Judean Mountains rift margins in the west (Israel and Palestine). The stratigraphic succession exposed on both sides of the LJRV and the Dead Sea starts with top Precambrian molasses sediments and volcanics, and terminates with young

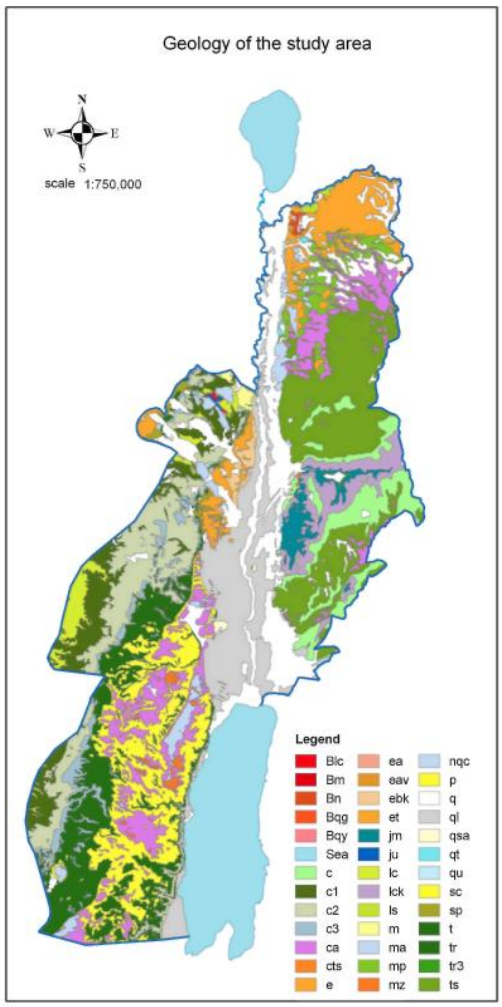


Figure 1: Location and geological map of the study area (after Sneh et al 1998)

Pleistocene and Holocene Rift Valley Fill (Sneh et al 1998). Valuable stratigraphic information is achieved by water wells, seismic data (Anker 2007; Anker et al 2008) and geological maps (Sneh et al 1998; Begin et al 1975). The deep-seated stratigraphy is based on deep oil wells in the valley itself, and on its margins.

The western and eastern banks of the Rift Valley are rather different structurally and stratigraphicly. The eastern side is higher structurally, and exposes deeper stratigraphic levels. Along the eastern shores of the Dead-Sea Precambrian Paleozoic and Triassic rocks crop out, whereas in the western side only Cretaceous sediments occur. Northwards, along the eastern Jordanian side Triassic to Neogene succession prevails and only Cretaceous to Neogene sediments expose in the west bank (Flexer and Yellin-Dror 2009).

The LJRV is the base level for the surrounding surface and groundwater. In the western side of the valley, the regional aquifers that supply most of the fresh water demand are the deep water bearing layers of the Judea Group mainly the Cenomanian-Turonian carbonates in the west and the Lower Cretaceous Kurnub Group and the Cenomanian-Turonian Ajlun Group in the east (Table 1). The natural replenishment is mainly from direct precipitation on the Judean Mountains outcrops (Fig. 1) and the Jordanian high

land.

The understanding of the flow pattern from the regional aquifers into the Rift Valley is important in building a conceptual model and in understanding the salinization problems. From practical point of view, the understanding of the flow mechanism is an important tool for the sustainable exploitation of the groundwater resources to the population in the region.

Table 1: The hydrological characteristics and the stratigrafic correlation of the rock sequence in the study area.

Western LJRV (Israel and Palestinian Authority)	**Eastern LJRV (Jordan)**
Eocene: Avedat Gr. Aquifer	Eocene
Senonian: Mount Scopus Gr., Aquiclude	B3: Muwaqqar Fm. Aquiclude
No equivalent in Israel and Palestinian Authority	B1,B2: Ghudram and Amman Fms. Aquifer
Turonian: Nezer & Shivta Fms. Aquifer.	Turonian A7: Wadi Sir Limestone Aquifer
Upper Cenomanian: Weradin Fm. Aquifer; Kefar Shaul Fm Aquiclude; Aminadav Fm. Aquifer.	A5-A6: Shueib Fm. Aquitard
Middle Cenomanian: Moza and Bet Meir Fms. Aquiclude.	A3: Fuheis Fm. Aquifer
Lower Cenomanian: Kesalon and Soreq Fms. Aquitard	A1-A2: Naur Fm. Aquifer
Aptian: Kefira Fm. Aquifer	No equivalent in Jordan
Lower Cretaceous: Kurnub Gr. Aquifer.	Kurnub Gr. Aquifer
No equivalent exposed in Israel and Palestine	Jurassic: Azab Gr. Aquifer

3. MODEL CONSTRUCTION

The construction of any computational model includes a number of stages as follows: 1) a conceptual model, i.e., a simplified representation of the complex geological and hydrologic reality, yet representative of the main features under scrutiny; and 2) translation of the computational model to a format that can be handled by groundwater flow simulation software.

The basic steps in the construction of a conceptual model are:

1. Prepare a map of the ground-surface elevation (Fig. 2a).
2. Define the aquifer layers under investigation, or the water bearing layers: in our case these are the upper and lower Cenomanian layers in the west, the alluvial aquifer in the valley and the lower cretaceous layers and Cenomanian layers in the east (Table 1).
3. Define the borders of the study area: ideally these would be defined by natural borders of the aquifer layers under investigation, for which the conditions are known (known fluxes or groundwater head elevation). In practice this is seldom the case and the borders are defined as a compromise between the need to limit the size of the flow domain and the necessity to place the borders such that any uncertainty about the prevailing conditions there would not affect the flow and transport processes in the area of interest (Fig. 1).
4. For each aquifer layer provide a structural map, describing its elevation and thickness (Fig. 2b).
5. Map geological features with a potential impact on the groundwater flow processes, such as faults with significant dips (Fig. 2c).
6. Provide initial values of the main hydraulic parameters (hydraulic conductivity, anisotropy ratio, effective porosity and storativity).
7. Identify water sources and sinks (natural, or man-made), such as springs discharging from the aquifers under investigation, wells, etc.
8. Outline the aquifer's outcrops, the area through which, the natural replenishment from rainfall occurs (Fig. 2a).
9. Identify the various water bodies in the aquifers under investigation, or in the case of the

LJRV aquifers with different salinity content (Guttman, 2000).

Gather relevant hydrologic and meteorological data, including springs location and their discharges, wells location, rain-stations.

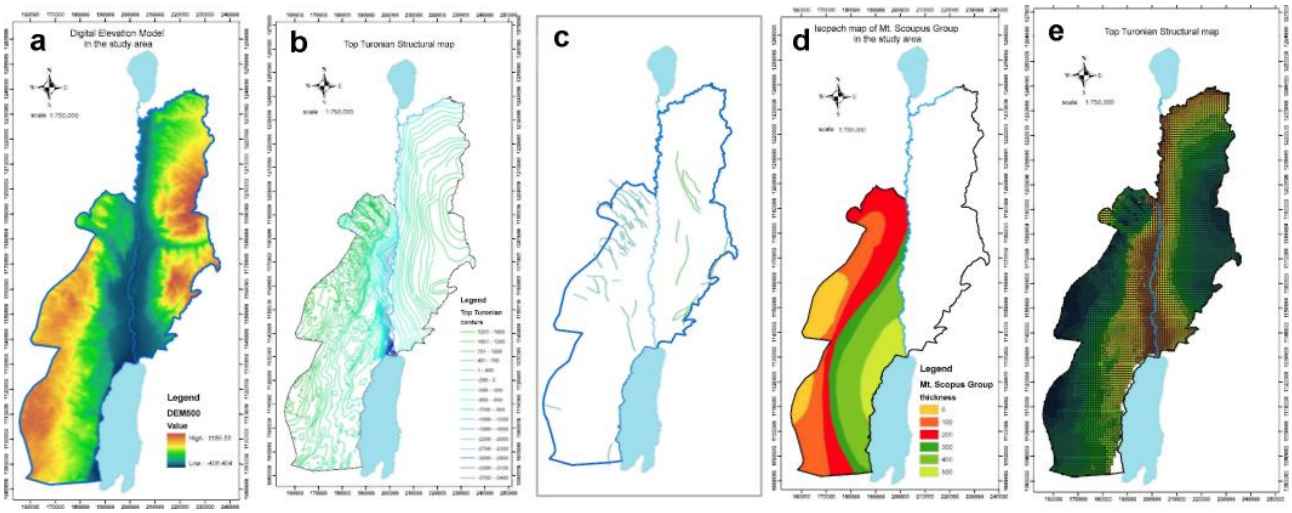


Figure. 2: Digital maps and data constructed for the conceptual geological model of the study area (From left to right: (a) Topography, (b) Top Turonian structural map,(c) fault pattern, (d) Isopah maps, (e) digital elevation model).

The borders of the LJRV model area were determined in close cooperation between the hydro-geologists of the SMART team and are illustrated in Figure 1. In the west (Israel and Palestinian Authority) the borders are identified with the water divide that separates the western aquifer from the eastern one, mainly along an axis from north to south, linking the cities of Ramallah, Jerusalem and Hebron. In the east the borders are identified with the subsurface water divide line of the Jordanian High land. The geological layers of interest are presented in the geology and Hydrogeology chapter (Table 1).

A digital elevation model (DEM) with a grid density of 1km was then constructed (Fig. 2e) using the following approach: 1) the elevation of the top Turonian was prepared (Fig. 2b) for each layer of interest above and below the top-Turonian layer thicknesses were calculate (Fig. 2d) the ground surface elevation was mapped (Fig. 2a) on the same grid points.

Major faults with a significant dip were mapped (Fig. 2c). For each fault a separated DEM was constructed, including, the fault points location and the elevation of the Top-Turonian on both sides of the fault.

Major outcrop areas were identified and digitized (Fig. 2a).

4. THE COMPUTATIONAL MODEL

The conceptual model was then translated into a computational one. This included: 1) the preparation of a computational three dimensional mesh; 2) the incorporation of the outcrop areas and 3) the inclusion of sources and sinks.

The computational mesh was constructed in two steps. We constructed a 2-D dimensional grid of the study area that included: 1) definition of point densities at different regions of the model domain; 2) generation of points using the Bubble pack algorithm of Shimada; 3) inclusion of wells and springs as mesh points; 4) inclusion of the major fault-paths as constraints and fault points as mesh points; 5) triangulation of the generated grid points and associated constraints using a CDT (Constrained Delaunay Triangulation) suggested by Vigo Anglada. The resulting mesh is illustrated in figure 3. It comprises 42660 2D mesh points and 84876 2D elements of triangular shape.

The tridimensional structure of the mesh was constructed using the digital elevation model (DEM) and an interpolation procedure, based on the compact support radial basis function (CSRBF) approach. The model includes 12 layers or 13 levels (a layer is defined by its top and bottom level). First the top turonian elevation was created and mapped to the mesh, and then the layers above the turonian and below it were generated using the thicnes information. Finally, the upper level corresponding to the groundsurafce elevation was created. The model layers were then truncated accoring to the groundsurface elevation. Any level with an elevation higher than the groundsurface elevation was reassigned to the uppermost level elevation. The resulting three dimensional model comprises 554580 3D nodes and 1018512 3D elements. Selected cross sections trough the model are illustrated in figures 4 and 5. Model consistency was checked against existing cross sections and geological maps.
The outcrops of the model domain were mapped and are illustrated in Figure 6

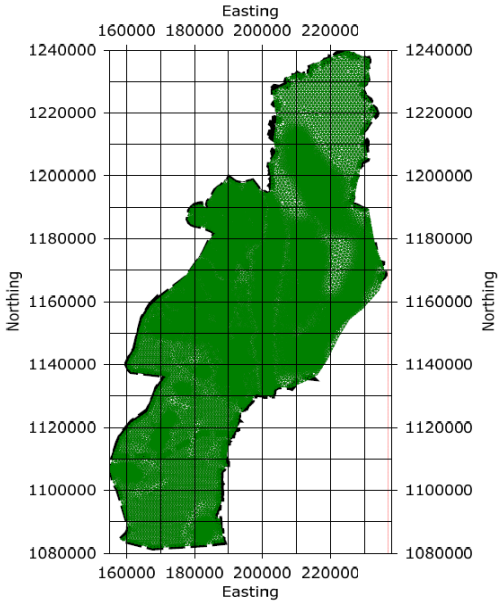


Figure 3: Plan view of computational mesh

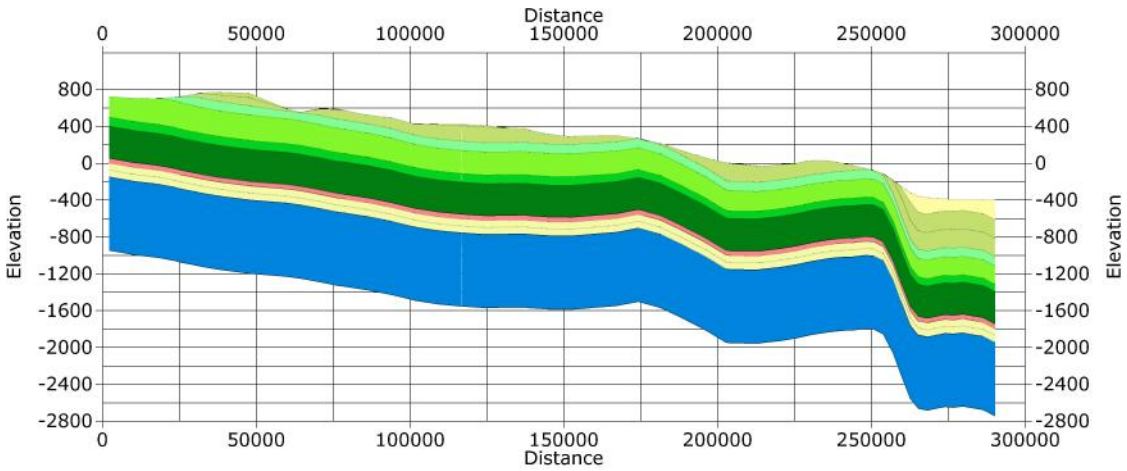


Figure 4: Vertical cross section I through the model

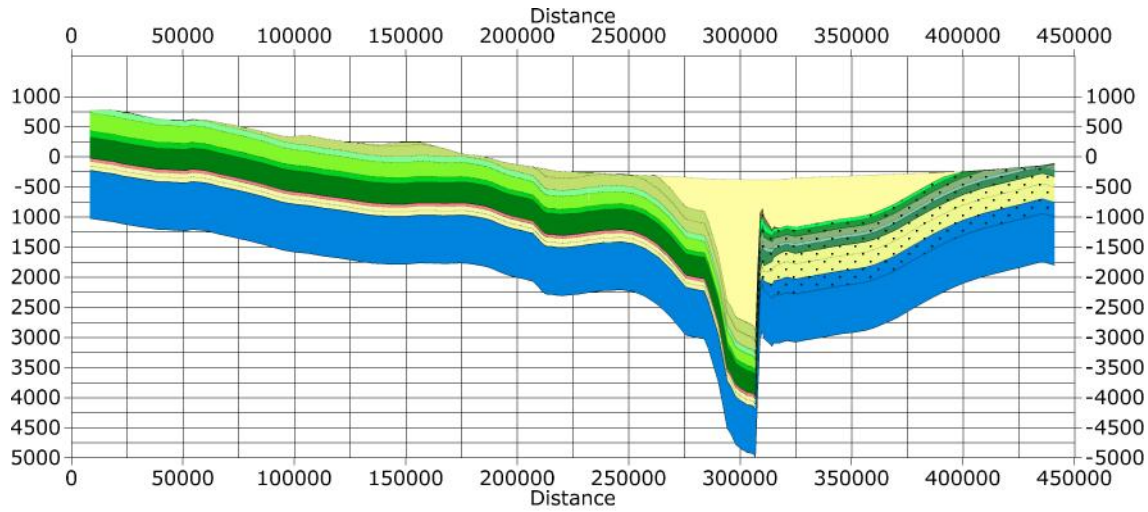


Figure 5: Cross section II though the model

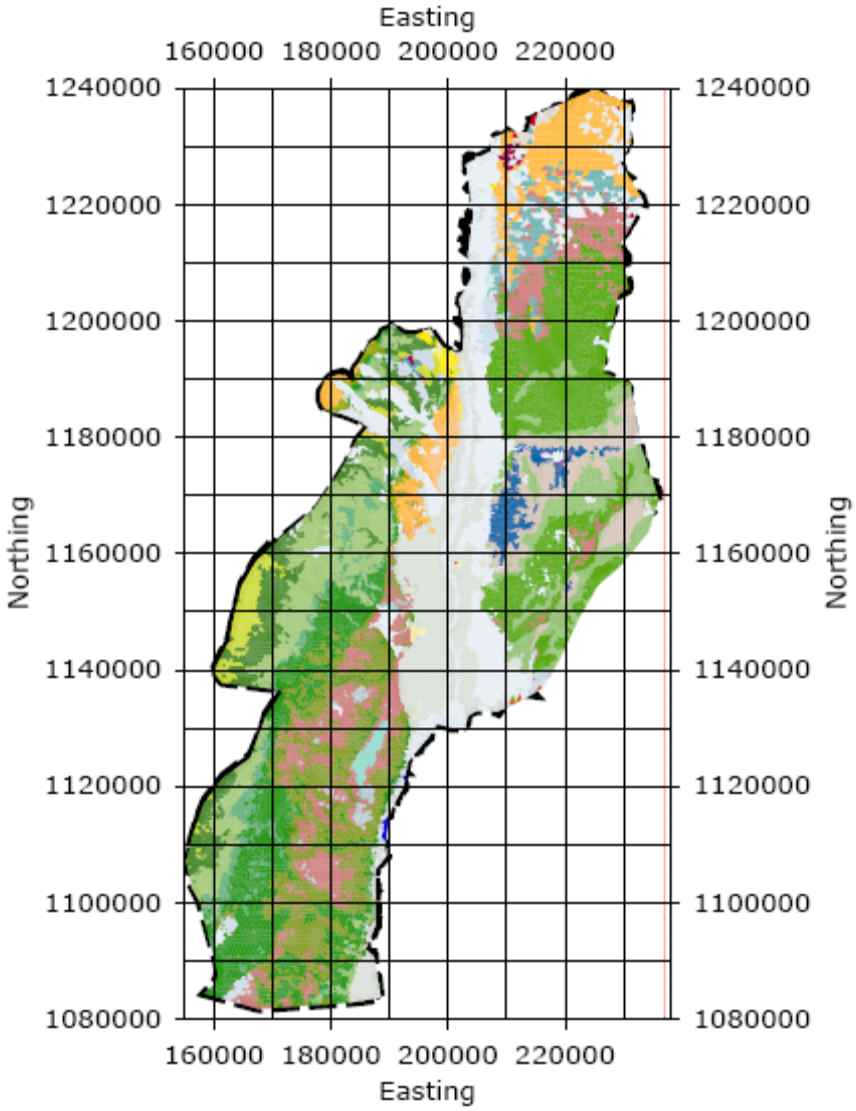


Figure 6: Model outcrops

5. SUMMARY

This paper describes the process of development of a three-dimensional computational model for the simulation of groundwater flow processes in the water bearing layers of the LJRV. The model has been prepared on the basis of a significant effort of conceptualization and has undergone substantial consistency checks. The model is now in the process of being used for the quantification of large scale flow processes in the LJRV, for the evaluation of the global replenishment rates, for the evaluation of global climate change impacts on the LJRV hydrologic balance and for the evaluation of alternatives for groundwater utilization.

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NON-PARAMETRIC TREND ANALYSIS OF PRECIPITATION DATA IN PIERIA REGION (GREECE)

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Abstract: Precipitation has been widely considered as a starting point towards the apprehension of climate changes courses, constituting one of the most substantial components of the hydrologic cycle. The purpose of this work is to investigate the temporal variability of precipitation in order to contribute to a better interpretation of the hydrological status of the study area located in Northern Greece within the prefecture of Pieria. The Aison River basin drains the water of its central part, forming the greatest receiver of surface water. The precipitation data used refer to the period 1974-2007.The methodology adapts various statistical approaches in order to detect possible precipitation changes in annual, monthly and seasonal basis, including both tests for monotonic trend (Mann-Kendall test, Sequential version of the Mann-Kendall test, Sen's estimator of slope) and a test for step change (the distribution-free CUSUM test). The application of this trend analysis framework resulted in the identification of a few trends, occurred mainly at smaller than annual time scales. An overall downward trend, even though no statistically significant, has derived for the stations located at lower altitudes within the agricultural zone of Pieria. Regional (River Aison basin scale) total precipitation series has also displayed trends similar to those for the majority of stations. Among the four seasons, a noticeable decrease of precipitation is observed in Spring by both ways of analysis (station based and regional).

Key words: precipitation, trend analysis, non parametric tests, Pieria Region

1. INTRODUCTION

Trend detection is an active area of interest for both hydrology and climatology in order to investigate climate changes scenarios and enhance climate impact research. The assumption of stationarity, connoting time-invariant characteristics of the time series under consideration, has accrued to be invalid as a result of anthropogenic influences and the natural features of the climate system (Jain and Lall 2000; Xu et al. 2003). Therefore, trend detection in precipitation time series is crucial for planning and designing regional water resources management. Several recent studies on climatologic trends conclude that trend in observed precipitation comprises a complex function of the climatic environment, precipitation intensity and season (Osborn et al. 2000; Ventura et al. 2002). Relevant reviews on trend analysis in precipitation time series include the studies of Lettenmaier et al. (1994), Turkes (1996), Zhang et al. (2000), Xu et al. (2003), Mosmann et al. (2004) and Partal and Kahya (2006).

The purpose of this work is to investigate the variability of precipitation in Pieria Region, located in Northern Greece, by detecting precipitation changes in the temporal structure for the period 1974 to 2007, in order to contribute to a better interpretation of its hydrological status. The precipitation regime features a high seasonal and annual variability in both temporal and spatial domains, providing the incentives for debating on precipitation variability and by extension on climate change.

The present study consists of four sections. The first section introduces the paper and explains the motives of this work. The second section describes the study area, the data employed and the

statistical techniques applied for trend and step change detection. The third section discusses the results accrued, while the fourth one outlines the major findings.

2. METHODOLOGY

2.1 Study area

The study area is located within the prefecture of Pieria in Northern Greece covering an extent of 123200 ha. Its boundaries are delineated by the coast at the eastern side, by Olympus Mountain at the southern and southwestern side, by Pieria Mountain at the western side and by river Aliakmon at the northern side. The climate is characterized as mild Mediterranean with a warm and dry summer period. The hydrological system is divided into three parts: the southern, the northern and the central one. The southern part of the hydrological system has an extent of 2500 ha and consists of small streams, some of them reaching the drainage network of Varico-Litochoro, while others flow directly into the Aegean Sea. The northern part has an extent of 41200 ha and drains at the sea via small streams. The Aison River basin constitutes the central part, covering an extent of 73048 ha and forming the greatest receiver of surface water (Figure 1).

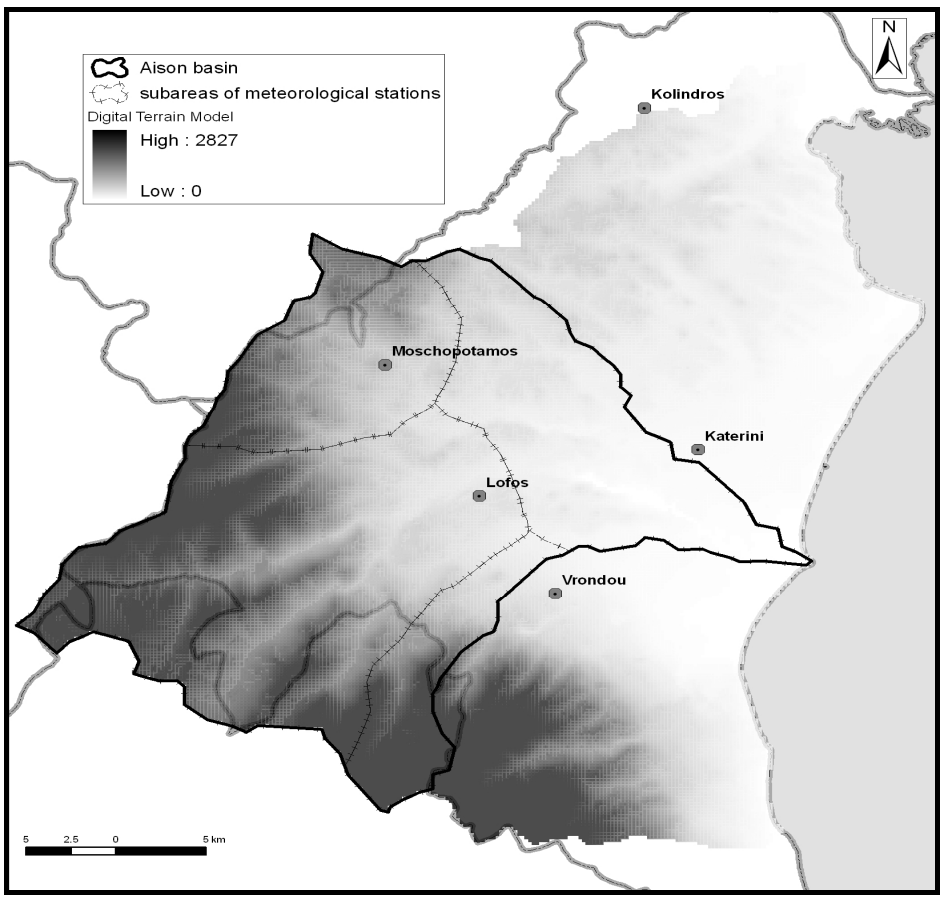


Figure 1. Aison River Basin – Pieria (Greece)

2.2 Materials and methods

The data includes the records of the precipitation stations of Vrontou, Moschopotamos, Katerini, Lofos and Kolindros, with an elevation range from 33 to 516 masl-meters above sea level (Table 1).

The first four above-mentioned stations belong to the Aison basin (Figure 1). The precipitation records include observations spanning from 1974 to 2007 and cover a period over 30 years that could be assumed to be long enough for a valid mean statistic and add up to specify hydroclimatic time series as non stationary with local periods of stationary (Kahya and Kalayci, 2004). Moreover, Burn and Elnur (2002) state that a minimum record length of 25 years ensures statistical validity of the trend results in a climate change research. Precipitation data involve annual totals, four-seasons totals (spring, summer, autumn, winter) and irrigation period totals (May to September) per year. All time series were tested for homogeneity through double mass curve method and the areal precipitation is calculated through a Thiessen based method in a GIS environment. In order to take into consideration seasonality and serial correlation, the different seasons of the year are considered separately. As a result, six precipitation variables at each station and six areal precipitation variables for the Aison Basin were subjected to trend detection analysis. Concerning the annual time series, the hydrological year (October to September) has been taken as reference.

Various distinct approaches, including both tests for trend and step change are adapted in the statistical analysis. It should be noted that a change may occur either gradually as a trend or abruptly as a step change. As many hydrologic time series data are not normally distributed, non parametric tests were considered as more suitable compared to parametric tests, even though they do not quantify the size of the trend identified. Therefore, all tests employed are non-parametric, as this category is generally distribution free. The next two subsections provide a brief presentation of the statistical techniques applied, whereas an analytical presentation of all the statistical procedures and mathematical equations is offered by Karpouzos et al. (2008).

Table 9. Precipitation Stations and Aison Basin data

	Elevation (masl)	Mean annual precipitation (mm)
Vrontou	182	836.7
Lofos	250	824.5
Katerini	33	628.3
Moshopotamos	516	769.5
Kolindros	330	631.3
Aison Basin	735	768.1

2.2.1 Tests for trend

The Mann-Kendall test (M-K test): This method tests if there is a trend in the time series data. It is a non-parametric rank-based procedure, robust to the influence of extremes and suitable for application with skewed variables (Hamed 2008). More particularly, this technique can be adopted in cases with non-normally distributed data, data containing outliers and non-linear trends (Helsel and Hirsch 1992; Birsan et al. 2005). According to this test, the null hypothesis Ho outlines that the deseasonalised data ($x_1 ... x_n$) is a sample of n independent and identically distributed random variables (Hirsch et al. 1982; Kahya and Kalayci 2004). The alternative hypothesis H_1 of a two-sided test is that the distributions of x_k and x_j are not identical for all $k,j \leq n$ with $k \neq j$. The resulting test statistic S indicates an upward trend if its value is positive and a downward trend otherwise.

The sequential version of the Mann-Kendall test: This method is used to test an assumption about the beginning of the trend development within a sample (x1…xm) of the random variable X, based of the rank series R of the progressive and retrograde rows of this sample (Taubenheim 1989; Sneyers 1990; Partal and Kahya 2006). The null hypothesis Ho presumes that the sample under investigation shows no beginning of a developing trend (Gerstengarbe and Werner 1999).

The Sen's estimator of slope: This test is applied in cases the trend is assumed to be linear, depicting the quantification of change per unit time (Sen 1968; Gilbert 1987). De Lima et al. (2005) point out the utility of this estimator in cases there are missing values or other gaps in the data, as it

remains unaffected from outliers or gross errors. The Q_i slope computation corresponds to the slope estimates of N pairs of data and finally, the Sen's estimator of slope Q derives from the N values of Q_i and equals to their median.

2.2.2 Tests for step change

The distribution-free CUSUM test: This non-parametric rank-based method tests whether the means in two parts of a record are different for an unknown time of change. In particular, successive observations are compared with the median of the series in order to detect a change in the mean of a time series after a number of observations (McGilchrist and Woodyer, 1975; Chiew and McMahon, 1993; Kundzewicz and Robson, 2004). The test statistic V_k of a time series data is the cumulative sum (CUSUM) of the k signs of the difference from the median (a series of values of -1 or +1) starting from the beginning of the series.

3. RESULTS AND DISCUSSION

Initially, the autocorrelation test was performed to all time series data for checking randomness ascertained by computing autocorrelations for data values at varying time-lags (Modarres and Da Silva, 2007). As all lag-1 serial correlation coefficients were statistically not significant, there was no need to pre-white the data, and all statistical tests described above are applied to the original time series (Luo et al., 2007).

Precipitation time series data were analyzed with a Mann-Kendall test for each station separately and the river Aison basin as well. Table 2 provides the calculations of the Mann-Kendall statistics and p-values derived for each distinct station and Aison Basin area respectively, according to time series steps described above. As it can be observed, a few trends were distinguished during the study period. More important trends are related to seasonal time series than to annual ones. With respect to irrigation period (May to September), a clear characterization of trend could not be obtained in the total number of the stations data sets. During this period there is a spread of trend over the stations, as one of these (Moshopotamos) has no trend, two of them (Vrontou and Katerini) hold an upward trend and the rest two (Lofos and Kolindros) possess a downward trend.

Concerning the seasonal analysis, the following remarks are noted. Spring reveals a downward trend in almost all stations (except Moshopotamos), which is statistically significant for Lofos. A weak upward trend is obtained for summer in the majority of stations (Vrontou, Moshopotamos and Kolindros). In autumn, three of five stations (Vrontou, Lofos and Kolindros) show a slight downward trend, while in the winter an equivalent upward trend is detected (Lofos, Moshopotamos, Kolindros). The contribution of each season to the total annual precipitation amount in each station, as well as for the whole area of Aison Basin is quite similar for spring, autumn and winter (varying from 25-32% for these seasons), meaning that the annual trend is not easily influenced by a single seasonal trend.

Annual time series did not reveal a statistically significant trend, although a downward one exists in three of five stations (Vrontou, Lofos and Katerini) situated at the lower altitudes. This is consistent with seasonal analysis, where Vrontou and Lofos appear to have three seasons with a downward trend (86.8% and 68.8% of total annual precipitation respectively) and Katerini has two seasons (53% of total annual precipitation) with a downward trend and one season with no trend. Kolindros presents no trend in annual precipitation time series as two seasons (57.5% of total annual precipitation) appear to have a downward trend, while the other two seasons (42.5% of total annual precipitation) show an upward one. On the other hand, Moshopotamos annual series present a weak but upward trend, as it was expected since all four seasons have the same positive sign of trend.

The results of the M-K test for the regional analysis of the River Aison Basin show a fragile downward trend annual time step in accordance to the majority of the stations located in the Aison

Basin. Irrigation period also shows a downward, but not statistically significant trend. Four seasons analysis ends to a weak downward trend taking its highest value during the spring.

Mann-Kendall test results were also checked with Spearman's rank test in order to enhance the reliability of trend outputs. The Spearman's rank test revealed almost the same results with M-K test. Trend direction in all cases was similar to the one detected by M-K test and the confidence intervals were more or less at the same levels. (Karpouzos et al, 2008)

Table 10. Mann-Kendall test results

Precipitation Totals	**Vrontou**		**Lofos**		**Moshopotamos**	
	S statistic	**p-value**	**S statistic**	**p-value**	**S statistic**	**p-value**
Annual (hydr.year)	-70	0.285	-70	0.285	38	0.569
Irrigation period	23	0.741	-51	0.459	1	1
Spring	-111	0.103	**-161***	**0.018***	17	0.81
Summer	35	0.617	-19	0.787	17	0.81
Autumn	-27	0.697	-58	0.395	23	0.741
Winter	-18	0.795	12	0.865	60	0.363
	Katerini		**Kolindros**		**Aison basin**	
	S statistic	**p-value**	**S statistic**	**p-value**	**S statistic**	**p-value**
Annual (hydr.year)	-38	0.569	0	1	-58	0.379
Irrigation period	26	0.711	-23	0.741	-29	0.674
Spring	-81	0.234	-87	0.204	-107	0.116
Summer	-7	0.928	17	0.81	13	0.857
Autumn	27	0.697	-11	0.881	-9	0.904
Winter	-30	0.653	52	0.43	-2	0.984

**statistically significant at 95% confidence level*

The most interesting results of the sequential version of the Mann-Kendall test are presented in Figures 2, 3 and 4. The sequential values of the u(t) and u´(t) statistics, both derived from a progressive analysis of a Mann-Kendall rank correlation are depicted respectively by solid and dashed lines respectively. Also, horizontal dashed and solid lines correspond to confidence limits at the 5% and 10% significance levels respectively. In all stations there is an apparent precipitation decrease during the period 1987-1993. Additionally, this decrease, as it can be shown from the Figure 3, was statistically significant at 95% confidence level at Vrontou. The intersection point of u(t) and u´(t) curves indicates the beginning of the decreasing trend (in 1987). Except Vrontou, there is no significant trend detected at all other stations since the graphical representation of u(t) and u´ (t) gives curves that do not point out an intersection point or overlap several times. From seasonal analysis, it has been considered more purposive to illustrate the case of spring trend, which has been found to be statistically significant at Lofos station. As it can be seen by Figures 2-4, the downward trend in spring appears very recently and the locations of each beginning can not be definitely assigned, although the start of the trend seems to be around the year 2003. However, the start position of this trend will be determined more precisely by taking into account the forthcoming years.

Sen's estimator of slope, following the Mann-Kendall test, was employed to figure out the change per unit time of the trends observed in all precipitation time series. Details are presented in the following table (Table 3), where a negative sign represents a downward slope and a positive sign indicates an upward one. At seasonal basis, a statistically significant downward slope identified at Lofos (Mann Kendall test) during spring season approximates 3.461 mm/ hydrologic year (Figure 5). As it concerns Aison Basin area, the total annual precipitation decreases slightly by 3.710 mm per hydrological year (Figure 5).

The distribution-free CUSUM test was used in order to test the hypothesis concerning the existence of a step (shift) change in precipitation time series during the period of study. The results derived on annual precipitation time series are presented in Figures 6, 7 and 8. As it can be observed from all figures, the maximum values of the statistic V_k have not reached the critical values at 90% confidence level, signifying that there is no statistically significant step change in all the tests applied. However, Vrontou approximates the critical value of 90% confidence level at 1986 and if a

step change were accepted in 1986, the period before the jump (1975-1986) would be characterized by an upward trend, whereas the period after 1986 (1986-2007) would feature a downward trend (Figure 6).

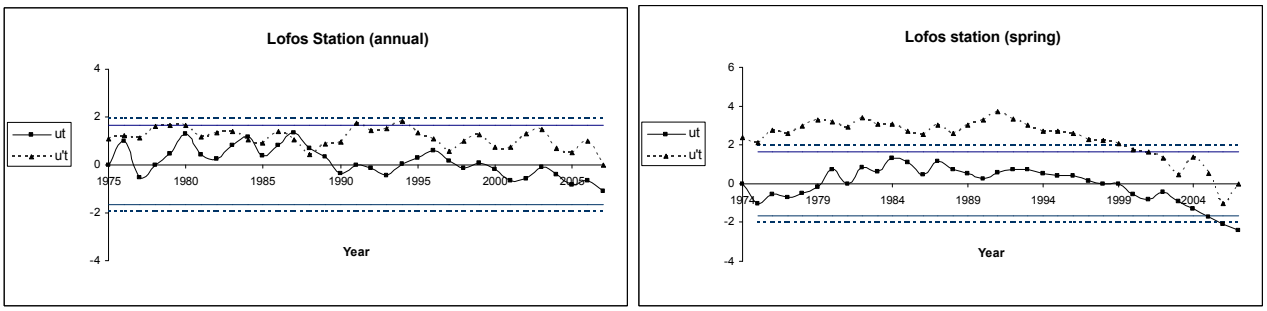


Figure 2. Sequential values of the statistics u(t) and u´(t):Annual and spring precipitation totals at Lofos

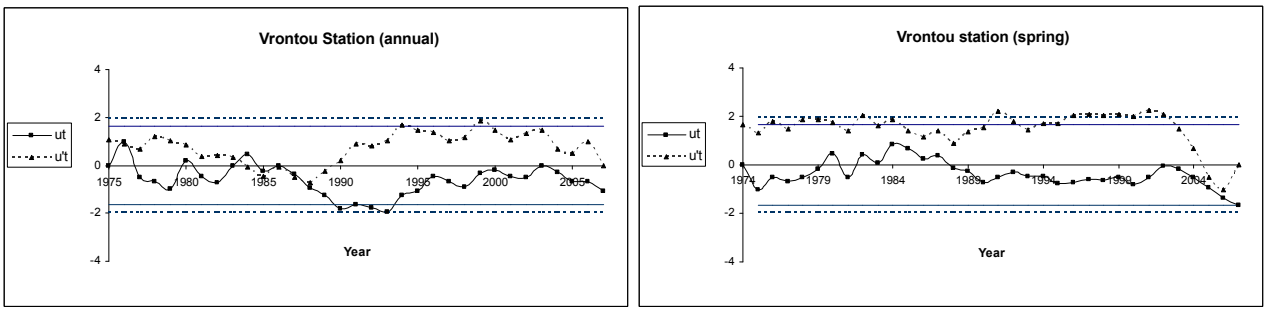


Figure 3. Sequential values of the statistics u(t) and u´(t):Annual and spring precipitation totals at Vrontou

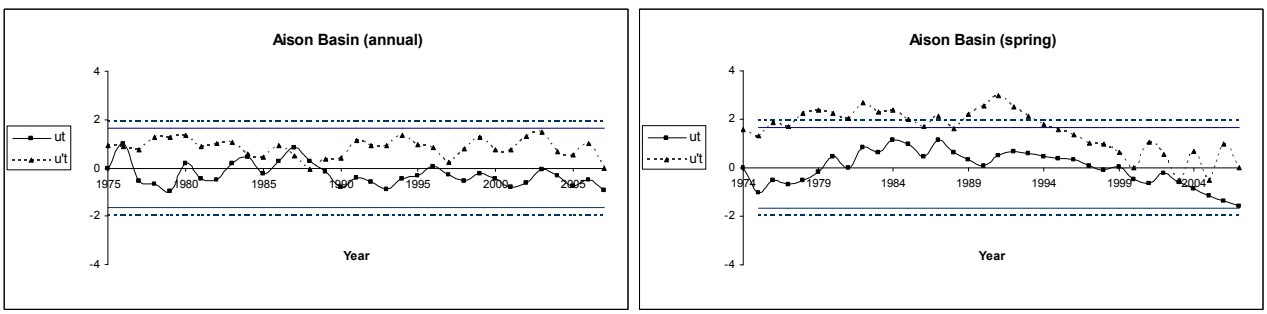


Figure 4. Sequential values of the statistics u(t) and u´(t):Areal annual and spring precipitation totals

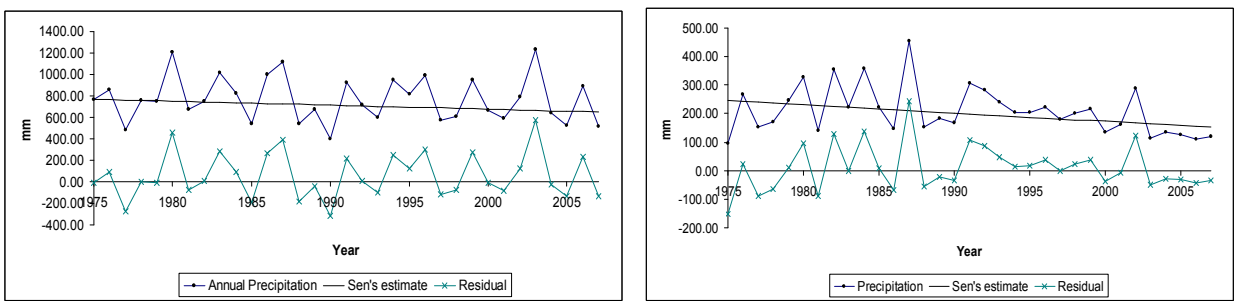


Figure 5. Sen's linear estimate: annual precipitation totals at Aison Basin (left) and Spring Precipitation at Lofos Station (right)

A decreasing short-term trend can be identified in all cases during the period 1987-1993 in a similar way to the Sequential Mann-Kendall test. This decrease is more apparent at low altitude located

stations (such as Vrontou, Lofos and Katerini) than the one observed at higher altitude located stations (such as Moshopotamos and Kolindros). An overall illustration of the regional analysis is provided in Figure 8, where it is shown that this decrease also concerns the whole area of Aison River Basin.

Table 3. Sen's slope estimate (mm/period)

Time series	Vrontou	Lofos	Moshopotamos	Katerini	Kolindros	Aison Basin
Annual	-6.968	-5.23	2.328	-2.037	-0.027	-3.71
Irrigation period	0.529	-1.114	0.03	0.506	-0.386	-0.428
Spring	-2.129	**-3.461***	0.65	-1.375	-1.54	-1.943
Summer	0.4	-0.165	0.4	-0.11	0.188	0.125
Autumn	-0.741	-1.767	0.767	0.6	-0.571	-0.6
Winter	-1.146	0.538	1.746	-0.94	0.911	-0.142

**statistically significant at 95% confidence level*

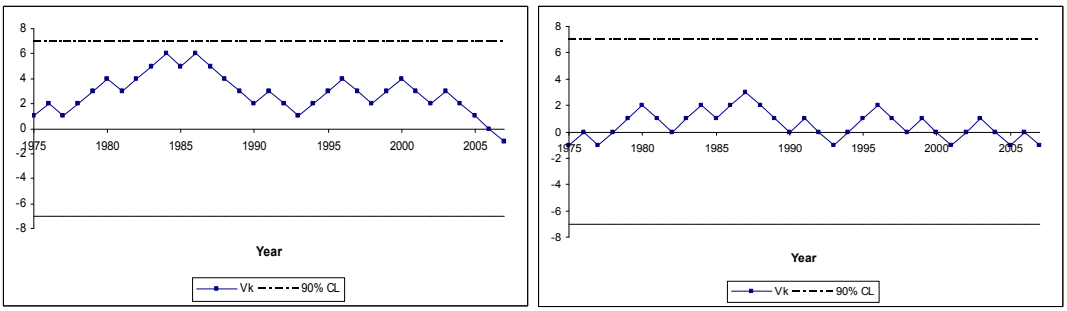


Figure 6. Distribution Free CUSUM chart at Vrontou(left) and Lofos(right) for annual precipitation totals

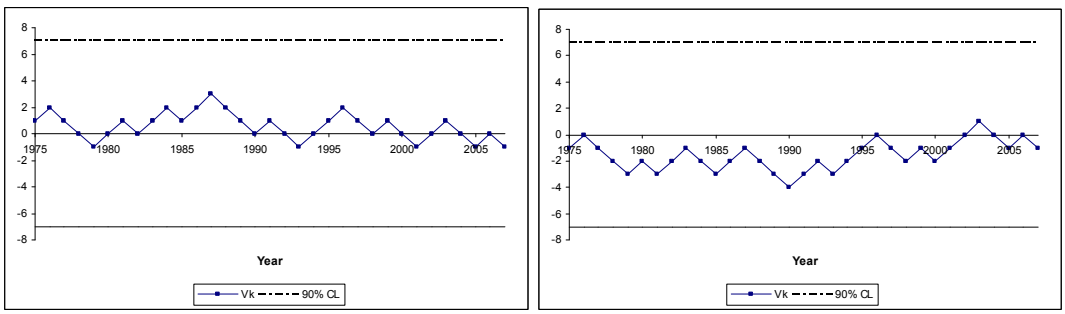


Figure 7. Distribution Free CUSUM chart at Katerini(left) and Moshopotamos(right) for annual precipitation totals

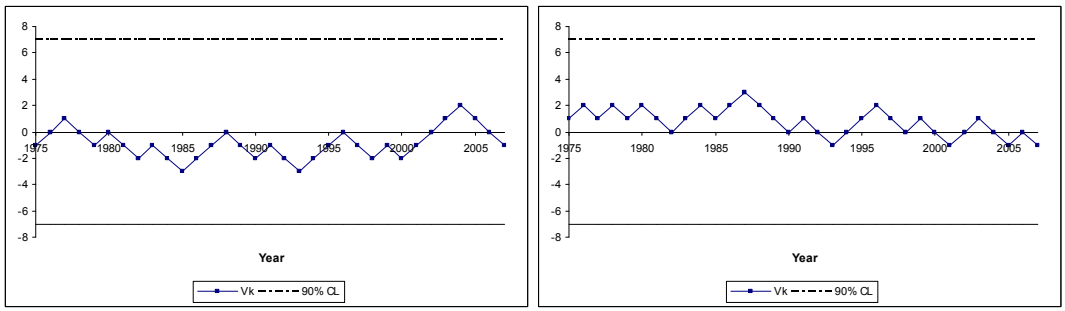


Figure 8. Distribution Free CUSUM chart at Kolindros(left) and Aison Basin(right) for annual precipitation totals

4. CONCLUDING REMARKS

The application of this trend analysis framework revealed an overall downward trend, even though no significant, for the stations located at lower altitudes within the agricultural zone of

Pieria. In all stations a precipitation decrease is identified during the years 1987-1993. Regional (River Aison basin) precipitation analysis has also displayed trends similar to those for the majority of stations. Among the four seasons, a noticeable decrease of precipitation occurred in spring in both station based and regional analysis. However, this decrease was found to be statistically significant at 95% confidence level at only one station. Finally, a statistically significant step change of precipitation was not identified during the study period in both ways of analysis.

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A CONTRIBUTION TO THE STUDY OF THE PHENOMENON OF HORIZONTAL INFILTRATION

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Abstract: In this work an attempt is made to compare experimental soil moisture profiles for a horizontal infiltration experiment (Poulovassilis, 1977) with calculated profiles obtained variously through the use of the computation code of HYDRUS 1D and through the semi-analytical solution of the appropriate diffusion equation when the soil water diffusivity is obtained directly from the experimental data. Necessary input data for running HYDRUS 1D are obtained in three different ways: First, the experimental data of the main wetting branch of the moisture retention curve (MRC) coupled with Ks ,the hydraulic conductivity at saturation, are used, second the predicted, according to the Parlange model (1976) main wetting branch of the MRC, when experimental data points of the main drying branch of the MRC are used and third the predicted, according to the Mualem model (1977) main wetting branch of the MRC, again when experimental data points of the main drying branch of the MRC, are used. In the previous three cases predicting appropriate hydraulic conductivity K(θ) relationship is obtained through the model of Mualem (1976). The comparison of the above soil moisture profiles leads to some useful comments and conclusions.

Key words: Horizontal infiltration, soil moisture profile, moisture retention curve, hydraulic conductivity simulation.

1. INTRODUCTION

Horizontal infiltration of water into the soil mass may be applied only under certain conditions in practicing irrigation, nevertheless it is thoroughly studied (Philip, 1955, 1957 a,b) since through this, the more complex and widely used process of vertical infiltration in irrigated agriculture (either by applying flooding or sprinkling) has become tractable and in depth analyzed. Of course the only difference between the two processes is the incorporation of gravity for the vertical infiltration, which makes the problem more complicated and not readily amenable to analytical solutions. The partial differential equation which governs horizontal infiltration is

$$\frac{\partial\theta}{\partial t}=\frac{\partial}{\partial x}\left[D(\theta)\frac{\partial\theta}{\partial x}\right] \qquad (1)$$

Where θ (L^3/L^3) is the volumetric soil moisture, t (T) is the time, x (L) is the distance and D (L^2/T) is the soil moisture diffusivity. The soil moisture diffusivity may be obtained (Childs and Collis-George, 1950) according to the equation

$$D(\theta)=K(H)\frac{\partial H}{\partial\theta} \qquad (2)$$

as a product of the hydraulic conductivity K and the reverse slope of the moisture retention curve (MRC), where H(L) is the matric head and θ(H) is the MRC, which one could determine in the laboratory. One should note that in doing this, always has to follow the same wetting (or drying) scanning curve of the MRC in order to comply with the hysteretical nature of both curves K(H) and θ (H). The determination of K(H) or K(θ) in the laboratory or in situ is a rather difficult and cumbersome endeavor, and this is the reason for the development of various conceptual models through which one may infer K from the MRC and K_s. (Childs and Collis George (1950), Burdine (1953), Marshall (1957), Jackson (1972), Mualem (1976)).

The solution of equation (1), $\theta=\theta(x,t)$ could be obtained only under certain initial and boundary conditions. In the experimental set-up (Poulovassilis, 1977) these were:

$t=0, \quad \theta=\theta_{in}, \quad x\geq 0$ (3)

$x=0, \quad \theta=\theta_s, \quad t>0$ (4)

$x\rightarrow\infty, \quad \theta=\theta_{in}, \quad t>0$ (5)

The imposition of a uniform initial θ, at t=0, $\theta=\theta_{in}$ throughout the column extent is enforced for practical reasons, in order to avoid unnecessary complications due to the phenomenon of hysteresis. Semi-analytical solution to the problem may be obtained through the application of the Boltzmann transformation (Philip 1955). This transformation combines the two independent variables x and t into one, according to the equation

$$\varphi=\frac{x}{\sqrt{t}} \tag{6}$$

which is expected to express θ as a single-valued function of φ, thus changing equation (1) from a partial differential equation to an ordinary one, which could be solved analytically or numerically, if finite differences are employed, achieving a very good volume balance criterion through iterations. From the solution, $\theta=\theta(\varphi)$, one could apply the inverse transformation and obtain the solution in the classical form $\theta=\theta(x,t)$. It is customary in solving equation (1) to ignore the phenomenon of hysteresis which characterizes the soil hydraulic functions θ(H) and K(H). What is usually applied is that experimental data points from the main drying branch of the MRC are used (these data points are rather more easily obtained and no one bothers to obtain experimentally the closed reproducible loop of the MRC) together with the value of K_s. The other necessary input functions K(θ) or D(θ) are calculated through conceptual models. It is, therefore possible, that some serious errors might occur in calculating moisture profiles or the integrated quantity of cumulative infiltration. The objectives of the present work are to investigate the various drawbacks of the above. More specifically we compare the experimental moisture profiles (Poulovassilis, 1977) with: a) The profiles obtained by the semi-analytical solution of Philip, where the appropriate experimental data points of both θ(H) and K(H) i.e those referring to the main wetting branches are used. b) The profiles obtained from the MRC experimental data points of the wetting branch and the K(θ) calculated from the K_s- value applying the conceptual model of Mualem (1976). In this case all necessary calculations were performed numerically through HYDRUS 1D model. c) The profiles obtained by using MRC experimental data points of the main drying branch, calculating next the main wetting branch of the MRC according to Parlange model (1976) and finally following (b) above assuming as experimental data points of MRC wetting branch, those obtained from the Parlange model (again HYDRUS ID is employed). d) The profiles obtained as above (case c), but using Mualem model (1977) to get the main wetting branch of θ(H). The comparisons above indicate the differences among the various empirical models used and the overall effects these exert on the final results, which are the moisture profiles and the cumulative infiltration versus time relationship.

2. THEORETICAL CONSIDERATIONS

2.1 General

The HYDRUS 1D program solves numerically in the finite differences scheme the well known equation of Richards, either in the horizontal or the vertical infiltration cases. The equation is written below for the horizontal case

$$\frac{\partial q}{\partial t}=\frac{\partial}{\partial x}\left[K\left(\frac{\partial H}{\partial x}\right)\right] \tag{7}$$

The finite difference scheme is

$$\frac{\theta_i^{j+1,k+1}-\theta_i^{j}}{\Delta t}=\frac{1}{\Delta\chi}\left[K_{i+\frac{1}{2}}^{j+1,k}\frac{H_{i+1}^{j+1,k+1}-H_i^{j+1,k+1}}{\Delta\chi_i}-K_{i-\frac{1}{2}}^{j+1,k}\frac{H_i^{j+1,k+1}-H_{i-1}^{j+1,k+1}}{\Delta\chi_{i-1}}\right]$$

where $\Delta t=t^{j+1}-t^{j}$ $\Delta\chi=\frac{\chi_{i+1}-\chi_{i-1}}{2}$ $\Delta\chi_i=\chi_{i+1}-\chi_i$, $\Delta\chi_{i-1}=\chi_i-\chi_{i-1}$, $K_{i+\frac{1}{2}}^{j+1,k}=\frac{K_{i+1}^{j+1,k}+K_i^{j+1,k}}{2}$,

$K_{i-\frac{1}{2}}^{j+1,k}=\frac{K_i^{j+1,k}+K_{i-1}^{j+1,k}}{2}$.

Indices i and j denote space and time respectively while k refers to iterations. Time and space steps Δt and $\Delta\chi$ are selected in such a way as to ensure stability and to secure a satisfactory mass or volume balance criterion, especially at the very early stages of the phenomenon. (More details are provided, Simunek et.al., 1998). Input functions to the HYDRUS 1D code are θ(H) according to van Genuchten (1980) analytical expression and K(θ) or $K(S_e(\theta))$ (Mualem 1976)

$$\theta(H)=\theta_r+\frac{\theta_s-\theta_r}{\left[1+|\alpha H|^n\right]^m} \tag{8}$$

$$K(\theta)=K_s S_e^{0,5}\left[1-\left(1-S_e^{\frac{1}{m}}\right)^m\right]^2 \tag{9}$$

where α,n,m are fitting parameters

θ_s = volumetric moisture at saturation

θ_r = residual volumetric moisture

$m=1-\frac{1}{n}$, $S_e=\frac{\theta-\theta_r}{\theta_s-\theta_r}$ (degree of saturation or effective saturation)

The parameters α,n and θ_r were obtained with the help of RETC code (van Genuchten et. al., (1991)

2.2 Parlange and Mualem models

Both of these models are predictive models of the main wetting branch of the hysteretic MRC, based on experimental data points of the main drying branch of the MRC.

2.2.1 Parlange model

The calculation of the main wetting branch of the MRC is based on the expression

$$\theta(H,H_1,H_2)-\theta_2=(H-H_1)\int_H^{H_2}\left[\theta(H,H_1)-\theta_2\right]/(H_1-H)^2\,dH \tag{10}$$

where H_1 is the matric head from which the initiation of the boundary drying branch takes place. H_2 is the value of the matric head at which reversing of the process takes place i.e the wetting process begins and H is the matric head, for which the value of θ (i.e during the wetting process for $H>H_2$) is sought. θ_2 denote the corresponding moisture at H_2. It is obvious that if H_1 denotes the matric head value corresponding to the initiation of the drying process, after saturation, then this value corresponds to the initiation of the main drying branch of MRC. Similarly, if $H_2\to\infty$ then equation (10) would predict the main wetting branch of MRC.

2.2.2 Mualem model

The prediction of the main wetting branch of MRC according to the Mualem model is obtained by the expression

$$S_w = 1-\left(1-S_d\right)^{1/2} \tag{12}$$

where

$$S_d = \frac{\theta-\theta_{min}}{\theta_u-\theta_{min}} \tag{13}$$

where subscripts w and d at the symbol of effective saturation S denote wetting and drying values respectively. $\theta, \theta_{min,}\ \theta_u$ denote the real value of moisture, the minimum and the maximum values of θ on the reproducible main hysteretic loop of MRC.

3. MATERIALS AND METHODS

The experimental θ(H) and K(H) relationships corresponding to the main wetting and drying branches, together with the moisture profiles for the horizontal infiltration experiment at times t=18,43,78,159,378 and 624 sec refer to a porous medium of sand. Details for the execution of the experiments and the data acquisition procedures are given in Poulovassilis (1977). The numerical code RETC of van Genuchten et.al., (1991) was employed for the estimation of the various fitting parameters α,n,θ_r to the analytical expressions of MRC according to the model of van Genuchten (1980). Similar methodology was followed for the determination of the best fit respective parameters, for the main wetting branch, obtained through the application of Parlange (1976) or Mualem (1977) models from the experimental data of the main drying branch of the MRC. To the residual moisture θ_r, the minimum value of measured θ was imposed $\left(\theta_r=\theta_{min}=0.06\ cm^3/cm^3\right)$. This, since we are referring to three different main wetting branches, the value of $\left(\theta_r=\theta_{min}\right)$ does not correspond to the same value of H. The initial condition for the application of HYDRUS 1D was set at $H=-50.5\,cm$ and in this respect the above value corresponds to slightly different θ_{in} values. These were for the Parlange model $\theta_{in}=0.072\ cm^3/cm^3$, for the Mualem model $\theta_{in}=0.071\ cm^3/cm^3$.

4. RESULTS AND DISCUSSION

In the figures 1-3 the experimental results for the main drying and wetting branches of the MRC and the respective branches of the hydraulic conductivity, K(H) function are given together with the predictions of the main wetting branch of the MRC (figure 3) obtained through the application of the conceptual models of Parlange (1976) and Mualem (1977). In table 1 the values of the parameters of the van Genuchten analytical expression of the MRC, are shown for the experimental data points curve-fit together with those obtained for the main wetting branch, as this was obtained according to the Parlange and Mualem models. The coefficient of determination (R^2) values are indicative of the quality of the fit.

Table 1: The values of the parameters of the van Genuchten MRC, obtained according to a curve-fitting procedure and the respective parameters for the main wetting branch, as this was obtained according to the Parlange and the Mualem models. The coefficient of determination R^2 is also shown.

	Main drying branch	Main wetting branch	Main wetting branch by Parlange	Main wetting branch by Mualem
α	0.0271	0.0475	0.0498	0.0312
n	8.8130	4.8920	4.2100	7.8945
m	0.8860	0.7956	0.7624	0.8730
θ_s	0.2840	0.2840	0.2840	0.2840
θ_r	0.0600	0.0600	0.0600	0.0600
R^2	0.9920	0.9970	0.9940	0.9910

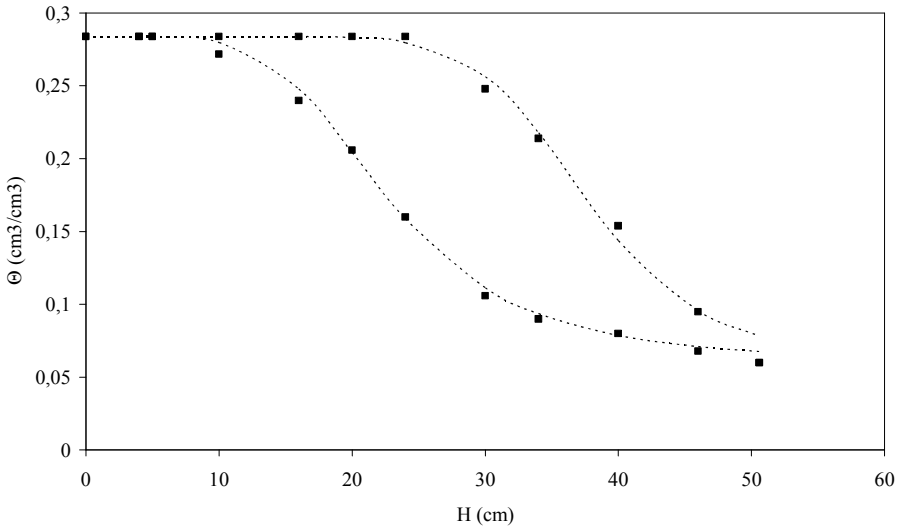


Figure 1: Experimental data points (Poulovassilis, 1977) for the main branches of the MRC (Drying and Wetting). Dotted lines represent MRC main branches obtained by curve-fitting of the data points to the analytical θ(H) expression of van- Genuchten (equation 8).

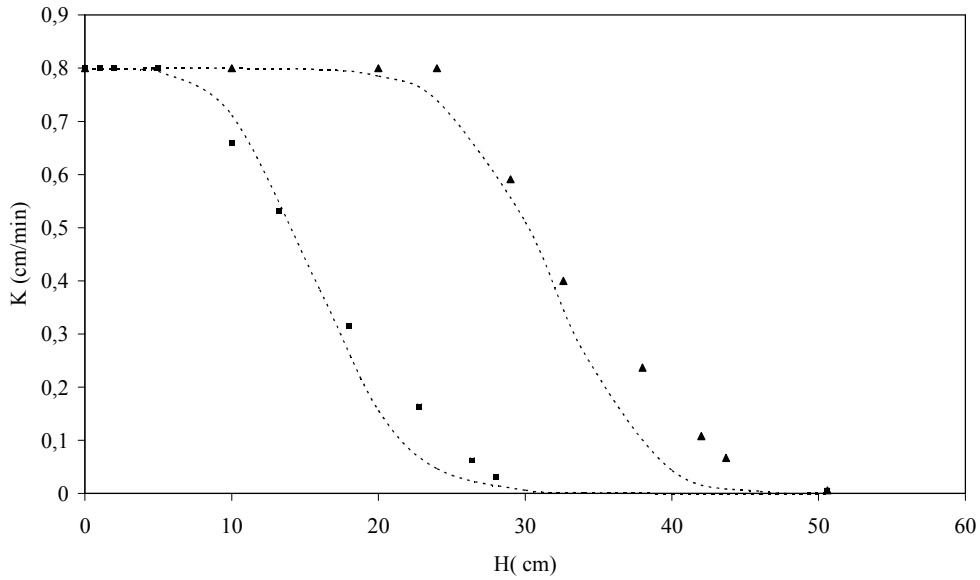


Figure 2: Experimental data points (Poulovassilis, 1977) for the main (Wetting and Drying) branches of the K(H) hysteretic function. Dotted lines represent the main K(H) branches obtained by curve-fitting of the data points to the analytical model (equation 9, van Genuchten- Mualem model)

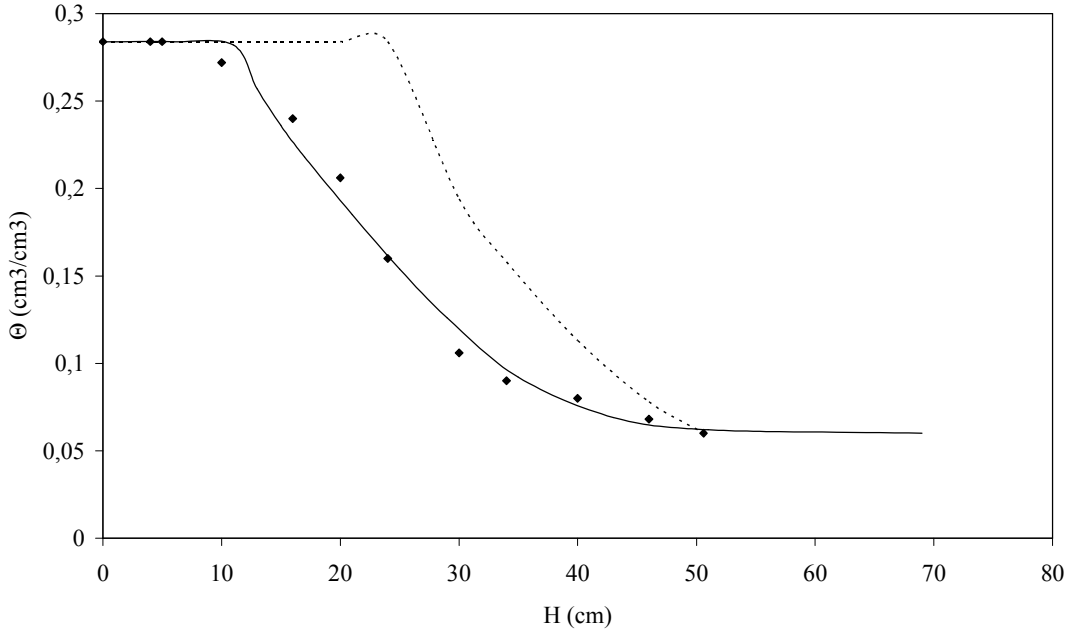


Figure 3: Experimental data points of the main wetting branch of MRC. Solid line represents Parlange (1976) main wetting branch, dashed line represents Mualem (1977) main wetting branch.

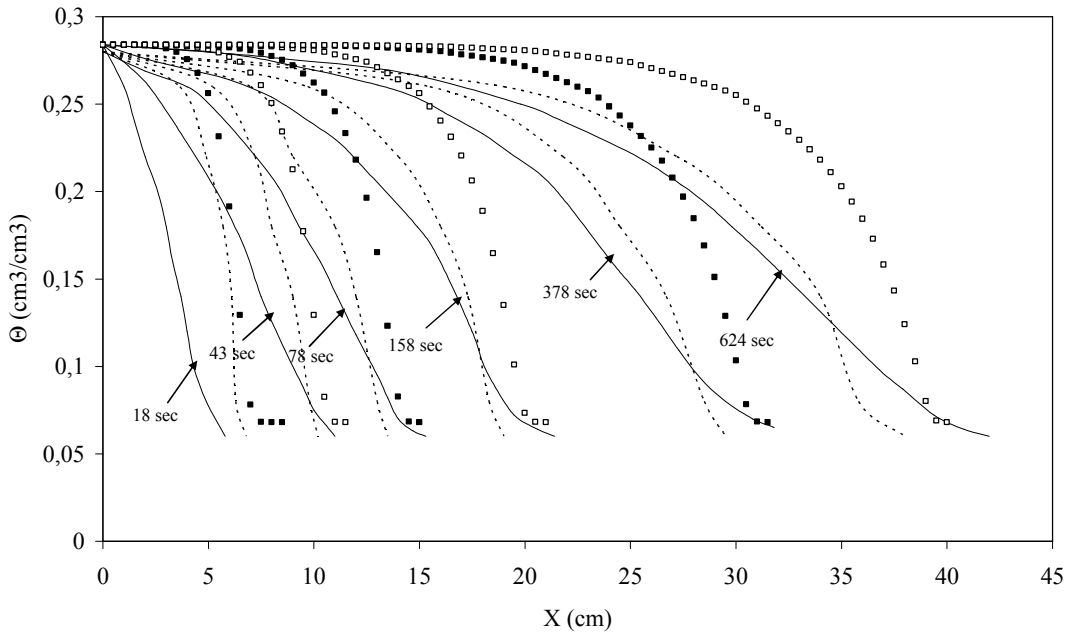


Figure 4: Experimental moisture profiles at six different times (solid lines). Dashed lines represent moisture profiles obtained by the semi-analytical solution of equation (1) and the points represent HYDRUS 1D simulation results where the experimental data points of the main wetting branch of the MRC together with K_s are used.

From figure 2 it is obvious that the van Genuchten-Mualem model predicts adequately the K(H) relationships (wetting and drying branches) except for some underestimated values in the region of relatively large values of H. From figure 3 it is obvious that the Parlange model predicts rather satisfactorily the main wetting branch from the experimental data of the main drying branch in contrast to the model of Mualem which fails to do so for the porous body of sand. In figure 4 the experimental moisture profiles $\theta(x,t)$ are shown together with the calculated ones, which have been obtained according to the solution of equation 1, where the diffusivity function $D(\theta)$ was estimated from the appropriate MRC (main wetting branch) and the main wetting K(H) branch, according to equation 2. Also shown are the results obtained by HYDRUS 1D (i.e through the

solution of the Richards equation (7)) where the experimental main wetting branch of the MRC was used together with the K_s value to obtain the other necessary K(H) wetting branch through the van Genuchten- Mualem model. The experimental soil moisture profiles show that at the early stages of the phenomenon, the advancement of the wetting front, especially for the high values of θ, close to saturation, is relatively retarded in comparison with the calculated profiles. Another aspect which one should mention is that the experimental data do not secure a unique θ(φ) solution and that its shape varies with the running time. In this respect the relationship D(θ), which can be calculated from the experimental moisture profiles will also not be unique, but it will be different relative to the moisture profile from which it is generated. The above could be attributed to a violation of Darcy's law at the very early stages of the phenomenon where exceptionally high values of the hydraulic gradients may prevail (Poulovassilis, 1977). Other researchers have reached similar conclusions (Moutsopoulos and Tsihrintzis, 2005), where the one-dimensional flow of water in a saturated porous medium under a sudden piezometric drop at the surface was investigated. From figure 4 it is also evident that the calculated wetting fronts (semi-analytical solution of Philip and HYDRUS 1D simulation results) are more deeply advanced in all times observed and that they exhibit a Green-Ampt-like shape. More specifically one could note that the two calculated profiles are rather close to each other at t=18 sec. This, changes in larger times with the profiles obtained according to the HYDRUS 1D code showing a more rapid advance. On the contrary one should observe that the calculated profiles obtained by the semi-analytical solution, at larger running times gradually come closer to the experimental ones.

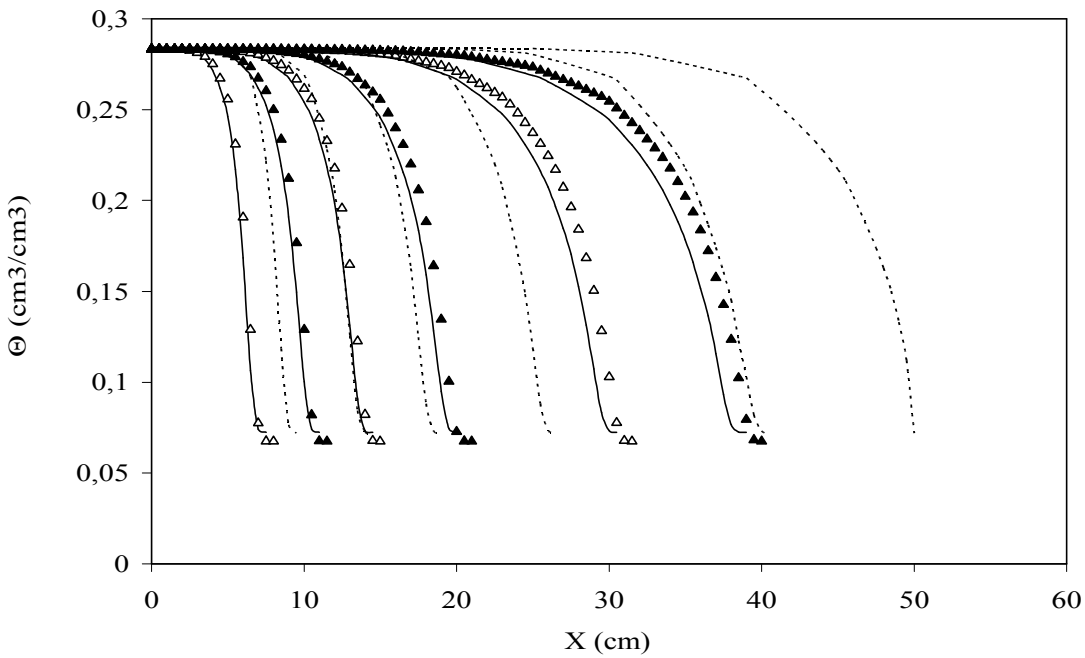


Figure 5: Comparison of the calculated profiles at six different times. Points represent the HYDRUS 1D simulation results where the experimental main wetting branch was used. Solid lines are the HYDRUS 1D simulation results where the experimental data points of the main drying branch of the MRC were used to infer the main wetting branch of the MRC according to the Parlange model (1976), and dotted lines are similar to solid lines, except that for the main wetting branch of the MRC the Mualem model (1977) was used.

From figure 5 it is obvious that the calculated profiles which are obtained from θ(H) experimental main wetting branch of the MRC and those obtained according to the model of Parlange (1976) are very close to each other, except for some relatively minor deviations at larger times. Contrary to the above, the moisture profiles obtained from the calculated main wetting branch of the MRC according to the Mualem model (1977) seem to advance faster than the other two profiles at all times. This failure is obviously due to the fact that the prediction of the main wetting branch of the MRC according to the model of Mualem is far from being adequate, as one may observe from figure 2. The calculated profiles (Parlange and Mualem, solid and dashed lines)

exhibit a close to Green- Ampt shape, something which might be expected for the case of sand. Generally one should underline that the differences among the experimental and calculated profiles, except those obtained according to the model of Mualem which present an overall failure, were rather large at large moisture contents. Moreover, one should mention that in all runs of HYDRUS 1D code the mass or volume balance criterion was exceptionally fulfilled. Another point one should mention is that in all runs the time steps, Δt, were kept similar, starting with small values (Δt=0.1 sec at the very early stages to increase to 0.5 sec later on, always securing accuracy and stability of the calculated profiles). Similar to the above were the conclusions reached by Elmaloglou and Kerkides (1983) in their investigation for a Yolo light clay, where various arithmetic schemes were applied for the solution of the problem of horizontal infiltration. They also noticed that the advancement of the wetting front was faster in relation to the semi-analytical solution of Philip (1955).

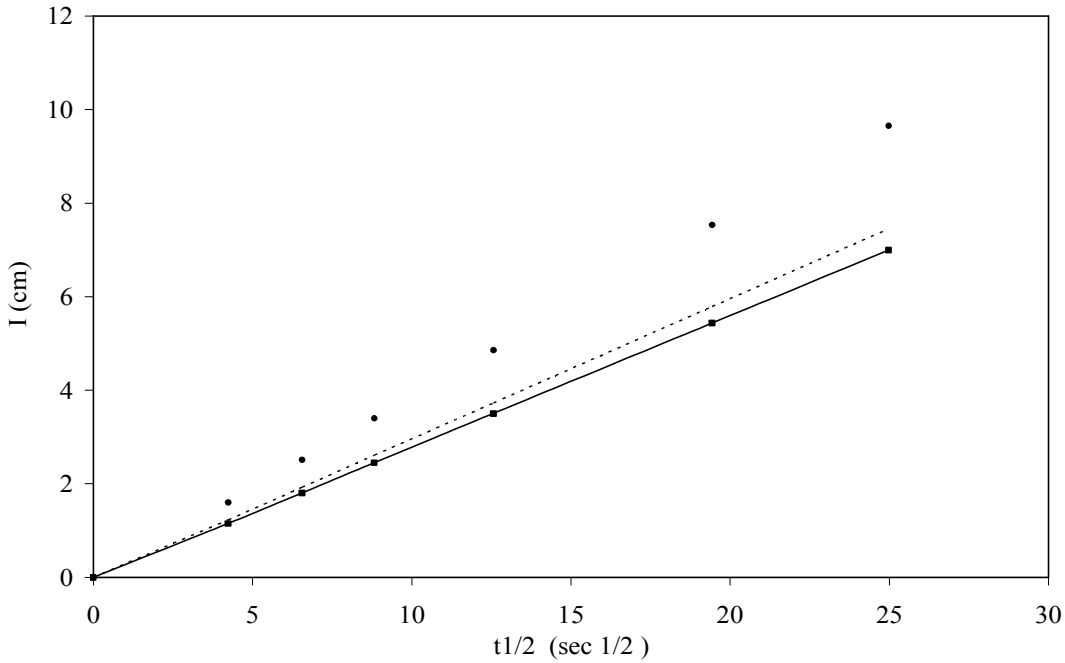


Figure 6: The cumulative infiltration versus square root of time $i\left(\sqrt{t}\right)$ relationship. Dotted line represents the simulation results (HYDRUS 1D) when the experimental data points of the main wetting branch of the MRC together with K_s are used, points represent the $i\left(\sqrt{t}\right)$ when the main wetting branch of the MRC is predicted from the experimental data points of the main drying branch of the MRC according to the model of Mualem and the solid line when Parlange model is used for the prediction of the main wetting branch of the MRC.

From figure 6 it is obvious that all solutions obtained by the HYDRUS 1D are represented by a straight line which passes through the origin, thus, fulfilling the requirements imposed by the Boltzmann (1894) transformation, but they do not give the same sorptivity value, since their slopes are different.

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TOWARDS RATIONAL WATER MANAGEMENT AND UTILIZATION OF WATER RESOURCES IN ARID ZONES: APPLICATION TO THE INTEGRATED MANAGEMENT OF WATER RESOURCES OF VARIOUS QUALITIES IN THE JORDAN RIFT VALLEY

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Abstract: The Jordan Rift Valley (JRV) is an arid zone with precipitations of 50 to 150 mm/year and potential evaporation of up to 2,600 mm/year. The main sources of fresh water are the relatively deep aquifers. These are naturally replenished by the rainfall falling in the surrounding mountains in the east (Jordan) and West (Israel and the Palestinian Authority).

Geologically, the JRV is an extensive geo-tectonic feature bordered by a deep-seated fault system that creates an elongated and narrow basin and acting as a drainage basin for groundwater originating both from the eastern and the western mountain aquifers that border the Rift basin as well as ephemeral streams and floods generated in these mountains. In this system, freshwater entering the aquifer from the outcrops in the Western and Eastern edges reaches the valley and mixes with the local saline water bodies. The management of the JRV water resources is rather complex, as saline water bodies and fresh water bodies coexist in the aquifers with a very delicate equilibrium between them. Inadequate exploitation of the fresh water sources may induce the migration of the saline water bodies and a rise in the salinity of the freshwater wells. On the other hand if the fresh water entering the aquifer is not captured before it mixes with the saline water bodies, substantial amount of fresh water that could be otherwise utilized for water supply could be lost. The main economic activity in this region is agriculture and its development critically depends on the availability of water for irrigation. The region already suffers from water scarcity and sustainability of the water resources utilization seems impossible under the actual demand situation. The future development depends on reliability of the water supply (quantities and qualities). In additional, the area is geographical isolation from areas with more favourable hydrologic conditions or large desalination centres (along the Mediterranean coast). Therefore, the development in this area primary depends on the local water sources. The water policy is to shift fresh water form agricultural uses to domestic purposes as a result of urban/rural development. The only way to reach sustainability is by developing the available sources (fresh, brackish and sewage) and integrating their utilization. The ever increasing demand for freshwater and to some extent brackish water requires proper management tools and by combining utilization of the freshwater and saline water sources. Proper allocation of the water resources within a wider frame of integrated water resources management (IWRM) which considers that waters of different qualities are allocated to a variety of sectors allows the optimization of the water exploitation scheme on one hand and the preservation of the aquifers on the other. In this study we regard all the existing waters sources (freshwater, brackish, saline water and effluent) as valuable source of water, which can greatly increase the overall yield. A judicious application of water treatment technologies, water mixing approaches and appropriate agricultural activities, capable of using freshwater, brackish water, treated effluent and mixtures of them for the irrigation of properly chosen crops.

Key words: Arid zone, IWRM, Jordan Rift Valley, Water supply, SMART.

1. INTRODUCTION

The Jordan Rift Valley (JRV) in its lower part is an arid zone with precipitations of 50 to 150 mm/year and potential evaporation of up to 2,600 mm/year. The main sources of fresh water are groundwater from relatively deep aquifers. Their replenishment is by the rainfall falling in the surrounding mountains in the east (Jordan) and West (Israel and the Palestinian Authority).

The main economic activity is agriculture. Its development critically depends on the availability of water for irrigation. Due to the scarcity there is a harsh competition for water uses in domestic,

agriculture and touristic applications. The development in this area prior depends on the local sources and the only way to reach long term sustainability are by developing these sources and integrating their utilization or by adaption of water demand. The existing waters sources are freshwater, brackish, saline water and effluents of treatment plants. All of them defined as valuable source of water, which can greatly increase the overall yield.

Unfortunately the area is geographical separated from the main national system and from the big desalination plants along the Mediterranean coast. This condition obligate the usage of local water sources.

The research on an integrated management approach was concentrated in this study on two pilot areas: the Kaliya area in the western side of the Jordan Valley and the Wadi Shueib area in the eastern side (Fig. 1). The paper presents the hydro-geological conditions, the available water resources of all qualities that exist in these pilot areas and the main water supply issues.

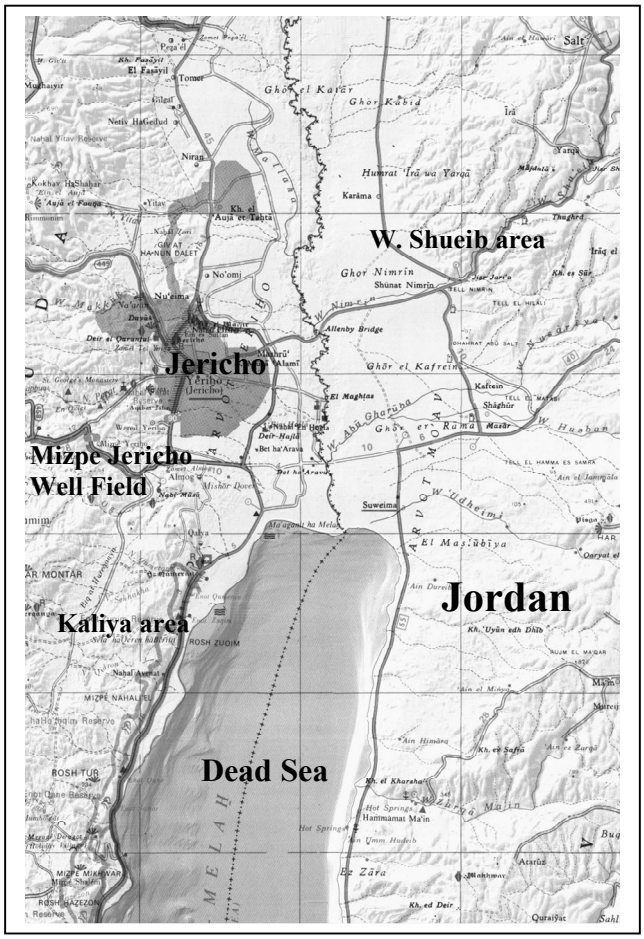


Figure 1: Location map including the Kaliya and Wadi Shueib test areas

2. HYDROGEOLOGY BACKGROUND

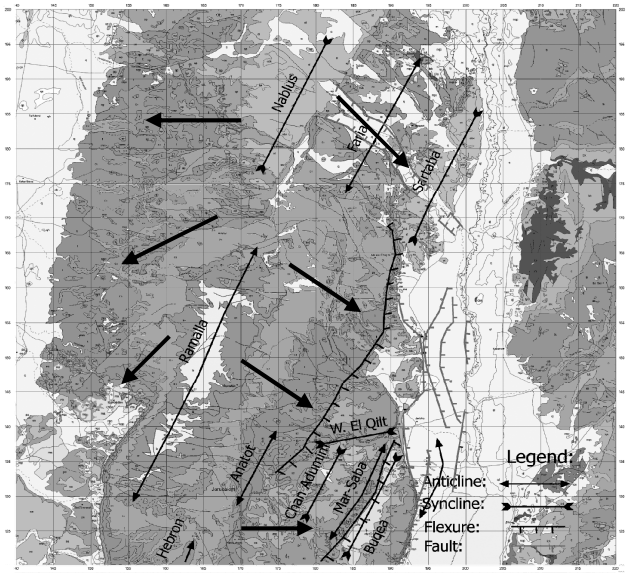
The structural formation of the Jordan and Dead Sea basin with its mountain ranges are the consequences of major geological processes which generated major fold systems, several horizontal and vertical faults systems and a series of pull-apart basins (Hötzl et al. 2007, 2008). There are two basic physiographic provinces; the highlands and the low-lying areas of the Jordan Rift Valley (JRV). A close correlation exists between physiographic features, geological structures and climate both on regional and local scale. Surface drainage of rainwater (surface runoff) has cut steep "canyon-like" wadis into the slope of the escarpments. These wadis generally cut into the rocks in an east-west direction following the overall drainage pattern.

The stratigraphic column on both sides is asymmetric and consists of carbonates, cherts, chalks, gravels, sandstones and evaporates which range in age from Triassic to Holocene (Hötzl et al. 2008). On the eastern side of the Rift the Triassic to Eocene successions are exposed, whereas on the western side, the exposed sequence starts only in the lower Cenomanian beds. Inside the Jordan Rift Valley the alluvial fill is composed of gravels, sand, clay and evaporates.

The Dead Sea-Jordan Rift Valley is the principal fault structure of the area. Along the western side of the Valley (Israel and the Palestinian Authority) the main transform fault runs north-southward along the western shore of the Dead Sea and further to the north. Around Jericho a deviation of the fault orientated to NW is known as the Ein Samiya fault strip (Begin 1976). On the eastern side (Jordan), another important fault deviates from the main rift structure near the village of Suweimeh (NE corner of the Dead Sea) and continues northeastward as the Amman-Halabat fault. Another important fault is the Wadi Shueib structure. These structures are clearly visible in the region and also consist of several faulted synclines and anticlines.

The Ramallah and Hebron Anticlines are the main folding structures in the western side of the valley. The anticlines are asymmetric structures, dipping steeply westwards and moderately eastwards to the Jordan Rift Valley and the Dead Sea forming a series of undulations and secondary structures (Fig.2). The structural elements and especially those that are close to the foothills, have an important impact on the flow regime and salinity on both sides of the Rift Valley (Hötzl et al. 2008).

Fig. 2: Structural elements in the Western side of the Jordan Valley and the groundwater flow directions



On the western side of the Jordan Valley the main aquifer is in the dolomite-limestone layers of the Judea Group (Albian to Turonian age). The recharge areas are along the Ramallah and Hebron anticlines. On the eastern side of the Jordan Valley the main aquifers are the limestone and sandstone layers of the Ram, Zerqa, Kurnub and Ajlun Groups (Triassic to Upper Cenomanian). The Jordan-Dead Sea Valley, which is the drainage base of the surface and groundwater coming from the surrounding mountains, is filled with clastic and evaporitic sediments that accumulated in the evolving basin since the transform movement began during the Miocene (Garfunkel and Ben–Avraham 1996). The Jordan Valley deposits create a multiaquifer system (Ankar, 2007).

The groundwater level in the recharge areas vary between 450 meter above sea level (asl) in Jerusalem area in the western side and around 500 to 600 meter above sea level in Salt area in the eastern side. The water levels descend to the Jordan Valley and the Dead Sea to levels of about 300 meter below sea level (bsl) in the Jordan Valley and about 400 meter bsl on the shore of the Dead Sea. It follows that the difference in the water levels between the recharge area and the aquifer outlets in the Jordan Valley and the Dead Sea are between 500 to 850 m over a distance of only about 20-30 km. This is a very steep flow gradient (assuming that the gradient is uniform). To account for the high water levels in the higher areas of the aquifer and the low water levels in the Jordan Valley, a series of hydrological barriers is required. These low permeability zones (mainly

more impervious layers in the flanks of the anticlines) coerce the groundwater to bypass them and to flow in individual paths towards the outlets in the Jordan Valley and the Dead Sea (via the springs that are located mainly along the western shore of the Dead Sea). Unlike the western side of the Jordan Valley, the geological conditions in the eastern side bring the Jordan Valley formations in contact with the regional aquifers creating a direct hydraulic connection between them and the groundwater flows from the regional aquifers into the Pleistocene layers in the Valley and to the Dead Sea.

The quality of water in the aquifers is variable, with low salinity near the recharge areas and high salinity in the Jordan Valley where the wells are close to salinization sources. The salinization sources are salt layers located in the Jordan Valley deposits; salt domes, deep seated brines and irrigation return flow. Over-pumping in some well fields in the Jordan Valley creates local depression that allows the saline water to flow into the fresh water body and raising their salinity. In some wells there are seasonal salinity fluctuations by tens to hundreds mg/l in parallel to changes in the pumping rate (Fig.3). During high demand periods in which the monthly pumping rate is very high, there is an increase in the salinity, and vice versa. Such behaviour indicates that very close to the radius of influence of these wells there is saline water and that the wells are pumping mixtures of fresh and saline water.

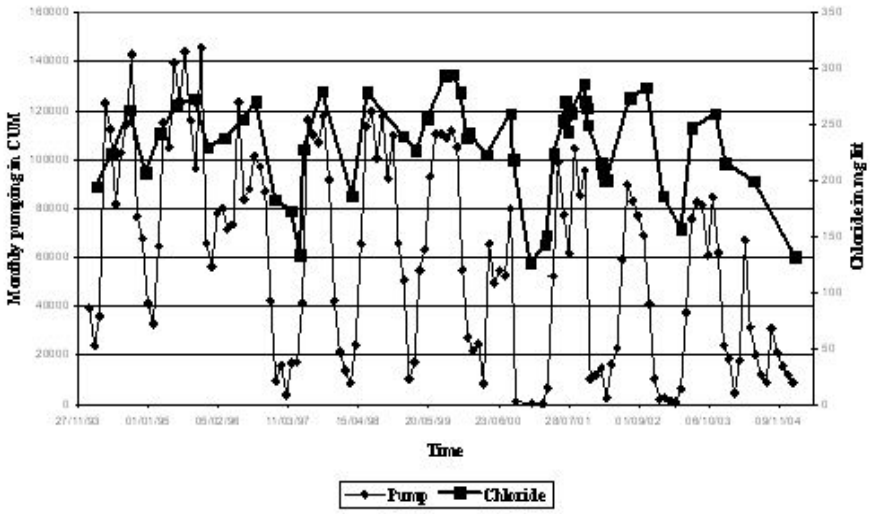


Fig. 3: Correlation between salinity and monthly pumping

From the above description it can be seen that the management of the water resources in the Jordan Valley is rather complex, as saline water bodies and fresh water bodies coexist in the aquifers with a very sensitive equilibrium between them.

Inadequate exploitation of the fresh water sources may induce the migration of the saline water bodies and a rise in the salinity of the freshwater wells. On the other hand if the fresh water entering the aquifer is not captured before it mixes with the saline water bodies, substantial amount of fresh water that could be otherwise utilized for water supply could be lost.

3. WATER SUPPLY ISSUES IN THE PILOT AREAS

Two pilot areas had been chosen. The Kaliya pilot area is located at the western side of the rift valley, along the north-western corner of the Dead Sea and Wadi Shueib pilot area located at the eastern side of the rift valley in the Jordanian side (Fig. 1).

3.1 kaliya pilot area

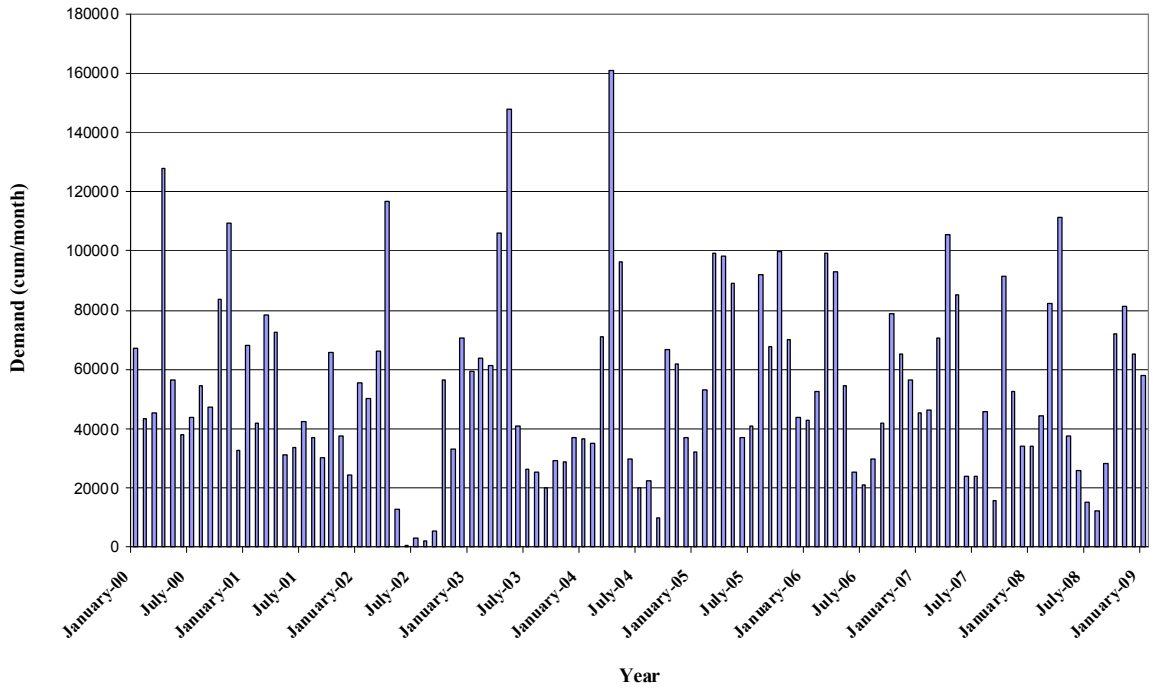
The Kaliya Pilot Area is divided into two sub regions. A northern sub region (Kaliya area) located in the northwest corner of the Dead Sea and a southern sub region (Mizpe Shalem area) located around 20 km south to the northern sub region. The climate is extreme arid (average precipitation is less than 100mm/year). The fresh groundwater resources originate from the rainfall

recharging the regional aquifer along the anticline axes about 30 km westward to the Kaliya area. Few wells located in the edge of the area at Mizpe Jericho well field (see Fig. 1) are supplying fresh water to the area. Along the Dead Sea shore there are two spring clusters. Their salinity varies between 600-6000mg/l Cl (Hötzl et al. 2008 chapter 3.3.2) and the total discharge is approximately 90-95 MCM/year. Most of the spring's water is flowing to the Dead Sea and lost there.

The domestic sector includes 6 small localities consuming approximately 230,000 cum/year. The population growth is extremely high (7.9% for 2006) and therefore water demand is expected to increase. The main economic activity is agriculture. Only a small amount (approximately 1 MCM/year) of brackish water (salinity up to 500 mg/lit Cl) is used today for the local agriculture. Sewage is another source for irrigation and it is generated at the cities located in the high mountain. The secondary treatment sewage is stored at a reservoir and later is mixed with the brackish water. It is being used for palm trees irrigation.

The water policies for this region are: increasing the use of marginal water (brackish and treated sewage) instead of using fresh water for agricultural and at the same time increasing the domestic water consumption as a result of urban/rural development. The future development depends on reliability of the water supply (quantities and qualities). It is important to mention that the current fresh water supply availability is the limiting factor for future development (agriculture, domestic use, tourism), especially during the peak demand seasons (Fig. 4).

Fig. 4: water demand for irrigation in a cultivation area (Kaliya)



In this area there are two agricultural seasons (March/April and September/October). At these periods the hourly and monthly water demand rises dramatically because all users need the water at the same time. To satisfy the high demand peak in short time, a large number of wells and large diameters of the pipelines are required. In between, the demand is low. Fig. 4 shows the fresh water demand at one of the major cultivation sites at Kaliya area. It is important to note that the beginning of the agricultural period depends on the last rain event which is the end of winter. When the last rain event occurs at the beginning of March, than the high pumping period is postponed to April. In very drought years the high pumping period may start at the end of February.

3.2 Wadi Shueib Pilot area

Wadi Shueib, located at the eastern side of the Jordan Valley (Fig. 1), is characterized by a steep relief with elevations ranging from -200 m bsl in the southwest up to 1240 m asl in the northeast.

The area belongs to the Balqa Governorate and comprises 5 larger municipalities (Salt, Fuheis, Mahis, Yarka, Ira) and several small hamlets. The population density, as well as most agricultural activity is concentrated in the higher altitudes in the northern part of the Wadi. The dense drainage network of the escarpment discharges the periodic flush floods of the rainy season to the Jordan Valley floor. The baseflow in the main channel is perennial and comprises the discharge from several springs, the outflow of the two sewage treatment plants in Salt and Fuheis as well as untreated sewage from the smaller municipalities (Ira and Yarka). In 1968 a dam was constructed in Wadi Shueib at the outlet in order to catch up the flows, but falls dry during the summer months.

The key water management challenges in the Wadi Shueib area are the contamination of springs and the incomplete connection of the population to the sewer system as well as major leakages in the water mains. Population growth, industrialization and irrigation of formerly uncultivated lands led to the deterioration in the surface and groundwater qualities and to the depletion of resources.

Only part of the houses within the catchments of the study area is connected to sewage collection and treatment systems. All other households in villages and settlements discharge their wastewater to leaky cesspools, where the wastewater infiltrates and contaminates the groundwater and springs or the surface water of nearby Wadis. Even the treated effluents do not have enough flow distances or underground flow paths for self-purification to take place. In some cases, leaky water supply and sewerage systems, irrigation return flows and other activities have lead to induced infiltration of water into the aquifers feeding the springs. In additional, in the catchment areas of some of the springs, wells were drilled, which lead to reductions in the discharge of the springs. Accordingly, the discharge rates and the chemistry of the water have also changed.

The over-pumping of aquifer, the various human activities of urbanization, irrigation and resources management are gradually leading to environmental and water resources damages, which in turn will negatively impact the socio-economic situation of the population.

A holistic water cycle diagram for Wadi Shueib is presented in Fig. 5.

For all of these problems the Ministry of Water and Irrigation in Jordan has started a range of different projects, many in cooperation with international donors:

- Vulnerability Mapping (GIJP-SMART).
- Delineation of spring protection zones (MWI-BGR).
- OMS (Operations & Management Support)-Program (GTZ/KfW).
- Modelling of Groundwater recharge from Wadi Shueib dam (SMART).
- Demonstration plant for decentralized wastewater treatment (SMART).
- Feasibility Studies for decentralised wastewater treatment of the villages Ira and Yarqa (SMART).
- Preparation of holistic water balances including all water sources (surface water, runoff, wastewater, groundwater) (SMART).
- Stakeholder consultation and problem screening exercise according to the DPSIR concept (SMART)
- Organizing and classifying the available knowledge.

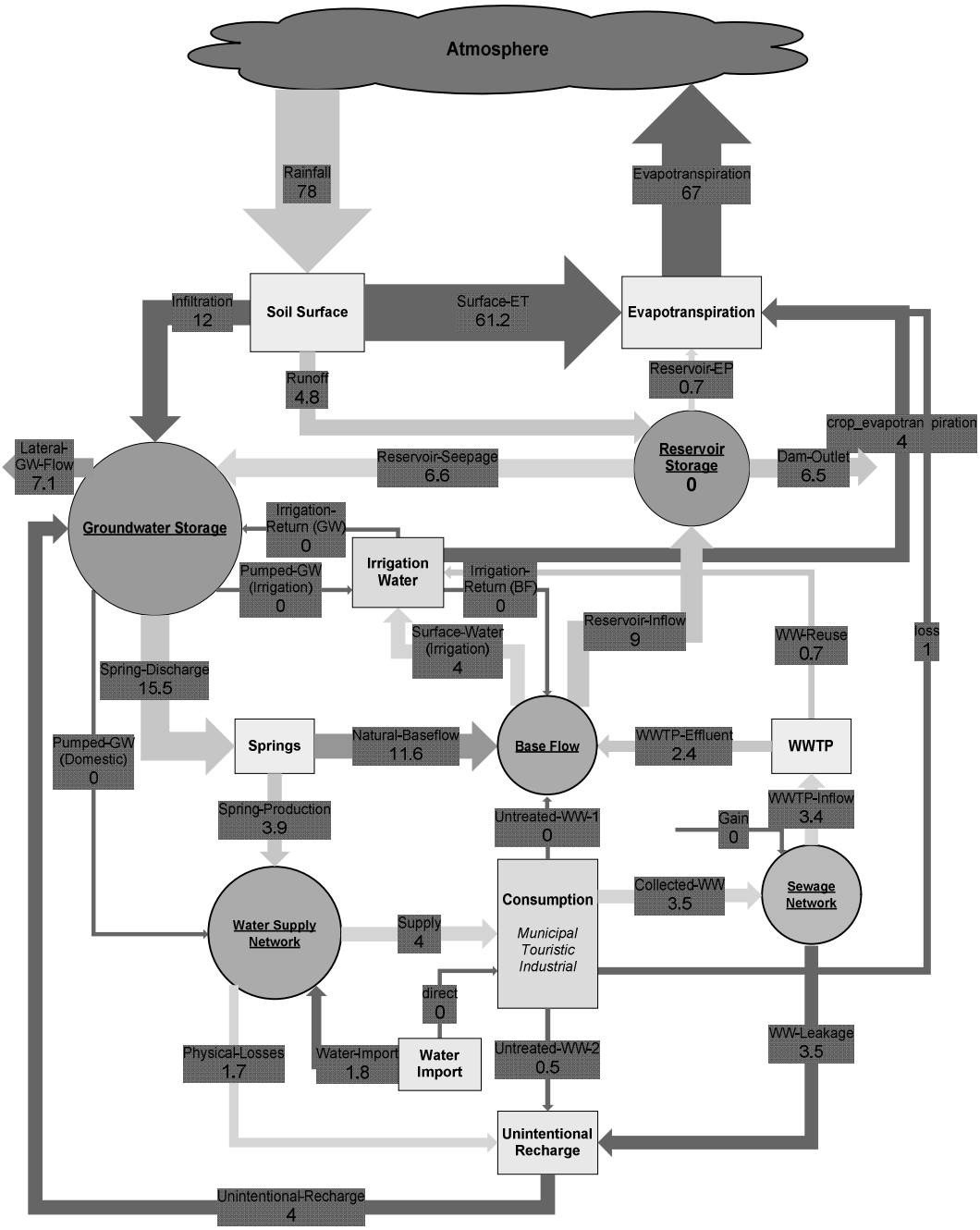


Figure 5: Holistic water cycle diagram for Wadi Shueib. Numbers on arrows indicate annual water flows in mm. Colours of arrows indicate reliability of the source or information (Riepl et al 2009).

4. CONCLUSION

The Jordan Rift Valley (JRV) is an arid zone. The main sources of fresh water are the relatively deep aquifers and some springs. The aquifers are naturally replenished by the rainfall falling in the surrounding mountains in the east (Jordan) and West (Israel and the Palestinian Authority). In these areas more favourable climatic conditions prevail, with average precipitations of 600-800 mm/year.

Geologically, the JRV is an extensive geotectonic feature bordered by a deep-seated fault system that creates an elongated and narrow basin and acting as drainage basin for groundwater originating both from the eastern and the western mountain aquifers that border the Rift basin as well as ephemeral streams and floods generated in the surrounding mountains. In this system freshwater entering the aquifer from the outcrops in the western and eastern edges reaches the valley and mixes with the local saline water bodies. The management of the JRV water resources is rather complex, as saline water bodies and fresh water bodies coexist in the aquifers with a very delicate equilibrium

between them. Inadequate exploitation of the fresh water sources may induce the migration of the saline water bodies and a rise in the salinity of the freshwater wells. On the other hand if the fresh water entering the aquifer is not captured before it mixes with the saline water bodies, substantial amount of fresh water that could be otherwise utilized for water supply could be lost.

The main economic activity in this region is agriculture and its development critically depends on the availability of water for irrigation. The future development depends on reliability of the water supply (quantities and qualities). It suffers from geographical isolation from areas with more favourable hydrologic conditions or large desalination centres (along the Mediterranean coast). Therefore, the development in this area primary depends on the local sources and the only way to reach to long term sustainability is by developing these sources and integrating their utilization. The water policy is to shift fresh water form agricultural uses to domestic purposes as a result of urban/rural development. The difference will come from marginal water sources (brackish and sewage).

Two pilot areas located in two sides of the Jordan Valley, had been chosen as a demonstration area. In Kaliya area the agriculture is the dominant occupation while in Wadi Shueib catchment area there are medium sized cities (domestic users) beside agricultural villages (agriculture users).

In Kaliya area there are few water resources (limited fresh water, brackish water and secondary effluent). The fresh water is limited and domestic purposes should be prioritized. Brackish and effluent can be used for selected agricultural crops (Palm and olive trees). The recommended scenarios for the Kaliya area are the alternatives of dilution of brackish water with effluent and dilution of fresh water with brackish water.

In Wadi Shueib, the major management options are given by a) water supply pipe renovation, b) implementation of decentralized WWTP, c) replacement of aged sewer systems, d) enforced implementation of groundwater protection zones. From an integrated management perspective, the feedbacks between these different implementation measures are not adequately quantified. The future reopening of contaminated springs will also minimize pumping costs significantly reducing water pumping from the floor of the Jordan valley (350 m bsl) to the City of Salt (700 m asl). However, even with this measure, increasing water imports or increased desalination of brackish water is necessary in future to accommodate a population growth of ca. 2% per year and an expected increase of average water consumption of 86 l/p/d in the Balqua Governorate in 2004 to the target of 100 l/p/d specified in the National Water Master Plan. This is still well below the per capita amounts utilized in the western pilot area, which points to even higher water demand in the future.

The examples from these two pilot areas and the way of defining the proper alternatives provide instructive case studies of IWRM concept in worldwide. In both pilot areas, the available water volumes are almost fully utilized, management challenges are now mainly water quality and the reduction of evaporation.

The ever increasing demand for freshwater and to some extent brackish water requires proper tools of management and combined utilization of the freshwater and saline water sources. Proper allocation of the water resources within a wider frame of integrated water resources management (IWRM) which considers that waters of different qualities are allocated to a variety of sectors allows optimizing the water exploitation scheme on one hand and preserving the aquifers on the other. In the study areas we regard all the existing waters sources (freshwater, brackish, saline water and effluent) as valuable water sources, which can greatly increase the overall yield, but a judicious application of water treatment technologies, water mixing approaches and appropriate agricultural activities, capable of using freshwater, brackish water, treated effluent and mixtures of them for irrigating properly chosen crops.

5. ACKNOWLEDGEMENTS

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INVESTIGATING THE OPTIMAL MANAGEMENT OF THE HELPS ROAD DRAIN URBAN STORMWATER HARVESTING SYSTEM WITHIN THE CITY OF SALISBURY, SOUTH AUSTRALIA USING CONDITIONAL VALUE-AT-RISK

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Abstract: We have created an SVG simulation tool for the analysis of urban stormwater harvesting systems and applied it to model the proposed Helps Road Drain scheme in the City of Salisbury, South Australia. The scheme consists of 5 inline storage facilities and an aquifer storage and recovery (ASR) system. The captured stormwater is harvested and may be subsequently pumped to consumers according to a contracted demand. We investigate a number of scenarios, such as extended wet or dry periods and reduced pumping, to evaluate the system's performance. We use Conditional Value-at-Risk (CVaR) as a measure for comparing the scenarios with regards to overflow and supply shortfall from the system. Across all of the investigations we observed that reducing the pumping rates at the storages resulted in more feasible amounts of water being stored in the aquifer whilst maintaining flows and satisfying demand for stormwater. The simulations of the scheme highlighted some possible issues with the system and some suggestions were made to resolve them and improve its performance.

Key words: Conditional Value-at-Risk, simulation, stormwater harvesting, urban stormwater

1. INTRODUCTION

We have created a computer-based simulation tool for the analysis of urban stormwater harvesting systems that serve to capture and harvest stormwater through a series of connected dams and an aquifer storage and recovery (ASR) system (Thomas et al. 2007). The simulation tool provides a visual representation of the system over time using Scalable Vector Graphics (SVG) which allows managers to visualise the limiting factors and interdependence of components within a system.

We have applied our simulation tool to model the Helps Road Drain urban stormwater harvesting system in the City of Salisbury, South Australia which is due to come online in late 2009. We consider the proposed system configuration as set out by the council and proceed to investigate a number of scenarios to evaluate the system's performance. The risk measure Conditional Value-at-Risk is used to compare the overflow and supply shortfall of the various water management policies.

The capital city of South Australia, Adelaide, relies on the ailing Murray River for its water supply. Stormwater is a valuable resource that may be used to reduce its dependence on the Murray.

2. HELPS ROAD DRAIN SYSTEM

The Helps Road Drain urban stormwater harvesting system is located in the City of Salisbury, in the northern Adelaide region of South Australia. The system consists of five inline storages each serving as a temporary water storage facility for flood mitigation and as a means of controlling environmental flows. The storages also allow the City of Salisbury to capture and hold stormwater

for harvesting and subsequent supply to local users. Stormwater runoff from the urban catchment flows into each of the storages together with any draining outflows from the storages directly upstream as shown in Figure 1. The system has a total of 2373 ML available for harvesting and flood mitigation with the individual storage capacities shown in Table 1.

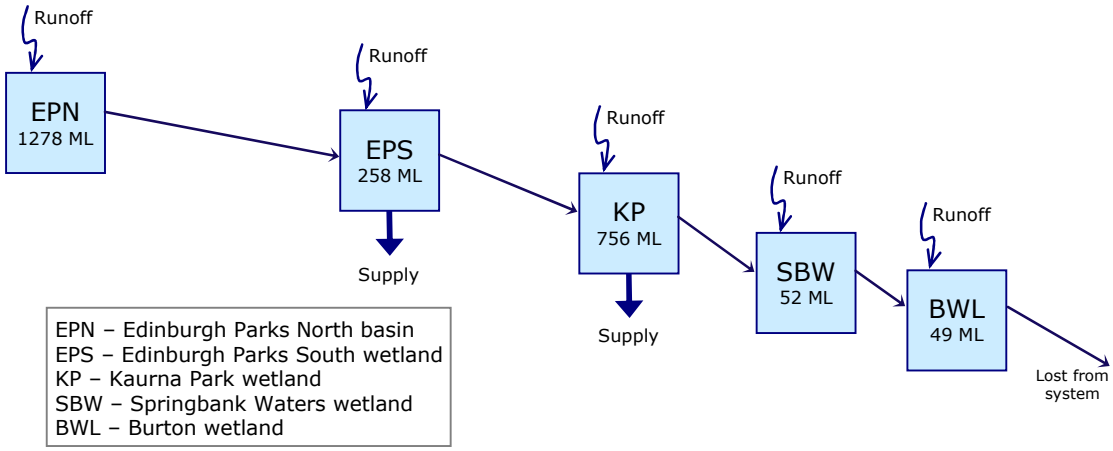


Figure 1: Diagram of the Helps Road Drain urban stormwater harvesting system

Table 1: Capacities of the storages in the Helps Road Drain system

	EPN	EPS	KP	SBW	BWL
Detention volume (ML)	1278	258	756	52	49
Harvest volume (ML)	610	67	92	33	17
Lowest weir height (m)	3	1.2	0.6	1.15	1.5

The first storage in sequence with a detention volume of 1278 ML is the Edinburgh Parks North basin. The main purpose of the first storage is to serve as a temporary storage for flood mitigation and to control the subsequent downstream flows. The outflow from the basin is over a V-notch weir at a height of 3 metres and through a valve at 1.5 metres. In this study we will assume that the valve remains closed. These outflows enter a 7 km open channel reach and subsequently flow into the downstream wetland, Edinburgh Parks South.

Outflow from the Edinburgh Parks South wetland is over a 5 metre long weir at a height of 1.2 metres. The local sub-catchment generates runoff from an industrial area to the east of the wetland which also contains users of the harvested stormwater at the site.

The third storage is the Kaurna Park wetland, a man-made urban forest with a detention volume of 756 ML. The vegetation within the wetland cleanses the water before it flows on to the next storage. The Kaurna Park weir consists of a 2 metre plate and a 20m long concrete-mesh wall. In practice the weir heights are adjustable but for our simulations they have been set to 0.6 metres and 0.8 metres, respectively.

The Springbank Waters wetland is located in the centre of a new urban development forming the main part of the local catchment area. The capacity of the storage is about 52 ML and the outflow is over a set of weirs spanning 70 metres with the lowest at a height of 1.15 metres and the highest at 1.4 metres.

The final storage in sequence is the Burton wetland, located directly next to the Springbank Waters wetland. A small storage of about 49 ML, the wetland receives runoff from the Burton residential area. The outflow from the storage is over a 3 metre long weir with sloping sides at a height of 1.5 metres. Water leaves the system at this point and eventually flows out to sea and cannot be recaptured.

Pumping stations are located at both the Edinburgh Parks South and Kaurna Park wetlands where water is pumped out of the storages at 85 L/s and 40 L/s, respectively. Each day where water is available to be pumped from the storage is known as a ‘harvest day’. Pumped water is stored in a

balance tank and is available for local industry to purchase and use according to a contracted supply. Any excess water is injected into the underground T1 aquifer. Water may also be extracted from the aquifer to assist in satisfying the demand. Extractions are limited to a maximum of 80% of the amounts that had previously been injected.

2.1 Mathematical model

The five storages of the Helps Road Drain system are modelled by a system of ordinary differential equations of the form

$$\frac{dV_i}{dt} = A_i(h_i)\frac{dh_i}{dt} = \mathsf{a}_i(t) - \mathsf{b}_i(t) - q_i(t) \tag{1}$$

where $V_i(t)$ is the volume of the storage and $h_i(t)$ is the height (or depth) of the water at time t, and where $A_i(h_i)$ is the horizontal cross-sectional area at a given height, a_i is the rate of inflow, b_i is the rate of draining and q_i is the constant pumping rate for the i-th storage in sequence.

The rate of inflow is

$$\mathsf{a}_i(t) = \begin{cases} f_i(r(t)), & \text{if } i = 1 \\ f_i(r(t)) + \mathsf{b}_{i-1}(t - \mathsf{t}_i), & \text{otherwise} \end{cases} \tag{2}$$

where f_i is the formula for the runoff into storage i for rainfall $r(t)$, and t_i is the number of days it takes for the draining outflow from the upstream storage $(i\text{-}1)$ to reach storage i.

The draining rate over a weir of constant length for storage i is derived from Torricelli's law (see Edwards and Penney (2000)) and is given by

$$\mathsf{b}_i(t) = \begin{cases} k_i\left[h_i(t) - w_i\right]^{3/2}, & \text{if } h_i(t) > w_i \\ 0, & \text{otherwise} \end{cases} \tag{3}$$

where w_i is the height of the weir and k_i is a constant.

2.2 Parameters of the system

The following parameters have been proposed by the City of Salisbury for the Helps Road Drain system which is due to come online in late 2009. The data inputs for the simulation are arrays of a representative range of depths for each of the storages and their corresponding horizontal cross-sectional areas and draining coefficients k_i (derived from flow formulae). The height-area profiles for each storage are shown in Figure 2.

By the fundamental theorem of calculus the volume is given by the relationship

$$V(h) = \int_0^h A(h)\, dh \tag{4}$$

which is numerically approximated in the simulation using the composite trapezoidal rule of the piece-wise linear $A_i(h_i)$ functions (Burden and Faires 2005).

The delays between the storages have been set to 2 days between the Edinburgh Parks North basin and the Edinburgh Parks South wetland, 1 day between the Edinburgh Parks South wetland and the Kaurna Park wetland, 30 minutes between the Kaurna Park wetland and the Springbank Waters wetland, and 6 minutes between the Springbank Waters wetland and the Burton wetland.

Figure 3 displays the monthly demand for harvested stormwater from the system totalling 546 ML/annum. This relationship has been derived from a history of customer usage at the two respective sites.

The simulation randomly generates daily rainfall from 2 locations: Smithfield and Edinburgh, using a mixed gamma distribution (Piantadosi et al. 2008). The expected monthly rainfall totals for both locations are shown in Figure 4A. Runoff from the Edinburgh Parks North basin depends on the Smithfield rainfall and the remaining wetlands receive runoff based on the Edinburgh rainfall. Rainfall-runoff relationships for each storage were derived by performing linear regression to match the observed rainfall to the simulated runoff from a WaterCress model by Richard Clark (personal communication). The expected monthly runoff totals for each storage are shown in Figure 4B.

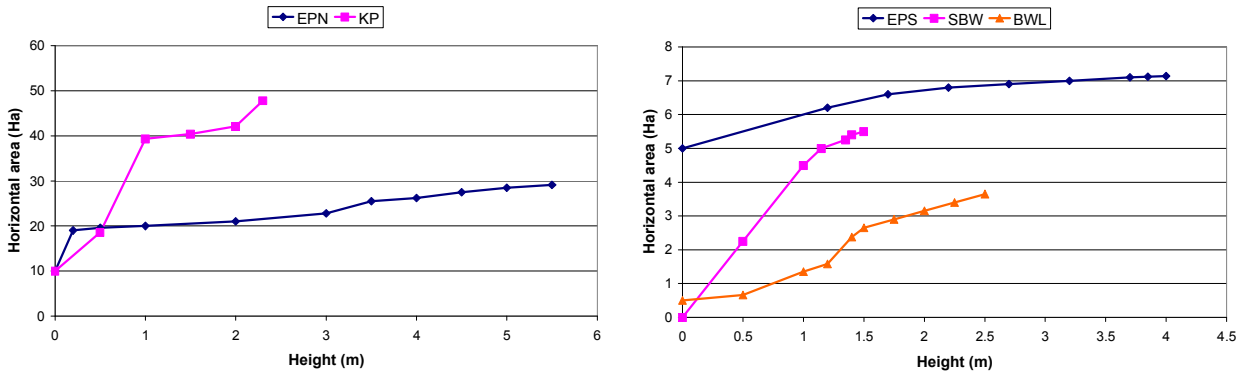


Figure 2: The horizontal cross-sectional areas of the storages at given heights

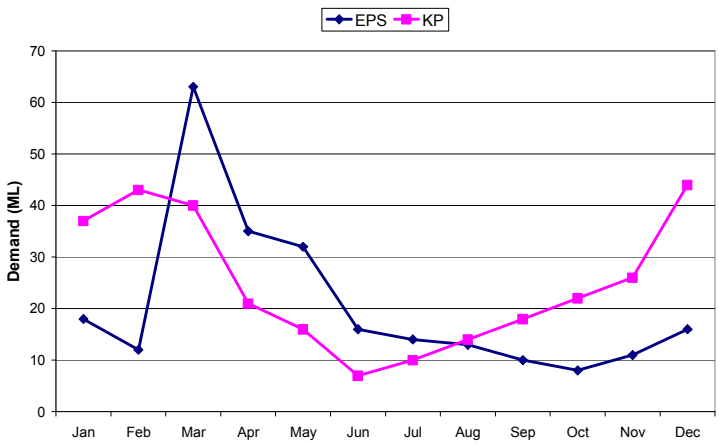


Figure 3: Monthly demand for stormwater from the Edinburgh Parks South and Kaurna Park wetlands

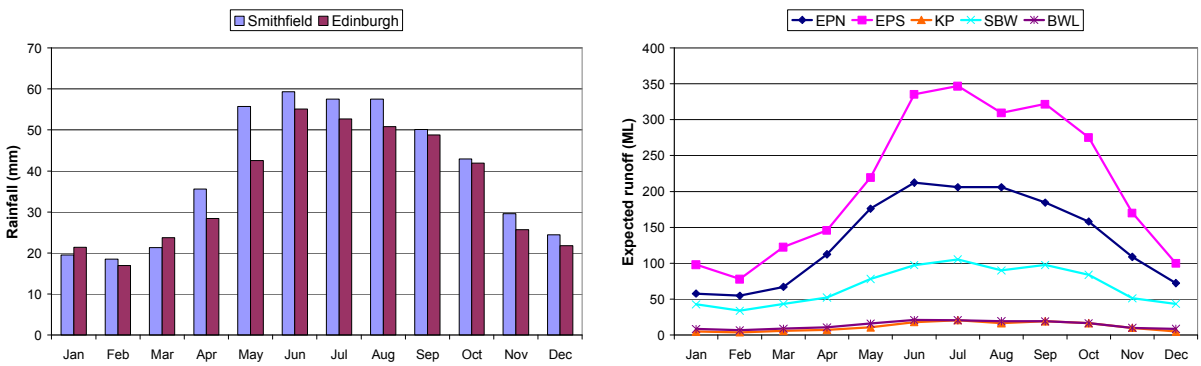


Figure 4: A) Expected monthly rainfall totals for the Smithfield and Edinburgh areas; B) Expected monthly runoff totals into each storage of the Helps Road Drain system

3. SIMULATION TOOL

The simulation tool has been written in SVG (Scalable Vector Graphics) and therefore may be viewed in an SVG-enabled web-browser. The system of differential equations is solved using a fourth order Runge-Kutta method (Burden and Faires 2005) and the changing volumes of water in the storages are drawn to the screen with two-dimensional graphics (SVG). Figure 5 shows the main screen of the simulation tool.

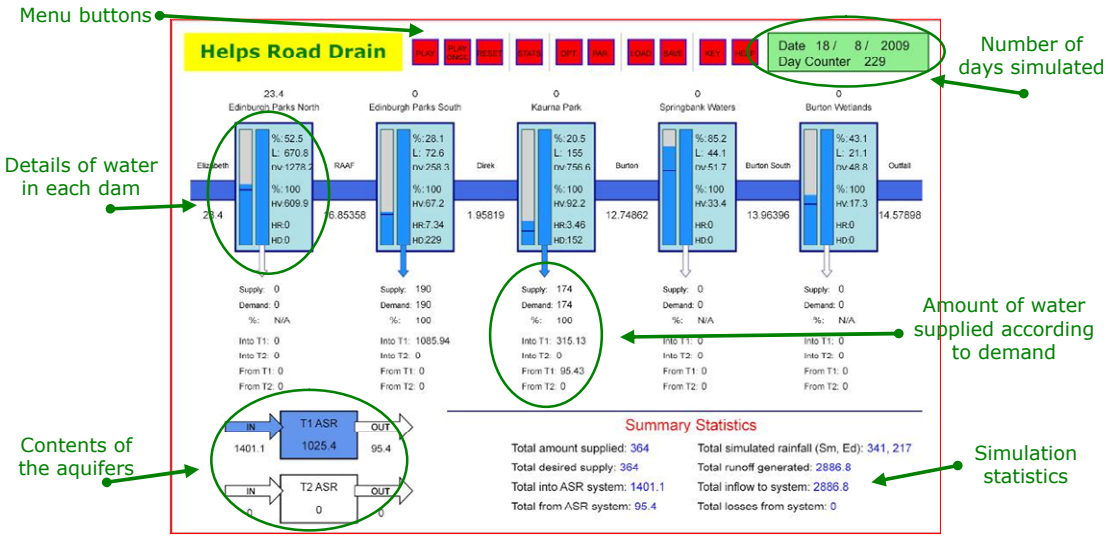


Figure 5: Main screenshot of the simulation tool with some highlighted features

3.1 Risk analysis

The simulation tool has the ability to perform risk assessment of various "loss" criteria such as overflow and failing to satisfy demand for stormwater. This allows the user to evaluate the effectiveness of a water management policy. The risk measure Conditional Value-at-Risk (CVaR) is calculated by costing long-term simulations. CVaR is defined as the expected value of loss given that the actual loss exceeds some significant Value-at-Risk (VaR) threshold and occurs with probability p (Rockafellar and Uryasev 2000). In the simulation tool the user may define either the probability or the VaR threshold. The relationship between them is shown in equation (5).

$$p = \Pr\{\text{Loss} > \text{VaR}\} \tag{5}$$

CVaR is then the average of all the observed loss values that are greater than VaR. The results are displayed to the screen together with a histogram of the losses observed from the simulation as shown in Figure 6A.

3.2 Stochastic convergence

Another feature of the simulation tool is the Comparison Mode (see Figure 6B for a screenshot) which compares two identical systems, with different initial states, subject to the same runoff inputs and demands. The simulation shows that two such systems, one initially full and the other empty, will converge to the same state and stops running when this occurs. This is called the stochastic convergence of the system and is measured by time-scales, that is, the number of days taken for the two systems to reach the same state. The time-scale is given as an indication of how a system recovers from an extreme event, such as flooding and drought. Systems with very long time-scales

may be difficult to manage. The time-scales and stochastic equilibria are dependent on the parameters of the system such as storage capacities and pumping rates. See Thomas et al. (2007) for the reasoning behind this feature.

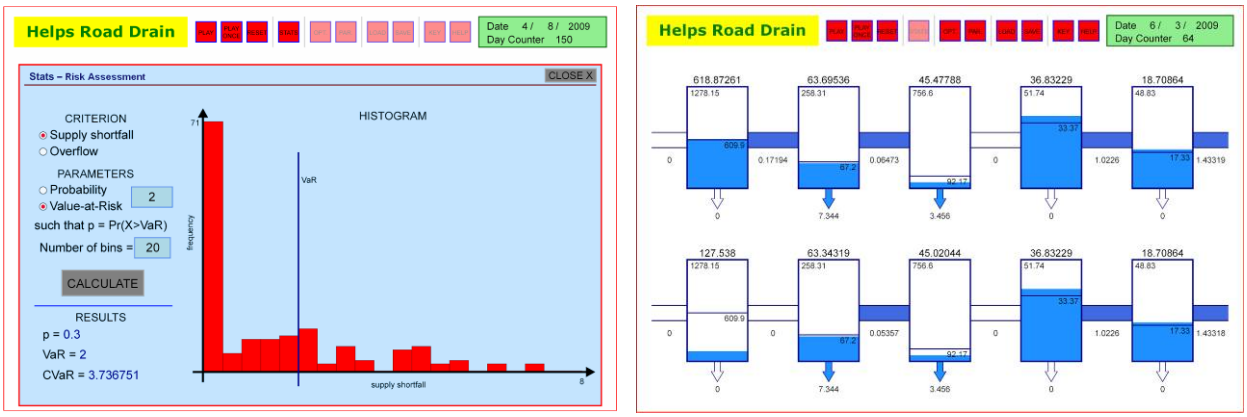


Figure 6: A) Screenshot of the risk assessment feature of the simulation tool where CVaR is calculated; B) Screenshot of the Comparison Mode of the simulation tool

4. SIMULATION RESULTS/INVESTIGATIONS

We have investigated a number of scenarios of the Helps Road Drain system using the simulation tool. Each simulation started with the storages and aquifers empty on the January 1st 2009 and was run over a time-horizon of 5 years (1826 days). The average results are summarised in Table 2. Across all the scenarios it was observed that reducing the pumping rates decreased the time-scales of convergence indicating that the approximate time for the system to return to normal after a drought or flood was decreased.

4.1 Investigation 1: Original configuration

In this scenario we considered the proposed configuration of the Helps Road Drain system as described in the previous sections of this paper. It was observed that the Kaurna Park wetland was often empty during the summer through to March-April. At this site significant amounts of water had to be extracted from the aquifer in order to satisfy demand. The Edinburgh Parks South wetland also emptied on occasion but not as often as the Kaurna Park wetland. The level of water at the Springbank Waters wetland would get close to capacity during the winter, increasing the risk of overflow. However there was only ever one case of daily overflow in the simulation runs. Since the wetland is located in a residential area the consequences of overflowing are greater and some measures may need to be put in place to reduce the risk of flooding.

Overall the system performed well: satisfying demand and not overflowing. However a large quantity of water was stored in the aquifer over the 5-year simulations which may not be feasible and may compromise the condition of the water table. These results suggest that the pumping rates may be reduced whilst maintaining flows, satisfying demand and injecting reasonable amounts of stormwater into the ASR system.

4.2 Investigation 2: Reduced pumping rates

In this scenario the pumping rates for the Edinburgh Parks South and Kaurna Park wetlands were reduced to 30 and 20 L/s, respectively. Once again the Kaurna Park wetland was observed to be emptying during the summer months but not as often as in the first scenario. If the emptying of the wetland became a serious problem the council would have to consider implementing a minimum

level at which pumping may occur at the site and hence not completely empty the wetland. The level of water at the Springbank Waters wetland was again approaching its capacity as in the previous scenario. A possible step to reducing the risk of overflow at this site would be to lower the heights of the weirs in order to let more stormwater drain out and flow into the downstream Burton wetland. Demand was mostly satisfied with a monthly shortfall CVaR of 35.34 ML with an average of 3597.9 ML being stored in the aquifer after the 5-year simulations.

4.3 Investigation 3: Unseasonably wet summers

To evaluate the system's ability to cope with extreme rainfall events we considered the scenario where the simulated daily rainfall for the summer months was doubled. The pumping rates were set to the values described in the first scenario. The Kaurna Park wetland continued to empty during the summer but naturally not as much as in the first scenario due to the increase in rainfall we now considered. Nevertheless, it gave further support to the proposal of a minimum level for pumping to be installed at the site. A reason behind the emptying could be that the wetland has a relatively low weir height. If it were raised then the outflows wouldn't be as large, resulting in the wetland not emptying as often and the downstream Springbank Waters wetland not approaching its capacity as often, thus minimising the risk of flooding.

The greater rainfall in this scenario resulted in greater amounts of stormwater being harvested and hence demand was almost always met. However large amounts of water were being stored being stored in the aquifer (an average of 10460 ML after 5 years) which may be infeasible. A reduction in the pumping rates may alleviate this problem.

4.4 Investigation 4: Unseasonably wet summers with reduced pumping rates

Next we considered the same scenario as above except with the reduced pumping rates of the second scenario. In this case it was observed that all the storages maintained a level of water above the height of the weir resulting in the reaches almost always containing water. This may be undesirable as the reaches never empty to maintain environmental flows through the catchment. This issue aside, demand was always satisfied, the contents of the aquifer seemed reasonable and the overflow from the system was minimal.

4.5 Investigation 5: Unseasonably dry winters

The fifth investigation considered the extreme of an unseasonably dry winter where the simulated daily rainfalls for June, July and August were halved. The pumping rates were as defined in the first scenario. In this case, although the demand was always satisfied, huge amounts of water were being stored in the ASR system over the 5 year time-horizon with an average of 8835.5 ML. With the reduction in winter rainfall it was observed that the Edinburgh Parks South and Kaurna Park wetlands were emptying during the autumn months and sometimes in winter. Once again, a reduction in pumping rates may reduce the frequency of the storages emptying and large amounts of water being injected into the ASR system.

4.6 Investigation 6: Unseasonably dry winters with reduced pumping rates

Lastly, we considered the same scenario as above but with the reduced pumping rates of the second scenario. Demand for stormwater from the system was almost always satisfied and an average of 3580.7 ML was stored in the aquifer over the 5 years. During winter the water levels hovered just above the weirs and during the summer flows were mostly maintained with the Kaurna Park wetland only emptying once or twice.

Table 2: Average simulation results

		Inves. 1	Inves. 2	Inves. 3	Inves. 4	Inves. 5	Inves. 6
Supply shortfall	p (%)	2.42	6.06	0.61	1.21	4.24	5.45
VaR = 2 ML	CVaR	35.31	35.34	7.74	12.14	29.11	31.79
Overflow	p (%)	0.054	0	0	0.055	0	0
VaR = 2 ML	CVaR	4.64	NA	NA	2.27	NA	NA
ASR contents after 5 yrs (ML)		9552.3	3597.9	10460	3759.2	8835.5	3580.7
ASR extraction totals (ML)	EPS	3.97	9.38	23.18	0.00	24.49	5.85
	KP	385.53	55.04	192.17	3.93	400.6	56.64
Harvest days over 5 yrs (1826 days)	EPS	1753	1777	1764	1809	1743	1782
	KP	1388	1710	1550	1796	1314	1709
Satisfied supply over 5 yrs (%)	EPS	99.75	98.71	100	99.75	99.75	99.76
	KP	96.33	92.43	99.39	98.32	95.1	92.69
Time-scales of convergence		234	216	231.4	208.2	295.8	267.4

5. CONCLUSION

Using our SVG simulation tool we have investigated a number of scenarios and observed how the Helps Road Drain urban stormwater harvesting system would perform. In most cases it was observed that reducing the pumping rates still satisfied demand for stormwater and resulted in feasible amounts of water being stored in the ASR system.

These investigations raised some possible issues especially overflowing/flooding and the significant emptying of the Kaurna Park wetland. In some cases we have even suggested possible solutions. We have only just begun to scratch the surface in the analysis of the Helps Road Drain system and further investigations are necessary to provide a comprehensive analysis of the system.

ACKNOWLEDGEMENTS

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COMBINED COST-BENEFIT ASSESSMENT OF GROUNDWATER PROTECTION MEASURES AND IMPROVED SANITATION CONCEPTS IN JORDAN

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Abstract: Both groundwater protection and sanitation infrastructure serve the same overall goals of safeguarding human health and preserving important ecosystems. However, in both segments the implementation of improvements is traditionally carried out by different professional and institutional sectors, which often results in a lack of mutual cost-benefit analysis. For example, sewer systems constructed within the catchment of groundwater resources (e.g. springs) provide the added benefit of conserving the spring water as drinking water. But, as a reality and often due to economic constraints, sewer systems are progressively leaking with age and even more, many villages in areas with steep topography or poor economical situation are not connected to sewer systems and still rely on cesspits. In this situation it is necessary to set priorities regarding groundwater deterioration and the financial investment into the sanitation system. The paper demonstrates how groundwater protection measures pay off in the course of 20 years at the example of a medium sized spring in Jordan. Further on, the use of vulnerability maps to set priorities of sewer and cesspit rehabilitation is shown.

Key words: Cesspits, economics, groundwater protection, IWRM, sanitation systems, sewers.

1. STUDY AREA INFORMATION

1.1 Location / Overview

From groundwater resources in Jordan, an estimated volume of 276 MCM/a is available as safe yield. However, currently about 392 MCM/a are abstracted, leaving behind an overabstraction of 124 MCM/a (BGR-MWI 2008). This results in frequent dropping of groundwater levels and also water quality deterioration. The dramatic situation calls for two responses: a) reduction of current abstraction rates by demand management or provision of alternative water sources, b) safeguarding and protection of available water resources in terms of water quality. Wadi Shueib, an eastern side wadi of the Jordan Valley, is characterized by a steep relief with elevations ranging from 200 m bsl in the southwest up to 1,240 m asl in the northeast (Figure 1). The area belongs to the Balqa Governorate and comprises 5 larger municipalities (Salt, Fuheis, Mahis, Yarka, Ira) and several small hamlets. The population as well as most agricultural activity is concentrated in the higher altitudes in the northern part of the wadi. The dense drainage network of the escarpment discharges the periodic flush floods of the rainy season to the Jordan Valley floor. The baseflow in the main channel is perennial and comprises the discharge from several springs, the outflow of the two sewage treatment plants in Salt and Fuheis as well as untreated sewage from the smaller municipalities (Ira and Yarka). In 1968, a dam was constructed at the outlet of Wadi Shueib in order to catch up the flows, but falls dry during the summer months.

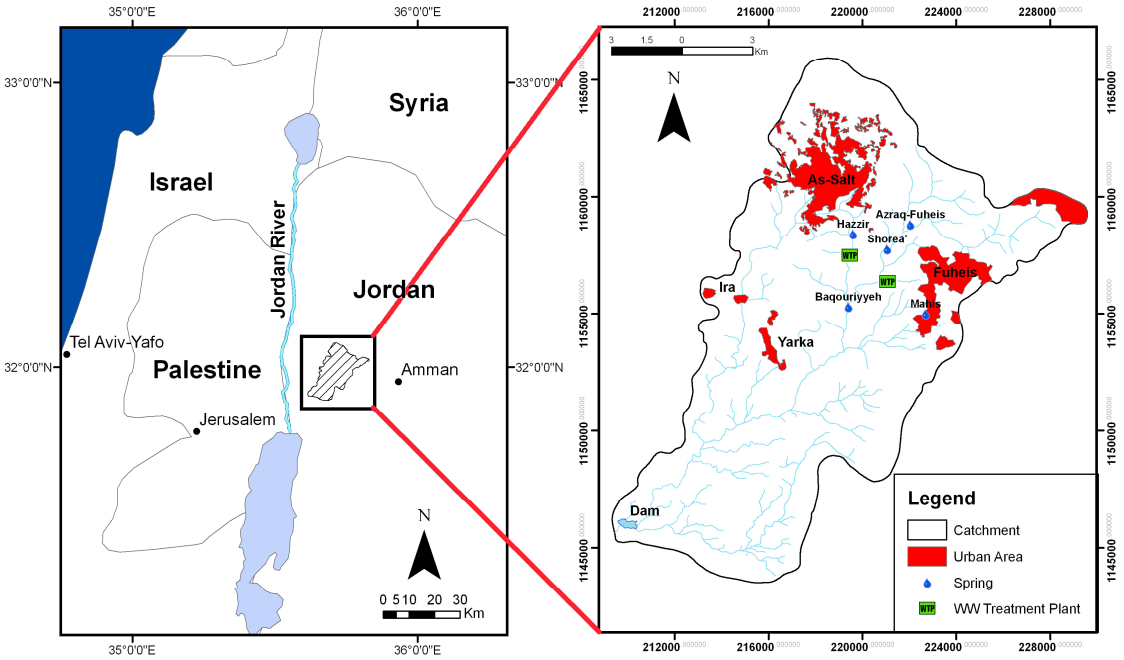


Figure 1. Overview over the Wadi Shueib catchment area.

1.2 Climate

Precipitation varies strongly in Wadi Shueib due to terrain complexity. Annual rainfall decreases rapidly westwards from over 600 mm at the town of Salt to less than 400 mm in the centre of the catchment area and to less than 200 mm at South Shoneh with a maximum of rainfall between December and March and a minimum in the months June until September. The annual amount of rainfall in the Wadi Shueib catchment for an average water year is estimated at 78 MCM (ranging from 50 – 133 MCM/a). Potential evaporation rates (Penmann-Montheith) in the area range from 1 mm/d (in the winter months at higher altitudes) to 8 mm/d (summer in the southern part).

2. DRIVERS AND MOTIVATION FOR CHANGE

2.1 Groundwater resources

The aquifer systems in the Wadi Shueib area are drained by several springs that are partly used for drinking water supply. A few wells were drilled in the area around Salt and near Shunat Nimrin penetrating the Hummar (A4) and Naur (A1/2) aquifer, others were drilled into the A7/B2 aquifer. However, today many of these wells, especially in the urban area of Salt are abandoned, sealed and no longer used for drinking water supply due to high concentration of nitrate (NO^{3-}). Figure 1 and Figure 4 show the location of the major springs in the study area. The total spring discharge (21 springs) was estimated by Taany (1992) to 9.4 MCM for an average water year (the NWMP assumes 15.6 MCM/a).

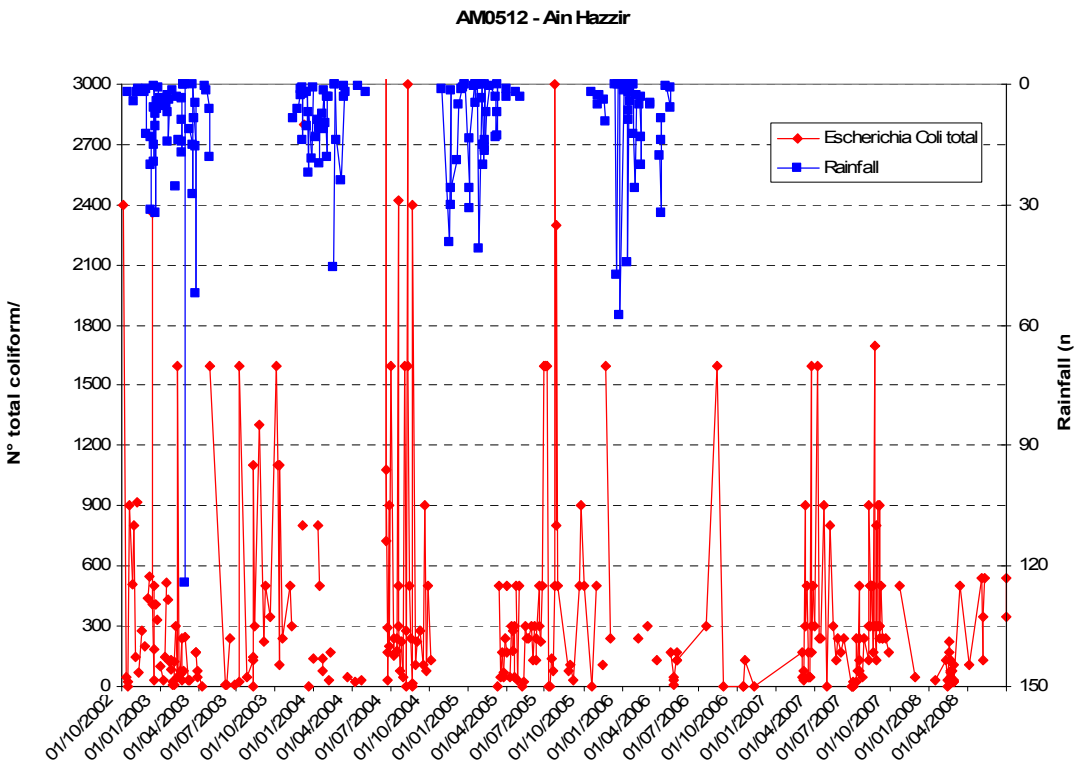


Figure 2. Occurence of total coliforms at Ain Hazzir spring in relation to rainfall events (BGR-MWI 2008).

The spring discharge not used for drinking water supply constitutes to the base flow along the Wadi Shueib. Long term records of observed baseflow (1981-99) at the Shueib Gauge (AM0008) give an average volume of 5 MCM per year.

The focus of this study is the Hazzir spring which is constantly exceeding the Jordanian Drinking Water Standard in terms of nitrate concentrations (60-80 mg/l in the spring water vs. 50 mg/l guideline value). Furthermore, Hazzir spring is subject to microbiological contamination as indicated by E.coli occurrence. Figure 2 shows the dependence of high total coliform counts after rain events in summer times.

The springs (without Hazzir) have an average pumped discharge of about 7 MCM/a. Hazzir spring has a pumped discharge of approx. 900,000 m³/year. In wintertime the spring discharge is more or less sufficient to cover the needs of the Salt/Fuheis water supply. In summertime, the spring discharge is not sufficient and therefore, wells are put in operation.

2.2 Water supply

Municipal water demand is the foremost water issue in the Wadi Shueib area. The average municipal water consumption in the Balqa Governorate is given at 86 liters per capita per day (MWI, 2004). This amounts to a total domestic water consumption of 3.5 MCM/a for the Wadi Shueib population. However, water supplied to the Wadi Shueib amounts to 6 MCM in 2004 or 151 L/c/d (DOS, 2004). This is due to high rates of physical losses in the water infrastructure. Unaccounted for Water (UFW) is estimated at 57.2% in the Balqa Governorate (MWI, 2004), including physical losses, illegal connections and wrong metering.

According to the National Water Master Plan, the first priority of water management is to allocate a share of 100 liters per captia per day to domestic water supplies. With the current rates of UFW and the strong population growth, this points to a supply need of ca. 8 MCM in 2010.

In many areas of Jordan continuous water supply is not sufficiently guaranteed. In these cases water tankers provide additional water to households. Costs for this service vary between JD 1.5-2.0 per m³.

2.3 Sanitation system

The Wastewater Treatment Plants of Salt and Fuheis receive inflow from the municipalities of Salt, Fuheis and Mahis. The total sewer connectivity in Wadi Shueib is 65%-79%. The plants discharge 2.1 MCM/a (in 2003) of treated effluent to the wadi, where it is mixed with the base flow from the springs and used downstream for unrestricted irrigation and aquifer recharge in the Wadi Shueib Dam. When the stream reaches the Shueib Dam, over 10 km away, treated effluent constitutes up to 30% of the dry weather flow.
Houses that are not connected to the treatment plants often use septic tanks or cesspits for their domestic wastewater which is mostly discharged to gardens.
However, regulations on proper disposal of waste sludge are lacking and there is practically no enforcement. Local municipalities which are in charge of regulating cesspits and septic tanks do not have the capacity or funding required for monitoring and regulating on-site sanitation.
.
Cesspits are pumped out by septage tankers more or less regularly depending on soil conditions and seepage, thus creating maintenance costs. Pumping costs are in the range of JD 13 per month. Note that these values may vary extremely. It seems that many households prefer a cesspit allowing seepage, in order to diminish the emptying cycle and consequently the associated costs.
A cesspit is estimated to cost around JD 600 (ECODIT/IRG 2004 in USAID 2005) or JD 105 per person (= in Balqa the average household size is 6 persons, consequently, the construction costs are in the range of JD 630) (Bataineh *et al.* 2002 in Abu-Madi 2004). Construction costs may extremely vary depending on storage capacity, ground and soil conditions.

The specific wastewater costs of households in urban areas that are connected to the sewerage are calculated from literature and average JD 0.19 per m³. Rural households with cesspits are found to disburse around JD 1.16 per m³. Average specific wastewater costs of households are given with JD 0.27 per m³.
Figure 1 shows the result of a household survey on the frequency of cesspit maintenance. 58% of the farmers indicate that they do not empty their cesspits at all (BGR-MWI 2008). Consequently, contamination of groundwater is widespread and in many cases contributes to the high coliform levels in spring waters mentioned above.

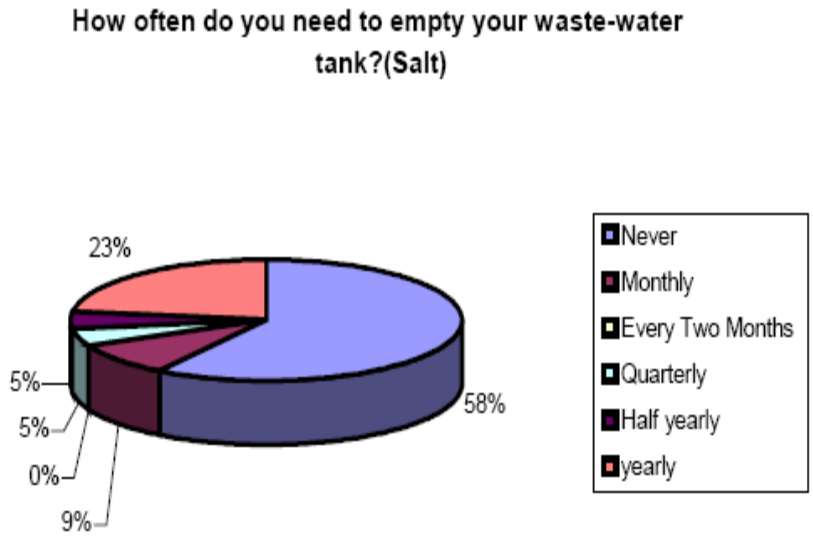


Figure 3. Result of a household survey on the frequency of cesspit maintenance (GTZ 2007).

In rural areas, households are rarely connected to sewerage systems. Mostly there are cesspits or septic tanks installed. Potential Pollution sources

Other potential pollution sources are provided by gas stations, slaughterhouses, hospitals, illegal waste dump sites, car service stations, carpenters, locksmiths and aluminium manufacturing. In addition, a large cement factory and two chemical industries are present.

3. ASSUMPTIONS OF THE SCENARIO CALCULATIONS

3.1 Business as usual

The Business-as-usual (BAU) Scenario, also called Status Quo or Baseline Scenario, describes unchanged continuation of the current management practices. For Wadi Shueib this means the continuation of water imports from Deir Allah pumping site at full volume flow, and the operation of the microfiltration plant. The full costs for imported water (including pumping from the base of the Jordan Valley, 1,000 m below Salt) were given as 0.29 JD/m³ by the MWI. The water from the Hazzir spring is not used due to its low water quality in this scenario.

3.2 Advanced water treatment

The advanced water treatment scenario assumes that a nitrate-removal plant is constructed in order to make the water of Hazzir spring directly usable for human consumption within the Jordanian Standard for drinking water. Different technological options such as electrodialysis, reverse osmosis or biological removal are possible. Basic cost comparisons are given in Rautenbach et al (1986) and Schoeman & Steyn (2003). Accounting for inflation and technological innovations, full treatment costs of 0.5 JD/m³ are assumed for the reverse osmosis treatment.

3.3 Removal of cesspits and rehabilitation of the sewer system

The rehabilitation scenarios assumed that the major sources of contamination, leaking sewers and cesspits, are brought to a state in which they do not pose a severe environmental threat any longer. Consequently, it is assumed that the nitrate levels in Hazzir spring will drop below the threshold value of 50 mg/L. While it can be assumed that also fecal contamination will be reduced significantly in this case, it is still recommended to maintain the existing facility for microfiltration in order to have maximum customer safety. The costs of the microfiltration plant were given as 0.01 JD/m³ by the MWI.

In order to calculate the costs for the cesspit & sewer rehabilitation program, first of all the number of cesspits and the respective sewer length within the groundwater contribution zone must be estimated. The number of inhabitants in Salt was given as 73,528 (DOS Population Census, 2004). Assuming 5 persons per household and the density of 8.1 persons per building, as recently measured for the adjoining Balqa-Urban area (DOS Population Census, 2004), a number of 9,077 buildings is estimated from Salt. Using the connection degree of Salt, given as 79%, it may be inferred that 21% of the houses rely on septic tanks. Thus, an amount of 1,906 cesspits is assumed for the Salt area. The length of the sewer network was directly obtained from GIS-analysis.

The further calculations of the total rehabilitation costs follow Table 1. Assumptions on rehabilitation cost

. Rehabilitation costs per item are based on Sorge et al (2008) for cesspits and GKW-Futuretec (2005) for sewers. Especially the rehabilitation costs for sewers might be too high for Jordan, since they were derived based on German data. Also the actual percentage of sewers requiring urgent rehabilitation is not known in sufficient detail. A more precise cost estimate is currently being developed by MWI & GTZ in the framework of the OMS Project. Nevertheless, the figures give a resonable idea about the upper limit of cost.

Table 1. Assumptions on rehabilitation cost

			% of Salt in Hazzir catchment	Items in catchment	% assumed to need rehabilitation	Rehabilitation costs per item [JD]	Total Rehabilitation costs [JD]
Cesspits in Salt	1906	[-]	59%	1118,822	81%	600	543,747.49
Sewer length	97000	[m]	59%	56939	60%	400	13,665,360.00
Sum							14,209,107.49

4. RESULTS

4.1 Scenario Analysis

Using the assumptions stated before, scenario calculations are set up for the costs arising for a time frame of 20 years. The results are detailed in Table 2.

Table 2. Scenario calculations for the estimated costs arising in a time frame of 20 years

Scenario			Baseline situation	Advanced Drinking Water Treatment	Rehabilitation of Cesspits
Consequence			Hazzir Spring is not used due to quality problems	Water from Hazzir Spring is fed into the local distribution network	Water from Hazzir Spring is fed into the local distribution network
Quantities	Utilized water production from Hazzir spring	m³/h	0	102,74	102,74
	Utilized water production from Hazzir spring	m³ in 20 years	-	18.000.000,00	18.000.000,00
	Water import from Deir Allah as substitute for spring water	m³ in 20 years	18.000.000,00	-	0
Assumptions	Cost microfiltation	JD/m³	0,01	0,01	0,01
	Pumping cost savings	JD/m³	0,29	0,29	0,29
	Cost reverse osmosis	JD/m³	0,5	0,5	0,5
Costs	Cost microfiltration	Mill. JD in 20 years	0		18
	Cost cesspit & sewer rehabilitation	Mill. JD in 20 years			10,56
	Cost reverse-osmosis treatment	Mill. JD in 20 years		180	
	Cost water import from Deir Allah	Mill. JD in 20 years	104,4	0	0
Total Costs		**Mill. JD in 20 years**	**104,40**	**180,00**	**28,56**

It is evident, that the scenario calculations are governed by the high costs of water import which causes also high total costs for the business-as-usual scenario. The total costs for the removal of nitrates as an end-of-pipe solution would amount to almost the double costs than the BAU-scenario. Compared to the BAU-scenario, the removal of cesspits and the rehabilitation of the sewer system is way more economical. However, it must be considered that additional water protection measures might be necessary to reach the targets. Also, the costs for the set up and maintenance of

groundwater protection zones were not available yet for the scenario calculations. Overall, it can be concluded, that just comparing the costs for microfiltration and the costs for water imports, an investment in the order of magnitude of 80 million JD might be available for groundwater protection measures if a time span of 20 years is considered. This is a large margin compared to the estimated costs of roughly 10 million JD for the sewer & cesspit rehabilitation. Thus, it is economically sensible to invest into protection measures and decentralized wastewater treatment facilities to utilize the local resource of the Hazzir spring.

4.2 Overlay analysis for defining rehabilitation priorities

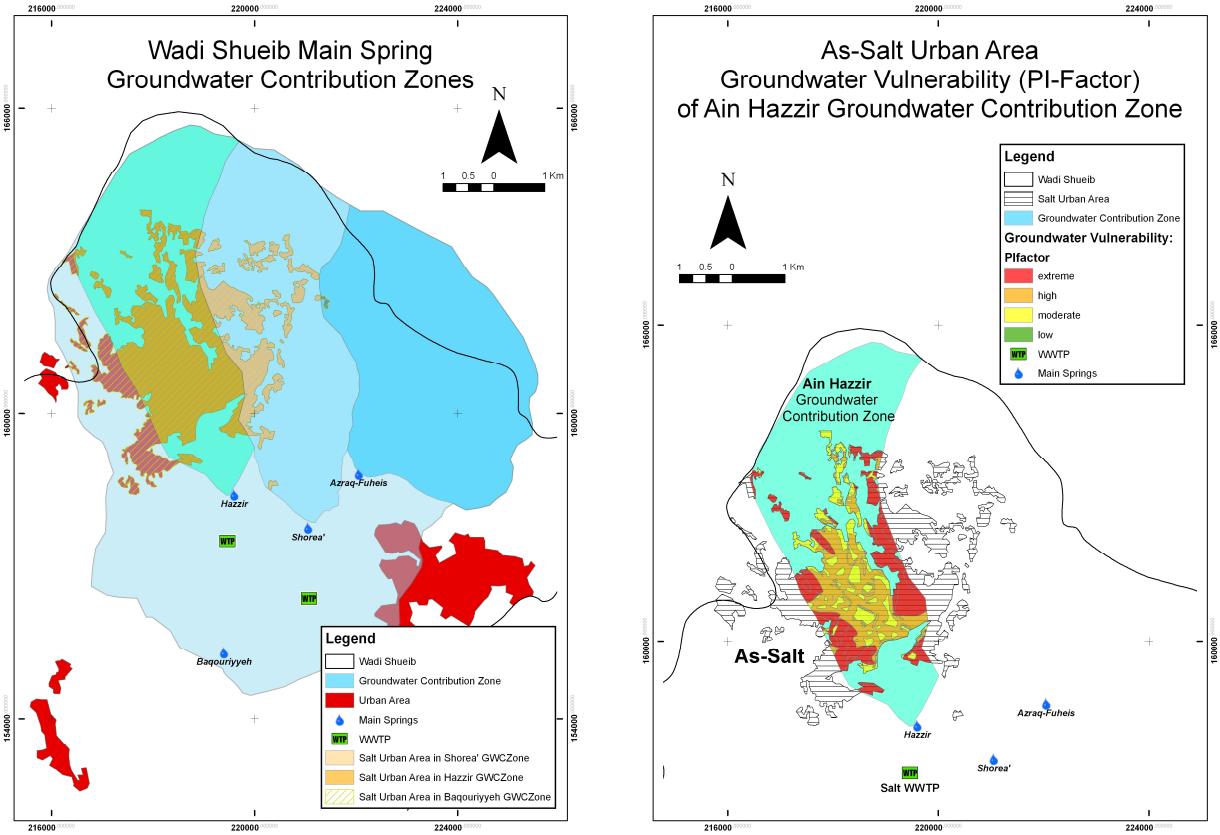


Figure 4. GIS-Overlay analysis of groundwater contribution zones as presented by BGR-MWI (2008) with vulnerability maps prepared by Werz(2006) and urban areas as identified from aerial photography.

In order to set priorities for sewer & cesspit rehabilitation planning, it is possible to utilize existing vulnerability mappings (Error! Reference source not found.) by Werz (2006). The groundwater vulnerability map was generated based on information deriving from the processed remote sensing image and the orthophotos, on data from soil investigations and field data including existing hydrogeological and hydrological data, on the DEM, and on geological maps (Werz 2006, Werz & Hötzl, 2008). For the assessment of the groundwater vulnerability the PI-method after Goldscheider (2002) was applied.

Figure 4 shows an example of the overlay analysis. Thus, from the 5.4 km² of urban area within the groundwater contribution zone, 2.2 km² are classified as extremely high vulnerable according to the PI-method while, 2.0 km² are classified as higly vulnerable. These areas should be treated with priority compared to the 1.1 km² exhibiting a moderate PI-Factor. However, the PI-factor vulnerability map constructed by Werz (2006) was not designed specifically for sewer systems and sources which are already 1-2 m below the surface. Thus the vulnerability is even higher than indicated by the PI method. In future, a specific sewage vulnerability map is recommended.

5. CONCLUSIONS

The paper demonstrates along the example of a contaminated spring in Jordan, how long term investments in water resources protection measures might pay off. Especially with respect to the high pumping and energy costs for water imports in this case, it is estimated that almost 80 million Jordanian Dinar (approx. 90 million Euro) could be invested into groundwater protection measures of a 100 m³/h spring while still being more economic than water imports. The key message of the paper is to demonstrate a methodology for the cost assessment. The numbers itself are just a rough approximation and subject to the addition of further costs upon in-depth analysis. Along with these statements, it must be kept in mind that the qualitative improvement of the spring water quality might take a long time due to the large underground reservoirs tapped. The investments into removal of contamination sources could be directed to septic tanks, which were reported by 58% of the users to be never emptied, thus, indicating complete leakage to the groundwater resource. In order to define priority areas for septic tank removal, rehabilitation of sewer pipes or the securing of other key pollution sources (e.g. slaughteries), groundwater vulnerability maps can be utilized.

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COMPREHENSIVE HYDROGEOLOGICAL, HYDROGEOCHEMICAL AND ISOTOPE BEHAVIOR OF NORTHEAST AFRICA INTRACRATONIC BASINS: CASE STUDY ABU SIMBIL BASIN

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Abstract: There are four mega intracratonic basins and five small basins in NE Africa. Abu Simbil Basin is one of them and did not extensively studies may due to the less potentialities from oil and minerals point of view. The newly accumulation of High Dam Lake and the intensive drilling of groundwater wells, encourage the governments for launching a major program for sustainable development of the basin. This article deals with identification of the groundwater aquifer system and their relationships with the lake. There are two groundwater aquifers namely; Sabaya and Abu Simbil. They hydraulically connected through faults and pores. They have groundwater salinity varied from 410 to 710 and from 1001 to 1972 ppm. Sabaya groundwater samples are isotopically depleted where they have hydrogen isotope values ranging from -82.4% to -62.6%, while Abu Simbil are less depleted that have values between -81.1% and -69.9%. The surface water recharge through pores reaches to 5.5 km in Sabaya aquifer that increases to more than 30 km through E-W faults. Owing to the effect of their coarser texture of Abu Simbil aquifer, the effect surface water increases to about 17 km through pores and extended to more than 40 km through E-W faults.

Key words: Northeast Africa, intracratonic basins, Abu Simbil basin, High Dam Lake, aquifer, isotope.

1. INTRODUCTION

Northeast Africa between the Mediterranean Sea in the north, the Wadi Howar in the south (north Sudan), the Tibesti (south Libya, northern Chad) in the west and the Red Sea in the east is characterized by a number of intracratonic basin, which is separated from each other by Pre-Cambrian outcrops of various size (Fig. 1a). The high grade polymetamorphic and granitoid rocks west of the Nile are part of the Pre-Pan Africa East Sahara Craton (Schandelmeier et al. 1987), while the basement east of the Nile comprises the Pan African metavolcanic-metasedimentary-ophiolite and related granitoid assemblages of the Nubian Sheild (Kroner 1985 and Vail 1986). The structural development in Northeast Africa between Cambrian and Carboniferous time was controlled by ENE-WSW oriented tensional forces, probably resulting from perpendicular regional compression, caused by the Pan African episode (Schandelmeier et al. 1987). The major consequence of this extensional regime was the formation of large NNW-SSE stricking structural blocks, which are generally wide, graben-type structures and smaller horst blocks. The major depocenter in the Early Paleozoic was recorded in the northwestern part of Egypt (Fig. 1b), while the others were located NW of Khartoum, Gebel Abyad, NW of Selima, Abu Simbil and Asyut (Schandelmeier et al. 1987).

Abu Simbil Basin (ASB) was selected due to the scare studies, the intensive drilling for groundwater exploration in the last decade gained a lot of information about subsurface geology, hydrogeology and hydraulic parameters of the basin; it occupies the largest artificial lake of the world (High Dam Lake that may change the hydraulic regime of the basin in the last decades; recently it takes attention of the government for launching a major program for the sustainable development. ASB is recorded south Egypt and north Sudan (Fig. 1 b and c), it is separated from other basins by arc volcanic rocks (gneiss and migmatic gneiss) in the western part and by Pan

African metavolcanic-metasedimentary - ophiolite of the Nubian Sheild in the southern and eastern parts, while it is connected with southern (Selima) and northern (Kurkur) basins through structural corridor (Fig. 1 c). The main objectives of this study are demarcating the groundwater aquifers, their extensions, thicknesses and hydrochemical characteristics. Besides, delineate the environmental isotope behavior and the mechanism of groundwater recharge.

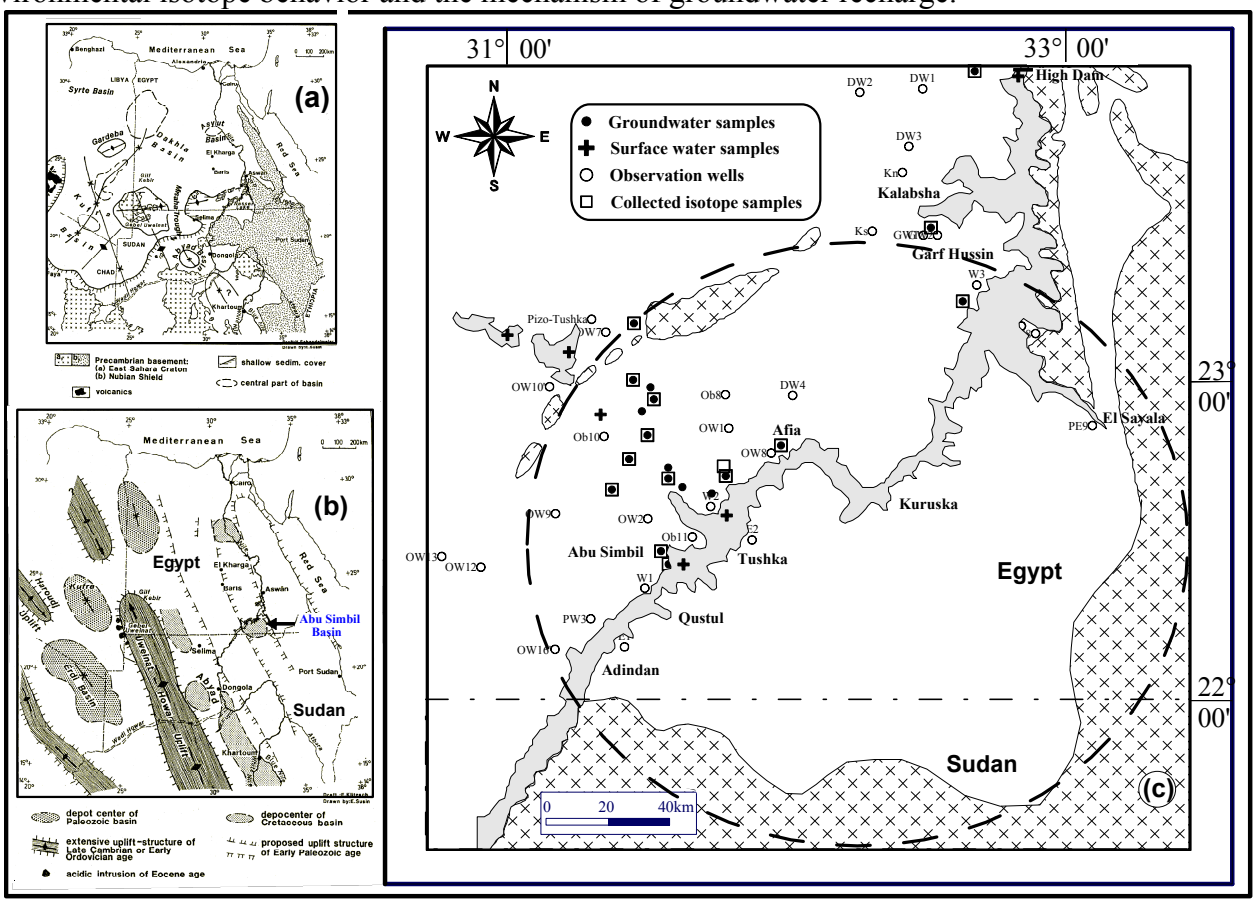


Figure 1. Northeast Africa intracratonic basins (a), Early Paleozoic Basins (b) (after Schandelmeier et al., 1987) and Abu Simbil Basin as well as location of the selected water and isotope samples (c).

2. DESCRIPTION OF THE AREA

ASB is located between latitude 21° 00\ and 24° 00\ N, and longitude 31° 00\ to 33° 20\ E (Fig. 1 c). It is quite interesting because of the varied rock types, the poly – phase tectonics and richness in mineral and soil deposits. It has high and rugged topography especially in the eastern and western limbs and decrease toward High Dam Lake. It lies in arid to semi-arid area, where the maximum temperature is 41.4 °C in August and the minimum temperature is 10.7 °C in December, the % humidity ranges from 39.5% in December to 19% in June. Wind speed reaches the maximum in September (3.7 m/Sec.) and the minimum in January and February (2.8 m/Sec.). The amount of rainfall is very little except sudden rain storms falling on the Red Sea Mountains (the eastern limb of the basin) and the evaporation varied from 29.2 mm/day in May to 5.4 mm/day in December.

Geomorphologically, ASB is divided into four units (Fig. 2) namely; high mountains, Nubian Sandstone Pedi-plain, structural plateau and lakes as follow:

1. High Mountains mainly occupy the eastern and southern limbs of the basin. They have an irregular topography which is varied from +400 m to +1000 m above sea level. They are composed mainly of igneous and metamorphic mountains that are represented the backbone

of the basin sediments. Isolated hills subunit is recorded as elongated shape in western (constitutes part of Aswan – Nakhali Uplift) and northern parts. They have irregular topography which range from about +300 m to +400 m.

2. Nubian Sandstone Pedi-plain constitutes large portion of the basin and is composed mainly of Abu Simbil (coarse to medium sandstone), Lake Nasser (siltstone), Sabaya (fine to coarse sandstone) and Abu Aggag (fine to medium sandstone) Fms that belonging to Cretaceous Period. The flatness of the sandstone pedi-plain is interrupted by other factors such as isolated flat tapped hills, isolated shallow basins and sand sheets that are accumulated as sand dunes in the southeastern part.
3. Structural plateau is recorded west of ASB and constitutes the southern part of the extended Sin El Kadab plateau. It has irregular topography that varied from about +150 m to about +500 m and is composed mainly of Paleocene-Eocene limestone.
4. Due to the construction of High Dam in 1970, the River Nile water storage in the low relief areas of Nubian Sandstone pedi-plain forming High Dam Lake (HDL) in the basin. On the other hand, Tushka Lakes (three) occupy the low relief areas of structural limestone plateau that varied from +140 m to +160 m above sea level. The water releases to these lakes through Khor Tushka when the water level in HDL reaches to +180 m.

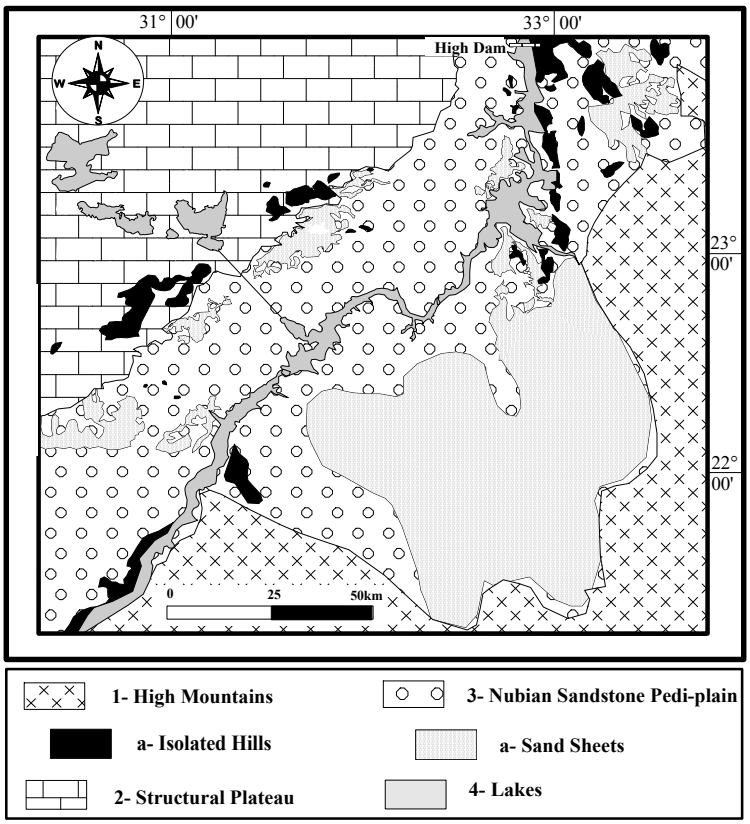


Figure 2 Geomorpologic map of the area of ASA.

The simplified lithostratigraphic description of ASB (Fig. 3) from the oldest to the youngest unit is given as follows:

1. The basement rocks occupy mainly the high mountains in the eastern and southern parts. They are composed mainly of ophiolitic mafic – ultramafic rock association, arc volcano and leucocratic rocks, while Aswan – Nakhali Uplift are composed mainly of arc volcanic rocks (gneiss and migmatic gneiss).
2. The Lower Cretaceous sediments (Abu Simbil, Lake Nasser, Sabaya Fms) arranged northward from the older to the younger and occupy large portion of the basin. Abu Simbil Fm sediments are outcropped in the southern part and composed mainly of coarse to medium sandstone with shale layers. The siltstone and claystone sediments of Lake Nasser

Fm are recorded to the south of Kor Tushka. El Sabaya Fm occupied large portion of the basin and is composed mainly of fine to coarse sandstone. Generally, the Upper Cretaceous sediments are restricted outside the basin in the northern part (Kurkur Basin) and represented by Abu Aggag (fluvial deposits with cross bedding sandstone), Temsah (marine shale with sand layers) and Kiseiba (clayey siltstone and fine sandstone with shale intercalations) Fms.

3. The Paleocene-Eocene sediments are outcropped outside ASB in the western part and represented by Kurkur (marl and marly limestone), Garra (limestone) and Thebes (Limestone with shale interbeds) Fms. The Quaternary sediments (gravel, sand, silt and clay) are recorded in the low relief areas around High Dam Lake.

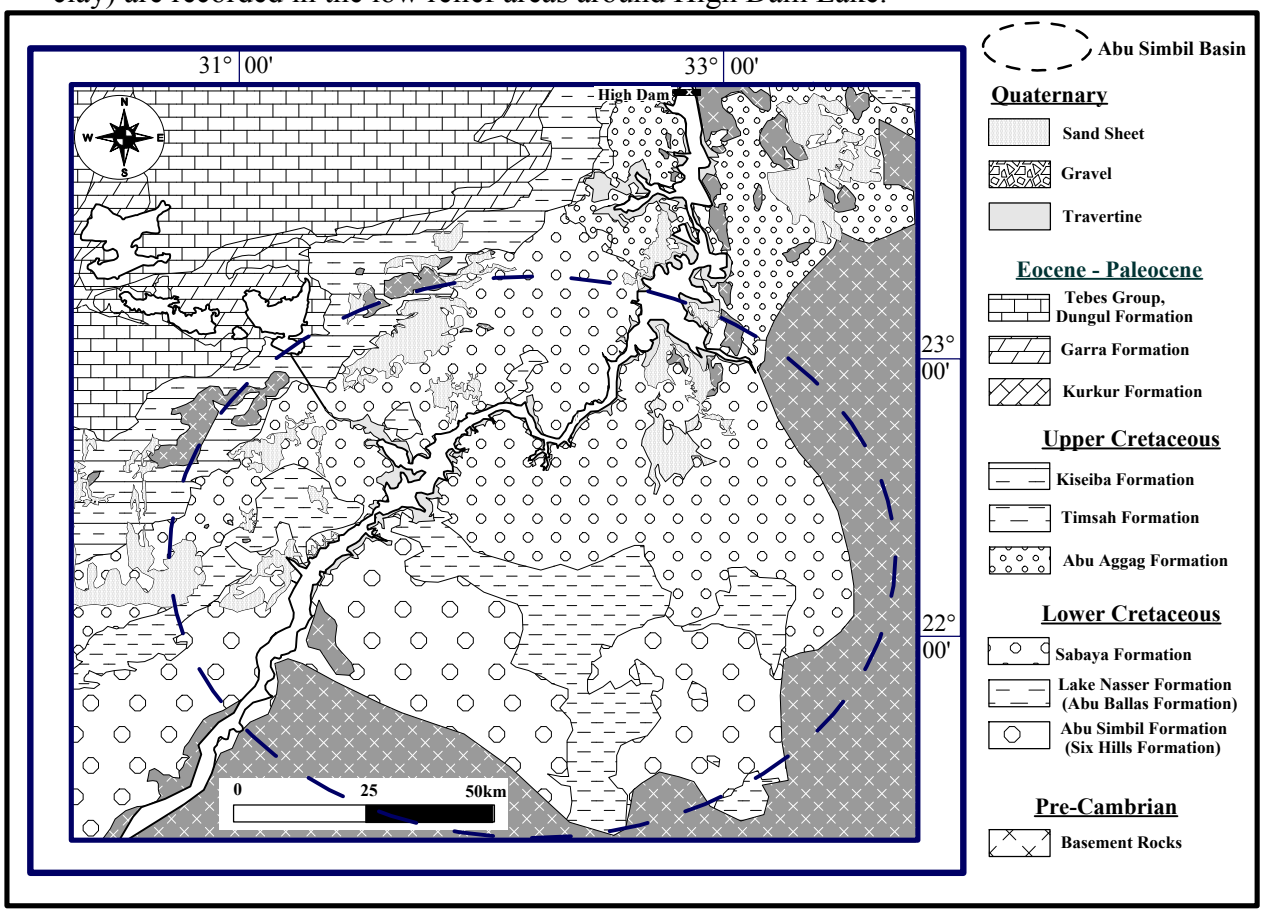


Figure 3. Geologic map of ABS (modified after GMRD, 1981 and Klitzsch et al., 1987).

3. APPROACH AND METHODOLOGY

Initially, in order to understand the geomorphology and surface geology of the ASB, different information from satellite images, geologic maps of GMRD (1981) and Klitzsch et al. (1987), and topographic maps were collected followed by processing, field observations and measurements. A lot of lithological description, well logging and composite logs data of the drilled groundwater wells were collected from Geoistrzivanja company (1962 and 1965), High and Aswan Dam Authority (1982 and 1984), Ahmed (1988 and 1996), Abd El Karim (1992), MPWWR (1997 and 2000), DASCO (1998), RIGWA (1998, 2000 and 2001), NWRC (2000), Salam (2001) and Mohamed (2005) to shed light on subsurface geology and hydrogeological condition.

To recognize the general variation in surface and groundwater chemistry of the study area, 26 water samples were collected during May 2006 and electrical conductivity (EC) and pH were measured. A Magellan global positioning system (GPS) was used to mark the location by noting down the latitude and longitude. Six water samples are represented all water bodies, three as

transect from south to north in HDL, and one sample from each of Zaid Canal, Tushka Lake No. 1 and No. 2. The groundwater aquifers are represented by twenty one water samples, fourteen (14) water samples from Sabaya aquifer (SA) and seven (7) water samples from Abu Simbil aquifer (ASA).

Three samples were collected from each location: (1) a filtered, acidified 100-mL sample for cation analyses; (2) an unfiltered, unacidified 125-mL sample for anion and alkalinity analyses; and (3) an unfiltered, unacidified 100-mL sample for stable isotope ratio analyses for hydrogen and oxygen. The groundwater samples were collected after pumping for 10 min. to remove groundwater stored in the well. In-situ measurements included EC, pH and bicarbonate which were measured using a portable field kit and titration respectively as per WHO (1996) recommendations as these parameters change with storage time. Major cations like Ca2+ and Mg2+ were analyzed by titration, while Na+ and K+ were measured on a AIMIL Flame Photometer (PE I). Chloride ion was analyzed by titration and the SO42- was measured by spectrophotometer. The chemical analysis was carried out as per the standard procedure given by ASTM (2002). After all sample analysis were completed, descriptive and multivariate statistical analysis were done using the software SPSS (SPSS Inc. 1999) on nine (9) hydrochemical variables (in this case EC, pH, Na+, K+, Ca2+, Mg2+, Cl-, HCO32- and SO42-).

Hydrogen and oxygen isotope ratios were determined by the methods of Coleman et al. (1982) and Socki et al. (1992), respectively. The isotope analyses (δ O18 and δ D) of the study area were done in the laboratory of the Department of Earth and Environmental Sciences, University of Illinois at Chicago, USA. The resulting sample values of δD and δ18O are reported in per thousand deviations relative to the corresponding ratios in Standard Mean Ocean Water (V-SMOW) (Coplen 1996).

4. RESULTS AND DISCUSSION

4.1 Hydrogeological Setting

a. Subsurface geology

Subsurface geology was necessary in the ASB for two major reasons: to delineate the groundwater aquifers thicknesses, facies and relationships, as well as from a geological point of view. The western and southern parts of ASB were classified stratigraphically by Klitzsch and Wycisk (1987) into Lakia, Abu Simbil, Lake Nasser, Sabaya, Abu Aggag, Temsah and Kiseiba Fms from bottom to top. The structure contour map at the top of basement rocks of ASB (Fig. 4a) shows the main center of the basin is located west Tushka with the maximum value about -410 m below sea level, while the northern Kurkur basin is shallower and has the maximum depth reach to -360 m west of High Dam.

According to Schandelmeier et al. (1987), from the Carboniferous to Lower Jurassic, ABS is located in the far east of mega main Selima continental basin in east Africa that received the eroded gravels and coarse sands from the surrounding basement rocks. These sediments constitute the base sediments of Nubian Sandstone (Lakia Fm). With the continuous sea regression during the Upper Jurassic and Lower Cretaceous, the terrestrial coarse to medium sandstone that eroded sediments from the eastern basement rocks are accumulated in the rejuvenated basin forming Abu Simbil Fm (Klitzsch and Wycisk 1987). Due to the difficulty to differentiate between Lakia and Abu Simbil Fms, the isopach map of them is plotted (Fig. 4b). It shows the marked effect of the structural elements at the top of basement and the center of deposition was shifted westward during the deposition with maximum thickness about 270 m south Tushka. The relatively huge thickness may be related to the rejuvenation along NW and E-W faults systems during deposition.

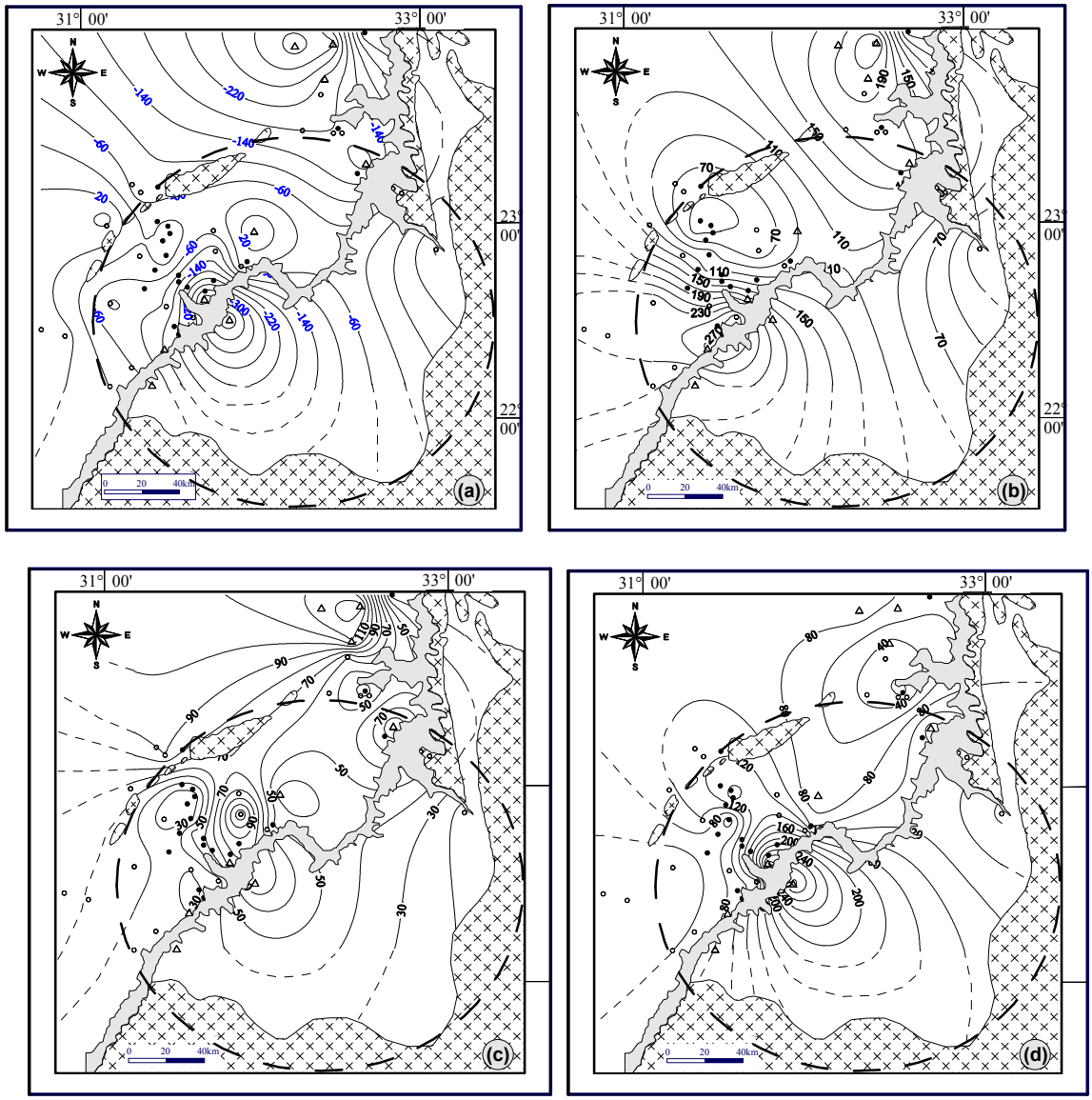


Figure 4. Structure contour map at top of basement (a), isopach map of Lakia and Abu Simbil Fms (b), thickness map of Lake Nasser Fm (c) and isopach map of Sabaya Fm (d).

During the Aptian Period (Lower Cretaceous), the sea transgressed southward until reached northern Sudan (Barthel and Bottcher 1978) and deposited Lake Nasser Fm of maximum thickness about 120 m north Tushka (Fig. 4c). It is composed mainly of claystones and siltstones with sandstone interbeds. The sea regressed northward during Albian due to the slight subsidence of the north Egypt (Klitzsch and Wycisk 1987). The eroded sediments (fine to coarse sandstone) from the surrounding basement rocks accumulated in the rejuvenated basin forming Sabaya Fm with maximum thickness about 310 m around Tushka (Fig. 4d). The area was affected by regional orogeny in the Upper Cretaceous that caused uplifting the southern part of the basin and restriction the Abu Aggag Formation in the northern part under sea regression during Turonian (Klitzsch and Wycisk 1987) which built mainly of coarse to medium sandstone with shale layers. With the continuous sea regression northward, Temsah Fm was deposited.

b. Aquifer system

Based on the available data of drilled wells, chemical analysis of the collected water samples (Fig. 1 and Table 1), hydrogeologic cross sections (Fig. 5) as well as literature, the groundwater aquifer of ASB are classified into Sabaya (SA) and Abu Simbil (ASA) aquifers from top to bottom. SA is outcropped in the central part of the basin, underlain Abu Aggag Fm in the northern part and

overlain Lake Nasser siltstone and claystone sediments i.e. unconfined to semi-confined. It has thickness ranges from few ten meters in the southern part to about 310 m west Tushka and composed mainly of fine to coarse sandstone with shale layers. It is recharged mainly from the southwestern part and from HDL through pores and fault systems. This appear in the relative high salinity of aquifer that varied from about 410 ppm to 813 ppm.

Table 1. Chemical and isotope analysis of the collected water samples.

No.	Cond. (µS/Cm)	TDS (ppm)	Ca^{2+} (mg/l)	Mg^{2+} (mg/l)	Na^{+} (mg/l)	K^{+} (mg/l)	Cl^{-} (mg/l)	SO_4^{2-} (mg/l)	CO_3^{2-} (mg/l)	HCO_3^{-} (mg/l)	δ 18 O% V-SMOW	δ D% V-SMOW
Surface Water												
1	230	123.3	16	2.5	26	5.7	7.3	12		108	2.6	20.4
2	250	130.3	19	5	22	5.1	9.23	10	7	106		
3	520	297.7	22	11	68	6.97	27.7	32	12	200		
4	260	139.3	18	3.5	28	6.32	14	10		119		
5	750	458.9	26	4	122	21.3	139	110		73.2		
6	430	254.3	26	2.4	55	13.3	18.46	72		134.2		
Sabaya Aquifer												
7	730	410.4	30	14	102	3.91	45	185		61		
8	750	436.9	32	17	100	3.91	129	88	6	122	-10.08	-81.4
9	750	441.6	40	5	111	5.09	122	93	5	121	-10.29	-82.4
10	785	450	64.1	22.2	57.8	11.4	140	51		207	-10.5	-78.5
11	850	503.3	34	14	122	6.32	161	111		110		
12	860	502.6	28	10	140	4.49	142	111		134.2	-10.2	-79.2
13	780	509.6	34	9	163	6.32	139	92.3	5	122		
14	850	514.5	32	12	130	5.7	180	106		97.6		
15	950	570.7	32	17	144	6.32	192	106.2		146.4		
16	1010	602.1	32	18	157	4.49	185	138.5		134.2		
17	1050	617.2	64	29	108	5.7	217	138.5		110	-10.2	-78
18	1180	669.9	109	29.3	85.2	6.4	248	104		176	-9.4	-72.5
19	1195	709.9	114	20.7	98.3	7.86	257	112		200	-8.1	-62.6
20	1330	813.3	44	15	220	5.7	280	184.6		128	-9.6	-77.8
Abu Simbil Aquifer												
21	1670	1001	76	20	244	6.32	268	338		97.6	-10.07	-81.1
22	2350	1410	84.8	56.4	320	12	128	641		335	-8.7	-70.1
23	2080	1257	145	40	233	4.49	286.1	500		97.6		
24	2200	1350	68	20	382	6.32	346	465		126	-9.1	-76.3
25	2370	1478	44	22	445	6.32	360	540		122	-9.14	-78.7
26	2730	1721	60	16	522	9.67	517	578		36.6	-8.87	-77.4
27	3170	1972	64	51	550	6.97	475	755		155	-8.15	-69.9

Because of the difficulty to differentiate between Lakia and Abu Simbil Fms and the approximate similarity of composition as well as hydraulically connected, they can be considered as one aquifer (ASA). It is outcropped in the southern part and overlain by Lake Nasser siltstone and claystone in the northern part (Figs. 3 and 5) with variable thickness from few ten meters in the southern part to about 270 m near Abu Simbil Town, and composed mainly of gravels and coarse to medium sandstone with shale layers. It is unconfined in the southern part to semi-confined in the northern part. It is recharged mostly from the southwestern part (Selima Basin) and partially from the HDL through pores and fault systems especially in the southern part where it is directly connected with High Dam Lake. Due to their lithologic nature and different sources of recharge, it has a wide range of salinity from about 1001 ppm to about 1972 ppm.

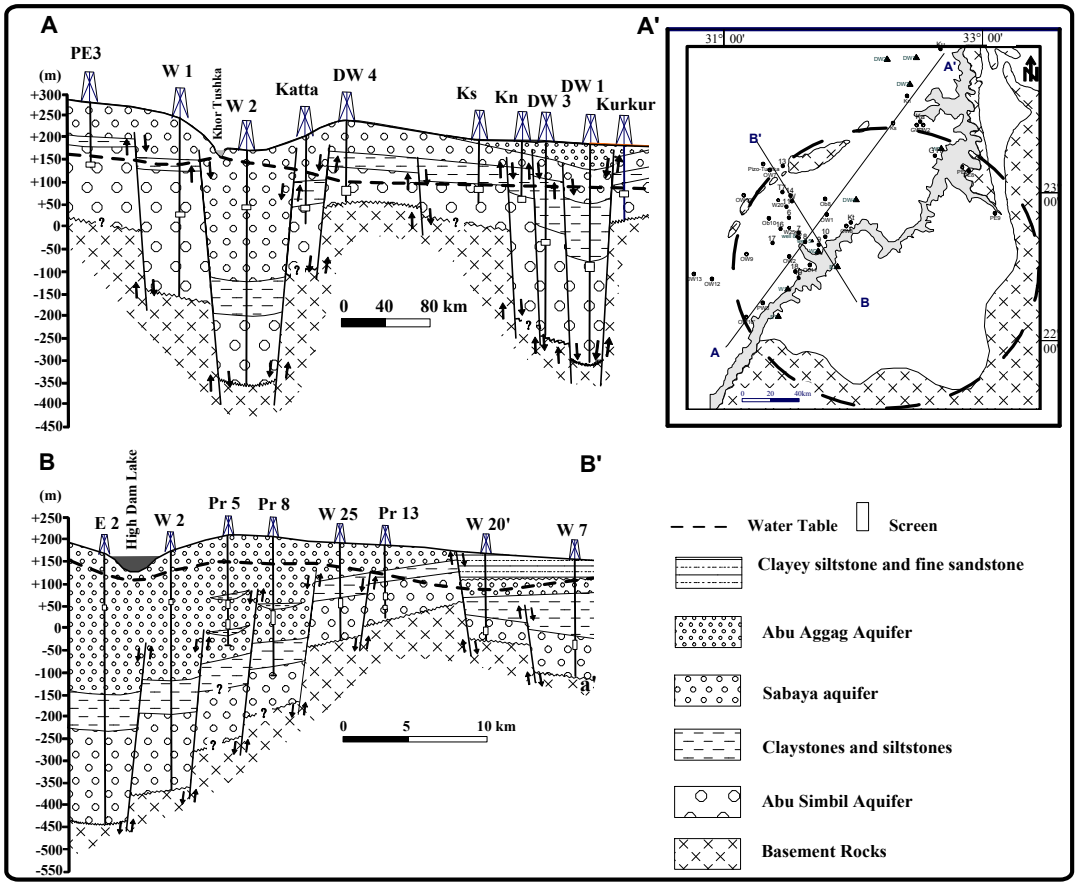


Figure 5. Hydrogeological cross sections of ASB.

Owing to the hydraulically connection between the groundwater aquifers through fault systems and facies, we trail to draw the water table map of the basin (Fig. 6 a). It shows the general water flow trend from the southwest to the northeast. There is also a local flow from HDL through pores and E-W faults, where the contour lines trending NW which is approximately vertical on the main groundwater flow. The narrow width in-between the contour lines are an indication of the relatively high rate of recharge from HDL and in contrast the wider width is an indication of the decrease of recharge. From the water table map, the impact of recharge from HDL varies from more than 10 km south of Abu Simbil Town to more than 30 km in the north. The groundwater salinity increases westward with the direction of local recharge from about 410 ppm near HDL to about 1970 ppm westward (Fig. 6b) which may be due to the effect of leaching processes.

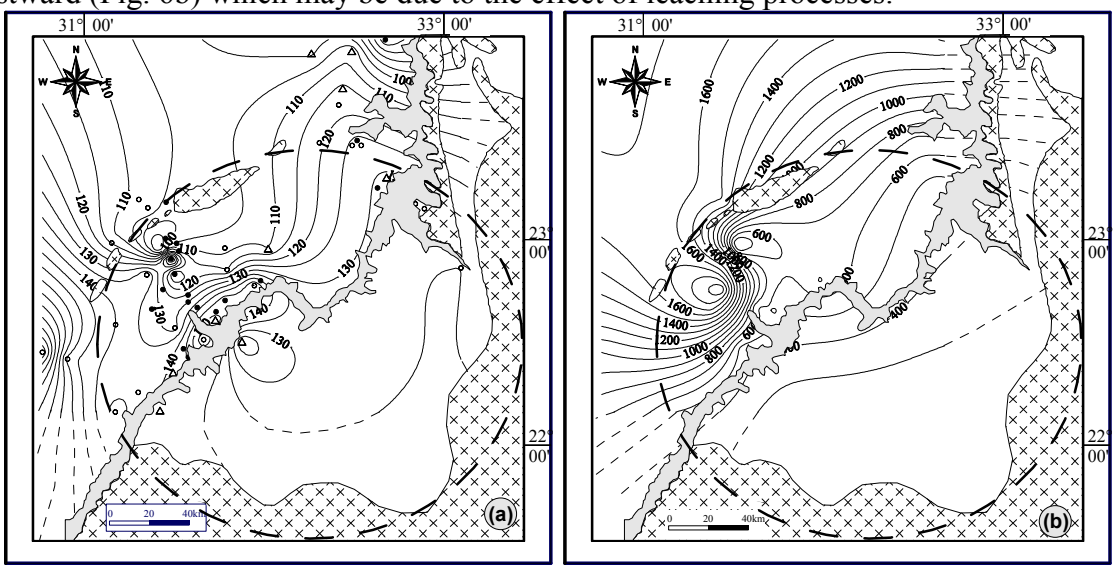


Figure 6. Water level (a) and iso-salinity (b) maps of Abu Simbil Basin.

c. Hydrogeochmistry

Water types have been classified on the basis of major ions using the Piper (1945) diagram (Fig.7). Different hydrogeochemical indicators pertinent to ground-water flow and circulation analysis are presented. There are three groups of water namely; surface water, SA and ASA. The surface water group is located in the western limb and it is the main source of recharge to SA and partially to ASA. It shows also the direct connection between SA and ASA.

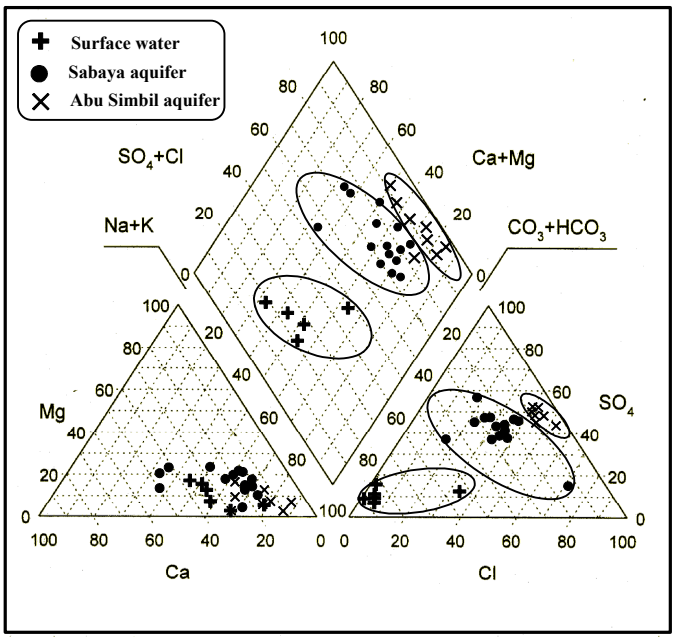


Figure 7. Piper plot of the hydrochmeical samples.

i. Surface water

The salinity of HDL samples changed from about 123 ppm near Abu Simbil Town (in the south) to about 297 ppm near High Dam (in the north) may be owing to the leaching processes and the effect of evaporation. Salinities of Zaid Canal (pumped from HDL at Tushka), Tuskha Lakes Nos. 1 and 2 (transported from HDL when the water level reaches to more than 180 m through spillway, where they are not feed by water from year 2001) are about 139, 459 and 254 ppm, respectively. The variation of the salinity of Tushka lakes may relate to the difference of the surrounding rocks and the rate of evaporation. Chemical hydrofacies of the surface water are essentially as follows: HCO3–Cl–Na–Ca (60%), and HCO3–SO4–Na–Ca (40%). The main salts are NaCl, Ca (HCO3)2 and CaSO4.

ii. Groundwater

1st. Geochemical evolution of groundwater and origin of salinity

The results are considered and interpreted according to a generalized southwest to northeast transect (main flow line). The samples are projected normal to the flow direction. This is a simplification to aid the interpretation. In reality, there may also existing flow path from the High Dam Lake. The evolution and the origin of salinity were investigated according to the distribution of chemical dissolved species concentrations. Non-conservative physico-chemical water parameters (pH, electrical conductivity and alkalinity) were measured in the field (Table 1). The physical and chemical parameters of the SA (shallow aquifer) indicated moderately large spatial variations. The electrical conductivity (EC) and total dissolved salts (TDS) of SA increased progressively from 750 µS/cm to 1195 µS/cm and from 410 mg/l to 710 mg/l along the flow lines (Fig. 8a), while ASA (deep aquifer) increased from 1330 µS/cm to 3170 µS/cm and from 813 mg/l to 1972 mg/l along the flow lines. The variation of aquifers values are related to the difference of aquifers facies and the groundwater flow mostly from the southwest especially in the deeper aquifer. Chemical

hydrofacies of the shallow aquifer are basically as follows: SO4–Cl–Ca–Na (62%), and Cl–SO4–Na–Ca (15%) due to the aquifer facies and the exchange reactions within the aquifer matrix, while in the deeper aquifer the first hydrofacies decreased into (43%), and the second one increased to (29%) as a long exchange reactions within the aquifer sediment.

2nd. The behavior of major elements

The spatial evolution of each major element concentration could be deduced from its relationship with the total mineralization (TDS) (Fig. 8b). There is a correlation with the total salinity showing an increase along the flow path for the whole of the major elements except for bicarbonate in deep aquifer (ASA) which has an inverse trend may due to some exchange reactions occurring within the aquifer matrix.

The major ion relationships with salinity in the shallow and deep aquifers are relatively straightforward. Chloride concentrations largely reflect the input conditions at the time of recharge and may be modified subsequently by inputs from formation waters or evaporites (Herczeg and Edmunds 1999). There is an increase of chloride and sodium ions concentrations along the flow path for the majority of the samples.

The plot of Na+ vs. Cl− (Fig. 9) shows that the representative points of shallow aquifer are distributed around the halite dissolution line, while of deep aquifer are close to the line. The Na/Cl weight ratio (Fig. 9) remains relatively constant around 1.5±0.5 which is consistent with stoichiometric dissolution of halite. This suggests a mix origin from different sources. However, some samples sometimes show a Na/Cl ratio that gets further away from unity. Those groundwater samples whose ratios are <1.5 generally correspond to waters coming mainly from the southwestern area. On the other hand, those water samples whose Na/Cl ratios are >1 match the samples collected in the northern part of ASB and in Kurkur basin. This fact may be explained by a sodium excess due to ion exchange between water and some minerals notably clayey ones that are presented within the aquifer matrix in that area. This is perfectly in accordance with the decrease of calcium concentrations with respect to salinity (Fig. 8).

Sulfate concentrations are relatively high in the eastern part close to the presumed recharge zone. They exhibit a relative variation along the flow path from 45 to 755 mg/l. These concentrations may be explained by exchange reactions occurring within the aquifer matrix during recharge. The very little SO4/Cl ratio in the east (0.8) increased slightly to 1.33 with the direction of flow. This indicates that the concentrations in these ions are monitored by their respective solubilities. On the other hand, high SO4/Cl values in the northern part that reach 3.9 may be related to the clayey layers that enriched by gypsiferous minerals. Calcium concentrations remain around 50 mg/l along most of the flow transects. Nevertheless, an increasing trend is observed towards the north with an Mg/Ca ratio which becomes <1. The latter may find its origin in a base exchange. Magnesium concentrations show a progressive increase and a good relationship both with sulfates and bicarbonates, which suggests that gypsum and calcite dissolve in parallel. Figure (9b) shows that Mg and SO4 of shallow and deep aquifers points lay between two end-members; the first end-member might be represented by shallow groundwater, while the other is the deep groundwater aquifer that is slightly recharged from HDL and where saline gypsum carbonated minerals are more present.

The samples show a good chemical homogeneity with SO4–Cl–Na and Ca respectively as the dominating species. The spatial evolution showed an increase along the flow path (east to west) for all major elements. In effect, the major ions exhibit a good relationship with the total salinity (Fig. 4b). The occurrence of flow path from southwest in the deep aquifer effect on the mineralization (gypsum carbonate) that increased from south to north in the flow direction.

iii. Statistical analysis

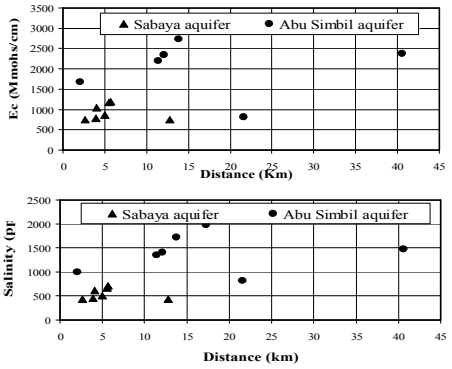
Numerical analysis of hydrogeochemical data has been attempted to determine the geochemical parameters of groundwater (Lawrence and Upchurch 1982). Correlation is widely used in statistical or numerical concepts for parametric classification of modeling studies (Balasubramanian et al.

1985). Statistical data generally provide a better representation than graphical data because (a) there are a finite number of variables that can be considered, (b) variables are generally limited by convention to major ions, and (c) superior relationships may be deduced by use of certain procedures.

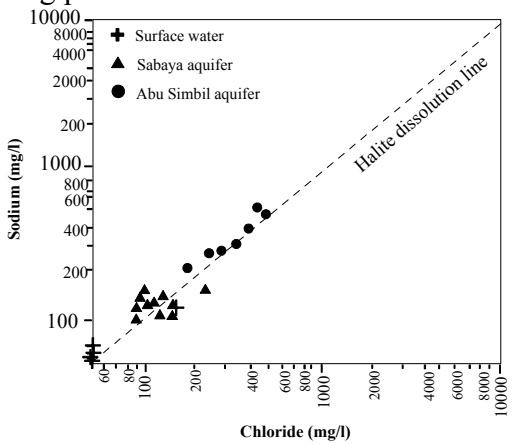
1st. Correlation matrix

The correlation matrix for surface and SA, ASA and both is shown in Table (2). Generally, it shows a good correlation for surface and SA than for surface and ASA, and the correlation for surface and all aquifers (SA and ASA) much similar for surface and SA. This may reflect the higher rate of recharge to the SA than ASA. It shows also, moderate good correlation for SA and ASA which is a signal for the connection in-between. Almost all analyses metals showed mainly good correlation with conductivity as it increased with dissolution of metals through ion exchange or oxidation- reduction in groundwater aquifers. TDS showed mostly good correlation with EC, Ca^{2+}, Mg^{2+}, Na^{+}; Ca^{2+}, SO_4^{2-} and Cl^{-}; Ca^{2+} with Mg^{2+} and SO_4^{2-}; Mg^{2+} with SO_4^{2-} and Cl^{-}; indicating anthropogenic input in groundwater and the effect of leaching processes

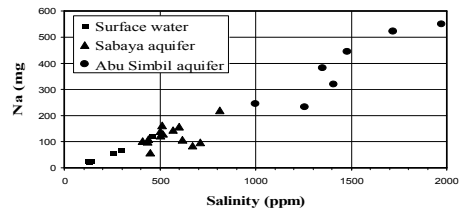
(a)

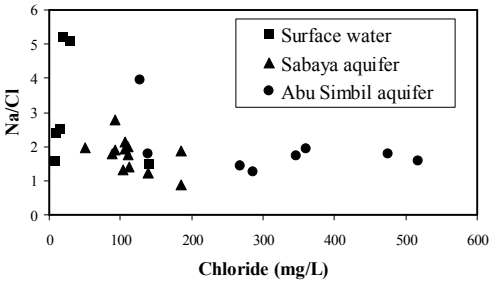


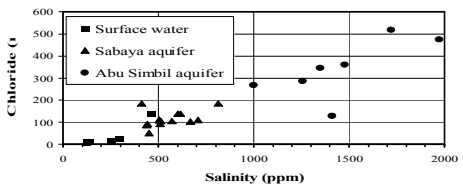
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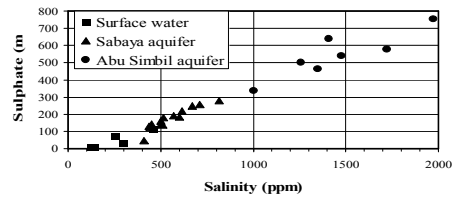


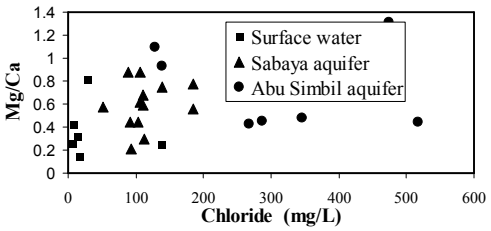
(b)

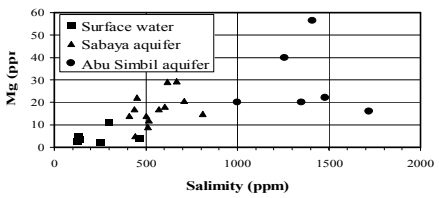


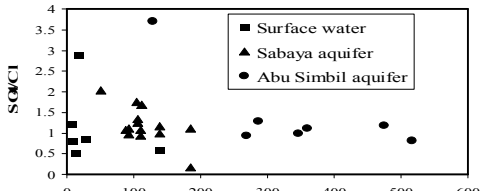












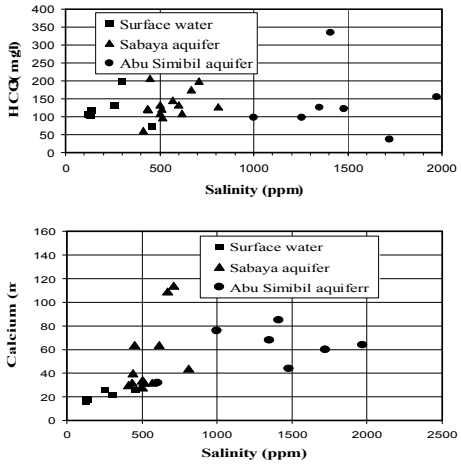


Figure 8. Relationships between both salinity and electrical conductance (EC) vs. distance to recharge zone. b Relationships between salinity and major ion concentrations (Na, Cl, SO_4, Mg, HCO_3, and Ca).

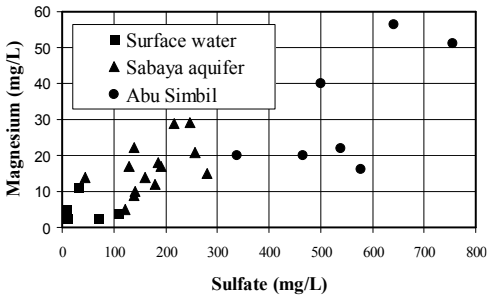


Figure 9. Relationship between sodium, Na/Cl, Mg/Ca and SO_4/Cl ratios and chloride (a), and magnesium vs. sulfate relationship (b).

Table 2. Correlation matrix.

	EC	TDS	Ca	Mg	Na	K	HCO_3	SO_4	Cl
a. Surface and Sabaya Aquifer (SA)									
EC	1								
TDS	0.996 **	1							
Ca	0.669 **	0.635 **	1						
Mg	0.745 **	0.705 **	0.750 **	1					
Na	0.769 **	0.809 **	0.087	0.271	1				
K	-0.091	-0.071	0.009	-0.239	-0.108				
HCO_3	0.25	0.228	0.560 *	0.426	-0.129	-0.008	1		
SO_4	0.959 **	0.961 **	0.697 **	0.720 **	0.704 **	-0.068	0.299	1	
Cl	0.804 **	0.807 **	0.281	0.467 *	0.814 **	-0.072	-0.289	0.659 **	1
b. Surface and Abu Simbil aquifer (ASA)									
EC	1								
TDS	1.000 **	1							
Ca	0.675 *	0.661 *	1						
Mg	0.796 **	0.782 **	0.747 **	1					
Na	0.972 **	.977 **	0.501	0.660 *	1				
K	-0.116	-0.11	-0.222	-0.143	-0.085	1			
HCO_3	0.116	0.098	0.115	0.562 *	0.012	0.037	1		
SO_4	0.989 **	0.987 **	0.705 **	0.860 **	0.940 **	-0.105	0.216	1	
Cl	0.919 **	0.927 **	0.544	0.523	0.950 **	-0.121	-0.265	0.859 **	1
c. Sabaya (SA) and Abu Simbil Aquifers (ASA)									
EC	1								
TDS	0.404	1							
Ca	0.659 **	0.597 **	1						
Mg	0.945 **	0.111	0.434 *	1					
Na	0.379	0.282	0.434 *	0.289	1				
K	0.095	0.304	0.578 **	-0.076	.615 **	1			
HCO_3	0.985 **	0.451 *	0.741 **	0.901 **	0.434 *	0.227	1		
SO_4	0.883 **	0.194	0.317	0.924 **	0.106	-0.358	0.794 **	1	
Cl	0.134	-0.157	0.29	0.161	0.530 *	0.686 **	0.247	-0.21	1
d. Surface and both aquifers (SA and ASA)									
EC	1								
TDS	0.999 **	1							
Ca	0.562 **	0.542 **	1						

Mg	0.751 **	0.732 **	0.697 **	1					
Na	0.955 **	0.964 **	0.313	0.566 **	1				
K	0.035	0.041	-0.022	-0.029	0.038	1			
HCO_3	0.12	0.102	0.292	0.531 **	-0.028	0.158	1		
SO_4	0.988 **	0.985 **	0.594 **	0.805 **	0.921 **	0.057	0.218	1	
Cl	0.911 **	0.918 **	0.387 *	0.485 *	0.939 **	-0.024	-0.272	0.844 **	1

** Good Correlation * Moderate good correlation

3.2 ENVIRONMENTAL ISOTOPES

a. Stable isotope geochemistry in ASB

Hydrogen and oxygen isotope ratios are ideal tracers of the origin and evolution of groundwater because they compose the water molecules and being sensitive to physical processes such as atmospheric circulation, groundwater mixing and evaporation (Clark and Fritz, 1997; Dansgaard, 1964). It is used to estimate the contribution of different groundwater to the water budget. The global meteoric water line (MWL) of Craig (1961) and paleometeoric line of Sonntag et al. (1978) were considered as a reference line. The point representing the isotope composition of Nubian Sandstone of the Western Desert and Gulf of Suez are of Sturchio et al. (2004), while the Eastern Desert and Wadi Qena are of Hamza et al. (1999). The present local precipitation of Egypt given by Nada et al. (1995) contain (δO^{18}= -2.6, δ D= +11.9) for comparison.

Fifteen water samples for stable ratios of hydrogen and oxygen analysis have been collected to represent water resources in ASB (Table 1 and Fig. 10); one sample from HDL, 8 samples from SA (shallow aquifer) and 6 samples from ASA (deep aquifer). The groundwater samples have a relative good distribution in the entire basin to determine the impact of HDL recharge on the groundwater aquifers. They have a wide range in hydrogen and oxygen ratio, from depleted composition of δ D = -82.4% and $\delta\ ^{18}O$ = -10.5 in southern part to less depleted composition of δ D = -62.6% and $\delta\ ^{18}O$ = -8.1 in the northern part. Table (1) showed the following:

1) SA samples are isotopically depleted where they have δ D values ranging from -82.4% to -62.6%, while ASA samples are less depleted that have δ D values varying from -81.1% to -69.9%.
2) SA in the southern part is more depleted than the northern one may due to the impact of Lake Nasser mudstone and siltstone that decreased the recharge from High Dam Lake.
3) The relative similarity of isotope values in ASA may due to the main source of recharge from southwest and the recharge from HDL occur in the southern part that characterized by coarse texture around the lake. The less depleted values of the aquifer are recorded near E-W fault system which increases the rate of recharge from the lake.
4) Generally, SA isotopic composition has relatively different isotopic composition (somewhat less isotopically depleted) than in the Western Desert, while ASA isotopic compositions are less depleted than SA (Fig. 10). They are located in a geographic trend of paleo-precipitation stored in the Nubian aquifer that is recorded by Sturchio et al. (2004) and the water samples near E-W fault system are less isotopic depleted due to the recharge from HDL (δ D = 20.4% and $\delta\ ^{18}O$ = 2.6).

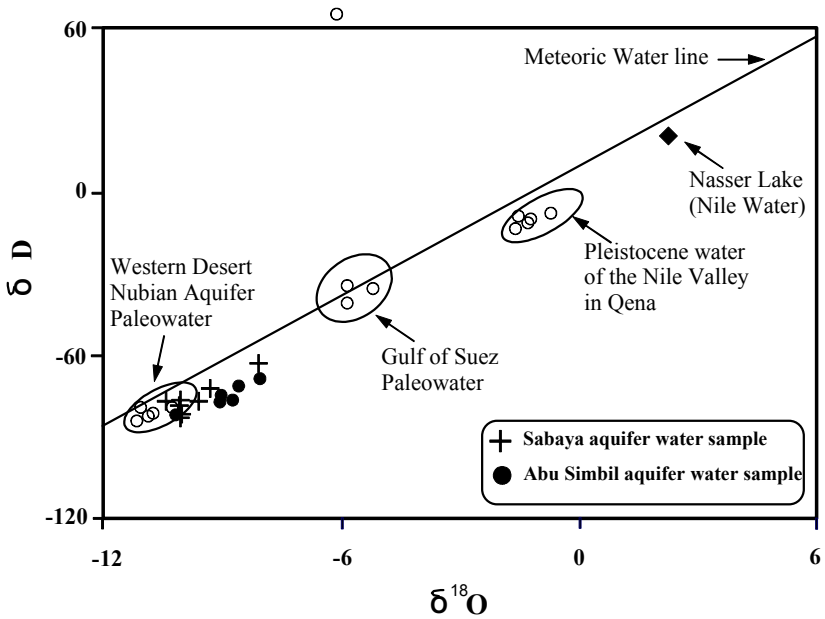


Figure 10. Stable isotope ratios of hydrogen (δ D) vs. oxygen (δ O^{18}) in Abu Simbil Basin. Samples from other studies of Western Desert, Gulf of Suez and Qena are shown as open circles.

i. Mechanism of recharge

By applying different factors that controlling the mechanism of recharge from HDL to the groundwater aquifers (Table 3), the following remarks can be obtained:

1) The main factors for recharge are pores media and E-W fault system.
2) The high rate of recharge to SA through pores reaches to 5.5 km from the lake that extended to less than 30 km near W-E fault system.
3) The direct recharge through pores is more than 17 km from the lake in ASA and extended to about 40 km near W-E fault system.
4) The variation of the effect of surface water (lake) on the surrounding aquifers may due to the difference of aquifer sediments, where ASA sediments are mostly coarse sediments and SA sediments are more finer which decrease the rate of recharge.
5) The degree of mixing is obviously in the shallower saturated thickness than the thicker type.
6) From the depth of screen, the degree of mixing with the shallower layers from surface water is higher than the deeper layers.
7) E-W active fault system increases the connection between the lake and the groundwater aquifers to about 40 km.

Table 3. Different factors that controlled the rate of mixing between HDL and the groundwater aquifers.

No.	Salinity (ppm)	d ^{18}O	d D	Depth to basement (m)	thickness Sediment (m)	Saturated thickness (m)	Screen depthfrom sea level (m)	W ater Level m(m)	Connection between first and second aquifers	Distance from HDL by (km)	E-W Faults
Sabaya Aquifer											
8	437	-10.08	-81.4	-190	410	330	60	145	Slightly yes	2.675	No
9	442	-10.29	-82.4	-167	385	310	70	143	Slightly yes	12.809	No
10	450	-10.5	-78.5	-165		297	41 0	132	Yes	3.952	Yes
12	503	-10.2	-79.2	-220	410	369	121 96 52	149	Slightly yes	4.532	No

	691	-10.0	-79.3	-20	210	120		100	No	27.354	Near
17	617	-10.2	-78	-180	380	322	124	142	Slightly yes	4.065	No
							67				
							37				
19	710	-8.1	-62.6	-5	235	145	41	140	Yes	5.712	Yes
18	670	-9.4	-72.5	-180		306	6	126	Yes	5.516	Yes
20	813	-9.6	-77.8	11	223	102	59	113	Yes	21.605	Far from
							40				
Abu Simbil Aquifer											
21	1001	-10.07	-81.1	-130	320	260		130	Slightly yes	2.031	No
22	1410	-8.7	-70.1	15	170	61	55	76	Yes	12.089	Far from
24	1350	-9.1	-76.3	5	183	103	63	108	Yes	11.383	Far from
							25				
25	1478	-9.14	-78.7	-80	260	188	78	108	No	40.608	Near
26	1721	-8.87	-77.4	-70	290	200		130	Yes	13.87	Yes
27	1979	-8.15	-69.9	-40	230	165	100	125	Yes	17.319	Yes

5. SUMMARY AND CONCLUSION

There are four mega intracratonic basins and five small basins in NE Africa. Abu Simbil Basin (ASB) is one of them and did not extensively studies may due to the less potentialities from oil and minerals point of view. The newly accumulation of Nile flood after the construction the High Dam Lake (HDL), the intensive drilling of groundwater wells and the occurrence of relatively good soil, encourage Egyptian and Sudanese governments for launching a major program for sustainable development of the basin. Water resources are of utmost importance to any development, thereby this article deals with identification of the groundwater aquifer system and their relationships with the HDL utilizing data sets of images, geological, hydrogeological, hydrochemical and isotope hydrology information.

There are three hydrofacies groups of water types namely; surface water, Sabaya aquifer (SA) and Abu Simbil aquifer (ASA). SA and ASA aquifers constitute the lower part of the meg-Nubian Sandstone aquifer system in NE Africa. They are hydraulically connected through pores and fault systems. They are recharged basically from southwest (the main trend of recharge in the meg-Nubian Sandstone aquifer system), while the newly introduced lake (HDL) changed the groundwater flow around it to become perpendicular on the main flow. The amount of surface water recharge varies from one aquifer to another, where it is relatively high in SA (shallower aquifer) and their groundwater salinity ranges from 410 ppm near the lake to 813 ppm westward (with the direction of local recharge). The salinity of ASA is greater than that of SA where it varies from 1001 to 1972 ppm may due to the aquifer matrix and the relatively low recharge from HDL.

The chemical hydrofacies of the surface water are essentially HCO_3–Cl–Na–Ca (60%) and HCO_3–SO_4–Na–Ca (40%). However, the main hydrofacies in SA (shallower aquifer) are SO_4–Cl–Ca–Na (62%), and Cl–SO_4–Na–Ca (15%) hydrofacies, while in deeper one (ASA) the first hydrofacies decreased into (43%), and the second increased to (29%) may as long exchange reactions within the aquifer sediment and a smaller amount of recharge from HDL. The behaviors of major ions in the collected water samples support a mix origin of groundwater aquifers. The resulted chemical analyses of the three water types are statistically analyzed using SPSS program for testing the collected data and define the relations between water resources. There is a good correlation for surface water and SA than for surface and ASA which means the advanced recharge from HDL to AS than ASA. There is a moderate good correlations for SA and ASA which is a signal for the connection in-between, while the correlation for surface water and groundwater aquifers (SA and ASA) relatively similar for surface and AS which support a small amount of surface recharge to ASA.

The groundwater of SA is isotopically depleted where the δ D values ranged from -82.4% to -62.6% and δ ^{18}O vary from -10.5 to -8.1, while ASA is less depleted and the δ D values vary from -81.1% to -69.9% and δ ^{18}O range from -10.07 to -8.15. SA and ASA are less isotopically depleted than in the Western Desert of Egypt by reason of the recharge from HDL. By applying different

factors that controlling the mechanism of recharge from HDL to the groundwater aquifers, the main factors for recharge are pores media and E-W fault system. The recharge of surface water through pores in SA reaches to 5.5 km from the lake that extended to 30 km near W-E fault system, while increased to about 17 km in ASA and extended to about 40 km through E-W fault may due to the coarse texture of aquifer sediments of ASA than SA.

ASB requires a lot of piezometric and production wells especially in the eastern and northern parts for testing and re-evaluating the aquifers geometry and hydraulic characteristics as well as distinguish the impact of lake fluctuation on the existing aquifers. Also detailed hydrological studies for determination the quantity of surface water recharge from HDL to the groundwater aquifers and impact of climate changes on the HDL storage must be done, at the side of groundwater storage estimation and building suitable plan for groundwater utilities in different purposes.

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INTEGRATED WATER RESOURCES MANAGEMENT THROUGH ACTIVE STAKEHOLDER DIALOGUE – CASE STUDY FROM PALESTINE

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Abstract: Implementing an IWRM process is in fact, a question of getting the three pillars "right" moving towards an enabling the environment of appropriate policies, strategies and legislation for sustainable water resources management and development, putting in the place the institutional framework through which the strategies and policies can be implemented and setting up the management instrument required by these institutions to do their job, however in order to put a strong ,sustainable platform for all pillars a high consultation and institutionalized dialogue among all stake holders is very necessary. The current paper will highlight some of the practical methodologies that has been followed to facilitate stakeholder dialogue in the Northern part of the West Bank. Moreover, it will show how such dialogue could improve good local water governance in that area. And feeding the concept of integrated water resources management

Key words: IWRM. SMART, STAKEHOLDERS, PARTICIPATORY APPROACH

1. THEORETICAL BACKGROUND OF IWRM

At the heart of Integrated Water Management is public participation, access to just process and participation of all stakeholders (CIS, 2003; UNECE, 1998; UNESCO, 2002, 2003, 2006; GWP, 2000, 2004). Public participation is both central to the implementation of the IWM and for the construction of complex societal problems. Just as there is no detailed understanding of, and objectives for IWM, so the definition and objectives of public participation, remain ambiguous. The meanings of both 'participation' and 'stakeholders' are changing (very unstable definitions) and evolve as people are involved. Many academics and practitioners distinguish public participation, stakeholder involvement or engagement based on the extent to which citizen or stakeholder are empowered in the final decision-making process (Collins & Ison, 2007; Forrester,1999a, 1999b; House, 1999; Rowe & Frewer, 2000, 2004, 2005; Wilcox, 1994). Others (Beierle & Cayford, 2002; Glass, 1979; Kessler, 2004; Parenteau, 1988; Ridder et al., 2005) emphases that the meaning of participation, involvement and engagement overlap and are used interchangeably to "denote a process by which individuals and groups come together in some way to communicate, interact, exchange information provide input around a particular set of issues, problems or decisions, and share in decision-making process to one degree or another" (Ashfort & Rest, 1999, p.3). Since there is no consensus on the meaning of those terms they are used in this study interchangeably (Kerrault, 2008). A similar semantic confusion surrounds the terms of stakeholders, citizens, the public, the community. English et al. (1993) and Ridder et al. (2005) distinguish stakeholder from the public by pointing out that the latter are unorganized, as opposed to stakeholders who are organized either around industrial interests or through unions and association. According to Kessler (2004) the community often includes not only those relying on the protection of the resource in need to protection, but extends to all of those interested or affected by environmental management. Clearly, the public, the community, citizens and stakeholders overlaps especially in IWM. In fact, the

boundaries of who has an interest in the decision-making process or who will be Implementing an IWRM process is in fact, a question of getting the three pillars "right" moving towards an enabling the environment of appropriate policies, strategies and legislation for sustainable water resources management and development, putting in place the institutional framework through which the strategies and policies can be implemented and setting up the management instrument required by these institutions to do their job (Clausen, 2004).

Setting water policy and budget priorities are one area for a more inclusive approach. Greater public participation and transparency in budget-setting activities can contribute to a more equitable distribution of resources for the poor." (Sohail and Cavill 2008, Tamimi 2008).

2. WHO ARE THE STAKEHOLDERS

Stakeholders are individuals, groups or organizations who, directly or indirectly, stand to gain or lose from a given development activity or policy. Distinction is drawn between:

- Primary stakeholders who are directly affected and would include the principal project beneficiaries
- Secondary stakeholders who are indirectly affected
- Key stakeholders who are the agents of change. These are often also "primary" stakeholders.

We undertake stakeholder analysis to:

- identify stakeholders' interests in, importance to, and influence over the intervention
- identify local institutions and processes upon which to build, and
- provide a foundation and strategy for participation

In case of Palestine, the most important player in the institutional building processes are the donors, however the fragmented, scattered uncoordinated agendas of the donors impacted very negatively the integrated water sector management (Tamimi, 2007)

The reform process suggested by different donors is not efficient because the above mentioned reasons and the different approaches to achieve the reform.

The global report on corruption mentioned clearly, a transparent and a good governance water institution will help in fighting corruption and empowering poor.

"A variety of innovative initiatives show how empowerment can translate into greater participation and a more powerful voice for the all stakeholders. At the same time, special efforts are needed to overcome the traditional exclusion of women and other vulnerable citizens from participatory processes. Their inclusion in activities needs to be targeted and a common respect created for their contributions (Sohail and Cavill, 2008).

3. ANALYSIS OF STAKEHOLDERS INVOLVEMENT

Methods of stakeholder analysis:

- Best done in collaboration with key stakeholder groups
- Not a desk study – use participatory methods such as stakeholder workshops and local consultations
- Use available and reliable secondary data to reduce costs

4. STAKEHOLDERS INVOLVEMENT APPROACH BASED OPTIMAL MANAGEMENT

4.1 Resources Scarcities

Scarcities, if not managed properly, bring conflicts that are not easy to resolve. This is especially so in an institutional environment where centralized and top down management is compounded by lack of stakeholders' involvement, difficult communication and fragmentation of responsibilities among many actors. (Leban et.al, 2006).

4.2 Different Perception and Interests

Conflicts are not necessary related to situations of war; they can also reflect situations in which competing actors have simply different interests they want to defend. In many cases such conflicts of interest have resulted from the different perception on the same issue. In some certain cases these conflicts of interest can be reasonably dealt with through direct negotiation among competing actors. However, in situations of war or occupation the margins of negotiation are often extremely narrow or non-existent.

In the water sector, as in many other domains, apparent conflicts may be resolved by bringing actors with different views together and have them discover the reasons behind each others opinion or even recognize that the other was right. The statement that “farmers are ignorant and have bad habits in water use” is often mirrored by “government officials have no idea what we are talking about”. Women may look to water mainly for its use for her household, while men may give first priority to it for their fields. Such different perceptions are two sides of the same coin. They form a polarity or two extremes of the same subject. Yet, any individual will not see both sides of this same coin or the two extremes of the same subject at the same time without assistance from someone else or without using proper tools.

Therefore, the need for more integrated and co-coordinated approaches towards water allocation, use and management is much more than what are currently practiced. Solutions to current and future water crises will most probably not be found anymore in technological advances or supply oriented approaches alone. The trend now is more toward integrated water management approach which, recognizes the needs to involve all actors at the different levels in a constructive dialogue. Such dialogue needs to be facilitated through the development and/or adaptation of adequate tools that assist them in reaching appropriate compromises (Tamimi 2008, Laban et al. 2006).

5. FACILITATION STAKEHOLDER DIALOGUE

Facilitating “vertical” and “horizontal” communication, coordination between all relevant actors (including local communities), recognizing different perceptions, negotiating conflicting interests, finding win-win solutions, in a specific domain, requires interactive dialogue among all actors. Such dialogue often requires also the skills of a professional mediator with no direct stakes in the issue to be discussed. A person or organization who is able to help others to listen to each other, to help actors to find common ground where possible; and if not, to come to reasonable compromise. Experience in many situations has shown that good process facilitation can contribute to better planning and implementation of viable and sustainable activities, facilitation here refers to mediating and guiding “brokering” processes in planning and decision making between government agencies (officials), local communities (end-users) and other relevant parties.

Process facilitators play a key role in stakeholder dialogue processes that aim to reach common goals and shared action plans. In such processes the different interests, perceptions, preoccupations, assumptions, and judgments among the actors involved, are made explicit. Opportunities are

identified to improve the exchange of information, social organization, and decision-making between stakeholders in order to create the proper conditions for innovation and better (water) management. At the same time this may contribute to create awareness with respect to constraints and opportunities that affect the performance of relevant actors. A stakeholder dialogue will identify potential actors who do or could act effectively together to remove constraints and make use of new opportunities. Such a process often enhances institutional and technological innovation through active networking, involving all relevant actors including community members, governments, NGOs, academic institutions, and the private sector (*Empower* project files, 2003-2006). *Empower* is EU funded project implemented in Jordan, Egypt and the Palestinian territories. The project's main objective is to increase the involvement of stakeholder at all levels of water management.

Moreover, facilitation should pay special attention to the less advantaged and poorer segments of the communities and actors. Every community has its own socio-economic configuration determined by culture, wealth, gender, land tenure, access to resources, etc. In most if not all communities there will be groups that are more vulnerable, have less resources and therefore, less influence in decision-making.

Involving different stakeholders at different levels, from national policy makers to end-users in local communities, does not happen by itself and is certainly not an easy job. It will require expertise and capacities to facilitate complex processes that pro-actively involve all these various stakeholders in planning, decision-making and implementation of water related issues. This is even more important where such processes should enable end-users and organizations at the community level to take larger shares and ownership in the management of local water resources.

Involvement stakeholders in all levels of project cycle does not mean to make every body happy and to meet all stake holders demand, in other words, stake holders involvement is a right based process (right to access information, to participate, to influence) but not only demand based involvement, dancing together doesn't mean going home together.

6. FROM THEORY TO PRACTICE

The *Empower Project* as a case study (The Palestinian Partner is Palestinian Hydrology Group):

Step One: Understanding the external environment of the project

The Middle East is one of the most water-poor and water-stressed regions of the world. While the region is home to 5% of the people of the world, it has less than 1% of its renewable fresh water. Today's annual per capita availability of fresh water in the region is only one third of its 1960 level, falling from an average of 3,300 cubic meters/year per person in 1960 to less than 1,250 cubic meters per person in 1995 (World Bank, 1996). This is the lowest per capita water availability in the world. It is estimated that by 2025, this limited availability is likely to drop to less than half the 1995 level. This average data, low as it is, is moreover misleading. With the possible exception of Iraq, Turkey and Iran, all Middle East countries have average per capita renewable fresh water availability levels even below the average of 1,250 cubic meters. Some of the Arab Gulf countries and the Palestinian West Bank & Gaza have per capita availability averages that are below 10% of the regional average of 1250 cubic meters, and even a country that is considered to be relatively water endowed like Lebanon shows an average of 1,200 cubic meters. Water scarcities are quite different in each country and are affected by many factors. In the three countries where the EMPOWERS Partnership Programme is implemented the situation can be summarized as follows:

Jordan and Palestine (West Bank/Gaza) have some of the world's lowest per capita availability of water. According to similar estimates (Arab Countries Vision 2000) water availability in Jordan is 169m3/year and in the West Bank is 78 m3/year. It is predicted that in Jordan this availability will further fall to 91m3/year in 2025 - even if all unconventional water sources are used - due to doubling of the population. At present there is said to be some availability of additional surface water, but most renewable groundwater reserves are fully exploited. Predictions are that by 2025

water supplied will exceed available renewable resources by 33 %.The result is that with increasing competition between rural (irrigation) and urban (drinking water) use, there is increasing focus on unconventional sources such as wastewater re-use and the improvement of demand management.

In the West Bank/Gaza water scarcity is further aggravated by Israel's excessive control over water resources. Moreover, Palestinians have not got a chance to manage their own resources and public affairs over the past nine decades. There has been always a foreign authority in power who was ruling. The most critical one has been the Israeli Occupation. During this time management of public resources including water have been completely within Israeli hands; decisions were made without Palestinian participation, and with minor or no regard for Palestinian need.

This in turn has created a large gap between the ruling power and Palestinian Public. Furthermore, it developed a distrustful, often antagonistic relationship with public authorities. Respect for public goods and public management decisions started to be seen as if it represents acceptance of, or agreement with the ruling power (the Israeli occupation).

This all has left a fragmented and not coordinated public service sector including water sector. Water supply management has been characterized by top down management approach with little or no room for public participation with no clear division of roles and responsibilities of the different bodies. Yet, the creation of the Palestinian Authority has improved the situation to a certain extent. However, there is still no adequate legal framework that ensures public participation in managing local resources. In this context, it is not easy to overcome the values inherited from the long term impact of occupation about this issue.

Involving different stakeholders at different levels, from national policy makers to end-users in local communities, does not happen by itself and is certainly not an easy job. It will require expertise and capacities to facilitate complex processes that pro-actively involve all these various stakeholders in planning, decision-making and implementation of water related issues. This is even more important where such processes should enable end-users and organizations at the community level to take larger shares and ownership in the management of local water resources.

In Gaza, current water use already exceeds the magnitude of renewable water resources by a factor of two. This requires a critical re-assessment of the current water use patterns and requires the development of proper scenarios to mitigate the current deterioration of the water sources in Gaza.

In the West Bank access to water is limited and carefully controlled by the Israeli Occupation. Although actual groundwater resources would amply satisfy demand of the West Bank itself, more than 80 % of the West Bank renewable water resources are used by Israel.

It is estimated that nearly 15% of the population in the West Bank have no access to piped water supply yet. More than 40% of the villages are not connected despite the fact that many of them demanded to be connected during the occupation period. They were always denied such access. On the contrary, all Israeli colonies in the West Bank are connected to water and many have swimming pools. This disparity in water supply has always been a source of tension. When the Palestinian villagers seeing the pipe that leads to an Israeli Colony passing through their land without supplying the village with water, they will not stay calm. Not only that, sometimes Israeli Army uses water to punish some villages by cutting the supply off for some days.

The situation is not much better in the case of the communities purchasing their water from the Israeli company (Mekorot). During, summer periods, water is reduced by various percentages in each area, it might goes up to 70% in some places. Some cities may have water once every week or month even. The same applies for some towns and villages too. This forces people to fetch water from unprotected sources which leads to increasing public health hazards and increase in the percentage of communities effected by water born diseases.

On the other hand, Egypt has with 920m3/year a relatively high per capita availability of water - in comparison to other countries in this region. However, demand reaches or exceeds the amount available because of low irrigation efficiency in most areas. It is not only dependent almost entirely on water originating from outside the country (98%), but also many farmers are dependent on the

inefficiencies of others. Moreover, over 25% of water is estimated to be applied twice to land, while 50% of farmers depend to some degree on drainage water El Zanaty Associates, 2001). Excess application may be targeted as an element requiring attention, but the consequences of improving application efficiency will need careful assessment if downstream drainage users are not to suffer, both from reductions in flow and deterioration of water quality. The additional factors of increased volumes and generally un- or poorly treated industrial and sewage effluents mean that to a large degree the present problems relate as much to water quality as they do to quantity. Demand and quality management are therefore regarded as equally important.

Summary of external Environment:

- Israeli water policy towards Palestinians
- Lack of resources
- Poor socioeconomic condition
- Institutional fragmentation
- Social Powers influence
- Lack of law Enforcement
- Competition among sectors
- Different Donors Agendas and lack of coordination
- Corruption at Different levels
- Lack of Governments Vision

7. HOW STAKEHOLDER DIALOGUE CAN BE FACILITATED?

Facilitating “vertical” and “horizontal” communication, coordination between all relevant actors (including local communities), recognizing different perceptions, negotiating conflicting interests, finding win-win solutions, in a specific domain, requires interactive dialogue among all actors. Such dialogue often requires also the skills of a professional mediator with no direct stakes in the issue to be discussed. A person or organization who is able to help others to listen to each other, to help actors to find common ground where possible; and if not, to come to reasonable compromise. Experience in many situations has shown that good process facilitation can contribute to better planning and implementation of viable and sustainable activities, facilitation here refers to mediating and guiding “brokering” processes in planning and decision making between government agencies (officials), local communities (end-users) and other relevant parties. (PHG Empower Project Document)

Process facilitators play a key role in stakeholder dialogue processes that aim to reach common goals and shared action plans. In such processes the different interests, perceptions, preoccupations, assumptions, and judgments among the actors involved, are made explicit. Opportunities are identified to improve the exchange of information, social organization, and decision-making between stakeholders in order to create the proper conditions for innovation and better (water) management. At the same time this may contribute to create awareness with respect to constraints and opportunities that affect the performance of relevant actors. A stakeholder dialogue will identify potential actors who do or could act effectively together to remove constraints and make use of new opportunities. Such a process often enhances institutional and technological innovation through active networking, involving all relevant actors including community members, governments, NGOs, academic institutions, and the private sector.

Moreover, facilitation should pay special attention to the less advantaged and poorer segments of the communities and actors. Every community has its own socio-economic configuration determined by culture, wealth, gender, land tenure, access to resources, etc. In most if not all communities there will be groups that are more vulnerable, have less resources and therefore, less influence in decision-making.

To ensure a balanced and interactive stakeholder dialogue, mediator or facilitator needs to have special skills and capacities. He/she need to make sure that the stakeholder dialogue process is well run and that all parties feel actively involved, without him/herself pursuing own agendas.

Step Two: Facilitation And Mediation

Ensure that every one is in the process
Facilitator has no own agenda
Marginalized groups are part of Process

EMPOWERS is a four year regional program for local water management in Egypt, Jordan and Palestine, funded by the EC MEDA Water Program and CARE International. It allies a Regional Partnership of fourteen organizations to empower local people and to improve long-term access to water by local communities. The main long-term goal of the project is to improve development and management of water resources at the intermediate and local level by promoting increased participation and representation of stakeholders in planning and decision-making processes. This approach will influence the bottom up planning and will lead to improved local water governance. In the mean time EMPOWERS operates as a Regional Information Program to disseminate information related to local water governance.

EMPOWERS strongly advocates and implements an approach of Stakeholder Dialogue and Concerted Action (SDCA), wih the assumption that stakeholder involvement leads to improved use and management of water resources, hence to a better local water governance. To this end, the project is developing a participatory planning cycle for Integrated Water Resource Management (IWRM). This cycle builds on the identification of water-related problems and the development of area specific long-term visions and strategies. This strategizing process is supported by the collection and analysis of relevant information on water resources, infrastructure, demand and access (RIDA) and the validation of this information in semi-quantitative Bayesian Networks (computer software). The aim of this planning cycle is to support stakeholders at local and intermediate levels in making the technical and political decisions to develop and manage their water resources within a commonly agreed future vision.

The EMPOWERS approaches are tested and developed since 2003 in the Governorates of Beni Suef (Egypt), Balqa (Jordan) and Jenin (Palestine) as well as in three selected communities in each Governorate. The EMPOWERS planning cycle for IWRM follows a Project Management Cycle approach, The activities for this planning cycle are firmly embedded in the above mentioned approach of SDCA that makes extensive use of tools for stakeholder analysis such as those developed in the RAAKS Guidelines.

Step Three: Create a common future vision

Use RAAKS guidelines and tools to create a greed common future vision

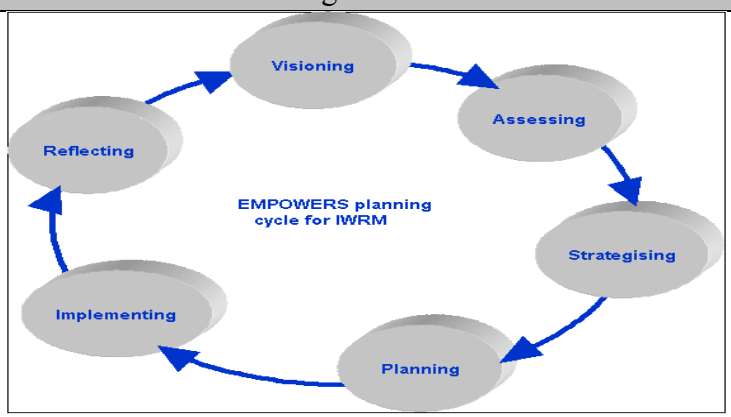


Figure 1. EMPOWERS planning cycle for IWRM (Source: Laban, 2006, and empower files).

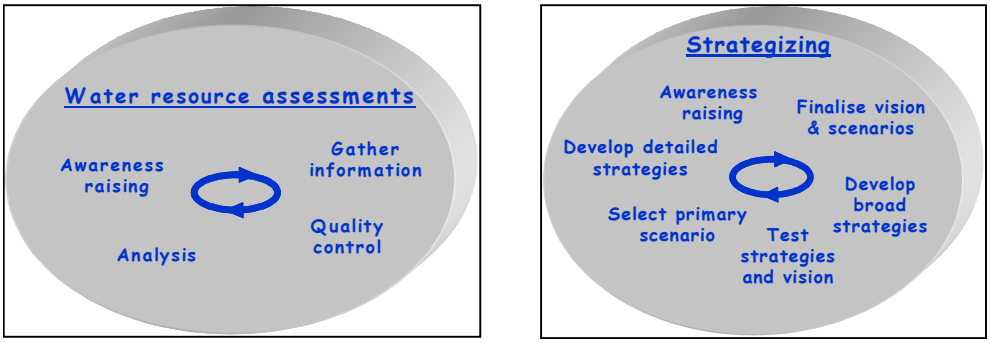


Figure 2. EMPOWERS water resource assessments and strategizing (Source: Laban, 2006, and empower files).

Step Four: Integrate Log Frame with Planning

Table 1. SDCA in the EMPOWERS Planning Cycle

Step	Objectives	SDCA for planning in IWRM	Outputs
Visioning	▪ Stakeholders involved and interested in work ▪ Broad scope of work identified and agreed	▪ Stakeholder identification and analysis ▪ Problem analysis ▪ Initial visioning and scenario building ▪ Identifying priority communities for action	▪ Stakeholder platforms ▪ Problem trees ▪ Initial visions at district or governorate level ▪ Initial scenarios
Assessing	▪ Main causes of water problems identified ▪ Agreed and shared information-base developed	▪ Stakeholders involved in: o Information collection and analysis o Quality control and cross-checking	▪ RIDA Analysis ▪ Belief Networks ▪ Provisional data base ▪
Strategizing	▪ Previous steps integrated to create shared basis for vertically and horizontally integrated action planning	▪ Update visions and scenarios ▪ Develop broad strategies ▪ Assess & validate scenario/strategy combinations using Bayesian Networks ▪ Select key scenario and related strategies ▪ Prioritize activities ▪ Define decision modalities	▪ WRA reports ▪ Community and District Water Fact Sheets ▪ "final" visions, scenarios and strategies for IWRM
Planning	▪ Detailed plan(s) for concerted action developed, budgeted and agreed	▪ Plan community and governorate level activities ▪ Identify tasks and responsibilities ▪ Define information flows ▪ Prepare project proposals ▪ Define M&E plans ▪ (acquire funding)	▪ Log frames for project proposals ▪ Funded IWRM Project Proposals for community, district and governorates
Implementing	▪ Activities implemented according to plans within a transparent and high quality approach and in a concerted way	▪ Implement activities ▪ Awareness raising ▪ Tendering (transparent) ▪ Capacity building ▪ Information sharing ▪ Quality control	▪ Achieved results ▪ Capacities build ▪ Information basis improved
Reflecting	▪ Implementation process documented ▪ Achievements monitored ▪ Lessons drawn out of preceding planning cycle	▪ Documenting processes (+ video) ▪ M&E ▪ Learning and reflecting	▪ Process reports & videos ▪ Evaluation reports ▪ Conclusions drawn as input for next planning cycle

Step Five: Institutionalize Stakeholder Dialogue

To what extent needs process facilitation be institutionalized and if so how? Facilitation of complex processes, as often the case in the water sector, needs to be done by preference by people that have no direct stake themselves in the process, i.e. who have not to gain directly or indirectly from the outcome of such a process. A facilitator needs to be independent from the "institutions" that are involved in the planning, decision and implementation process. With 'institutions", reference is made to all those bodies that have something to do with water, and that are relevant for specific issues under discussion at a certain time and place. Institutions may well comprise government agencies, NGOs and private sector at different levels (from national to municipality) as well as different community based organizations. Specific institutions have their own mandate and professional interest, which are often translated in sub-sector agendas that are more narrow than the water sector at large (mandates of drinking water authorities, agricultural departments, or the priorities of specific community groups). External facilitation of "horizontal" and "vertical" communication and coordination among these players may well become then a must, if overall planning and implementation of IWRM is done in an integrated and holistic way. In particular, attention is required to ensure that the often narrow sub-sector agendas of key stakeholders do not come to dominate. Breaking down barriers to horizontal and vertical communication is a key part of a good facilitation process, as is now experienced in the SDCA and Planning Cycle approach developed and tested by the EMPOWERS programmed.

In the water sector planning and decision-making are not one-off events. They occur in an iterative way before, during and after implementation of specific activities (see above planning cycle). Facilitation of such an iterative process cannot be restricted to a number of planning workshops at the outset of a programmed. The implementation of a planning phase and the resultant activities itself will be confronted with new and often unforeseen constraints, conflicts and opportunities that need to be dealt with and that often need the skills of an outsider to come to the necessary compromise. Matching and "translating" government concerns and policies to the realities on the ground and the priorities of different community groups is not an easy job and go often beyond bureaucratic solutions.

Criteria for identification of facilitation host institutes

Institutional position

- Relatively neutral, no specific sector agendas and independent from government
- An existing and well-known non-profit organization
- Indigenous and well-rooted in the country's civil society
- Unconventional and non-bureaucratic
- Wide geographic presence through field offices as well as development program in at least 30 -50 rural communities (country-wide in Jordan and Palestine; in at least two governorates in Egypt)
- Accepted by most if not all sectors in civil society and government

Capacitates

- Capacity in facilitating interaction and decreasing gaps between local communities and government agencies
- Experience in working with local communities (community development, capacity building,)
- Capable and experience in communicating with government agencies
- Interdisciplinary and divers staff capacities
- General (but not necessarily very specific) knowledge about the water sector
- Familiar with EMPOWERS approaches (SDCA, RAAKS, PTD, PRA, Participatory Planning framework, RBA, ……..)

9. LESSON LEARNED AND EVALUATION

Several lessons can be obtained from the EMPOWER project

- Stakeholders dialogue is a dynamic process and it should be integrated with all steps of project cycle
- All stakeholders' participation will lead to successful story
- Over expectation might be cause a serious damage and interruption during the implementation
- Institutionalization the stake holder dialogue will be the part of guarantees for future sustainability of the project
- Marginalized and poor group can use the dialogue as a platform to guarantee their rights
- Stakeholder dialogue is not demand driven it is a participatory approach

10. CONCLUSION

This paper has discussed the need for process facilitation in situations of increasing water scarcities, such as those in most of the countries of the Middle East. Also based on the Author and Palestinian NGOs experiences the stakeholders dialogue and consultation should take into consideration the local conditions, the different kinds of externalities, and the socioeconomic and political un-certainties. Facilitation is even more necessary in order to pro-actively involve marginalized and voiceless groups women and the poorest layers of society in rural communities. It gives an example of how a stakeholder approach for changing values from charity based to right based, institutionalizing the stake holders involvement will help for optimal management through creation community ownership of the project.

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TOWARDS IWRM IMPLEMENTATION – SCENARIOS DEFINITION FOR KALIYA REGION

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Abstract: The concept of IWRM is to apply a holistic approach to the water resources management problem (from both quantity and quality aspects) and its interaction with social implications, in a watershed or a group of watersheds, possibly across national borders. The implementation of IWRM is demonstrated for the Kaliya region, which is one of the pilot study areas in the SAMRT project. The Kaliya region is located at north-west of the Dead Sea, which is an arid zone with average precipitation of less than 100mm per year. The area is isolated from the main Israeli national water systems and the water supply depends solely on local sources. The water is used for domestic purposes and for agriculture (which is the main economic activity and source of income). Any increase in water demand has to be drawn from the local water sources, thus imposing substantial stresses. A Multi Criteria Decision Making (MCDM) process was applied for qualifying and ranking various alternatives of water utilization, according to a number of criteria of different and often competing nature. MCDM aims at highlighting these conflicts and deriving ways to reach a compromise in a process that is rather transparent. The goals of a certain project or action have to be defined by decision variables and need to be maximized subject to constraints. Multiple conflicting criteria can be formally incorporated into the water management planning process by using the Analytic Hierarchy Process (AHP) method which is based on comparative judgment of alternatives and synthesis of priorities. The AHP method is designated to assist the decision maker to rank predefined alternatives according to a set of predefined criteria, following a pair-wise comparison process. The water policies for this region are: converting water for agricultural uses; increasing domestic water consumption as a result of urban/rural development; and prioritizing water supply for the tourism (as this sector has the highest potential of income/per allocated cubic meter). The future development depends on reliability of the water supply (quantities and qualities). The availability of the fresh water is the limiting factor for any future development, especially during the peak demand seasons. The constraints are bound with availability of the different water resources (fresh water, brackish water from springs and sewage). The water clients were classified into 4 categories (domestic use, agriculture, tourism and nature) and their water demand was recognized. Numerous policy complex problems for this region were characterized as goals (multiple criteria), after defining four main future management scenarios for each client. The criteria of which the policy makers needed to analyze and facilitate their decision making were: Protecting the fresh water body from deterioration, minimizing the gap between demand and supply, especially on peak days, and maximizing the client's economic profit. The next step was to analyze efficiently the multi criteria problems and determine which objective out-weight another, by recognizing the necessity for trade-offs between the alternatives to best serve the common interest of the study area.

Key words: IWRM, Socio-Economic, SMART, Groundwater, Arid Zone, Water Supply, Dead Sea, Israel

1. INTRODUCTION

Integrated Water Resources Management (IWRM) expresses the concept that water resources should be managed in an holistic way, coordinating and integrating all aspects and functions related to water abstraction, quality control and water-related service upply so as to bring sustainable and equitable benefit to those who are dependent on the resource (http://iwrm-smart.org), also taking into consideration the effects of each use on the others in the context of social, economic and environmental objectives (http://www.archive.cap-net.org/iwrm_tutorial/mainmenu.htm; Bouwer 2000).

Understanding the interactions between water, environment and society is the foundation of the IWRM for a specific area.

This paper presents the implementation of one element of a decision support system (DSS) for regional IWRM, applied to the Kaliya region in the lower Jordan Rift Valley.

Water management decisions are typically involves a relatively large number of alternatives and multiple, conflicting and often incommensurate evaluation criteria. To this end, a modeled-based methodology for Multi Criteria Decision Making (MCDM) has been implemented for this study. MCDM is a discipline aimed at supporting decision makers who face the need to make numerous and conflicting evaluations. MCDM aims at highlighting these conflicts and deriving a way to come to a compromise in a transparent process. The goals of a certain project or action have to be defined by decision variables and need to be maximized subject to constraints. MCDM problems involve a number of criteria that may not be of equal importance to the decision maker. Therefore, each criterion is assigned a weight that represents the preference of the decision maker with regard to the objectives. This represents the main core of the Analytical Hierarchy Process (AHP), which was developed by Thomas L. Saaty in the 1970's and has been extensively studied and refined since then. The AHP is a mathematical theory of value, reason, and judgment, based on ratio scales for the analysis of multiple-criteria decision-making problems (Saaty, 2008). The AHP method is designated to assist the decision maker to rank predefined alternatives according to a set of predefined criteria, following a pair-wise comparison process (Saaty 2001; Saaty 2005; Saaty 2006; Saaty and Hu 1998; Barzilai and Golany 1994). It is important to note that some combinations of alternatives are contrary to the others and these tools are aiming in defining and selecting the proper alternative (combination).

2. RESEARCH METHOD

2.1. The study area

The implementation of IWRM should link between land and water uses in a basin scale, which is the smallest hydrological unit of analysis and management (Jønch-Clausen and Fugl 2001). The Kaliya region was selected as a pilot study area for demonstrating the IWRM concept. The Kaliya region is located at the north-west coast of the Dead Sea and in the eastern edge of the Judea Desert (figure 1). The main aquifer is the dolomite limestone layers of the Judea Group aquifer (Cenomanian age). This is an arid zone with an average precipitation of less than 100mm per year. The recharge is in the high mountainous area about 25 km westward to the Kaliya region. The aquifer system outlet is through two groups of springs, En Feshcha in the northern part and Kane and Samar in the southern part (Figure 1). The water in the springs is brackish to saline (600-6000 mg/lit Cl-). The total discharge of the springs is approximately 90-95 MCM/year. This water flows to the Dead Sea and is lost there (Hötzl et al., 2008). The water supply depends solely on local sources and it is isolated from the main Israeli national water systems without any backup.

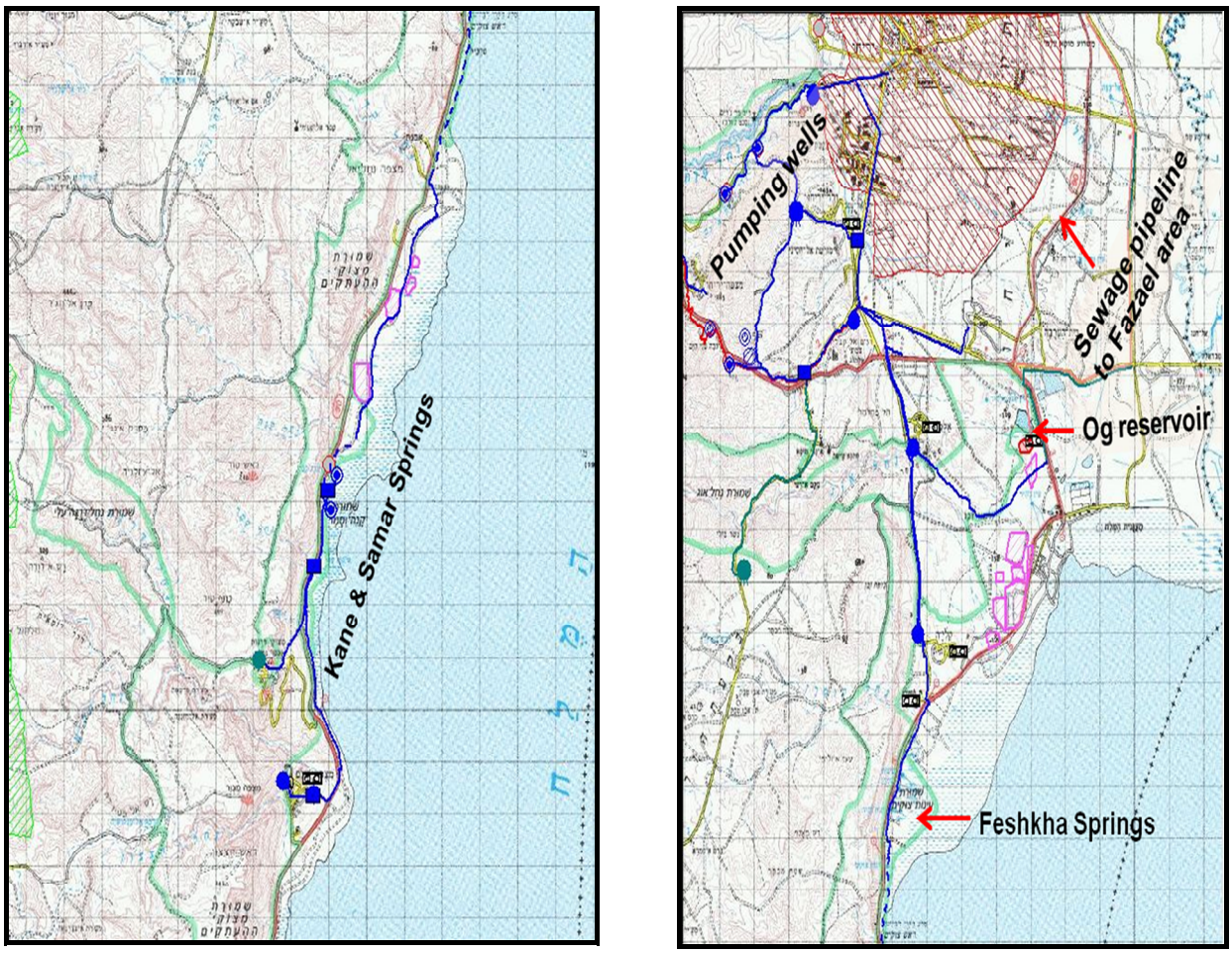


Figure 1: Location map of the study area and the water supply network (left side-Kaliya area, right side-Mizpe Shalem area)

2.2. Methodology

In order to make decisions in the complex environment of multiple criteria, we need to understand **the problem**, the need and purpose of the decision (**the goal**), **the criteria** with which we screen the decisions, to define a set of sub-criteria, consult the stakeholders and other affected groups and finally to define a set of **alternatives or scenarios** that can meet the stated goal. We then try to determine the best alternative, or in the case of resource allocation, we need priorities for the alternatives to allocate their appropriate share of the resources. The decisions involve many intangibles that need to be traded off. Through tradeoffs it clarifies the advantages and disadvantages of policy options under circumstances of risks and uncertainty (Saaty, 2008).

The methodology chosen for breaking down the multiple problems of the Kaliya region and aggregating the solutions into a conclusion is the Analytic Hierarchy Process (AHP). According to this method, prioritization process is held through comparison of alternatives and is measured by rating. The AHP is a systematic procedure to construct and represent the elements of a problem in a hierarchy formant. The basic rational of AHP is organized by the breakdown of the problem into smaller constituent parts at different levels. An AHP model typically consists of an overall goal, a set of criteria to specify the overall goal decomposed to sub-criteria, and finally, at the lowest level of the hierarchy, the decision alternatives to be evaluated. In AHP, each element in the hierarchy is considered to be independent of all the others. The hierarchy can be visualized as a diagram like the one below (Figure 2).

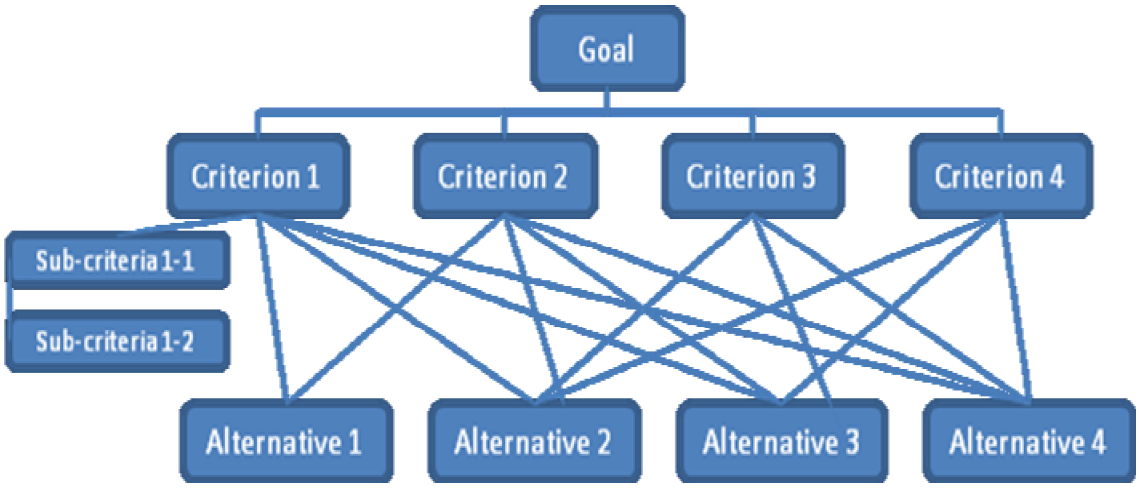


Figure 2: Illustration of the Analytic Hierarchy process
Based on: Saaty, 87, 2008

Beyond the decomposition principle, the AHP is based on pair-wise comparisons of elements in a decision hierarchy with respect to the parent element at the next higher hierarchical level (i.e., among criteria and lower level elements). The pair-wise comparison is aimed in providing a cardinal scale to evaluate objects according to some subjective preference criteria. Pair-wise comparisons are made on a scale of relative importance (Table 1) where the decision maker has the option to express the preferences between two elements on a ratio scale from equally important (i.e., equivalent to a numeric value of one) to absolute preference (i.e., equivalent to a numeric value of nine) of one element over another. Ratings of decision makers are arranged as numerical numbers in a comparison matrix. Based on this, relative weights for all elements of the hierarchy are calculated with the Eigen-value method, indicating the priority level for each element in the hierarchy (Saaty, 2001). The alternatives performances are weighted with the relative weights of criteria and sub-criteria, and added to an overall priority for each alternative (i.e. how they contribute to the goal), which allows a cardinal ranking of the alternatives (Saaty, 2006). An alternative calculation of the weights, called the logarithm least squares method (LLSM) has been suggested and it can be applied to rank criteria too. There is substantial debate regarding the most suitable way to calculate the weights (the Eigenvector method (EVM) or the LLSM method). Many papers have been indicating their advantages and shortcomings, but so far there no clear recommendation regarding the best method. This technique was applied to the Kaliya region and the results of the evaluation will be presented as fallows.

Intensity of Importance	Definition	Explanation
1	Equal Important	Two elements contribute equally to the objective.
3	Weak Importance	Experience and judgment slightly favor one element over another.
5	Strong Importance	Experience and judgment strongly favor one element over another.
7	Very Strong Importance	One element is favored over another; its dominance is demonstrated in practice.
9	Absolute Importance	The evidence favoring one element over another is of the highest possible order of affirmation
Intermediate values used to interpolate between adjacent scale values		

Table 1: A nine point scale for pair-wise comparison (Saaty, 7, 2005)

3. RESULTS AND DISCUSSION

3.1. Defining the problems and the goals for the Kaliya Region

First, the constraints of the Kaliya region were defined. The constraints bound with the different water resources (fresh water, brackish water, springs and sewage) that are exploitable in this region. Fresh water is supplied from Mizpeh Jericho well field, which is located few kilometers westward to the region. This well field is the eastern point where the groundwater is fresh prior to it mixing with the Dead Sea brines. The water in the springs is a mixture of the fresh water with the Dead Sea brines. The salinity varies between 600-6000mg/l Cl (Hötzl et al., 2008). The water up to salinity of 1500 mg/l Cl^- is currently used for local cultivations. The brackish water from the Feshkha springs is supplied to the Kaliya area, and the brackish water from the Kane springs is supplied to the Mizpe Shalem area. Although the total discharge of the springs is approximately 90-95 MCM/year, only small portion (approximately 1 MCM/year) of brackish water is being used today for the local agriculture. Most of the spring's water flows to the Dead Sea without utilization.

The sewage generated in the eastern neighborhoods of Jerusalem and the Ma'ale Adumim locality is stored in the Og reservoir (Volume of 1.3 MCM). The secondary treated wastewater is utilized for the local agriculture (only palm trees are irrigated). Until 2008 some amount was transferred northward to Fazael area. Today the sewage from the Og reservoir provide effluent for irrigation mainly to the Kaliya area.

The Israeli water policy (the goals) for this area is: to convert water for agricultural uses; to increase domestic water consumption as a result of urban/rural development; and to develop the water supply for the tourism sector due to its high income.

The future development depends on reliability in water supply (quantities and qualities) along the year. Thus, the current fresh water supply availability is the limiting factor for future development (agriculture, domestic use, tourism), especially during the high demand seasons (see table 2).

	Cubic meters per year	Peak day
Existing water production	5,800,000	18,040
Demand	5,491,000	24,242
Total balance	309,000	-6,202

Table 2: Fresh Water Production & Demand - Mizpe Jericho and Jericho2 wells (2004)

We then established an inventory of all clients in the study area and their water needs/demands. These include:

1. The **domestic sector**, with a total population of 1,150 residents live in 6 localities, with yearly demand of approximately 230,000 cum/year. The population growth is extremely high 7.9% (data valid for 2006) and therefore water demand is expected to increase.
2. Six types of **agricultural cultivations** exist in this study area: 2,000 Dunam (1 Dunam = 1,000m^2) of palm trees; 1,000 Dunam of cherry tomato; 500 Dunam of pepper; 300 Dunam of grapes; 100 Dunam of basil; and 30 Dunam of asphodel - with total demand of 5,850,000 cum/year (data valid for 2008).
3. Four areas of **tourist** accommodations include 167 guest rooms which consume 81,600 cum/year. During 2007, total of 360,000 visitors were numerated visiting Feshcha spring and Kumeran natural parks and consumed approximately few thousands cum/year (there isn't a separated measurement of the water that is supplied to these parks). It is impossible to evaluate the visitors attending the other 7 Natural Reserves in the region.
4. The **natural environment** (the spring's natural reserves): To maintain the live of the flora and the animals in these natural reserves a minimum amount of water should continue to flow from these springs to the Dead Sea. The actual number is unknown. In

some policy-meetings the numbers of approximately 10 MCM/year per spring area was suggested (personal comments: Guttman, 2008). The safe yield that can be drawn in Mizpe Jericho well field is between 8-10 MCM/year (Guttman et al. 2004).

The findings show that most of the water demand is for tourism and agricultural uses.

3.2. Constructing the multi criteria problem for the Kaliya Region

The third step was to analyze efficiently the multi criteria problems and determine which objective out-weight another, by recognizing the necessity for trade-offs between the alternatives to best serve the common interest of the study area.

In this process, numerous policy complex problems were identified and were characterized by means of four criteria. In this current research, the criteria of which the policy makers will need to analyze and facilitate their decision making are: Protecting the aquifer from deterioration according to the hydrological recommendations (C1), minimizing the gap between water demand and supply (C2), especially on peak days (C4), and maximizing the client's economic profit (C3). In the prioritization process the policy makers needed to pair-wise compare between the criterions and give relative weights for all elements of the criteria hierarchy. According to the criteria rating results, both of the LLSM and the EVM show that the criteria hierarchy is: C4>C2>C3>C1. These results indicate that the policy makers prioritize to minimize the gap between water demand and supply on peak days more than the other 3 criteria.

Then, the policy makers needed to analyze the uses of different water resources that are accessible in this region. The alternatives for each criterion bound with the water resources, and they are: fresh water (A1), brackish water (A2), effluent (A3), dilution of fresh and brackish water (A4), and dilution of brackish water and effluent (A5). The policy makers needed to pair-wise compare between all five alternatives of water resources and give overall priority for each alternative. This process was done for each goal (i.e. analyzing all five alternatives for four criteria). The outcome of the calculation process is 4 comparison matrixes, which allow a cardinal ranking of the alternatives for all elements of the criteria. According to the alternative rating calculation, both of the LLSM and the EVM show similar alternative hierarchy. The results according to the EVM calculation are summarized in table 3, and they show the overall alternative hierarchy: A5>A3>A4>A2=A1 (When alternative A is said to be better alternative B, we write A > B). These results indicate that the policy makers prioritize the dilution of brackish water and effluent usage, and the fresh water brackish water alternatives have equal importance.

	AHP Weighted relative values	A1	A2	A3	A4	A5
C1	0.078	0.003	0.01	0.032	0.006	0.027
C2	0.103	0.016	0.009	0.009	0.019	0.049
C3	0.154	0.007	0.016	0.047	0.014	0.071
C4	0.665	0.031	0.033	0.158	0.07	0.366
Results		0.068	0.068	0.246	0.116	0.513

Table 3: The outcome comparison of water resources alternatives for all policy criteria, according to the Eigenvector method

3.4. Defining the Scenarios for the Kaliya Region

The scenarios were defined, and they are summarized in table 4.

Increasing the fresh water demand	Increasing the agriculture sector	Domestic water use	Tourism
Drilling new wells.	Broadening existing plots.	Development and populating this area is first of all a political issue.	Develop new and attractive sites and beaches along the Dead Sea shore.
Importing water from existing sources.	Developing new areas for plantation.		Develop the potential climate-therapy, thermal and spa sites.
Desalination of brackish water.	Bring additional water and increasing the reuse of brackish water and sewage.		Build hotels and other tourism facilities.
Improving the water supply system (pipelines, reservoirs).			

Table 4: scenarios definition for the study area

4. SUMMARY AND CONCLUSIONS

A Multi Criteria Decision Making (MCDM) process was applied in the Kaliya region for qualifying and ranking various alternatives of water utilization, according to a number of criteria of different and often competing nature. The Kaliya region is located in the northern part of the Dead Sea. This is an arid zone with an average precipitation of less than 100mm per year. The area is isolated from the main Israeli national water systems and the water supply depends solely on local sources. The water resources in this area are fresh groundwater, brackish water from springs and effluent. The agriculture sector is the main water consumer and uses all types of water (directly or mixed). Tourism is the sector that has the maximum challenge to develop in the future.

Four criteria were defined: Protecting the aquifer from deterioration according to the hydrological recommendations, minimizing the gap between water demand and supply especially on peak days and maximizing the client's economic profit. The four criteria were compared between themselves and were given relative weights for all elements of the criteria hierarchy. The research results indicate that minimizing the gap between water demand and supply on peak days is the most important criteria. Afterwards, a set of alternatives for each criterion bound with the water resources were defined. The alternatives are the local water resources (fresh water, brackish water and effluent) and dilution between fresh and brackish water and between brackish water and effluent. Again, the alternatives were compared between themselves giving overall priority for each alternative. This process was done for each goal (five alternatives for four criteria). The final result indicates that dilution of brackish water and effluent usage is favorable, and fresh and brackish water alternatives have equal importance.

5. ACKNOWLEDGEMENTS

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SOCIO-ECONOMIC PROFILING AS PART OF IWRM FRAMEWORK: ISRAELI LOCALITIES WITHIN THE LJRV

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Abstract: The implementation of Integrated Water Resource Management (IWRM) requires multidisciplinary information. The leading assumption is that socio-economic characteristics rely on and affect the implementation of water technologies, utilization and management plans. Analyzing the socio-economic profile of a given community in a defined region that is a client of water, which is supplied by local water resources, will allow better definition of the future water exploitation, utilization and management scenarios. The objective of this study is to delineate a socio-economic profile of the Israeli localities within the hydrological basins of the Lower Jordan Rift Valley (LJRV), in the context of an IWRM framework. In addition, this profile also included information on water consumption and water that is supplied within the LJRV. The findings are based on literature review, interviews and statistical information data collected from the Central Bureau of Statistics in Israel (CBS) on population and economic census. The findings show that the socio-economic level of the population living in LJRV is ranked as medium to low. The population growth rate is much higher than the national average, and the age structure of the population pyramid shows slight differences from the national one. The reason for this is the demographic and socio-economic characteristics of the local population living in these frontier localities. Within the study area there are 33 localities with a population of 12,292 Jewish residents. The LJRV localities are mainly classified as agricultural cooperated rural localities, while industrial branches support the cultivation activities and market the productions for export and for the local markets. The findings show that agriculture was and still is the main water consumer. The area is intensively used in irrigated agriculture. Water is drawn from the Jordan River, numerous springs and wells penetrating the water bearing layers of the Mountain aquifer. The Domestic effluents which are generated by the small localities are treated in local wastewater plants. Treated waste water is supplied to the agricultural consumers, and it is used mainly for irrigating palm trees. Irrigation of palm trees with this type of water enables allocation of fresh water to other crops, or for domestic uses. However, water scarcity due to semi-arid and arid climate, the growing consumption, and the water quality are the bottle-neck for the development of growing population. Furthermore, this fringe region has no upcoming future water demand and supply plan. Due to the anticipated increase in tourism, agriculture and domestic water use in the near future, economic growth and viability are linked to the preservation and the increase in sustainable water sources.

Key words: IWRM- Integrated Water Resource Management, Socio-Economic and Demographic Profile, SMART- Sustainable Management of Available Resources with Innovative Technologies, Water Consumption and Supply, Water Shortage.

1. INTRODUCTION

Freshwater is a finite and vulnerable resource, and it is essential to life, economic development and the environment. Conversely, water demands and pollution increases as a result of continued growth in population and economic activity, and thus competition and conflict over limited water resources increases (Bouwer 2000; Jønch-Clausen and Fugl 2001; Keskinen 2006; Jorgensen and Rast 2007). Studies show that the major dynamic factor that causes stress on the available water supplies is the growth of population, which has unprecedented impact on socio-economic development, increasing costs of water development and limitations on development. Stresses include extraordinary demands for agriculture (particularly irrigation and drainage), the provision of domestic water supply and sanitation, industrialization, energy production, environment, tourism, rural and urban shifts, and migration (Al Radif 1999).

Integrated Water Resources Management (IWRM) framework overcomes the complex water problems, which traditional approaches have not been able to address this complexity and interdependency in water resources management (Jønch-Clausen & Fugl 2001; Jewitt 2002; Biswas 2004; Ballweber 2006; Jorgensen and Rast 2007). In spite of the fact that there is no agreement on IWRM definition, most international institutions have endorsed this concept. IWRM expresses the idea that water resources should be managed in an holistic way, coordinating and integrating all aspects and functions of water extractions, water control and water-related service delivery so as to bring sustainable and equitable benefit to those entire dependent on the resource (http://iwrm-smart.org), and considering the effects of each use on the others in the context of social, economic and environmental objectives (http://www.archive.cap-net.org/iwrm_tutorial/mainmenu.htm; Bouwer 2000).

The implementation of IWRM framework is viewed in a country or a specific region, because water issues vary from one area to another, and often within a single country (Al Radif 1999; Jønch-Clausen and Fugl 2001; Bowen and Riley 2003; Biswas 2004; Letcher and Giupponi 2005; Ballweber 2006). In order to integrate all aspects and functions of water resources in a specific region, this study will present the social and economic dimensions, which are often mistreated. The aim of this paper is to define the socio-economic profile of community in a fringe region, in the context of an IWRM framework. A secondary aim derivative from this research is to promote broad perspectives and awareness among decision makers about socio-economic issues which are highly related to water resources management and considered into any measure of sustainability.

2. RESEARCH METHOD

2.1. The study area

IWRM must link land and water uses across the whole of a catchment, which is the smallest complete hydrological unit of analysis and management (Jønch-Clausen and Fugl 2001). This basic unit is defined by natural geographic and hydrologic boundaries. In this current research, the hydrological basins are the reference borders of the study area which is the Lower Jordan Rift Valley (LJRV) located in the eastern Mediterranean basin. The LJRV, an extensive geotectonic fault system, is shared by three nations: Israel, Kingdom of Jordan and Palestine authority – the west bank. Since the origin of the water on which these countries depend is not entirely within their national territory, this issue created and still creates substantial political tensions and conflicts. Each nation has its priority developmental and economic goals set according to environmental, social and political realities. The area is intensively used by irrigated agriculture, because water is drawn from the Jordan River, numerous springs located on the valley's flanks, and wells penetrating the water bearing layers of

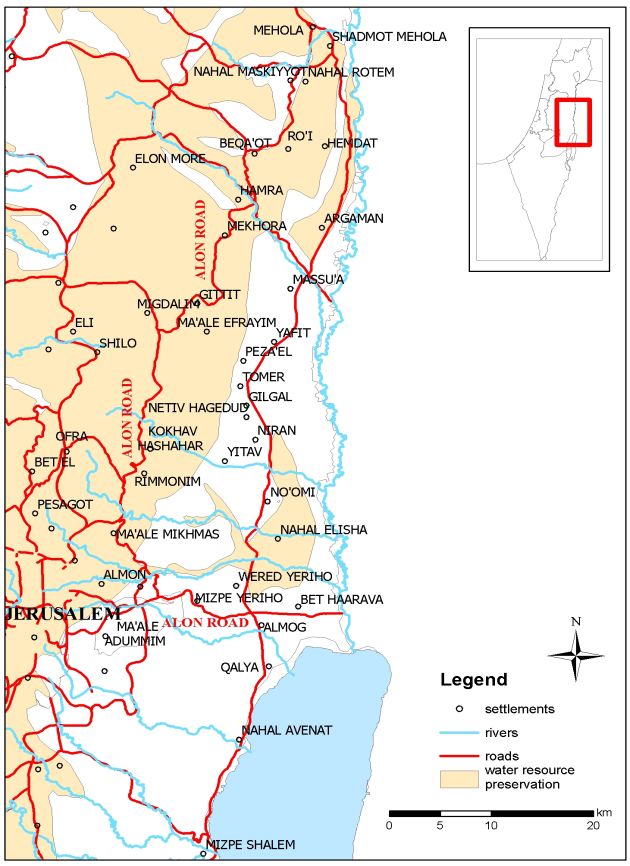


Figure 1: The LJRV Study Area

Data Source: computerized data layers of localities, rivers, roads and water resources preservation areas – provided from the Ministry of Interior.

the Mountain aquifer. However the scarcity of water due to semi-arid and arid climate (annual precipitation of 100-300 mm) requires irrigation water for agricultural development, and it is the bottle-neck for the development of growing population.

IWRM must be seen in the specific context and stage of economic and social development because these factors differ greatly between the three nations. Thus, this study deals only with the socio-economic profile of the Israeli local population living in LJRV localities. The Israeli part of the LJRV is an elongated valley and is located in the extreme east of the country, as seen in figure 1. The eastern strip includes the Jordan Valley and the northern shores of the Dead Sea. The Eastern strip lays between two longitudinal axes, one along the west side of the Jordan River and the Dead Sea and the other, along the eastern slopes of the Samarian Hills, effectively the east side of the Alon Road (road no. 458). This section also marks most of the border between Israel, the west bank and Kingdom of Jordan.

2.2. Methodology

The leading assumption is that socio-economic characteristics rely on and affect the implementation of water technologies, utilization and management plans. The objective of this study is to delineate the socio-economic characteristics of the Israeli population living within the hydrological basins of the LJRV, in the context of an IWRM framework. In addition, the socio-economic analysis also included information on water consumption and water that is supplied from the local resources within the LJRV. The following components are:

1. Characterization of the Israeli localities in the LJRV area.
2. Economic activities of the local population (employment recourses such as agriculture, industry and manufacturing activities, tourism etc.).
3. Demographic data on the current population and expected growth trends.
4. Water consumption and water supply.

The findings of all these components are based on literature review and interviews, when combination of knowledge usually exceeds the limit of either source of knowledge alone:

1. Literature review: studying and screening existing data on the prominent characteristics of the localities and population within the LJRV.
2. Census: statistical information data collected from the Central Bureau of Statistics in Israel (CBS) on population and economic census. The information includes physical data (population, education, welfare, etc.) on population living in Israel.
3. Interviews: The results of the literature review were complemented by interviews with some of the key persons in the local authorities and from different sectors.

3. RESULTS AND DISCUSSION

3.1. Characterization of the Israeli Localities in the LJRV region

The Israeli localities in the study area were established after the 1967 war, in order to create a string of settlements in fringe areas (Tenenbaum and Eiran 2005). This idea was the main core of what is known as the “Alon Plan”. The Jordan Rift Valley was estimated to comprise 66,000 Dunam (1 Dunam = 1,000m^2) of cultivable land, unevenly distributed along the Valley. The north is more suitable for cultivation than the southern part. Therefore, more localities were established there. The potential amount of land and water available determined the number of agricultural units and settlements. Agriculture provided most of the employment in the study area. Industries and craft were established in order to supplement income and to provide alternative employment during slack

seasons. Owing to the paucity of agricultural resources in the southern part of the Valley, the localities were based mainly on tourism. The settlements pattern in the Jordan Valley was more or less similar to that found in other parts of the country, a combination of farming and industry wherever this has been found to be necessary. The idea was that with the aid of large scale capital investment in the development of water resources and the use of agro-technical knowledge, it would be possible to raise large crops of winter vegetables and fruits, which can be sold at high prices in the European markets. Since the 1980's many of these localities have branched out into industry and tourism, while some of their members have begun to work as salaried employees in the adjacent urban centers (Efrat 2006).

The study area generally follows a rural settlement pattern, and the localities are classified as agricultural rural ones. These localities are unionized into 3 Regional Councils (RC - refers to a number of rural localities, which provides various municipal services for the villages within its jurisdiction boundaries). Only one urban Local Council (LC) was found in the study area. Rural localities (include less than 2,000 inhabitants) are sub divided in the country to the following classic models: Kibbtz, Moshav, collective Moshav, Institutional locality, Communal locality and other rural locality. The variation between them derives from the equality and the level of cooperation in ownership of property and of means of production (CBS 2006b). The statuses of the Israeli rural localities in the study area are presented in table 1. The locality type and the level of cooperation define the water consumption. The agricultural sector is the main water consumer, consuming more than 95% of the freshwater supplied to the LJRV region. Jordan Valley RC consumes approximitlly 83% of the freshwater supplied to the LJRV region because cultivable land is distributed at its jurisdiction.

Table 2: Socio-Economic Data of the LJRV localities

Data Source: http://www.cbs.gov.il/www/publications/local_authorities2006/excel/p_excel.xls

	Jurisdiction area	**Association to a Council**	**Total Localities in the study area**	**Locality Type**	**Type of residents living in the localities**	**Occupation**	**Socio-Economic Rank**
Jordan Valley	Northern Regional Council: along the length of LJRV	Regional Councils	21	2 Kibuzim 15 Moshavim 4 communal localities	13 secular 6 religious 2 mixed	28% tourism 28% light industry 100% agriculture	6
Megilot-Dead Sea	Southern Regional Council: along the northern - western part of the Dead Sea	Regional Councils	6	4 Kibuzim 1 Moshavim 1 communal locality	5 secular 1 religious	67% tourism 50% light industry 83% agriculture	5
Mate-Biniamin	Eastern Council, partly included	Regional Councils	5	5 communal localities	1 secular 3 religious 1 mixed	20% tourism 100% light industry 40% agriculture	4
Ma'ale Efraim	western side of the Jordan valley eastern strip	Local Council	1	Urban	mixed	light industry	5

3.2. Demographic and socio-economic data

Israel population is growing at an annual rate of 1.8 percent, while population growth in the study area is much higher, mostly due to fertility rates and slightly from migration (population growth: 3.3% at Jordan Valley, 7.9% at Megilot-Dead Sea, 7.4% at Mate-Biniamin, -2.7% at Ma'ale Efraim). Population growth is encouraged in correspondence to the predominant ideological and religious beliefs in the Jewish sector in Israel. This set of beliefs includes ideological-nationalistic values (Zionism), religious values (Judaism), traditional patriarchal family structure, military-security and economic considerations. This has directly and indirectly influenced a series of national policies that promote high fertility (Orenstein 2004). Within the study area there are 33 localities with a population of 12,292 Jewish residents. Israel population is composed of similar percentage of female and male. Life expectancy tendency is increasing both for male (78.3) and female (82.3). The age and sex structure of Israel's population is one of the clearest reflections of the demographic place of Israel among societies of the world. Population pyramids of Israel in the 1950's closely resemble those of third world countries. In the early 1990's the population pyramid was becoming more like the European-American structure (Goldscheider 1996). As it is detailed in figure 2, age structure shows slight differences of the population pyramid from the national one.

The findings indicate that population characteristics living in LJRV region is correspondence to developing countries. Population trends are a crucial point in projections of natural resources consumptions, water being one and the most vital of them all (Dungumaro and Madulu 2003). Population growth and higher standards of living will cause ever increasing demand for high quality drinking and industrial water, and at the same time, more and more irrigation water will be needed to meet increasing demands for food. Also, more water will be required for natural environmental concerns.

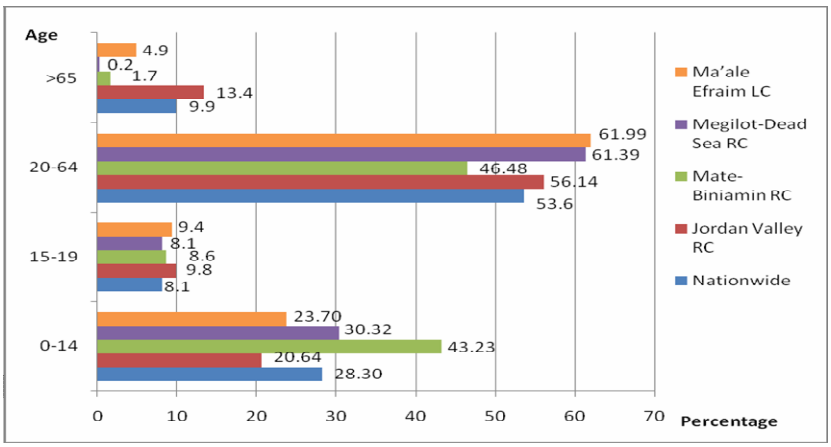


Figure 2: Population pyramid of the LJRV localities
Data Source: http://www.cbs.gov.il/www/publications/local_authorities2006/excel/p_excel.xls

The Israeli CBS classified all the localities by their socio-economic characterization of the population residing in the localities, according to different demographic and socio-economic variables (such as education, standard of living, labor force, health, etc.). The ranking is from 1 to 10, when 10 is the highest grade meaning high socio-economic level. The localities in the study area are socio-economic ranked according to the regional councils which they belong to. The socio-economic level of the population living in LJRV is medium to low (table 1).

3.3. LJRV Water Consumption and Supply

Israel's overall water inventory is limited, unevenly distributed, and subject to rather high, climatically determined fluctuations. The region suffers from scarcity of usable water, and water is considered as a national resource of utmost importance. Around the world, water is generally utilized under a system of user rights. In Israel a quota allocation system assigns the amount of

water to be used on an annual basis (or a seasonal basis, in the case of farmers). The Water Law (1959) established this annual allocation and licensing system in order to meet the future demand with adequate quantities of water and to maintain its quality. Irrigation water is subsidized, while water for the urban and industrial sectors is not (Arlosoroff 2001). In order to respond to increasing socio-economic pressures and to ensure more sustainable water resources management, the Water and Sewage Governmental Authority was established in January 2007 after unionizing all water and sewage laws, institutional authorities, and powers (http://www.water.gov.il). Three primary directives implement water planning in Israel: agriculture development, population dispersal, and development of arid regions. Water production and distribution is mainly carried out the by the Israel Water company "Mekorot", which supplies 90% of Israel's drinking water and 70% of all the water supply in the country (http://www.mekorot.co.il).

According to the Water and Sewage Governmental Authority and Mekorot, LJRV fringe region has no upcoming future water demand and supply plan, and the development in this arid area is first of all a political issue.

Mekorot is the main fresh water supplier for the consumers in the study area, for agricultural and domestic users. The water supply is from a local distribution network, which is separated from the Israeli national water systems, mainly drilling from the mountain aquifer. The drilling wells are connected to a reservoir (water plant) and are transferred from it to the consumer by pipe lines. Figure 3 summarizes the total water supply (fresh and saline water) provided by Mekorot to the study area, between the years 2000 to 2007. Somewhat fluctuation in water consumption exists from year to year due to changes in cultivation types and their irrigation requirements, changes in population growth and migration balance, and the use extent of treated waste water.

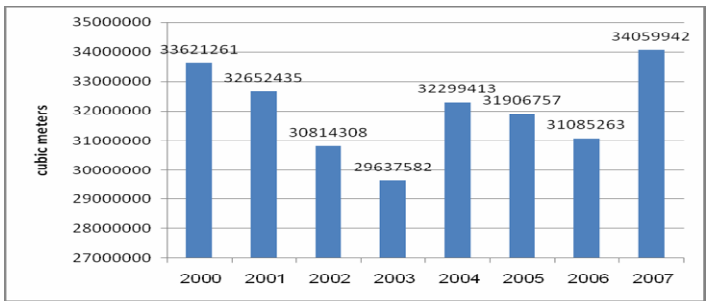


Figure 3: Water Supply Provided by Mekorot to the LJRV Study Area (cubic meters)
Data Source: provided by Mekorot (a governmental authority), 2008

Alternative water resource such as water reuse and desalination, increases the availability of potable water, and increases the management efficiency of conventional resources and schemes to optimize the water usage. These solutions are completely in line with sustainable and IWRM approaches (Thomas and Durham 2003). Two water associations exist in the study area, which supply water to agricultural use, and each operate a water reservoir, Tirza at the northern part of the LJRV and Og at the south. Tirza reservoir collects flood-water flowing downstream from Triza stream, and Og reservoir collects wastewater from eastern Jerusalem neighborhoods and from Ma'ale Adumim locality. Water supply is occasionally mixed by different types such as salt water, flood water and treated waste water. The domestic effluents generated by the small localities are treated in local wastewater plants. Waste water flow into artificial reservoirs where in this process contaminants (mainly organic compounds) are removed from wastewater throughout bio-oxygenation ponds. The Inhabitants in twenty one localities (64 percent from total of 33 localities) are connected to oxygenation ponds. Four localities discharge effluents to Septic tanks, which is a small scale sewage treatment system. This system was common in some towns and villages in Israel, but due to environmental problems it is no longer utilized. Therefore, Septic tanks are converted to oxygenation ponds or wastewater treatment plants. Only four domestic wastewater treatment plants collect and transport effluents from localities and discharge or reuse treated sewage back into the environment (for agricultural use or to the streams).

4. SUMMARY AND CONCLUSIONS

Practical implementation of IWRM must reflect variations in local conditions according to specific context and stage of economic and social development. IWRM must be extremely sensitive to national, political, cultural, and social conditions (Jønch-Clausen and Fugl 2001). Analyzing the socio-economic profile of a given community living in a defined region that is a client of the water supplied by the local water resources will allow better definition of the future water exploitation, utilization and management scenarios. To this end, this paper addresses the IWRM framework by presenting the socio-economic profile of a given community, and its water needs. The Israeli population living in the LJRV is presented by including characterization of the localities, demographic and socio-economic data on the local population, and information on water consumption and water supplied within the LJRV.

Demographic and socio-economic characteristics of fringe localities are expected to be different than the national ones. For example, the age structure of the study area shows a different population pyramid than the national one. Furthermore, population growth rate of small localities is expected to be higher than the national average. The annual national population growth rate is 1.8 percent, while it is 3.3% in Jordan Valley, 7.9% in Megilot-Dead Sea, 7.4% in Mate-Biniamin, and -2.7% in Ma'ale Efraim (due to negative migration balance). According to the CBS, the socio-economic level of the population living in LJRV is ranked as medium to low. The socio-economic profile of this region demonstrates the stress on water resources. Observation from the LJRV basin area have demonstrated conflicts and competition over water use in agriculture and domestic use as population growth increases. This trend brings about water resource stress since more people refers to less water per capita. When there is competition for water resources it exposes the need to justify the allocation of water to one user rather than to another. Water use conflicts are especially noted between agricultural activities and domestic uses. Acceleration in the competition of the water uses (domestic, industrial and agricultural) calls for effective and sustainable water resources management. An integrated water resources perspective ensures that social, economic, environmental and technical dimensions are taken into account in the management and development of water resources (Dungumaro and Madulu 2003).

The study area generally follows a rural settlement pattern and the localities will continue to rely on agriculture as main employment sector. The LJRV localities are mainly classified as agricultural cooperated rural localities (Cooperative Moshav, Kibbutz, Communal localities, and only one urban locality). The industrial branches support the cultivation activities and market the productions for export and for the local markets. Treated waste water is supplied to the agricultural consumers, and it is used mainly for irrigating palm trees. Irrigation of palm trees with this type of water enables allocation of fresh water to other crops, or for domestic uses. The findings indicate that farmers are willing to use water of different qualities. Recognizing the inter-related nature of different sources of water and thus also the inter-related nature and impacts of the differing water uses is a major step to the introduction of IWRM.

It is obvious that there is not enough water in the LJRV region, that water consumption is increasing, and that water quality has been deteriorating rapidly. Furthermore, due to the anticipated increase in tourism, agriculture and domestic water use in the near future, economic growth and viability are linked to the preservation and the increase in sustainable water sources. Local communities and the local trends are ignored by water planners in developing and managing land and water resources (Mulwafu and Msosa 2005). Lack of upcoming water demand and supply plan for this fringe region is the farthest illustration for this saying. Demographic and socio-economic characteristics of the local population should be stronger linked between water management and regional water planning. This multi-disciplinary approach is meant to ensure that local socio-economic aspects will be integrated into future water development and management plan. Through the case study of LJRV, the importance of generalizing socio-economic variables in the management of water resources is shown.

5. ACKNOWLEDGEMENTS

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STRATEGIC CHOICE OF DAMS SITES WITHIN INTEGRATED MANAGEMENT OF WATER RESOURCES

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Abstract: Dam Construction is considered as one of the main aims for Water resources Development in Yemen. However, Dam Construction has many drawbacks such as the lack of considering integrated management in dams construction. The choice of dam location is considered as one of the challenges for the development of dam construction in Yemen. In this research; many necessary criteria have been shown for the choice of dam locations. The program prepared can be used in Yemen and other countries also.

Key words: Dams, dam sites.

1. INTRODUCTION

In Yemen, there is an increase in water demand especially for agricultural, industrial and domestic uses. Also, there are limited water resources in addition to frequent repetition of dry years. Therefore, construction of dams becomes very necessary which started in early eighties. Usually, in the selection of Adam site, physical, economical, social, environmental and political factors should be taken into consideration. Unsuitable of some dam sites in Yemen caused technical problems in those dams. In this study, all factors which influence the selection of Adam site are considered.

2. METHODOLOGY

2.1 Literature Survey

Literature Survey was made to show the aims of dam site selection, and their requirements, and identify the available factors that affect the selection of the dam site.

2.2 Field work

In this part of the research the possible criteria for the selecting suitable dam site in Yemen have been identified by using three ways:

(a) Field survey for the most dams have been constructed and the dam which are under construction (b) Field questionnaire were made for collecting data from the experienced engineers on the construction dam projects and the selection of dam site , selecting of the research sample from The engineers who have no less than five years experience in executing the dam projects. The length of their experience is ranged from (5-20) years, making a questionnaire forms which were made as a result of a literature survey they content the construction dam important, construction methods and comprehensives list of the factors that affect the selection of dam site in Yemen. These factors have further been used as a base to build a management system for selecting suitable dam

site in Yemen. (c) Interviews were made with experts who have a good site experience in execution of dam projects to know the factors and the criteria that affect the selection of dam site in Yemen. These experts gave their decisions on the selection of dam site and the reasons behind their decisions were recorded and arranged as a rules and criteria in suggested management system.

2.3 Developing the suggested management system

In developing a suggested management system for dam site selection, the concepts have been developed to construct a management system that is capable of advising the engineers on the dam projects logistics and selection of dam site. The researcher depended on the literature survey and the field questionnaire answers which both include the influencing factors, in building up system structure. The system has been developed by using six stages as illustrate in figure (1). These six stages reflect the analysis procedures that progress from broad generalization to the specified selection of dam site.

2.4 Running of the developed system

The developed system can be running from the main interface which contains the main six factors (Social factors, Physical factors, Economical factors, Environmental factors, Political factors and Human factors) as illustrate in figure(2). A computer program (Visual C++) was made for this purpose.

2.5 Evaluating the system

Preparing evaluation study has been made to show the system efficiency in selecting the suitable dam site. This made by using a group of engineers were selecting to evaluate the suggested management system.

3. RESULTS

From analyzing the answers in table (1):

All the answers consider that the following factors which are arranged according to their important in the selecting dam site in Yemen are social factors, Physical factors, Political factors, Economical factors, and Environmental and health factors.

Table (2) shows that:

(a) The orders of the importance of the social factors that affect the selection of dam site are citizen desired to accept the project, Establishment of people and prevent migration, Right of water distributions in the region and improve the standard living for the person income.

(b) The orders of the importance of the physical factors which affect the selection of dam site are topographic factors, hydrologic factors, geological factors, available of materials, shape of the shoulders, site and the capacity of the spillway and earthquake factors.

(c) The economical factors that affect the selection of dam site according to their important are cost of the dam, availability of water resources, cost of operation and maintenance, availability of local hand and cost of temporary constructions.

Depending upon the previous factors and the results shown in table (1) and table (2), the researcher reached to build the management system which can be used to select the suitable dam site as shown on fig (1).

The evaluation of system was achieved by presenting the facilities of developed system to the respondents and then being asked to answer a questionnaire form. The years of experience of those selected personnel ranged from (5-20) years of experience as represented in table (3).

The results show:

(a) All the answers show that the running system can be used in selecting suitable dam site .

(b) All the answers show that the running system is simple and easy to be used.

(c) To the question about the program was including all the factors that affect the selection dam site, all the users answered (Yes).

(d) To the question on efficiency of the program in providing selection suitable dam site, six of the users answered that the program was very good, and four of them answered that the program was good.

(e) To the question about if the program were save in time and cost, all the users answered that the program would have a good saving in time and cost that usually spent on the selection of dam sites.

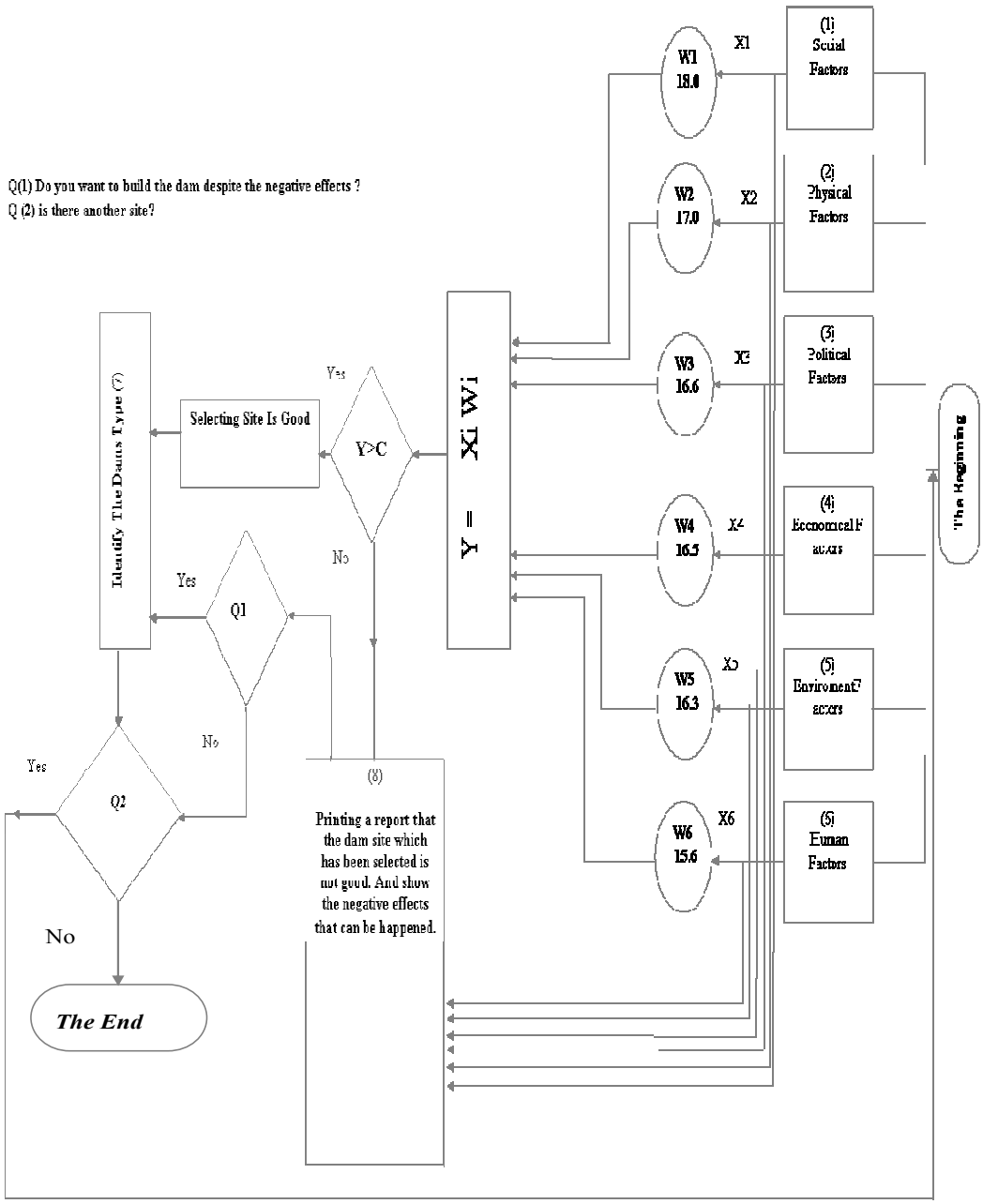


Figure 1. Flowchart of the developed system.

Table 3. percentage of the mean of frequency for collecting data (main factors).

Main Factors	Mean of frequency answers (N)	% of mean of frequency answer (W)
Social Factors	6.94	21
Physical Factors	6.76	20.5
Political Factors	6.46	19.6
Economical Factors	6.43	19.5
Environmental Factors	6.35	19.3

$$W= 100N/\sum \quad (1)$$

Table 2. Percentage of the mean of frequency for collecting data (secondary factors)

Secondary Factors	Frequency answer (N)	%Mean of Frequency (M)
Social Factors, such as:		
Citizen desired to accept the project.	7	26.5
Right of water.	6.45	24.5
Physical Factors, such as:		
Topographic Factors	7.32	8.8
Hydrologic Factors	6.86	8.24
Geological Factors	6.83	8.22
Availability of materials	6.62	7.95
Political Factors, such as:		
Population Social Stability	6.08	26
Providing national security	5.59	24
Economical Factors, such as:		
Cost of the dam.	7.1	19.5
Availability of water resources.	6.54	18
Cost of operation and maintenance	5.78	16
Availability of local labors.	5.75	15.8
Environmental Factors such as:		
Flood mitigation.	6	37
Health improvement.	5.13	31.6
Environment recreation.	**5.08**	**31.4**

$$M=100N/\sum N \quad (2)$$

Table 3. Program evaluation answers.

Res. no	Experience (year)	Running prog	Training for.m. site.	Consider all.f	Prog. applicability	Saving in T & C
1	20	Simple/easy	Very useful	Yes	Very good	Yes
2	15	-	-	-	-	-
3	13	-	-	-	-	-
4	12	-	-	-	-	-
5	10	-	useful	-	-	-
6	10	-	-	-	-	-
7	10	-	-	-	Good	-
8	8	-	-	-	-	-
9	6	-	-	-	-	-
10	5	-	-	-	-	-

4. CONCLUSION

Based on the results obtained on this study, the following conclusions are drowned:

(1) Most of the site engineers have a lack of knowledge about the factors that affect the selection of dam site.

(2) The important factors that affect the selection of dam site are: Social factors, Physical factors, Political factors, Economical factors, Environmental and Human factors.

(3) The social factors that affect the selection of dam site are, Citizen desired to accept the project, Establishment of people and prevent migration, Right of water distributions in the region and improve the standard living for the person income.

(4)The computer program that provides the management system with data bases will be extremely useful especially in cases when experts are hard to find or their consolation is too expensive. This situation is very common in Yemen construction projects.

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A COMPUTER SYSTEM OF ESTIMATING REFERENCE CROP EVAPOTRANSPIRATION USING THE MODIFIED PENMAN'S (FAO - 24) METHOD

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Abstract: The rational designing of irrigation systems requires the calculation of the evapotranspiration in the best possible way. Reference crop evapotranspiration (ETr) is the rate at which water, if available, would be removed from the soil and plant surface of a specific crop, arbitrarily called a reference crop. The classical Penman method is a combination equation that considers both the energy and aerodynamic aspects of the ETr process. Food and Agricultural Organization (FAO) modified the original Penman's method in 1977. This method uses mean daily climatic data, with an adjustment for day and night time weather conditions. The modified Penman's (FAO-24) method is considered to be one of the most accurate methods for the computation of evapotranspiration and must be used when all the required climatological data are available. The required calculating procedure when the above modified Penman's (FAO-24) method is used, is very complex and calls for a lot of calculations. In this paper we present a computer system of estimating reference crop evapotranspiration using the modified Penman's (FAO-24) method. The system facilitates irrigation network designers in calculating reference crop evapotranspiration. It is simple in use, do not demand special informatics knowledge, and it is approachable to a wide range of researchers. System's architecture includes a user-friendly interface for the data input, a computational module and the output subsystem, where the user is given a detailed table of results. The computer system provides a quick and accurate solution to the problem of estimating reference crop evapotranspiration. In addition, it could be incorporated as subroutine of integrated software dealing with hydraulic calculations.

Key words: Computer system, Crop evapotranspiration, Irrigation, Penman's method.

1. INTRODUCTION

The rational designing of irrigation systems requires the calculation of the evapotranspiration in the best possible way. Reference crop evapotranspiration (ET_r) is the rate at which water, if available, would be removed from the soil and plant surface of a specific crop, arbitrarily called a reference crop. The classical Penman's method (Penman 1963) is a combination equation that considers both the energy and aerodynamic aspects of the ET_r process. Food and Agricultural Organization (FAO) modified the original Penman's method in 1977. This method uses mean daily climatic data, with an adjustment for day and night time weather conditions. The modified Penman's (FAO-24) method is considered to be one of the most accurate methods for the computation of evapotranspiration and must be used when all the required climatological data are available.The required calculating procedure when the above modified Penman's (FAO-24) method is used, is very complex and calls for a lot of calculations. In this paper we present a computer system of estimating reference crop evapotranspiration using the modified Penman's (FAO-24) method. The system facilitates irrigation network designers in calculating reference crop evapotranspiration. It is simple in use, do not demand special informatics knowledge, and it is approachable to a wide range of researchers. System's architecture includes a user-friendly interface for the data input, a computational module and the output subsystem, where the user is given a detailed table of results. The computer system provides a quick and accurate solution to the problem

of estimating reference crop evapotranspiration. In addition, it could be incorporated as subroutine of integrated software dealing with hydraulic calculations.

2. THE MODIFIED PENMAN'S (FAO-24) METHOD

Food and Agricultural Organization (FAO) modified the original Penman equation (Penman 1963) in 1977. The modifications included a revised wind function, an adjustment factor to account for local conditions and the assumption that the daily average ground heat flux is zero. This method uses mean daily climatic data, with an adjustment for day and night time weather conditions. The modified equation used in this method (Doorenbos and Pruitt 1977; Jennifer, 2001) is:

$$ET_r = \frac{C}{\rho_w}\left[W\frac{(R_n - G)}{\lambda} + (1-W)f(u)(e_a - e_d)\right] \quad (1)$$

where ET_r is the reference evapotranspiration, mm/day; W is a temperature and the altitude of the area related weighting factor; R_n is the net solar radiation, $MJm^{-2}d^{-1}$; G is the soil heat flux in $MJm^{-2}d^{-1}$; λ is the latent heat of evaporation, MJ kg^{-1}; ρ_w = 1000 kgm^{-3}, is the density of water; f(u) is the wind related function, kg $hPa^{-1}m^{-2}d^{-1}$; e_a is the saturation vapour pressure at mean air temperature, hPa; e_d is the mean actual vapour pressure of the air, hPa; and C is an adjustment factor to account for day and night weather conditions.

2.1 Weighting Factor (W)

This is a weighting factor for the effect of solar radiation, the wind and humidity on ET_r. It is given by the equation:

$$W = \Delta/(\Delta + \gamma) \quad (2)$$

where Δ is the rate of change of saturation vapour pressure with temperature, hPa $^0C^{-1}$, and γ is the psychometric constant, hPa $^0C^{-1}$, which are calculated from the equations provided by Brunt (Brunt 1952), and Bosen (Bosen 1960) respectively:

$$\Delta = \frac{4098}{[T_a + 237.3]^2}\ 6.11\ \exp\left[\frac{17.27\ T_a}{T_a + 237.3}\right]\ \text{hPa}\ ^0C^{-1} \quad (3)$$

and

$$\gamma = \frac{\left(1 - \frac{0.0065}{273 + T_a}h\right)^{5.2586}}{1.5156 - 0.0014307\ T_s}\ \text{hPa}\ ^0C^{-1} \quad (4)$$

where T_a and T_s are the daily mean air and water temperature, ^0C; and h is the altitude of the area, m.

2.2 Humidity (e_a-e_d)

Humidity is expressed as the difference between the mean saturation water vapour pressure (e_a) and the mean actual vapour pressure (e_d). From Tetens (Tetens 1930), saturated air vapour pressure as a function of temperature, e_a, can be computed as follows:

$$e_a = 6.11\exp\left[\frac{17.27\ T_a}{T_a + 237.3}\right]\ \text{hPa} \quad (5)$$

The actual vapour pressure is calculated using saturation vapour pressure and the mean relative humidity:

$$e_d = e_a Rh_{mean} \quad hPa \tag{6}$$

Therefore:

$$e_a - e_d = (1 - Rh_{mean}) 6.11 \exp\left[\frac{17.27\, T_a}{T_a + 237.3}\right] \quad hPa \tag{7}$$

2.3 Wind Factor (f(u))

Wind Factor expressing the effect of wind on ET_r using the modified Penman's (FAO-24) method, is calculated (Doorenbos and Pruit 1997) from:

$$f(u) = 0.27(1 + 0.86 u_2) \quad kg\ hPa^{-1} m^{-2} d^{-1} \tag{8}$$

Wind speed is measured at many different heights above the ground surface. The Penman's equation requires wind speed at a height of 2.00 m. It can be adjusted to a height of 2.00 m by:

$$u_2 = u_z \left(\frac{2}{z}\right)^{0,2} \quad ms^{-1} \tag{9}$$

where: u_2 and u_z is the wind speed corresponded at height of 2.00 m and z m, in ms^{-1}; and z is the height of wind measurement, m.

2.4 Net Radiation (R_n)

Net Radiation is the difference between incoming and outgoing solar radiation. It can be measured, but such data are seldom available. Instead, it is calculated from solar radiation on sunshine hours, temperature, and humidity data (Koutsoyiannis and Xanthopoulos 1997; Papazafiriou 1999). The FAO-24 Penman's approach calculates R_n as follows:

The net radiation is the algebraic sum of the net shortwave radiation (R_{ns}) and net longwave radiation (R_{nl}). As R_{nl} constitutes a loss, this relationship is:

$$R_n = R_{ns} - R_{nl} \qquad MJm^{-2} d^{-1} \tag{10}$$

Net shortwave radiation (R_{ns}), can be calculated by correcting the incoming solar radiation for surface albedo (α) of the crop surface by:

$$R_{ns} = (1 - \alpha) R_s \quad MJm^{-2} d^{-1} \tag{11}$$

where: $\alpha = 0.05$ for water surface; $\alpha = 0.15$ for bare soil surfaces; and $\alpha = 0.23$ for green vegetated surfaces. For most crops $\alpha = 0.25$.

Solar radiation (R_s), is calculated using the amount of radiation received at the top of the atmosphere (R_a). This value is calculated (Duffie and Beckman 1980,1991) by:

$$R_a = 3.6 \frac{2\, I_s\, d_r}{15} \left(\omega_s \sin\varphi \sin\delta + \frac{180}{\pi} \sin\omega_s \cos\varphi \cos\delta \right) \quad MJm^{-2} d^{-1} \tag{12}$$

where: $I_s = 1.367\ kWm^{-2}$ is the solar constant; d_r is the relative distance from the sun to the earth; φ is the station latitude, degrees; δ is the declination of the sun, degrees; ω_s is the sunset hour angle, degrees; The values of d_r, δ and ω_s are calculated respectively from:

$$d_r = 1 + 0.034 \cos\left(\frac{360}{365} J - 2.86479\right)^0 \quad (13)$$

$$\delta = 23.45^0 \sin\left(\frac{360}{365}(284 + J)\right)^0 \quad (14)$$

and

$$\omega_s = \arccos(-\tan\varphi \tan\delta) \quad \text{degrees} \quad (15)$$

In the above relations, J is the Julian day of the year considered given from:

$$J = 30{,}5M - 14{,}6 \,(\text{integer}) \quad (16)$$

where $M = 1,2,3,\ldots,12$ is the number of the month considered.

After that solar radiation (R_s), is obtained with the Angstrom formulation:

$$R_S = \left(0.25 + 0.50\frac{n}{N}\right) R_a \quad MJm^{-2}d^{-1} \quad (17)$$

where n and N are correspondently the actual and the maximum possible sunshine hours , hd^{-1}; and

$$N = \frac{2}{15}\omega_s \quad hd^{-1} \quad (18)$$

Net longwave radiation (R_{nl}), can be determined from available temperature, the vapour pressure and the n/N ratio data by the equation:

$$R_{nl} = \varepsilon_n f_L \sigma (T_a + 273)^4 \quad MJm^{-2}d^{-1} \quad (19)$$

where $\sigma = 4.903 * 10^{-9}$, MJ K^{-4} m^{-2} day^{-1}, is the Stefan-Boltzmann constant ; ε_n is the net emissivity factor which is calculated by $e_n = 0.34 - 0.044\sqrt{e_d}$; and f_L is the cloudiness adjustment factor which is calculated by $f_L = 0.1 + 0.9\frac{n}{N}$.

Therefore R_{nl} can be determined by:

$$R_{nl} = \sigma(T_a + 273)^4 \left[0.34 - 0.044\sqrt{e_d}\right]\left[0.1 + 0.9\frac{n}{N}\right] \quad MJm^{-2}d^{-1} \quad (20)$$

2.5 Soil Heat Flux (G)

Soil heat flux is calculated from the relation provided by Jensen et al. (1990):

$$G = c_s d_s \frac{|T_{n+1} - T_{n-1}|}{\Delta t} \quad \text{or} \quad G = 2 c_s d_s \frac{|T_n - T_{n-1}|}{\Delta t} \quad MJ\ m^{-2}d^{-1} \quad (21)$$

where T_{n+1}, T_n and T_{n-1} are respectively the mean temperature on next month n+1, month considered n, and on previous month n-1, ^0C; c_s is the soil heat capacity = 2.1MJ m^{-3} $^0C^{-1}$; Δt is the length period ; d_s is the estimated effective soil depth and $d_s = 0{,}18$ m for daily temperature fluctuations and $d_s = 0{,}03$ m for monthly temperature fluctuations (Jensen et. al. 1990).

Therefore G can be determined by: $G = 0{,}38|T_d - T_{d-1}|$ MJ $m^{-2}d^{-1}$ for daily temperature fluctuations and by: $G = 0{,}14|T_n - T_{n-1}|$ or $G = 0{,}07|T_{n+1} - T_{n-1}|$ MJ $m^{-2}d^{-1}$ for monthly temperature fluctuations.

Note: The soil heat flux is generally very small and it is assumed to be $G = 0.01R_n$ during the day and $G = 0.02R_n$ during the night. According to FAO-24 modifications it is assumed that the daily average ground heat flux is zero.

2.6 Adjustment Factor (C)

The Penman equation assumes that the most common conditions occur when radiation is medium to high, maximum relative humidity is medium to high and moderate daytime wind about double the night time wind. These conditions are not always met. The Penman equation has to be corrected in such situations using an adjustment factor. The adjustment factor as developed by Allen et al. (1989) and obtained from Jensen et al. (1990) is given by:

$$C = 0.892 - 0.0781u_d + 0.00219\,u_d\,R'_s + 0.000402\,Rh_{max}\,R'_s + 0.000196\frac{u_d}{u_n}u_d\,Rh_{max} + 0.0000198\frac{u_d}{u_n}u_d\,Rh_{max}R'_s + 0.00000236\,u_d^2 Rh_{max}\,R'_s - 0.0000086\left(\frac{u_d}{u_n}\right)^2 u_d Rh_{max} - 0.0000000292\left(\frac{u_d}{u_n}\right)u_d^2\,Rh_{max}^2\,R'_s - 0.0000161Rh_{max}\,R'^2_s \quad (22)$$

where u_d and u_n are correspondently the daytime and night time wind speed, ms^{-1};

$$\lambda = 2.501 - 0.002361T_s \ \text{MJ kg}^{-1}\ ; \quad (23)$$

$$\text{and, } R'_s = \frac{R_s}{\rho_w \lambda} \ \text{mmd}^{-1} \quad (24)$$

Note: The modified Penman's (FAO 24) method requires day and night wind speed. If the wind data to evaluate the correction factor are not available, it is adequate to estimate that the u_d/u_n value is 1.5 during the convective season and 1.85 during the frontal season.

2.7 Reference Crop Evapotranspiration (ET_r)

After combining the above equations, the modified Penman's (FAO-24) equation is given by:

$$ET_r = \frac{C}{\rho_w}\left[W\frac{R_n}{\lambda} + (1-W)f(u)(e_a - e_d)\right] \quad (25)$$

where all the parameters used, are as above have been described.

3. ESTIMATING REFERENCE CROP EVAPOTRANSPIRATION ACCORDING TO THE MODIFIED PENMAN'S METHOD (FAO-24) USING PERSONAL COMPUTER

The required calculating procedure when the above modified Penman's (FAO-24) method is used, is very complex and calls for a lot of calculations. For this reason, we found necessary to develop a simple computer system that enables researchers and students to easily calculate reference crop evapotranspiration. This program, being accessible by those who need it, could be embedded as subroutine in larger computer systems dealing with hydraulic calculations.

3.1 Organising the Input Parameters

The first step is to organise the data for the problem on the spreadsheet. In Figure 1, the input parameters for the altitude of the area, h in m, the height of wind speed measurement, z in m , the wind speed corresponded at height z, u_z in m/s, the number of the month considered, M, the station latitude, φ in degrees, the daily mean air temperature, Ta in 0C, the daily mean water temperature, Ts in 0C, the actual possible sunshine hours, n in h/d, the ratio of daytime and nighttime wind speed ud/un, the mean relative humidity, Rhmean in %, and the maximum relative humidity, Rhmax in %, were entered in cells e15, i15, m15, e18, i18, m18, e21, i21, m21, e24 and i24 respectively.

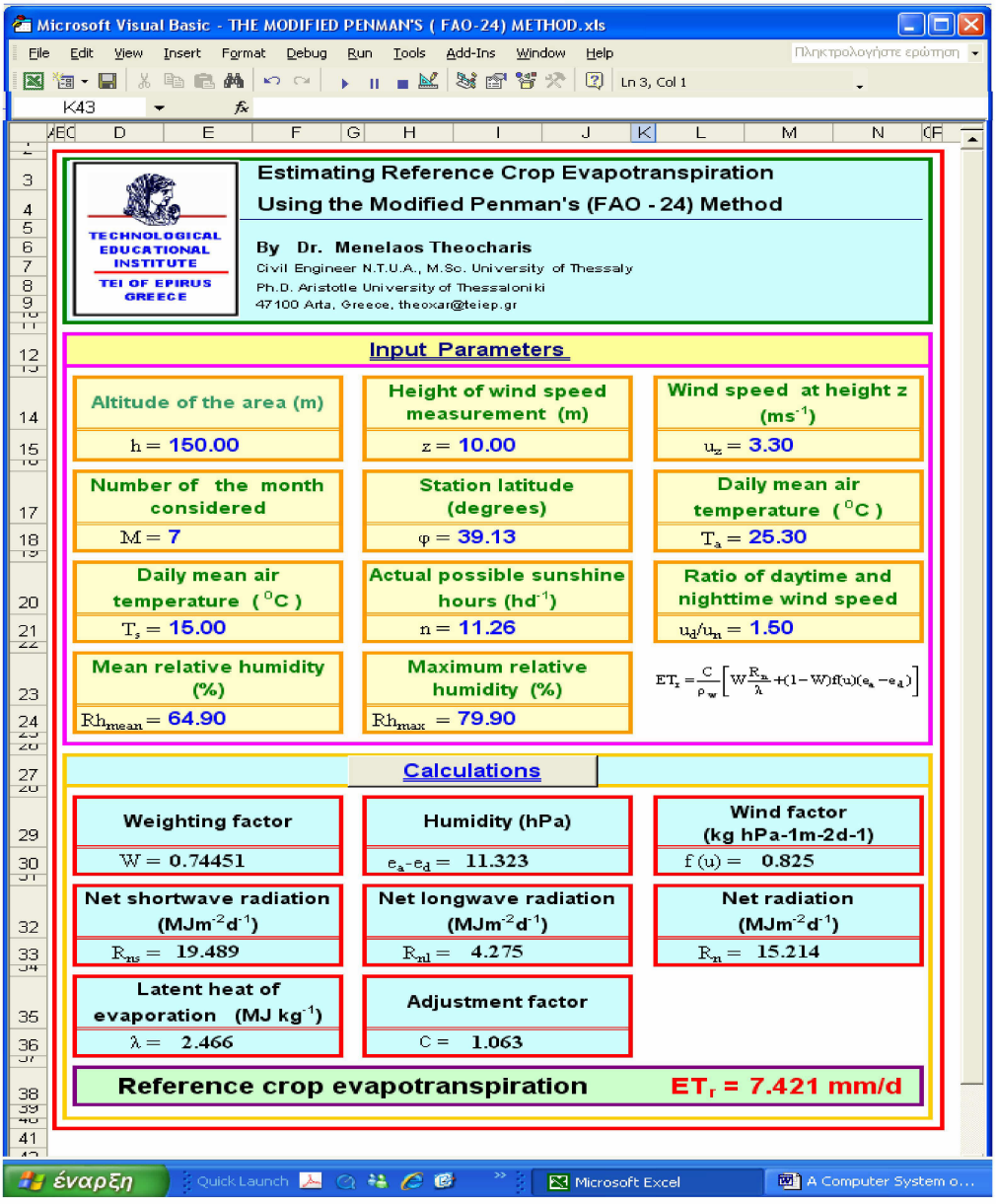


Figure 1. Spreadsheet of data and calculations of ETr using the modified Penman's (FAO-24) method.

3.2 Solving the Problem

In order to solve the problem the following macro command is created:

```
Sub Crop_Evapotranspiration ()
'Macro recorded in 24/1/2009 by M. Theocharis
```

```
 Range("f3") = "Estimating Reference Crop Evapotranspiration "
 Range("f4") = "Using the Modified Penman's Method (FAO 24)"
 Range("h12") = "Input  Parameters "
 h = Range("e15").Value
 z = Range("i15").Value
 uz = Range("m15").Value
 M = Range("e18").Value
 φ = Range("i18").Value
 Ta = Range("m18").Value
 Ts = Range("e21").Value
 n = Range("i21").Value
 Ru = Range("m21").Value
 Rhmean = Range("e24").Value
 Rhmax = Range("i24").Value
 Δ = (4098/((Ta+237.3)^2)*6.11*exp(17.27*Ta/(Ta+237.3)))
 γ = ((1-0.0065*h/(273+Ta))^5.2586)/(1.5156-0.0014307*Ts)
 W = Δ/(Δ+γ)
 Range("e30") = W
 ea = 6.11*Exp(17.27*Ta/(Ta+237.3))
 ed = ea*Rhmean/100
 Range("i30") = ea-ed
 Fu = (0.27*(1+0.86*uz*(2/z)^0.2))
 Range("m30") = Fu
 J = Int(30.5*M-14.6)
dr =1+0.034*Cos((4*Atn(1)/180)*(360*Int(30.5*M-14.6)/365-2.86479))
 δ = 23.45*Sin((4*Atn(1)/180)*(360*(284+Int(30.5*M-14.6))/365))
 cosωs = -Tan(4*Atn(1)*φ/180)*Tan(4*Atn(1)*δ/180)
 ωs = (180/(4*Atn(1)))*(Atn(-cosωs/Sqr(-cosωs*cosωs+1))+2*Atn(1))
 Ra = 1/15*(3.6*2*1.367*dr)*(ωs*Sin(4*Atn(1)*φ/180)*Sin(4*Atn(1)*δ/180)+(180/(4*Atn(1)))*
        Sin(4*Atn(1)*ωs/180)*Cos(4*Atn(1)*φ/180)*Cos(4*Atn(1)*δ/180))
 N = (2/15)*ωs
 Rs = (0.25+0.5*n/N)*Ra
 Rns = (1-0.25)*Rs
 Range("e33") = Rns
 Rnl = 4.903*10^-9*((Ta+273)^4)*(0.34-0.0442719*ed^0.5)*(0.1+0.9*n/N)
 Range("i33") = Rnl
 Rn = Rns-Rnl
 Range("m33") = Rn
λ = 2.501-0.002361*Ts
 Range("e36") = λ
 C = 0.892-0.0781*(uz* (2/z)^0.2)+0.00219*(uz*(2/z)^0.2)* Rs/λ+0.000402* Rhmax* Rs/λ+
      0.000196* Ru*(uz* (2/z)^0.2)* Rhmax+0.0000198* Ru*(uz* (2/z)^0.2)* Rhmax *Rs/λ+
      0.00000236* (uz*(2/z)^   0.2)^2*Rhmax*Rs/λ-0.0000086*Ru^2*(uz*(2/z)^0.2)*Rhmax-
      0.0000000292* Ru*(uz*(2/z)^0.2)^2* Rhmax^2*Rs/λ-0.0000161*Rhmax*(Rs/λ)^2
Range("i36") = C
ETr = C*(W*Rn /λ+(1-W)*(0.27*(1+0.86*uz*(2/z)^0.2))*(ea-ed))
Range("m38") = Round(ETr,3)&" mm/d"
End Sub
```

After that we click the **“Solutions”** button and the solution of the problem results.The described procedure is shown in Figure 1.

4. APPLICATION

Below the reference crop evapotranspiration in Arta, Greece area for the month July are estimated, using the above described computer system, Figure 1. For the input parameters: h = 150.00 m; z =10.00 m ; u_z = 3.30 m/s; M=7 ; φ = 39.13 ^{0} ; T_a = 25.30 ^{0}C ; T_s =15.00 ^{0}C ; n =11.26 hd^{-1}; u_d/u_n = 1.50 ; Rh_{mean} = 64.90 % ; and Rh_{max} = 79.90 % ; it emerges that: W = 0.7445 ; ea-ed = 11.323 hPa ; f(u) = 0,825 kg hPa^{-1}m^{-2}d^{-1} ; R_{ns}= 19.489 MJm^{-2}d^{-1}; R_{nl} = 4.275 MJm^{-2}d^{-1} ; R_n =15.214 MJm^{-2}d^{-1}; λ= 2.466 MJ kg^{-1}; C = 1.063 and finally the reference crop evapotranspiration ET_r =7.421 mm/d.

5. CONCLUSIONS

Solving problems of estimating reference crop evapotranspiration is of prime importance for the study of irrigation networks.

The required calculating procedure when the modified Penman's (FAO-24) method is used is very complex and calls for a lot and very laborious calculations. This is why FAO developed various tables where the values of the calculating parameters are provided.

But these tables are very unpractical in use and include significant inaccuracies in values determination. Besides, it is impossible to use them when trying to solve the problems using a computer.

The computer system presented in this paper is simple in use, does not demand special informatics knowledge and is approachable to a wide range of researchers. In addition, it could be incorporated as subroutine of integrated software dealing with irrigation calculations.

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COMPUTATION OF HYDROGRAPHS IN EVROS RIVER BASIN

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Abstract: Evros River basin is a transboundary basin which extends in three countries: Bulgaria, Greece and Turkey. The whole basin area is about 53 000 km2. 66% of the basin area belong to Bulgaria, 28% to Turkey and 6% to Greece. Evros River length in Bulgaria (Maritsa River) is about 321 km, in Greece about 218 km, while 203 km are the natural border between Greece and Turkey. In this study, continuous hydrographs, due to continuous hyetographs of the time period October 2004 – September 2006 including some extreme flood events, were computed at the outlets of the sub-basins of Evros River by means of the well-known hydrologic software HEC – HMS. In concrete terms, the Bulgarian part of Evros (Maritsa) River basin was divided into 27 sub-basins and the Greek part into 13 sub-basins. Apart from that, the basin of Tountza River, which is a tributary of Evros River flowing through Bulgaria and Turkey, was divided into three sub-basins. Finally, the basin of Erythropotamos River, which is a tributary of Evros River flowing through Bulgaria and Greece, was also considered. The computed values of water discharge in the Greek part of Evros River basin are very extreme for certain time periods.

Key words: continuous hyetograph, continuous hydrograph, Evros River, transboundary basin

1. INTRODUCTION

The extreme floods occurred last years in the Greek part of Evros River were the motivation for the present computational study. A time period including some extreme rainfall events (October 2004 – September 2006) was selected for this study.

Evros River basin is a transboundary basin which extends in three countries: Bulgaria, Greece and Turkey. The whole basin area is about 53 000 km^2. 66% of the basin area belong to Bulgaria, 28% to Turkey and 6% to Greece. Evros River length in Bulgaria (Maritsa River) is about 321 km, in Greece about 218 km, while 203 km are the natural border between Greece and Turkey.

By means of the well-known hydrologic software HEC – HMS (2000), continuous hydrographs at the outlets of the sub-basins of Evros River basin in Bulgaria and Greece were computed on the basis of continuous hyetographs in the sub-basins considered for the time period October 2004 – September 2006.

Generally, by means of the above hydrologic software, the hydrograph for any cross-section of Evros River with high flood risk can be computed.

2. SHORT DESCRIPTION OF HYDROLOGIC MODEL

The hydrologic software used in this study contains four models: a rainfall excess model, a model for the transformation of rainfall excess to direct runoff hydrograph, a baseflow model and a routing model for the hydrographs. These models are briefly described in the following sections.

2.1 Rainfall Excess Model

According to Soil Conservation Service method (SCS 1972), the rainfall excess is computed by the relationship:

$$h_R = \frac{(h_r - 0.2S)^2}{h_r + 0.8S} \tag{1}$$

where
h_R: rainfall excess (mm)
h_r: rainfall (mm)
S: maximum hydrologic losses (mm)

The maximum hydrologic losses are computed by the equation:

$$S = \frac{25400}{CN} - 254 \tag{2}$$

where CN is the curve number which can be estimated as a function of land use, soil type and antecedent moisture conditions, using tables published by the SCS.

2.2 Model for the Transformation of Rainfall Excess to Runoff Hydrograph

The hydrograph of direct runoff is computed on the basis of the theory of unit hydrograph. In the dimensionless synthetic unit hydrograph of SCS (1972), discharges are expressed as a portion of the peak discharge q_p and time steps as a portion of the rise time of the unit hydrograph T_p. Knowing the peak discharge and the lag time t_p for a rainfall excess of specific duration, the unit hydrograph can be estimated by the dimensionless synthetic unit hydrograph for the study basin.

The values of q_p and T_p are estimated by a triangular unit hydrograph, where the recession time is equal to $1.67T_p$. The area which is surrounded by the curve of the unit hydrograph must be equal to the rainfall excess of 1 cm. The peak discharge is

$$q_p = \frac{CA}{T_p} \tag{3}$$

where
$C = 2.08$
A: basin area (km^2)
T_p: rise time of unit hydrograph (hr)

In addition, the studied unit hydrographs in small and large basins show that the lag time of the basin t_p is approximately equal to $0.60t_c$, where t_c is the concentration time. The time T_p is a function of the lag time t_p and the duration of the rainfall excess t_R:

$$T_p = \frac{t_R}{2} + t_p \tag{4}$$

2.3 Exponential Recession Model

The time variation of baseflow is represented by an exponential recession model (Chow et al. 1988), which is described mathematically by the following equation:

$$Q_{bt} = Q_{b0} k^t \tag{5}$$

where

Q_{bt} : baseflow at any time t

Q_{b0} : initial baseflow (at time zero)

k : exponential decay constant; it is defined as the ratio of the baseflow at time t to the baseflow one day earlier.

According to the baseflow model, a threshold value has to be specified as a flow rate or as a ratio to the computed peak flow (Figure 1).

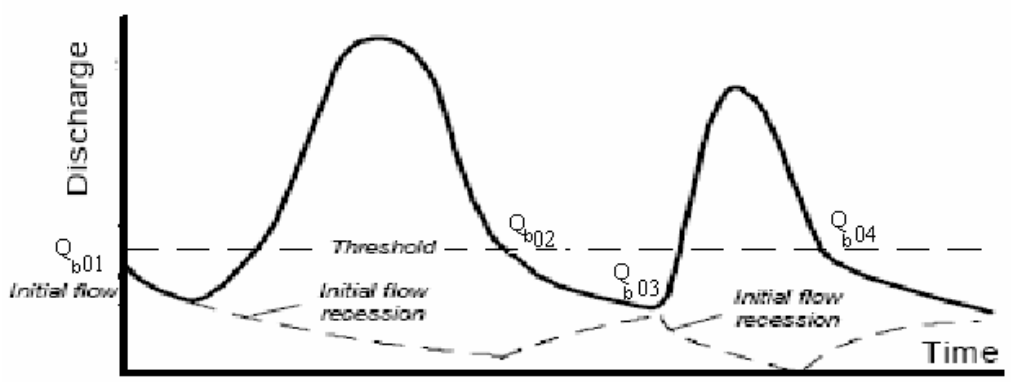


Figure 1. Recession with multiple runoff peaks

Equation (5) is applied twice to a hydrograph: (a) to simulate the initial flow (baseflow) recession; in that case, the value Q_{b01} lies on the discharge axis, and (b) to simulate the total runoff recession; in that case, the value Qb_{02} corresponds to the point of intersection between the threshold line and the recession limb of the hydrograph (Figure 1). For the subsequent second hydrograph of Figure 1, the value Q_{b03} corresponds to the end of the first hydrograph, while the value Q_{b04} is defined as the value Q_{b02} in the first hydrograph.

2.4 Hydrograph routing model

The routing of the total hydrograph (direct runoff + baseflow) from the outlet of a sub-basin to the outlet of the whole basin is enabled by means of Muskingum – Cunge model (HEC – HMS 2000). This model is based on the widely known hydrologic routing Muskingum model, including the parameters K and x that can not be easily estimated. The basic equation of the model is given below:

$$Q_{i+1}^{k+1} = C_o Q_i^{k+1} + C_1 Q_i^k + C_2 Q_{i+1}^k \tag{6}$$

where Q is the total runoff discharge (direct runoff + baseflow), i designates the space step Δx and k designates the time step Δt. The coefficients C_o, C_1 and C_2 are defined by the following equations:

$$C_o = \frac{c\lambda - 2x}{2(1-x)+c\lambda} \qquad C_1 = \frac{c\lambda + 2x}{2(1-x)+c\lambda} \qquad C_2 = \frac{2(1-x)-c\lambda}{2(1-x)+c\lambda} \qquad K = \frac{\Delta x}{c} \qquad \lambda = \frac{\Delta t}{\Delta x} \tag{7}$$

The product $c\lambda = c(\Delta t / \Delta x) = C$ is called the Courant number and is equal to the ratio of the celerity of small waves c to the grid celerity $\Delta x / \Delta t$.

When $x = 0.5$, there is no numerical diffusion, while when $x < 0.5$, there is numerical diffusion. In addition, when $C = 1$, there is no numerical dispersion, while when $C \neq 1$, there is numerical dispersion.

By equating the hydraulic physical diffusion coefficient $\nu_h = q_o / 2S_f$ with the numerical diffusion coefficient $\nu_n = c\Delta x(0.5 - x)$ of the numerical scheme, the value of the parameter x is obtained:

$$x = \frac{1}{2}\left(1 - \frac{q_o}{S_f c \Delta x}\right) \tag{8}$$

where
c: small wave celerity
q_o: reference discharge per unit width (from the inlet hydrograph)
S_f: energy slope

By using Equation (8) the parameter x is computed by means of the physical characteristics of the routing stream (Δx, S_f, c, q_o).

3. MODEL APPLICATION TO EVROS RIVER BASIN

For the application of the hydrologic model described above to Evros River basin, the Bulgarian part of Evros (Maritsa) River basin was divided into 27 sub-basins and the Greek part into 13 sub-basins. Apart from that, the basin of Tountza River, which is a tributary of Evros River flowing through Bulgaria and Turkey, was divided into three sub-basins. Finally, the basin of Erythropotamos River, which is a tributary of Evros River flowing through Bulgaria and Greece, was also considered (Figure 2). In the Bulgarian part of Evros River basin there are some dams, whose basins were neglected in the present study (Figure 2, shaded areas). The basin of Erginis River, which flows through Turkey, was also neglected, because it discharges its waters into the lower part of Evros River basin (Figure 2, shaded area).

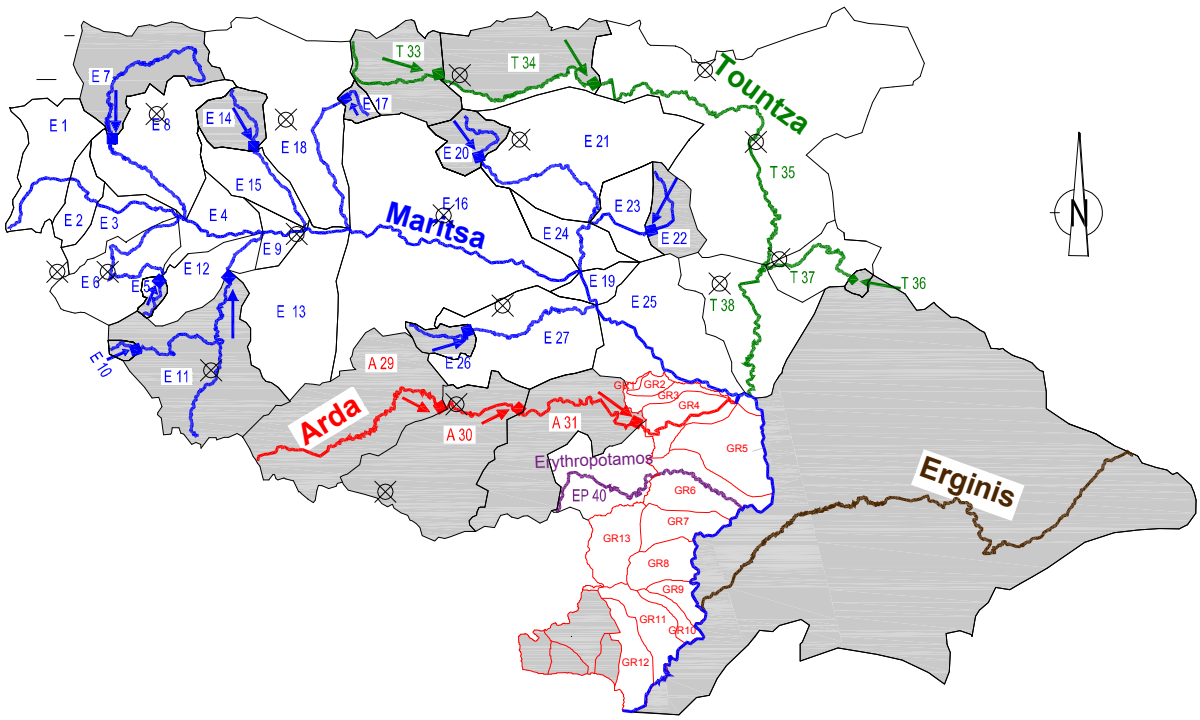


Figure 2. Sub-basins of Evros River basin in Greece and Bulgaria

The system of the sub-basins and the routing reaches of the hydrographs, according to the software HEC – HMS, is depicted in Figure 3.

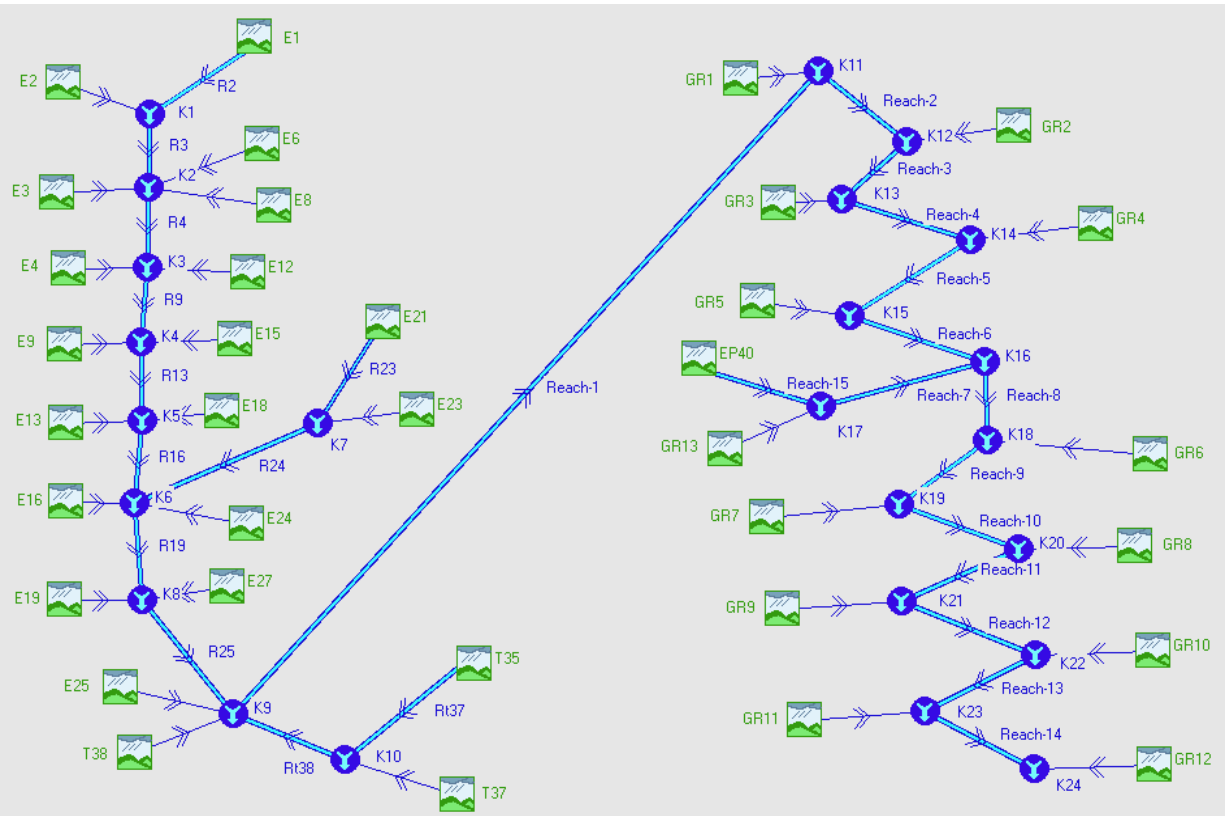


Figure 3. System of sub-basins and routing reaches according to the software HEC – HMS

Daily rainfall amounts of 17 stations in Bulgaria and 12 stations in Greece (Evros District), for the time period October 2004 – September 2006, were used in this study.

In Figure 4, the hyetograph for the sub-basin E16 (Bulgaria, Figure 2), for the time period October 2004 – September 2006, is given indicatively (Mystakidis 2008).

4. COMPUTATIONAL RESULTS

In Figure 4, the hydrograph at the outlet of the sub-basin E16 (Bulgaria), due to the corresponding hyetograph in the same figure, is given indicatively. The area of the sub-basin E16 is 3720 km^2. According to Figure 4, the peak discharge is 533 m^3/s (July 5, 2005), while the corresponding rainfall amount is 25 mm. However, if the routing of the hydrographs of the upstream sub-basins is taken into account, the discharge at the outlet of the sub-basin E16 is 3222 m^3/s.

In Figure 5, the hydrograph at the junction K15 (Figure 3) is given indicatively. This junction corresponds to the village Pythio (Greece). According to Figure 5, the peak discharge is 3307 m^3/s (July 7, 2005).

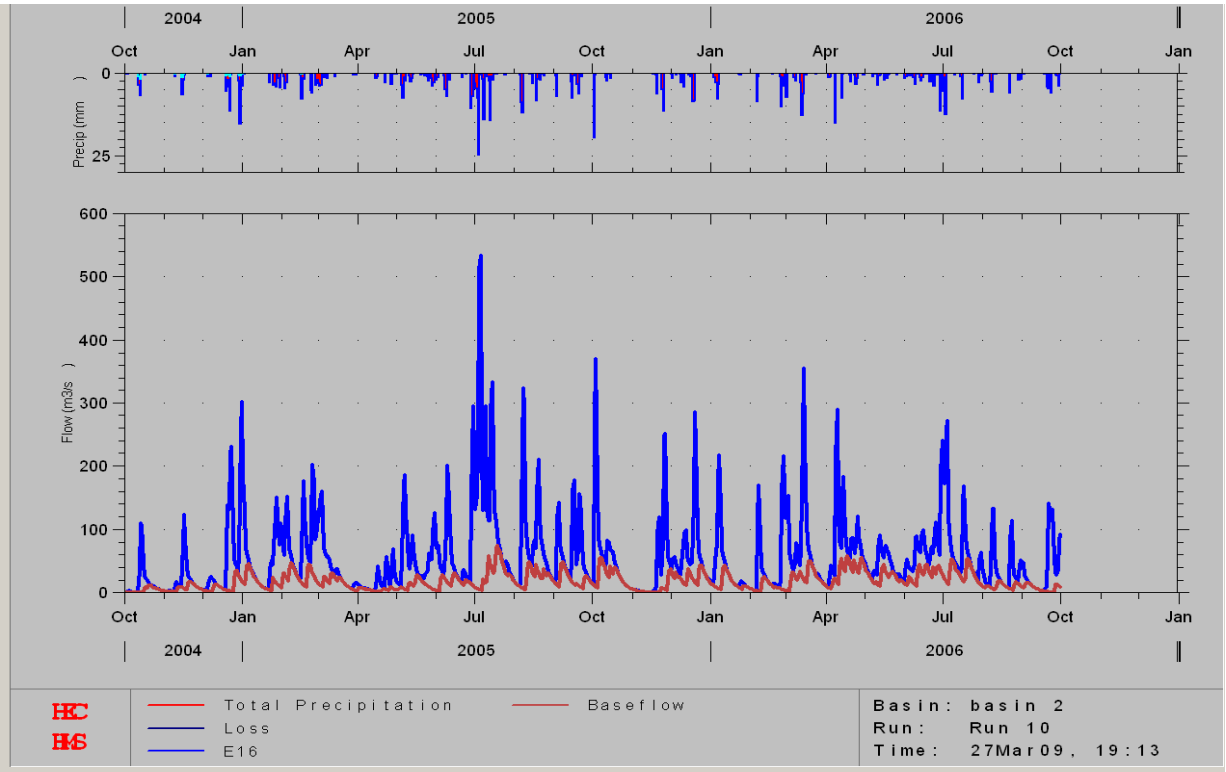


Figure 4. Hydrograph at the outlet of sub-basin E16 (Bulgaria) due to the corresponding hyetograph

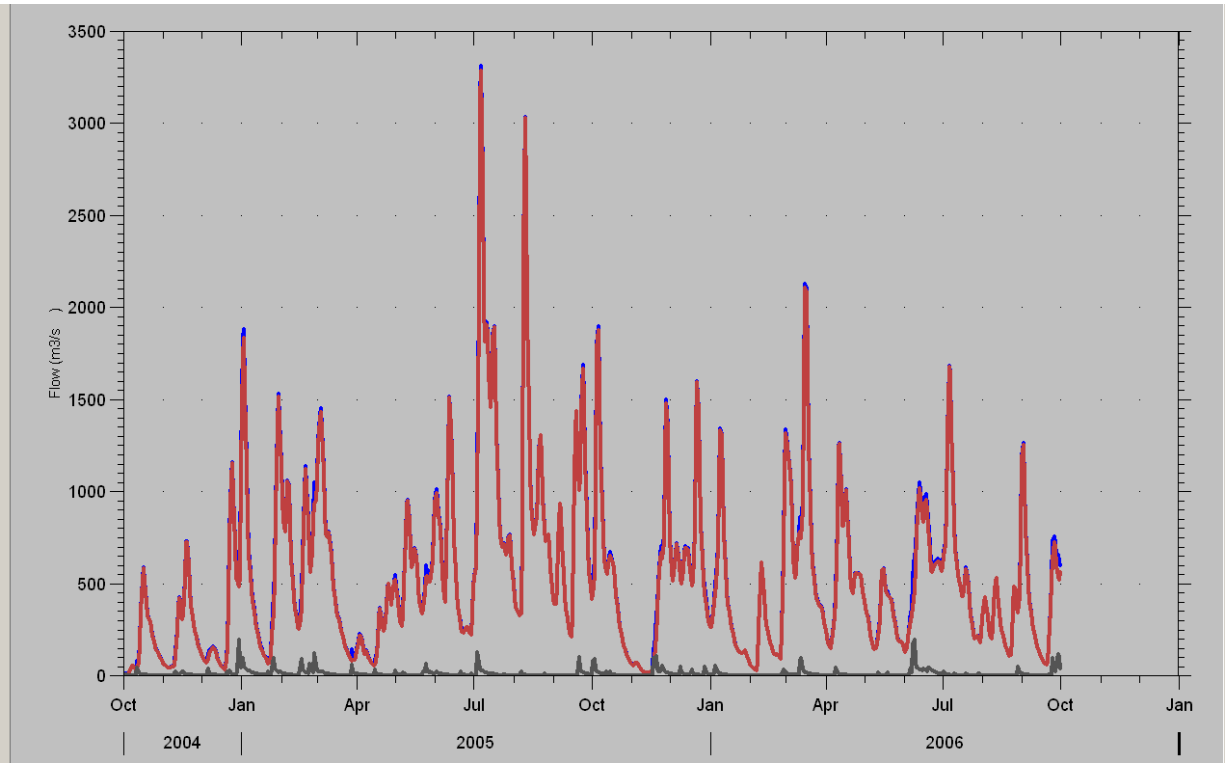


Figure 5. Hydrograph at the junction K15 (Pythio, Greece)

5. CONCLUSIONS

The computed continuous hydrographs in the transboundary basin of Evros River (Bulgaria, Greece and Turkey), for a time period of two years (2004 – 2006), include many extreme flood events.

The hydrologic software HEC-HMS used in this study is a very useful tool for predicting extreme flood events in large basins.

The size order of the computed discharges in the village Pythio (Greece) is comparable with the size order of the measured discharges in the same place.

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A COMPARATIVE STUDY OF GEOSTATISTICAL METHODS FOR DAILY RAINFALL

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Abstract: Prediction of daily rainfall over a region is one of important issues in water resources management and planning. Accurate estimation of the spatial distribution of rainfall is critical to the successful modeling of hydrologic processes. The ability of obtaining high resolution estimates of spatial variability in rainfall fields becomes important for identification of locally intense storms which could lead to floods. The accurate estimation of the spatial distribution of rainfall requires a very dense network of instruments, therefore, it is the necessary to estimate point rainfall at unrecorded location from values at surrounding sites. In this study, 3 methods were used to estimate annual rainfall using interpolation techniques in GIS system. These methods were: Inverse Distance Weighted (IDW), Kriging, and Spline. These geostatistics method, which is based on the theory of regionalized variables, is increasingly preferred because it allows one to capitalize on the spatial correlation between neighboring observations to predict attribute values at unsampled locations and dose not that ignore both the elevation and rainfall records at surrounding stations. The data set including daily rainfall collected from rain gauge stations of study area were used. Normality and homogeneity of data were examined. Elevation was used as co variable and its correlation with rainfall evaluated. Variographic analysis using three different techniques were applied for predicting annual rainfall then statistical comparison of the results obtained by three methods.

Key words: Geostatistics, GIS, Interpolation, Iran, Rainfall.

1. INTRODUCTION

Continuous rainfall data in grid format are required to run models for hydrological and agricultural research as well as water resources planning and management. There are different methods for regional prediction of precipitation such as Theissen and arithmetic average approaches. These methods are easy to use but do not consider correlation and location of data which may lead to low accuracy of the results. Application of geostatistics techniques in hydrological sciences is a useful approach to avoid such errors and increase of calculation accuracy as well (4). In classic statistics samples taken from a population are lack of spatial properties. Therefore the calculated values of a parameter in a homogeny sample do not include any information of the same parameter in another sample with a defined distance. Geostatistics consider the value as well as location of the sample. Then it is possible to analyze value and location of the samples together. To achieve this purpose it is necessary to relate spatial properties (distance, direction) of different samples using mathematical formula called spatial structure (2).

Obviously geostatistical methods use various variables and then produce different results. In many causes a method which is selected to reach the prediction process but it is vital to find out the most appropriate interpolation technique for precipitation prediction. The first application of Geostatistics was happened in mine exploitation and extended to other branches of earth science. Amini (2002) applied kriging and co-kriging techniques for predicting Cl- concentration of soil in Roudash, Isfahan which showed that kinging method provides more accurate and low cost results (1).

Azimzadeh et al. (2005) used kriging technique to estimate the area of desert pavement and threshold velocity of wind erosion in Mehriz, Yazd. The results indicated that common kinging is a precise and relevant method to produce the map of the effective factors of wind erosion including Reg area and iso-velocity threshold lines (3). Mahdavi et al. (2004) studied the capability of geostatistical techniques for estimating spatial distribution of annual rainfall in arid and semi-arid regions of south eastern Iran which showed TPSS method using elevation as a variable is the most relevant method of rainfall estimation while kinging and WMA methods having second power placed in next orders (4). Campling et al. (2001) estimated regional average precipitation in southern Spain using Theissen and geostatistical methods and concluded that the results obtained by kriging has the highest correlation with the data (5). Haberlandt and Kite (1998) applied geostatistical techniques for hydrological estimation in Canada. They concluded that application of geostatistics along with compositional data produces more accurate results (9).

Prodhorome and Reed (1999) used geostatistical techniques for estimating extreme rainfall in mountainous regions of Scotland that showed kriging and modified residual kriging due to simplicity are more applicable methods. But modified residual method has more precise results than other approaches (12).

Campling et al (2007) were used kriging method to estimate spatial distribution of rainfall as well as to optimize the number and location of rain gages in Spain. Drogue et al. (2002) were used geostatistical techniques for estimating and quantifying monthly and annual rainfall in France that showed efficiency of the method.

Rousso et al. (2005) evaluated rainfall in urban areas which showed that kriging method is a good technique for this purpose. Diodata (2005) evaluated the influence of topography on spatial variability of precipitation over small watersheds of complex terrain in mountainous regions of Italy. The results showed that common kriging method has the highest error while multivariate regression produced the lowest error.

Diodato and Ceuarelli (2005) were studied spatial variation of monthly and annual precipitation in southern Italy which showed that weighted moving average has the highest error while linear regression and common co-kriging methods produce the lowest errors. Johanson and Chen (2003) studied the influence of wind and topography on precipitation distribution is Sweden that showed high efficiency of these methods for predicting spatial distribution of precipitation.

In the current study, Inverse Distance Weighted (IDW), Kriging, and Spline were applied for estimating annual precipitation in south Alborz mountain of Tehran province in Iran.

2. MATERIALS AND METHODS:

2.1. Study area:

The study region includes Latian basin in south Alborz Mountains. The geographical coordinates of the region are 51° 22′-51° 51′ E longitude and 35° 45′-36° 50′ N altitude. The total area is also 425.17 km2. Temperature variation in the region is high due to elevation. The climate of study area is mostly semi-arid and the annual precipitation ranges from 573 mm. Figure 1 shows the position of gauging stations in the study area.

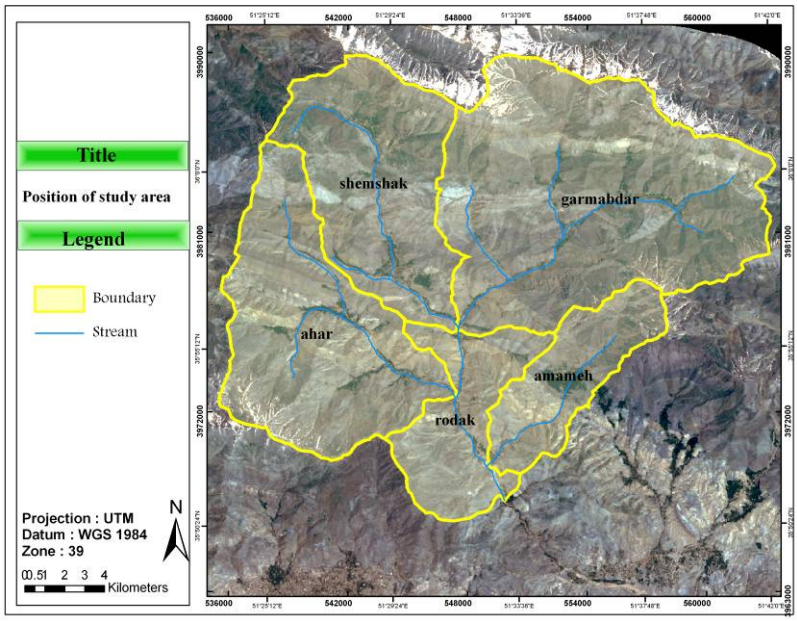


Figure 1.The position of the study area

3. METHODS:

ArcGIS Statistical Analyst has the capability to apply many type of spatial interpolation to input point data. For this study, the initial interpolation methods used were: Inverse Distance Weighted (IDW), Kriging, and Spline. The ordinary kriging includes spherical, circular, exponential, Gaussian and linear methods of interpolation was used (McBratney and Webster, 1986).

3.1. Theory of common kriging

Kriging is a prediction that considers values of a variable in ensample points as a linear composition of the values of surrounding points. Considering the values of variable Z in n measured points as follow:

Z= (z(x1), Z (x2),….Z (xn)) (1)

Estimation of Z in point X0 using kriging estimation is defined as:

$$Z^*(x_o) = \sum_{i=1}^{n} \lambda_i z(x_i) \quad (2)$$

The most important part of kriging is statistical weighs assigned to λ_i .

To avoid bias of estimation, the weighted should be determined in a way that summation is equal to one $(\sum_{i=1}^{n} \lambda_i = 1)$ and the variance of estimates should be minimized as:

var [z*(x0)] =E [(z*(x0) -z(x0))2] =Min (3)

3.2. Splin theory

The Spline function uses the following formula for the surface interpolation:

$$S(x,y) = T(x,y) + \sum_{j=1}^{N} \lambda j R(rj) \quad (4)$$

where : j = 1, 2, ..., N N is the number of points, λj are coefficients found by the solution of a system of linear equations, rj is the distance from the point (x,y) to the jth point,

While T(x,y) and R(r) are defined differently, depending upon the selected option. For computational purposes, the entire space of the output raster is divided into blocks or regions equal in size. The number of regions in x and in y directions is the same and they are rectangular in shape. The number of regions is determined by dividing the total amount of points in input point dataset by the value specified for the number of points. For data less uniformly distributed, the regions may

contain a significantly different number of points, with the value for the number of points being only the rough average.

3.3. Theory of IDW

Inverse distance weighted interpolation determines cell values using a linearly weighted combination of a set of sample points. The weight is a function of inverse distance. The surface being interpolated should be that of a location ally dependent variable.

IDW lets the user control the significance of known points upon the interpolated values, based upon their distance from the output point. By defining the higher {power} option even more emphasis can be put on to the nearest points. Thus, nearby data will have the most influence, and the surface will have more detail (be less smooth). Specifying the lower value for power will provide a bit more influence to those of the surrounding points which are a little farther. The power is a positive, real number. The characteristics of the interpolated surface can also be controlled by limiting the input points for calculating each interpolated point. The input can be limited by the number of sample points to be used, or by a radius within which there are all points to be used in the calculation of the interpolated points. Considering the values of variable WA measured points as follow:

$$W_A = \frac{\frac{1}{d_A^2}}{\frac{1}{d_A^2}+\frac{1}{d_B^2}+\frac{1}{d_C^2}+.....} \tag{5}$$

3.4. Variogram analysis

Variogram method is a suitable technique for estimating spatial variability of a variable. Calculation of variogram graph is one of essential stages in geostatistics which is defined as follow:

$$\gamma(h)=\frac{1}{2n(h)}\sum_{i=1}^{n(h)}\left[Z(xi)-Z(xi+h)\right]^2 \tag{6}$$

Where:

γ(h): value of variogram for pair points with distance h

n(h): Number of pair points with distance h

Z(xi): observed value of variable X

Z(xi+h): Observed value of the variable with distance h from x

For variogram plotting it is necessary to compute y (h) for different values of h and then to plot the values for different distances of h. In another word variogram is the variance of different points with distance h. The obtained variograph of measured samples is called experimental variogram which is a vector value that is a dependent of distance and direction.

The properties variogram include threshold (sill=C0). The threshold is the maximum value of variogram which is spatial variance of the variable. The lowest value of variogram includes partial effect which shows variance of errors of measurements. The effective distance demonstrates the distance. That variogram has the highest value. Instead of variance, covariance shows the similarity of variables. Since arithmetic summation of similarity and dissimilarity is constant then it is possible to replace it with the average difference of points of distance h. The relation between variogram and co-variogram is defined as:

$$y(h)+c(h)= s^2 \tag{7}$$

Where:

σ^2=threshold

y (h): variorum value

C (h): co-variance value

4. RESULTS:

The IDW and splin methods are part of a method of interpolation called 'deterministic' interpolation as they allocate interpolated values to locations as a result of surrounding measured values and mathematical formulas to create a surface from point data. Kriging, however, falls into a category of geostatistical methods; which adds the ability to determine some evaluation of accuracy of the resulting predicted surface. The kriging method assumes the variation in the z value is statistically homogenous throughout all location in the surface. The result of interpolation of rain has showed Fig 2, 3 and 4.

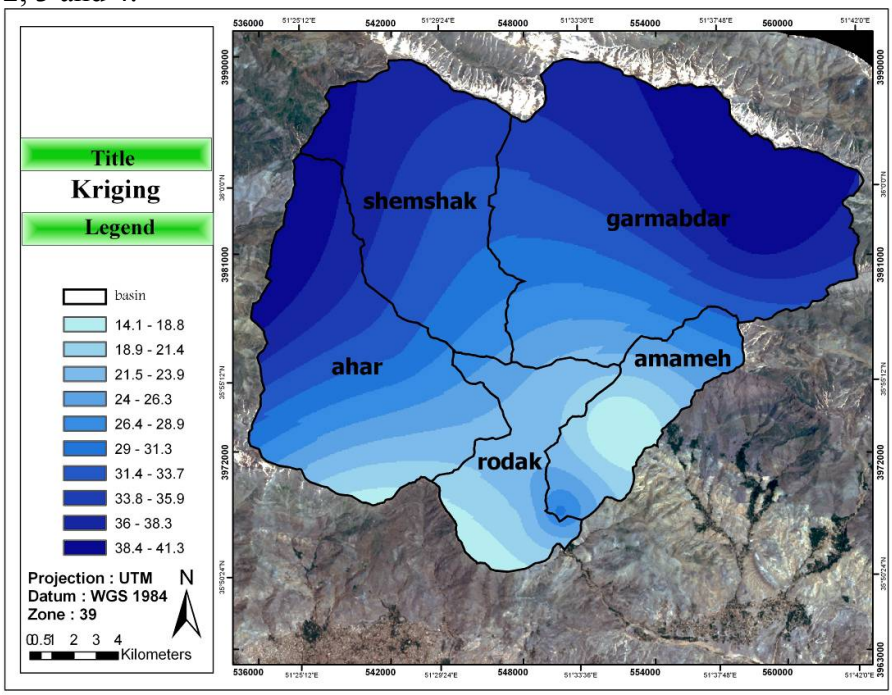


Figure 2. Kriging method in study area

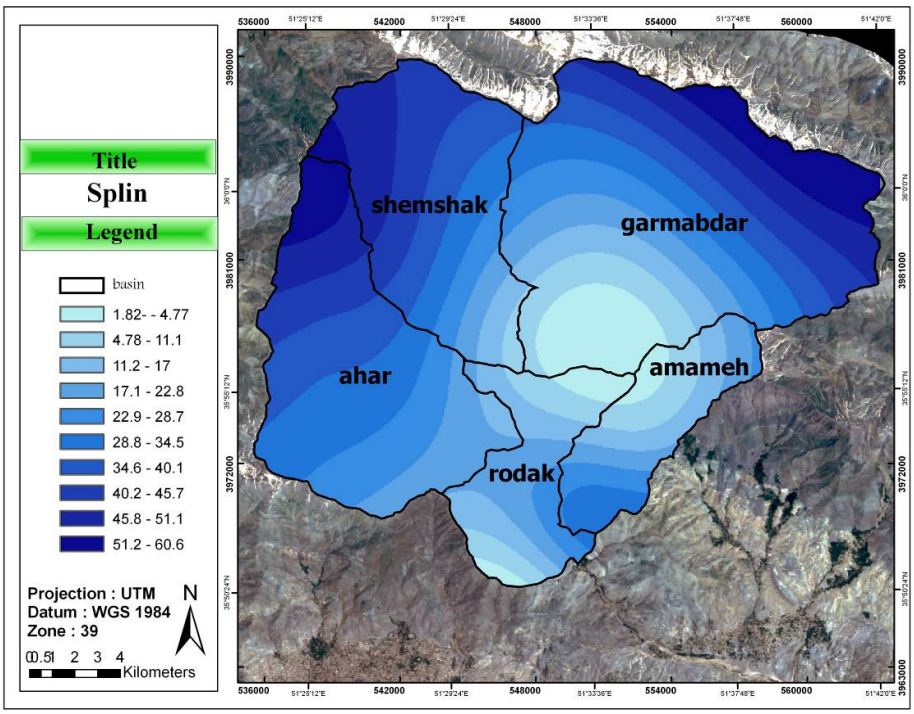


Figure 3. Splin method in study area

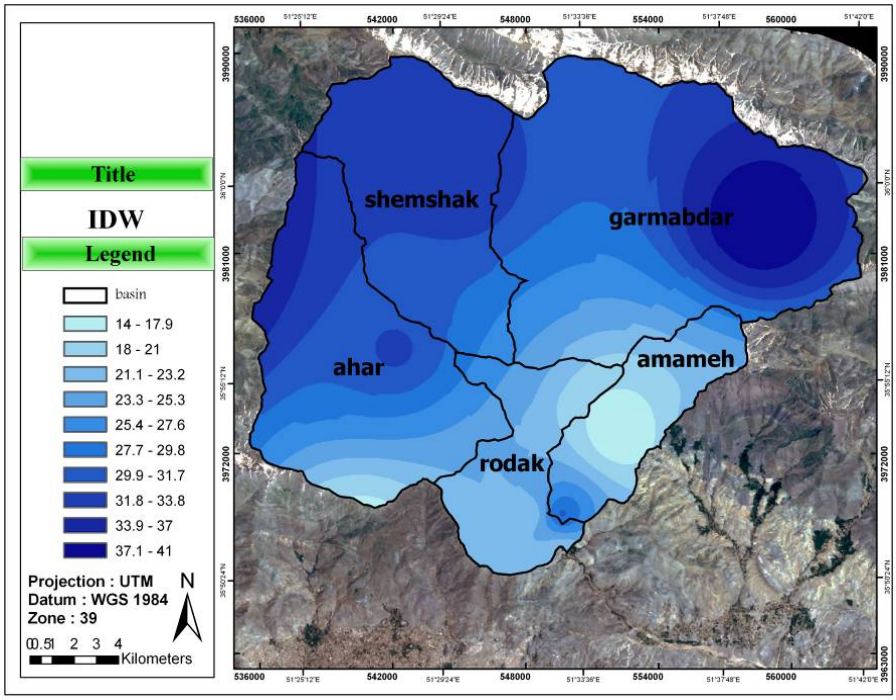


Figure 4. IDW method in study area

5. DISCUSSION AND CONCLUSION

As seen in the variogram results the most appropriate models fitted to precipitation, and elevation and elevation-precipitation variables are spherical and Gaussian models respectively. However the results of current study show weak spatial structure of the variable data but the most appropriate results based on the statistical comparisons showed high capability of kriging technique. The results also confirm the research conducted by Mahdavi et al. (2004) in arid and semi arid regions of southeastern Iran, Diodata in southern Italy (2005) and Russo (2005) in central Italy.

Since there was a weak correlation between elevation and precipitation in the study area then application of elevation data did not increased the precision of estimates while in case of high correlation between elevation and precipitation, considering elevation data would increase the precision of estimated results.

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SPATIAL AND TEMPORAL ANALYSIS OF PAN EVAPORATION IN GREECE

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Abstract: During the ongoing efforts for validating or invalidating the accelerating hydrological cycle, pan evaporation has come to the light, as an index of potential evaporation, often used together with precipitation measurements to obtain the surface moisture balance, and estimate its variability. Several studies have reported declines in pan evaporation rate throughout the Northern and Southern hemisphere for various periods since the 1950s. At individual sites, both increases and decreases in pan evaporation have also been reported. In this study, temporal analysis of daily pan evaporation measurements (Epan), Penman (ETp) and Penman-Monteith (ETp-m) evapotranspiration estimates, is presented for 16 stations in Greece, during the period 1979-1999. The high correlation between Epan and ETp-m on a yearly basis and the similarity in their annual time evolutions (in almost all stations of the study) indicate that pan evaporation can be used as an index of potential evapotranspiration in Greece, as has been reported in the literature for other regions. Furthermore, trends in Epan are examined, in order to investigate if they can represent the ETp-m trends as well. The decreasing Epan trend, reported widely in the literature, is confirmed in Greece for 10 stations, during the warm season only. In contrast, almost all stations indicate negative rainfall trends on annual, warm and cold season basis. When all stations are considered 'as a whole', annual, and cold season pan evaporation estimates show insignificant increases while rainfall trends are negative.

Key words: Pan evaporation, Penman potential evaporation, Penman-Monteith evapotranspiration, trend.

1. INTRODUCTION

Changes in the climate are not only limited to changes in the energy balance, but they also extend to the water cycle. The moisture balance at the terrestrial surface can be described as a balance between the atmospheric supply (precipitation) and atmospheric demand (potential evaporation). Changes in terrestrial precipitation around the world are well documented, and the general trend has been reported rather increasing, over both the last 50 and 100 years. Unfortunately, evaporation is rather difficult to be measured and indirect approaches are used for estimating it, from variables directly related to evaporation. Evaporation is usually obtained from firstly estimated potential evaporation. Recently, pan evaporation has been taken into account as an index of physically based measure of potential evaporation. Potential evaporation is also calculated from readily available climate variables using models such as the Penman-Monteith reference evapotranspiration model (Allen et al., 1998).

During the ongoing efforts for validating or invalidating the accelerating hydrological cycle, it is important to understand past and current evaporation trends and several studies have examined the trends in potential evaporation, in places all over the world. Declines in physically measured pan evaporation have been reported for various periods since the 1950s (Peterson et al., 1995; Chattopadhyay and Hulme, 1997; Brutsaert and Parlange, 1998; Quintana –Gomez, 1998; Lawrimore and Peterson, 2000; Golubev et al., 2001; Asanuma, 2004; Hobbins et al., 2004; Liu et al., 2004; Liu and Zeng, 2004; Groisman et al., 2004; Roderick and Farquhar, 2004, 2005; Chen et al., 2005; Tebakari et al., 2005; Shenbin et al., 2006; Zhang et al., 2007; Papaioannou et al., 2007; Roderick et al., 2007; Burn and Hesch, 2007; Hobbins et al., 2008; Jovanovic et al., 2008). Positive

Epan trends have been also referred in different areas (Cohen et al., 2002; Xu et al., 2005; Kirono et al., 2008) and at individual sites, both increases and decreases in pan evaporation have commonly been observed. Penman-Monteith reference evapotranspiration (ETp-m) has also been reported as decreasing (Thomas, 2000; Cohen et al., 2002; Chen et al., 2005; Wu et al., 2006; Zhang et al., 2007) or increasing (Yang et al., 2005; Oguntunde et al., 2006; Kirono et al., 2008) in some regions.

Some of the studies in the literature have attempted to identify the meteorological factors causing the observed trends in evaporation. A lot of researchers have shown that the trend is not determined by temperature alone. Decreasing Epan was attributed to decreased diurnal temperature range (Peterson et al., 1995) or decreasing vapour pressure deficit or decreasing net radiation (Asanuma and Kaminmera, 2004) or wind speed (Zhang et al., 2007). Reduced Epan has been reported as consistent with the observed large and widespread decreases in solar radiation in the Nothern Hemisphere, resulting from increasing cloud coverage and aerosol concentration, by Roderick and Farquhar (2002) and as attributed to the decrease in solar irradiance in China, due to increased concentrations of aerosols, by Liu et al., (2004). Chen et al., (2005), Gao et al., (2006) and Xu et al., (2006) further have demonstrated that solar radiation and wind speed were the major contributors to the changes in Epan, as well as ETp-m. Roderick et al., (2007) have resolved the declining Epan in Australia, as caused by reductions in wind speed and some regional changes in solar radiation. In the contrary, Xu et al. (2005) had reported increasing Epan in Tibetan Plateau, as resulting from temperature increases coupled with a lack of surface water being in line with observed water vapour pressure increases. Thomas et al., (2000) indicated that wind, relative humidity and maximum temperature are associated with changes in ETp-m. Cohen et al., (2002) found significant increase in Epan for Israel, accompanied from decreases in global irradiance and increases in water vapour deficit and wind speed.

Differences or similarities between the different estimates of evaporation trends derived from instrumental records or from Penman-Monteith model have been reported in recent studies (Cohen et al., 2002; Chen et al., 2005; Zhang et al., 2007; Kirono et al., 2008), as a key research. The present study focuses on the relationship between pan-evaporation measurements and reference evapotranspiration estimates by Penman-Monteith (or Penman) method, for Greece. Thus, temporal analysis is presented for daily pan evaporation measurements and daily Penman-Monteith reference evapotranspiration estimates, as well as, for other meteorological attributes, as daily precipitation, temperature, net radiation, solar radiation, vapour pressure deficit, and wind velocity. Their trends (during the period 1979-1999) are evaluated for 16 sites across Greece. The existence or no of the widely reported decline of pan evaporation measurements is investigated either at individual sites or for all sites considered 'as a whole' in Greece, on different timescales (yearly, warm and cold season basis). The analysis is furthermore extended to Penman-Monteith estimates and the above mentioned, as affecting it, meteorological attributes.

2. MATERIALS AND METHODS

Daily measurements of temperature, humidity, wind velocity and sunshine hours, provided by the National Meteorological Service of Greece, are used for estimating potential evaporation by Penman equation (ETp) and reference evapotranspiration by Penman-Monteith method (ETp-m) for 16 stations in Greece. Table 1 shows the characteristics of the stations.

ETp is given by the Penman equation (Penman 1948) as:

$$ETp = [\Delta/ (\Delta+\gamma) (R_n – G)/L] + [\gamma/ (\Delta+\gamma) E_A]$$

where, Δ is the slope of saturated vapor pressure in relation to air temperature (kPa $^oC^{-1}$), γ is the psychrometric constant (kPa $^oC^{-1}$), R_n is the net radiation, G is the soil heat flux density, L is latent heat of evaporation and E_A is the drying power of the air given as: $E_A = f(u) (e_s - e_a)$, where, f (u) is a function of the wind speed u (m/s) and (e_s-e_a) is the vapor pressure deficit (kPa).

ETp-m, given by Penman-Monteith equation (Monteith, 1981), is used to estimate potential evaporation from hypothetical reference grass with an assumed height of 0.12 m, a fixed surface resistance of 70 s m^{-1}, and an albedo of 0.23 (Allen et al. 1998) and is expressed as:

$$ET\ p\text{-}m = [0.408\ \Delta\ (R_n – G) + \gamma\ (900/T +273)\ u_2\ (e_s-e_a)] / [\Delta + \gamma\ (1+0.34u_2)]$$

where T is the mean daily air temperature at 2m above ground level (^oC) and u_2 is the wind speed (ms^{-1}) at 2m above ground level.

As it is known, pan evaporation is often taken as an indicator of potential evaporation (Brutsaert and Parlange, 1998). Thus, in this study, daily experimental values of pan evaporation (from class-A pans) available for the above mentioned 16 stations are also considered for the period 1979-1999. These stations were chosen on the basis of the records being available for as long as possible. Figure 1 shows the correlation between annual values of Penman (ETp) or Penman-Monteith (ETp-m) evapotranspiration and pan evaporation (Epan). It is evident that Epan was highly correlated on a yearly scale with ETp and ETp-m. Epan was better correlated with ETp-m (coefficient of determination r^2=0.83) rather than with ETp (r^2=0.79). These results suggest that Epan seems to be a good indicator of ETp-m for the Greek area. The slope of the regression line between ETp-m and Epan (Figure 1) is 0.734. It must be noticed that the slope between ETp and Epan was 0.86 and that a very high correlation existed between ETp-m and ETp (r^2=0.99).

Furthermore, the daily experimental pan measurements were reduced by the estimated average pan coefficient, although it is known that pan coefficients vary seasonally (Allen et al., 1998). Their average yearly values (Epan) are shown in Table 1. Average yearly values of precipitation (P), and potential evaporation ETp-m (or ETp), are also presented in Table 1. It is obvious that the average yearly estimations of Epan are in a good agreement with the ones obtained from Penman-Monteith method in most of the stations. Epan values range between 920.53 mm/year (Trikala) and 1764.19 mm/year (Ierapetra), with corresponding ETp-m values 954.04 and 1672.31, respectively.

The annual variations of the average mean monthly pan evaporation (Epan) and Penman-Monteith evapotranspiration (ETp-m) are presented in Figure 2, for all stations. It is observed that averaged monthly values of Epan and ETp-m have uniform yearly route in almost all stations. Part of any observed difference must mainly be attributed upon the unique pan coefficient used. Hence, it seems that pan evaporation (Epan) can be an indicator for reference evapotranspiration (ETp-m).

In order to investigate and explain the temporal variation of pan evaporation in Greece the analysis is taken over warm and cold season besides the yearly basis. Warm season is defined from May through October. The slopes of the trends of the annual, warm and cold season averages, of either pan evaporation (bEpan) or other meteorological attributes, as precipitation (bP), net radiation (bRn), solar radiation (bRs), vapour pressure deficit (bVPD), temperature (bT) and wind velocity (bU) are calculated, for the period 1979-1999. The slopes of the trends of reference evapotranspiration (bETp-m) are also examined. Table 2 shows the number of the stations with negative and positive trends of the considered parameters and the ones with the significant trends at the 95% level of significance (shown in the parenthesis). Thus, it is apparent that eight (8) sites appear to exhibit negative Epan trend and 6 of them have this negative trend statistically significant at the 95% level of significance, on a yearly basis. In warm season there are 10 stations with negative Epan trend (8 statistically significant) and in the cold season 9 stations with positive Epan trend (4 statistically significant). In the contrary, 10 sites show positive ETp-m trends on the annual and the warm season basis, while ETp-m decreases in 9 stations, during the cold season. The sign of the Epan and ETp-m trends are in agreement in 12 or 11 stations on a yearly or warm season basis but in disagreement in 50% of the sites during the cold period. When similar, the pan trends are greater in magnitude in all cases. Precipitation is mainly decreasing either in all or 13 or 14 sites on the annually or warm or cold season basis, respectively, with 14 or 8 or 12 trends statistically significant. Trends in incident solar radiation (Rs) are examined, since Rs provides the major energy input to any evaporation process. A positive trend of Rs is observed in almost all the stations except of Serres (for all three considered timescales) and Kalamata (for yearly and warm season). None of them is statistically significant. Trends in net radiation (Rn) are also calculated in order to evaluate changes in long and short wave radiation on the surface. Statistically insignificant positive trends are observed for Rn, in all stations and all timescales of the analysis. Vapour pressure deficit (VPD) trend is positive in 12 sites (except of Serres, Mytilini, Lamia and Andravida) for yearly and warm season timescales. During the cold season the positive trends are reduced to 9 (negative trends are increased due to the added negative trends of Agrinio, Iraklio and Tymbaki). All the referred trends

for VPD are statistically insignificant. Temperature trends for this period are positive for all stations on a yearly and warm season basis. Half of the stations exhibit increases (Serres, Tricala, Mikra, Larisa, Aghialos, Agrinio, Andravida and Kalamata) and the other half decreases, during the cold period. All of them are statistically insignificant. Wind speed (U) is mainly decreasing in 11, 10 or 13 sites for yearly, warm or cold season basis, respectively, being though statistically insignificant in all stations or timescales (yearly, warm, cold season).

When considering, all available stations of the country 'as a whole', annual values of (Epan), (ETp-m) and (P), averaged over 16 sites, are obtained for each year of the period 1977-1999. Figure 3, shows the annual, warm and cold season pan evaporation rate (Epan), potential evaporation (ETp-m) and precipitation rate (P), averaged over the 16 sites, as a function of time. The overall slopes of the corresponding trends, on the yearly basis, are positive for Epan (0.445 mm/y^2) and ETp-m (0.078 mm/y^2) and statistically significant negative for precipitation (-3.406 mm/y^2). Statistically insignificant negative slope of Epan trend 'as a whole' (-0.838 mm/y^2) is observed during the warm season, when ETp-m and precipitation exhibit positive ones (0.476 mm/y^2 or 0.435 mm/y^2, respectively). In the contrary, during cold season, the slopes of the trends are presented as negative for ETp-m (-0.480 mm/y^2) and precipitation (-5.407 mm/y^2) and positive for Epan (1.049 mm/y^2).

3. RESULTS AND METHODS

A more complete report of trends in physically measured pan evaporation in different places of the world and an investigation of the underlying physical reasons for these trends is essential, in order to describe how the trends of potential evaporation relates to the enhanced greenhouse effect. Usually, Epan trends are examined mainly on an annual basis and the estimation of a general trend based on the all-stations average time series is rather pronounced. In this study, similarities between magnitude and trends in measured pan evaporation (Epan) and estimated Penman-Monteith reference evapotranspiration (Ep) are examined for Greece (which is characterised by a variety of climatic regimes) for the period 1979-1999. Besides the annual, analysis on a warm and cold season basis (based on monthly data) is also conducted. Considering that an all-stations average time series from a network of 16 stations may smooth out the original spatial variability, within the country (hence, it may not really represent the whole Greek area), a general picture of the trend is obtained by examining the distribution of the trends across the 16 sites.

The slope of the regression line between ETp-m and Epan is found equal to 0.734 being approximate to the widely adopted value of 0.7 for the pan coefficient. Within the same period, the agreement between estimated potential evapotranspiration calculated by Penman's (ETp) or Penman Monteith's (ETp-m) equation and pan evaporation measurements (Epan) is very good, with Epan being better correlated with ETp-m (r^2=0.83) rather than with ETp (r^2=0.79) on a yearly basis. Epan and ETp-m exhibit similar interannual variability, even though their magnitude may differ. These results suggest that Epan seems to be a good indicator of ETp-m for the Greek area. Part of the observed differences may be attributed upon the unique pan coefficient used.

When considering, all available stations of the country 'as a whole', the overall slopes of the corresponding trends, on the yearly basis, are positive for Epan and ETp-m but significant negative for precipitation. Similar results have been reported by Yang et al., (2005), Oguntunde et al., (2006) and Kirono et al., (2008). Negative slope of the trend of the Epan is observed, during the warm season only (statistically insignificant), whereas ETp-m and precipitation exhibit positive ones. In the contrary, during cold season, the slopes of the trends are negative for ETp-m and precipitation and positive for Epan. The decreasing Epan trend reported widely in the literature and observed in southern Greece (when stations in Crete are considered 'as a whole') (Papaioannou et al., 2007) is confirmed through all the country for only 10 stations and only during the warm season. In contrast, almost all the trends in rainfall were negative for annual, warm and cold season. The trends in annual pan evaporation (or ETp-m) and annual rainfall have opposite trends in 8 (or 10) sites.

The results show that the trend in pan evaporation varied from site to site, with a trend for a decrease in pan evaporation, to be strongest in yearly and in warm season. In 75% (or 69%) of the stations the Epan measurements show the same trend direction as those of the ETp-m estimates, on a yearly (or warm season) basis, respectively, but are greater in magnitude in most cases. The sign of the Epan and ETp-m trends are in disagreement in 50% of the sites during the cold period. As expected, trends in monthly Epan and ETp-m varied from site to site and from warm to cold season. It is worthy noting that most of these trends were not significant at the 95% level of significance. On an annual basis, the trends also varied spatially. This study shows that the temporal and spatial variability of Epan tends to be higher than that of ETp-m. Similar results for the comparisons of temporal variations between Epan, and ETp-m or point potential estimates derived from the Morton method, are referred by Chen et al., (2005) and Kirono et al., (2008), respectively.

The widespread trend in declining pan evaporation has prompted research into the reasons for the decline. In terms of the surface energy balance, there are three possibilities to explain changes in pan evaporation: changes in one or more of net radiation, vapour pressure deficit of the air and wind speed. The importance of each of the three causes varies from place to place. In this study, temperature trends for the examined period are positive (statistically insignificant) for all stations on a yearly and warm season basis, whereas, half of the stations exhibit increases and the other half decreases, during the cold period. This is in agreement with the observed small and positive (statistically insignificant) trend of vapour pressure deficit (VPD). Air temperature does not influence pan evaporation over time scales of greater than about a week or so (Roderick and Farquhar, 2004), but can influence evaporation indirectly i.e. increasing temperature causes a simultaneous increase in both VPD and outward long-wave radiation. The increase in long-wave radiation decreases Rn and the increase in VPD increases advective water transfer, and this may be seen in this study, with Epan increasing. Trends in net radiation (Rn) are calculated in order to evaluate changes simultaneously in long and short wave radiation on the surface. Statistically insignificant positive trends are observed for Rn, in all stations and all timescales of the analysis in agreement with the general increasing Epan or ETp-m. Trends in incident solar radiation (Rs) are also examined, since Rs provides the major energy input to any evaporation process, and is an excellent estimator of Rn (Hobbins et al., 2004). A positive trend is observed in almost all stations for the three considered timescales but none of them is statistically significant. Recently, the trends in surface solar radiation are reported as showing a widespread reversal from dimming to brightening in the mid-1980s (Wild et al., 2005) but this can not be clearly distinguished in this study, due to the short time lines of the data and the fact that the period 1979-1999 includes both conditions. However, the data 'after 1986' attribute more in the analysis and as a result statistically insignificant increases of Rs are apparent. Positive trends of Rs are better reflected in ETp-m than the physically based measures of Epan. Wind speed (U) is found mainly decreasing. Evaporation increases in this study, may be attributed to the observed weakly increases in Rn and the possibility of an increase caused by the observed small increase in VPD being higher than the decrease caused by the observed reductions in wind speed. It must be mentioned that reductions in wind speed have been combined mainly with declining Epan or Penman-Monteith evapotranspiration (Chen et al., 2005; Gao et al., 2006; Xu et al., 2006; Zhang et al. 2007; Roderick et al., 2007) whereas increase in wind speed have been referred, with increase in Epan (Cohen et al., 2002).

Table 1. Characteristics of the stations and average annual precipitation (P), pan evaporation (Epan), Penman-Monteith reference evapotranspiration (ET p-m) and Penman potential evaporation (ET p).

Station	Lat.	Long.	Alt. (m)	P (mm/y)	Epan (mm/y)	ETp-m (mm/y)	ETp (mm/y)
Serres	41°05'	23°34'	34	444.93	969.53	914.88	1103.70
Trikala Im.	40°36'	22°33'	0.8	514.88	920.88	954.04	1134.98
Mikra	40°31'	22°58'	4	443.51	1255.87	1160.09	1366.75
Alex/poli	40°51'	25°26'	2.5	498.11	1302.10	1055.59	1243.12
Larisa	39°39'	22°26'	71	440.34	1222.60	1062.19	1252.24
Aghialos	39°13'	22°48'	12.2	479.15	1438.66	1134.13	1321.36

Mitilini	39° 04'	26°36'	4.2	606.88	1473.60	1276.50	1527.94
Agrinio	38° 37'	21°33'	24	853.07	1264.33	1038.40	1242.22
Aliartos	38°23'	23°06'	110	591.40	1122.44	1131.48	1345.20
Lamia	38°51'	22°24'	14.3	565.56	1442.56	1185.20	1376.22
Adravida	37°55'	21°17'	11.1	759.17	1241.27	1130.07	1331.28
Pirgos	37 40'	21°26'	12	900.11	1226.08	1133.42	1332.69
Kalamata	37°04'	22°06'	222	770.09	1356.11	1166.39	1360.31
Iraklio	35°20'	25°11'	37	470.52	1631.03	1442.08	1706.83
Ierapetra	35°00'	25°44'	10	426.36	1764.90	1672.31	2006.06
Tymbaki	35°00'	24°46'	5	452.89	1740.19	1437.99	1687.13

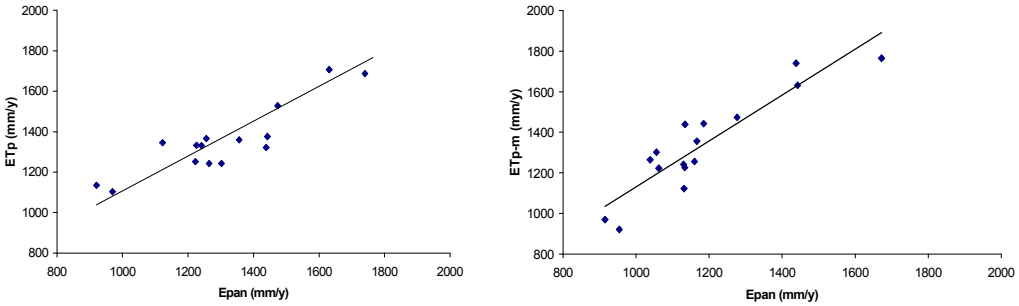


Figure 1. Relationships between Penman potential evaporation (ET p) or Penman-Monteith reference evapotranspiration (ET p-m), and measured pan evaporation (Epan).

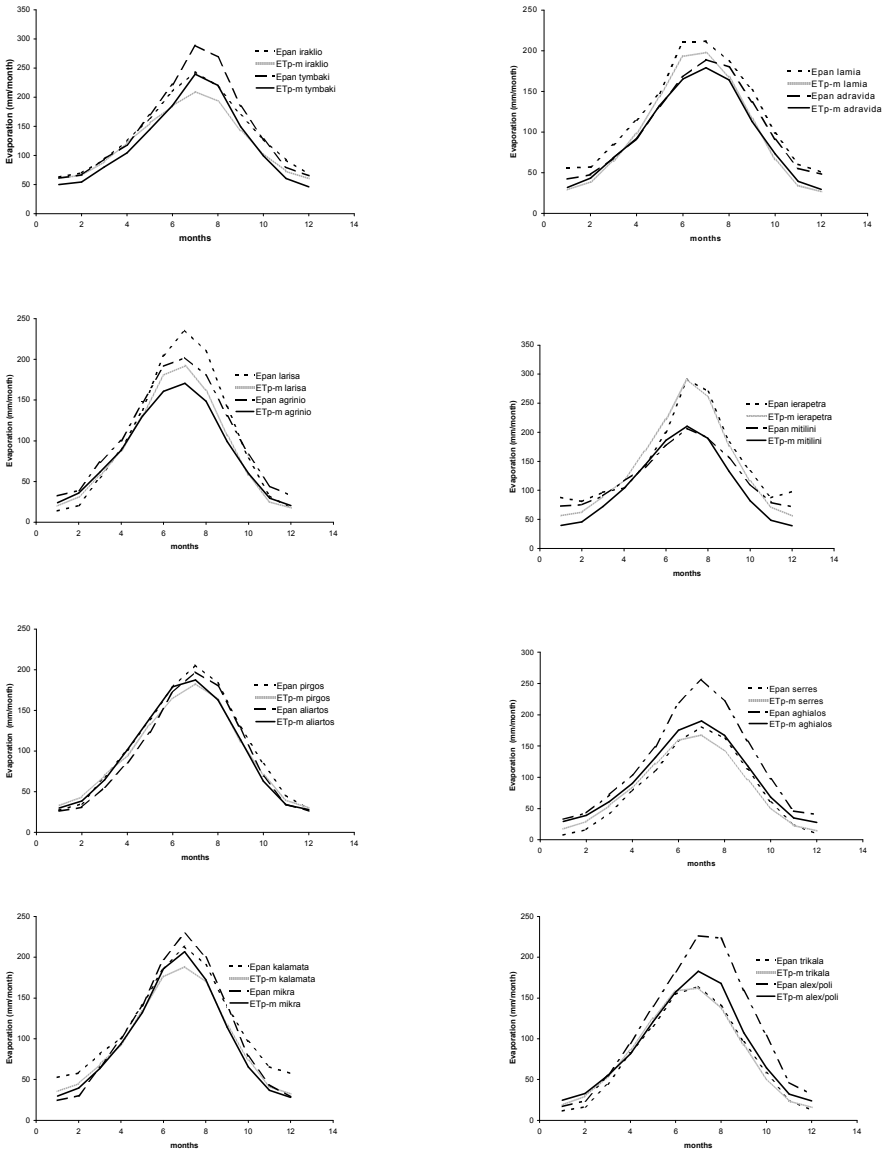


Figure 2. Annual variation of mean monthly pan evaporation (Epan) and Penman-Monteith reference evapotranspiration (ETp-m), for all stations.

Table 2. Number of stations with positive or negative trends of Precipitation (p), Pan evaporation (Epan), Penman-Monteith evapotranspiration (ETp-m), Wind velocity (U), Temperature (T), Vapour pressure deficit (VPD), Net (Rn) and Solar (Rs) radiation.

Parameters	Yearly		Warm season		Cold season	
	Decrease	Increase	Decrease	Increase	Decrease	Increase
P	14(14)	2(0)	13(8)	3(0)	14(12)	2(0)
Epan	8(6)	8(6)	10(8)	6(3)	7(5)	9(4)
ETp-m	6(3)	10(5)	6(2)	10(5)	9(3)	7(1)
U	11(0)	5(0)	10(0)	6(0)	13(0)	3(0)
T	0	16(0)	0	16(2)	8(0)	8(0)
VPD	4(0)	12(0)	4(0)	12(0)	7(0)	9(0)
Rn	0	16(0)	1(0)	15(0)	0	16(0)
Rs	2(0)	14(0)	2(0)	14(0)	1(0)	15(0)

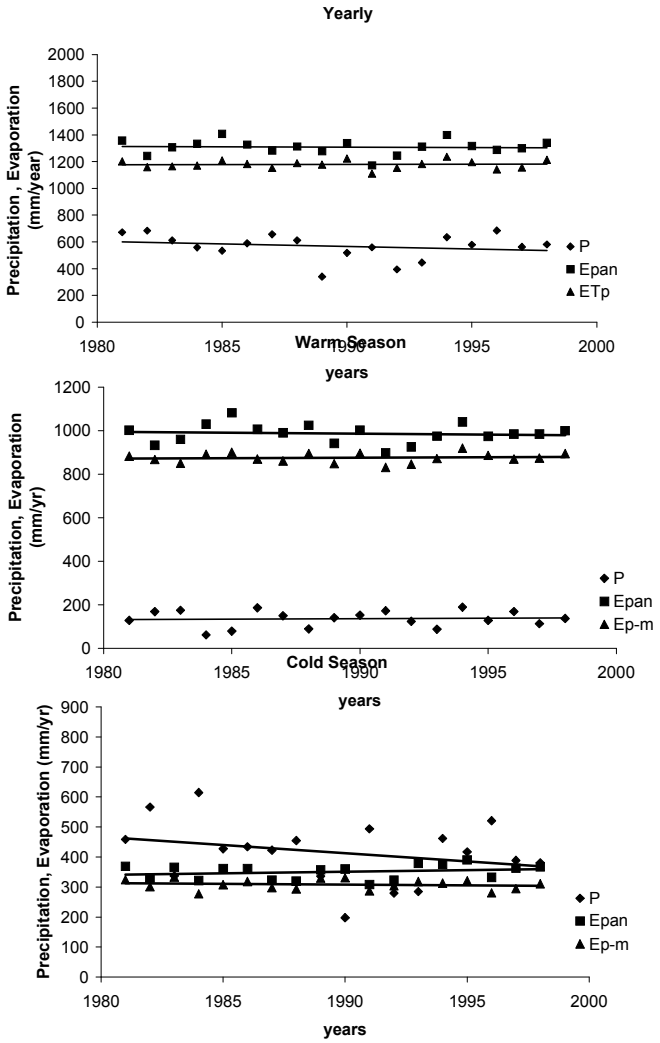


Figure 3. Overall trends in annual, warm and cold season precipitation (P) pan evaporation (Epan) and Penman-Monteith evapotranspiration (ETp-m) averaged over 16 sites.

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PREDICTION OF CROP PRODUCTION USING THE STANDARDIZED PRECIPITATION INDEX (SPI) AND CLIMATIC FACTORS TO MANAGE DROUGHT RISK: CASE STUDY HEMADAN, IRAN

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Abstract: Drought causes a great damage to dry and water farming. Therefore, finding a scientific and suitable way to avoid the harmful effects of drought is essential. Thus, proceeding to agricultural drought risk management can be very useful. For this, we considered two steps, the first step is to evaluate drought, based on indices that can determine its intensity and continuity rate. In which the Standardized Precipitation Index (SPI) is the most important one. The second step is to predict the reduction of crop production when drought happens. Having a clear prediction, we can plan for a more considerate farming. To do this, we need to scrutinize the relation between the SPI index and crop production. The goal of this paper is to evaluate and recognize drought's that have occurred in the region and examine significant correlation between the SPI index at one, three and six month time-scales and dry farming wheat production. Precipitation data from twenty weather stations were used for the calculation of SPI. Monthly evapotranspiration for all station were calculated with both Penman and Thornthwaite method. Wheat production data were obtained from the agriculture ministry and at the end, all the collected information was analyzed with the SPSS software. The results show significant correlation among crop production, SPI index, evapotranspiration and temperature. As, from the several SPI time-scales, the one and three month SPI had a stronger correlation with crop production. The predictive model for the region allows crop prediction with the SPI index within drought assess time.

Key words: Drought, Crop prediction, Standardized Precipitation Index (SPI), evapotranspiration, temperature.

1. INTRODUCTION

Generally, the cause of moistness decreasing is the rainfall lacking in a specific period that is happened constantly and almost it outbreaks in any climate which means drought. The out breaking of this phenomenon is inevitable [11].

The possibility of happening of drought is in three parts, meteorology's aridity, hydrology's drought and agriculture's drought that they have different and specific definitions [7].

What has been pointed in this article is from point of agriculture's view about the drought, which account on the soil's moistness with its need for water. So, in this view whenever drought will be happened that the soil's moistness become less than necessity of that region's plants and it leads to damage on productions [8]. The most damaged sectors in agriculture are dry and water farming in duration of the drought. The most important damage is the shortage of productions. Hence, it is too significant to know how encounter and prevent the drought. Therefore, finding the scientific and practical way to prevent the badly effects of drought could be so important [2].

So far, models for prediction of crop production have been used in large and homogeneous agriculture regions of some countries such as Africa and the USA [17]. But, there is less study and research on the benefits of using the standard precipitation index (SPI) and production's rate for creating a model to predict crop production in small and non-homogeneous areas [14]. Using the

drought's index and the amount of crop production all together for crop prediction has a good result. Although, so far both of these indexes have been used separately [10] [13].

Up to now, many researchers evaluate the monitoring of drought with standard precipitation index (SPI), Therefore, this is the best validity for that, in among of the researchers' associations throughout of the world.

Bordi accompanied his colleagues used the SPI index for monitoring the long time drought in Italy and they drew the results of that for 6, 12 and 24 months also, they have shown the behaviors of SPI since January 1950 till august 2000 about Italy for the 24 month SPI. They also brought up that the SPI has so many fluctuations in short period(3 month). However, in long terms its fluctuations are decreased [11].

In 1998, Mr. Hayes with his colleagues proved the efficiency of SPI about the drought, which happened at Colorado in the U.S in 1996. Not only this index showed the progress of the drought but also it pressed the time of beginning of the drought in 1996. Therefore, comparing with Palmer's index, SPI worked better in a short-term period [15].

In 2001, Giruxetal accompanied his co-workers studied drought affections into Raiment in the U.S since 1998 to 1999 and they brought up that SPI works better than Ballmer's index in one month period and also SPI works much more better than Ballmer at registering the beginning of drought conditions and its intensity [16].

Nassaji, Zavare and Saneii after their monitoring the drought in Esfehan and Zabol with different duration, such as 3, 6, 12, 24 and 48 months in 2001 they brought up the possibility of prediction the dryness periods have been prepared with different temporal measurement on the regions under study [6].

In 2006 Sergio M. Vicente-Serrano accompanied their co-workers began to research and finding the way for prediction of wheat harvest by standard precipitation index (SPI) on a region in Spain. After gaining good results, they expressed that it is possible to know the mass of harvest in February, which means the production, is predictable four months before harvest [14].

Unfortunately, drought has been analyzed much less, than it is eligible to consider in Iran and unknown things around the drought phenomenon are too much. To solve drought problems it is necessity to assess drought on the regions that are threaten by it [4]. Hamedan province is one of the regions that exposed to drought constantly and this province is significant from point of view of dry farming wheat production, too. The prediction of crop productions in this province is important, it helps so much the managing of drought's risk with proper, and highest policy at economy that could deduct to least damage in country. Hence, at the first, the intensive and the drought's duration have been analyzed by researching and monitoring the dryness in Hamedan province on method of counting the standard precipitation index (SPI). Then, with the combination of this index with production's rate of dry farm wheat has been used for a crop prediction model.

2. MATERIALS AND METHOD

Between 34 degree and 35 degree, 46' latitude north and between 47 degree, 48' and 49 degree, 28' longitude the Hamedan province is located with area of 19493 kilometers square. It equals 1.2 percent of entire country size. This province has semi dryness and cold climate with average 12-degree temperature and its average rainfall is 330mm. In category of Selsianof, it is located in semi dry and cold region [3].

First time McKee accompanied his colleague specified the standard precipitation index (SPI) for verifying the probability of drought's expansion in 1993. This symbol has been designed for lacking the rainfall in different periods and it represents the drought's effects in various terms over the amount of water resources. In 1995, McKee with his colleagues took this matter into consideration and they put the SPI under study in three, six, twelve, twenty four and forty eight month periods [11].

Standard precipitation index (SPI) is used to evaluate the dryness in this research, which has much more benefits than others indexes do. Other drought's indexes are complicated but SPI is easy used. This style can work at different tense and place aspects. SPI is comparable in different geography's conditions and topography's circumstances that those are affect on it, too. This index is able to turn the amounts of drought's intension, duration and expansion to quantities and evaluate them into the periods. It only needs the monthly rainfall's data. The strongest point of this way is that it is able to compare various terms such as one-month duration or length of years [12]. That is choose-able for any specific period and it often depends on the hydrology's system that is on top. Positive amounts of SPI represents the more than the normal rainfall's rate and negatives numbers express happening of less than common (normal) raining. The rate of deviation from zero represents the probability of occurrence. So, in this manner on the base of this amount of SPI can decide and makes a plan [5].

The statistics of existed weather stations in region are preparing monthly and considering different scales such as having long-term statistics, having short time and uncompleted data and proper variance on region. Twenty-nine weather stations in and outside of the precinct were chosen with jointly statistics' terms. The stations that are under study are;

AGH TAPE, BAHAR, GONBAD, JOKAR, KITO, KORIJAN, NASHR, SHORIN, DARGAZIN, HAMADAN (airport), MALAYER, NOZHE, SOLAN, GHAZVIN, KERMANSHAH, SAGHEZ, SANANDAJ, KHORRAM ABAD, ARAK and ZANJAN.

Uncompleted data from some stations were rebuilt by the SPSS software and making them to long-term statistics. In addition, it was rebuilt by rate of unity on the base of foundation's stations and verity and homogeneity of data were tested by RUN-TEST method, too. On this research, SPI has been used over one, three and four month periods [1].

The statistics' production of dry farm wheat since 1991 until 2008 belong to Nahavand, Malayer, Toyserkan, Asad Abad, Kabodar Ahang, Razan, Bahar and Hamedan was taken from ministry of agriculture.

The parameters of evapotranspiration and temperature were under considered for more correlation. The method of Penman Mantis was used for counting the rate of potential of evapotranspiration for synoptic stations. In addition, for other stations, that did not have enough data for using Penman Mantis method but had other data, including the minimum temperature, was counted by Thornthwaite's method. But on consideration of Fao's persistence over Penman Mantis's method in comparing with others ways, at this manner, potential of evapotranspiration are used and verified on this research by two ways of evaluation and the connections between both methods [9].

The correlation of crop production's rate, evapotranspiration, temperature and SPI by analyzing the various variable Regression for prediction of wheat's production was under study in various time-scales of one month, three months and six months. Those analysis have been done through the SPSS and Excel software.

3. RESULTS AND DISCUSSION

The most important features of drought from point of performance the under study plans and doable at region are shown in table 1 for duration of six months.

Table 1. Drought's features on stations under studied in six months duration

Station	**The most intensity observed**			**The longest duration (month)**			**The most intensity of duration on purpose of drought magnitude**		
	SPI Value	Month	Year	Duration (month)	DM	ADI	Duration (month)	DM	ADI
Agh Tape	-2.04	9	1977	34	-21.93	-0.645	26	-24.04	-0.92
Bahar	-1.99	11	1991	49	-18.31	-0.37	43	-44.03	-1.02
Gonbad	-2.47	10	1967	28	-34.67	-1.24	28	-34.67	-1.24

Jokar	-1.69	3	1988	87	-51.98	-0.6	87	-51.98	-0.6
Kito	-3.08	11	1970	66	-79.73	-1.21	66	-79.73	-1.21
Kkorijan	-2.55	11	1991	43	-51.07	-1.19	43	-51.07	-1.19
Nashr	-2.31	5	1999	31	-35.96	-1.16	31	-35.96	-1.16
Shorin	-2.9	5	1984	47	-42.24	-0.9	31	-59.87	-1.94
Dargazin	-2.48	11	1978	42	-33.22	-0.79	42	-33.22	-0.79
Hamadan (airport)	-2.85	6	1999	36	-31.25	-0.87	25	-35.91	-1.44
Malayer	-3.04	2	1975	32	-31.56	-0.99	32	-31.56	-0.99
Nozhe	-3.43	12	1965	62	-82.61	-1.33	62	-82.61	-1.33
Solan	-1.88	8	1999	39	-37.49	-0.96	39	-37.49	-0.96
Ghazvin	-2.35	8	1997	35	-37.1	-1.06	35	-37.1	-1.06
Kermanshah	-2.11	12	1995	53	-37.43	-0.71	53	-37.43	-0.71
Saghez	-2.66	6	1999	34	-22.72	-0.67	34	-22.72	-0.67
Sanandaj	-2.68	6	1999	37	-31.08	-0.84	37	-31.08	-0.87
Khoram	-2.58	12	1973	22	-20.6	-0.94	22	-20.6	-0.94
Arak	-3.07	10	1999	21	-14.13	-0.67	17	-14.86	-0.87
Zanjan	-2.04	9	1977	34	-21.93	-0.645	26	-24.04	-0.92

SPI: standard precipitation index.
ADI: average drought intensity.
DM: drought magnitude.

The minimum amount of SPI has been observed in six-month duration, which belongs to February in 1965 from Nozhe station with -3.43. According to SPI's category, the region is suffering from too intensive drought. Kito and Malayer stations are standing on the next ranking one after the other with less than -3.43. From the aspect of long term period of drought, Jokar station with suffering 87 month constantly dryness with -0.6 average drought intensity allocated to itself first step in among of long stretch drought duration. By that ranking Kito station has a silver medal. In among those stations that are suffering from drought duration, Shorin station with thirty one month dryness with 1.93 average drought intensity (ADI) allocated to itself the golden medal form the aspect of the most intensity and size. Comparison of the 1, 3 and 6 month SPI at figure one has been shown. Having concentration over the SPI's graph, during the short time-scale (one month), it comes out with lots of fluctuations and on the contrary, fluctuations are going down in long time-scales. It could be comment that the SPI with short time-scale is so sensitive to moist situations and it responses with a tiny changing over the monthly rainfall. The amount of SPI stands above zero if the changing be positive. Therefore, SPI goes beneath the zero if the fluctuations be negative. These are showing that SPI is a strong device with an ability of monitoring the moist situations and short time-scale changing (agriculture drought), also the most intensive droughts in the SPI's three month periods is more than SPI's six month length periods and this is resulting from principal effecting of monthly rainfall over whole rainfall of three-month period. Therefore, it is clear that SPI is sensitive to decreasing the rainfall. Drought frequency is increased on short time-scale and their duration are small but on the contrary, on long time-scale drought frequency is decreased and their durations are more stable. At the first, droughts occur in one-month period and in case of continuation, the drought situations occur.

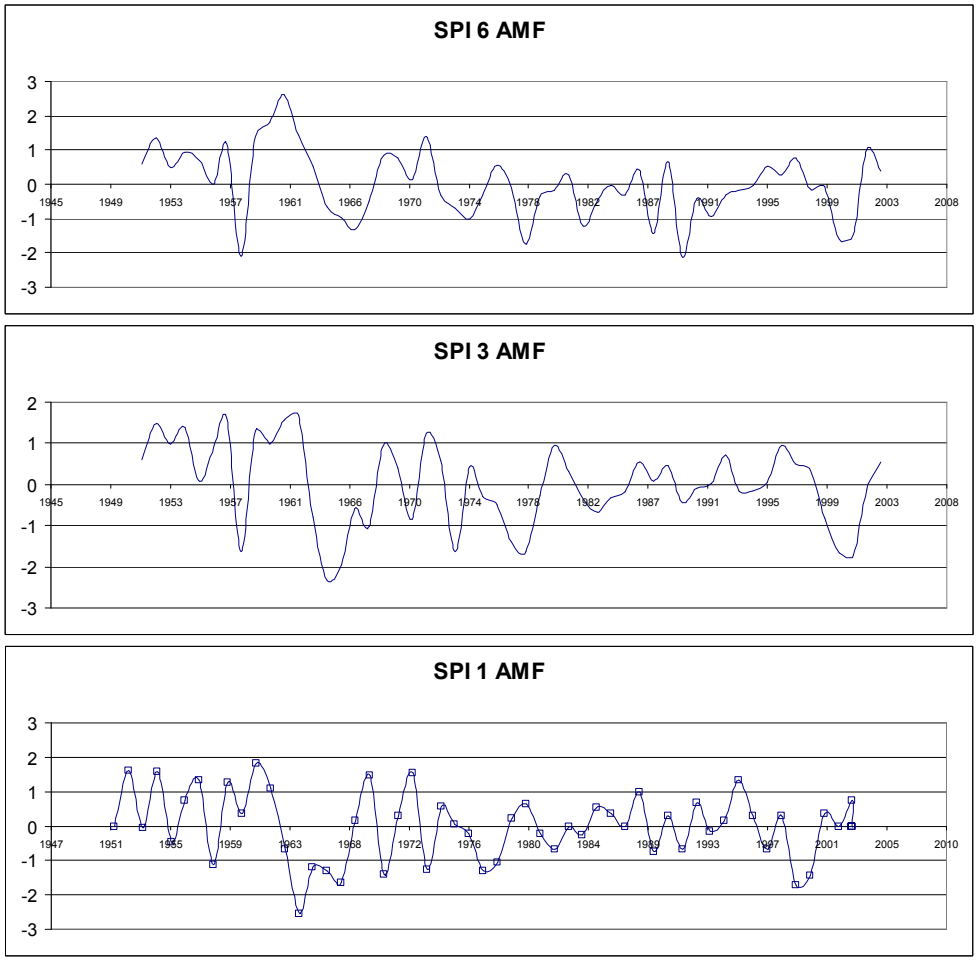


Figure 1. Various periods of SPI for under study region

Tables two, three and four show the analyzing results in among of wheat production, standard precipitation index (SPI), temperature and evapotranspiration.

Table 2. Factor of Regression (R^2) and standard division that connected to one-month SPI

month	with out evapotranspiration and temperature	with evapotranspiration and temperature	Standard Division
August	0.489	0.953	92.24
September	0.45	0.936	111.33
October	0.523	0.948	101.36
November	0.523	0.948	101.36
December	0.568	0.975	95.54
January	0.612	0.986	88.65

Table 3. Factor of Regression (R^2) and standard division that connected to of three-month SP

month	with out evapotranspiration and temperature	with evapotranspiration and temperature	Standard Division
August	0.412	0.805	159.53
September	0.436	0.864	157.92
October	0.498	0.897	95.5
November	0.478	0.902	142.89
December	0.499	0.952	117.56
January	0.541	0.976	125.36

Table 4. Factor of Regression (R^2) and standard division that connected to six-month SPI

month	with out evapotranspiration and temperature	with evapotranspiration and temperature	Standard Division
August	0.475	0.569	261.33
September	0.477	0.589	288.76
October	0.547	0.829	100.85
November	0.557	0.867	98.25
December	0.578	0.905	95.36
January	0.597	0.912	93.22

As it is observed, analyses have been form by two steps. First, without any counting on evapotranspiration and temperature but counting those parameters happened in the second step. At the first step, in the manner that Regression's factor was down and there was not seen any proper correlation between SPI and crop production because, the others parameters such as, evapotranspiration and temperature are effectible over the production's rate. It means that having just only the SPI index for prediction of crop production's rate does not give the correct result. Therefore, putting parameters of evapotranspiration and temperature into consideration causes the increasing of Regression's factor and it makes great correlation.

The results have expressed SPI of monthly duration since November until April, has a great correlation in among of production's rate, SPI, evapotranspiration and temperature (table three). It is the same on three-month SPI duration (table four). There is a great correlation on six-month SPI period onwards January. The R^2 is 0.569 in November in six-month SPI and it equals 0.589 in December, which are less than 0.8, that is the desirable amount. The highest amount of R^2 is observed in April in SPI of one-month duration and the minimum rate of that belongs to November in SPI of six-month period. The purpose of one-month SPI, three-month SPI and six-month SPI in a row mean that the amount of rainfall during the last month, three and six-month before the month's name which is mentioned after SPI. Having concentration over great correlation in among of one-month SPI, three-month SPI and the months January, March and April, of the six-month SPI, we can create a good model for prediction of crop production, months before the harvest season. The R^2 of November and December did not enter the model because of their low size.

The standard precipitation index had a strong role at this relationship. Evapotranspiration and temperature on this linkage are the causes of raising its accurateness and they could brief the others factors inside of themselves such as, production and harvest's management. Although, the most significant factor for having harvest is access to water. The prediction's model of SPI for one, three and six-month are close to %95 which was gained by the following path.

Prediction model for crop production for one-month SPI:

$$Y = -1667.55 + 1.4634X_1 + 219.5412X_2 - 73.1728X_3 + 413.2994X_4 - 146.483X_5 - 281.854X_6 \quad (1)$$

Table 6. Parameters used in the model

Y	X_1	X_2	X_3	X_4	X_5	X_6
(kg/h)crop production rate	(mm/year) evapotranspiration	(°c) temperature	SPI 1- 8	SPI 1- 9	SPI 1- 10	SPI 1- 12

Prediction model for crop production for three-month SPI:

$$Y = -293.817 + 12.30229X_1 + 53.4323X_2 + 23.4521X_3 - 405.072X_4 + 439.1102X_5 - 160.248X_6 \quad (2)$$

Table 7. Parameters used in the model

Y	X_1	X_2	X_3	X_4	X_5	X_6
(kg/h) crop production rate	(mm/year) evapotranspiration	(°c) temperature	SPI 3- 8	SPI 3- 11	SPI 3- 12	SPI 3- 1

Prediction model for crop production for six-month SPI:

$$Y = 3311.806 - 23.45X_1 - 174.365X_2 - 123.256X_3 + 668.667X_4 - 539.754X_5 + 624.8019X_6 \quad (3)$$

Table 8. Parameters used in the model

Y	X_1	X_2	X_3	X_4	X_5	X_6
(kg/h) crop production rate	(mm/year) evapotranspiration	(°c) temperature	SPI 6- 10	SPI 6- 11	SPI 6- 12	SPI 6- 1

Prediction for two years 2007 and 2008 were done with the models. Predictions, using the one-month SPI model had a closer result to the observed amount, followed by the three-month SPI model and after them came the six-month SPI model.

Table 9. Wheat prediction using the models

year		spi-1	spi-3	spi-6
2007	Observed(Kg/h)	1200	1200	1200
	Predicted(Kg/h)	1100	1095	1000
2008	Observed(Kg/h)	1060	1060	1060
	Predicted(Kg/h)	988	899	878

4. CONCLUSION

From the viewpoint of economic, wheat production is too important for Hamedan province. Drought makes variant and unknown conditions for gaining the production. Consideration on the constantly drought of breaking out these latest years, it is necessary to have the management of drought's risk and making plan on the base of prediction's models for decreasing the production's damages.

This article represents proper paths for predicting the production with SPI, evapotranspiration and temperature few months before the harvest time in Hamedan province. The results of this research specifically belonged to Hamedan province. Therefore, matching this method to other regions needs more data to have correct predictions. However, this method looks so sufficient for crop prediction. This can inform farmers about their harvest also it is a good way for insurance companies to get rid of unknown situations.

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IS ROMANIA PREPARED FOR CLIMATIC CHANGES FROM LAND RECLAMATION AND IMPROVEMENT ARRANGEMENTS POINT OF VIEW?

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Abstract: 2005 and 2007 were two years which demonstrate that Romanian authorities presented difficulties in managing water crisis situations. Water excess periods followed at short time by water scarcity periods with significant negative impacts on population and economy proved the inexistence of a coherent management plan regarding the maintenance and exploitation of land reclamation and improvement systems. The investments were important during the last years and, more than that, it was adopted a legislative framework in order to promote the efficiency and effectiveness of this type of arrangements. Unfortunately, the adoption of integrated BMP's (best management practices) for farms at hydrographical basin level continues to be a target with an unknown term of appliance. The paper will present a situation of land reclamation and improvement arrangements at national level will discuss the Romanian legislation regarding these problems and will present opinions connected with Romania's level preparation for climatic changes from land reclamation and improvement point of view.

Keywords: land reclamation, arrangements, climatic changes

1. INTRODUCTION

At this hour is little doubt that climate change is happening. This process is now recognized as one of the most serious challenges facing the world – the peoples, the environments and the economies. It was scientifically proved that the high concentration of greenhouse gases in the atmosphere is causing the phenomenon known as global warming.

The temperature in Europe increased by almost 1° Celsius in the past century, faster than the global average. Climatic conditions have become more variable and sometimes, with unexpected results on humanity. Rainfall and snowfall are presenting significantly majored values in northern Europe while in the southern Europe the precipitation fallen as volumes and phenomenon as drought are more frequently. Temperatures become more extreme and floods more often, sometimes these phenomenons followed one to other at very short period of times.

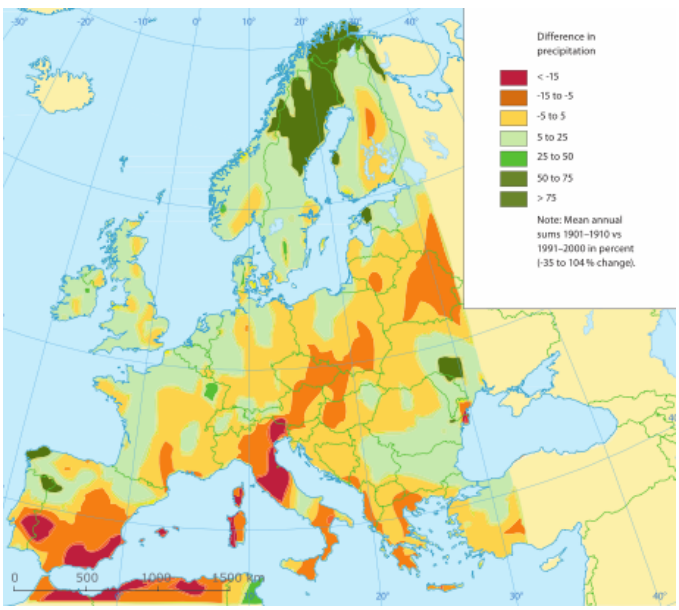


Figure 1. Trend in mean annual rainfall 1900 - 1998 (EEA source)

The last 7 years in Romania presented a distinctive variability regarding the precipitation regime and the temperatures variation. These factors (temperature, precipitation) impose to be as restrictions of agricultural productions from the most Romania's geographic regions in the same time proving to be drastic limiters or, on the other side, powerful sources with negative effects about the available resources for hydroameliorative measures.

Years which had been remarked by a strong drought were followed or preceded by periods with floods with different degrees of intensification. The causes are numerous, the anthropic pressure proving to be the most communes.

Land reclamation and improvement arrangements, especially drainage and surface drainage works, if till a few decades ago were indispensable for agriculture in present they are proving to be harmful for the soil fertility. Aridity phenomenon has installed in many areas and stressed the water scarcity one additional cause being the practice of an intensive drainage which decreased the water table. In the same time, the inadequate maintenance of surface drainage channels proved to be a major cause of the flooding from 2005 with disastrous effects about a large area with significant population.

Table 1 Natural Disasters – number of people killed at national level (1983 – 2008)

Disaster type	Date	Number of people killed
Flood	1991	108
Extreme Temperature	2006	68
Extreme Temperature	1998	60
Extreme Temperature	2007	38
Flood	2005	33

Table 2 Natural Disasters – number of people affected at national level (1983 – 2008)

Disaster type	Date	Number of people affected
Flood	July 1997	122,320
Flood	April 2000	60,431
Flood	September 2005	30,800
Flood	March 2006	17,071
Flood	July 1991	15,000
Flood	July 2005	14,669
Flood	July 2004	14,128

Table 3 Natural Disasters – economic damages at national level (1983 – 2008)

Disaster type	Date	Damage US Dollars
Flood	July 2005	800.000.000
Flood	April 2005	596.000.000
Drought	2000	500.000.000
Flood	August 2005	313.000.000
Flood	June 1998	150.000.000
Flood	June 2001	120.000.000
Flood	July1997	110.000.000
Flood	April 2000	100.000.000

2. LAND RECLAMATION AND IMPROVEMENT ARRANGEMENTS IN ROMANIA

In Romania, the land reclamation and improvement arrangements had as component parts the following types of constructions: pumping stations and/or gravitational evacuation, canals network, drainage networks, pipes networks, bridges and/or small bridges, underground crosses, aqueducts.

Romanian land reclamation and improvement works are organized according to Law 138/2004 as it follows:

a) Irrigation arrangements – 3.077.069 ha, from which:
- sprinkler irrigation: 2.761.803 ha
- furrow irrigation: 263.026 ha
- irrigation by flooding (rice arrangements): 52.240 ha
- pumping stations for irrigations: 3.210
- reversible (irrigation – surface drainage) pumping stations: 41

b) Surface drainage arrangements – 2.950.451 ha, from which:
- gravitational: 1.521.268 ha
- with pumping: 1.429.183 ha
- surface drainage pumping stations: 699

c) Drainage – 246.539 ha

d) Soil erosion control arrangements: 2.210.234 ha

e) Surfaces with work defence against flooding:
- dams at Danube river and interior rivers: 821.204 ha
- barrages for fast flooding attenuation: 190.904 ha

The evolution of these works till 2004 is presented in the next table:

Table 4 The evolution of land reclamation and improvement works (1944 – 2004)

Year	Dammed	Irrigated	Drainage	Preventing soil erosion
1944	622.0	18.0	358.0	-
1950	642.0	42.5	368.1	2.0
1955	668.8	93.1	404.4	9.4
1960	827.1	199.6	505.7	100.0
1965	856.7	229.9	587.0	197.5
1970	1331.9	731.3	1111.4	435.3
1975	1455.2	1474.2	1965.5	983.1
1980	1545.0	2301.0	2462.5	1609.7
1985	-	2965.3	2948.8	2095.5
1990	-	3168.7	3168.7	2222.3
1991	-	3125.8	3194.1	2282.4
1992	-	3197.2	3182.1	2264.4
1993	-	3202.3	3188.5	2253.4
1994	-	3202.8	3191.7	2267.9
1995	-	3205.2	3196.2	2267.8
1996	-	3211.1	3199.5	2279.1
1997	-	3190.6	3198.8	2276.2
1998	-	3184.0	3198.8	2276.5
1999	-	3179.8	3201.5	2276.9
2004	-	3077.1	2950.5	2210.2

2.1 Flood risk management plan

A sustainable and competitive agriculture cannot exist without these kinds of arrangements, given been the climatic conditions from Romania. As an example, the activity of defence against flooding is primordial. The arguments for sustainability in this domain are based on economical, human, social principles. Also, we must take in consideration the necessity of a quick removal regarding the negative effects and the reestablishment of the normal works exploitation.

According to Government Decision (G.D.) 1.854, flooding risk managements means the appliance of some policies, procedures and practices having as objectives risk identification, their analysis and evaluation, the risk monitoring and re-evaluation in order to assure for the human communities a sustainable environment from physical and social point of view. Flooding risk about human life and activities can be reduced by creating and implementing the risk maps at flooding. They were decided as a primordial necessity on the basis of H.G. no. 1854/2005, published in M.O. no. 72/2006. Their creation was started by Romanian Government having from E.U. final term of realization being 2013.

A good flood risk management is the result of some interjectorily activities, interdisciplinary which containing waters management, territory and arrangement and the urban development, nature protection, the agricultural and forestry development, transport infrastructure protection, the protection constructions and tourist zones protection, communitarian and individual protection, every sector having his attributions in the realization of a specific activities.

2.2 Drainage arrangements in Romania

From the last researches made by Romanian Academy of Soil Science, more than 8.6 million hectares are affected by humidity excess, so we can understand the importance of surface drainage and subsurface drainage works. Through surface-drainage is apprehended the first stage of the water excess evacuate drainage from the agricultural grounds, through leakages at surface, with help of surface drainage open channels, the drainage representing the second stage of surface drainage for lowering high water table level (0.0 to 0.5 m) level imposed by agricultural cultures requests towards the air-water regime from the soil profile where these develop their radial system.

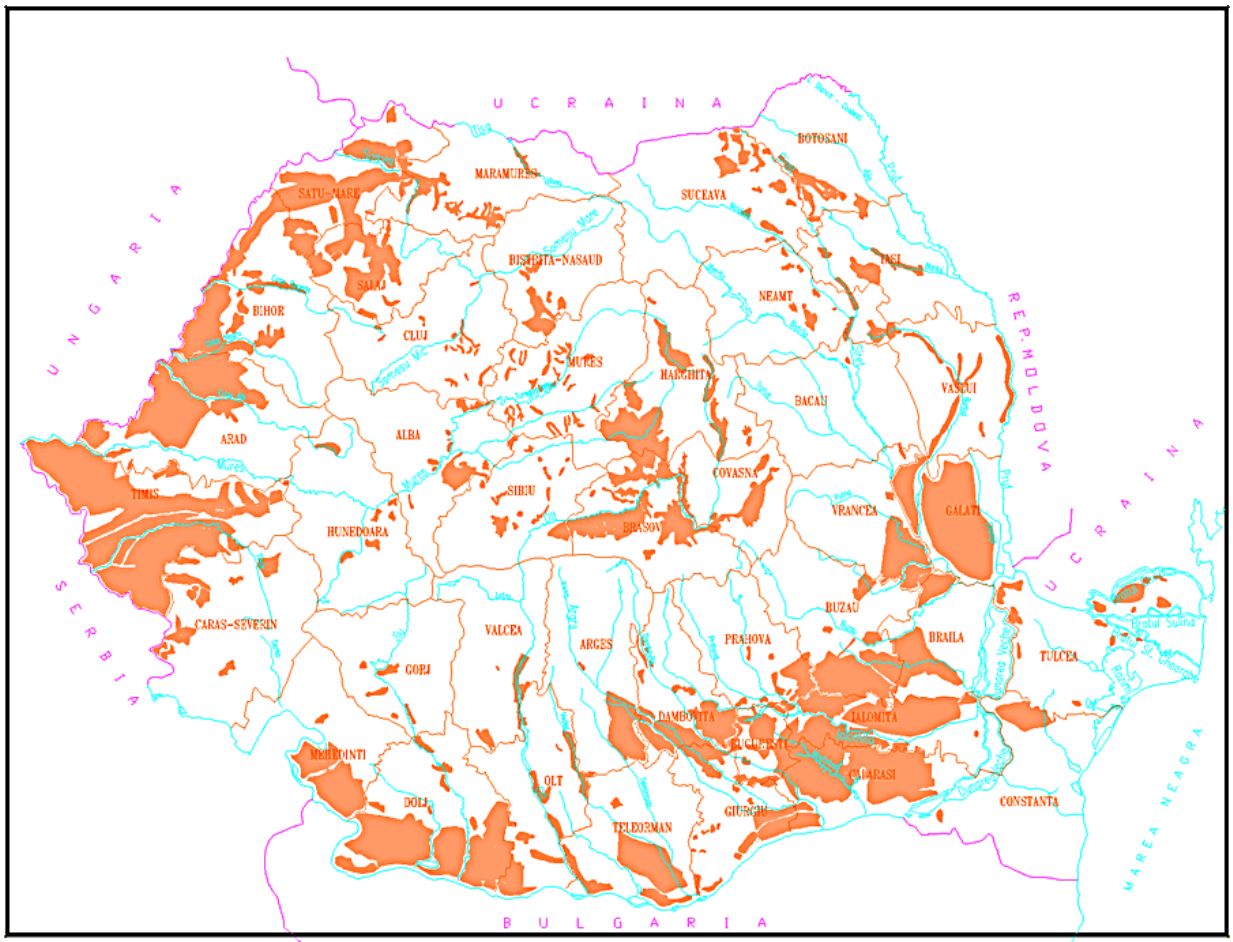


Figure 2. Surface drainage and drainage arrangements in Romania

Unfortunately, on many situations, these works are not sufficient for humidity excess removal being necessary the completion with other specific works as: deep loosening, land shaping, soil levelling and mole drainage. Another supplementary and completion exploitation works are: the surface leakages assurance from depression zones in channels network, through the execution of leakages gutters; agro-technical works executed for soil humidity decrease; clearing surfaces invaded by reeds; increase the arable horizon humidity in the droughty periods through irrigations; descent the water table level through supplementary drainage works.

2.3 Irrigation arrangements in Romania

Humidity excess is not the only problem for Romanian agriculture. Drought periods, which appear more and more often, must be struggled, the most important option being the appliance of irrigation. The irrigation has the role to complete the water deficit from soil profile, realizing in this way normal condition for agricultural development in drought periods. The irrigation types practiced in Romania are: sprinkler irrigation, furrow irrigation (furrow and border strip), drip irrigation, underground irrigation (drainage - subirrigation reversible areas), special irrigations (fertilization, against frozen). The choosing of irrigation method depends by the source and water quality, the land topography, soil textures, the cultures which will be irrigated etc. The most of the irrigation areas from our country use the sprinkler irrigation, with big energetic costs for water pumping, that why only the territories near Danube water with big irrigation systems allowed this type of water distribution. National Administration of Land Reclamation and Improvement and National Society "Land Reclamation and Improvement" are the juridical persons which must apply an efficient management for the mentioned works. Also, an important contribution, especially in irrigation management, belongs to the private sector where was created a special organism for the efficient exploitation of irrigated areas, The Organization of Irrigation Water Users.

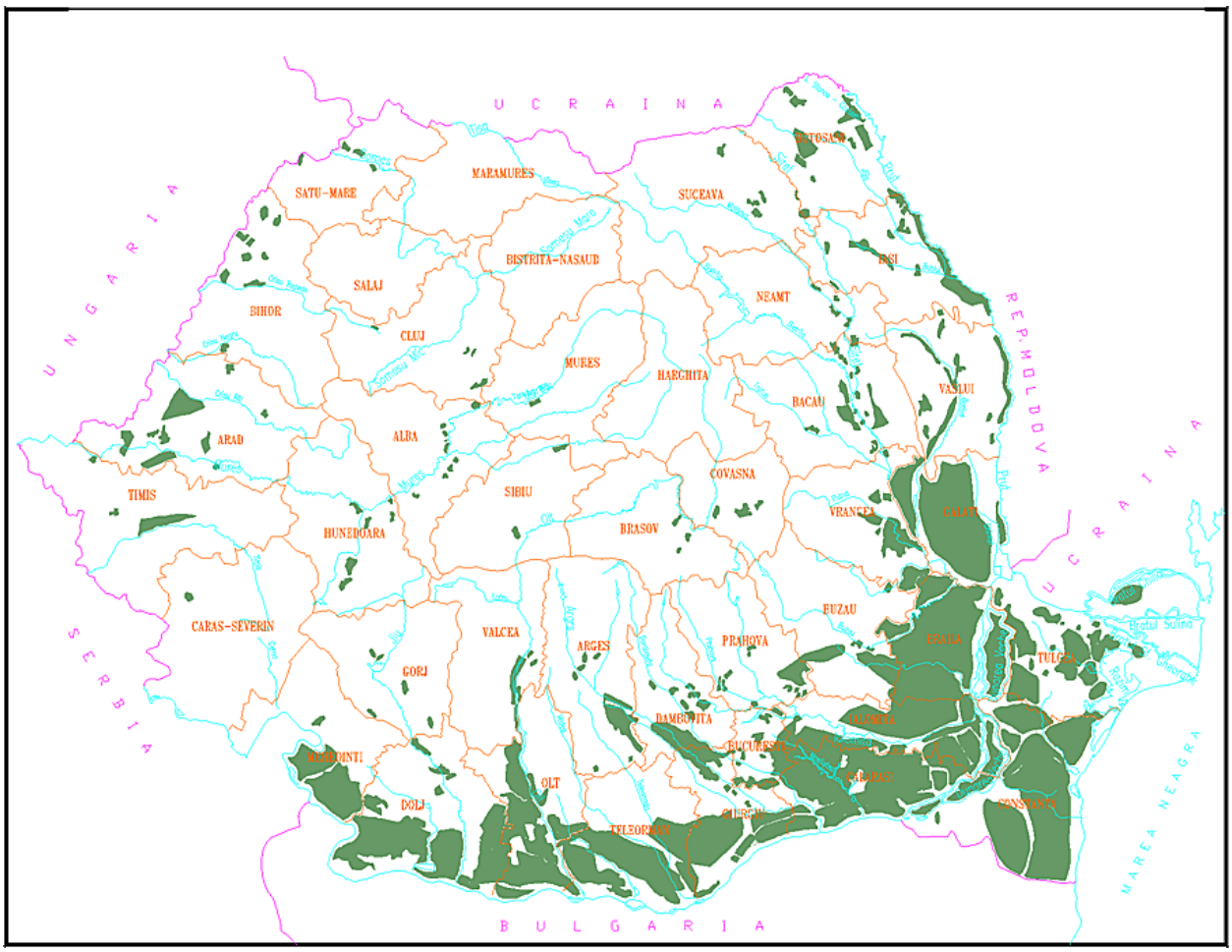


Figure 3. Irrigation arrangements in Romania

2.4 Soil erosion control

Soil erosion control arrangements may complete another land reclamation and improvements works (as irrigation or drainage) or can be individual systems. The soil erosion manifests intensely and below different forms, facts which can lead to a continuously impoverishment and soil

destruction and implicit to the agricultural production decrease, sometimes catastrophic. Therefore, the regularization of the versants leakages, soil erosion control and the grounds economic capitalization from the hills zones constituted and constitute an important preoccupation in agricultural area and respective in the land improvements sector. At country level, the total surface arranged with works and measures for the soil erosion control is of 2.2 millions ha. Among the main technical works which do the mentioned surface object, we mention: coastal and marginal channels (14.000 km), terraces (25.000 km), markets (6.000 km), concrete drops (35.000 km), drain tubes, terraced drain (30.000 km), plantations against erosion (35.000 ha).

3. ROMANIAN LEGISLATION REGARDING CLIMATIC CHANGES

Romania ratified the United Nations Framework Convention on Climate Change (UNFCCC) by Law no. 24/1994 manifesting clearly its concern to mitigate this problem at global level and its political will to fulfill its commitments. In 1999, Romania signed the Kyoto Protocol being the first UNFCCC Annex I Party that ratified it by Law no. 3/2001. The value of the greenhouse gas emissions reduction target adopted by Romania is of 8% compared to the base year 1989. The Kyoto Protocol entered into force at international level on the 16th of February 2005. In July 2005, the first National Strategy on Climate Change was approved by the Governmental Decision no. 645/2005. The Strategy represents the general framework for implementing climate change policies and measures in the period 2005-2007.

The National Action Plan on Climate Change was approved by the Governmental Decision no. 1877/2005 following the strategy adopted earlier. The Action Plan contains concrete actions, including the financing sources and the responsible institutions, needed for the accomplishment of the objectives adopted in the strategy. Although the Kyoto Protocol's flexible mechanisms are "voluntary", Romania got involved successfully in the development of "Joint Implementation" projects based on the cooperation with different countries, for the reduction of greenhouse gas emissions, being at this moment the most attractive country on JI, according to the Point Carbon Agency. Romania has initiated and continues to develop bilateral cooperation with different states, for the development of this type of projects

5. IS ROMANIA PREPARED FOR CLIMATIC CHANGES FROM LAND RECLAMATION AND IMPROVEMENT ARRANGEMENTS POINT OF VIEW?

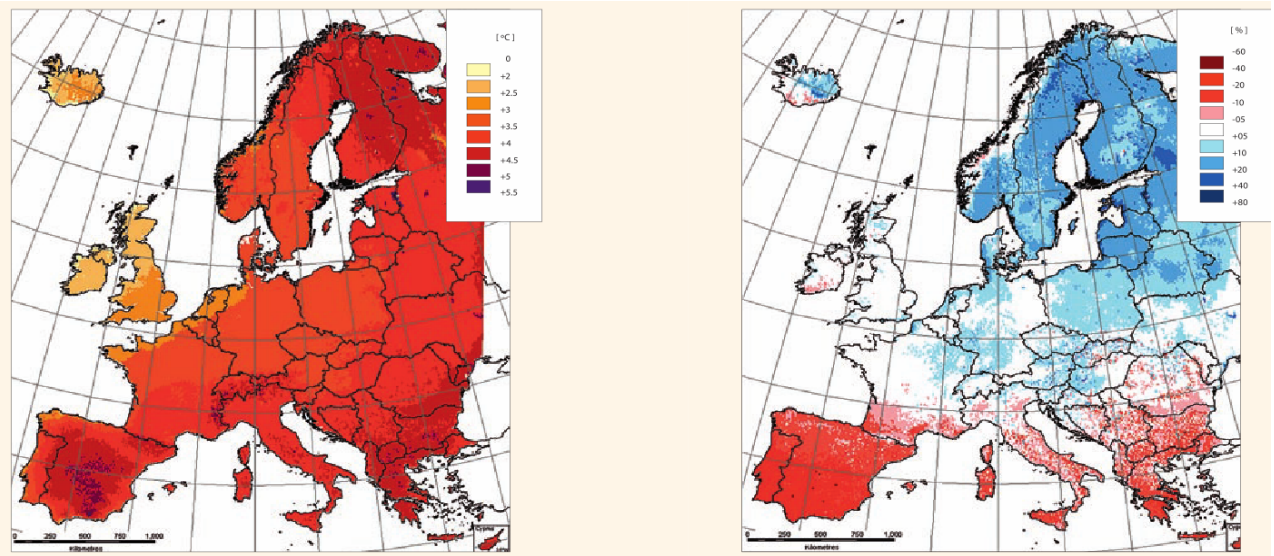


Figure 4. Projected changes in mean annual temperatures and precipitations by the end of 21th century (source: http://prudence.dmi.dk)

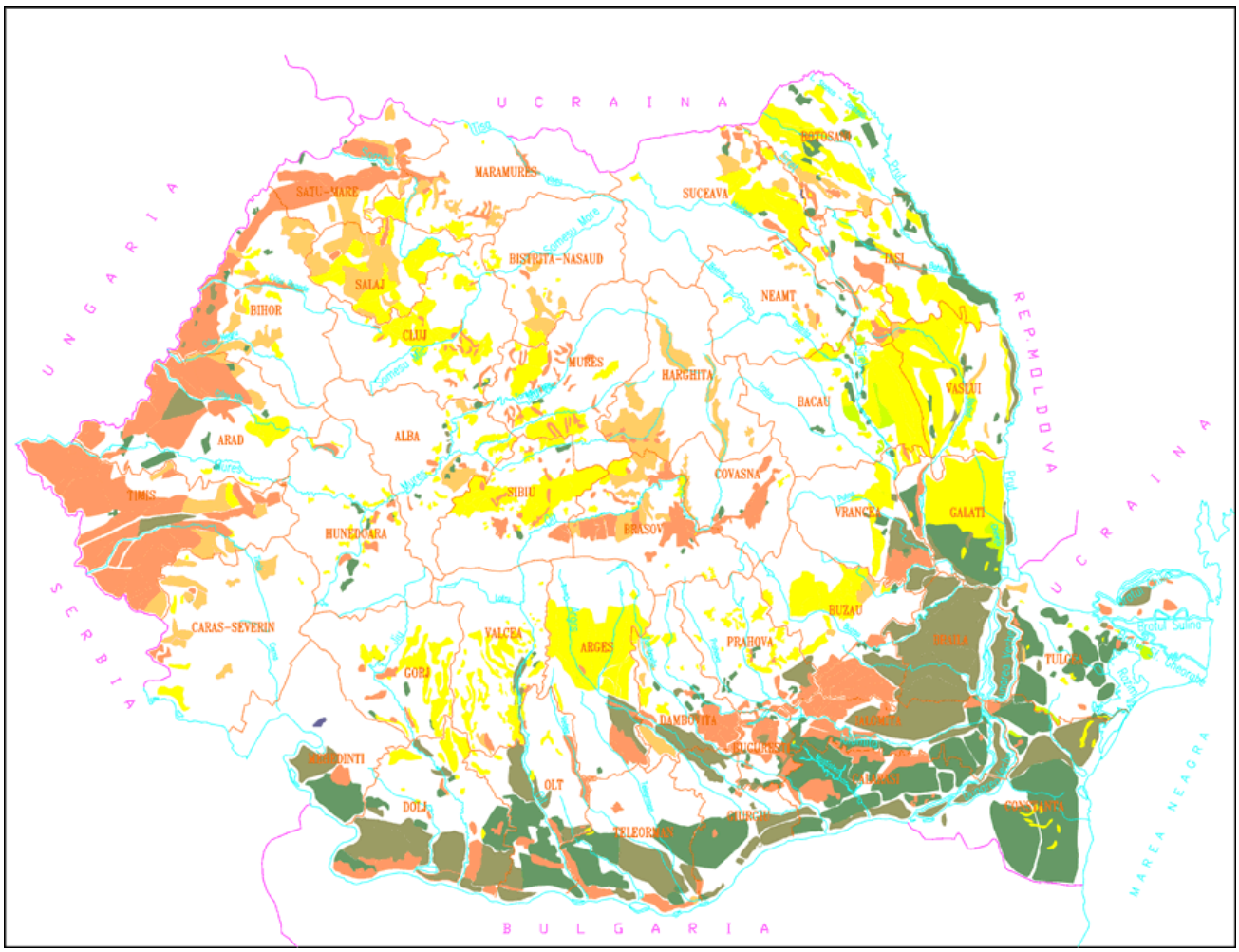


Figure 5. Land reclamation and improvement arrangements in Romania

As a result of adherence to European structures, Romanian agriculture will have to face the competition with other countries for agricultural products capitalization and gaining new markets, partially lost in the transition period 2000 – 2006. Taking in consideration the climatic conditions and soil resources from arable land, Romanian agricultural system depends crucially, among other agricultural elements, on land reclamation and improvement works. Simple existence and functioning of these doesn't presuppose their efficiency in the creation of proper conditions for obtaining rich crops. It is impose the implementation of an adequate management, efficient for the European legislation implementation also in this specific domain, part of Romanian agriculture, which works in private system, in order to assure a sustainable development.

The physical and technical state of these arrangements imposes as a primordial necessity their re-technologization through rehabilitation and/or modernization for assuring their efficient functioning.

Romanian legislation is adapted to the European requests not being resolved the problem of arrangements rehabilitation which are in service. Through Rural Development National Program 2007 – 2013 are foreseen supporting measures of water users association, land reclamation and improvement organizations, etc.

Governmental organizations (as A.N.I.F. R.A.) from land reclamation and improvement elaborated strategies of rehabilitation and modernization for the existent arrangements, of encouraging the beneficiaries to constitute land reclamation and improvement organizations and to take in administration, at request, without payment, according to the existent legislation, land reclamation and improvement arrangements.

6. CONCLUSIONS

The climatic changes, humidity excess, drought and the permanent lack of water are interconnected but these phenomenon/ processes mustn't be confused. The permanent lack of water is linked by aridity as natural phenomenon in time while a wrong exploitation of available resources (during the normal periods or with a surplus), the missing of an un-sustainable water management can constitute, easily, the way to desertification appearance, result of a major negative impact from anthropic pressure. Drought is acting like a temporary un-balance of request/ water resources report, often in combination with the influences produced by anthropic factor.

From legislative point of view, the adoption and implementation of Water Framework Directive imposes to Romanian Government and its structures to adopt and apply strategies in concordance with European legislation and to have in view the transboundary water management plans. Bringing and implementing in Romania new techniques as integrated drainage management can constitute solutions for the central and local authorities in this domain.

Despising the fact that Romania adhere at United Nations Convention Against Desertification by the Law 111 from 5 June 1998, the legislative framework is very poor in this sense existing a strategy but without efficiency and effectiveness. Following the floods from 2005, in a relative short time, were elaborated handbooks for local authorities, these including measures and directions for actions in case of calamities. Perhaps that, following the drought from this year, will be elaborated a similar handbook for this kind of situation, unfortunately after its producing and deployment with severe effects over the society. I propose to be prepared and elaborated as soon as possible this kind of handbooks, for drought situation, necessary to the local and central authorities.

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CLIMATE CHANGE IMPACTS ON GROUNDWATER RESOURCES IN THE JAFFNA PENINSULA, SRI LANKA

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Abstract: The groundwater resources of the Jaffna Peninsula, Sri Lanka, constitute an example of aquifers subjected to severe and excessive pumping. Groundwater is the prime source of water for agricultural development in the peninsula, but its potential is limited as over-abstraction from the aquifers results in saline intrusion. The recharge to the groundwater in the peninsula is almost entirely from rainfall percolation and the climate is characterised by distinct dry and wet seasons. A three-dimensional groundwater flow model has been applied with the view to assess the impacts of the climate on the groundwater resources system in the area. Two different future scenarios have been used to evaluate the climate change impacts in the peninsula and were compared with a scenario of the existing conditions. The first scenario examined the effects of increasing the rainfall and the temperature, and the second scenario considered an additional increase in sea level and runoff. The results showed that during the dry season, water levels increased in the middle of the aquifers but were almost the same in the coastal areas. On the other hand, during the wet season, there was a significant increase in the water levels. Furthermore, the model showed that a significant quantity of fresh water ends up in the sea through seepage and through the lagoons. Consequently, the results demonstrated that the aquifers can be used as underground reservoirs to store the excess water in the rainy season and to satisfy water demand in the dry season. In addition, the changes in rainfall and temperature were identified as the predominant factors of impact, while changes in sea level rise and runoff were found to be only of secondary influence for the groundwater resources. It was concluded that climate change could improve the existing condition of the aquifer in the peninsula under a capable and efficient water management strategy.

Key words: Climate change, Sea level rise, Temperature, Rainfall, Runoff, Groundwater model.

1. INTRODUCTION

Water resources in coastal aquifers are an important source of fresh water. In the Jaffna Peninsula, in Sri Lanka, the fresh water resources, which play a key role to the development of the region, are characterised by variation in the water levels through the year. However, climate change can significantly affects groundwater resources and thus influence the availability of fresh groundwater.

Previous studies related the variation in temperature and precipitation during the year with the direct impact on changes in groundwater levels and quality (Brouyere et al. 2004; Chen et al. 2004). Such studies used different modelling techniques such as water balance models (Kruger et al. 2001; Arnell 1998), empirical and conceptual models (Chen et al. 2002; Cooper et al. 1995) and numerical models (Mustadraft et al. 2008). However very little research has focused on the potential effects of climate change on groundwater resources (Gleick 1986; Amell 1999; Mimikou et al. 2000).

The Intergovernmental Panel on Climate Change (IPCC) developed long-term emission scenarios referred to as Special Report on Emission Scenarios (SRES), which have been widely used to analyse possible climate change, its impact and mitigation strategies. Climate change projections have been developed for each scenario based on expected changes in population, carbon dioxide concentration, global mean annual temperature and mean sea level rise (IPCC 2007).

Droggers (2004) studied the climate change impacts in Walawe, Sri Lanka, by using climate projections from the Hadley General Circulation Model (HadCM3) and the climate change

scenario (A2). He stated that precipitation and temperature would both increase in the future with high variations in climate. In addition, he showed that the number of consecutive years when precipitation is severe and the number of years with extreme temperatures is expected to be higher in the future. Overall, the impact of climate change in Walawe was shown to have more positive than negative effects.

Based on the IPCC climate change scenarios, an integrated approach was applied to evaluate the impacts of climate change on the water cycle in the Jaffna Peninsula. To access the impacts of potential climate change on fresh groundwater, we focused on changes in precipitation, temperature, sea level rise and runoff.

2. THE STUDY AREA

2.1 Geographic Information

Jaffna Peninsula forms the northern part of Sri Lanka with an area of approximately 1017 km^2 (Figure 1). The peninsula is about 70 km long and 10 to 36 km wide. It is delimited by the Palk Strait on its western and northern side, by the Bay of Bengal on the east and by mainland Sri Lanka on the south. Jaffna includes two internal lagoons, namely the Vadamarachy lagoon or Thondamanaru lagoon and the Upparu lagoon, with an area of 87.1 km^2 and 34.7 km^2 respectively. The peninsula is separated from the mainland by two external lagoons, the Jaffna lagoon and the Elephant Pass lagoon.

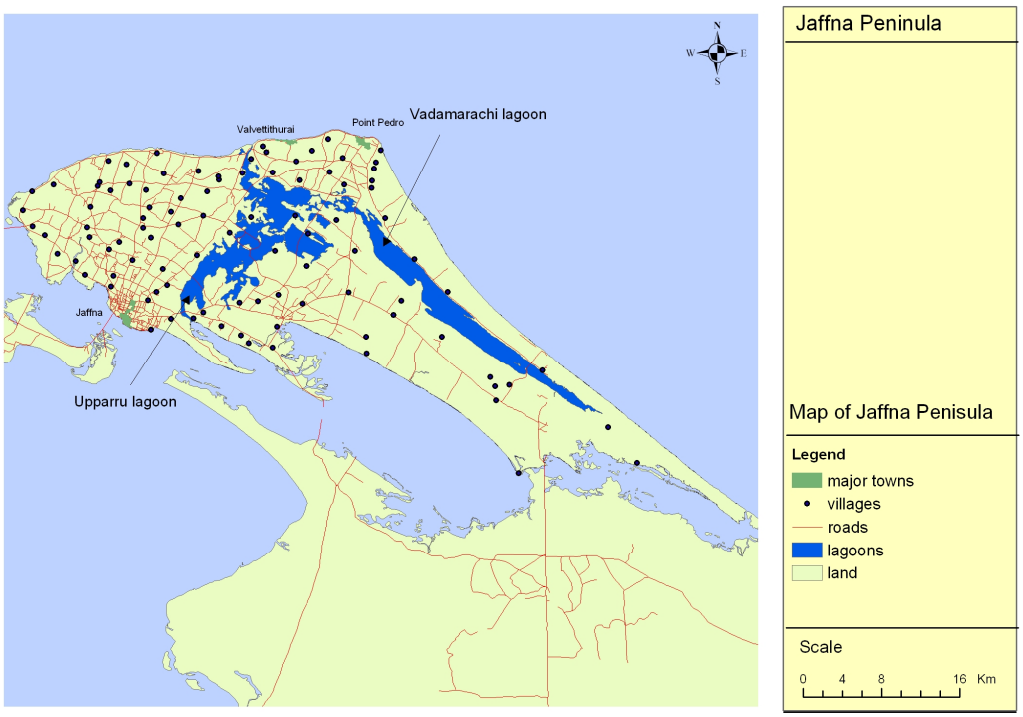


Figure 1. Geographical location of the Jaffna Peninsula, Sri Lanka.

2.2 Climate

The climate in the Jaffna Peninsula is characterised by distinct wet and dry seasons. A rainy season occurs from September to December, while in the remaining months the influence of the monsoon is characterised by very low rainfall (Navaratnarajah 1994; Nakagawa et al. 1994).

There are 16 rainfall stations on the peninsula and the nearby mainland. Although 80% of the precipitation falls between September and December, however the peninsula receives most of the rainfall during November. This leads to heavy runoff. It is estimated that 10-15% of the annual rainfall is lost as runoff (Nandakumar 1994).

According to Fernando (1973), while the rainfall follows a periodic pattern, there is a significant variation from year to year. It is approximated that the annual rainfall is 1300 mm. Records for the period between 1975 and 1994 show an annual rainfall of 1266 mm while for the period 1999 to 2004 it was calculated to be 1490 mm. It is noticeable that for 2003 it was 1328 mm and for 2004 the annual rainfall was 1868 mm. Figure 2 presents the average monthly and total annual rainfall.

It can be observed that the average annual rainfall has increased between 1999 and 2004. This change occurs in the wet season, as illustrated in Figure 2 and shows the increase in rainfall during extreme events, which is an important indicator of climate change impacts.

The annual potential evapo-transpiration is calculated at 1858.9 mm and the pan evapo-transpiration ETpan is 1224.9 mm. According to Nandakumar (1994) the actual rate of evaporation of the peninsula is 2260 mm and the total evaporation during the period from mid-January to mid-April is just over 600 mm.

From the total rainfall, 10-15% is lost as direct runoff, 40-48% as evapo-transpiration and only 30-32% is left over for groundwater recharge (Navaratnarajah 1994).

Simulations of the chloride concentrations for 1999 to 2005 showed that there is a correlation between the rainfall and the salinity in the Jaffna Peninsula (Tyriakidis et al. 2006). The results suggested that after heavy rain the concentration of chlorides is reduced while after a light rain the levels of chlorides are increased.

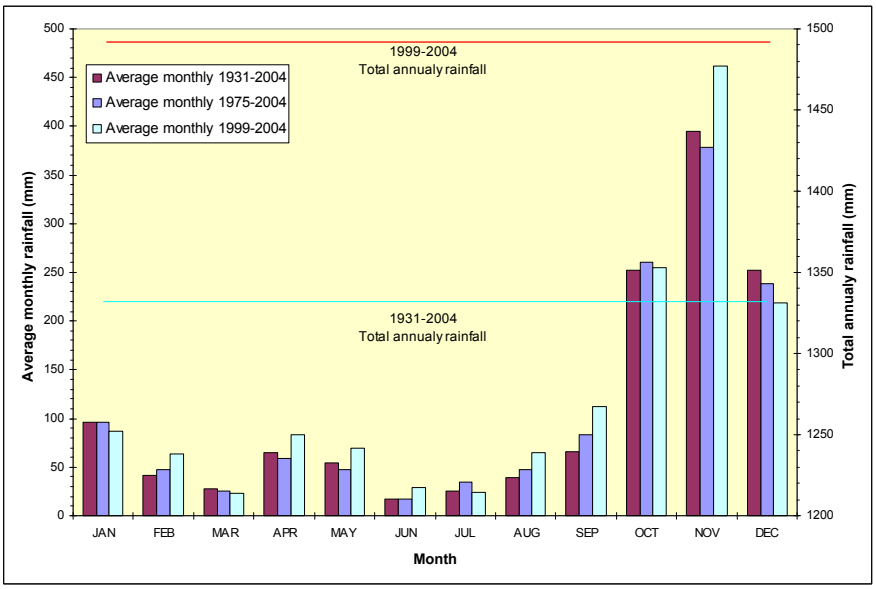


Figure 2. Average monthly and total annual rainfall.

2.3 Geology and Hydrogeology

Jaffna peninsula is mainly composed of Miocene limestones, which are exposed at the north central part (Balendran et al. 1968; Cooray 1984), extending in a NNW to SSE direction. The limestone is underlain by a thick sandstone formation, known as Mannar Sandstone Formation. Bordering with the limestone on the western part are patches of red bed formations, encompassing the brown sand formation which occupies a large area on the peninsula (Moorman 1961). Enclosing these along the western coast are lagoon deposits. The area east of the lagoon is occupied mainly by brown sandy loams in the south and sand dunes in the north along the eastern coast of the peninsula.

The hydro-geological system of Jaffna Peninsula consists of three aquifers separated by the lagoons, which worked as hydraulic barriers. The first aquifer occupies the west part of the peninsula and is called Chunnakam aquifer, the Point Pedro aquifer lies on the east part of the peninsula and the third aquifer is located on the southwestern part of the peninsula and is called Chavakachcheri aquifer. Fresh water is mainly located in limestone but on the Point Pedro and Chavakachcheri aquifers it can be found in sand deposits. All aquifers are unconfined and the core of fresh water lenses has a maximum thickness of 15-24 m as measured from the water table to the top of the transition zone (Kraft 2002).

3. METHODOLOGY

3.1 Numerical Modelling

A three-dimensional groundwater flow and quality model has been used to evaluate the impacts of climate change on the groundwater. The groundwater model for the region has been developed by the first author, by using the software HYDRO-3D (Guganesharajah 2001), which simulates spatial and temporal distribution of water levels, velocities and several water quality parameters. The model cells are tetrahedral and the model uses the finite integral element numerical method to predict flows and water levels in the system.

The modeled area covers 1019 km^2 and consists of the entire peninsula, including the two lagoons. The model boundaries of the hydro-geological system are constant head (0.43 mAOD) along the coastline, which is the mean sea level. The top of the layer represents the water level of the unconfined aquifer and the bottom layer corresponds to the base of the limestone, which is represented by the no flow boundary.

The domain of the study is discretised with normal triangular elements, which vary from 350m on the land to 200m on the lagoons. The elements are well adapted to precisely describe the aquifers and the lagoons and also to manage the model with short computer running time. The grid consists of eight faces and seven layers extending to a depth of 94 m, with 13 865 nodes for each face and 552 510 tetrahedrals in total.

The first layer represents limestone, lagoonal and estuarine deposits, dune sands and yellow and brown sands. The base of the first layer varies from 5-15m. The remaining layers are part of the limestone formation and their thickness has been chosen wisely to simulate the water quality of the aquifer (Figure 1).

The values of the hydraulic conductivity for the limestone have been estimated by calibrating the model until the simulated piezometry is adjusted to the observed piezometry during the period between 1999 and 2004. Porosity (n) and storativity (S) of the aquifers and the hydraulic conductivity for Point Pedro and Chavakachcheri aquifers have been estimated from the aquifer tests (Engineering Sciences, 1984) and presented in Table 1.

The calibrated model for the period 1999 to 2004 used the CropWat code (Clarke 1998) to estimate the evapo-transpiration of the area. The CropWat code uses the Penman-Monteith method. The required data for estimating the potential evapo-transpiration in Jaffna Peninsula is presented in Table 2. Runoff was assumed to be 20% of the total rainfall and the model was run by using monthly time steps.

Table 1. Hydraulic properties of the model

Formation	n	S	Kx (m/d)	Ky (m/d)	Kz (m/d)
Sand	0.28	0.1	3	3	3
Dune sand	0.25	0.15	28	28	28
Deposits	0.3	0.1	2	2	2
Limestone	0.2	0.05	112 - 280	112 - 280	112 - 280

Table 2. Climate data of Jaffna Peninsula.

Month	Maximum temperature (°C)	Minimum temperature (°C)	Humidity (%)	Wind speed (km/d)	Sunshine (hours)	Solar radiation (MJ/m2/d)	ETo (mm/d)
January	27.78	23.78	74.59	137.40	8.20	19.50	4.03
February	29.05	25.05	68.31	129.60	8.90	21.80	4.71
March	30.37	26.37	68.07	138.60	9.10	23.30	5.27
April	31.71	27.71	70.39	148.20	8.80	23.10	5.45
May	32.04	28.04	73.80	317.40	8.30	21.80	5.75
June	31.20	27.20	73.75	330.00	7.50	20.20	5.40
July	31.44	27.44	73.24	298.20	6.80	19.30	5.24
August	30.67	26.67	74.60	303.60	7.40	20.60	5.28
September	30.66	26.66	75.71	297.00	7.70	21.10	5.25
October	29.78	25.78	80.00	173.40	7.00	19.30	4.28
November	28.42	24.42	84.03	111.60	6.70	17.60	3.57
December	27.57	23.57	79.93	145.20	7.10	17.50	3.58
Average	30.06	26.06	74.70	210.85	7.79	20.43	4.82

The main objectives of the groundwater flow model of the Jaffna Peninsula were to simulate the piezometric levels and chloride concentrations under conditions predicted to exist by 2028 and to predict the behaviour of the quantity and quality of the groundwater under two different climate change scenarios.

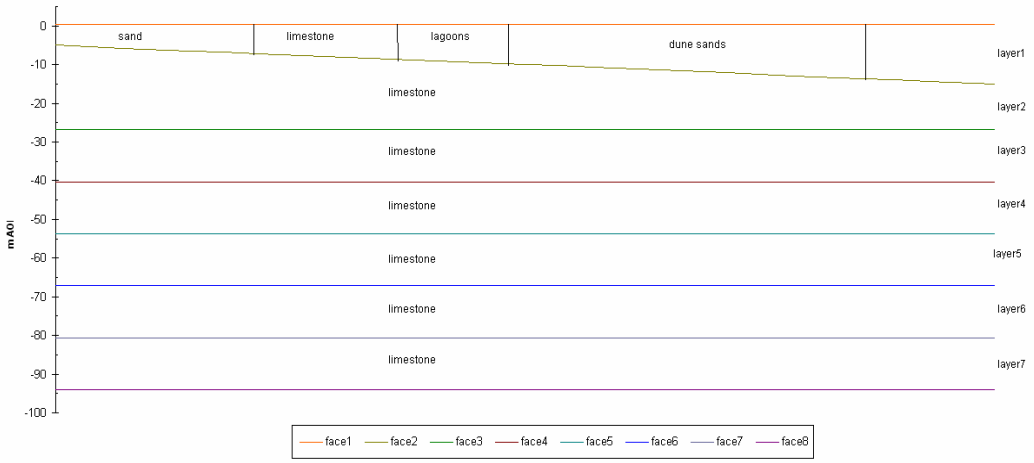


Figure 3. Conceptual model of the Jaffna Peninsula.

3.2 Climate Change Scenarios

Climate projections from the Hadley general circulation model have been used to predict climate change for all the SRES emission scenarios. The lowest emission scenario gives a global-mean temperature rise of about 1°C by year 2030 (Johns et al. 2003). Sea level will change owing to expansion of the oceans, melting of glaciers, snow and ice. Global sea level is predicted to have risen by about 0.1 m by 2030 as calculated by the Hadley model (Gregory and Lowe 2000). Finally, maximum annual five-day rainfall events have changed significantly over the last decades and showed an increase of 10% in northern mid-latitudes (Met Office 2005).

Two different scenarios were examined to show changes in groundwater in Jaffna Peninsula and compared with a base case scenario with no influence by the climate. The simulation used the calibrated values for the rainfall, temperature and evapo-transpiration to predict the situation by the year 2019. The first scenario assumed an increase in rainfall and runoff by 10% and 20% respectively and rises in temperature by 1° C. The second scenario considered an additional increase in sea level by 0.15 m and an increase in run-off by 24%. All the scenarios were simulated for 18 years, covering the period between 2005 and 2022.

4. RESULTS

A comparison between computed results for the scenarios has been prepared, providing a useful way to show the possible impact of climate change on the groundwater resources (Figure 4). Results for the base case scenario showed that the simulated water levels in the dry season of 2019 would vary from 0.43 m to –1 m. During the wet season of the same year simulated water levels would fluctuate from a mean sea level up to 4.29 m. The simulation indicates the significant relationship of the rainfall with groundwater levels.

In the first climate change scenario, computed water levels for the dry season vary from –0.57 m to 2.78 m and for the wet season from a mean sea level to 7.77 m. Comparison of results with the base case reflects that water level changes occur in the middle of the aquifers, while water in the lagoon has risen only slightly.

The second climate change scenario presents simulated water levels between –0.43 m and 2.18 m for the dry season and 0.43 m to 6.87 m for the wet season. Water levels in the lagoons have higher values than in the first scenario, but levels in the aquifers are slightly lower. Consequently, when applying the climate change scenarios there is a significant increase in the water levels in the aquifers.

Analysis of the water balance of the model showed that the boundary flow from the sea to the aquifer compared with the base case was reduced by 37% and 32% for the first and second scenario respectively during 2019. In addition, the sea water flows towards the lagoons reduced by 24% for the first scenario and 21% for the second scenario. The boundary flow from the aquifers to the sea increased by 33% and 30% for scenarios one and two respectively. For the same period the flow from the lagoon to the sea was estimated to have an increase of 27% for the first scenario and 30% for the second scenario. Differences between the base case and the climate change scenarios of the boundary and lagoon flows are illustrated in Table 3. It can be observed that lagoons and boundary flows to the sea are increased for the scenarios while flows from the sea towards the aquifers and the lagoons are reduced. However, differences for the second scenario have less influence on the water resources, excluding the flow from the lagoon to the sea, owing to an increase in runoff. In addition, sea level rise can have negative impacts on the system, but it is mainly controlled by changes in rainfall. Therefore, rainfall and temperature rises are the predominant factors, with changes in rainfall and runoff to play a secondary role on the changes in groundwater resources.

Overall, the results showed that during the dry season, water levels increased in the middle of the aquifers but were almost the same in the coastal areas. On the other hand, during the wet season, there was a significant increase in water levels. The increase in the levels indicates that the aquifers can be used as underground reservoirs to store water during the wet season to be provided for the dry season.

Table 3. Simulated differences in boundary and lagoon flows between base case and climate change scenarios.

Scenarios	Boundary flow change (from sea to the aquifer)	Lagoon flow change (from sea to the lagoon)	Boundary flow change (from aquifer to the sea)	Lagoon flow change (from lagoon to the sea)
Scenario 1	-37%	-24%	33%	27%
Scenario 2	-32%	-21%	30%	30%

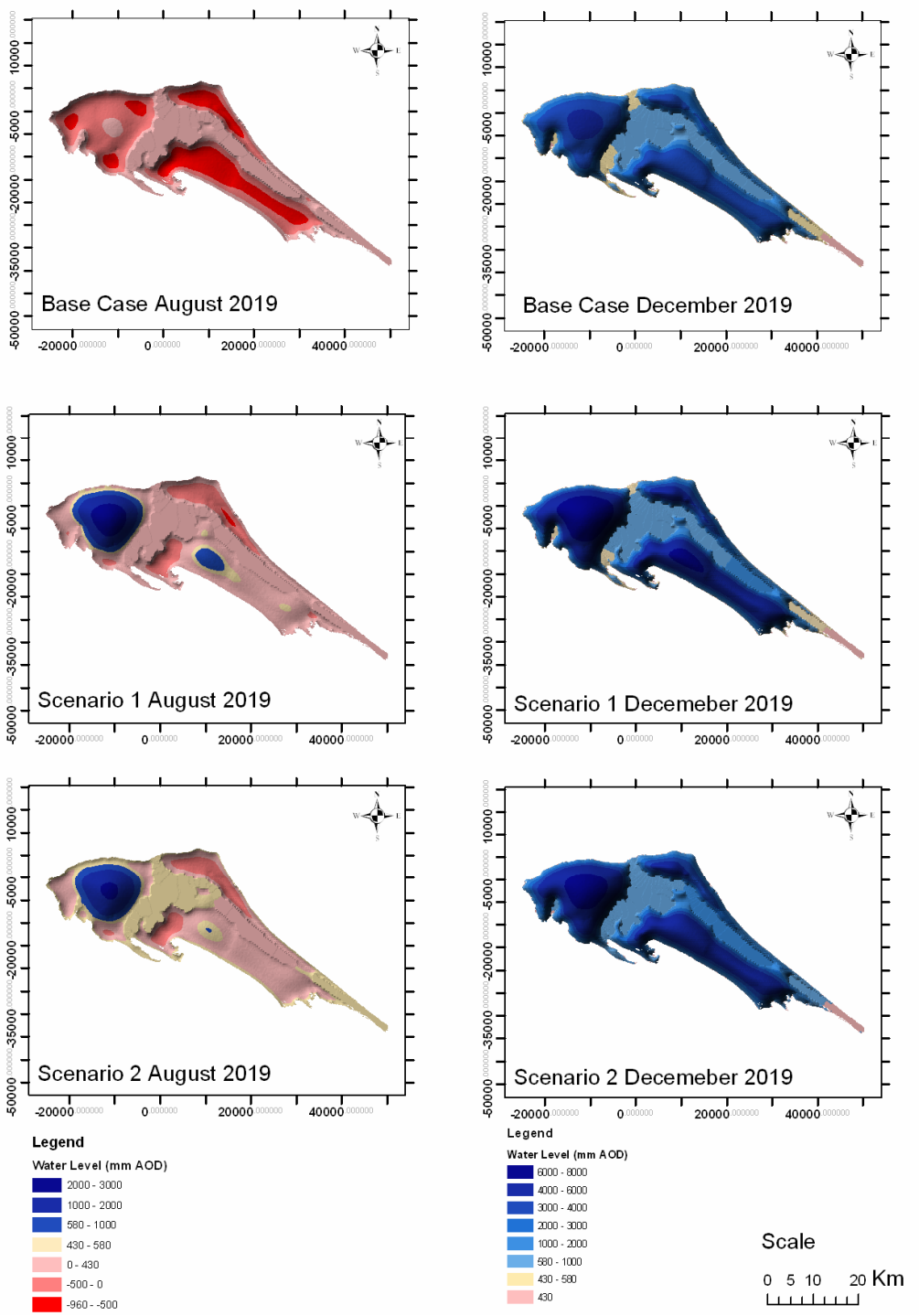


Figure 4. Simulated water levels for the base case and climate change scenarios.

5. CONCLUSIONS

The analysis of the groundwater levels highlighted the impact of climate change on the water resources of this system. Results showed that climate change can have a positive impact in the region. Changes in rainfall and temperature are the predominant factors, followed by the sea level rise and runoff. Although there is a significant improvement on the groundwater resources, fresh water in the wet season moves from the lagoons to the sea. This excess water can be used efficiently under a capable water management, to provide a cost-effective and valuable solution for the water problems encountered in the Jaffna Peninsula.

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LAKE VEGORITIS IN GREECE: A PARADIGM OF CLIMATE CHANGE AND MISMANAGEMENT EFFECTS ON ITS QUANTITY AND QUALITY CHARACTERISTICS

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Abstract: Lake Vegoritis is a very important natural impoundment in Greece. The lake has been under a continuous deficit in the water balance since the mid fifties. As a consequence about 65 % of its water has been lost, while the physical, ecological and water quality properties of the lake are influenced as well. The main cause of the quantitative problems is the lack of a proper management plan for its hydrological catchment and the human activities. Temperature and dissolved oxygen are two of the most important water quality parameters of the lake, while evaporation is the most important components of its water balance and is depended on meteorological parameters as air temperature, solar radiation and humidity that in turn are affected from climate change. The mismanagement of water resources and climate change has affected the water temperature distribution, stratification duration, dissolved oxygen distribution and biological characteristics of the lake in the same manner. Results of these variables in Vegoritis Lake for three different years (1981, 1993 and 2003) show that duration of stratification has extended, the dissolved oxygen depletion in hypolimnion became more intense and the evaporation during summer months was lower.

Key words: Climate change, dissolved oxygen, evaporation, temperature, Vegoritis Lake.

1. INTRODUCTION

The impacts of climate change on environment are one of the main components of human concern about the future changes. These impacts could influence many social-economic parts of development and the ecosystems (European Commission, 2005). Lakes are very sensitive systems and significant effects are expected.

A number of investigations have already examined the effects of global warming on water quantity (Arnell, 1999) but very little attention has been paid to the effects on water quality.

Water movement in lakes and impoundments can be affected by many factors such as wind, air temperature, topography, earth's rotation, air pressure and bottom friction (Lam and Schertzer, 1999). The climate change significantly affects many of these factors, which are related to the hydrodynamic circulation and the resulting lake responses. For example, the movement of warm and cold water masses in lakes is strongly affected by the factors of thermal stratification as air temperature, solar radiation and wind.

The extent and duration of the thermal stratification will in turn alter significantly the oxygen depletion and the primary production. Thus, a series of causal relationships need to be established through the meteorological forcing to the physical, chemical and biological responses.

Lake Vegoritis is a very important natural impoundment in Greece. It has been under a continuous deficit in the water balance for up to four decades (Papamichail and Antonopoulos, 1998). The water quality degradation from the inflow wastewaters of industrial, municipal and agriculture activities in its catchments (Antonopoulos et al., 1996) is a very serious problem. The problems of lake's water resources decrease are similar to climate change impacts on water resources systems. In this article the causes of water balance deficit in Vegoritis Lake are examined

and investigated under the climate change prospective and the subsequent effects on quantitative, qualitative and biological components of the lake.

2. CLIMATE CHANGE AND LAKE RESPONSE

The local climate affects the hydrodynamics of a lake, the water quality and the ecosystem components.

Increasing water temperatures change hydrodynamics in lakes, expand the thermal stratification period, and deepen the thermocline (e.g. Hassen et al., 1998; Blenckner et al., 2002). These shifts then increase nutrient release from sediments and lead to an alteration in nutrient circulation (Malmaeusa et al., 2006). As a result, climate variability is expected to have a significant influence on lake ecosystems. Water temperature is a key parameter in most biological systems, directly influencing water chemistry, biochemical reactions and growth/death of biota.

Climate changes affect also the seasonal variation and the hydrology of the lake and as a consequence the surface level variation during the year and for long time period, the inflow and outflow and the redistribution of inflow from streams. Changes of hydrological components in lake's catchments affect significantly the hydrodynamics of a lake.

Due to their large water volume, lakes have high heat capacity which in conjunction with the exchanges of thermal energy between the water and the atmosphere results in modification of the local climate. Thus, during spring and summer, the lakes hold cold the surrounding air temperature, while during autumn and winter they retain a warmer environment. The high heat capacity is the reason of long time periods without ice cover in deep lakes.

The effect of global warming on the aquatic ecosystems of a specific lake depends on numerous factors, such as the geological and geographical features of watershed, lake dimensions, water quality, sediment quality, and original trophic state.

Increased concern over the potential impacts of climate change on lake environments have prompted interest in evaluation of water temperature using numerical models. The response of lake water temperature to climate change can be assessed using physically based models that simulate seasonal thermal evolution in response to atmospheric forcing (Stefan et al., 1993, 1998; Hondozo and Stefan, 1993; Hostetler and Small, 1999; Fang et al., 1999). These models in combination with Global Circulation Models (GCMs) provide results of lake temperature, depth and duration of mixing for long-term responses to climate change. Other more integrated models of water quality and ecological components of lake ecosystems can describe, besides temperature, the quality, as well as the biological parameters of lakes such as phytoplankton, zooplankton and fishery (Stefan et al., 1993; Stefan et al., 1998; Antonopoulos and Gianniou, 2003).

Primary production is controlled by a combination of temperature, light, and nutrients. When nutrient availability is high, overall production varies with water temperature. When it is low, overall production is set by the nutrient level and is relatively insensitive to changes in temperature (Downing et al. 1990). It is likely that rates of primary production will be altered by climate change and this may result in significant consequences for aquatic ecosystems and the human communities that use them. On the one hand, decreases in primary production will reduce food availability at the base of the food web, ultimately causing reductions in productivity of fish at the top of the food web. On the other hand, excessive increases in primary production will produce eutrophic (highly productive) conditions, exemplified by degraded water quality and noxious blue green algal blooms.

3. LAKE VEGORITIS DESCRIPTION

Lake Vegoritis is a very important natural impoundment in Greece (Figure 1). It is located in the northern part of Greece at an altitude of 510 m (nowadays) and a northern latitude of 40^{o} 47' (Antonopoulos et al. 1996). The lake occupies the deeper part of the hydrological basin, an area of 1853 km^2, where other three lakes (Zazari, Chimaditis and Petron) are also located (Figure 1). The

volume of the lake has been decreasing since the mid of fifties. About 65% of the lake water has been lost (Papamichail and Antonopoulos, 1998). In 1981, the surface elevation, the volume and the surface area of the lake were 523.5 m, 1575×10^6 m^3, and 52 km^2, respectively, and in 1993 they were 513.5 m, 810.4×10^6 m^3, 39.1 km^2, respectively. In 1981, the maximum depth of the lake was 58 m and in 1993 it was 48 m In 2003 the surface elevation decrease at 509 m slightly below 1993 elevation. In Figure 2 the variation of surface elevation and the decrease trend from 1960 is presented. The lake appears to have great quantitative, qualitative and ecological problems. The trophic status of the lake has been changed from oligotrophic at the beginning of the 80s to oligo-mesotrophic later during the 90s. The presence of free ammonia, pesticides and industrial wastes has created an increased toxicity in the lake water having adverse effect on fish (Antonopoulos et al, 1996).

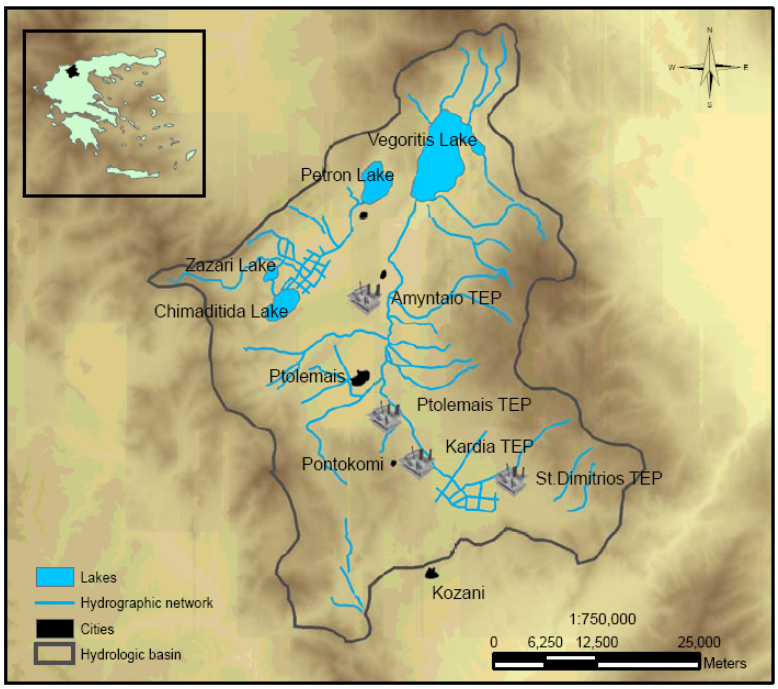


Figure 1. Lake Vegoritis catchment with hydrographic net and the industrial activities (TEP, Thermoelectrical Power Plant).

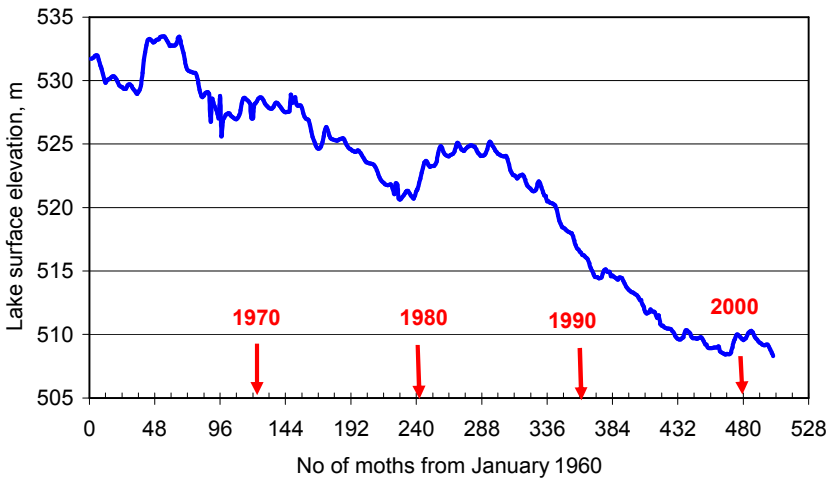


Figure 2. Variation of Lake Vegoritis surface elevation from January 1960.

The reasons of lake volume decrease and the surface elevation decline could be attributed to mismanagement of water resources in the hydrological basin and the effects of climate changes. The foremost paradigm of water resources mismanagement is the huge amounts of water discharged out of the Vegoritis Lake basin to Nisiou lake basin, through the tunnel of Arnissa, between the years of 1956 and 1985 to satisfy hydroelecrical demand for Agras power station. Regarding the impact of climate change on water quantity and quality problems of the lake, there are proofs that the microclimate of the region was affected and changed due to the over usage of natural resources (water pumping from the lake and the groundwater, changes in hydrodynamic conditions of surface and groundwater systems) and the high industrial development in the area (coal based thermoelectric power plants, other industries) and population movement.

4. LOCAL CLIMATE IN VEGORITIS BASIN

The local climate of Vegoritis lake basin is semi-dry, Mediterranean, with alterations between warm-dry and cold-wet periods (Aschonitis et al., 2006). The mean annual precipitation ranges between 450 and 600 mm. The mean relative humidity during cold-wet period ranges from 70 to 80 %, while during warm-dry period is from 50 to 60 %. The maximum monthly evaporation is observed during the months of July and August and is from 120 to 180 mm. The temperature during the winter months falls under 0^{o}C for 50-70% of winder days, while at some days of summer it could exceed 40 ^{o}C.

The precipitation data of the period 1951-2001, show that the mean annual rain depth is 528 mm, with the maximum mean monthly depth at November and the minimum at August. From the analysis of data, two distinguish periods were observed for 1951-1975 and 1976-2001. The mean annual precipitation of the 1st period is 574 mm and of the 2nd is 482 mm, showing a decrease of 16 % (Aschonitis et al., 2006).

From the analysis of air temperature data for the period of 1951-1990, the mean annual temperature is 12 ^{o}C, with maximum monthly temperature at July and minimum at January. The mean temperature of the two subperiods (1951-1975 and 1976-1990) is 12.6 ^{o}C and 11.1 ^{o}C, respectively.

The industrial development in the lake basin (4 thermoelectrical power stations and the extended surface coal mining areas), the topographic characteristic of the area, the air pollution from industrial activities, the changes in land use (decrease in soil covered with plants) and in climate conditions affect the amount of solar radiation, as well as the fraction of solar radiation absorbed by the soil and reflected by the soil surface. The result is a decrease in mean annual temperature at the layer over the soil surface and a decrease in temperature range between day and night time.

5. TEMPERATURE, DISSOLVED OXYGEN AND EVAPORATION IN VEGORITIS LAKE.

Water temperature, dissolved oxygen and evaporation are characteristic variables sensitive to climate change. They are affecting many of the physical, chemical and biological parameters of lakes. Water temperature is a key parameter for many biological processes, as it influences the chemistry of water, the biological reactions and the growth/decay processes of organisms. The availability of suitable dissolved oxygen (D.O.) concentrations controls the presence of freshwater fish species and fish populations in lakes. Fish have D.O. concentration requirements, below which mortality is more likely to occur or growth is impaired (U.S.EPA, 1976; Fang and Stefan, 2000). The D.O. thresholds of many species of fresh-water fishes lie between 1.0 and 2.0 ppm.

Evaporation is one of the main components in both the energy and water budgets of lakes and a primary process of water loss for most of them. Accurate estimates of lake evaporation are necessary for water and energy budget studies, lake level forecasts, water quality surveys, water

management and planning of hydraulic constructions. Evaporation process depends on the availability of thermal energy and the vapor pressure deficit between the evaporating surface and the overlying air, which in turn depend on meteorological factors such as temperature, relative humidity and wind speed. Besides climate, lake characteristics such as its size, shape, depth, water quality and circulation, even its location can affect the rate of evaporation.

Water temperature of lakes and its vertical distribution exhibits variations throughout the year. From late spring to mid autumn lakes of temperate regions are thermally stratified. Thermal stratification affects the density of water and inhibits its circulation. The prospective increase of temperature due to climate change will alter the hydrodynamics of lakes and increase the time period of thermal stratification and the depth of thermocline. These changes in turn will have an effect on the release of nutrients from the lake sediments.

A one-dimensional temperature stratification and dissolved oxygen prediction model named QALAKE-DOT was used to simulate the annual temperature and dissolved oxygen distribution of Lake Vegoritis in different years. The model is based on the thermal and oxygen diffusion equations, which are solved using the Galerkin finite element method (Antonopoulos and Gianniou, 2003). The one-dimensional vertical transport of heat is described by

$$A\frac{\partial T}{\partial t}=\frac{\partial}{\partial z}\left(AK_z\frac{\partial T}{\partial z}\right)-\frac{1}{\rho c_p}\frac{\partial Aq}{\partial z} \qquad (1)$$

where T(z,t) is the water temperature (^oC) as a function of depth (z) and time (t), A(z) is the horizontal area as a function of depth (m^2), q(z) is the internal distribution of heat sources due to solar radiation absorption inside the water column (cal/m^2-day), K_z ($=D_o+E$) is the coefficient of diffusivity (m^2/day), D_o is the molecular diffusivity (m^2/day), E(z,t) is the turbulent diffusivity (m^2/day), ρ is the water density (g/cm^3) and c_p is the specific heat of water ($cal/^oC$-g).

The one-dimensional vertical transport of dissolved oxygen in lakes is described by

$$\frac{\partial C}{\partial t}=\frac{1}{A}\frac{\partial}{\partial z}\left(AK_z\frac{\partial C}{\partial z}\right)-\frac{u}{A}\frac{\partial C}{\partial z}+P-R-S \qquad (2)$$

in which C is the concentration of dissolved oxygen (mg/L), A is the horizontal area of the lake at the depth z (m^2), u is the vertical velocity of water (m/day), P is the oxygen production by plant photosynthesis, R is the oxygen losses by respiration of lake organisms and S is the oxygen consumption by BOD (biochemical oxygen demand) and SOD (sediment oxygen demand) (mg/m^3-day),. The oxygen production by plant photosynthesis (P) is a function of depth and time and is calculated as the product of the maximum specific photosynthetic oxygen production rate (mg O_2/μg Chla-day), the chlorophyll-a concentration (μg/L), and the maximum value of algae growth.

The following mass transfer (aerodynamic) equation, derived under the conditions of Vegoritis Lake, was used for the estimation of evaporation from its surface

$$E=0.1431u_2(e_{sw}-e_d) \qquad (3)$$

where u_2 is the wind speed measured at some specific height above the water surface, e_{sw} is the saturation vapor pressure at the temperature of the water surface (mbar) and e_d is the air vapor pressure above water surface (mbar).

The water temperature and dissolved oxygen concentrations of Lake Vegoritis were simulated for the climate conditions of 1981, 1993 and 2003. The results of daily water temperature and D.O. simulations obtained by the model were tested against water temperature and D.O. measurements during the years 1981 and 1993 (Antonopoulos and Gianniou, 2003).

In the current study, these two essential water quality parameters of lakes are examined as they are affected by the climate changes and in turn they have a profound effect on the chemical and biological processes of a lake. As an example, water surface temperature controls the evaporation rate and the exchanges of energy through the atmosphere-water interface, as well as the temperature of the hypolimion, where important biological processes take place. Dissolved oxygen concentrations depend on temperature as the D.O. saturation concentration is reversely proportional to temperature. In turn D.O. concentrations affect the processes of aerobic and anaerobic mineralization of dead organic matter in hypolimnion and consequently the survival of fishes and

other aquatic organisms. The increase of lake's water temperature, either as a result of its volume decrease or due to climate change will have significant impacts on the biological, chemical and quantitative characteristics of the lake.

According to field data and simulations results for the three yearly examinations, the lake presents the characteristic annual temperature cycle. Temperature stratification was observed from the first days of June at the depth of 6 m. The thermocline was deeper at the end of summer and it remained so until October. The fall overturn occurred between the end of October and the beginning of November. The maximum depth of the thermocline exceeded 10 m in all the years of the simulation. It means that during 1993 and 2003 the hypolimnetic stratum decreased about 10 m and 13 m, respectively, a fact that has caused significant effects on the biological activity and ecology of this layer. Figure 3 shows the variation of thermocline depth from the beginning of temperature stratification until its destruction (Antonopoulos and Gianniou, 2002). The smaller layer of hypolimnion means that the dead organic matter from aquatic organisms which grow in the epilimnion (euphotic zone) and the pollutants which accumulate in this strata, are under smaller dilution. The decrease of the hypolimnetic layer is clearly presented in Figure 4. In this figure the temperature distributions in August 4, 1981, in July 20, 1993 and in July 20, 2003 are presented.

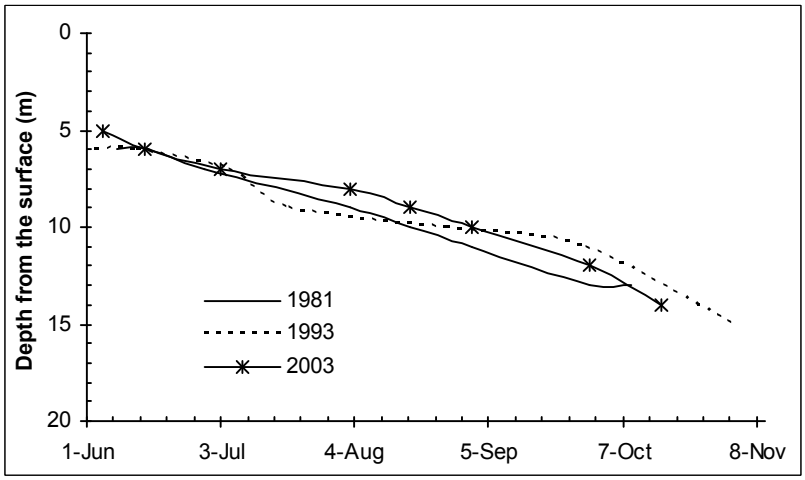


Figure 3. Variation of thermocline depth of Vegoritis Lake during the years of 1981, 1993 and 2003.

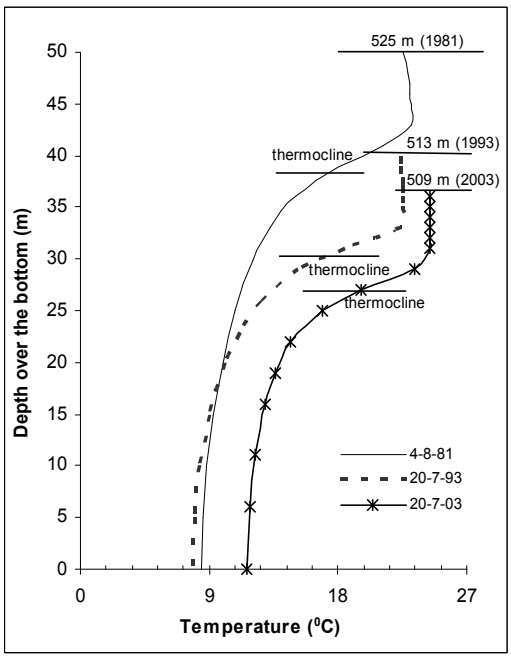


Figure 4. Comparison between the temperature distribution in Vegoritis Lake of August 4, 1981, July 20, 1993 and July 20, 2003.

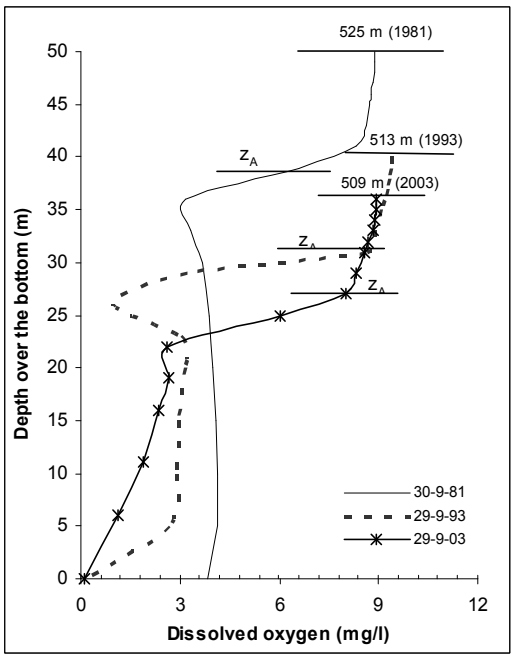


Figure 5. Comparison between the dissolved oxygen distribution in Vegoritis Lake of August 4, 1981, July 20, 1993 and July 20, 2003.

Regarding the dissolved oxygen simulation the homogeneous D.O. profiles near saturation at the first months of the year changed later in the summer. The D.O. concentration profiles during the summer months and the first part of autumn show a similarity to temperature distribution. In the euphotic zone D.O. concentrations are uniform and near saturation. The formulation of metalimnetic oxygen minimum started from August in 1981 and from July in 1993 and 2003. It is more distinguished during 1993 and 2003 with values of D.O. less than 4 mg/L, which for a short period approach zero. The hypolimnion is cut off from the high epilimnetic oxygen concentrations and only minor vertical exchange occurs because of the stratification. This situation prevails until November, when stratification becomes weaker and wind mixing is more effective.

In Figure 5 the distribution of dissolved oxygen in Vegoritis Lake at the end of September of 1981, 1993 and 2003 is presented. The depth of euphotic zone z_A slightly changed, approximately at 10% for 1993 and 2003 in comparison to 1981 (11.5 and 9.2 m respectively), whereas the metalimnetic depth of oxygen depletion is observed at the same level from lake surface. There is a significant decrease though in the years 1993 and 2003 of the non-euphotic zone of about 10-14 m, which is related to the decrease in the thickness of the hypolimnion layer.

Evaporation rates for Vegoritis Lake were estimated using the mathematical model QUALAKE-DOT and the mass transfer equation (3) for the years 1981, 1993 and 2003. In Figure 6 the monthly simulated evaporation rates for Vegoritis Lake for these three years are presented (Gianniou and Antonopoulos, 2007a and b). Mean annual evaporation rates are 2.20 mm/day, 2.10 mm/day and 2.0 mm/day for the years 1981, 1993 and 2003, respectively.

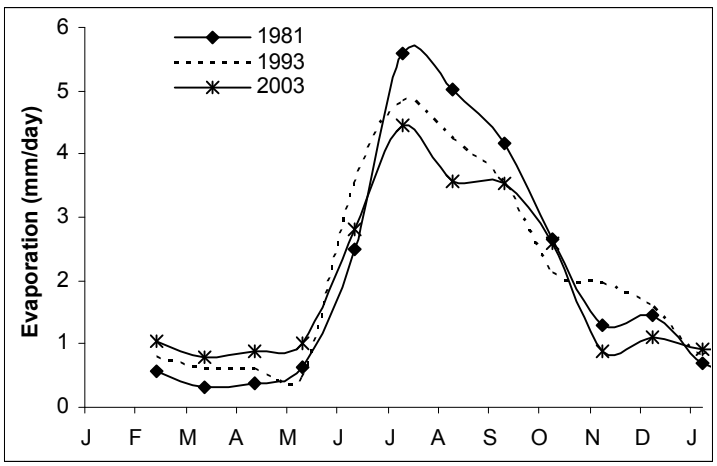


Figure 6. Monthly variation of evaporation values for Lake Vegoritis during the years 1981, 1993 and 2003

CONCLUSIONS

Lake Vegoritis has been under a continuous water balance deficit since the decade of 50's. The mismanagement of water resources in the hydrological basin has led in water degradation of the lake. The industrial activities of coal mining and the 4 thermoelectical power station in lake's basin have affected the microclimate. The mean annual temperature of the last 25 years decreased by 1.1 ^{o}C in relation to that of the past 25 years, while the mean annual precipitation decreased from 567 to 440 mm respectively. Many problems which were not solved with the appropriate water management might accelerate and become more intense with the climate change.

The volume decrease has affected significantly the cycle of thermal stratification, which is related to the lake circulation, the heat balance of the lake and the dissolved oxygen distribution in the lake. The oxygen decrease in the hypolimnion is more intense during the last years and it is due to the decrease in lake's volume and the increase of pollution.

The climate change will increase and intense the problems which are related to temperature, evaporation, water circulation, oxygen, productivity and biology of the lake. It is required to develop a water resources management plan as soon as possible and to initiate intensive monitoring of all of the climate change related metrological parameters and the monitoring of quantity, quality and biological parameters of the lake.

The protection of Vegoritis Lake is related to the activities and pollution sources in lake's hydrological basin. The agriculture and industrial development in the basin should be affiliated to the sustainability of the Vegoritis Lake itself and to the resources in the hydrological basin and the climate change impacts.

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UNCERTAINTY OF CLIMATE CHANGE IMPACT ON THE FLOOD REGIME, CASE STUDY AIDOGHMOUSH BASIN, EAST AZARBAIJAN, IRAN

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Abstract: This research work is aimed to describe a methodology for quantifying uncertainties involved in climate change impact on flood regime studies due to discrepancies in AOGCMs (Atmosphere-Ocean General Circulation Model) simulations of regional and local climate parameters; applied on Aidoghmoush basin in north-west of Iran.. This research work excludes uncertainties due to parameters and concepts of the rainfall-runoff model, climate sensitivity, emission scenarios, and downscaling techniques. Monthly temperature and precipitation change scenarios of Aidoghmoush basin were constructed from 7 AOGCM models (CCSR/NIES, CGCM2, CSIRO MK2, ECHAM4/OPIYC3, GFDL R30, HadCM3 and NCAR DOE PCM) using baseline period (1971-2000) data and future period 2040-2069 (2050s) under the SRES emission scenario, namely A2. Then, these climate change scenarios were downscaled spatially to the basin using original grid box information of each AOGCMs. Results showed that the temperature increase and the precipitation vary in 2050s relative to the baseline period. Monthly probability distribution functions (pdf) of temperature and precipitation change scenarios in 2050s were constructed by a weighting method (comparing 30-year monthly average of observed and AOGCMs temperature and precipitation data in the baseline period). A daily rainfall-runoff conceptual model (IHACRES) was calibrated for the basin. Carrying out Monte Carlo method, 2000 samples of temperature and precipitation were sampled from corresponding pdfs. Then 2000 temperature and precipitation time series for 2050s were constructed using change factor method. These time series were introduced to IHACRES. By this, 2000 30-year time series of daily runoff were simulated in 2050s and 30 maximum annual flood of each time series were extracted. Appropriate theoretical probability distribution functions were fitted to each series as well as maximum annual flood of baseline. Finally changes in two flood indicators were discussed: Severity of flood events by comparing the magnitude of flood using fixed return periods and frequency of flood events by comparing the return period of a flood using fixed magnitude. Results show that for the return period less than 50-year, the probability of decrease in magnitude of future flood event relative to the baseline is more than the probability of increase. For the return period more than 50-year both situations have nearly same probability to occur. Analyzing the flood frequencies show that in the future, floods less than 20 m^3/s have less chance to occur relative to the baseline. On the other hand for flood magnitudes more than 60 m^3/s this chance will increase. Finally it can be concluded that over-reliance on a single or very few AOGCMs climate scenarios could lead to inappropriate flood magnitude or frequency. Therefore for providing scientifically based advice to decision makers it is essential that climate change impact studies consider a range of weighted climate scenarios derived from different AOGCMs. The methodology and the final results of this research can play the base role for other disciplines such as dam operation, dam break risks, agricultural damage risks and adaptation policies that are greatly affected by changes in flood regime.

Keywords: Climate Change, Flood, IHACRES, Uncertainty

1. INTRODUCTION

It is common knowledge that intensified human activity and growing population have changed the climate and the land biosphere. The linear warming trend over the last 50 years (0.13°C per decade) is nearly twice that for the last 100 years. Other climatological parameters, such as precipitation, cloudiness and evaporation have been investigated which show strongly varying trends on both global and regional scales (IPCC 2007). Anthropogenic climate change not only has been affecting climate variables but also extreme events (e.g. drought, flood …) although it is not widely recognized- (Robson 2003). Trends in fluvial flooding are more difficult to detect, as changes in factors such as land use, reservoirs, drainage or flood alleviation schemes will impact on the flood regime in addition to changes due to the climate (Prudhomme et al. 2003).

In the future, hydrologic cycle will be most severely affected by climate change. Any changes in the hydrologic cycle will affect flood to such an extent that water management adaptation measures will very likely be brought in (Mirza, 2001). To assess the likely magnitude of climate change in the future and its resulting impacts on flood, the scientists rely on simulations from general circulation models (GCMs), driven by plausible scenarios of future emissions from human activities (Carter

2007). Although GCMs are the most credible tools to assess the climate change impacts, they are subjected to some source of uncertainties. Examples include poorly understood climate physics and computing limitations, both of which limit the accuracy of GCM climate change simulations.

A broad range of literature exists regarding managing uncertainty in climate change impact assessments, often using Monte-Carlo approach (New and Hulme 2000; Katz 2002; Prudhomme et al. 2003). Such studies have not generally weight GCMs, and have imagined the uniform prior distribution for all GCMs. This can affect the final results of impact assessment studies.

This study demonstrates one attempt to introduce a procedure to assess the impact of climate change on flood using a weighting method. Section 2 of this paper, describes the various input data sets and the characteristic of the study area. In section 3 the methodology of the research includes rainfall runoff, GCM-Weighting method and flood frequency analysis will be presented. This is followed by an examination of the weighted scenarios on the regime of flood (Section 4). We conclude in section 5 with a discussion of the advantages of this approach and the implications for climate change impact and adaptation assessments.

2. STUDY AREA AND DATA SETS

The river Aidoghmoush with 80 km length is the largest river in the Aidoghmoush Basin (drainage area _ 1802 km^2) (Fig. 1). It originates in the north- west of Iran and traverses the east. The hydrometer gauging station Motorkhaneh, considered in this study, is located in the end east of Aidoghmoush basin. The mean discharge at this gauging station is 170 MCM/year with the coefficient of variation of 65%. The seasonal distribution of flow relative to the total annual flow is 23.4, 65, 1.4 and 10.4 for winter, spring, summer and autumn respectively. The daily precipitation (from 10 stations), temperature (from 2 stations) and daily peak discharge (hydrometer gauging station Motorkhaneh) are available for the baseline period 1971–2000. These data were obtained from the Meteorological Institute and ministry of energy of Iran.

Seven GCM configurations are considered in this study; CCSR: Japanese Centre for Climate Research Studies model, CGCM2: Canadian Centre for Climate Modeling and Analysis GCM, CSIRO-Mk2: Australian Commonwealth Scientific and Industrial Research Organisation, ECHAM4 – OPYC3: German Climate Research Centre, European Centre/Hamburg, GFDL-R30 : US Geophysical Fluid Dynamics Laboratory, HadCM3: UK Hadley Centre for Climate Prediction and Research Coupled and NCAR-DOE–PCM: US National Centre for Atmospheric Research model, DOE version (Carter 2007). For each configuration two runs were considered, one for the control period 1971–2000 and one for the period 2040-2069, based on the SRES-scenario A2.

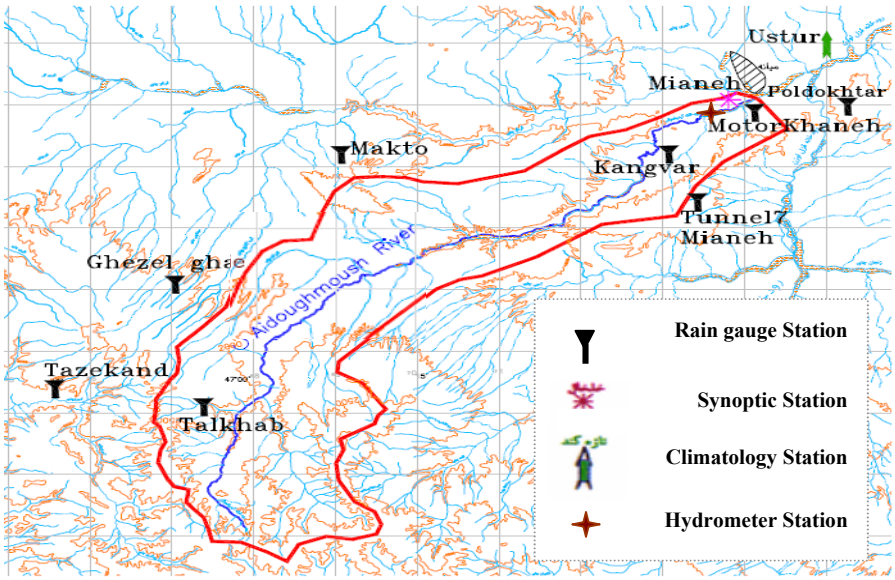


Figure1, Location of study catchment and stations

3. METHODOLOGY

In order to assess the impact of climate change uncertainties on flood regime, seven major steps were followed. First, a conceptual relationship between rainfall and runoff was determined. Second, climate change scenarios were constructed for the Aidoghmoush basin in 2040-2069 using the results of seven GCMs under A2 scenario. Third, discrete temperature and precipitation pdfs were constructed using a GCM- weighting method. Fourth, Monte Carlo method was carried out to sample from the pdfs. Fifth the climate change scenario samples were applied to rainfall-runoff model in order to determine the magnitude of changes in runoff at the Motorkhaneh gauging station. Sixth, 30 annual maximum flood were retrieved from 30 year daily runoff and the return period of each flood was determined. Seventh, steps fourth to sixth were iterated 2000 times. In the following the methodology of each steps are described.

3-1 Rainfall-Runoff Model Description

The IHACRES module structure consists of a non-linear loss module, which converts observed rainfall to effective rainfall or rainfall excess, and a linear streamflow routing module, which extends the concept from unit hydrograph theory that the relationship between rainfall excess and total streamflow is conservative and linear (Jakeman and Hornberger 1993).

The non-linear loss module allows one to take into account the effect of antecedent weather conditions on the current status of soil wetness index s_k and vegetation conditions, and evapotranspiration effects. Here the effective rainfall u_k is calculated from the measured precipitation r_k and temperature t_k by the recursive relations:

$$s_k = \frac{r_k}{c} + (1 - \frac{1}{\tau_w(t_k)})s_{k-1} \quad (1)$$

$$u_k = r_k(s_k + s_{k-1})/2 \quad (2)$$

$$\tau_w(t_k) = \tau_w \exp(20f - t_k f) \quad (3)$$

The constant c is calculated so that the volume of excess rainfall is equal to that of the total streamflow for the period over which the model is calibrated, τ_w and f are parameters which should be optimised, τ_w is a time constant reflecting the rate of drying (in days) of the catchment at 20°c and f is a factor which modulates this rate as temperature varies.

The linear module allows any configuration of stores in parallel or series. In the two-store configuration, at time step k, quickflow, $x_k^{(q)}$, and slowflow, $x_k^{(s)}$,combine additively to yield streamflow (discharge), q_k :

$$q_k = x_k^{(q)} + x_k^{(s)} \quad (4)$$

with

$$x_k^q = -\alpha_q x_{k-1}^{(q)} + \beta_q u_k \quad (5)$$

$$x_k^s = -\alpha_q x_{k-1}^{(s)} + \beta_s u_k \quad (6)$$

where U_k is the effective rainfall. The parameters α_q and α_s can be expressed as time constants for the quick and slow flow stores, respectively:

$$\tau_q = -\Delta / \ln(-\alpha_q) \quad (4)$$

$$\tau_s = -\Delta / \ln(-\alpha_s) \quad (5)$$

where Δ is the time step (daily here). In catchments which are modeled with only one store only Equations (2) and (4) are relevant.

Model calibration was performed in the basin with daily area averaged precipitation, temperature and runoff time series during the period of observation, 1971-2000.

3-2 Construction of climate change scenarios

A simple downscaling approach, the 'change fields' procedure was applied to derive the monthly climate change scenarios for the basin. The climate change scenarios were obtained by computing the differences (or ratio) between the averages of the AOGCMs dataset for the future periods (2040-2069) and the corresponding averages of the models simulated for the baseline period (here, 1971-2000). The changes for temperature are usually presented based on the differences (e.g. 2040- 2069 minus 1971-2000), while for precipitation change, ratios are commonly used (e.g. 2040-2069 divided by 1971-2000) (Carter 2007; Diaz-Nieto and Wilby 2005; Wilby and Harris 2006).

3-3 Weighting GCMs

Single forecasts of the climate response to increasing greenhouse gas levels, are far more useful to policy makers when they are accompanied by some measure of the associated uncertainty [Schneider, 2001]. In ensemble forecasting, it is customary to take the arithmetic ensemble mean as a prediction quantity and in most cases this already provides a better skill than any of the ensemble members alone. The term "Multi-model ensemble" indicates a set of simulations from various climate models with different structures. The use of such ensembles allows to asses the structural uncertainty included in model output. The simplest way of presenting climate change scenarios from different GCMs is with no weighting estimates. On the other hand by this approach all models take account the same weights. However, this does not take any account of differences in GCM model performance for the present day. Ensemble weights are calculated by comparing model output for the present day with observed data. Accordingly we will evaluate the sensitivity of the expose unit to a multi-model probabilistic prediction by using the following equation:

$$R_i = \frac{\frac{1}{B_{x,i}}}{\sum_{i=1}^{N} \frac{1}{B_{x,i}}} \tag{6}$$

where R_i is a measure of the model reliability in terms of the model bias in simulating present-day temperature/precipitation and $B_{x,j}$ is the model bias that defined as the difference between simulated and observed monthly mean temperature/precipitation for the baseline period (here 1971-2000). It is obvious that the higher the bias, the lower the model reliability. On the other hand, models with more ability are assigned higher weights and hence given more emphasis.

3-4 Flood frequency analysis

In flood frequency analysis one should determine a relationship between magnitude and return period of the flood of the river. Accordingly, different procedures exist; AM (Annual Maximum flood) which appropriate probability distribution function should be fitted to Annual Maximum flood series of a river, and POT (Peak Over Threshold) that a thresholds is assumed for maximum flood and then runoffs with higher magnitude than threshold extracted from the runoff series, finally an appropriate distribution function will be fitted to the extracted data (Kite 1977). Here we used the AM procedure for flood frequency analysis.

4- RESULTS

To assess the impact of uncertainties due to climate change on flood regime of Aidoghmoush basin, we at first calibrated the IHACRES model. In this regard the area-averaged daily temperature and precipitation data of the basin and the Motorkhaneh daily runoff data over 1971-2000 were used. Different calibration and verification periods were tested using three criteria; correlation coefficient (r), Root Mean Square Error (RMSE) and Mean Absolutely Error (MAE). Figures 2 shows the best result which obtained from calibration and verification of IHACRES. As shown in the figure, although in some cases the peak runoff can not be simulated well by the IHACRES, the performance of the model in simulating the daily runoff of the river is desirable. Finally, as we needed a model that can be iterated a lot of times; we developed a FORTRAN code using equation 1 to 5 with calibrated coefficient from IHACRES.

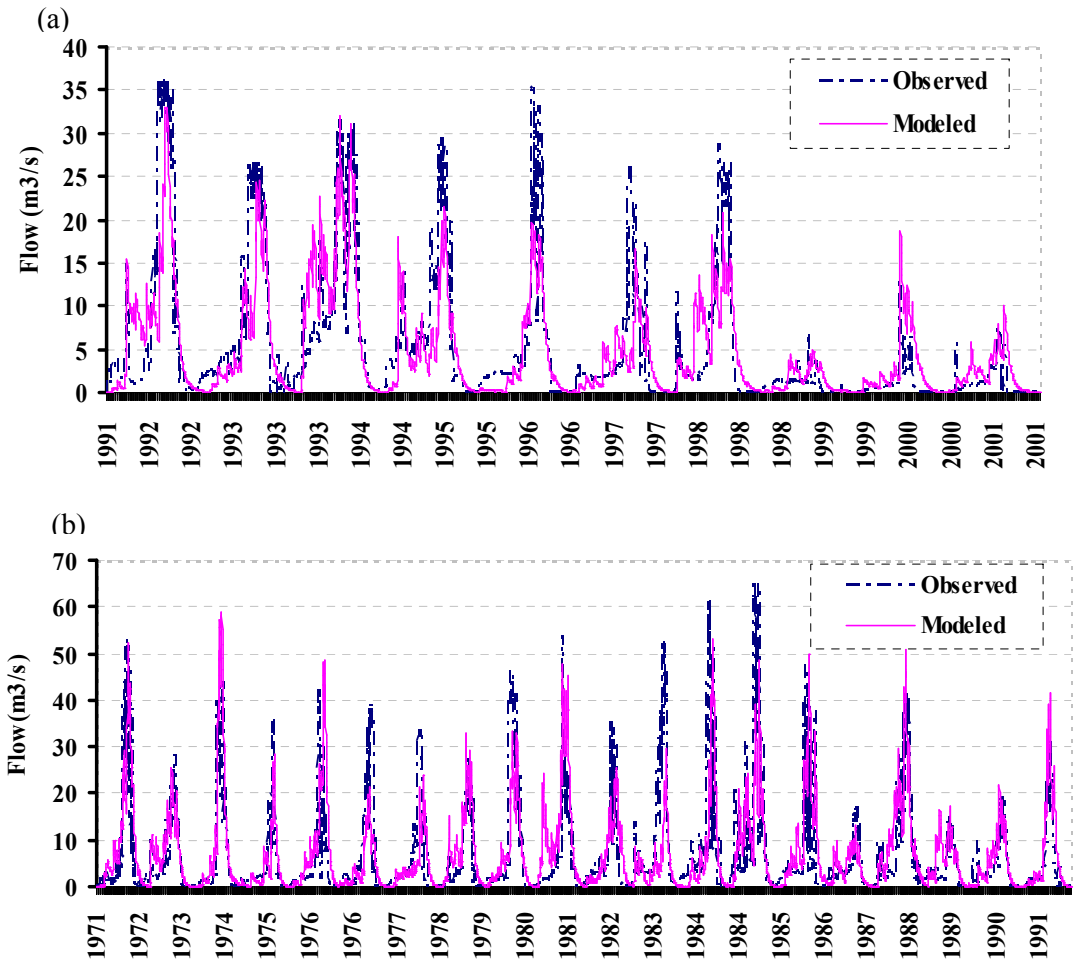


Fig 2, Observed and modeled runoff for (a) calibration period (r=0.79, RMSE=4.62, MAE=1.66) and (b) verification period (r=0.66, RMSE=7.84, MAE=2.1)

To simulate the flood of the Aidoghmoush basin in the future, three steps were followed. Constructing climate change scenarios of the basin, constructing pdfs of climate change scenarios, sampling from pdfs, and introducing samples to IHACRES. Accordingly monthly temperature and precipitation change scenarios of the basin were constructed for the period 2040-2069 from 7 AOGCM models under A2 scenario. Results showed that (not shown here) there is good agreement between AOGCMs in simulating the temperature, since all models estimate 1 to 6°c temperature increase in the future relative to the baseline. For precipitation this agreement was poor and both decrease and increase changes relative to the baseline were simulated by the models. On the other hand most of the models show increase in precipitation for the seasons; winter and autumn (should be noted that the pattern of the precipitation of the basin in the baseline is winterly).

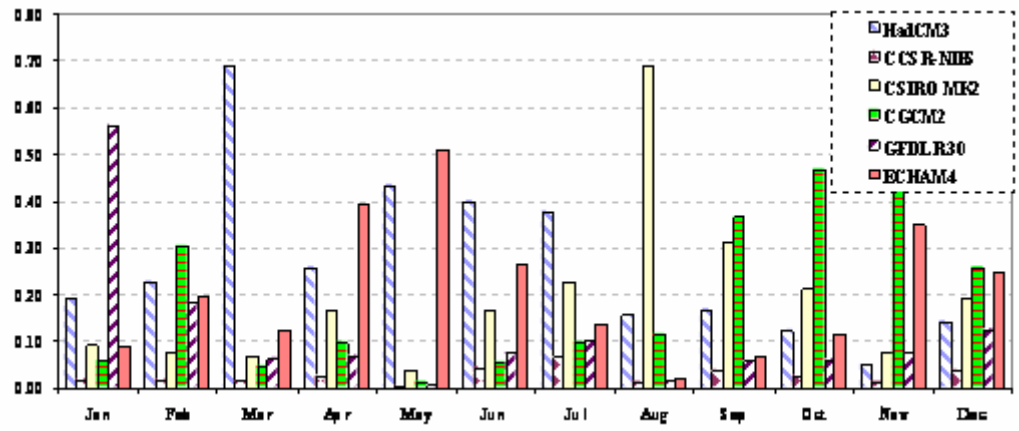
To construct the monthly pdfs of climate change scenarios of the basin, at first daily temperature and precipitation data of 7 GCMs over the period 1971-2000 were downscaled to the study area using the information of the main grid box of each model. Then 30-year mean monthly modeled data were compared with observed area averaged temperature and precipitation data of the basin in same period. The performance

of each GCM in simulating the temperature and precipitation of the basin was evaluated using correlation coefficient (r), Root Mean Square Error (RMSE) and Mean Absolutely Error (MAE) (Table 1). As shown, the ability of GCMs to simulate the temperature is better than precipitation. On the other hand NCAR DOE PCM has very low performance in simulating the precipitation of the basin, so we omit this model for the next analyses. Using equation 6, the monthly weights of each GCM were calculated (fig 3). As shown in figure 3, each model takes into account different weights in each month. For example, HadCM3 takes into account the highest weight in simulating temperature in March (November for precipitation), while in November it is one of the lowest one (January for precipitation).

Table 1, Performance of GCMs in simulating observed climate variable of the basin

Climate Variable	Precipitation			Tmperature		
Performance / Model	MAE (mm)	RMSE (mm)	r (%)	MAE (oC)	RMSE (oC)	r (%)
HadCM3	6.82	8.67	91.0	1.00	1.18	99.6
CCSR-NIES	14.30	19.12	67.0	7.84	7.89	99.6
CSIRO MK2	10.93	14.60	77.0	1.25	1.34	99.1
CGCM2	36.12	52.64	77.0	1.94	2.50	97.9
GFDL R30	10.67	12.30	97.0	3.14	3.74	99.4
NCAR DOE PCM	23.00	28.91	16.0	3.99	4.20	99.4
ECHAM4-OPYC3	16.79	20.24	89.0	1.72	2.37	98.9

(a)



(b)

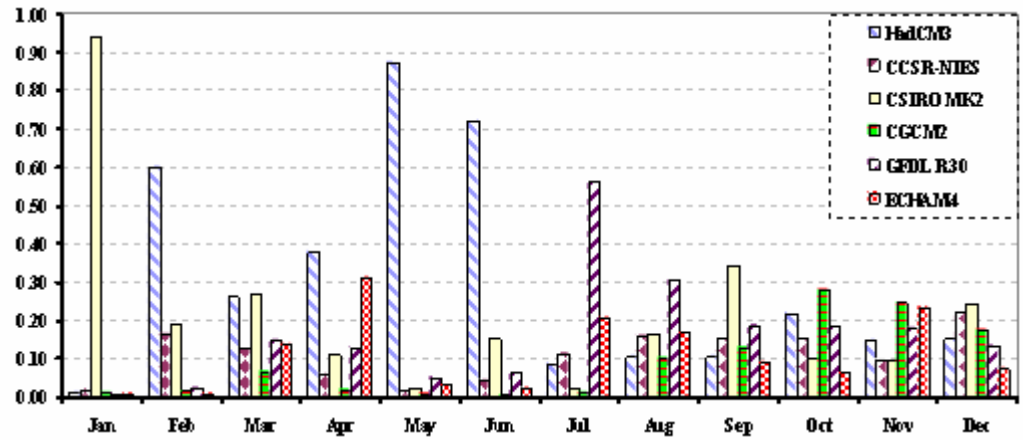


Fig 3, Monthly weights of GCMs in simulating (a) temperature and (b) precipitation of Aidoghmoush *basin*

To assess the flood of the Aidoghmoush basin in 2040-2069, discrete monthly pdfs of temperature and precipitation change scenarios of the basin in 2040-2069 were constructed using each GCM weights. Then Monte Carlo method was carried out to simulate 2000 samples from each monthly pdf. Finally each monthly sample added to the observed daily temperature and precipitation time series to produce 2040-2069 daily time series of desired variables. Then each set of temperature and precipitation time series of period 2040-

2069 individually introduced to the IHACRES and 2000 daily time series of runoff of the Aidoghmoush River were simulated. In each runoff time series the annual maximum floods were extracted and appropriate pdf were fitted to them. Results showed (fig 4) that in most cases Log Pearson Type III was the best pdf (as well for the baseline flood).

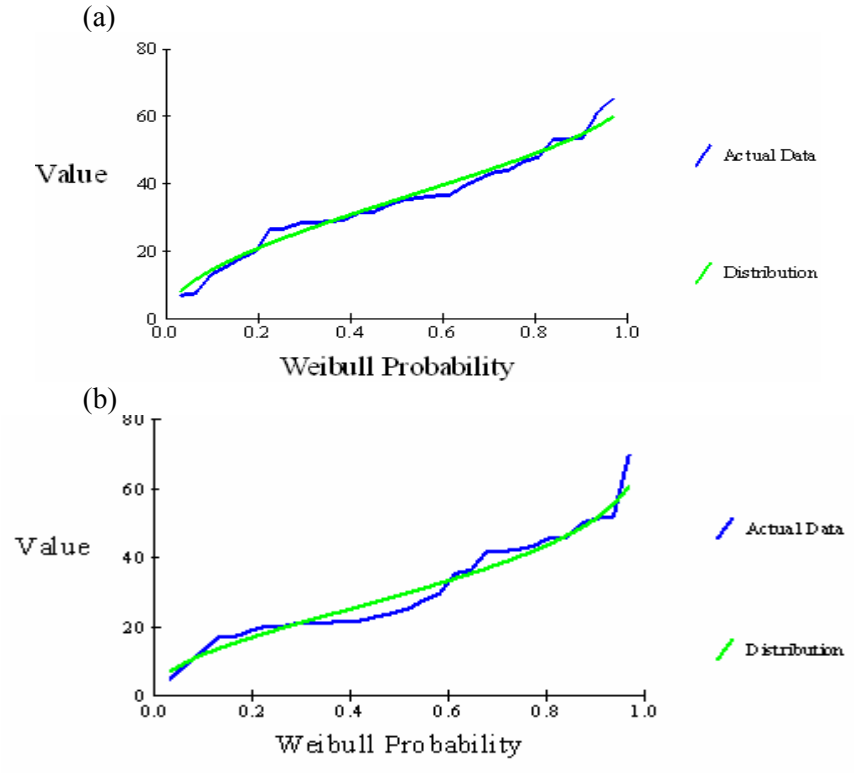


Fig 4, probability distribution function of flood in (a) baseline period and (b) a sample in 2040-2069

Finally changes in two flood indicators were evaluated: Severity of flood events by comparing the magnitude of flood using fixed return periods and frequency of flood events by comparing the return period of a flood using fixed magnitude. Results show that for the return period less than 50-year, the probability of decrease in magnitude of future flood event relative to the baseline is more than the probability of increase. For the return period more than 50-year both situations have nearly same probability to occur (Fig 5-a). On the other hand, analyzing the flood frequencies show that in the future, floods less than 20 m3/s have less chance to occur relative to the baseline and for flood magnitudes more than 60 m3/s this chance will increase (Fig 5-b).

5- CONCLUSION

In this study, climate change scenarios that included A2-SRES emission scenario were generated from 7-TAR GCM and downscaled to an appropriate river basin scale. These data weighted and served as input to a hydrologic model to generate climate impacted runoff for a study basin. These were then used in a statistical river basin simulation model to investigate the sensitivity of a reservoir system to climate change for flood risk. It can be concluded that over-reliance on a single or very few GCMs climate scenarios could lead to inappropriate flood magnitude or frequency. Therefore for providing scientifically based advice to decision makers it is essential that climate change impact studies consider a range of weighted climate scenarios derived from different GCMs. The methodology and the final results of this research can play the base role for other disciplines such as dam operation, dam break risks, agricultural damage risks and adaptation policies that are greatly affected by changes in flood regime. It should be noted that, although applying the uncertainties due to GCM simulation in impact assessment studies can improve the final results, one should use the methodology that

applies other uncertainties due to; emission scenarios, downscaling method, rainfall-runoff modeling and weighting method in the analyses.

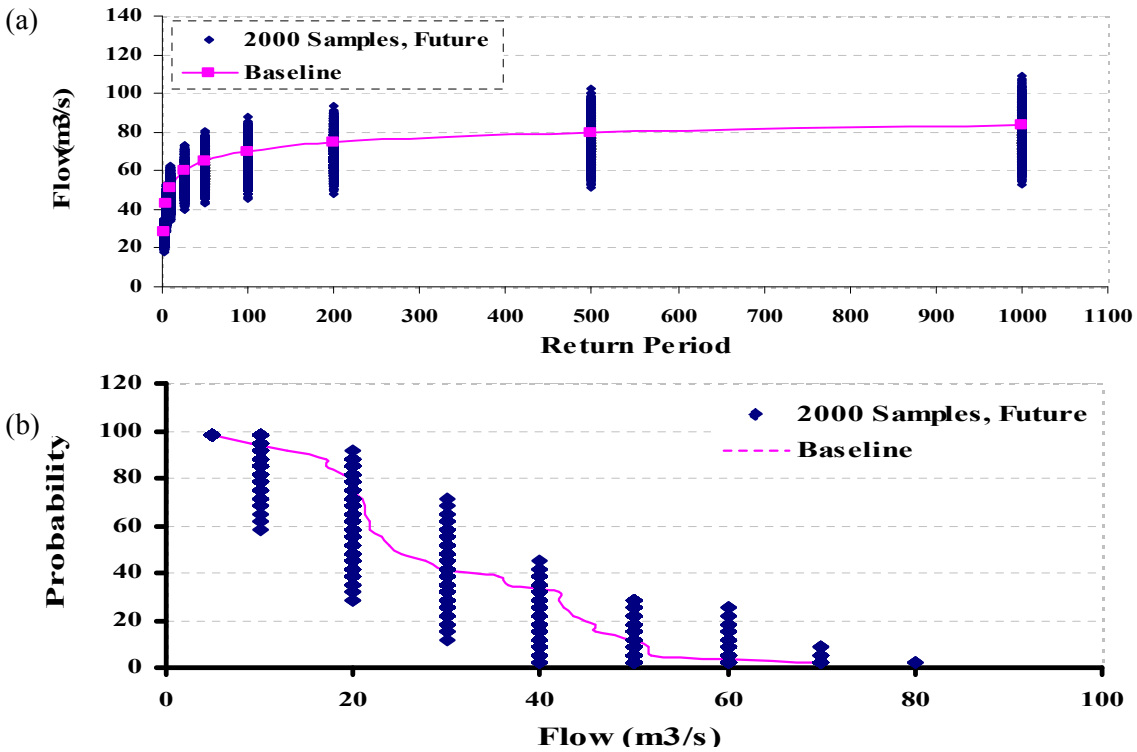


Fig 5, changes in (a) return period and (b) frequency of flood of Aidughmoosh river in 2040-2069 relative to the baseline

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THE EFFECT OF CLIMATE CHANGE ON GROUNDWATER LEVEL IN THE WEST OF SABZEVAR, IRAN

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Abstract: The study area is located west of Sabzevar, Iran. In this region, the ground water level of piezometers existing in the study area measured during 12 years (1995-2006). In this period, there is a meaningful behavior in the changes of groundwater level. The temporal variations in groundwater level may be divided into three sections. Moreover, the value of rainfall during the mentioned period varies considerably. Artificial recharge of aquifer using flood spreading method has been performed in the study area. The obtained results show that the temporal variations of ground water level in the study area depend upon both the changes in the annual rainfall and the values of recharged water due to artificial recharge.

Keywords: artificial recharge, climate change, ground water level, Sabzevar

1. INTRODUCTION

Water is a one of the most important needs for people, and groundwater is the most important available source of fresh water in the world (Todd 1980). Groundwater has been more uniformly spread over the world than surface water. Such that in most arid and semi-arid regions, there is no surface water but groundwater is usually available (Abdin 2006).

Dettinger et al. (2007) expressed that groundwater resources are more stable than surface water against climate change. However, some workers (e.g. Changnon 1987, Dickinson et al. 2004) have stated that climate change (even slow ones) can affect the groundwater resources. In addition, it is a truism that the artificial recharge will increase the groundwater level (e.g. Al Battashi and Syed Rashid 1998, Bacellar et al. 2002, Murray and Tredoux 2002, Sheng 2005).

The study area is located in 45 km west of Sabzevar and it is an arid region. Figure 1 shows the location of study area. In the dry and wet years, the annual precipitation may range between 100 and 300 mm, respectively. According to 12 years data (1994-1995 to 2005-2006) the mean annual precipitation of the study area is almost 178 mm. During the periods of drought, the groundwater level decline is relatively high. In addition, the quality of groundwater from upstream to downstream decreases considerably. Therefore, to improve the quality and quantity of the groundwater in this arid region, m an artificial recharge was performed in this area.

This paper gives information concerning the impacts of climate change on groundwater level in the study area (west of Sabzevar). In addition, it gives the effects of artificial recharge of aquifer using flood spreading method. As it pointed out earlier, the study area is a semi-arid region and there is no surface water in this region. Therefore, in this area alike to most of arid and semi-arid regions of Iran the dependency on groundwater for water supply is 100%.

Although groundwater will be less directly and more slowly impacted by climate change, as compared to surface water but groundwater level in the study area has been considerably affected by variations in annual rainfall. This is because the studied aquifer is located in a alluvial fan and the permeability of the aquifer is very high. Such a high permeability allows the water percolated into the unsaturated zone of aquifer to catch the water table rapidly.

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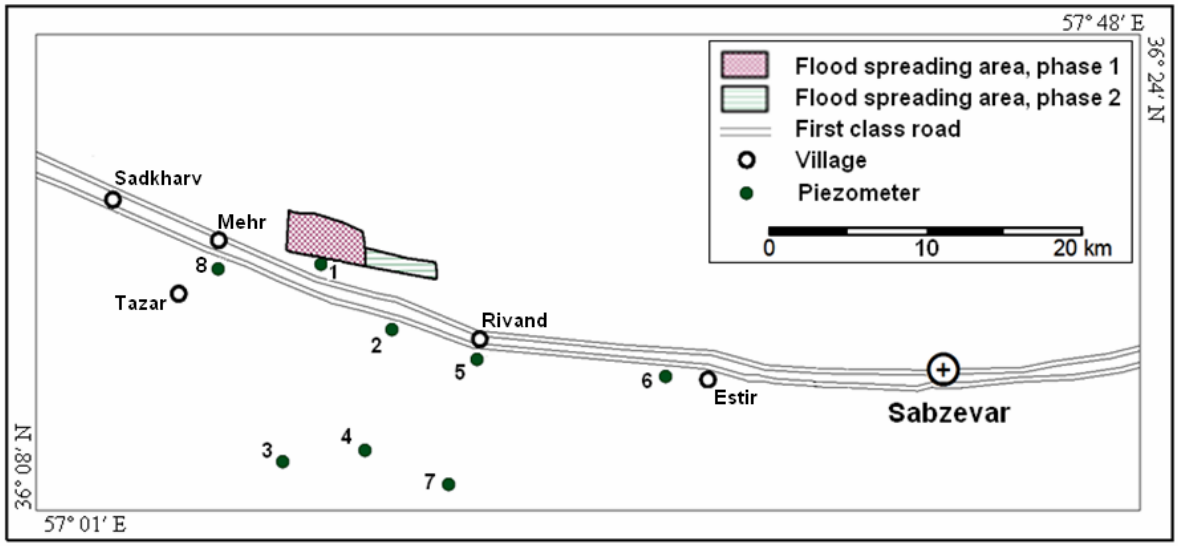


Figure 1. The location of study area

2. PROCEDURE

In order to evaluate the effect of climate change and the values of artificially recharged water through flood spreading on groundwater in the west of Sabzevar, water level data-sets of eight piezometers for 12 water years (1994-1995 to 2005-2006) in the study area were considered. The water levels in the mentioned piezometers have been monthly measured and all of them corrected to the midmonth. In addition, the values of annual precipitation during the above-mentioned period were considered. Moreover, the volume of water recharged to the aquifer through flood spreading sites was taken into account.

3. DISCUSSION

Figure 2 shows the histogram of annual rainfall for 12 years period (1994-1995). As can be seen from the Figure 2, although the values of rainfall vary for different years but this period may be divided into three sub-periods of 4 years (i.e. 1994-1997, 1998-2001, and 2002-2005 sub-periods). It can be seen from the chart that the average value of annual rainfall for first, second and third sub-periods is 188.9, 132.5, and 213.5 mm, respectively. Therefore, on the basis of mentioned values, the average value of annual rainfall for second sub-period in comparison with that for first sub-period has been diminished. However, the average value of annual rainfall for third sub-period in comparing with the second sub-period has been markedly increased.

In order to improve the quality and quantity of the groundwater in the study area which is an arid region; a flood spreading site was constructed in 1996. Artificial recharge using flood spreading method was performed in this area in 1997. Table 1 gives the information on time, number and volume of floods entered to flood spreading sites. As can be seen from the Table 1, the considerable water entered into the flood spreading sites is 15.5 million cubic meters which has occurred in 1997-1998 water year (the year after constructing the sites). Thereafter between 1998- 1999 and 2001-2002 water years, due to low annual rainfall, the volume of water entered into the flood spreading sites has been markedly decreased. Then from 2002-2003 to 2004-2005, due to increase in annual rainfall, the amount of water entered into the flood spreading sites has considerably increased.

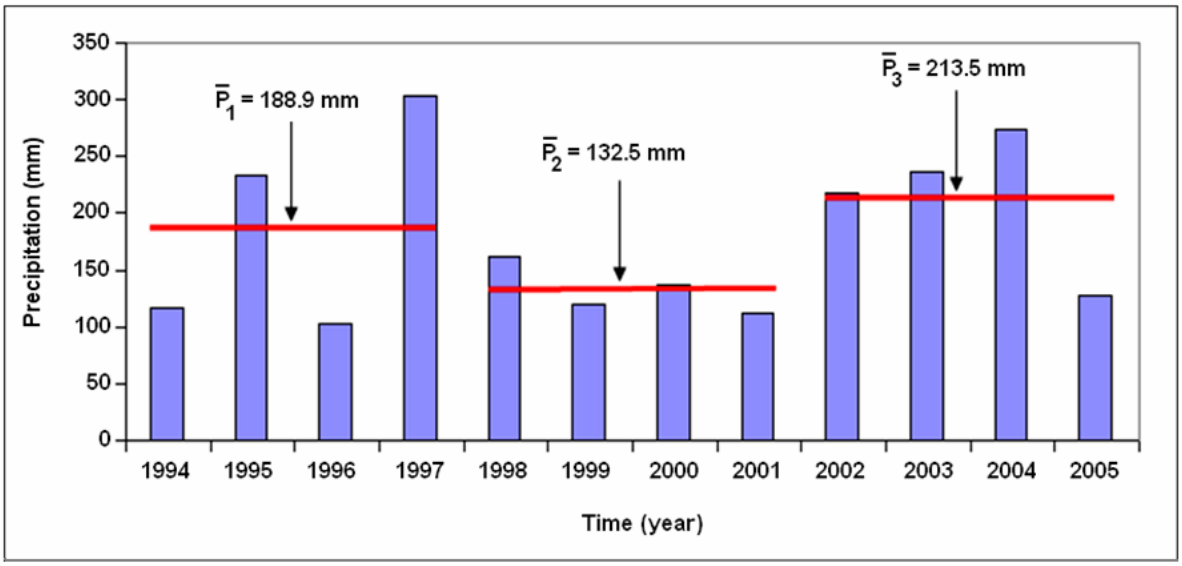


Figure 2. The histogram of annual rainfall from 1994-1995 to 2005-2006

Table1. Time, number and volume of floods entered to flood spreading sites

Water year	Time and number of floods entered to flood spreading sites	Volume of flood $\times 10^6 m^3$
1997-1998	Winter and spring 1998, 7 floods	15.5
1998-1999	Winter and spring 1999, 4 floods	1.6
1999-2000	Winter 2000, 4 floods	0.2
2000-2001	Winter 2001, 4 floods	0.5
2001-2002	Fall 2001 and winter 2002, 3 floods	1.1
2002-2003	All seasons, 10 floods	7.0
2003-2004	Winter and spring 2004, 6 floods	7.1
2004-2005	Winter and spring 2005, 6 floods	6.0
2005-2006	Winter and spring 2006, 5 floods	0.3

Figures 1, 2, and 3 show the groundwater level fluctuations in three piezometers of study area. As can be seen from these Figures, in most piezometers the fluctuations of groundwater level can be divided into three equal four-year sections. The values of water level gradient were calculated in each section for all piezometers. Table 2 gives the values of water level gradient in each section for eight piezometers.

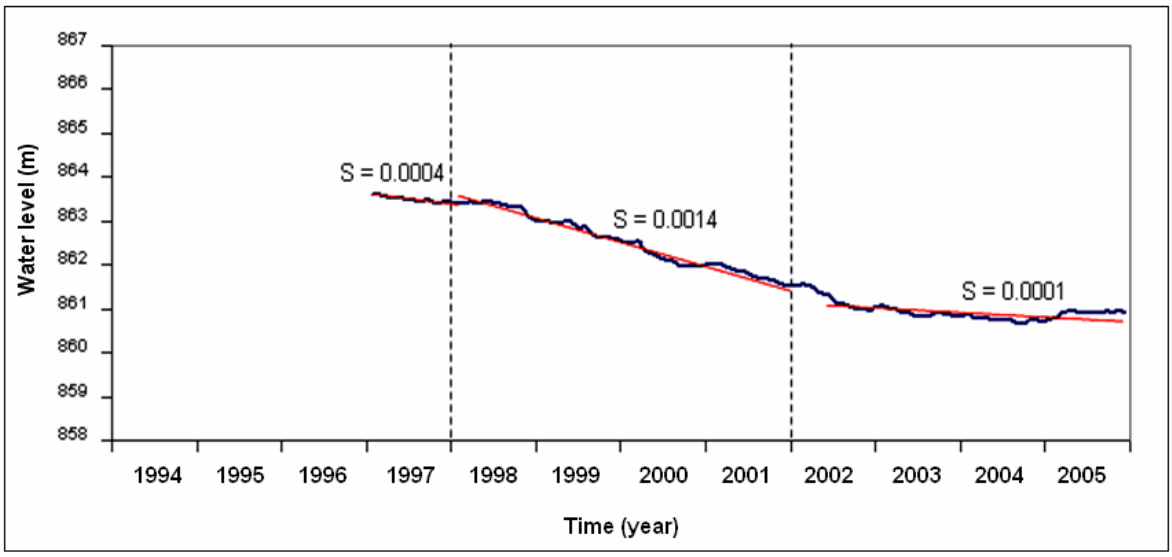


Figure 3. Groundwater level fluctuations in piezometer No.1

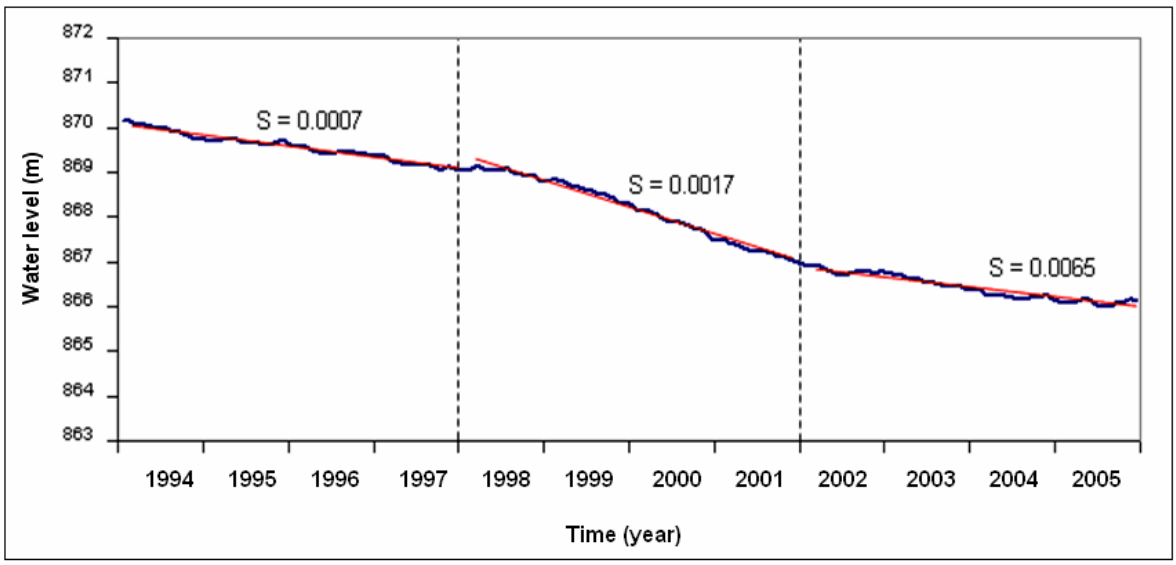


Figure 4. Groundwater level fluctuations in piezometer No.2

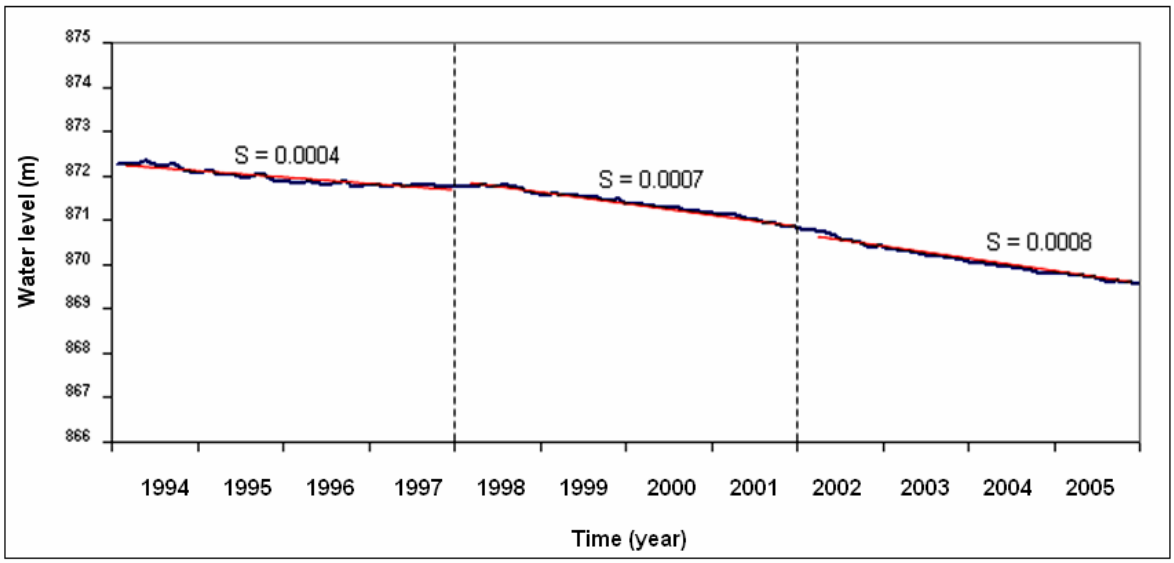


Figure 5. Groundwater level fluctuations in piezometer No.3

Table 2. Water level gradient in eight piezomeres in study area

Piezometer	WLG in Period 1	WLG in Period 2	WLG in Period 3	$\times 100 \frac{\text{WLG in period 3}}{\text{WLG in period 2}}$
1	0.0004	0.0014	0.0001	7 %
2	0.0007	0.0017	0.0006	35 %
3	0.0023	0.0029	0.0030	103 %
4	0.0008	0.0020	0.0015	75 %
5	0.0010	0.0012	0.0008	67 %
6	0.0004	0.0007	0.0008	114 %
7	0.0014	0.0016	0.0011	69 %
8	- - -	0.0008	0.0005	63 %
WLG = Water level gradient				

As can be seen from Table 2, that variations of water level gradient in different piezometers are not the same. To evaluate the variations in water level for all piezometers, the ratio of water level gradient of period 3 to that of period 2 was calculated (last column in Table 2). This ratio indicates to degree of variations in water level in the mentioned periods. According to the ratio of water level gradient of period 3 to that of period 2, these studied piezometers may be classified into two groups. First ones are piezometers 1 and 2 which affected by both variations in value of rainfall and recharged water through flood spreading sites. Second groups are those piezometers (No. 3 to 8) which subjected to variations in annual rainfall only. Figure 6 illustrates the ratio of water level gradient of period 3 to that in period 2 in all piezometers. As can be seen from this histogram, the ratio of water level gradient of period 3 to that in period 2 in piezometer No.1 is minimum one. It is because the groundwater level in period 3 in comparison with period 2 has been slowly decreased in this piezometer due to effect of considerable water entered to flood control site near this piezometer. In piezometer No.2, the effect of recharged water through flood control site may be seen as well. In other piezometers which have been affected by variations in annual rainfall only, the groundwater level in period 3 in comparison with period 2 has been considerably decreased.

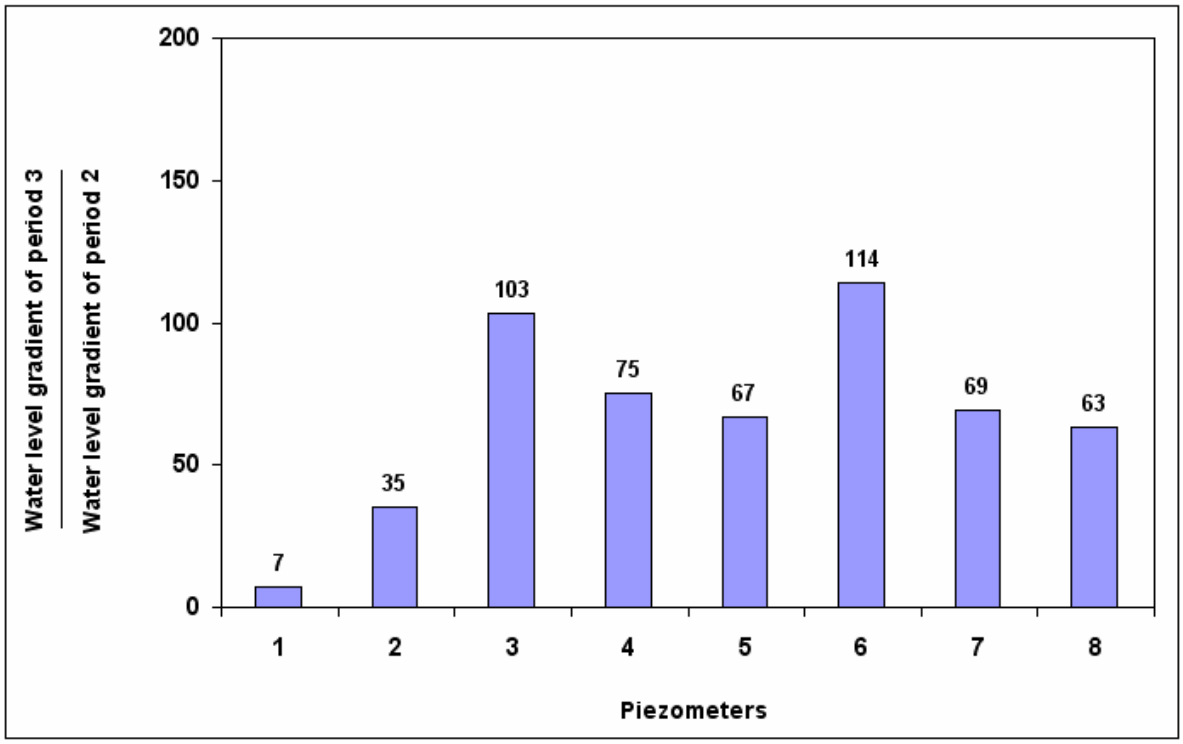


Figure 6. The ratio of water level gradient of period 2 to that in period 3 in all piezometers

4. CONCLUSION

To assess the effects of climate change and artificial recharge on groundwater level in west of Sabzevar, data obtained from eight piezometers and also the annual precipitation during 12 years (1995-2006) were used. According to the groundwater level gradient in the piezometers and the variations in annual rainfall, the study period (i.e. 1995-2006) can be divided into three identical sections. Artificial recharge using flood spreading method was performed in this area in 1997. The obtained results show that in piezometers which have been affected by artificial recharge through flood control sites the temporal variations of ground water levels respond to both climate change (i.e. change in annual rainfall) and the volume of water added to aquifer by artificial recharge. However, for those piezometers which have not been affected by artificial recharge, temporal variations of ground water levels respond to climate change only. For instance, in piezometer No.1, due to marked decrease in rainfall during second sub-period (i.e. 1998-2001), the groundwater level have rapidly been declined. In this piezometer during the third sub-period (i.e. 2002- 2005), because of effect of water added to the aquifer through artificial recharge, the groundwater level have slowly been declined. However, most piezometers the groundwater level show rapid decline even in third sub-period which the annual rainfall has been increased.

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IMPACT OF GLOBAL DIMMING AND BRIGHTENING ON REFERENCE EVAPORTRANSPIRATION OVER COMPLEX TERRAINS ESTIMATED FROM MINIMUM CLIMATOLOGICAL DATA

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Abstract: For climate trend studies or for use in climate models, in areas where time series of weather measurements are often available for more sites or for longer time periods only as monthly values, it is desirable to be able to use minimum monthly climatological data to estimate reference evapotranspiration, accurately. A new 'reference evapotranspiration model for complex terrains' (REMCT), with monthly time step, is used for estimating trends of reference evapotranspiration in Greece, incorporating rainfall occurrence, and air temperature and the effect of climate and site elevation. REMCT is applied after developing a methodology for calibrating its parameter values with monthly Penman-Monteith estimates obtained from daily values of the appropriate meteorological parameters, in 29 sites evenly distributed over Greece for the period 1974-2001. Furthermore in this study, trends in reference evapotranspiration based on REMCT are examined for 16, 22 or 29 stations across Greece, during the periods 1950-2001, 1958-2001 or 1986-2001, respectively. The consistency of the reported trends in radiation and temperature records and their implications for the hydrological cycle and especially the trends in the modelled reference evapotranspiration are investigated. Thus, the signs of trends in reference evapotranspiration are examined according to the global dimming or brightening periods, taking into account some of the studies suggesting that the widespread reduction of surface solar radiation ('global dimming') resulted in reduced evaporative cooling at the land surface, (consistent with reported decreasing pan evaporation for many regions throughout the world, with widely different climates, over the past 50 years), was effective up to the 1980s and gradually transformed into 'brightening'.

Key words: Complex terrain, evapotranspiration, global brightening, global dimming, Penman Monteith, REMCT, trends

1. INTRODUCTION

The climate of the 20th century has undergone significant changes of natural and anthropogenic origin. Many studies document the changes in surface temperature over the past decades and a few studies concentrate on the surface energy balance components, which are the processes that govern the evolution of surface temperature. Additionally, much evidence from the changes of precipitation, runoff, and soil moisture suggests that the hydrological cycle has been intensified in many parts of the world during the past century, along with the increasing global temperature. Thus, changes in the climate need not to be limited to changes in the energy balance, but may also be extended to the water. The connecting term between water and energy balance is actual evapotranspiration, which, is rather difficult to be directly measured and indirect approaches are used for estimating it. One of the common ones is to firstly estimate potential evaporation, commonly measured by evaporation pans. It is also estimated from readily available climate variables using models such as the Penman-Monteith reference evapotranspiration model or simpler methods based on minimum data. Consequently, several studies have examined the trends in potential or reference evapotranspiration, in order to investigate the changes to latent heat (the energy equivalent to evaporation) over land. Contrary to the expectation that a warmer climate will increase evaporation, many studies have reported decreasing pan evaporation over the past 50 years for many places of the world (Peterson et al., 1995; Chattopadhyay and Hulme, 1997; Quintana-Gomez, 1998; Brutsaert and Parlange, 1998; Lawrimore and Peterson, 2000; Golubev et al., 2001; Roderick and Farquhar, 2002; Groisman et al., 2004; Hobbins et al., 2004; Asanuma et al., 2004; Liu et al., 2004; Liu and Zeng, 2004; Roderick and Farquhar, 2004, 2005;

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Chen et al., 2005; Tebakari et al., 2005; Shenbin et al., 2006; Roderick et al., 2007; Zhang et al., 2007; Burn and Hesch, 2007; Papaioannou et al., 2007; Jovanovic et al., 2008; Hobbins et al., 2008). These decreases in pan evaporation were not universal, and data from a few regions indicate increases (Cohen et al., 2002; da Silva, 2004; Xu et al., 2005; Kirono et al., 2008). At individual sites, both decreases and increases have commonly been observed. Like the trends in pan evaporation, trends in reference evapotranspiration estimated from readily available climate variables have been reported negative in some regions and positive in other regions. Thus, trends in Penman Monteith estimates have been referred negative (Thomas, 2000; Cohen et al., 2002; Chen et al., 2005; Wu et al., 2006; Gao et al., 2007; Zhang et al., 2007) and positive (Yang et al., 2005; Oguntunde et al., 2006). There are also reports on increases in estimates based on other methods, as Kondo and Xu method (Xu et al., 2005), Morton method (Kirono et al.,2008)and Thornthwaite method (Hobbins et al., 2008).

The interpretation of the above mentioned negative trend has been controversial, and so far agreement on its significance, regarding terrestrial evaporation has remained elusive. At first it was thought as indication of a general negative trend in terrestrial evaporation in those areas, which seemed incompatible with the reported general increases in global precipitation in the same period. According to Brutsaert and Parlange (1998), this ‘evaporation paradox' was suggested that may indicate increasing terrestrial evaporation, due to the complementary relationship between potential and actual evaporation. This explanation was generally consistent with other evidence by Lawrimore and Peterson (2000), Golubev et al. (2001), Hobbins et al. (2004) and Ramirez et al. (2005). On the other hand in some studies (Cohen et al., 2002; Roderick and Farquhar, 2002; Ohmura and Wild, 2002; Liu et al., 2004; Wild et al., 2004) the decreasing pan evaporation was attributed in decreasing solar irradiance (global dimming) giving evidence of decreasing landscape evaporation. Recently, Brutsaert (2006) has shown that both effects (global dimming and the complementary relationship) can be combined, whereas, Zhang et al. (2007) have referred that actual evapotranspiration and pan evaporation exhibit a complementary relationship in Tibetan Plateau but not Bouchet’s complementary relationship. Additionally, in recent studies, an emerging evidence for a widespread decline of solar dimming during the 1980s and reversal to a brightening thereafter (Wild et al., 2005; Pinker et al., 2005) is also reported. Furthermore, Wild et al. (2007) showed that the decadal changes of land mean, maximum and minimum surface temperature, as well as temperature range, are in line with the proposed transition in surface solar radiation from dimming to brightening during the 1980s and with the increasing greenhouse effect. Thus, in this study, the consistency of trends in radiation and temperature records and their implications for the hydrological cycle and especially the trends in reference evapotranspiration are examined. Since temperature and rainfall attributes are often available for more sites or for longer time periods, a new ‘reference evapotranspiration model for complex terrains’ (REMCT), with monthly time step, developed by Diodato and Bellocchi (2007) is used for estimating trends of reference evapotranspiration in Greece, incorporating rainfall occurrence, air temperature and the effect of climate and site elevation. REMCT is applied after developing a methodology for calibrating its parameter values with Penman-Monteith estimates, obtained from daily values of the appropriate meteorological parameters, in 29 sites evenly distributed over the country for the period 1974-2001. Furthermore, trends in reference evapotranspiration based on REMCT are examined for 16, 22 or 29 stations across Greece, during the periods 1950-2001, 1958-2001 or 1986-2001, respectively. The sign of trends in the modelled reference evapotranspiration is examined according to the global dimming or brightening periods.

2. MATERIALS AND METHODS

In this study, daily measurements of dry and wet bulb-temperature, wind velocity and sunshine hours obtained from the National Meteorological Service for 29 stations in Greece during the period 1974-2001, are used for estimating reference evapotranspiration by Penman-Monteith equation (Monteith, 1981). The characteristics of the stations are presented in table 1. The Penman-Monteith reference evapotranspiration (ET_{PENMON}), is expressed as:

$$ET_{PENMON} = [0.408\ \Delta\ (R_n - G) + \gamma\ (900/T + 273)\ u_2\ (e_s - e_a)] / [\Delta + \gamma\ (1 + 0.34 u_2)] \quad (1)$$

Where, Δ is the slope of saturated vapor pressure in relation to air temperature (kPa $^oC^{-1}$), γ is the psychrometric constant (kPa $^oC^{-1}$), R_n is the net radiation (MJ m^{-2} d^{-1}), G is the soil heat flux density (MJ m^{-2} d^{-1}), $(e_s - e_a)$ is the vapor pressure deficit (kPa), T is the mean daily air temperature at 2 m above ground level (^oC) and u_2 is the wind speed (m s^{-1}) at 2m above ground level.

Monthly values of mean (T) and maximum (Tmax) air temperature, precipitation (PR) and number of rainy days (Rd) available for the mentioned 29 stations during the period 1974-2001, respectively, are used to obtain monthly reference evapotranspiration (ET_{DIOBEL}) based on the reference evapotranspiration model for complex terrain (REMCT), as developed by Diodato and Bellocchi (2007). ET_{DIOBEL} values are estimated by the following nonlinear equation:

$$ET_{DIOBEL} = (R_{so}/\lambda)\ (T +17.8)\ [(T_{max}\text{-ref}(T_{min}))^{0.5}\text{-f(w) Rd})]\ f(clim) \quad (2)$$

where, R_{so} (MJ m^{-2} d^{-1}) is the mean monthly extraterrestrial solar radiation, λ (MJ kg^{-1}) is the latent heat of evaporation, T(°C) is the mean monthly air temperature, T_{max} (°C) is the mean monthly maximum air temperature, Rd ($month^{-1}$) is the number of rainy days in a month, ref(T_{min}) (°C) is a reference minimum air temperature and f(clim) and f(w) (month) are semi-empirical functions modulating the rate at which the energetic forcing, involved in the evapotranspirative processes (principally the term under square root in equation 2), is approached as air temperatures and rainy days change. The reference minimum air temperature is determined as: $ref(T_{min})=a\ [1- \alpha \cos(\ 2\ \pi\ ((m-b)/c)]$ where, m is the month and a, α, b and c are empirical parameters equal to:–2.64 (°C), 1.79, 6.5 and 12.0, respectively. The two semi-empirical functions are determined as: $f(clim)=d-e\ [((g-E)\ P/1000)^{0.5}\text{- hE}]$ and $f(w)=k+q(T_{(\alpha)max}-10)^3$

where, $T_{(a)max}$ (°C) is the long-term yearly mean daily maximum temperature, *E* is the site's elevation (m), *P* is the long-term mean annual precipitation (m) and d, e, g, h, k and q are empirical parameters equal to 0.00247, 0.000000831 (m^{-1}), 2000 (m), 0.1 (m^{-1}), 0.0397 ($°C^{0.5}$ $month^{-1}$) and 0.000079228 ($°C^{-3}$). The parameter g (in f(clim) function), set equal to 2000 m, is the upper elevation for REMCT to be applied.

The obtained monthly ET_{DIOBEL} (uncalibrated) values based on the original parameters of REMCT are compared to monthly ET_{PENMON} estimates for each of the 29 stations for the period 1974-2001. Their linear regression is presented in figure 1 (uncalibrated). The results of the linear regressions (determination coefficient R^2 and slope a) and 'diffrence measures' (root mean square error RMSE, mean bias error MBE and index of agreement IA) are estimated for each station. It is shown that the MBEs are greater than 10% in a lot of stations and the RMSEs are quite high, although the indices of agreement are excellent (ranging from 0.9343 to 0.9935). As a result, it was considered that there is a need for the model to be calibrated. A calibration procedure was developed and the parameter values of REMCT were basically recalculated from comparisons against mean monthly Penman Monteith estimates. Keeping constant the parameters of ref (T_{min}), the following calibration process was employed for equation 2: First, the values were determined for f(clim) and secondly for f(w), fitting the estimates of the depended upon $R_{so,}$ λ, T, $T_{max,}$ ref(T_{min}), Rd terms of equation 2, against the Penman Monteith estimates. Next, the parameters of the functions f(clim) and f(w), were determined by multiple regression analysis. The best results (as compared to Penman Monteith estimates) were evaluated, when these parameters were determined independently for groups of stations, and not uniquely for the whole country. The best local calibration was achieved by grouping the stations of the study according to their monthly values of T, T_{max} and Rd, following cluster analysis. For each group, some stations are selected as calibration set (marked with *) and the rest ones are used as validation set. The empirical parameters d, e, h, k and q (for each function f(clim) and f(w)) were determined from the calibration data set only, by multiple regression analysis The obtained values were: (d=0.001342, e=0.0000000477 m^{-1}, h=2.760639 m^{-1}, k=0.00562 $°C^{0,5}$ $month^{-1}$, q=0.00000316 $°C^{-3}$) for group 1 (Serres, Florina*, Kavala*, Alex/poli, Kozani, Tripoli*, Ioannina*) or (d=0.001766, e=0.0000002 m^{-1}, h=0.151184 m^{-1}, k=0.017697 $°C^{0,5}$ $month^{-1}$, q=0.0000136 $°C^{-3}$) for group 2 (Mikra*, Larissa*, Kerkira*, Agrinio, Aghialos, Lamia, Andravida, Kalamata*) and (d=0.002158, e=0.00000046 m^{-1}, h=0.423593 m^{-1}, k=0.02619 $°C^{0,5}$ $month^{-1}$, q=0.0000388 $°C^{-3}$) for group 3 (Mitilini*, Argostoli, Chios, Elliniko, Naxos, Methoni, Milos, Kithira*, Sitia*, Rhodes*, Iraklio*, Ierapetra, Tymbaki*, Pirgos*). These new parameters were used in the REMCT for estimating new monthly ET_{DIOBEL} (calibrated) values, for each station (of either calibration or validation set). The calibrated ET_{DIOBEL} values are compared to mean monthly Penman Monteith estimates and their linear regression for all stations is also presented in figure 1 (calibrated). The results of the linear regressions and 'difference measures' (R^2, a, RMSE, MBE and IA) for each station show that the excellent indices of agreement (referred above for the uncalibrated ET_{DIOBEL} values) are improved (ranging from 0.952 to 0.995) and the MBEs are mainly reduced, being smaller than 10% in 25 (out of 29) stations. Only 3 (out of 14) stations belonging to the validation set and 1 station from the calibration set present MBE of the order of 14% and 11%, respectively. The RMSEs are also reduced, ranging mainly from 8.7 % to 13.8 % for 24 stations but remaining up to 15.9 - 22% for 5 stations. Thus, the performance of the REMCT model is very high over Greece (a country characterized by a complex terrain) and this is strengthened especially from the validation data set results. The REMCT model, linking reference evapotranspiration to

minimum available data, as rainfall occurrence and mean and maximum air temperature, but including the effect of climate and site elevation can be used for estimating reference evapotranspiration and investigating its time evolution for longer time periods over Greece.

Thus, in this paper, according to the availability of the mentioned data, the modelled ET_{DIOBEL} is calculated furthermore for 22 and 16 stations (evenly distributed across the country), during the longer periods 1958-2001 and 1950-2001, respectively. The selected 16, 22 and 29 stations are shown in table 2. The slopes of the trends of reference evapotranspiration based on REMCT (bET_{DIOBEL}) and precipitation (bPR) are estimated for 16, 22 or 29 stations, during the periods 1950-2001, 1958-2001 or 1986-2001, respectively and the results are shown in table 2, with the statistically significant ones noted as bold. The signs of trends in the modelled ET_{DIOBEL} are examined during these selected periods, according to the global dimming or brightening periods, although the period 1986-2001 is quite short for safe conclusions. The number of stations with increasing or decreasing trends in ET_{DIOBEL} and Pr is apparent in table 3. (The statistically significant ones are shown in parenthesis). It is obvious that more stations (9 out of 16, or 17 out of 22, or 28 out of 29) exhibit positive trends during the selected time periods. The tendency for even more positive trends is indicated, as more years belonging to the solar brightening period, are included in the period considered for the time evolution. Precipitation seems decreasing in all stations (16 out of 16 and 21 out 22) during the longer periods, whereas almost equal positive and negative trends (all statistically insignificant) are evident during the shorter period. Finally, the trends of reference evapotranspiration (ET_{DIOBEL} and ET_{PENMON}), precipitation (PR), number of rainy days (Rd), mean (T), maximum (Tmax) and minimum (Tmin) air temperature for all stations considered 'as a whole' are examined. According to the available data and the selection of stations (in order to be evenly distributed across the country), these trends are evaluated 'as a whole' for 22 or 29 stations during the periods 1958-2001, 1974-2001 and 1986-2001 (table 4) or 1974-2001 and 1986-2001 (table 5), respectively. The slopes of the trends either of ET_{DIOBEL} or ET_{PENMON} (table 4 or 5), are positive from both analyses, reassuring (the referred from the analysis of the independent stations) tendency for increasing positive slopes, as the period 1986-2001 is more involved in the data series. Precipitation and rainy days exhibit also statistically insignificant negative trends, in both analyses 'as a whole' (for 22 or 29 stations) with their slopes indicating weakening, as their time evolution is considered towards the latest years. In the contrary, T and Tmax indicate a negligible, a small (0.03 ^{0}C/y) and a further increasing trend (0.06 and 0.05 ^{0}C/y, respectively) during the periods 1958-2001, 1974-2001 and 1986-2001, respectively, when considering 22 stations 'as a whole' (table 4). These results are also confirmed from the analysis referring to 29 stations and two available time periods (table 4). In addition, it must be noticed (table 4 or 5) that the trend of Tmin is positive (0.01, 0.04, 0.06 ^{0}C/y) or (0.03, 0.06 ^{0}C/y) for 22 or 29 stations, respectively, considered 'as a whole'. It is also increasing towards the latest time periods.

3. RESULTS AND DISCUSSION

Reference evapotranspiration can be computed from meteorological data by means of different methods. The Penman-Monteith formulation (Allen et al., 1998) is regarded as a good reference evapotranspiration estimator for a wide variety of climatic conditions, but a major drawback to its application is the relatively high data demand, requiring air temperature, wind speed, humidity, and global solar radiation data. Therefore, an accurate approach of estimating reference evapotranspiration for complex terrains, from minimum data of weather measurements, only available as monthly summaries, appropriate for climate trend studies and for use in climate models, is desirable since it can increase spatial resolution in data-sparse areas. Since temperature and rainfall attributes are frequently available on a regional scale, the new reference evapotranspiration model for complex terrains, (REMCT), with monthly time step, linking reference evapotranspiration to rainfall occurrence and air temperature, and including the effect of climate and site elevation, developed by Diodato and Bellocchi (2007), is selected for estimating reference evapotranspiration in Greece, a country characterized by a complex terrain. The model is applied for estimating reference evapotranspiration in Greece, after developing a methodology for calibrating its parameter values with monthly Penman-Monteith estimates obtained from daily values of the appropriate meteorological parameters, in 29 sites evenly distributed over the country, for the period 1974-2001. The model has been applied successfully to a range of terrains in Italy (Diodato and Bellocchi, 2007), and it was selected for Greece, taking into account that it was developed, in order to skip over limitations observed with other simplified models [as the ones by Hargreaves and Samani (1985) and Droogers and Allen (2002)] in complex terrains. The obtained

uncalibrated ET_{DIOBEL} values (based on the original parameters of the proposed model) compared to ET_{PENMON} values present indices of agreement very high. The MBEs though, are greater than 10% in a lot of stations and the RMSEs quite high. This can be attributed to the fact that REMCT appears sensitive to variation in its several parameters and its application to Greek sites may be limited by its ability to provide appropriate site-representative parameters, able to capture the temporal variability of evapotranspiration data. Its generic optimized parameters, being crude estimates over multiple years have been determined over a pool of data from all sites in Italy to ensure a generic spatial and temporal representation, but specific geographical locations in Greece, may have particular characteristics that cause the parameters to deviate from reference values and require local calibration. Thus, after calibrating locally the parameters of the model, with monthly Penman-Monteith estimates (according to the developed methodology), the accuracy of the predictions of the model is improved. The indices of agreement become even better, the MBEs are reduced, becoming smaller than 10% in most sites and the RMSEs become smaller, ranging from 8-14% in most stations. It must be mentioned though, that a high RMSE of the order of 22% remains apparent in Ierapetra, a site characterized by very high values of the aerodynamic term, which rather could not be approximated by the model. As a conclusion, the performance of the REMCT model is very high over Greece (a country characterized by a complex terrain) and this is strengthened especially from the validation data set results. This may be attributed to the fact, that rainy days number in a month is used rather successfully as an air humidity proxy in the model, modulated by the function of the long-term maximum air temperature f(w). In addition, the function f(clim) helps matching the reference evapotranspiration fluctuations for different combinations of precipitation (proxy of air humidity) and elevation (proxy of solar transmittance). The dependence of reference evapotranspiration on air temperature, seems also successfully limited in the model, by daytime air temperatures and thus roughly matched by maximum air temperatures (T_{max}), since air temperature lapse rate is usually greater for T_{max} than for T_{min}, and as a result, smaller values of estimated radiation transmittance and reference evapotranspiration at higher elevations are obtained. Thus, the analysis shows that REMCT based on minimum data, is not limited only to Italian sites (Diodato and Bellocchi, 2007), but is also suitable to estimate accurately the monthly mean reference evapotranspiration on heterogeneous surfaces in Greece and hence to investigate its time evolution for longer time periods over the country.

Furthermore in this study, the consistency of trends in radiation and temperature records and their implications for the hydrological cycle and especially the trends in reference evapotranspiration are examined. Various studies suggest that solar energy at the surface showed significant changes during the last 45 years. Especially, solar dimming was effective in masking greenhouse warming, but only up to the 1980s, when dimming gradually transformed into brightening. Since then, the uncovered greenhouse effect is apparent with a rapid temperature rise over land since mid-1980s. In the other hand, the greenhouse gas induced enhancement of the downward long-wave radiation, has not exceeded the short-wave loss during the first period, resulting in a reduction of surface net radiation and as a consequence, reduced evaporative cooling at the land surface, (consistent with reported decreasing pan evaporation for many regions throughout the world with widely different climates over the past 50 years) explains the apparent inconsistency between reduced surface radiative heating and increased temperature (Wild et al., 2004). Thus, in this paper, trends in reference evapotranspiration based on REMCT are examined for 16, 22 or 29 stations across Greece, during the periods 1950-2001, 1958-2001 or 1986-2001, respectively. The sign of trends in the modelled ET_{DIOBEL} is examined according to the global dimming or brightening periods although the period 1986-2001 is quite short for safe conclusions. It is obvious from the analysis (table 3) that most stations exhibit positive trends especially during the period 1958-2001 and almost all, during the period 1986-2001, which is characterized as the solar brightening period. The trends of reference evapotranspiration ET_{DIOBEL} as well as ET_{PENMON}, for 22 or 29 stations, considered 'as a whole' during the periods 1958-2001, 1974-2001 and 1986-2001 (table 4) or 1974-2001 and 1986-2001 (table 5), respectively are also examined. ET_{DIOBEL} or ET_{PENMON} from both analyses (table 4 or 5), exhibit positive trends. The tendency for increasing positive slopes, as the period 1986-2001 is more involved in the data series is also evident. These results are in agreement with positive trends in reference evapotranspiration reported by Yang et al. (2005) and Oguntunde et al. (2006) or Xu et al. (2005) or Kirono et al. (2008) or Hobbins et al. (2008), based on Penman Monteith or Kondo and Xu or Morton or Thornthwaite method, respectively, and in disagreement with reported negative trends in Penman Monteith estimates (Thomas et al., 2000; Cohen et al., 2002; Chen et al., 2005; Wu et al., 2006). The tendency for even more positive trends is indicated, as more years belonging to the solar brightening period, are included in the period considered for the time evolution and this increase may be consistent with

increased surface radiative heating (Wild et al., 2004, 2007). In the contrary, precipitation seems decreasing in all independent stations during the longer periods, whereas almost equal positive and negative trends (all statistically insignificant) are evident during the shorter period (table 3). However, precipitation and rainy days exhibit also statistically insignificant negative trends, in both analyses 'as a whole' (for 22 or 29 stations) with their slopes indicating weakening, as their time evolution is considered towards the latest years (table 4 or 5). General decrease of rainy days and negative trend of rainfall, consistent with the negative trend of the cyclonic circulations types during the period 1958-2000 have also been reported for Greece by Maheras et al. (2004).

Finally, the trends of mean, maximum and minimum air temperature for 22 or 29 stations considered 'as a whole' are examined during the periods 1958-2001, 1974-2001 and 1986-2001 (table 4) or 1974-2001 and 1986-2001 (table 5), respectively. T and Tmax indicate a negligible, a small and a further increasing trend during the 3 periods, respectively, when 22 stations are considered 'as a whole' (table 4). These results are also confirmed from the analysis referring to 29 stations and the 2 available time periods (table 4). In addition, it must be noticed (table 4 or 5) that the trend of Tmin is positive for 22 or 29 stations, considered 'as a whole' and increasing towards the latest time periods. These findings are consistent with the ones reported by Wild et al. (2007) for a rapid mean or maximum or minimum temperature rise over land surfaces since the mid-1980s (0.38 or 0.37 or 0.40^0C/decade, respectively). Especially, they suggested that 'solar dimming was effective in masking greenhouse warming, but only up to the 1980s, when dimming gradually transformed into brightening and since then, the uncovered greenhouse effect is apparent with a rapid temperature rise over land since mid-1980s'.

In the present study, positive trends in reference evapotranspiration estimated either from the REMCT or Penman Monteith method can be attributed to the fact that potential additional energy from solar brightening, noted during the 1990s, may not only have gone into heating of the surface, but also into additional evaporation. This is also supported by pan evaporation measurements in energy limited environments which partially indicate, after decades of decrease (Roderick and Farquhar, 2002; Ohmura and Wild, 2002), a recent recovery, in line with changes in surface solar radiation (Liu et al., 2004). The rapid increase in land surface temperature, despite evidence for a concurrent increase in evaporative surface cooling also implies a strong increase in surface radiative forcing in the 1990s (Wild et al., 2007).

Table 1.Characreristics of the stations and average values of dry (T) and maximum (Tmax) temperature and number of rainy days (Rd)

Station	Latitude	Longitude	Altitude (m)	T (^{o}C)	Tmax (^{o}C)	Rd (month^{-1})
Serres	41^o 05'	23^o 34'	34	15.23	20.48	7
Florina	40^o 48'	21^o 25'	4	12.07	17.38	7
Mikra	40^o 31'	22^o 58'	4	15.83	20.35	9
Kavala	40^o 56'	24^o 23'	15	14.68	19.03	7
Alexa/poli	40^o 51'	25^o 26'	3.5	14.9	19.15	7
Kozani	40^o 17'	21^o 47'	625	13.08	17.78	7
Kerkira	39^o 37'	19^o 55'	2	17.55	21.93	10
Ioannina	39^o 40'	20^o 51'	483	14.24	19.47	10
Larissa	39^o 39'	22^o 26'	71	15.71	21.38	9
Aghialos	39^o 13'	22^o 48'	12.2	16.32	20.81	8
Mitilini	39^o 04'	26^{o}36'	4.2	17.62	20.96	7
Agrinio	38^o 37'	21^{o}33'	24	17.03	22.99	8
Lamia	38^o 51'	22^o 24'	14.3	16.58	21.87	8
Andravida	37^o 55'	21^o 17'	11.1	17.23	21.79	8
Argostoli	38^{o}11'	20^o 29'	21.2	18.2	21.28	8
Chios	38^o 21'	26^o 09'	4	17.48	20.85	4
Pirgos	37^o 40'	21^o 26'	12	17.58	22.87	7
Tripoli	37^o 32'	22^o 24'	652.4	13.94	19.6	9
Elliniko	37^o 54'	23^o 45'	10	18.43	21.99	7
Kalamata	37^o 04'	22^o 06'	222	17.36	22.52	9
Naxos	37^o 06'	25^o 23'	9	18.24	20.49	5
Methoni	36^o 50'	21^o 42'	53	17.85	21.13	8

Milos	36° 44’	24° 26’	164	17.54	20.67	6
Kithira	36° 08’	23° 01’	165.8	17.7	20.04	6
Rhodes	37° 24’	28° 05’	34.7	19.22	22.01	6
Iraklio	35° 20’	25° 11’	37	18.68	21.7	8
Ierapetra	35° 00’	25° 44’	10	19.71	23.3	5
Sitia	35° 12’	26° 08’	114.5	18.58	21.52	5
Tymbaki	35° 00’	24° 46’	5	18.83	23.02	5

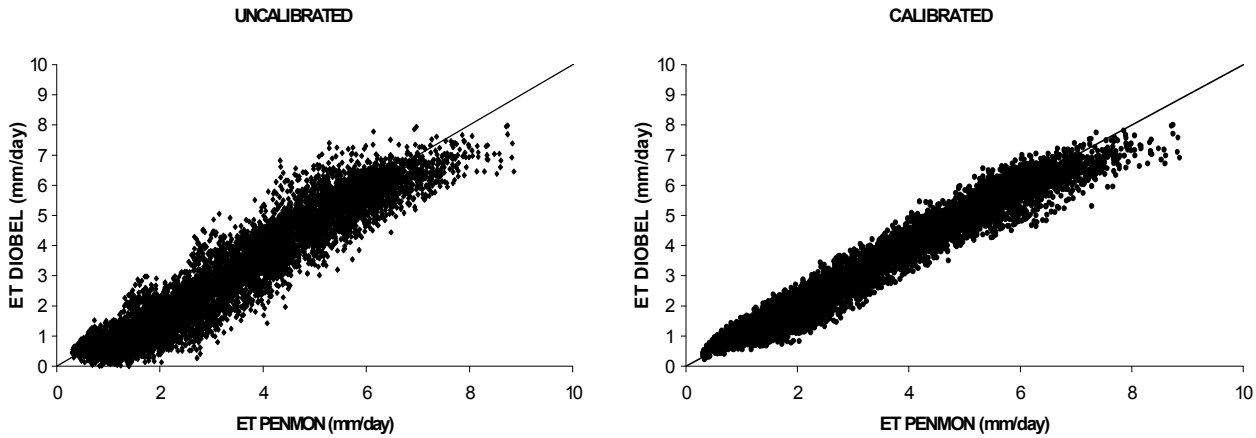


Figure 1.Uncalibrated or calibrated Diodato Bellocchi Evaportranspiration ($ET_{DIOBELL}$) as a function of Penman Monteith Evaportranspiration (ET_{PENMON})

Table 2. Slopes of the trends of Precipitation (b PR) and Diodato Bellocchi Evapotranspiration (b ET_{DIOBEL}), during the periods 1950-2001, 1958-2001 and 1986-2001, for 16, 22 and 29 stations, respectively. Slopes of the trends of Penman Monteith evapotranspiration (b ET_{PENMON}), during the period 1986-2001, for 29 stations.

	1950-2001		**1958-2001**		**1986-2001**	
Stations	**bPR**	**bET_{DIOBEL}**	**bPR**	**bET_{DIOBEL}**	**bPR**	**bET_{DIOBEL}**
	(mm/y^2)	**(mm/y^2)**	**(mm/y^2)**	**(mm/y^2)**	**(mm/y^2)**	**(mm/y^2)**
Florina	**-4.26**	0.13	**-6.67**	0.38	4.43	2.16
Kavala	**-12.62**	**-1.67**	**-20.47**	**-2.61**	5.99	**3.03**
Alex/poli	**-3.98**	0.17	**-4.32**	0.53	3.69	2.53
Kozani	**-3.36**	0.47	**-5.33**	**0.94**	-5.28	3.74
Kerkira	**-9.65**	**0.57**	**-10.30**	**1.08**	14.91	1.50
Ioannina	**-7.49**	-0.24	**-7.78**	0.01	-15.35	2.01
Larisa	**-2.26**	-0.32	-0.23	0.39	-2.63	**5.20**
Mitilini	-1.84	**0.98**	-3.20	**1.54**	12.74	**5.51**
Agrinio	-4.85	-0.39	**-7.28**	0.43	4.81	**4.78**
Argostoli	**-9.72**	-0.43	**-12.57**	-0.15	-5.68	**1.95**
Tripoli	**-3.76**	-0.22	**-5.15**	0.34	-10.40	-2.07
Methoni	**-3.93**	0.06	-2.65	0.43	-11.58	**1.84**
Milos	-1.15	**1.24**	-0.17	1.81	2.154	8.20
Kithira	-2.11	**0.63**	-1.59	0.51	-6.98	3.89
Rodos	**-5.12**	-0.01	-3.61	0.30	5.05	2.81
Irakleio	-0.68	0.31	-2.08	-0.02	5.92	**4.06**
Mikra			**-2.04**	**0.85**	-7.16	**5.67**
Aghialos			**-3.39**	**0.90**	-1.22	4.29
Andravida			-5.74	-0.23	-2.55	**2.32**
Elliniko			0.12	**1.97**	0.39	7.46
Naxos			-0.33	**1.98**	3.86	3.22

Tymbaki	-2.37	-0.24	-4.65	3.40
Serres			-0.68	**3.32**
Lamia			5.42	4.97
Chios			3.80	2.67
Pirgos			1.66	2.61
Kalamata			2.69	3.05
Ierapetra			-13.42	**5.12**
Sitia			-9.78	**3.85**

Table 3.Number of stations with positive or negative slopes of the trends of Precipitation (b PR) and Diodato Bellocchi Evapotranspiration (b $ET_{DIOBELL}$) for the periods 1950-2001, 1958-2001 and 1986-2001 for16, 22 and 29 stations, respectively.

	bET_{DIOBEL}		**bPR**	
	increace +	**decreace -**	**increace +**	**decreace -**
1950-2001	9(4)	7(1)	0	16(11)
1958-2001	17(7)	5(1)	1(0)	21(11)
1986-2001	28(12)	1(0)	15(0)	14(0)

Table 4. Slopes of the trends of Diodato Bellocchi (bET_{DIOBEL}) and Penman Monteith (bET_{PENMON}) evapotranspiration, precipitation (bPR), number of rainy days (bRdays) and, dry- (bT), maximum- (bTmax) and minimum- (bTmin) air temperature for the periods 1958-2001, 1974-2001 and 1986-2001 for 22 stations considered 'as a whole'.

Parameter	**1958-2001**	**1974-2001**	**1986-2001**
bET_{DIOBEL} (mm/y^2)	0.60	2.58	3.70
bET_{PENMON} (mm/y^2)		1.34	3.11
bPR (mm/y^2)	-4.79	-3.97	-0.55
bRd (month y)$^{-1}$	-0.49	-0.38	-0.13
bT (^{0}C/y)	**~0**	0.03	0.06
bTmax (^{0}C/y)	**~0**	0.03	0.05
bTmin (^{0}C/y)	0.01	0.04	0.06

Table 5. Slopes of the trends of Diodato Bellocchi (b ET_{DIOBEL}) and Penman Monteith (bET_{PENMON}) evapotranspiration, precipitation (bPR), number of rainy days (bRdays) and dry- (bT), maximum- (bTmax) and minimum- (bTmin) air temperature for the periods 1974-2001 and 1986-2001 for29 stations considered 'as a whole'.

Parameter	**1974-2001**	**1986-2001**
bET_{DIOBEL} (mm/y^2)	2.57	3.99
bET_{PENMON} (mm/y^2)	1.50	3.16
bPR (mm/y^2)	-4.22	-0.46
bRd (month y)$^{-1}$	-0.39	-0.02
bT (^{0}C/y)	0.03	0.06
bTmax (^{0}C/y)	0.03	0.06
bTmin (^{0}C/y)	0.03	0.06

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CLIMATE CHANGE AND WATER FOR AGRICULTURE: IMPACTS – MITIGATION – ADAPTATION

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Abstract: The aim of this paper is to briefly present the main effects of climate change and variability on water resources with respect to water availability for agriculture in the Mediterranean region. The projected increase of temperature and decrease of precipitation, combined with the more frequent occurrence of extreme events, such as drought, will have a great impact on agricultural production and the economies of the Mediterranean region. Results of undertaken studies in Greece, in precipitation patterns and drought assessment are presented. Based on a precipitation frequency analysis, evidence of precipitation reductions is shown. Calculated values of a drought index, namely the Reconnaissance Drought Index (RDI), for drought assessment in Thessaly are delineated. The importance of the formulation of adaptation strategies is highlighted and an adaptation scheme for agriculture in water scarce areas is presented.

Key words: climate change, agriculture, drought

1. INTRODUCTION

The observed combined increase of temperature and the decrease of precipitation in areas in southern Europe where already water scarcity is at place will have profound effects on land uses and the economy. Semi-arid and arid areas are particularly exposed to the impacts of climate change on freshwater. Climatic projections for the Mediterranean basin indicate that the area will suffer a decrease in water resources due to climate change. The key climatic trends identified for the Mediterranean region (IPCC, 2007) are continuous temperature increase, further drying with precipitation decrease in the order of 20% and the accentuation of climate extremes, namely floods, drought, heat waves and forest fires. With higher temperatures, the water-holding capacity of the atmosphere and evaporation into the atmosphere increase, and this favours increased climate variability, with more intense precipitation and more droughts. Also, changes in temperature, radiation, atmospheric humidity, and wind speed affect potential evapotranspiration, and this can exaggerate further the effect of decreased precipitation on surface waters. Efforts to offset declining surface water availability due to increasing precipitation variability may also be hampered by the fact that groundwater recharge will decrease considerably in some already water-stressed regions, where vulnerability is often exacerbated by increases in population and water demand (IPCC, 2007).

Irrigated agriculture is globally the largest water-use sector. Regionally, large changes in irrigation water demand as a result of climate change are expected. Current water management practices are very likely to be inadequate to reduce the negative impacts of climate change on water supply reliability, flood risk, health, energy, and aquatic ecosystems. Irrigation water withdrawals account for almost 70% of global water withdrawals and 90% of global consumptive water use (the water fraction that evapotranspires during use) (Shiklomanov and Rodda, 2003). Of all sectoral water demands, the irrigation sector will be affected most strongly by climate change, as well as by changes in the effectiveness of irrigation methods.

This paper presents selected aspects of the climate change and variability influence on water resources, focusing on the impacts concerning water and agriculture in the Mediterranean region, along with a review of selected climate change mitigation and adaptation actions and measures,

with concern to water and agriculture. In this context, results of studies are briefly presented, concerning precipitation trends and patterns, drought and evapotranspiration monitoring and assessment in Greece, focusing particularly on the plain of Thessaly in central Greece, where agriculture is the dominant land-use and water consumer.

2. CLIMATE CHANGE AND WATER AVAILABILITY FOR AGRICULTURE

Global climate change will affect all economic sectors to some degree, but the agricultural sector is perhaps the most sensitive and vulnerable (Alexandrov and Hoogenboom, 2000). World agriculture, whether in developing or developed countries, remains very dependent on climate resources. The impact of climate variability on agricultural production is important at local, regional, national, as well as global scales. Agriculture of any kind is strongly influenced by the availability of water. Climate change will modify rainfall, evaporation, runoff, and soil moisture storage patterns. Changes in total seasonal precipitation or in its pattern of variability are both important. Crop yields are affected by variations in climatic factors such as air temperature and precipitation, and the frequency and severity of extreme events like droughts, floods, hurricanes, windstorms, and hail (Alexandrov and Hoogenboom, 2000).

Agriculture is already the largest consumer of water resources in semi-arid regions (Yeo, 1999). Present water consumption for agriculture is 60% for the whole Mediterranean basin, and about 80% or more in the southern, eastern and north-eastern parts. Transformations in agriculture (expansion of cultivation to southern semi-arid areas where pastoralism was usual, depopulation of remote mountains) and land-use intensification (overgrazing, land degradation, irrigation) are major changes (Cuddenec, 2007). In Greece, 84% of the total annual demand for water under the present conditions in all water districts is allocated for irrigation purposes, with the remaining available water being potable water (12%), allocated for industrial and energy purposes (3%) and for livestock (1%) (NTUA, 2008). Cases of groundwater overexploitation combined with inefficient use of energy for pumping from great depths, have also been reported in agricultural areas in Greece, where they cause additional problems such as extensive soil subsidence and deterioration of water quality in aquifers due to agricultural chemicals (NTUA, 2008).

Irrigation has always been important for crop production in most Mediterranean countries due to high evapotranspiration rates and restricted rainfall inputs. The demand for water for irrigation is projected to rise in a warmer climate, increasing the competition between agriculture and urban as well as industrial users of water (Olesen and Bindi, 2002). More water will be required per unit area under drier conditions, and peak irrigation demands are also predicted to rise due to more severe heat waves.

The predicted increase in extreme weather events, e.g., spells of high temperature and droughts, is expected to increase yield variability and to reduce average yield (Olesen and Bindi, 2002). In particular, in the European Mediterranean region, increases in the frequency of extreme climate events, such as spells of high temperature, heavy storms or droughts, during specific crop development stages (e.g., heat stress during flowering period, rainy days during sowing time), together with higher rainfall intensity and longer dry spells, are likely to reduce the yield of summer crops (e.g., sunflower). Cereal yields are also likely to be reduced as they are limited by water availability, heat stress and the short duration of the grain-filling period. Where certain varieties of crops are grown near their limits of maximum temperature tolerance, heat spells can be particularly detrimental (IPCC, 2007). Permanent crops (olive, grapevine, fruit trees etc.) are also very important in this region and are affected by extreme weather events (such as hail and storms), which can reduce or completely destroy yield.

3. STUDY AREA AND DATA

The results presented in this paper will focus on studies concerning the region of Thessaly, where agriculture is the main water consumer. Thessaly lies in central - eastern continental Greece and characterized by a highly variable landscape. The terrain is such that tall mountains surround the plain, which is among the biggest in Greece. At the western side the climate is continental; the winters are cold and the summers are hot and the temperature difference between the two seasons is large. At the eastern side the climate is typical Mediterranean climate. Summers in Thessaly are usually very hot and dry, and in July and August temperatures can reach 40 °C. Mean annual precipitation over the whole Thessaly region is about 700 mm and it is distributed unevenly in space and time. The mean annual precipitation varies from about 400 mm at the central plain area to more than 1850 mm at the western mountain peaks. Generally, rainfall is rare from June to August. The mountain areas receive significant amounts of snow during the winter months and transient snowpacks develop.

The Thessaly plain is one of the major agricultural areas of Greece. Total water availability is about 3.209 hm and consists of 2.596 hm^3 surface water and 613 hm^3 groundwater. The amount of the water used for irrigation purposes accounts for about the 96% and for water supply about the 3,3 % of the total water consumption in the area.

Data used for the studies presented in this paper include conventional meteorological data (temperature and precipitation) from ground stations of the National Meteorological Service network. Meteorological data for the region of Thessaly were taken from the meteorological station of Larissa.

4. IMPACTS

4.1 Impacts on precipitation patterns

Studies show increase in the frequency of droughts for almost most parts of Europe. The regions most prone to an increase in drought risk are the Mediterranean (Portugal, Spain) and some parts of central and eastern Europe, where the highest increase in irrigation water demand is projected (Donevska and Dodeva, 2004). Studies of changes in precipitation extremes in Europe (Giorgi et al., 2004) agree that the intensity of daily precipitation events will predominantly increase, also over many areas where means are likely to decrease. The number of wet days in Europe is projected to decrease (Giorgi et al., 2004), which leads to longer dry periods except in the winters of western and central Europe.

4.2 Precipitation Frequency Analysis in Greece

Studies on the impacts of climate variability and change to water availability investigate the trends of precipitation and evapotranspiration rates. Bampzelis et al. (2006) used precipitation time series from five meteorological stations evenly distributed in Greece, and investigated changes in the range and frequency of daily precipitation for two separate periods in the dataset by calculating precipitation frequency diagrams for each period separately. The climate in Greece varies from humid in the north-western regions to dry in the eastern regions of the country. The country falls into five main climatic types: West Mediterranean Sea region; Mountainous area; Mainland area; Desert type Mediterranean region; and East sea region. The availability of water resources within the area covered by each climatic category is proportional to the general precipitation pattern.

Data used in this study include daily values of temperature and precipitation covering a period of 46 years (1955-2000), and were taken from five meteorological stations in Greece (Methoni, Larissa, Iraklion, Corfu and Elliniko in Athens), which are representative of the main climatic categories. The precipitation data were divided into nine classes ranging from light to heavy and

extreme precipitation events, and the number of precipitation events per class was calculated (Bampzelis et al., 2006).

Based on calculated temperature diagrams using daily temperature records, it was observed that the temperature trend for all stations, for the period 1955-1975 is declining while for the period 1976-2000 is increasing. Thus, the study was separated into two periods, 1955-1975 and 1976-2000. Since the two periods differ in their duration, the precipitation frequencies were normalized.

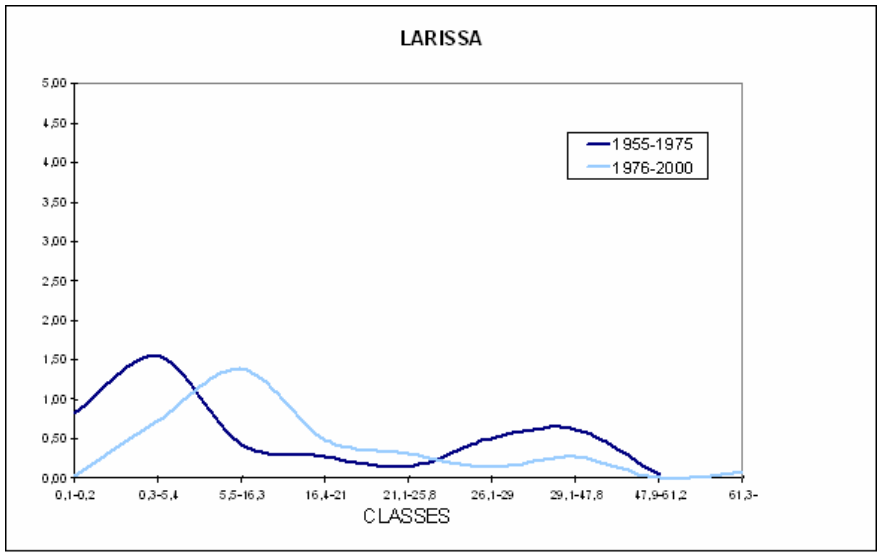


Figure 1. Precipitation Frequency episodes for Larissa station.

The results of the frequency analysis showed that the whole study period (1955-2000) was characterized by small fluctuations in precipitation episodes. A small decline in precipitation was observed, especially in the western part of Greece (Corfu and Methoni stations) (Bampzelis et al., 2006). Concerning the Thessaly plain, there were indications of a shift towards higher precipitation intensities for the second period (after 1975). In particular, precipitation of a slightly greater volume but less frequent was observed in the Larissa station for the period 1976-2000 compared to the previous period (1955-1975) (Figure 1). This fact indicates a trend for less precipitation episodes but of more intensity causing floods and greater loss of water. Reduction of precipitation inputs and decline of water availability could adversely affect future crop production in the area and consequently the income of the agricultural population.

5. MITIGATION

Mitigation plans for water shortages in agriculture have three essential components, relating to provincial, national or regional scale, and include early warning systems, risk assessments and mitigation actions (Klein et al., 2005). Examples of mitigation actions include spatial planning for land use and land use change and initiatives for water distribution systems for crop irrigation that can be implemented at different scales. Risk management tools focusing on agriculture include drought mapping, development of models and methods, drought assessment and forecasting and earth observation in support for risk management technologies.

5.1 Drought Assessment in Thessaly

An example of an early warning system for agriculture is the implementation of the annualised Reconnaissance Drought Index (RDI) (Tsakiris et al., 2007) used for the identification and monitoring of the 2007 drought in Greece, focusing on the Thessaly plain.

Drought is a temporary variation from normal conditions, unlike aridity, which is a permanent feature of climate and occurs in both high and low rainfall areas and virtually all climate regimes.

Droughts can be characterized by their severity, frequency and duration, and areal extent. Practices for water resources management and plans for drought mitigation need an assessment of drought conditions in a particular area and its spatial-temporal variability (Dalezios et al., 2000). Because drought is a very complex phenomenon several indices have been developed describe its intensity and severity and the potential impacts (Loukas et al., 2002). In the results presented here, RDI was used for the drought assessment and monitoring and the description of the occurrence or not of meteorological drought in the region of Larissa in Thessaly. RDI is a station-based index, and was calculated using monthly cumulative precipitation and mean monthly temperature values for the hydrological years, from 1955 to 2002, taken from the meteorological station of Larissa (Kanellou et al., 2008).

The RDI is based on the ratio between two aggregated quantities of precipitation and potential evapotranspiration. The initial value of the index for a certain period, indicated by a certain month (k) during a year, is calculated by the Equation:

$$a_k = \sum_{j=1}^{j=k} P_J / \sum_{j=1}^{j=k} PET_j \quad (1)$$

where, P_j and PET_j are the precipitation and the potential evapotranspiration of the j month of the hydrological year. The RDI could be calculated for any period of the year. It can also be started from any month of the year. In Mediterranean, the hydrological year usually begins on October, so for this month k=1. The α_k parameter of RDI is advisable to be calculated for 3, 6, 9 and 12 month time steps. The Normalized RDI (RDI_n) is computed using the Equation:

$$RDI_n(k) = ((a_k - \overline{a_k}) - 1 \quad (2)$$

where, $\overline{a_k}$ is the average value of α_k for each time period. The Standardized RDI (RDI_{st}) is computed using the Equation:

$$RDI_{st}(k) = y_k - \overline{y_k} / \hat{s}_k \quad (3)$$

where, y_k is the ln α_k, $\overline{y_k}$ its arithmetic mean and $\hat{s}_k$ its standard deviation.

The conventional precipitation dataset from the Larissa station was analysed in order to extract the RDI_{st} for the 1955-2002 period. RDI_{st} values range from 4 and more to -3.99, representing extreme wet weather conditions to severe drought, respectively. Annual calculated RDI_{st} values for the Larissa station are shown in Figure 2.

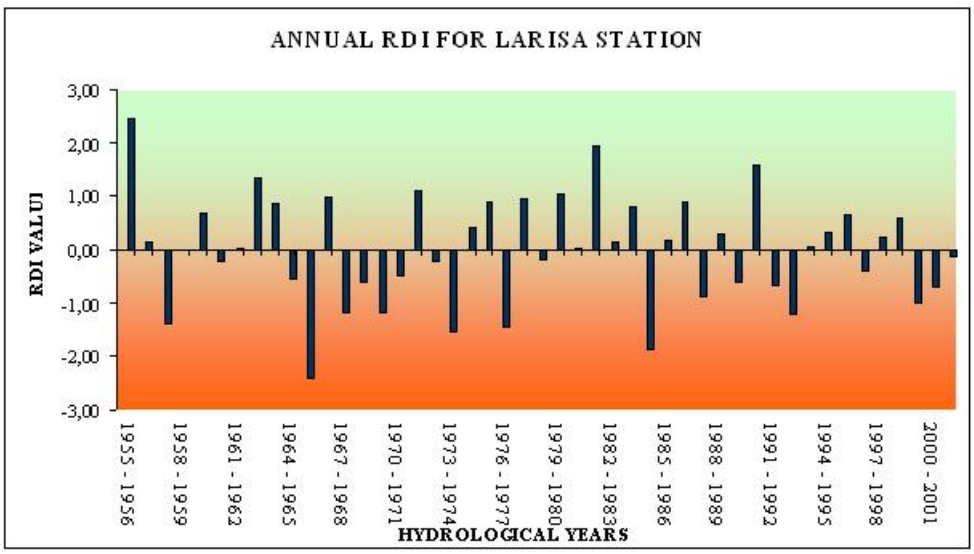


Figure *2. Annual calculated RDI_{st} values for the Larissa station*.

The analysis of the results indicates that Thessaly and, specifically the Larissa region, suffered from extreme drought events mainly during the 1966-67 and 1985-86 hydrological years. However, several years of the dataset indicate severe drought conditions (e.g. 1957-58, 1974-75, 1977-78, 1993-94). In many cases, the phenomenon appears persistence up to one or two sequential years, indicating moderate or mild drought. These conditions seem to appear more often over the last decades. The results indicate that the RDI is able to identify the occurrence of drought and quantify drought intensity using readily-available precipitation data. Thus, drought indices such as RDI can be used in mitigation plans and risk management strategies for water in agriculture, especially in the areas of most importance for agriculture, such as Thessaly in Greece. Future research aims to investigate the use of meteorological satellite imagery for the calculation of drought indices, in order to assess the spatial distribution and extent of drought severity for large areas.

6. ADAPTATION

6.1 Adaptation concepts

Adaptation aims at reducing the risk of damage from current and future harmful impacts cost-effectively and by exploiting potential benefits. Assessing and adapting to the impacts of climate change requires balancing social, economic, and environmental factors in the context of an ever-expanding range of objectives, uncertainties, and management options (Pyke et al., 2007), aiming at developing comprehensive and integrated methodologies for the assessment of the impacts, vulnerabilities and cost-effective adaptation. Synergies between adaptation and mitigation measures are essential provided that there are no contradictory conflicts between the mitigation and adaptation measures and their influence on different sectors.

The diversity of climate change impacts requires a range of different adaptation measures specific to each impact. In general, adaptation measures are categorised as “soft” (low intensity-low cost) and “hard” (technical-high cost) (Hallegatte, 2009). Concerning water shortages in dry regions, the “hard” measures include the construction of reservoirs and dams, water supply and water diversion infrastructure and desalination plants. These are long-term investments with high potential for tackling with water shortages in dry regions, but are costly and their success relies on the level of integration of climate predictions and climate uncertainty in the development and design stages.

Thus, technical solutions cannot be considered as the only way of adapting to changing climates, since “soft” adaptation measures and institutional and financial tools can also be efficient. Focusing on agriculture, crop production is very sensitive to water availability. However, where annual precipitation will decrease, problems in the agriculture sector may come first from extreme events in water scarcity (e.g., long droughts) rather than from the decrease in average water availability. In the Mediterranean region, average yields are expected to decrease, but most of the concern is about the recurrence of extreme droughts with disastrous consequences (Hallegatte, 2009). For example, the construction of water reservoirs will be efficient if climate change remains moderate, but cannot cope with the most pessimistic projections.

6.2 Adaptation scheme for water use in agriculture

“Soft” adaptation measures for water availability for agriculture include efficient use of scarce water in dry regions, enhancement of drought early warning systems, integration of drought preparedness in relevant policies across sectors and levels of the society, and the development and introduction of drought-resistant crops.

Based on the results of undertaken studies in Greece (e.g. Tsiros et al., 2008; Tsiros et al., 2009), an adaptation scheme for agriculture is proposed for agricultural areas focusing on the plain of Thessaly, integrating "soft" measures: a) changes in sowing and planting dates based on crop phenology and temperature trends analyses using satellite data for depicting the spatial variability and extent; b), crop rotations and selection and introduction of drought tolerant crops based on agroclimatic zoning; c) changes in irrigation requirements by water conservation, through estimated evapotranspiration on a daily basis using satellite data; and d) changes in fertilisation and reduction of inputs by incorporating environmental indicators.

Results from studies such as the on-going EU-funded PLEIADES ((Participatory multi-Level EO-assisted tools for Irrigation water management and Agricultural Decision-Support) project, can contribute significantly to the formulation of the adaptation scheme.

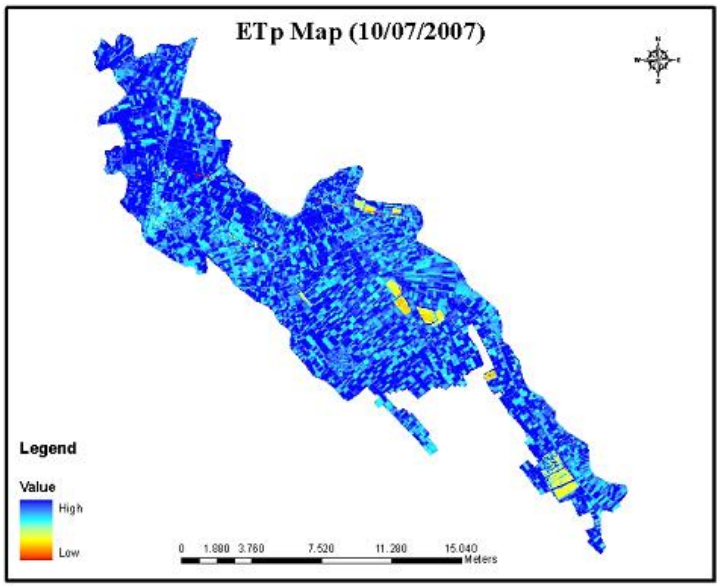


Figure 3. Range of calculated ETp using satellite imagery from a pilot area in Thessaly.

Monitoring water needs for agriculture in Thessaly, in the context of the PLEIADES project, requires a combination of field observations (crop characteristics, crop demands in water, fertilizer, pesticides and energy use) in pilot areas, micro-meteorological data and analysis of satellite data. Ground based micro-meteorological measurements include air temperature, wind speed, humidity, and precipitation on daily basis, and the data are used to calculate reference evapotraspiration (ET_0). The spatial distribution of evapotranspiration is assessed using satellite imagery (Landsat-5 TM) covering the study area. Following geometric an atmospheric correction and in conjunction with field crop data and reflectance and evapotranspiration equations, the imagery was used to calculate reflectance, NDVI, fractional cover, crop coefficient (K_c) and range of potential evapotranspiration (ET_p) (Figure 3) maps over the Thessaly plain.

The PLEIADES project is on-progress and the results are expected to contribute significantly in the formulation of adaptation strategies for sustainable water use and irrigation management in areas with intense agriculture.

7. CONCLUSIONS

Climatic projections indicate increases in temperature and decreases in precipitation, combined with the more frequent occurrence of extreme events, such as drought, in the Mediterranean region. Agriculture is expected to be most influenced by the adverse impacts of climate change and variability, with high probability of crop production reductions, due to less availability of water for irrigation. In order to reduce the risk of climate change impacts on agriculture, comprehensive methodologies need to be developed, integrating the results of studies on drought and

evapotranspiration assessment and monitoring, for the formulation of cost-effective mitigation measures and adaptation strategies.

ACKNOWLEDGEMENTS

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HYDROLOGIC RESPONSES OF CLIMATE CHANGE ON THE KARKHEH RIVER BASIN

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Abstract: Future of the Karkheh River Basin and its people's livelihoods clearly depends on natural resources like water, soil, vegetation and livestock. As water is the most limiting natural resource in this basin, any increase in water productivity will almost certainly benefit rural livelihoods. To ensure the sustainability of the improvements in water productivity, assessment of the possible impacts of climate change on hydrology and water resources in the basin is necessary.

In this study the potential impacts of climate change on hydrology in the Karkheh River Basin were assessed using a macroscale hydrology model driven by 21st century simulations of temperature and precipitation downscaled from runs of CGCM2 model with two emissions scenarios (A2 and B2) for 3 periods: (2010-2039), (2040-2069), (2070-2099). At first, the monthly precipitation and temperature output of each GCM was bias-corrected and statistically downscaled to a 1/2° grid using a statistical technique. Then, we forced a hydrologic model using the downscaled data to generate streamflow at strategic points. The hydrologic model used in this study was the variable infiltration capacity (VIC) model. The model was run at a daily time step at a 1/2-degree resolution over the Karkheh River Basin. The downscaled data from each of 2 projected climates were used to force the land surface hydrology model to simulate hydrologic responses in the Karkheh River Basin, which produced projected streamflow at inflow points to major reservoirs.

As the results of this study, by 2070-2099 the median warming relative to 1961-1990 was 1.3°C and 1.9°C under B2 and A2 emissions, respectively. For the same periods, the models project median precipitation increases of 7% (B2) and increases of 11% (A2). Median changes by 2070-2099 in reservoir inflow were 15% (B2) and 21% (A2), with largest flow reductions during the rising limb of the seasonal hydrograph, from April through September.

Key words: Climate Change; Downscaling; VIC Model; Hydrologic Impacts; Karkheh River Basin

1. INTRODUCTION

Climate change impacts on hydrology and water resources have been widely recognized and a variety of regional assessments of such impacts have been done in many countries over the past two decades. The intensification of the hydrological cycle anticipated as global warming continues will manifest itself distinctly in different regions (Stocker, 2001; Trenberth, 1999). This effect has already been observed as a global phenomenon, with generally increasing precipitation at mid- to high-latitudes and decreasing precipitation in the sub-tropics (Folland and Karl, 2001). Worldwide precipitation is reported to have increased about 2 percent since 1900. While global average precipitation has been observed to increase, changes in precipitation over the past century vary in different parts of the world. Some areas have experienced increased precipitation while other areas have experienced a decline (NOAA, 2005). The cumulative effects of warming and precipitation changes are integrated by watersheds to produce changes in intensity, duration, and frequency of both droughts and floods.

In this study, we assessed the hydrologic impacts of projected climate change on the Karkheh River Basin (KRB). We used projections of CGCM2 model, under a higher (A2) and lower (B2) greenhouse gas emissions scenario. We tried to address the following questions: First, what are the projected changes in precipitation and temperature for the KRB under higher and lower emissions scenarios? Next, what are the impacts on projected inflows to major reservoirs on the KRB? In order to answer the first question we had to have downscaled data in KRB. Unfortunately no previous research had been done on KRB in order to make categorized data, so we had to downscale GCM output ourselves.

2. STUDY AREA

Karkheh River Basin is, most notably, the eastern flank of the "cradle of civilization" (ancient Mesopotamia). This major river basin of western Iran has unique agricultural and hydrological aspects; but also much in common with other catchments in developing countries around the world: debilitating rural poverty and land degradation, low water and agricultural productivity, a dry climate, and growing upstream-downstream competition for water.

Changes in land use patterns in recent decades, especially overgrazing and conversion of natural pastures to rainfed cropping, have taken a heavy toll. Ninety percent of the upper watershed's rangelands and 70 percent of its forests have been significantly degraded. At the same time, excessive use of fertilizers and pesticides is polluting groundwater, putting human health at serious risk and because of that surface water resources reflect the excesses of inappropriate farming practices. Sustainable water management in irrigated-dominated river basins attempts to ensure a long-term, stable, and flexible water supply to meet crop water demands, as well as growing municipal and industrial water demand, while mitigating negative environmental consequences.

The future of the Karkheh River Basin and its people's livelihoods clearly depends on more holistic, basin-wide management and monitoring of natural resources—water, soil, vegetation and livestock. As water is the most limiting natural resource in this basin, any increase in water productivity will almost certainly benefit rural livelihoods. To ensure the sustainability of the improvements in water productivity, assessment of the possible impacts of climate change on hydrology and water resources in the basin is necessary. As a result of population growth and economic development, water demand and urbanization is increasing in the basin. Both the climatic and non-climatic changes are dynamic. These changes complicate water resources management (i.e. water supply, flood management, drought management). Understanding the impacts of climate change on hydrology and water resources of the basin will help decision-makers to cope with these changes and manage water resources adaptively in the future. Figure 1 illustrates the position of Karkheh River Basin and the 20-cell grid-mesh that we have considered on the basin in order to make downscaled data in 0.5 x 0.5 degree scale and run VIC model.

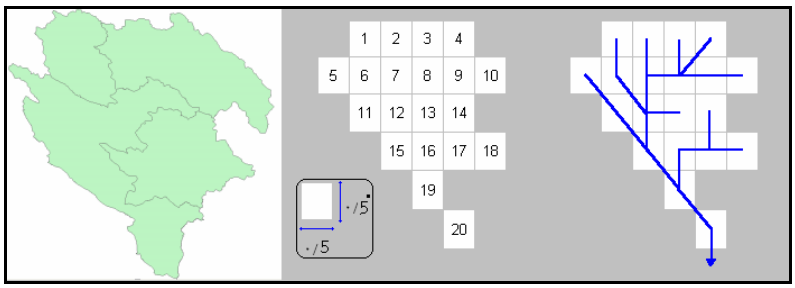


Figure 1. Karkheh River Basin and 20-cell grid-mesh

3. METHODOLOGY

In this study the potential impacts of climate change on hydrology in the KRB were assessed using a macroscale hydrology model driven by 21st century simulations of temperature and precipitation downscaled from runs of a GCM with two emissions scenarios (A2 and B2). At first, the daily precipitation and temperature output from each GCM was bias-corrected and statistically downscaled to a 1/2° grid using a statistical technique with SDSM model. After that, we forced a hydrologic model with the downscaled GCM data to generate streamflow at strategic points. The hydrologic model used in this study was the variable infiltration capacity (VIC) model. The model was run at a daily time step at a 1/2-degree resolution over the KRB. The downscaled data from each of two projected climates (CGCM2 model which had been run with two emissions scenarios) was used to force the land surface hydrology model to simulate hydrologic responses in the KRB, which produces projected streamflow at inflow points to major reservoirs in the basin.

The approach of this study is to produce ensembles of projected future climates through the 21st century. The consensus among GCMs for the projected changes in precipitation and temperature

were assessed for the KRB. Each of two projected climates is used to drive a distributed land surface hydrology model, which produces an ensemble of projected streamflow at inflow points to major dams in the KRB. Changes to the inflows are statistically analyzed to assess the confidence in various levels of projected changes in reservoir inflows.

Global Climate Model Simulations. All data were obtained from the IPCC date distribution center and Canadian center for Climate Modeling and Analysis data center. The emissions scenarios used in this study, A2 and B2, are described in detail by Nakicenovic et al. (2000). Each scenario produces different atmospheric concentrations of future greenhouse gases. While A2 does not represent the highest CO2 emissions (at least through 2100) of the SRES scenarios (IPCC, 2001), it is the highest emission scenario for which most modeling groups have completed simulations. As such, although it is by no means a "worst case," A2 does represent the higher emission case in this study. B2 generally does not represent the lowest CO2 emissions (at least through 2100) of the SRES scenarios (IPCC, 2001); it is the lowest emission scenario which used to run CGCM2.

Because the spatial scale of GCMs output is too large to characterize climate over small areas like the KRB, some type of downscaling was necessary. The monthly precipitation and temperature output from each GCM was bias-corrected and statistically downscaled to a 1/2° grid using the SDSM (statistical downscaling model) and an empirical interpolation aggregation method. The downscaled data are used to force the land surface hydrology model to simulate the hydrologic response of the KRB to the ensemble of future climate projections.

Downscaling approach. Downscaling approaches have subsequently emerged as a means of relating largescale atmospheric predictor variables to local-scale or station-scale meteorological series (Xu, 1999). This step is a Data Generation Step in Climate Change Studies. Even if global climate models in the future are run at high resolution there will remain the need to downscale the results from such models to individual sites or localities for impact studies (DOE, 1996; p34). Two downscaling methods are commonly used, dynamical and statistical downscaling (Xu, 1999):

1. Dynamic methods, involving the explicit solving of the process-based physical dynamics of the system (e.g., Giorgi and Mearns, 1991; Jones et al., 1995);

2. Statistical methods, that use identified system relationships derived from observational data (e.g., Wigley et al., 1990; Wilby, 1995; Hewitson and Crane, 1996)

Statistical downscaling methodologies have several practical advantages over dynamical downscaling approaches. In situations where low–cost, rapid assessments of localized climate change impacts are required, statistical downscaling (currently) represents the more promising option.

Statistical Down Scaling Model. The statistical techniques are embodied in the Statistical Downscaling Model (SDSM) developed by Rob Wilby et al., King's College, London. SDSM facilitates the rapid development of multiple, low-cost, single-site scenarios of daily surface weather variables under current and future regional climate forcing. Additionally, the software performs ancillary tasks of predictor variable pre-screening, model calibration, basic diagnostic testing, statistical analyses and graphing of climate data. SDSM is a hybrid of multiple regression and stochastic downscaling methods. SDSM enables the construction of climate change scenarios for individual sites at daily time–scales, using grid resolution GCM output. Observed data sets (predictands) are first regressed against a 'selection' of climate predictor(s) to develop regression equations. SDSM is said to be calibrated when the regression coefficients, explained variance, and standard error are within acceptable limits for each regression model. An internal random number weather generator takes the calibrated output and stochastically synthesizes a number of ensembles that are statistically related to the original output. (SDSM can synthesize up to 100 ensembles, 20 being the default number). The means of these ensemble simulations can then be used as the synthesized downscaled values. These synthesized values must be 'validated' to see how closely they replicate the actual data. Best fit is obtained by making adjustments to the two stochastic parameters (variance inflation and bias correction). The regression equations, once validated, can now be used to 'generate' future downscaled sets of these climate variables, using output from different GCM models and experiments. The significance of the downscaled variables is inferred

from the monthly, seasonal, and annual variation in explained variance (R2), and standard error (SE). (Lines, 2006)

The software reduces the task of statistically downscaling daily weather series into five discrete processes: (1) screening of predictor variables; (2) model calibration; (3) synthesis of observed data; (4) generation of climate change scenarios; (5) diagnostic testing and statistical analyses.

In this study, we applied historical data from 1960 to 1975 for model calibration and from 1976 to 2000 for model validation. Figure 2 illustrates calibration and validation of the SDSM model for the Karkheh River Basin.

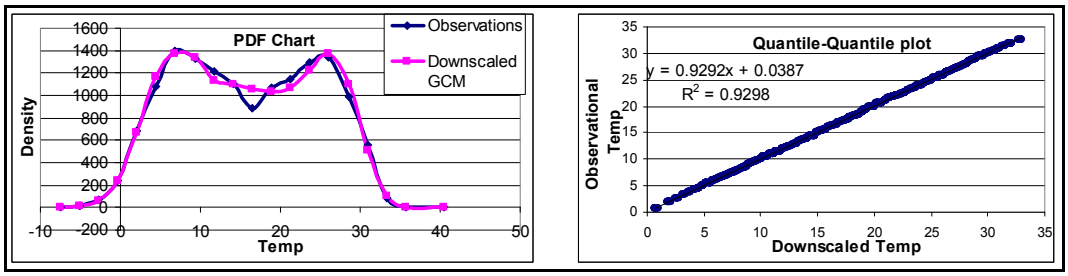


Figure 2: calibration and validation of SDSM for the KRB

4. Hydrology Modeling

The hydrologic model used in this study is the variable infiltration capacity (VIC) model (Liang et al., 1994). The VIC model is a macroscale semi-distributed, physically-based hydrologic model that balances both surface energy and water budgets over a grid mesh, typically at resolutions ranging from a fraction of a degree to several degrees latitude by longitude. The VIC model uses a "mosaic" scheme that allows a statistical representation of the sub-grid spatial variability in topography, infiltration and vegetation/land cover, which is important when simulating hydrology in heterogeneous terrain. The resulting runoff at each grid cell is routed through a defined river system using the algorithm developed by Lohmann et al. (1996).

The VIC model has been successfully applied in many settings, from global to river basin scale (Maurer et al., 2001; Nijssen et al., 1997; Nijssen et al., 2001b), as well as in many studies of hydrologic impacts of climate change (Christensen et al., 2004; Hayhoe et al., 2004; Nijssen et al., 2001a; Payne et al., 2004). For this study, the model was run at a daily time step at a 1/2-degree resolution (measuring about 3000 km2 per grid cell) over the KRB.

The VIC model was calibrated for the two locations for years 1960-1975, and validated for years 1976-2000. The correlation of simulated and monthly flow was relatively high, with a Pearson correlation coefficient of r=0.87. Note that, due to limited data availability, no adjustments have been made to the observed flow data to account for upstream diversions or other anthropogenic impacts.

In this study, we applied historical data from 1960 to 1975 for model calibration and from 1976 to 2000 for model validation. Figure 3 illustrates calibration and validation of the VIC model for the Karkheh River Basin.

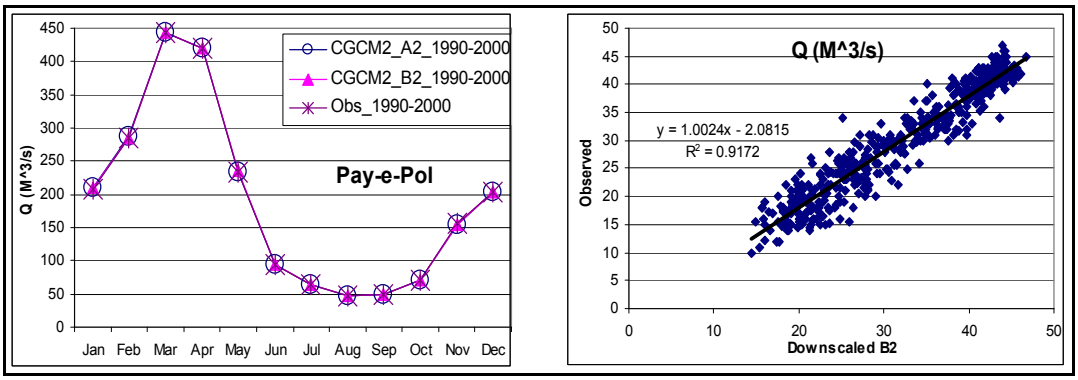


Figure 3: calibration and validation of the VIC model for the KRB

5. RESULTS AND DISCUSSION

As is common in climate change impact studies with an ensemble of GCMs (Christensen and Lettenmaier, 2007; Maurer and Duffy, 2005; Milly et al., 2002), we assume each of the GCMs produces an equally probable projection of future climate. While some studies of hydrologic impacts of climate change have examined skill-based weighting of future projections, the difference in outcomes from equal weighting has been found to be small (Dettinger, 2005; Wilby and Harris, 2006). Figure 4 illustrates the projected changes in temperature for 2 emission scenarios A2 and B2 on the Khorramabad station. Figure 5 illustrates the projected changes in precipitation for 2 emission scenarios A2 and B2 on the Kermanshah station.

The median projection of temperature changes between these periods varies from 1-1.3 °C under the B2 emissions scenario, and 1.5-1.9 °C under the A2 emissions scenario. The greatest warming is focused generally to the North and East of Karkheh River Basin.

Figure 6 illustrates the projected changes in temperature and precipitation, respectively, between the historic (1961-1990) and future (2070-2099) periods. The median precipitation projections for Karkheh River Basin in Figure 6 shows drying and wetting trends, with changes up to 20% in some areas. There is more severe drying under the lower B2 emissions scenario.

The seasonality of the changes in precipitation and temperature are non-uniform. For the A2 emissions scenario, precipitation raises through the 21st century in all of the basin, whereas under the B2 emissions scenario, through the 21st century in the east and north parts of the basin which is mountainous precipitation raises and in other parts it will decreases.

Figure 7 illustrates the impact of the projected climate changes on inflows to the downstream reservoirs, At Pay-e-Pol station just before the Karkheh dam inlet, under the B2 emissions scenario, annual average inflow (across the ensemble of GCMs) is projected to decline by 80 m3/s by 2010-2039 and by 150 m3/s by 2070-2099. Under the A2 scenario, the projected drop in annual average inflow is 60 m3/s by 2010-2039 and 120 m3/s by 2070-2099. By the end of the century, these drops represent 18% (B1) and 14% (A2) of total annual inflows at the Karkheh reservoir.

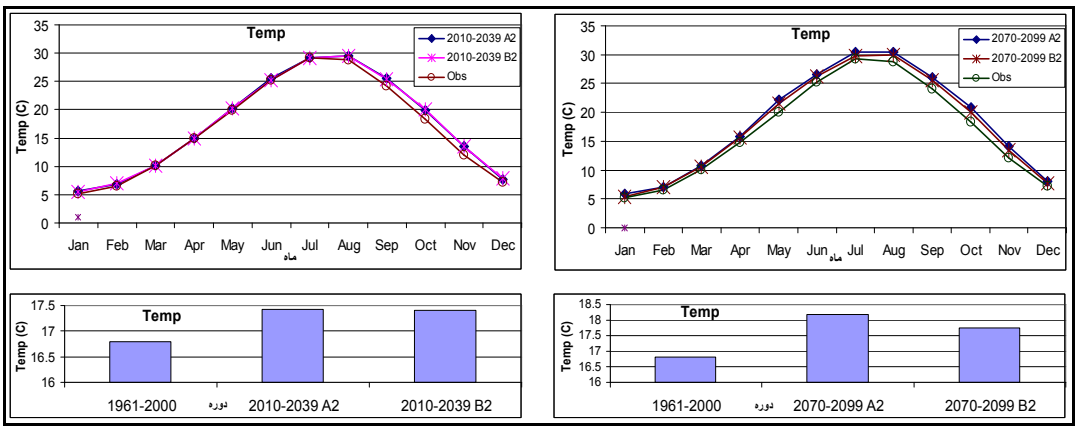


Figure 4: the projected changes in temperature for 2 emission scenarios A2 and B2 on the Khorramabad station.

All of the declines in reservoir inflows are statistically significant at very high confidence levels for March through September. Similar to precipitation projections, the GCM-based flow projections vary enough among GCMs for October and February that the confidence assigned to the changes is lower.

As noted above, for A2 emissions by 2070-2099, the median projection for the KRB was 11% increase in annual precipitation and 1.9°C average temperature increase, which produced a 15% reduction in annual average flow. Thus, the results obtained here are plausibly representative of the sensitivity of the hydrologic system to climate change.

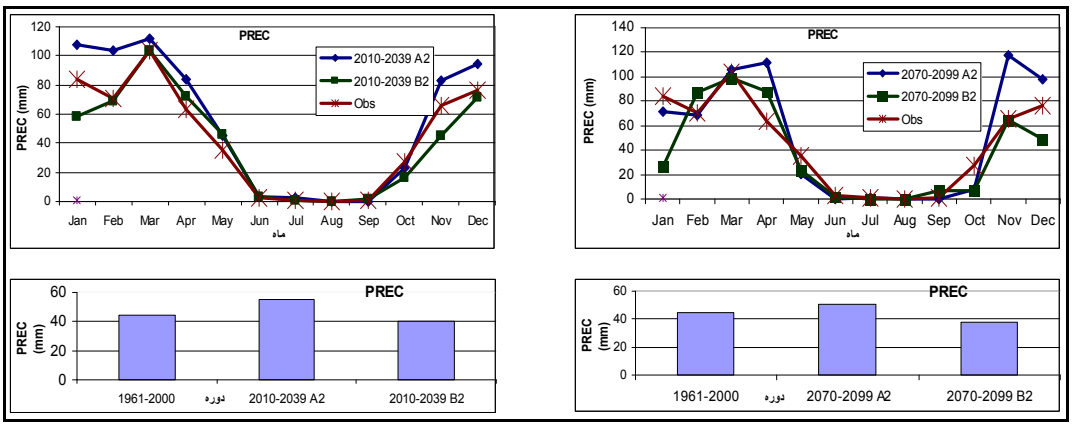


Figure 5: the projected changes in precipitation for 2 emission scenarios A2 and B2 on the Kermanshah station.

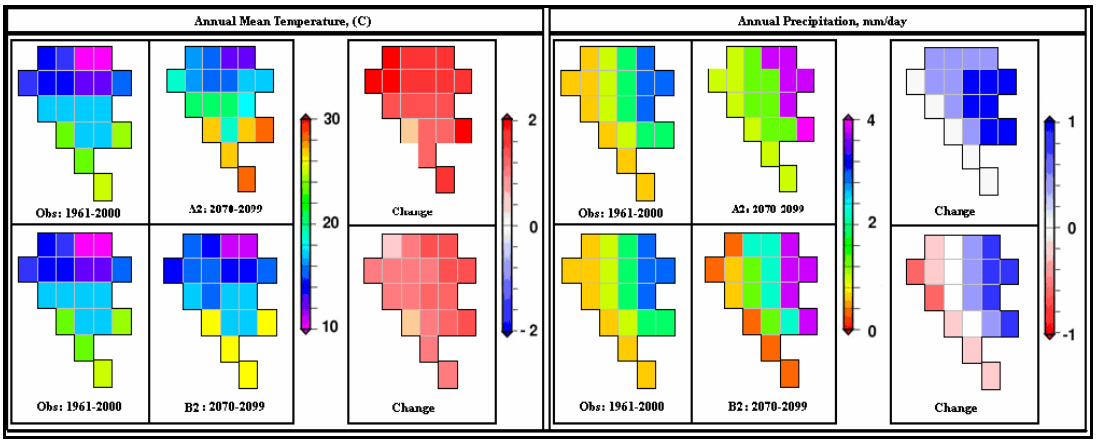


Figure 6: Projected temperature and precipitation change for Karkheh River Basin from 1961-1990 and 2070-2099 under higher (A2) emissions (upper row) and lower (B2) emissions (lower row).

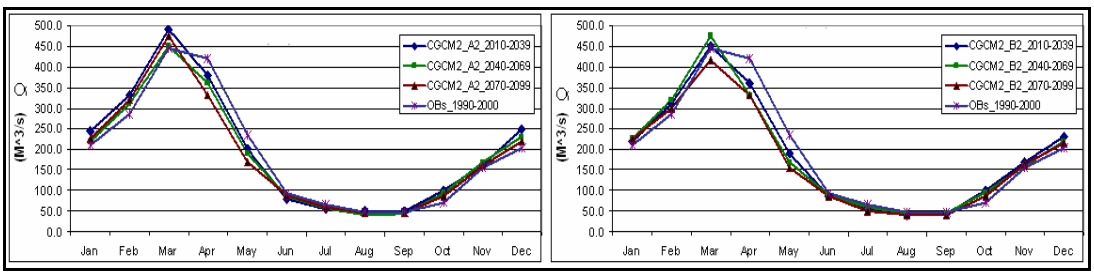


Figure 7: the impact of the projected climate changes on inflows to the downstream reservoirs, At Pay-e-Pol station

6. CONCLUSIONS

As noted by the recent IPCC Working Group II report (IPCC, 2007a), changes in temperatures and precipitation patterns will force many countries to adapt to inevitable changes in water supplies,

and the effectiveness of adaptation efforts depends in part on the availability of general information on vulnerable areas and projected impacts. This study provided a quantitative, probabilistic assessment of potential changes to the hydrology of the Karkheh River Basin. The study incorporated climate projections by CGCM2 model using both a lower and a mid-high emissions scenario, and used these to drive a distributed hydrology model to estimate streamflow impacts.

We find that by the end of the 21st century for the Karkheh River Basin:

- Average temperatures will rise from 1.3-1.9°C, with the greatest increase in June-July.
- The consensus of GCM indicated a wetter future, with an overall reduction in precipitation of 7-11%.
- The majority of the drop in precipitation will occur in December-March, corresponding to the first half of the rainy season.
- Inflows to the major reservoirs will decline on average by 15-21%.
- Peak declines in reservoir inflow will occur in Jun-August.

In all cases, the most severe impacts occur under the higher emissions A2 scenario, and are roughly a factor of two greater than the impacts under the lower B1 emissions scenario.

Acknowledgements. We acknowledge Robert Wilby and Christian Dawson, co-designers of the SDSM model. We acknowledge the modeling group (Canadian Center for Climate Modeling and Analysis) for making their simulations available for analysis. We acknowledge Prof. Lettenmaier laboratory University of Washington because of VIC model that have been available in their site for analysis and their helps and guides, specially we acknowledge Ted Bohn who answered our questions about VIC model and its way of running. We acknowledge Eleonora Demaria because of her helps during the study with VIC model. We acknowledge International water management institute - Integrated Database Information System (IWMI-IDIS) because of all meteorological datasets they have given us about Karkheh River Basin.

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URBAN IRRIGATION: A PRODUCTIVE USE OF WATER OR AN OPTIONAL EXTRA?

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Abstract: Australian water policy makers commonly consider urban irrigation water use to be more discretionary than indoor water use. Consequently compulsory restrictions on irrigation water use are universally used by urban water utilities as an acceptable means to reduce demand at times of drought. This paper highlights the importance of urban irrigation, which is vital for the survival of the lifestyle horticulture industry. This industry sector employs over 110,000 people across Australia, and values more than 1% of the Australian GDP. Using Kogarah Municipality in South-eastern Sydney, as a case study site, the paper provides water use profile of a local government authority in Sydney. By discussing various social, environmental and significant economic benefits associated with urban irrigation, the paper challenges the idea of compulsory water restrictions. Within Kogarah, high social value of urban irrigation is evident from annual User Hours figure of over 43,680 for each of its sports fields. Open space supported by urban irrigation also seems to help property values in Kogarah by as much as 20%. Furthermore, urban irrigation supported local golf club has an annual value of approximately $1.95 million to the local economy. Non-potable water reuse is therefore proposed as an urban water policy alternative. Investment by local governments and private golf clubs in decentralised water recycling demonstrates market driven evolution in urban water management. Urban irrigation in Australia is no longer an optional extra, but in fact is a much valued productive use of water.

Key words: Non-Potable Reuse, Urban Irrigation, User Hours, Golf Economy, Fit for Purpose, Decentralised Reuse, Lifestyle Horticulture.

1. BACKGROUND

Each year Australians consume more than 24,000 GL of water. Irrigated agriculture coupled with other rural activities consume close to 79% of this water, with remaining 21% going to urban uses (Environment Australia, 2003). Non-residential consumption represents around 30% of total water consumed in urban Australia, which is collectively the amount of water used by industry, commercial, and local government entities (ABS, 2004).

Some 17.6 million Australians, constituting eighty five percent of the population live in cities. According to CSIRO (2003) greater than 90% of the national population growth over the next quarter century, an additional 4.5 million residents, will occur in Sydney, Melbourne, South East Queensland and Perth. Consequently there are growing pressures upon urban water supplies, exacerbated by the recurrence of droughts. Achieving the goal of sustainable water management will therefore be a major challenge for most Australian cities in the 21st Century.

2 URBAN IRRIGATION

Urban irrigation is defined as irrigation which takes place within urban landscapes and includes irrigation of lawns, home gardens, parks, sporting fields and golf courses mainly for recreational purpose and to improve the quality of life (Stein, 2009). Urban irrigation is predominately concerned with non-food crops, ranging from grass for lawns and sporting facilities to different ornamental plants, shrubs and trees. Urban irrigation is vital for preserving the aesthetics of our

urban landscapes and to provide for lifestyle facilities that urban dwellers in the developed world take for granted.

The stream of horticultural industry that relies on urban irrigation has been termed as environmental horticulture, amenity horticulture, and more recently as the lifestyle horticulture (Aldous, 2008). The lifestyle horticulture sector includes businesses involved in the production of non-food horticulture products, such as ornamental plants, flowers and turf. It also incorporates a range of services including landscape design, sales of lifestyle horticulture products and services, construction and maintenance of parks, gardens and golf courses, and technical horticultural advice and information dissemination (Queensland DPIF, 2008).

Australian water policy makers commonly consider outdoor water use (urban irrigation) to be more discretionary than indoor water use. Consequently restrictions on outdoor water use are universally used by water utilities as an acceptable means to reduce demand at times of drought (Brennan et al, 2007). The Productivity Commission (2008) states that such prescriptive rationing by water utilities denies consumers an opportunity to choose the water use they value most. However, from an urban water utility's perspective irrigation demand, especially non-residential urban irrigation demand constitutes a very small portion of the total water sales. Mandatory restrictions are therefore an easier policy option then finding alternatives.

2.1 Value of Urban Irrigation

Urban irrigation through provision on urban green space provides several direct and indirect benefits. For instance, urban green space significantly impacts the microclimate of a region by cooling effect, improves the hydrological cycle and increases urban biodiversity. These benefits can be grouped into environmental, social and economic categories, and rely on the vegetation being irrigated during drought. (Fam et al, 2008).

Community benefits from green space from an aesthetic point of view, as well as by its contribution to improving social cohesion. There is now ample research evidence on the positive effects of urban green space on blood pressure, cholesterol and stress-reduction. Table 3 below summarises some of the key environmental and social benefits associated with urban green space, as identified by Fam et al (2008).

Table 3: Key Environmental and Social benefits of urban green space

Environmental Benefits	Social Benefits
Cooling Effect through evapotranspiration and increased solar reflection	Mental Health benefits of contact with nature and green space.
Enhanced air quality in urban areas	Physical exercise equally as effective as medication for treatment of depression.
Hydrological advantage through increased infiltration and reduced runoff	Parkland views from window during work hours proven to assist with job satisfaction
Increased urban biodiversity with trees providing canopy habitat for birds and fauna	Urban parks necessary for healthy development of children Physical inactivity significant health risk factor

Power (2005) suggested a methodology to quantify the value of a grassed sports surface to the community based on its User hours. Use hour is defined as the product of the number of participants and the hours of use by each participant. A sports field is considered of low use if the usage rate is less than 10,000 User hours per annum and of high use if the rate exceeds 30,000 User hours per annum.

$$\text{User Hours} = (N)*(H)*(W) \quad (1)$$

Where N = Number of players using the venue,
H = Hours of Use per week per player, and
W = Number of weeks used per annum.

Connellan (2007) further suggested dividing annual User hours with the total water consumption to measure the benefits (User hours/ML) of the irrigated site.

While grouped as environmental and social benefits, it is important to note that these benefits also have associated economic efficiency effect in the form of reduced costs to society. For instance, in 2005 the total cost of obesity to the Australian economy was estimated to be $3.767 billion, 12.7% of this cost was borne by the society (Australian Sports Commission, 2008). Providing sports fields and parks for more physical activity is the best solution to this health epidemic, and is likely to result in reduced health care costs. Similarly environmental benefits from urban irrigation result in reduced pollution abatement costs for the community (Fam et al, 2008). In addition to these economic efficiency effects, there are also direct commercial impacts of urban irrigation in terms of employment generation and improvement to asset values.

According to Irrigation Association Australia (2005), lifestyle horticulture sector employs an estimated 110,000 people. The total value of the sector is estimated to be approximately $9.39 billion per annum, which is more than 1% of the Australian GDP, and similar to the value of rural irrigation and what it produces for our economy (Irrigation Association of Australia, 2005).

Water is a critical input to the Golf industry. In the year 2004 the value of goods and services that make up the Australian golf economy was estimated at $2.71 billion, or approximately $2,110 per golfer (PGA, 2006). It also identified that approximately two jobs are created at golf facilities for every 100 people that play the game. (PGA, 2006)

Another economic benefit of urban irrigation supported green space is evident in the increased property values. According to Crompton (2000), people are willing to pay a more for property located close to parks, open space and greenery. Surveys discovered that 50% of respondents would be willing to pay 10% more for a house near a park and 57% said that if they were in the market for a home they would they would select neighborhoods close to parks and open space (Crompton, 2000).

3. POLICY ALTERNATIVE: DECENTRALISED NON-POTABLE REUSE

Given the social, environmental and economic benefits from urban irrigation, the value it delivers to the urban communities is undeniable. At the same, given it's relatively small size in the 24,000 GL of annual Australian thirst, urban irrigation is often the forgotten water use. It is only in the past decade that this area of urban water use has commanded significant attention of water planners.

Albeit slowly, a viable policy alternative to compulsory water restrictions is now emerging to meet the demand of urban irrigation at times of drought. Within metropolitan Australia decentralised water reuse, under schemes that are not owned by major water utilities was virtually non-existent. However, in the last 5 years, with drought assisted realisation of the value of urban irrigation, a number of water reuse projects have been implemented or are currently underway to cater for urban irrigation. In Sydney, local government authorities and private golf clubs are the two primary players in implementing such schemes.

Investment by private golf clubs and local government in decentralised water recycling is extremely useful in increased promulgation of water reuse for irrigation, and demonstrates market driven evolution in urban water management. Urban irrigation in Australia is no longer an optional extra, but in fact is a valued productive use of water. This evolution is evident in the case study project at Kogarah Municipal Council, which was Sydney's first ever water mining plant and was also the winner of the International Water Association's 2008 Project Innovation Award for the Asia Pacific Region.

4. CASE STUDY – MUNICIPALITY OF KOGARAH

Municipality of Kogarah is located in the heart of St George Region, in South-east of Sydney Metropolis. Kogarah local government area (LGA) encompasses an area of 20 km^2, with a population of 52,641. Approximately 10% of the total local government area of Kogarah is zoned as open space, including neighborhood parks and playgrounds, sports fields and ovals, and bushland.

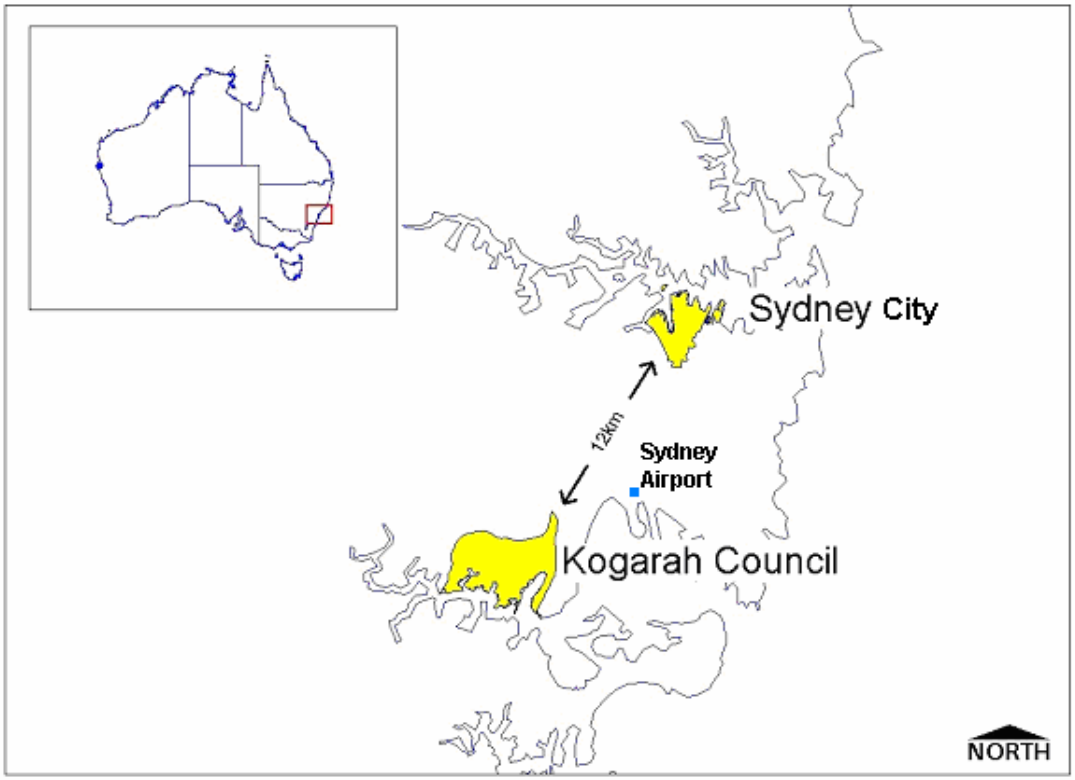


Figure 1: Location of Kogarah Local Government Area in Australia.

Land use within the LGA can be categorised primarily as urban, with residential dwellings, commercial centres, roads and transport facilities, open space and minor areas used for light industrial development. Kogarah's population density is high (and increasing), at approximately 5,600 people per square kilometre.

Table 1 provides a snapshot of water use in Kogarah LGA allocated between single dwellings, multi-units and non-residential users. Of the total 14.35 ML of water used in Kogarah every day, residential customers consume more then half. Non-residential water use in Kogarah amounts to 18% of the total urban water use, which is less then Sydney's average and can be attributed to lack of heavy industries within Kogarah LGA.

Table 1: Water Use in Kogarah (Kogarah Council, 2006)

	Single Dwellings	**Multi-Unit Dwellings**	**Non-Residential Accounts**
Number of Dwellings/Accounts	12069	8530	1075
Average Daily Water Use (Litres/dwelling or account/day)	649	460	2424
Total Daily Water Use (ML/day)	7.83	3.92	2.6

For the purpose of this paper, we will focus on the non-residential aspect of the water use profile. As highlighted in Table 1, during 2004/05 the 1,075 non-residential accounts in Kogarah LGA consumed 951 ML of water. To further analyse the non-residential demand, water accounts were categorised into low users (<5ML/a), medium users (<26 but >5ML/a) and high water users (>26ML/a). This classification was adopted from Sydney Water Corporation's Every Drop Counts

Program (Institute for Sustainable Futures, 2004; Kogarah Council, 2006). A breakdown of non-residential water demand for the whole LGA is presented in Figure 2.

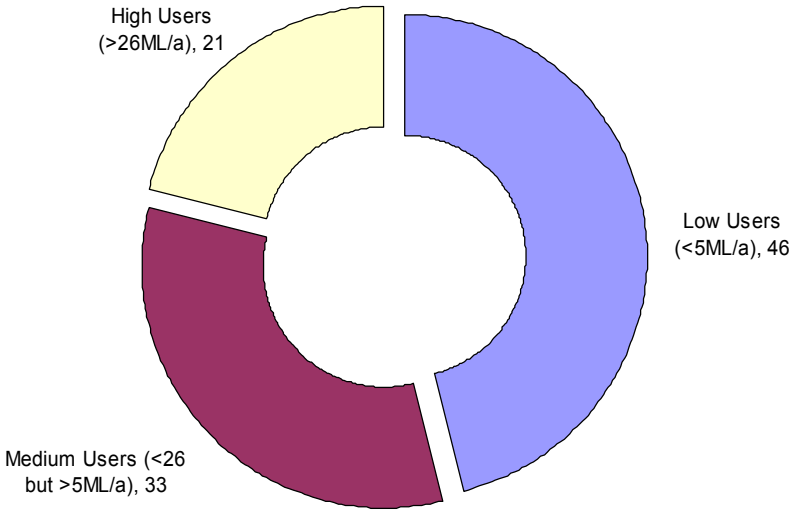


Figure 2: Breakdown of Non-Residential Water Use in Kogarah (Kogarah Council, 2006)

As highlighted in the figure above, 21% of the total non-residential water demand is consumed by high water users, which are a total of 9 accounts (Kogarah Council, 2006). Three of these 9 high water user accounts are Beverley Park Golf Club, the St George Rugby Leagues Club, and the St George Hospital. Kogarah Council itself is also amongst the high water users, and due to diverse portfolio of assets around the LGA, Council holds a number of water accounts distributed among high and medium users.

According to Mulley et al. (2007), local government on average uses 55% of potable water consumed for irrigation of public open space, with some council's using up to 82%. This represents a significant use of potable water for a purpose that does not necessarily need potable quality water.

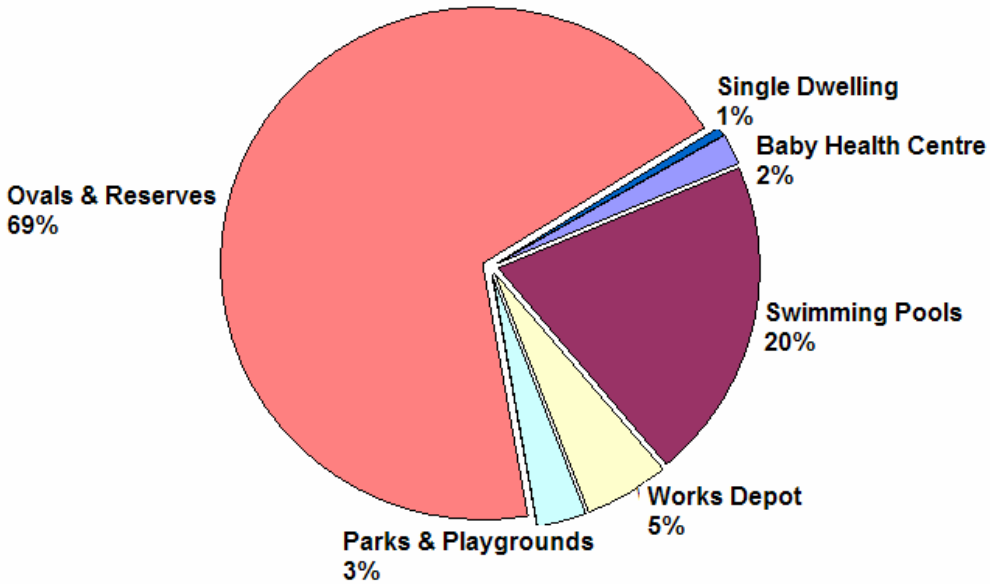


Figure 3: Kogarah Council's own water use profile (Kogarah Council, 2006)

Consistent with the findings of Mulley et al (2007), Figure 3 shows that more then 70% of Kogarah Council's own water usage is attributed to irrigation and maintenance of its parks and sports grounds. With local government being the principle providers of sports grounds in the state of New South Wales (NSW), high irrigation water use is explicable. According to NSW Parliament Standing Committee on Public Works (2008), local government manages 1230 out of the 1500

sports fields and ovals in Sydney metropolitan area. Table 2 below provides a breakdown of various management entities responsible for these sports fields.

Table 2: Providers of Sporting Fields/ovals in Sydney Metropolis (NSW Parliament Standing Committee on Public Works, 2008)

Managing Entity	Number of Parks	Percentage
Local Government	1230	82%
Private Sector	105	7%
State Government Trusts	75	5%
State Government Departments	15	1%
Other Institutions	75	5%

4.1 VALUE OF URBAN IRRIGATION FOR KOGARAH

The Municipality of Kogarah is the medical hub of southeastern Sydney. It is home to the St George Public Hospital, a wide range of allied health services as well as Australia's second largest private hospital (currently under development). It is therefore important that Kogarah's urban landscape maximises on the potential health benefits of urban greenery. Through provision of a range of open space Kogarah Council has a reputation of being the 'garden suburb', which is cherished by its residents.

There are also sound financial reasons behind cherishing the garden suburb image. The median house pricing in adjacent suburbs within Kogarah Municipality show a difference of over 20%, a significant portion of which could be attributed to the presence or absence of parks and open space. As outlined in Table 4 below, garden suburbs of Hurstville Grove and Beverley Park have higher median property prices then localities literally across the street.

Table 4: Median House Prices in Adjoining Suburbs within Kogarah Municipality (Domain Real Estate, 2009)

Suburb Name	Urban Green Space/Parks	Median House Price
Carlton	No major regional park	$550,000
Beverley Park	Located around the Beverley Park Golf Course	$700,000
Penshurst	No major regional park	$575,000
Hurstville Grove	Two large regional parks on eastern and western end	$772,000

The Beverley Park Golf Club with a membership base of over 925 golfers plays an important role in Kogarah's local economy as well as a local employer. It is estimated that the Club creates approximately 20 jobs in the local area (PGA, 2006). The annual value of Beverley Park Golf Club to the Kogarah economy is approximately $1.95 million (PGA, 2006).

According to Kogarah Council (2007), there are 22 sports fields located at ten venues within Kogarah that are widely used by 300 registered sports clubs for Winter sports alone. Table 5 above provides an estimation of average User hours associated with each sporting field at Kogarah for winter sports.

Table 5: Average User Hours for Winter Sports at Kogarah's Sports fields (Kogarah, 2007)

Users Per field (No. Teams x Players)	Hours Used/Week/Player (Training + Match hours)	Number of Weeks in Winter Season	Average User Hours/Field
13 x 15 = 195 Users	5 + 3 = 8 Hours	28 Weeks	43680

According to Power (2005) sports fields with annual User Hours of 30,000 are considered high value sports fields. With an average User Hour figure of 43,680 without even considering summer

sports, it would appear that sporting venues in Kogarah are in extreme demand, and extensively valued by the community. It is important to note that in delivering these extremely high User hours Kogarah Council uses a total of 31ML of irrigation water per annum (Kogarah Council, 2006a). The annual irrigation water use benefit for Kogarah's sporting community is over 1409.03 User Hours/ML.

Given the above benefits associated with urban irrigation, Kogarah Council was the first local government in Sydney to invest in a water mining facility that recycles wastewater for urban irrigation. The facility supplies irrigation water to the local golf club, rugby league club and Council's own sporting fields. While subsidised by the New South Wales State Government with a grant of $1.6 million, it is important to note that the matching funds were generated from the Kogarah community through an environmental levy, confirming community's willingness to pay for urban irrigation.

5. CONCLUSION

Kogarah case study shows high social value of urban irrigation, evident from annual User Hours figure of over 43,680 for each of its sports fields. As well as economic benefits from urban irrigation such as increased property values by as much as 20%. Furthermore, a viable golf club with annual contribution of $1.95 million to the local economy. These figures coupled with Kogarah's image as the garden suburb and the medical hub of southern Sydney, makes it obvious for the Council to appreciate the impacts of compulsory restrictions on urban irrigation. This in turn makes local governments and golf clubs ideal candidates to invest in decentralised reuse schemes. A policy shift from compulsory restrictions to alternatives such as decentralised water reuse would therefore require local governments to play a key role in urban water management.

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ASSESSMENT OF WATER SYSTEMS VULNERABILITY TO DROUGHT

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Abstract: Water shortage may be caused by intense drought events and/or mismanagement of available water resources. To face water shortage caused by drought, proactive measures should be taken in a timely and well organised way. The paper gives guidelines for the estimation of water shortage risk in the strategic and operational preparedness plans of a complex water system. It is concluded that for the operational preparedness plan important tools are required: a) a simplified drought monitoring system, b) a forecasting tool for assessing annual drought severity, c) a deterministic mechanism for estimating the consequences, and d) a prioritised water allocation methodology based on annualised risk.

Key words: complex water system, drought, Preparedness Planning, vulnerability assessment, water shortage risk.

1. INTRODUCTION

Water shortage may be caused by intense or long lasting drought episodes and/or mismanagement of available water resources. In order to face water shortage caused by drought, proactive measures should be taken in a timely and well organised way which can assist affected areas either to prevent or mitigate the associated consequences. This proactive approach has gained acceptance by various research groups working in the area (Wilhite, 2000; Rossi *et al.*, 2007; Tsakiris, 2008a; Garrote *et al.*, 2007; Iglesias *et al.*, 2009) although no systematic application of this approach has been implemented in the real world so far. Many research projects have recently adopted this proactive approach when devising strategies to combat drought and water shortage.

This approach includes two phases. In the first phase preparedness plans are devised for identifying the long and short term measures which are viable, efficient and environmentally and socially acceptable and could be applied at various levels of severity. The second phase refers to the implementation phase of such plans provided that the necessary information is available from the drought/water shortage monitoring system. In short, Water Shortage Preparedness Plan should comprise a Strategic and an Operational Plan.

The Strategic Plan uses historical data on drought and water shortage episodes together with economic, social and environmental data, trying to formulate relationships between drought scenaria and consequences. The cornerstone of this procedure is the estimation of maximum potential consequences corresponding to the worst possible scenario. In the Strategic Plan the severity level of each scenario (represented by a return period) is identified and the (relative) vulnerability of each affected system is estimated. It is advisable for a complex water system to be divided into a number of sub-systems. Normally three or four sub-systems can be examined separately. In each of the sub-systems the annualised risk is calculated for each option of measures taken against drought and water shortage. Annualised risk can then be compared with the acceptable risk leading to the rational determination of design of protection measures.

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Apart from annual isolated droughts causing water shortage, the multi-year droughts could be also analysed. In the next paragraphs both the isolated annual droughts and the multi-year droughts are analysed.

For the Operational Plan three principal tools are of importance: a) the drought monitoring system, b) the forecasting mechanism for an early assessment of annual drought severity and c) the reliable deterministic mechanism for estimating the anticipated consequences based on the Strategic Plan. Finally a prioritised water allocation methodology should be devised for achieving the most efficient allocation of limited water resources.

2. CALCULATING DROUGHT SEVERITY

Drought is a recurring natural regional phenomenon associated with severe reduction of water supply availability for a significant period of time. When modelling drought three dimensions (variables) characterising each drought episode, the severity (or intensity), the duration and the areal extent, need to be modelled. When performing frequency analysis this multidimensional character of drought leads to obscure or difficult to understand conclusions.

In an attempt to simplify the identification and characterisation process Tsakiris (2008b) proposed the uni-dimensional analysis of droughts appropriate for management decisions. According to this simplification the river basin or sub-basin is used as a territorial unit for drought analysis whereas prespecified reference periods are introduced for replacing the duration of the drought events on the temporal scale. Finally drought severity is represented by the Reconnaissance Drought Index (RDI) and supplementarily by the Standardised Precipitation Index (SPI) (Paulo *et al.*, 2007).

As known, RDI is based on the ratio of cumulative precipitation and potential evapotranspiration values (Tsakiris and Vangelis, 2005; Tsakiris et al, 2007). Three expressions of RDI are used; the initial, the normalised and the standardised. The last is of the same nature as SPI (a well known drought index using only precipitation values) and uses similar rules and thresholds with SPI.

To promote comparability of drought severity data, the reference period of 3, 6, 9 and 12 months were proposed starting from the beginning of each hydrological year.

In case of historical data, annual severity values can be associated to streamflow data and recharge of groundwater aquifers. The latter are the principal components (apart from the depletion rates) for estimating the water shortages (Nalbantis and Tsakiris, 2009).

In the case of the Operational Plan the values of drought severity of the first or second reference periods of 3 and 6 months duration are used to forecast the final annual drought severity level, which in turn has been associated with the anticipated consequences. At the same time the annual drought severity level of the year analysed can be compared with the severity levels corresponding to each scenario based on selected return period.

It should be recognised that the complex water system of a river basin or the adjacent basins should be divided into a number of simpler uniform sub-systems mainly associated to water uses. Thus at least three sub-systems may be distinguished: the urban (municipal) system, the agricultural water system and the industrial water system. Some other sub-systems such as the commercial and the tourist sub-systems may be studied separately or they can be incorporated in the urban and industrial system, respectively. The need for disaggregating the entire system into sub-systems stems from the fact that each drought scenario with a certain return period has different consequences in each of the sub-systems and therefore each sub-system exhibits different vulnerability leading to a different risk level.

As mentioned earlier, isolated annual drought episodes can be studied and return period may be attributed to each severity level. However, of particular interest is the occurrence of multi-year droughts. The analysis of multi-year drought episodes is attempted in the next paragraph.

3. THE MULTI-YEAR DROUGHT CONSEQUENCES

A certain river basin can be affected by drought of varying intensity for more than one year. Several persistent droughts lasting for several years have been recorded with increasing frequency over the last decades.

In general terms the consequences from these multi-year droughts are much more significant compared to the aggregated consequences from non consecutive single-year droughts of the same intensity.

The consequences from a multi-year drought are mostly dependent on the system characteristics and its vulnerability. A system with a more robust coping capacity, which is well prepared and its people are educated and responsive, is expected to withstand the stress created by a multi-year drought more successfully than a less prepared and more vulnerable system.

Analysing the drought conditions for four areas in Greece (Thessaly, Cyclades, Athens and Eastern Crete) for the period 1955-2002, the single drought years as well as the multi-year drought events were recorded. The statistical results of this survey are presented in Figures 1 and 2. Figure 1 refers to droughts of varying intensity whereas Figure 2 refers to severe and extreme droughts (Tigkas, 2008).

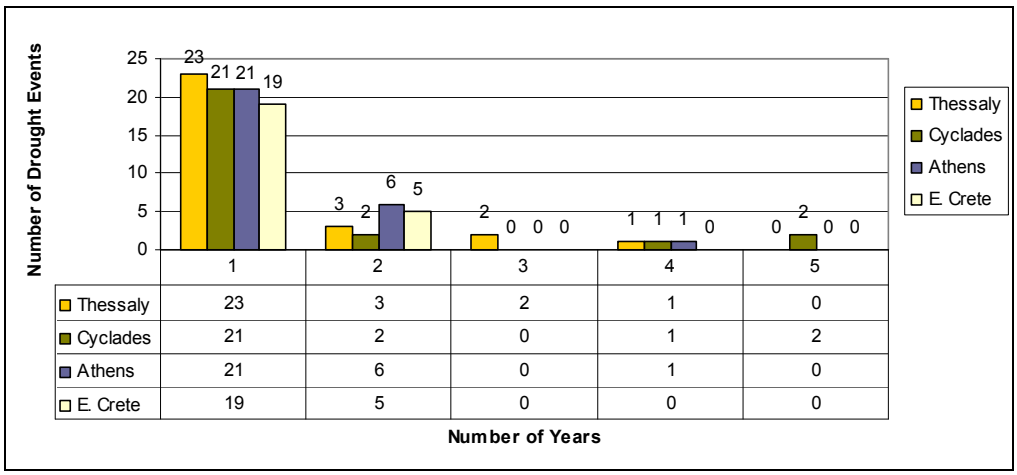


Figure 1. Single and multi-year drought events during the period 1955-2002.
(1: single year, 2: 2-year, 3: 3-year, 4: 4-year and 5: 5-year)

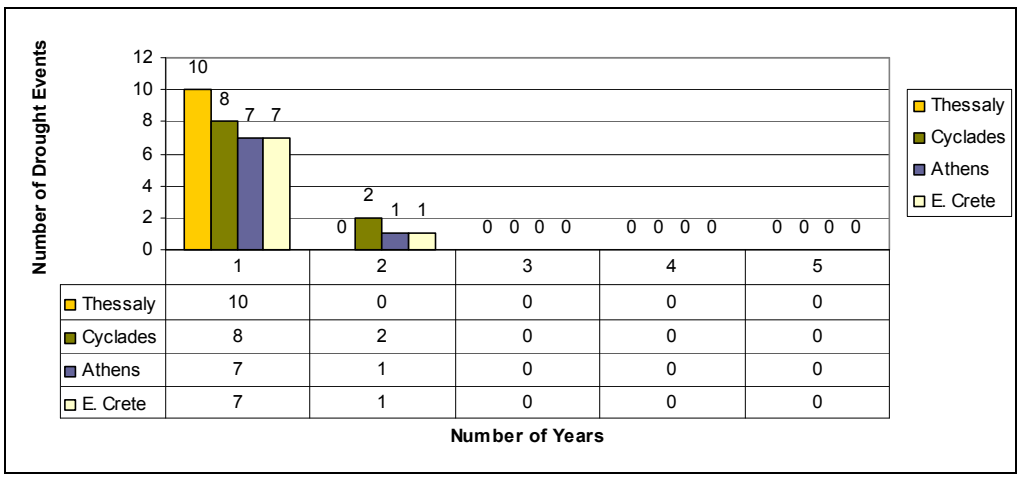


Figure 2. Severe and Extreme single and multi-year drought events during 1955-2002.

As can be seen from Figure 2, the severe or extreme droughts occurring for two consecutive years are very seldom phenomena. It can be also noticed that there is no multi-year severe drought event with duration more than two consecutive years.

In order to assess the multi-year drought impacts on a certain system a thorough analysis of all its components is required in response to various drought intensities for one or more consecutive years. For most of the cases such information is seldom available. For this reason in order to reach practical results, empirical vulnerability functions may be used for each year. Their aggregation over a number of years (multi-year drought) can be assumed to be of an additive or multiplicative form.

For a number of situations examined with inadequate and scattered data it may be concluded that in the case of multi-year droughts the aggregated vulnerability function is of a multiplicative rather than an additive type.

Using the first expression of RDI for the entire hydrological year, the response function can by approximated by a logistic or power function. For simplicity the following analysis refers to the simplest power function type of the response function:

$$Y = (\alpha_{12})^{\lambda} \tag{1}$$

in which: Y is the response function of the year ($Y \leq 1$)
α_{12} is the first expression of RDI for the 12-month period of the hydrological year (usually October-September)
λ is the sensitivity index

If $\lambda=1$ the response function Y is linear. For values of $\lambda>1$ the function is concave whereas for values of $\lambda<1$ the function is convex as can be seen from Figure 3. Therefore if the system is robust and well prepared λ should be less the 1, whereas for a more vulnerable system λ should be taken greater than 1. For simplicity purposes the sensitivity index λ may be considered constant for a certain system throughout the period examined.

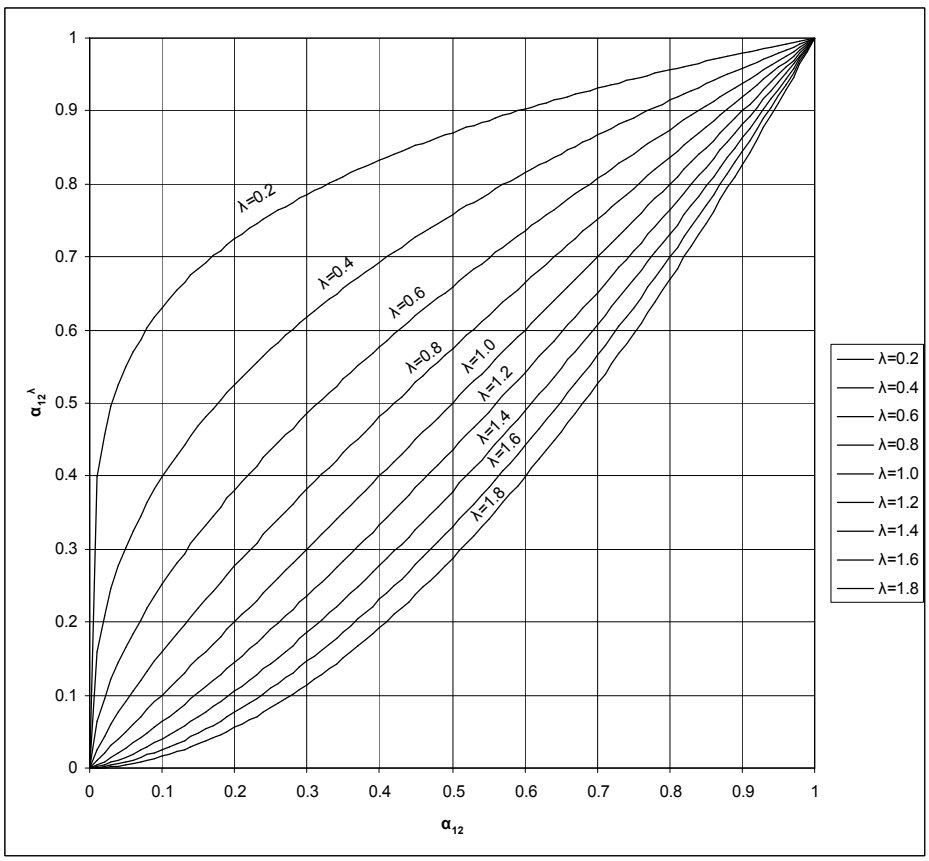


Figure 3. Several forms of "power" vulnerability function.

According to the multiplicative principle for n consecutive drought years the final response may be calculated as:

$$Y_n^* = \prod_{i=1}^{n} Y_i \qquad i=1(1)n \tag{2}$$

in which the final response Y_n^* is applied to the last year of the drought period and Y_i refers to the response of single-year drought in the ith year.

If the value of the item at risk A is constant during all the years of drought, the consequences are:

$$L_k = A \cdot \left(1 - \prod_{i=1}^{k} Y_i\right) \qquad \textit{(for the k-th year)}$$

and (3)

$$L_n = A \cdot \left(1 - \prod_{i=1}^{n} Y_i\right) \qquad \textit{(for the final year)}$$

In the special case that $Y_1=Y_2=Y_3=...=Y_n=Y$, which practically means that the intensity of drought remains the same through the entire period of the multi-year drought, Eq. 2 may be written as:

$$Y_n^* = Y^n \tag{4}$$

and the consequences of each year of the multi-year drought are estimated as:

$$L_k = A \cdot \left(1 - \prod_{i=1}^{k} Y_i\right) \qquad \textit{for } k=1(1)n \textit{ and } k \geq 1 \tag{5}$$

For illustration purposes let us consider a system with sensitivity index λ=0.25 affected by a three-year drought of average annual intensity represented by α_{12}=0.70. Then, for each of the three years the anticipated consequences measured by the economic loss is calculated as follows, given that the total value at risk is 1 million Euros (1 M €).

first year loss: $L_1 = A \cdot (1 - Y_1) = 1\ M€ \cdot (1 - 0.70^{0.25}) = 1 \cdot (1 - 0.91) = 0.09\ M€$

second year loss: $L_2 = A \cdot (1 - Y_1\, Y_2) = 1 \cdot (1 - 0.84) = 0.16\ M€$

third year loss: $L_3 = A \cdot (1 - Y_1\, Y_2 Y_3) = 1 \cdot (1 - 0.77) = 0.23\ M€$

As can be seen from the numerical example, although the intensity of drought is the same for all the three years of drought, the resulted consequences are considerably increasing from the first towards the last year of this multi-year drought event.

It would be interesting to derive the aggregated loss over the entire multi-year drought period. Let us consider the three-year drought of the previous example. Then:

$$\begin{aligned} L_{total} = L_1 + L_2 + L_3 &= A \cdot (1 - Y_1) + A \cdot (1 - Y_1\, Y_2) + A \cdot (1 - Y_1\, Y_2 Y_3) \\ &= A \cdot [3 - a_{12,1}^{\lambda} - a_{12,1}^{\lambda} \cdot a_{12,2}^{\lambda} - a_{12,1}^{\lambda} \cdot a_{12,2}^{\lambda} \cdot a_{12,3}^{\lambda}] \end{aligned} \tag{6}$$

If the intensity of drought is considered uniform during the entire period of the three years, Eq. 6 becomes

$$L_{total} = A \cdot [3 - \alpha^{\lambda} - \alpha^{2\lambda} - \alpha^{3\lambda}] \quad (7)$$

For n-year drought Eq. 7 finally becomes:

$$L_{total} = A \cdot [n - \alpha^{\lambda} - \alpha^{2\lambda} \ldots - \alpha^{n\lambda}] \quad (8)$$

Considering the data of the previous example (α_{12}=0.70, λ=0.25, A=1 M €) the aggregated loss over the three-year drought period will be calculated according to Eq. 7 equal to 0.483 M €. As expected this figure is much greater than the aggregated loss of the three independent drought years which is calculated 0.255 M €.

Based on the above calculations and the most common values for λ and α_{12} it becomes clear that for a multi-year drought the vulnerability is increasing as more drought years are added. Regarding the distribution of the intensity of drought it can be observed that the consequences are higher if the intensity of drought is greater during the first years of the multi-year drought, followed by milder drought years when compared to the opposite case in which there is a mild drought at the beginning followed by intense drought during the last years of the entire period.

The last step in this multi-year drought is how to calculate sensitivity index λ which is compatible with the results produced for the multi-year drought. This task is difficult and is mainly dependent upon the complexity of the system which is analysed. For the simple systems an empirical rule is to study the consequences of two to three multi-year events from the recent historical record and try to estimate an average λ through a simple optimisation procedure based on trial and error. A more complicated approach is resulted if λ is not constant and is changing from year to year as the persistent drought progresses. Even in this case the procedure may be simplified by adopting a linear increase of λ in relation to the years from the beginning of drought. However, it should be clarified that these simplifications are introduced for formulating procedures which can reach practical results. In case of complex systems it is advisable to disaggregate the system to relatively independent sub-systems, as was mentioned previously.

The above procedure can be explained by a numerical example of simple system affected by a three-year drought. Let the RDI first expression for each of the three years be α_{12} = 0.80, 0.90 and 0.70, respectively. The annual streamflow of the river is decreased to 75, 65 and 40% of the mean annual streamflow for the three drought years, respectively. Estimation of sensitivity index λ of the system is required.

According to the above, if λ is considered as a constant, then:

$$0.80^{\lambda} = 0.75 \Rightarrow \lambda = 1.29$$

$$0.80^{\lambda} \cdot 0.90^{\lambda} = 0.65 \Rightarrow \lambda = 1.31$$

$$0.80^{\lambda} \cdot 0.90^{\lambda} \cdot 0.70^{\lambda} = 0.40 \Rightarrow \lambda = 1.34$$

Therefore an average $\bar{\lambda}$ = (1.29 + 1.31 + 1.34) / 3 = 1.31, can be selected.

A second approach could accept a gradual increase of λ of say 2% per year. In this case λ=1.29 during the first year, λ=1.31 during the second year and λ=1.33 during the third year. Therefore, instead of using a unique value of λ=1.31 we could expect a slight increase from year to year.

If the above procedure is tested for two to three multi-year drought events, some round figure can be obtained for being used in future multi-year drought events.

4. VULNERABILITY AND RISK IN COMPLEX WATER SYSTEMS

As mentioned, this paper focuses on the vulnerability of complex water systems in response to drought. However, each water system reacts differently than another depending on its capacity and preparedness to withstand less water availability. Even in the same complex system, the sub-

systems exhibit different vulnerability. It should be also mentioned that different level of water security for each system or sub-system may be required. For instance, a municipal water system should be more carefully protected against water shortage than an agricultural system. This is reflected at the level of acceptable probability of failure which should be smaller in the municipal water system.

For a simple water system depending principally on annual water availability (annual regulation) the Strategic Preparedness Plan has the main task to determine the level of the probability of failure due to lack of water and propose the necessary engineering and management measures in order to fulfil this requirement. In case of an existing system with a certain infrastructure, analysis of drought events is needed for the rational determination of the drought severity (corresponding to a certain return period) and therefore the necessary additional engineering and management measures which are required.

It should be stressed that water shortage risk is not totally dependent on natural hazards such as drought. Therefore, the probabilistic procedure should also incorporate a deterministic component including the permanent characteristics of the system and the management practices adopted in this water system. Therefore if risk is calculated as the probability of failure (due to water shortage) it should always refer to given conditions and capacities (Tsakiris, 2009).

In case of the Operational Preparedness Plan, one more constraint should be fulfilled: the initial conditions. In this case we have to calculate the so called conditional risk.

For the Strategic or the Operational Plan both single year and multi-year droughts should be considered.

Since probabilities are not easily understood by the decision makers and the stakeholders and they do not create the appropriate awareness for the severity of water shortage, risk can be evaluated in terms of economic, social and environmental loss. This may be achieved by the concept of annualised risk which incorporates the drought scenario with certain probability of exceedance, their impact on water availability and therefore its impacts on economic, social and environmental sectors. Figure 4 shows a composite evaluation of anticipated consequences from a certain drought or water shortage scenario on these sectors. This figure is the outcome of an Index Method (such as the Checklist or Multi-Attribute Utility method). According to the procedure followed, the individual criteria are transformed into numbers on a common notional scale as in Figure 4. Further by weighting them in terms of their relative importance the overall score for each scenario is derived. By using probability classes and the average impacts between drought scenaria at the borders of each class, the anticipated impacts (economic, social and environmental) from the entire drought events process and the given system characteristics can be estimated.

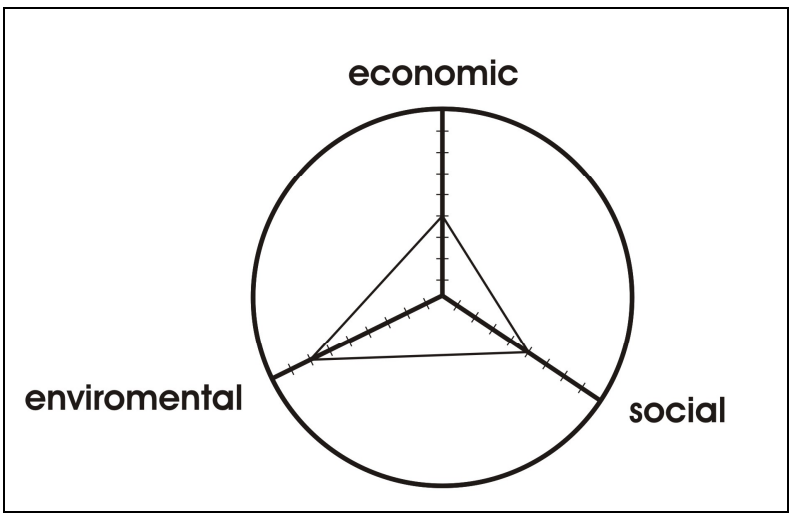


Figure 4. Evaluation of a drought episode based on the consequences on economic, environmental and social sectors.

The estimation of annualised risk is difficult and requires a deep knowledge of economic, social and environmental processes. This may be derived from experts' knowledge of the system under study and is obviously applied only to the system under study.

In complex water systems there is also one more complication related to allocation rules of limited water resources during the period under stress. Based on the above proposal, Tsakiris (2008a) proposed a prioritisation scheme in which the lack of each unit of water during a certain month has a certain impact. Through a multicriteria evaluation all units of demand may be characterised by a certain priority. The task of setting priorities for each unit of water available in each month is much more difficult in a complex system requiring advanced multicriteria evaluations and enhanced stakeholders participation, which cannot be discussed in this paper.

5. CONCLUSIONS

Preparedness Plans to face water shortage, need simple and meaningful tools and calculation procedures, so that they can be understood and adopted by decision makers and stakeholders. In this paper attention is focussed on the simplified analysis of single and multi-year drought events through simple vulnerability functions.

Complex water systems are disaggregated into a small number of sub-systems and vulnerability analysis is performed in each sub-system separately. Then annualised risk in each sub-system is proposed to be used for setting priorities for engineering and management measures (strategic level) and for the allocation of limited water during the period under stress (operational level).

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INVESTIGATING THE SPATIAL AND TEMPORAL DISTRIBUTION OF DROUGHT TREND IN IRAN

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Abstract: Drought is a normal and repeatable climatic phenomenon. The prediction of spatial and temporal distribution of drought attributes is one of the main aspects of water resources management, especially in arid and semiarid regions. More than 75% of Iran is located at such climatic conditions. In recent years, due to drought, very severe economical and agricultural damages were occurred in Iran. The drought analysis indices are common applied for assessing the frequency and severity of dry and wet periods in each region. In this study, using some attributes and indices the drought trends in Iran is investigated and its spatial and temporal distribution during the last 5 decades is compared over the whole country. The 49-year precipitation data of 21 weather stations were provided to analyze the temporal and spatial variations of drought in northern and southern Iran. Iran is a vast country and is affected by northern, Mediterranean, Sudanian and Indian Ocean fronts. Due to this state, the severity and frequency of drought in different part of the country is not the same. The standard precipitation index and 5-year and 7-year moving average were used to assess the drought in the study area. The results represents several wet and dry periods at the selected stations. The analysis of data for some identified dry and wet periods shows the occurrence of different severity and frequency of drought in northern and southern Iran.

Key words: Drought trend, Moving average, Standard Precipitation Index

1. Introduction

Drought is a normal, recurrent feature of climate that may occur everywhere even if its characteristics and impacts vary significantly from region to region (Wilhite 1997). It is defined as a natural temporary imbalance of water availability, consisting of a persistent lower-than-average precipitation, of uncertain frequency, duration and severity, of unpredictable or difficult to predict occurrence, resulting in diminished water resources availability and carrying capacity of the ecosystems (Pereira et al. 2002). Thus, an objective evaluation of drought condition in a particular area is the first step for planning water resources in order to prevent and mitigate the negative impacts of future occurrences. For this purpose, along the years, several indices have been developed to evaluate the water supply deficit in relation to the time duration of precipitation shortage. Among various indices and criterions, the most commonly used for drought monitoring are the Palmer Drought Severity Index (PDSI, Palmer 1965) and the Standardized Precipitation Index (SPI, McKee et al. 1993). The PDSI is based on the supply-and-demand concept of the water balance equation for a two-layer soil model. It depends on several local coefficients that are estimated using local hydrological norms related to temperature and precipitation averaged over some calibration period (at least 30-year period, according to the World Meteorological Organization recommendation).

The basis of the SPI is the difference between the amount of precipitation required to retain a normal water balance level and the actual precipitation. Nevertheless, if we wish to compare drought conditions of different areas, which often have different hydrological balances, the most important characteristic of any index is the standardization. The SPI complies with this requirement. It is, in fact, a standardized index that can be computed on different time scales, so as to allow monitoring most of drought types (i.e. meteorological, agricultural, and hydrological). The SPI

computation for any location is based on the long-term precipitation record cumulated over the selected time scale.

Rezaei et al. (2005) investigated annual rainfall trend in arid and semi-arid regions of Iran using 79 climatological stations with 36 years of data record. Monthly and annual precipitation data were studied from homogeneity and randomness point of view. A number of homogeneity tests were applied and discontinuities were adjusted in no-homogeneous stations. Annual precipitation time series from 1965 to 2000 were checked for climate variability and possible trend. The results showed that there is no evidence of climate change in the study area. Although many stations showed negative trends indicating the decrease in precipitation, this trend was not statistically significant at 95 percent significant level. The results indicated that southeast corner of Iran has experienced climate change in the form of negative precipitation trend.

Morid et al. (2006) examined the performances of seven drought indices requiring only rainfall data for drought detection and monitoring in the Tehran province of Iran. They concluded that, despite different underlying statistical distributions, the SPI performed in a similar manner with regard to drought identification and drought onset, and that the SPI and Effective Drought Index (EDI) could be recommended for operational drought monitoring in the region; however, the EDI requires daily precipitation, which constitutes a serious limitation for its operational use.
Thus, due to its advantages, the SPI appears to be the most powerful drought
index. Many authors including, Hayes et al. (1999), Szalai and Szinell (2000), Lloyd-Hughes and Saunders (2002), Lana et al. (2001), Vicente- Serrano et al. (2004), Tsakiris and Vangelis (2004), used the SPI to monitor drought in many regions.

Zhai et al. (2008) investigated on spatial and temporal pattern of precipitation and drought in Gansu Province, Northwest China. They used Standardized Precipitation Index (SPI) to calculate and reconstruct the drought at different time scales and analyze the frequency of drought occurrence in the recent 50 years. They concluded that the SPI is applicable in Gansu Province. The number of severe droughts differs among regions: it is more obvious as a 3-month drought in the Yellow River Basin and the Yangtze River Basin than in the Inland River Basin, and other droughts at 6-, 9-, and 12-month time scales have the same effect in the three regions.

McKee et al. (1993) have defined the SPI as the number of standard deviations that the observed cumulative rainfalls at a given time scale would deviate from the long-term mean. Zhai et al. (2008) calculated standardized Precipitation Index (SPI) to reconstruct the drought at different time scales and analyze the frequency of drought occurrence in the recent 50 years. The results showed that the SPI is applicable in Gansu Province of China.

2. Material and methods

2.1 Standardized Precipitation Index

The SPI is widely accepted and used throughout the world in both research and operational management since it is normalized to a location and normalized in time (Wu et al. 2007). Practically, when one computes the SPI, there is no need to investigate the soil moisture, temperature, among others, in contrast to the process of calculating the Palmer Drought Severity Index (PDSI). SPI is theoretically unbound; it is extremely rare empirically to observe values greater than +3.0 or less than - 3.0. The SPI is usually computed over multiple time scales ranging from 1 to 48 months, which reflect the impact of drought on the availability of the different water resources. Among users there is a general consensus about the fact that the SPI on shorter time scales (say 3 and 6 months) describes drought events affecting agricultural practices, while on the longer ones (12 and 24 months) it is more suitable for water resources management purposes.

In this paper, results concerning the 3-, 6-, 9-, and 12-month time scale are discussed. This is because in Iran there are areas where significant portions of the available water reservoirs are

managed on a time scale of a single year (i.e. they are filled in during the rainy season and empty out in the dry season).

The time and space variability of drought in Iran were studied using long term rain gauge data from 21 stations distributed uniformly over the country. These stations with the record length of 49 years (1955 – 2003) were selected to identify the probable different drought periods. The missing data at the stations was reconstructed using regression method between the stations with highest regression coefficient. Drought conditions were assessed through the SPI and moving average computed on 3-, 6-, 9-, and 12 months time scale rainfall amounts. The data set of all stations was smoothed using a 5-and 7-year moving average filter in order to damp time series fluctuations.

2.2 Study area

Iran with area of about 1,648,000 square kilometers is located in the southwest of Asia and lies approximately between 25N and 40N in latitude and between 44E and 64E in longitude. Iran's important mountains are Alborz and Zagros ranges. Alborz and Zagros Chains stretch in northwest-northeast and northwest- southeast direction. These two ranges play an important role in the non-uniform spatial and temporal distribution of precipitation in of western Iran. Because of its geographical position, drought is the dominant hazard and precipitation is an important moisture input in the area.

Precipitation in Iran has a high spatial and time variability. There are regions in the south of Caspian Sea, which receive up to 2,000 mm of annual precipitation, whereas portions of central and eastern part of the country get less than 50 mm. Furthermore, most of the precipitation in Iran falls during the winter and autumn seasons, due to the prevalence of humid westerly winds of Mediterranean origin. However, there are regions in the northwestern part of the country that are characterized by high precipitation also during spring. These features make difficult the management of water resources in Iran, especially during water shortage periods. The SPI appears to be a useful tool for comparing the climatic conditions of these areas characterized by different hydrological regimes.

Table 1. The station characteristics

NO.	Weather Station	Longitude	Latitude	Elevation
1	Arak	386234.42	3773930.4	1708
2	Isfahan	562545.45	3608988.02	1550.4
3	Babolsar	647359.04	4064710.78	-21
4	Tabriz	12547.63	4215838.61	1361
5	Rasht	380194.38	4117912.83	36.7
6	Zanjan	275130.58	4062695.82	1663
7	Sanandaj	681772.17	3911844.69	1373.4
8	Kerman	496792.89	3346488.37	1753.8
9	Mashad	736559.43	4016743.39	999.2
10	Yazd	254112.12	3532300.95	1230.2
11	Orumieh	507362.31	4154042.7	1313
12	Ahvaz	277995.8	3468896.13	22.5
13	Khoramabad	255320.74	3707974.77	1125
14	Bushehr	483763.78	3206150.53	19.6
15	Tehran	528653.86	3948872.6	1190.8
16	Zahedan	294745.09	3261555.65	1370
17	Shiraz	648484.79	3275444.19	1488
18	Kermanshah	694834.26	3795599.87	1322
19	Hamadan	292131.42	3897610.97	1679.7
20	Shahrekord	485882.91	3577394.03	2061.4
21	Birjand	705844.07	3638651.04	1491

The rainy period in most of the country is from November to May followed by dry period between May and October with rare precipitation. The average annual rainfall of the country is about 251 mm with maximum amounts in the Caspian Sea plains, Alborz and Zagros slopes with more than 1,800 and 480 mm, respectively. Going inland at the central and eastern plains, the ranges of precipitation decreases to less than 100 mm annually depending on the location. From the synoptic aspects, the climate of most part of Iran is dominated by subtropical high in most part of the year. This phenomenon causes hot and dry climate in summer. The rainfall in the country is produced by Mediterranean synoptic systems, which move eastward along with westerly winds in cold season (Rezaei et al. 2005).



Figure1. Geographical location and topographic features of Iran

Synoptic systems and year-to-year variation in the number of passing cyclones cause high variability in annual rainfall. Frontal Mediterranean cyclones associated with the westerly air flows produce most of precipitation in the whole country in late autumn and particularly in winter. In addition to the frontal Mediterranean cyclones, rainfall bearing systems called Sudanian cyclones which come from the southwest make an important contribution to increase annual rainfall amount of the west and southwest of the country. In northwest mountainous regions, convective and frontal thunderstorms are important atmospheric process responsible for rainfall in spring and early summer. These rainfall bearing systems just are active in the west portion of the country and have no more energy and moisture to pass far through east. These systems sometimes may reach central and east dry regions of the country when have no potential to produce rainfall due to long trajectory and loss of moisture. This region is the most drought prone area in the country due to high inter- and intra-annual irregularity in rainfall and high coefficients of variation. This region that account for over half of the country area, is surrounded by Alborz mountain range from the north and Zagros range from the northwest to southwest. Zagros range acts as a wall to prevent Mediterranean moisture bearing systems to pass through to the east. Hence these two ranges prevent the arid and semi arid regions of the central and eastern part of Iran from access to moisture. This phenomenon gives rise to high irregularity in rainfall in the study area. Lack of rainfall in May to October compounded with high temperature leads to high evapotranspiration and water deficit in this region. Prolonged drought in this area in recent years makes us wonder if we are facing to a drier climate.

Increasing water deficit problem in conjunction with increasing population and water demands in this region gave the region a priority for study of climate change analysis.

3. Results and Conclusion

The objective of this paper is to identify the drought with SPI and moving average at different time scales and with higher resolution, detect the trend of precipitation, and analyze the distribution of precipitation trend and the frequency of drought in Iran. The results could be utilized as a reference for decision making regarding water resources management, ecological recovery, and drought hazards mitigation.

SPI is theoretically unbound; it is extremely rare empirically to observe values greater than +3.0 or less than -3.0. The 3-, 6-, 9-, and 12 months time scale rainfall amounts of all stations were smoothed using a 5-and 7-year moving average filter in order to damp time series fluctuations.

The results show 3 wet or drought periods is identifiable since 1955 over Iran. Some of the weather stations represent wet period and at the same period others show a drought situation. Figures 2 to 7 represent different behavior of annual precipitation trend in some provinces located at north, middle and south parts of Iran. For example, as it is shown in figure 2, two different trends is identifiable in Tabriz weather station in the north part of the country and Bushehr station in the south part.

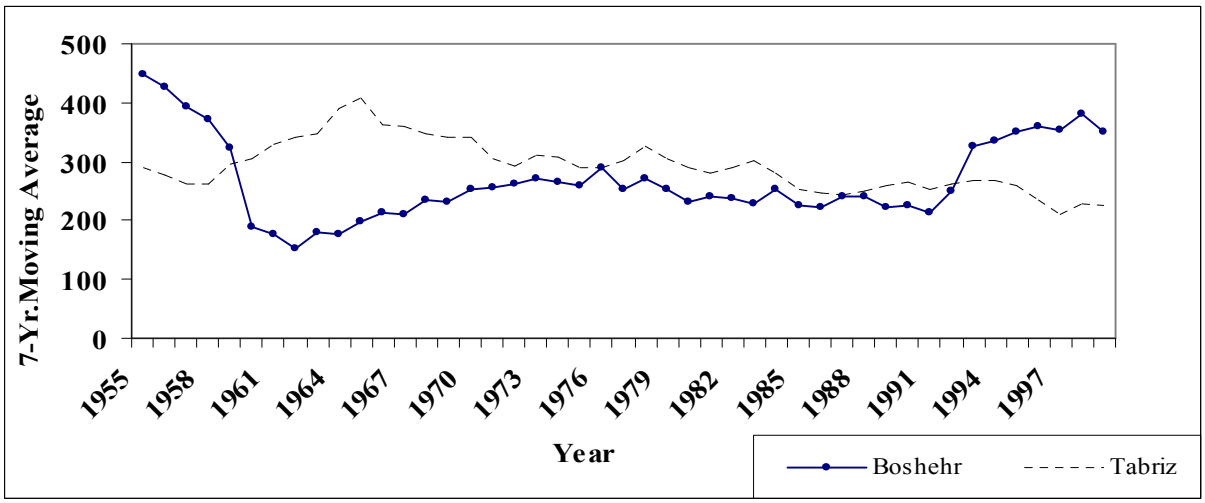


Figure 2. 7-year annual moving average at Tabriz (in the northwest part) and Bushehr (in the south part) stations in Iran.

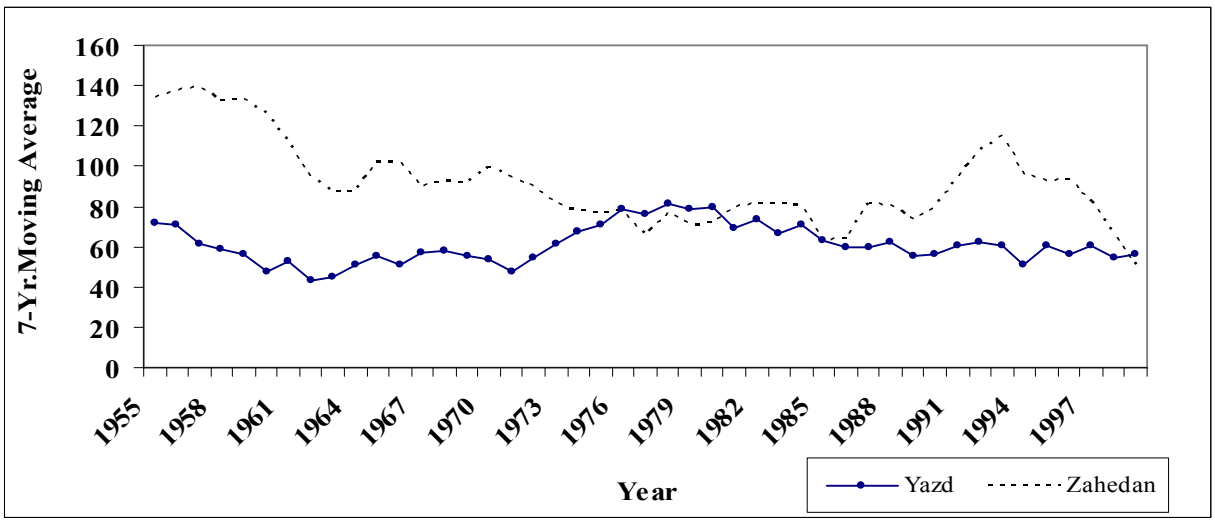


Figure 3. 7-year annual moving average at Yazd(in the middle) and Bushehr stations.

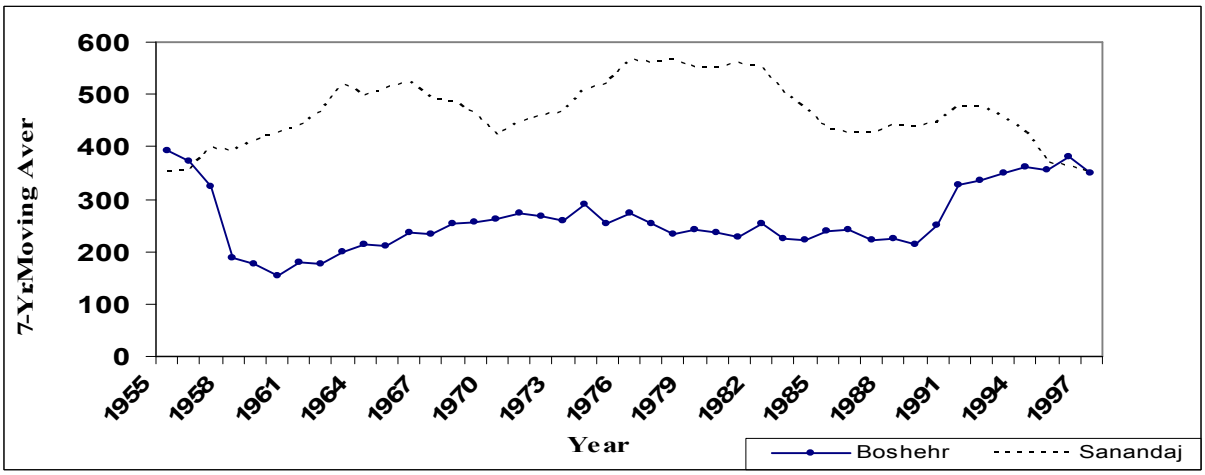


Figure 4. 7-year annual moving average at Bushehr and Sanandaj (in the northwest part) stations.

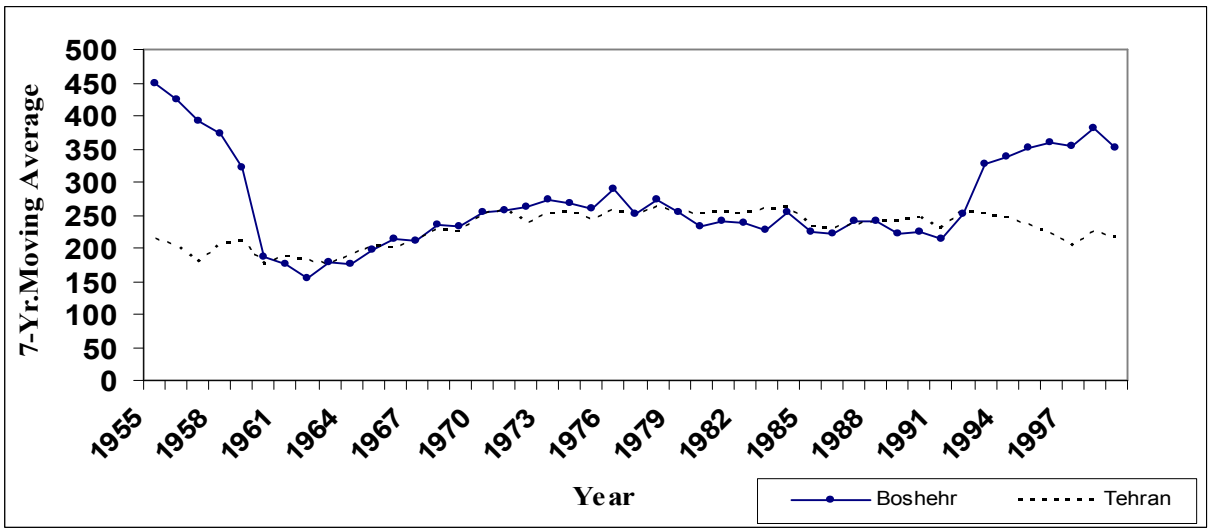


Figure 5. 7-year annual moving average at Bushehr and Tehran stations.

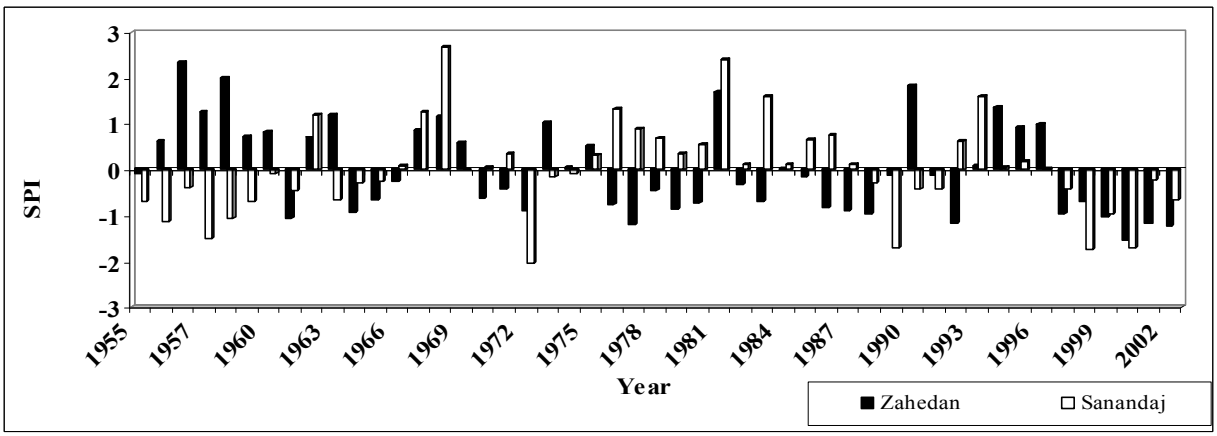


Figure6. Annual rainfall SPI for Sanandaj and Zahedan (in the southeast part) stations in Iran.

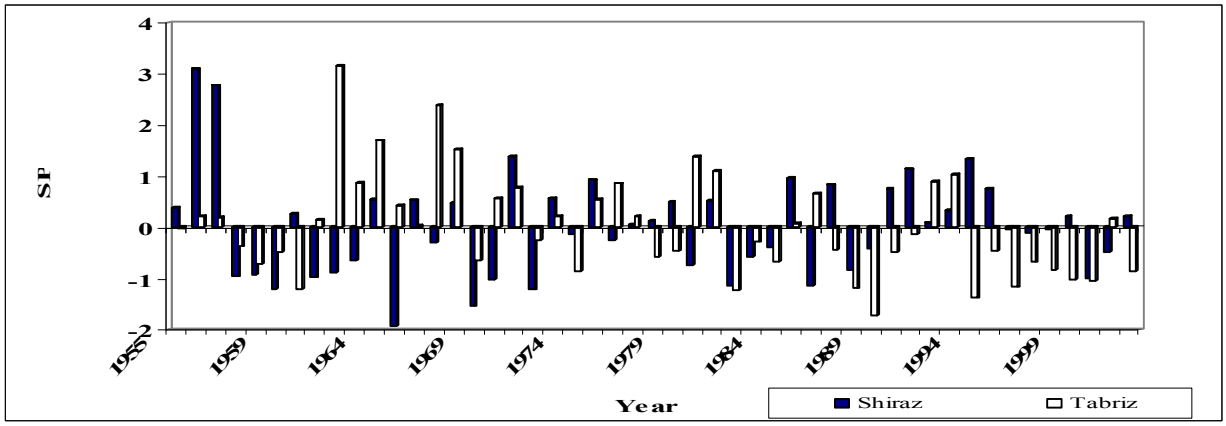


Figure7. Annual rainfall SPI for Shiraz (in the south part) and Tabriz stations in Iran.

Investigation of long term data series (49 years) of annual rainfall in 21 meteorological stations of arid and semi-arid regions of Iran showed that high temporal variability of annual rainfall is a natural feature of this climate. Coefficients of variation in all stations indicated that the uniformity of rainfall in this region is very low due to high irregularities. Most of the stations were characterized with different trend during 1955-2000 period. Therefore, for some stations can be concluded that annual rainfall was nearly stationary in the studied period. However, inherent irregularities and fluctuations in rainfall time series are the main factor responsible for scarcity of water resources and prolonged droughts.

- Water resources management strategies must be adjusted according to changing trends in rainfall and the spatial patterns of drought frequency. Most portions of Iran receive little precipitation and show a downward trend in rainfall in specific years, and at the same time, it can be seen an upward trend in precipitation at some parts of the country. For instance, some of these different behaviors are shown in figure 2 to 7.
- The southern, central and northern Iran have experienced different wet and drought severity at the selected long term period. The existence of a high drought severity condition over some parts and at the same time a low drought severity or even, a wet condition should be caused the water resources managers and decision-makers to design appropriate strategies in each part of the country.
- More attention should be focused on precipitation trends and drought distribution during the agricultural development decision-making process and drought hazards mitigation.
- Future studies need to investigate the specific effect of drought on sandstorms and the regional environment.

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AN APPLICATION OF GPCC AND NCEP/NCAR DATASETS FOR DROUGHT MONITORING IN IRAN

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Abstract: The lack of reliable and updated precipitation datasets is the most important limitation that hinders establishing a drought monitoring and early warning system in Iran, which is one of the most drought prone countries in the world. To overcome this obstacle we have evaluated the applicability of GPCC and NCEP/NCAR precipitation datasets for drought analysis in Iran. For this purpose, drought variability across the country has been analyzed by computing the Standardized Precipitation Index (SPI) on 12-month time scale based on the common period 1951–2005. For each dataset, by applying the S-mode PCA to the SPI field and Varimax rotation, the studied area has been regionalized into a few distinctive sub-regions characterized by independent climatic variability. To verify the results against observations at rain gauges, 32 synoptic stations having monthly precipitation data for the study period have been considered and the same procedure has been applied to analyze the space/time variability of drought across the country. Results show that both datasets identify the same sub-regions of drought variability and they are in good agreement with those found using observations. However, the NCEP rotated principal components associated with the sub-regions show different time variability with respect to the behaviors captured by GPCC and observations. Such differences that concern mainly the period 1951–1970 have been investigated. Thus, the study suggests that GPCC precipitation dataset is a useful tool for drought monitoring in Iran and can be used to complement the information provided by rain gauge observations. The NCEP/NCAR reanalysis dataset shows a good agreement with observations for the period 1970–2005, and its discrepancies with GPCC and observations should be taken into account when periods before 1970 are considered.

Key words: Drought monitoring, Principal Component Analysis, Reanalysis data, Standardized Precipitation Index.

1. INTRODUCTION

Drought monitoring, which is a crucial component of drought management, aims to provide information that enable and support people and organizations to take actions to reduce potential drought-related impacts and damages. However, we usually tend to focus on drought when it is occurring and to react when crises strike. Recent experiences and developments on drought knowledge are encouraging societies to shift from the traditional crisis-based management to a risk management approach (Wilhite et al. 2000). The difficulty in adopting the latter approach is mainly due to the lack of reliable and updated data that unable to fulfil the different components of drought risk analysis (Hayes et al. 2004). As described by Hayes et al. (2004), hazard analysis is the first step of drought risk assessment, and it may be an easy or difficult task depending on the availability of climatological records.

While some developed countries, such as USA and Australia, enjoy drought monitoring and early warning systems (http://drought.unl.edu/dm and http://www.bom.gov.au/climate/drought), the lack of updated and reliable meteorological data is still the major limitation in establishing such useful tools in many developing countries like Iran. According to Bordi et al. (2006), for a drought monitoring system and in assessing dry or wet periods in any given area it is highly desirable to have at hand a dataset that: (i) is easy to access, (ii) uniformly covers the area, (iii) has a time-duration sufficiently long to be trustworthy in a statistical sense and (iv) is optimal in the sense of capturing consistently dry and wet events. Most of the available records may match one or more of

these requirements, but not (especially the first) all of them. In Iran, the organizations responsible for collecting meteorological data delay 3 to 5 years in publishing the updated data. So, the lack of updated data is one of the most important obstacles for drought monitoring. Moreover, many station's records in Iran have substantial gaps leading to many low quality and in-homogeneity problems. Also the shortness of climatological records represents a limitation since the stations with the longest records, from 1951 up to present, are very limited and sparsely distributed across the country. On the other hand, the high spatial and temporal variability of precipitation over Iran, the frequent dry periods and the increasing water demands for an ever growing population, as well as for industry and economic development, including irrigation, aggravate water scarcity and make difficult a rationale water management (Raziei et al. 2009). Thus, it is highly desirable to have an updated dataset for Iran that allows capturing the historical drought occurrences and uniformly covers the country in order to properly represent the spatial variability of drought phenomenon within the area. For this purpose two kinds of data are nowadays available: the gridded data based on observations and the reanalysis data based on assimilation schemes.

The Global Precipitation Climatology Centre (GPCC) has produced and released several products, among them the Full Data Reanalysis Product Version 4 consisting of global land surface monthly precipitation at different spatial resolutions. The analysis is based on all stations, near real-time and non real-time, and covers the period 1901–2007 (Schneider et al., 2008). These data satisfy the criteria described above and, therefore, may be used to assess dry/wet periods over the globe for approximately the last hundred years.

Dinku et al. (2008) have recently evaluated the performance of the GPCC Full Data Reanalysis Product Version 3, covering the period 1951–2004, using a gauge network over complex topography in Africa. They found a good agreement between the gridded product and the reference rain-gauge data. However, for what concerns the GPCC Full Data Product Version 4, due to its recent update (September 2008), no studies are yet documented in the international literature, motivating our attempt to use this dataset for drought monitoring in Iran.

The reanalysis products represent another source of data. The analyzed fields are the result of complex interactions between all available observations and model results, and the final products are uniformly gridded fields on a global scale. In the latest decade or so, two of such reanalyses became easily available: one produced by the National Center for Environmental Prediction/National Center for Atmospheric Research (NCEP/NCAR) and the other one by the European Centre for Medium-Range Weather Forecasts (ECMWF), the ERA-40 (Kalnay et al. 1996; Uppala 2002). Precipitation, which is the main variable for drought assessment, is not a primary variable of the reanalysis products, i.e. precipitation observations are not assimilated into the model, thus, precipitation fields generated by the reanalysis system are short-range model forecast accumulations. Some authors (Bordi et al. 2006) have attempted to evaluate the performances of these reanalysis data in capturing dry/wet periods around the globe. Since, differently from ERA-40 product that covers the period 1957–2002, the NCEP dataset is continually updated every month from 1948, in the present study we use the latter for our purpose that is to evaluate the applicability of GPCC and NCEP/NCAR precipitation datasets for drought monitoring in Iran. The reliability of these datasets has been checked against observations at 32 rain gauges that have the longer and more reliable data records.

Dry and wet conditions have been assessed applying the Standardized Precipitation Index (SPI) as defined by McKee et al. (1993). The index is widely used in drought monitoring centres and in several drought studies (see for example Lloyd-Hughes and Saunders 2002; Tsakiris and Vangelis 2004). The SPI, based only on precipitation, is in fact a useful tool for capturing the climate variability associated with water shortage or surplus in different areas, and has several advantages (Keyantash and Dracup 2002). Former applications of SPI for assessing the spatial variability of droughts in Iran proved appropriate (Raziei et al., 2008, 2009). Furthermore, in analyzing the spatial and temporal variability of drought across Iran we applied the Principal Component Analysis (PCA) to the SPI field and Varimax rotation.

2. DATA AND METHODS

Data used for the analysis are monthly precipitation from GPCC Full Data Product Version 4, from NCEP/NCAR reanalysis dataset, and from 32 synoptic stations having the longest historical data records.

The Full Data Product Version 4, updated in September 2008, is a gauge-based gridded monthly precipitation dataset for the global land surface, available in spatial resolutions of 0.5° x 0.5°, 1.0° x 1.0° and 2.5° x 2.5° geographical longitude by latitude. The dataset covers the period from 1901 to 2007 and is based on both non real-time and real-time stations (Schneider et al. 2008). Several quality controls have been applied before gridding (Beck et al. 2005). The Full Data reanalysis is freely available via Internet (http://gpcc.dwd.de) and will be updated at irregular time intervals subsequent to significant data base improvements. In the present study we have considered the spatial resolution 2.5° x 2.5° longitude by latitude, and only the grid points over Iran.

The NCEP/NCAR monthly mean precipitation rates have been retrieved from the NCEP/NCAR reanalysis archive covering the period from January 1948 to present. Precipitation data are available on a regular grid 1.9^{o}x1.9^{o} in longitude and latitude and have been derived from the primary meteorological fields of the NCEP medium range forecasting spectral model based on the assimilation of a set of observations (see Kalnay et al. 1996 for details on the reanalysis procedure). Data were obtained via internet (http://www.cdc.noaa.gov). In the present study we have considered only the grid points over Iran.

To evaluate the applicability and performance of GPCC and NCEP/NCAR data sets for drought monitoring in Iran, monthly precipitation time series from 32 historical Iranian synoptic rain-gauge stations have been considered. The time records in these stations vary from 1951–1953 to 2005, which is the latest release of meteorological data in Iran. These time series have a few missing values and are the more reliable precipitation data available for meteorological and climatological analyses in Iran. Missing values have been estimated using MOVE4 technique (Maintenance of Variance Extension, Vogel and Stedinger 1985).

In order to assess hydrological drought conditions over Iran, the SPI on 12-month time scale has been computed for GPCC, NCEP/NCAR and observation datasets for the common period 1951–2005. The SPI is based on the precipitation field alone and its computation for any location is based on the long-term precipitation record cumulated over the selected time scale. The long-term record is fitted to a probability distribution, usually a Gamma distribution, which is then transformed through an equal-probability transformation into a normal distribution. The index is a useful tool for monitoring the different kinds of drought and comparing climatic conditions of areas governed by different hydrological regimes.

To capture the patterns of drought co-variability at different stations/grid points, the PCA (Rencher 1998) was applied to the SPI time series of GPCC, NCEP/NCAR and observations. The method consists in computing the covariance matrix of the SPI data with the corresponding eigenvalues and eigenvectors. The projection of the SPI fields onto the orthonormal eigenfunctions provides the principal components or PC score time series. In guiding a proper interpretation of the results shown in the next section, we remark that the spatial patterns (eigenvectors), properly normalized (divided by their Euclidean norm and multiplied by the square root of the corresponding eigenvalues), are called “loadings”; they represent the correlation between the original data (in our case, the SPI time series at single stations/grid points) and the corresponding principal component time series.

In order to find more localized spatial patterns of variability we applied the Varimax rotation to the loadings. This method allows finding areas within the region that have rather independent climatic variability, i.e. the rotated principal components are temporally orthogonal (Rencher 1998). Following the rule by North et al. (1982), we have estimated the sampling errors of the eigenvalues associated to principal components and we have established how many loadings to retain for rotation.

3. RESULTS

To capture the principal modes of spatial and temporal variability of hydrological drought across Iran, we have applied the PCA to the SPI time series computed using GPCC, NCEP/NCAR and rain-gauge data. Based on the scree plot and the North's rule of thumb, we have selected for each data set the first four loadings for Varimax rotation. The cumulative percentages of the total variance explained by the selected PCs for GPCC, NCEP/NCAR and observations are 72.2%, 77.5% and 58.5%, respectively (Table 1).

Table 1 Percentages of the explained variances of un-rotated and Varimax rotated PCs for GPCC, NCEP/NCAR and observations.

	GPCC		NCEP/NCAR		OBSERVATIONS	
PC	Un-Rotated (%)	Varimax Rotated (%)	Un-Rotated (%)	Varimax Rotated (%)	Un-Rotated (%)	Varimax Rotated (%)
PC1	45.4	25.5	45.3	28.1	31.6	20.4
PC2	13.3	23.8	16.8	26.1	11.6	16.6
PC3	8.4	15.1	8.5	14.5	8.6	12.4
PC4	5.0	7.7	7.0	8.9	6.7	9.2
Total	72.2	72.2	77.5	77.5	58.5	58.5

In Fig. 1 the rotated loadings (R-Loadings) for the three datasets are displayed and the percentages of the total variance explained by each PC are listed in Table 1.

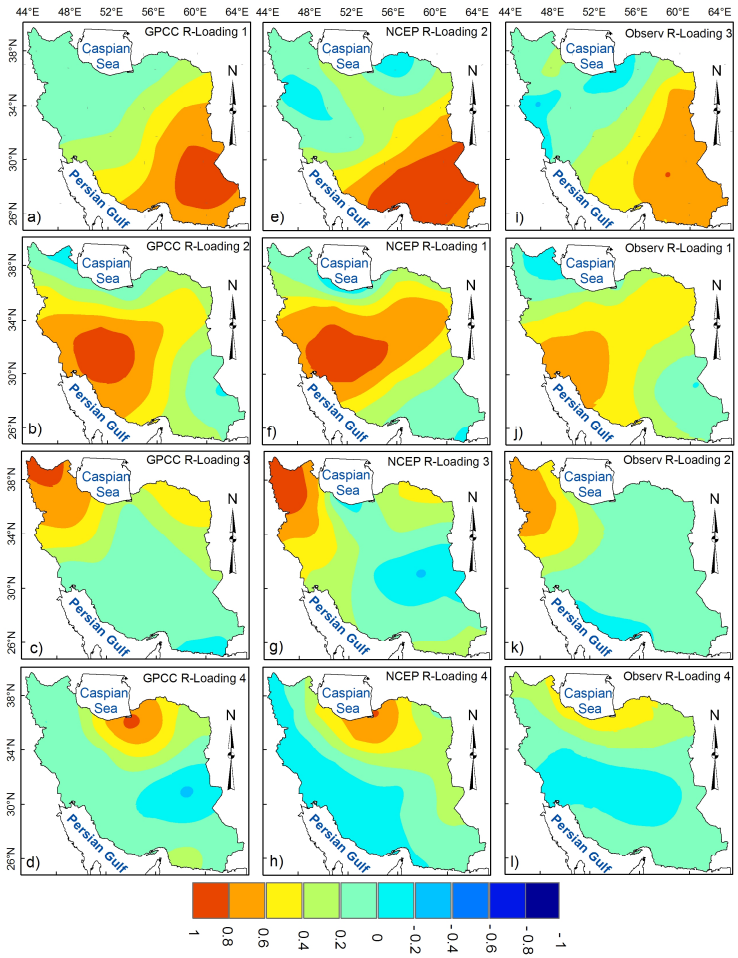


Figure 1. Rotated loadings obtained for GPCC (a to d), NCEP/NCAR (e to h) and observation datasets (i to l).

Figures 1a–d refer to the spatial patterns obtained for GPCC dataset. The first rotated loading is characterized by high positive values in south-eastern Iran, while R-Loading 2 has high values on

grid points located in central-western area. R-Loading 3 shows positive values greater than 0.6 in north-western Iran, and R-Loading 4 identifies an area in the north of the country, close to the Caspian sea, as a distinctive sub-region with positive values. As shown in Figs. 1e–h and Figs. 1i–l, the NCEP/NCAR dataset and the observations identify similar spatial patterns of variability, however the last with values smaller than those corresponding to the other two data series. Results are therefore coherent relative to the three different sources of data. Note that, in all cases the fourth rotated loadings explain a small percentage of the total variance (about 8–9%), suggesting that the robustness of the northern sub-region should be checked using longer time records. Moreover, in the case of the observations, R-Loading 4 reaches relative high values (0.4–0.6) only in a limited area close to the Caspian Sea, indicating that the time variability of the SPI there is not well captured by the corresponding rotated PC score. Thus, this feature should be analyzed further using a denser rain-gauge network.

In Fig. 2 the time series of the rotated PC scores (RPCs) for GPCC and NCEP/NCAR are displayed and compared with the ones obtained using observations.

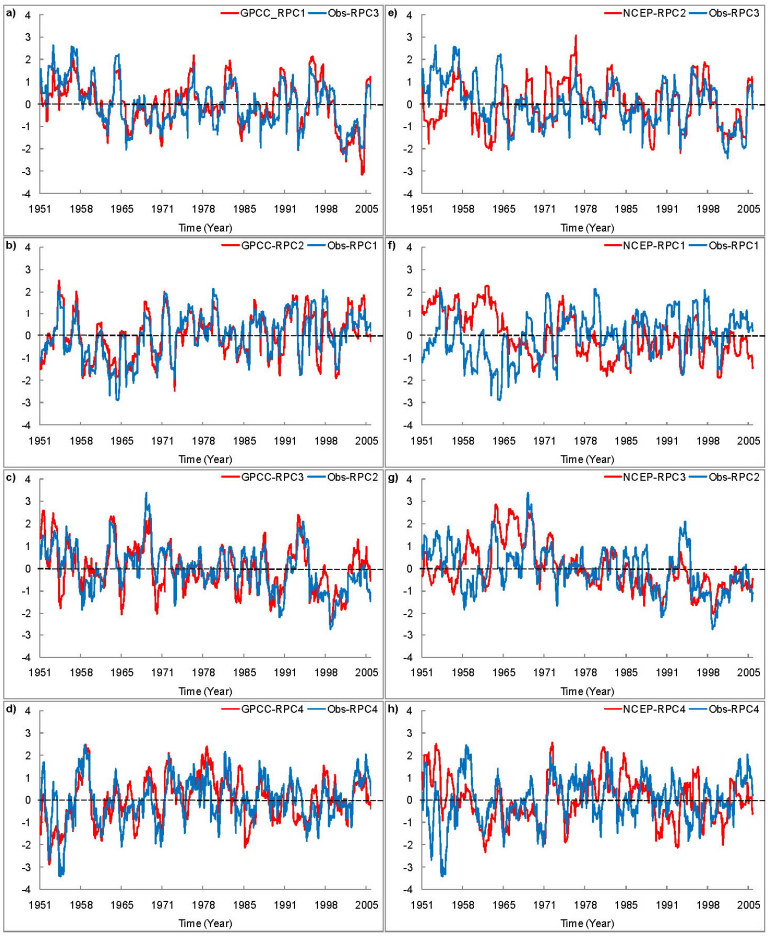


Figure 2. Standardized RPC score time series for GPCC and NCEP/NCAR dataset (red lines), and observations (blue lines).

From Fig. 2a–d it can be noted that the GPCC RPC scores (red lines), associated to the rotated loadings shown in Fig. 1a–d, are in good agreement with those obtained for observations and corresponding to the same identified sub-regions (blue lines). All the RPC time series are characterized by multi-year variability and there are not noticeable (statistically significant) long-term linear trends for the time section here considered. For what concerns the NCEP RPCs (Figs. 2e–h), it seems that the rotated scores associated to the south-eastern and north-western sub-regions (Figs. 2e and 2g, respectively) have a satisfactory agreement with the counterparts RPCs obtained for observations, while noticeable differences are found for the rotated scores associated with the other two sub-regions (Figs. 2f and 2h). In particular, both NCEP and observations capture a linear

trend (here not shown) towards negative values (dry periods) in the RPC scores associated with the north-western region (Fig. 2g): the fitting linear trend for NCEP accounts for about 27% of variability, while that for observations 18%. Also for the central sub-region (Fig. 2f) a linear trend is detectable, which is towards negative values for NCEP, accounting for about 33% of the RPC score variability, while it is towards positive values for observations and explains about 11% of variability.

Thus, GPCC, NCEP/NCAR and rain-gauge datasets are in good agreement in identifying the following four sub-regions as the principal spatial modes of drought variability: north-western, south-eastern, central-western and northern areas. These regions have different and independent time variability that is not univocally represented by the three datasets. In the next section we will investigate this discrepancy.

4. DISCUSSION

The correlation coefficients between the RPC time series for the three datasets are listed in Table 1, and those associated with the same sub-region are denoted in bold.

Table 1. Correlation coefficients between the RPCs obtained for GPCC, NCEP/NCAR and observations. Correlation coefficients between RPCs associated to the same sub-regions are in bold.

Dataset		GPCC				Observations			
	RPC	RPC1	RPC2	RPC3	RPC4	RPC1	RPC2	RPC3	RPC4
Observations	RPC1	0.15	**0.89**	-0.09	0.17	1	0	0	0
	RPC2	-0.10	0.18	**0.79**	-0.10	0	1	0	0
	RPC3	**0.85**	0.01	0.12	-0.12	0	0	1	0
	RPC4	0.07	-0.14	0.38	**0.66**	0	0	0	1
NCEP/NCAR	RPC1	0.28	**0.22**	0.14	-0.27	**0.08**	0.14	0.43	-0.21
	RPC2	**0.72**	0.27	-0.08	0.27	0.43	-0.09	**0.49**	0.20
	RPC3	-0.03	-0.25	**0.54**	0.15	-0.36	**0.54**	-0.03	0.19
	RPC4	0.07	0.01	0.34	**0.25**	0.06	0.27	0.20	**0.30**

As noted before, it appears that GPCC data have a good agreement with rain-gauge observations in identifying similar behaviour of drought variability in the four sub-regions. For what concerns NCEP/NCAR, the RPCs associated with north-western and south-eastern regions show good correlation with the counterpart RPCs of both GPCC and observations (correlation coefficients greater than 0.5), while very low correlation coefficients are found for the other two sub-regions, i.e. central-western and northern regions.

To better understand this feature, let us focus on northern and central-western regions. For the northern region, differences between the NCEP and observations RPCs (Fig. 2h) are noticeable in all the time record, suggesting an overall disagreement between the SPI time series in the area. For the central-western region (Fig. 2f), instead, the main differences between the two rotated scores seem to be particularly evident for the period 1951–1970. This is supported by the correlation maps (Fig. 3) between the SPI time series computed using NCEP/NCAR and observations for the time sections 1951–2005 and 1970–2005. The correlation coefficients have been computed after gridding the SPI data on the common grid 1^{o}x1^{o} in latitude and longitude. Considering the period 1951–2005 (Fig. 3a), high correlations characterize the north-western and south-eastern regions of Iran, while very low correlations are found in central-western and northern regions. When the period 1970–

2005 is considered (Fig. 3b), and the SPI is re-computed using this calibration period, the correlation map suggests an overall agreement among the two datasets and very low correlation coefficients are limited to the north-eastern sector, close to the Caspian sea.

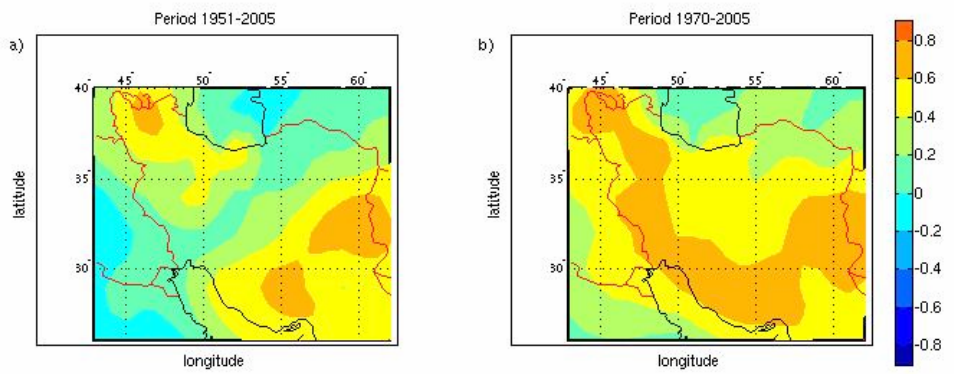


Figure 3. SPI correlation map for the periods: a) 1951-2005 and b) 1970-2005.

In view of these results, we have repeated our analysis for the period 1970–2005. Results for the two datasets (NCEP and observations, here not shown) suggest that the first two loadings can be retained for Varimax rotation. Two sub-regions are identified: northern (including Caspian sea region) and central-southern with independent climatic variability. The RPCs obtained from the two datasets show similar time behaviours, suggesting a very good agreement between NCEP/NCAR and observations (correlation coefficients of about 0.8). This means that the emergence of the other two distinct sub-regions, i.e. central-western and northern, are strongly related to the SPI time variability in the period 1951–1970.

On the other hand, for the GPCC dataset considering the shorter period 1970–2005 the first four loadings can be retained for Varimax rotation. As a consequence, four sub-regions result from the analysis as obtained for the longer period 1951–2005. The RPCs associated with the northern and southern regions are in good agreement with those obtained for NCEP and observations. Thus, further analysis is needed to better understand both the disagreement between NCEP and observations and the stability of the four sub-regions as a function of the record length.

5. CONCLUSIONS

In the present paper the application of GPCC and NCEP/NCAR datasets for drought monitoring in Iran has been investigated against observations at 32 historical rain-gauge stations. The common period 1951–2005 has been considered for the analysis that was based on the PCA of the SPI on 12-month time scale. The aim was to identify the sub-regions within the country characterized by independent drought variability. Results suggest that GPCC, NCEP/NCAR and observations identify four sub-regions: north-western, south-eastern, central-western and northern (close to the Caspian Sea) regions. However, while both GPCC and observations agree in identifying the time variability of the SPI in the four sub-regions, the NCEP/NCAR shows different time behaviours. The discrepancies between the NCEP RPC time series and those of GPCC and observations are particularly noticeable for the central-western and northern areas, and for the period 1951–1970. Since more observations were added into the assimilation scheme along the years, it is expected that the errors in NCEP reanalysis are greater for the first decades than for recent years, especially in regions situated in the lee side of mountains like central Iran. However, at this stage of the analysis the origin of these differences remains not clear and need further investigations.

In concluding, the study suggests that GPCC dataset is a useful tool for drought analysis in Iran and can be used to complement the information provided by rain-gauge observations. The NCEP/NCAR reanalysis dataset shows a good agreement with observations for the period 1970–2005, and its discrepancies with GPCC and observations should be taken into account when periods

before 1970 are considered. Moreover, further work should be done to investigate the stability of the four sub-regions with respect to the record length.

ACKNOWLEDGEMENTS

NCEP/NCAR data were obtained from the NOAA/CDC web site http://www.cdc.noaa,gov, while GPCC data were freely provided by the Deutscher Wetterdienst through their web site http://www.dwd.de. Rain-gauge data were provided by the Iranian Meteorological Organization. We thank Prof. Alfonso Sutera for his useful suggestions.

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USING TIME SERIES ANALYSIS AND ARIMA MODEL IN DROUGHT SIMULATION (A CASE STUDY: FARS PROVINCE, IRAN)

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Abstract: Drought is one of the most important natural disasters that can affect sectors with its very much frequency particularly in dry and semidry areas of Iran. Lack of raining has various effects on agriculture, surface water and groundwater resources, humidity of soil and rivers. Regarding the location of Fars province in dry and semidry climate area and the incidence of repeated drought, especially in recent years, has led to pay more attention to drought simulation and forecasting. Quantitative description of environmental process, the anticipation and estimation of its behavior at the time and situations without observation and sample, requires having a pattern and model of that process. One of the methods to access the goal of simulation in drought incident is the use of time series model. In this study, the raining statistics of 90 stations of pluviometry and synoptic in Fars province were used 30 years of statistic period. To determine the grade of drought severity, the standardized precipitation index (SPI) was also used due to its advantages in local analysis and establishing time relationship between drought incidents. Determination of desirable local model according to drought condition of Fars province was conducted after drought severity monitoring and evaluation. The suitable model for each region was provided by using time series analysis and based on provided local pattern in different regions of Fars province. The ARIMA method was selected in drought incident simulation using time series models based on the Autocorrelation and Partial-autocorrelation. Considering stability, parameters, and different methods, all probable models were evaluated and the most suitable model was selected and presented for drought simulation in each region.

Key words: Drought, Simulation, Time Series Analysis, ARIMA Model.

1. INTRODUCTION

Drought is one of the silent natural disasters which can leave destructive effects in different fields such as economy, society and environment. If the drought effects are not resolved by proper solutions, it would remain in the area for years. There is a unanimous agreement saying that drought phenomenon happens when there is not enough moisture due to rain shortfall in several periods of time. In general, drought problems are related with the time period in which these deficits are accumulated and their relation in rain shortfall appears in the shortages in resumed water resources [Mckee et. al. (1993)]. At the beginning of drought, agriculture section usually bears the most damage due to its excessive dependence on the water resources. If drought and short term rainfall continues, moisture in soil disappears rapidly and the agriculture section relies on surface and groundwater resources therefore it severs the water crisis. When the rainfall resumes, the condition turns back to normal and drought ends. Until the surface and groundwater resources revive, its ill consequences continue and its effects on surface and underground resources take for months or even years. Thus it can be concluded that recharging period of resources is a function of severity and continuity of drought and precipitation. It can be said that, in recent decades, drought has been the most common natural disaster which has affected human societies. Hence, there are two kinds of managements provided to control these climate disasters: (1) crisis management and (2) risk management.

Crisis management is a confronting approach against drought in which actions are taken without predefined planning and sometimes leads to ineffective, immediate and inharmonic actions by people and governments. Risk management is a reverse approach in comparison with the crisis management. This applies before drought occurs as a conservative method, to decrease the drought effects and organize required decisions on decreasing damages during drought period.

Evaluations on drought incidents have shown that applying crisis management actions has led to confusion and deficient actions and ineffective cooperation and planning which would increase vulnerability against drought and cost excessive budgets to pay for these damages [Ashgar toosi and Alizadeh(2003)]. Therefore, using different methods such as simulation, modeling and anticipating drought incidents, will result in presenting plans for equipment against drought and decreasing its damages. Since starting and ending times and continuity are the most features of drought, study on these subjects could generate anticipation, simulation and forecasting the drought incidents. Besides, quantitative description and perception of environmental process and the anticipation and estimation of its behavior at the time and situations without observation and sample requires have a pattern and model of that process. Without a model, the analyst only has a set of data and there would be no perception and conclusion on underlying variables taken at place no sampling has taken place. Generally speaking, modeling can be regarded as the art of simplification. In this case a model was made to present a proper framework and approach to acquire knowledge and process management and control. Achieving these objectives requires applying a wide range of data processing statistic methods for modeling [Mohammadi (2006)]. Time series models can be used for simulation and modeling the drought for behavior perception. Before drought simulation, this phenomenon should be measured and evaluated using time series analysis in order to perceive its behavior in the past and acquire a proper basis to make the model. To determine the severity of drought it should be turned from qualitative and descriptive state to quantitative and numeric, and define and present some indices for it. Basically the drought index is a function of different environmental factors. of drought ultimately showed as a number which can be used to evaluate and take decisions on drought and it is much easier and useful than several rows of various data. (To make decision making and evaluation more practical and easier, drought severity is shown by numbers instead of revealing several rows of various data.)Then by using the results from drought measurement and indices, the changes in drought can be examined during a given statistical period and forecast, and therefore simulate its incident. With the role of drought indices in determining severities of drought, time series and modeling this natural disaster, it should be used an index, have a high accurate in modeling and forecasting the drought. McKee et al. (1993) used the standardized precipitation index in Colorado and examined different features of drought in 3, 6, 12, 24 and 48 month scales and upon the results they provided a classified system to define the severity of drought. The results of examining the drought in 1996 by the standardized precipitation index showed that it revealed the incident of drought, one month before the Palmer index [Hayes et al. (1999)]. In a study [Tsakiris and Vangelis (2004)], the severity and time of drought defined as two main parameters which are very important in making decisions and they are commonly used parameters among the evaluation indices. The results show that SPI can be used as a drought watch system. In another study [Ghavidel rahimi (2004)], the annual precipitation data was used in a 43 statistical period for the weather stations in Eastern Azerbaijan province in order to analyze and modeling precipitation and calculate dry and humid years. Different precipitation-based indices used to determine quantity and therefore classify humid, normal and dry years. Because of its more abilities and less constraints, the standardized precipitation model was found the better option among the valid precipitation models, the standardized precipitation model and Niche model that have been used in the this study.

Once the given index is selected, time series methods should be used to provide simulation models and forecasting drought. Since 1970s the time series analysis, practically and theoretically, have been rapidly developed for forecasting and control. This analysis is usually related to dependant data correlate sequentially. This correlation is sequential between those observations which are more popular and would be used more for forecasting. One of the most important

methods of perception of future results which are based on the past events is the time series analysis in forecasting [Niroomand and Bozorgnia (2006)]. In a study on modeling precipitation procedure and drought forecasting for catchment area of a lake by using time series method, precipitation at selected sub-catchment areas of Oroumiyeh Lake has been examined. The statistical analysis of precipitation at the stations showed there were sever frequencies on annual precipitation during 39 years of statistical year which is an important climate feature for the catchment area and its stations. In general, in terms of drought incidents, the underlying stations imply weak to moderate droughts and examining precipitation procedure and the related calculations of forecasting future precipitation using time series method, drought incidents at most stations are forecasted [Ghavidel rahimi and Jahanbakhsh asl (2003)]. In another study [Parvin (2001)], using ARIMA model, forecasting of four stations at catchment area of Oroumiyeh Lake was forecasted and their precipitation was classified using Z-score index. The results showed a nearly normal state with ascending precipitation range, from 2000 to 2001. In another study in Khorasan province using SARIMA model patterns and MINITAB software, in a statistical period between 1970 to 2002, seasonal precipitation was modeled and finally using the provided models at each station, precipitation at four seasons (spring, summer, autumn and winter) was forecasted for 2004. After comparing with long-term average at each season, their anomaly maps were drawn [Ashgar toosi and Alizadeh (2003)]. Using time series methods in forecasting weather droughts is a study in which with descriptive methods and ARMA models, monthly forecasting was achieved with Palmer severity drought. The monthly data from 1929 to 1969 were used as non-linear computer calculations with minimum level for evaluating and estimating the parameters in ARMA model. The results show that Box-Jenkins model in compare with the other models is a better option. The results also show there was a relation between ARMA models and time series analysis [Davis and Rappoport (1974)].

Thus the present study is aimed to measure and evaluate drought across Fars province using standardized precipitation index (SPI), determining legalization and detecting the phenomenon behavior and presenting an optimal model for simulation and forecasting drought incident using time series models.

2. PROCEDURE

The time series are different in type. They include one and multi variable, correlated and uncorrelated, consistent and inconsistent. Consistency means that the possible governing rules are intact in time, or in other words, the governing procedure on the process is fixed in time. Since the consistency is the initial condition in modeling time series, it is necessary to stabilize the data series. There are two ways to stable the time series.

(A) Differentiation process: though many time series are inconsistent, for some equilibrium forces, different sections show very similar behavior and their differences is at local average. This sort of behavior is called homogenous inconsistency. As it is natural, these homogenous inconsistent time series are independent of their level. Such series are converted to consistent series with an appropriate differentiation.

(B) Variance stability conversions: not all inconsistency problems can be solved by differentiation. There are many time series that are consistent in average but inconsistent in variance. It is used a proper variance stability conversion to solve the problem, in other words, instead of main series of data it is used converted series which is consistent.

In this study, the raining statistics of 90 stations of pluviometry and synoptic in Fars province were used. These numbers of stations were selected among 363 stations accounting for different parameters such as: having a long term statistics, low statistical deficits and proper variance throughout the province. Also it was used a 30 year common statistical period (1972-1973 to 2001-2002) for statistical analysis. For utilizing the statistics, at first it was examined the monthly

precipitation data for all the stations using run-test statistics method for their validity and homogeneity. Once it is assured that the data is homogenous, the missing data for each station is estimated using the data at the closest station and Expectation Maximization algorithm with SPSS software. Comparing with the other methods, it has a high accuracy for estimating the missing data and can act based upon the expected maximum estimation and provides the best estimation processing sequential frequencies [Figueiredo (2004)]. For determining drought severities standardized precipitation index was used due to its advantages on location analysis and time connection between the drought incidents.

Because the data is used to analyze time series, severity of standardized precipitation index (SPI) and their average and variance are standardized. Most data are consistent and both methods were used for consistency test and validity of data consistency. Once it was assured the drought data was consistent, the proper model was built. In calculating the proper model for simulating drought incident in Fars province, based on the results of simulation and drought variance analysis, topography is provided using geographic information system in Fars province [Shamsnia et. al. (2007)], and similarity of drought incident procedure throughout the province (Fig. 1). Based on the results of classification maps, those areas which are similar in drought incident are considered as one area in modeling and one model is provided for them. Time series analysis is achieved through some methods. ARIMA or Box-Jenkins method is one of them which is also called (p, d, q) model. In every (p, d, q) model, p is related to the number of autoregressive values, d shows the number of integrated values and q indicates the number of moving average in ARIMA model. This analysis is done in several stages. In the first stage p, d and q values are determined using Autocorrelation and Partial-Autocorrelation methods. Then in the second stage, it is examined whether p and q values (autoregressive and moving average, consequently) can be remained in the model or they should be removed. At the third stage, it is discussed whether the remainder value (remainder error) is random and has a normal variance or not. If it is so it can be said the model is appropriate.

3. MODEL SELECTION CRITERIA

In time series analysis, or generally in data analysis, several proper models are used for showing definite data. Sometimes it is an easy selection but at other times it may be so hard. Thus indefinite criteria have been introduced for comparing the models differ from the model recognition method. Some of them are based on the shortened statistic from the remainders (which are calculated from a fit model) and some others are determined based on the anticipation error (calculated from the anticipations out of sample). For first method, there are Akaike Information Criterion (AIC), Bayesian Information Criterion (BIC) and Schwarz Bayesian Criterion (SBC) criteria and for the second method, upon the anticipation error, there are Mean Percentage Error (MPE), Mean Square Error (MSE), Mean Absolute Error (MAE) and Mean Absolute Percentage Error (MAPE). The model which uses the least statistics is selected as the proper model. These criteria can be tested and examined by much statistical software.

In the present analysis, at first ARIMA method was used and different values of p, d and q were tested within the stages. Since AIC criterion, in using ARIMA model, has a high accuracy, and comparing the two ARIMA models acts better for selecting the best fit equations [10]. Thus in the present study AIC criterion has been used, which is one of the criterion used for selecting the model upon the shortened statistics from the remainders.

Then the desired model for each specific area was provided using the minimum AIC value. By having the drought severity values within the range of 95% safety coefficient and with the topography of Fars province, the equation of model for each area was provided and presented. Equation (1) shows the general format of the time series model.

$$\hat{Y}_{(t)} = \mu + Y_{(t-1)} + \phi_1 (Y_{(t-1)} - Y_{(t-2)}) - \phi_2 (Y_{(t-2)} - Y_{(t-3)}) + \phi_3 (Y_{(t-3)} - Y_{(t-4)}) + \theta_1 * e_{(t-1)} - \theta_2 * e_{(t-2)}$$

[1]

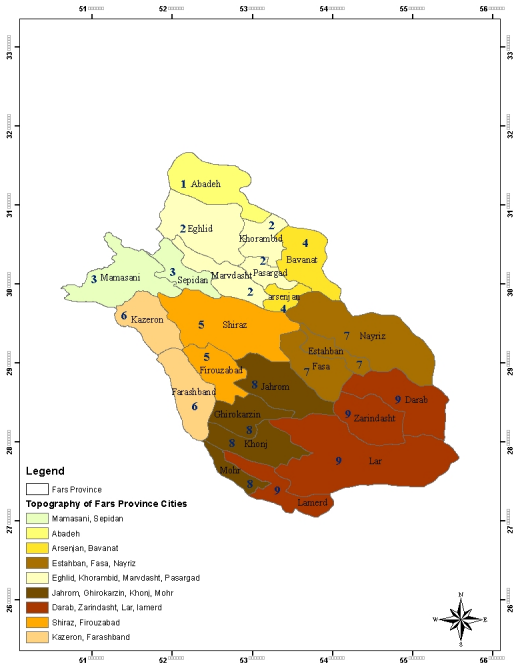


Figure 1. Topography of Fars province cities for simulation and drought incident map.

4. CONCLUSIONS

As it was mentioned earlier, the model was built based on the topography of cities in Fars province. Name of the cities and the numbered regions are provided in the topography and the desired ARIMA model is shown in terms of the number of p, d and q values for simulation and drought incidents. For the desired model, the results show that after doing various tests, ultimately the model ended up to p and q values. In all states d value was zero and the final model formed to the ARMA method. But since the work achieved by the ARIMA method, it is better to call it the ARIMA method.

Table 1 – Topographical features for Fars province cities and the appropriate model of simulation and drought incidents

Type of ARIMA model	Name of cities	Number of region
ARIMA(2 0 1)	Abadeh	1
ARIMA(3 0 2)	Eghlid, Khorambid, Marvdasht and Pasargad	2
ARIMA(3 0 2)	Mamasani and Sepidan	3
ARIMA(3 0 2)	Bavanat and Arsenjan	4
ARIMA(3 0 2)	Shiraz and Firouzabad	5
ARIMA(3 0 2)	Kazeron and Farashband	6
ARIMA(4 0 2)	Estahban, Fasa and Nayriz	7
ARIMA(1 0 1)	Jahrom, Ghirokarzin, Khonj and Mohr	8
ARIMA(1 0 0)	Darab, Zarindasht, Lar and Lamerd	9

Upon the number of autoregressive and moving average values at teach region and according to the general equation of time series, the simulation and drought incident model was shown (Table 2) for each 9 regions. On every equation, $Y^{(t)}$ is a time variable provided upon the number of autoregressive values. The numeric values related to $Y^{(t)}$ variable are also the autoregressive values upon the Autocorrelation and Partial-Autocorrelation methods which are left

after different tests. In the model equations, e implies the remainder error related to the moving average values. In all provided models, AIC index value considered at the minimum level when providing the desired map the variance of points around the fit line remain at the best conditions. As the present models, are simulated upon the drought similarities in different regions and incident procedure, thus they can be used following evaluation and reliability toward forecasting drought incidents across Fars province.

Table 2 – The appropriate models for simulating and drought incident in Fars province in terms of region upon the topography

Suitable model for simulation of drought incidents	Number of region
$\hat{Y}_{(t)} = Y_{(t-1)} + 1.52(Y_{(t-1)} - Y_{(t-2)}) - 0.6(Y_{(t-2)} - Y_{(t-3)}) + 0.7 * e_{(t-1)}$	1
$\hat{Y}_{(t)} = Y_{(t-1)} + 1.89(Y_{(t-1)} - Y_{(t-2)}) - 1.35(Y_{(t-2)} - Y_{(t-3)}) + 0.33(Y_{(t-3)} - Y_{(t-4)}) + 1.22 * e_{(t-1)} - 0.68 * e_{(t-2)}$	2
$\hat{Y}_{(t)} = Y_{(t-1)} + 1.8(Y_{(t-1)} - Y_{(t-2)}) - 1.27(Y_{(t-2)} - Y_{(t-3)}) + 0.32(Y_{(t-3)} - Y_{(t-4)}) + 1.16 * e_{(t-1)} - 0.71 * e_{(t-2)}$	3
$\hat{Y}_{(t)} = Y_{(t-1)} + 1.65(Y_{(t-1)} - Y_{(t-2)}) - 1.29(Y_{(t-2)} - Y_{(t-3)}) + 0.42(Y_{(t-3)} - Y_{(t-4)}) + 1.05 * e_{(t-1)} - 0.83 * e_{(t-2)}$	4
$\hat{Y}_{(t)} = Y_{(t-1)} + 0.54(Y_{(t-1)} - Y_{(t-2)}) - 0.85(Y_{(t-2)} - Y_{(t-3)}) + 0.74(Y_{(t-3)} - Y_{(t-4)}) - 0.19 * e_{(t-1)} - 1 * e_{(t-2)}$	5
$\hat{Y}_{(t)} = Y_{(t-1)} + 2.07(Y_{(t-1)} - Y_{(t-2)}) - 1.64(Y_{(t-2)} - Y_{(t-3)}) + 0.47(Y_{(t-3)} - Y_{(t-4)}) + 1.34 * e_{(t-1)} - 0.77 * e_{(t-2)}$	6
$\hat{Y}_{(t)} = Y_{(t-1)} + 1.52(Y_{(t-1)} - Y_{(t-2)}) - 1.36(Y_{(t-2)} - Y_{(t-3)}) + 0.73(Y_{(t-3)} - Y_{(t-4)}) - 0.14(Y_{(t-4)} - Y_{(t-5)}) + 0.81 * e_{(t-1)} - 0.77 * e_{(t-2)}$	7
$\hat{Y}_{(t)} = Y_{(t-1)} + 0.62(Y_{(t-2)} - Y_{(t-3)}) - 0.73 * e_{(t-1)}$	8
$\hat{Y}_{(t)} = Y_{(t-1)} + 0.83(Y_{(t-1)} - Y_{(t-2)})$	9

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DROUGHT MANAGEMENT IN THE IBERIAN PENINSULA SHARED RIVER BASINS: DEVELOPMENTS AND PERSPECTIVES

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Abstract: In Europe, the 40 shared river basin districts represent more than 60% of the whole territory. In that context, the Water Framework Directive (WFD) strongly promotes the cooperation between the Member States sharing those river basin districts in order to develop a "River Basin Management Plan" (RBMP) and also ensure that 'good status' for all of Europe's surface and groundwater within a 15-year period is achieved.

This context applies to the Iberian Peninsula and is part of the frame of the management of drought situations and, namely, of the common implementation of drought management plans (DMP). In order of that, following a short description of the specificities of the Iberian Peninsula regarding water resources management and the active bilateral treaties and conventions, some insights on the efforts that have been developed by the two countries to reform their institutional, legislative and administrative water frameworks in order to comply with the WFD requirements will be presented. Also, some bilateral efforts and actions concerning some important interrelated water issues namely on the management of extreme events and on the establishment of an ecological river flow regime will be referred. This analysis will be presented having in mind the "Convention on Co-operation for Portuguese-Spanish River Basins Protection and Sustainable Use", approved in 1998 that aims at embracing the sustainable development of all the shared river basins of the Iberian Peninsula. This Convention is of great importance in the current context as it introduces a basin-based water management and a decision-making process in compliance with the WFD, framed on coordinated River (sub-) Basin Management Plans.

The last part of the work will be specifically dedicated to droughts management and planning in the Iberian Peninsula context. For that purpose, a characterization of the corresponding existing conditions and of the expected developments will be presented, trying to highlight not only the current still existing insufficiencies but also the perspectives regarding those issues in each country. This comparison will be further enlarged to the existing bilateral cooperation, promoting the application of the know-how and experience of each country on droughts management to the definition and implementation of a cohesive and thorough Iberian shared river basins system of indicators that was already contemplated in the Albufeira Convention, but that has not been created yet. The importance and impact of climate change will also be referred in the context of the WFD.

Key words: Albufeira Convention, Drought Management, Drought Indicators, Water Framework Directive.

1. INTRODUCTION

During the last decade, shared water resources management in the Iberian Peninsula has been having important developments under the framework: (i) at the Iberian level, of the Albufeira Convention signed in 1998 (active since 2000); and, (ii) at the European level, of the Water Framework Directive, approved in 2000.

The Albufeira Convention, as it will be briefly explained in the following sections, represents the logical evolution of the existing treaties and conventions between Portugal and Spain, since the 18th century. For the first time, the context of bilateral cooperation between the two countries was set regarding the development and management of the shared river basins and not only (as before) of the shared river stretches, anticipating some of the requirements that would be defined in the Water Framework Directive (WFD) two years later, this last being an important trigger of reform of the water policy and of the institutional settings in the two countries (Maia, 2008).

Both the Albufeira Convention and the WFD emphasize the need of coordination and cooperation on transboundary river basins management by the riparian countries. According to

WFD that shall be translated on: (i) the guarantee that an international river basin is formed; and, (ii) the production of a single RBMP (River Basin Management Plan) or, in case that it is not possible, an RBMP for each Member State territory's river basin part. The RBMP may be complemented by specific programs and management plans, at sub-basin, sectoral or problem level. In fact, the need for a real European droughts policy has recently emerged, and it also became clear that the institutional framework of the different European countries is not prepared to face real droughts occurrence. Increasing Europe's vulnerability to droughts (combined with increasing water needs) urges that drought mitigation procedures are implemented and coordinated, especially when shared river basins are concerned. The paper emphasizes the importance of the elaboration and common implementation of drought management plans (DMP) and policy and analyses related and common issues and present corresponding perspectives of expected and foreseen developments in the Iberian Peninsula.

2. THE IBERIAN PENINSULA CURRENT CONTEXT

As previously referred, the Iberian Peninsula, presents a dense past of treaties and conventions that started in the 18th Century. The five shared river basins (Minho. Lima, Douro, Tejo and Guadiana) cover more than 45% of the Iberian territory but only 21% of that share corresponds to Portugal, although in terms of internal (surface and ground) natural water resources of those basins the portuguese share is almost one third (32,1%). Nevertheless, more than 70% of total water use on those shared river basins does occur in Spain, where more than 2/3 of the corresponding existing storage capacity is located (Maia, 2008).

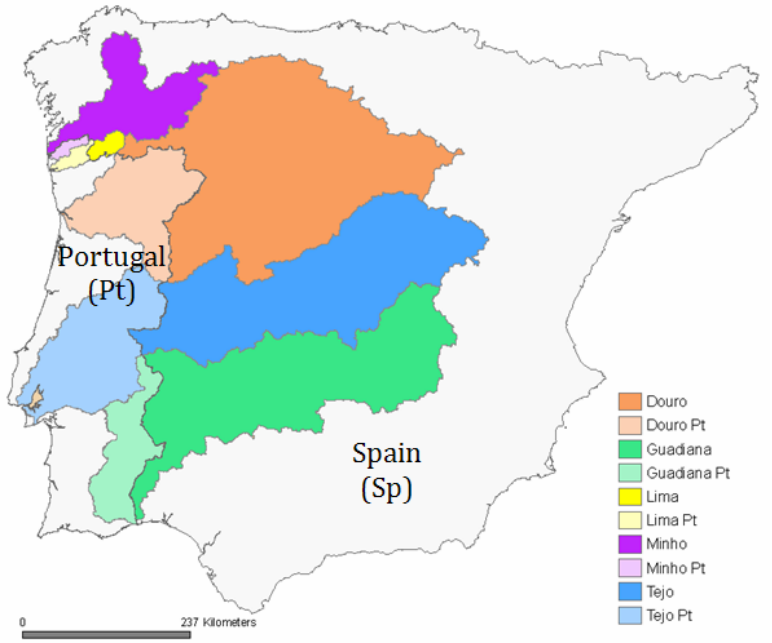


Figure 1. The Iberian Peninsula and the Five Shared River Basins (adapted from INAG – www.inag.pt).

Since 1864, when the Treaty of Limits was signed, several Agreements and Conventions were established between Portugal and Spain strengthening the cooperation on shared rivers water use between the two countries. The still active (but mostly ineffective) conventions on the Douro river (dating 1964) and on the other shared rivers (dating 1968) were mostly concerned with the bordering river stretches and aiming the hydroelectric potential share. By the end of the last century, following the rich context of bilateral cooperation, an adequacy of the bilateral agreements between the two countries was recognized as needed not only to solve the 1964 and 1968 Conventions mutually recognized limitations but also to adapt that cooperation to the (by then) common policy for water resources management meanwhile on preparation by the EU. That led to the celebration of the Convention on Co-operation for Portuguese-Spanish River Basins Protection and Sustainable

Use, usually referenced as the Albufeira Convention, signed in 1998 and that came into force in January 2000. The Convention embraced all the river basins territory and was a notorious step towards the definition of the framework of bilateral co-operation for freshwater and groundwater protection and also for sustainable use of the five shared rivers watershed resources. It should be emphasised that the two countries put a significant highlight on introducing a basin-based water management and a decision-making process, framed on coordinated River (sub-) Basin Management Plans, one per country (as it is being followed on the first cycle of WFD implementation).

The creation and (increasing) role of the Institutional Boards defined within this convention, the CADC (Commission for Convention Development and Appliance) and the Parts Conference, acknowledge the importance recognized by the two countries in shared river basins' management. In that context, different Work Groups were created to specifically deal with some particular issues as, more recently: (i) flow regime, droughts and emergency situations; (ii) information exchange; (iii) hydraulics structures security and floods; and (iv) WFD and water quality. A sub-commission dedicated to public participation was also formed.

Following the increasing role of the CADC on common WFD implementation, and to ensure a good efficiency and efficacy of this board, a Permanent Technical Secretariat was more recently (2008) formed. Among its main work objectives for 2008-2010, one should refer: (i) the definition of the methodologies for the elaboration of the Management Plans of the five River Basin Districts; (ii) the adequacy of the flow regime of the River Basin Districts Management Plans with the flow regime of the Convention, and (iii) the assessment of the bi-national public participation process.

Meanwhile, both Portugal and Spain have been working on making homogeneous their water resources management units and to reform their internal laws, in compliance with the WFD. In order of that, Portugal and Spain transposed the WFD into national Law and undertook strong legislative efforts, managing to introduce the necessary adjustments to reach homogeneity in the RBD's definition. In Portugal, eight continental River Basin Districts (RBD'S, referred internally as Regiões Hidrográficas, RH) were defined under the correspondent new Water Law and had clearly in mind the corresponding river basin parts in Spain (see Figure 1): Minho and Lima were joined in one (RH1), all the others alone (Douro, in RH3; Tejo, in RH5; Guadiana, in RH7). Spain, where water resources management has been made for a long time on a river basin basis, later adapted to that by sub-dividing the former Confederación del Norte to create a specific RBD (Demarcación Hidrográfica) to include Minho and Lima rivers, all the other shared rivers RBD's keeping basically the same geographical definition as since the last century. Maia (2008) gives a more in depth description on the process of institutional adequacy carried out by the two countries.

3. THE WATER FRAMEWORK DIRECTIVE IMPLEMENTATION: CURRENT SITUATION

The WFD is a "dynamic" Directive not only because its implementation process is foreseen to be continuously under scrutiny and updated over six years cycles but also because it may foster different Directives to fill some gaps of the WFD, as it is the case of the ones already implemented on groundwater (Directive 2006/118/CE) and on floods (Directive 2007/60/CE). In what concerns drought mitigation issues, which are of relevance for southern European countries, namely Portugal and Spain, some important efforts have still to be made, as referred later more in detail.

The report on the first stage of the WFD implementation (SEC, 2007) reveals that Portugal managed to, globally, respond to the three main requirements that were evaluated: (i) the transposition of the WFD; (ii) the report on Article 3 – River Basin Districts and Competent Authorities; and, (iii) the report on Article 5 – Analysis of the River Basin Districts characteristics. By then, Spain had achieved less satisfactory results, with a global reporting performance below 50%, against 85% for Portugal (SEC, 2007), namely because of some delay in complying with WFD requirements on RBD's delimitation. The report pointed also that despite the main

improvements achieved by both countries for promoting joint work in order to attend the WFD goals, a need for a more dynamic and coordinated implementation process is still needed.

In what concerns the current key issues of the WFD implementation process, both Portugal and Spain are seen as having difficulties in complying with the EU scheduled contributions up to the end of 2009 (WISE, 2009). In fact, in Portugal, although approved in 2005 and expected to be fully operational by 2007, the (5) River Basin Districts Administrations (ARH, Administrações das Regiões Hidrográficas) became only really operative in June 2008 and only now (May 2009) the process for RBMP's elaboration is to be launched. This fact translated naturally not only in the delay of RBMP (draft and final version preparation) process but also of the corresponding one on public participation and consultation, although the work program consultation sessions had (still) occurred in due time (1st semester 2007). The public discussion sessions on Significant Water Management Issues (SWMI) for each of the eight RH's (based on reports prepared by the correspondent ARHs in January) have been occurring (to finish end of May), the ones on shared River Basin Districts with double sessions (one in Portugal and one in Spain), with the participation of both RBD Administrations and CADC.

WISE (2009) refers Spain as with some already published RBMP's. According to the (good) online information available from the responsible Ministry (Ministerio de Medio Ambiente y Medio Rural y Marino) and from the "Demarcaciones Hidrograficas", that does not refer to mainland, where no draft versions of the RBMP's are yet available. Only the Balearic Islands have already submitted a version of the RBMP to public consultation process, ended in March 2009. In fact In both mainland Portugal and Spain, still no date for the consultation process on RBMP is known yet.

It is clear that both countries have important efforts to develop to be able to comply with the deadlines set by the Water Framework Directive.

4. BILATERAL COMMON ISSUES

In the context of the five shared river basins, Portugal and Spain have some important interrelated bilateral water issues, resumed by Maia (2003). Some of those, although having already required and benefited from the development and intensification of bilateral cooperation, still possibly need or require further efforts, namely: (i) institutional adequacy; (ii) establishment of a river flow regime; and, (iii) coordination of extreme hydrological situations.

Considering the institutional adequacy, and although the major efforts that have already been emphasised on the previous sections, the effective role and authority of the CADC on the envisaged real coordinated and integrated water resources management of the Portuguese-Spanish river basins still needs to be developed and framed in the next future to better adequate it to WFD provisions.

In what concerns the establishment of a river flow regime, an important evolution has been achieved. Although the Albufeira Convention states the need of the establishment of an adequate flow regime for each shared river basin (having in mind, namely, the fulfilment of environmental goals), only ten years after the Convention's signature a guarantee (stated as provisory in 1998) of minimum annual flows (except for Lima) at established monitoring (bordering and estuarine, in this case only for Douro and Tejo) stations (and also instantaneous flow values for two stations, each one at the upstream section of the two bordering stretches), but only for "non-excepcional" years, was approved by the two countries. That regime, approved by the Parts Conference in February 2008 by proposal of a specific CADC Work Group (WG on flow regime, droughts and emergency situations) created in 2006, also established minimum trimester flow values and (in this case only for Douro and Tejo) a weekly flow volume guarantee at the monitoring stations.

Nevertheless, it shall be emphasised that this flow regime may be revised in accordance with the provisions of the RBMP's, still to be approved.

Concerning the coordination of extreme events, relating to floods, as explained in Maia (2008), Portugal and Spain have already an operational and effective bilateral cooperation (more active for the Tejo river basin) that needs to be institutionalized. A Protocol Agreement for bilateral data exchange was approved in April 2002, by then only automatically for Tejo, although still in 2007

coordination procedures for flood events were scheduled to be agreed. Meanwhile, both countries have to comply with the WFD's complementary directive on Floods (2007/70/EC) by implementing a three stage process: (i) carrying out a preliminary assessment (by 2011) to identify zones at risk of flooding and, for such zones; (ii) drawing up flood risk maps (by 2013); and, (iii) establishing flood risk management plans (by 2015).

In what concerns droughts, a lot of work is to be done, namely by Portugal and on bilateral terms, as Spain has already prepared specific plans for drought situations and Portugal is still behind, working independently. That still will now be analysed more in detail.

5. DROUGHT MANAGEMENT AND PLANNING IN PORTUGAL AND SPAIN

The Albufeira Convention (Article 19) defines that both Portugal and Spain shall "coordinate actions to prevent and control drought and water scarcity situations, setting the exceptional mechanisms to mitigate consequent effects and define the nature of exceptionality to the general regime established in the present Convention...". The same document states that both sides should "undertake joint studies of drought and water scarcity situations to define measures to be applied and define the criteria and indicators of the exceptional regime…".

In fact, in 2003, the CADC (by then) specific workgroup on droughts agreed on a two-phase work, aiming at: the establishment of an indicator's system and respective trigger values; the identification of the main uses to be assured under special circumstances,

Nevertheless, concerning droughts, the 2007-2008 plan of action (CADC, 2007) of the (broader) WG on flow regime, droughts and emergency situations still included as goals the: selection of the hydrometereological variables and definition of monitoring points to assess drought situations; definition of indicators and alert trigger values; definition of exceptionality and management on drought situations.

In reality, presently (mid 2009), no common and homogeneous indicator system has yet been created, and that necessity does not seem to be taken into account by the two countries, currently at different stages on addressing drought management and planning issues:

- Portugal:

Drought situations were traditionally managed on a reactive response approach. Drought situations are monitored by the Water Institution (INAG) based on cumulated precipitation values along the hydrological year, with a drought situation confirmation the 8th month (May).

The severe drought period of 2004/05, that was ranked as the worst rainfall affection of the last 60 years (based on PDSI classification in terms of area affected), was a trigger for action on those situations. In fact, a special Drought Commission was created to monitor and implement appropriate mitigation measures for drought effects minimization and it stated the need for new and systematic procedures regarding drought management, namely: the creation of an integrated drought early warning and management system; the establishment of a reliable monitoring system of (surface and ground) water sources availabilities as well as the estimation of the main water uses for different sectors (volumes and monthly distribution); wider promotion of water efficient use.

Although the Portuguese EU Presidency (2nd semester 2007) dedicated a special attention and gave a special priority to water scarcity and droughts, only very recently the work on the framed Drought Alert and Management System (DAMS) could effectively begin. DAMS is to be a drought early warning system enabling (Figure 2):

(i) the continuous evaluation of the set of the chosen indicators, aimed to translate different meteorological, hydrological and socio-economic situations;

(ii) the stipulation of correspondent levels of drought's severity alert (and measures);

(iii) the (model) simulation of possible meteorological and water uses foreseen scenarios and the evaluation of corresponding potential impacts.

That will imply that the chosen indicators are validated through the comparison with real drought impacts.

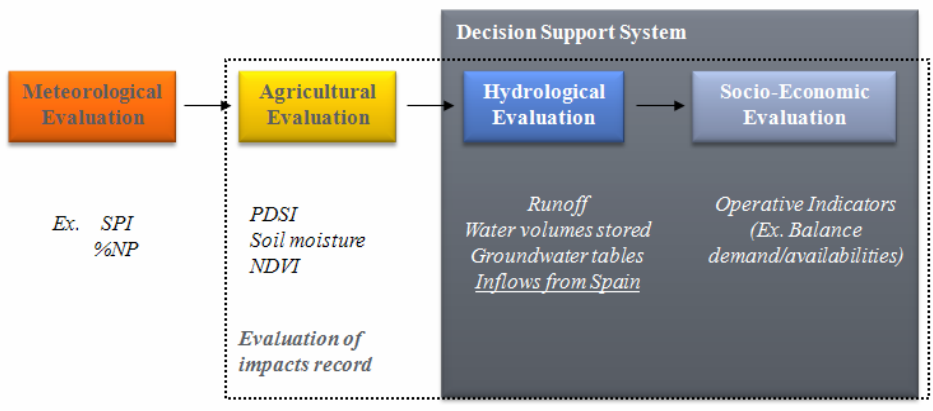


Figure 2. DAMS – Drought Alert and Management System.

The anticipation of the possible drought impacts shall enable to adopt the adequate planned prevention measures. Nevertheless, in Portugal no Drought Management Plans are already envisaged, since most of the effort is currently being concentrated on the (late) RBMP.

- Spain:

The last severe drought situation, in Spain extended from 2004 to 2007, led to an intense activity on drought issues, of which resulted: Special Plans for Drought situations for all the "Demarcaciones Hidrográficas"; a supporting system of indicators; Emergency Plans for settlements with populations bigger than 20000 inhabitants.

The supporting indicators' system is based on operative indicators (hydro-meteorological variables, e,g.: stored volumes on reservoirs; groundwater tables on aquifers; river flow levels; reservoir's discharges; cumulated precipitation in significant meteorological stations; snowpack, where significant), with the drought's severity final evaluation being achieved with a standardization of the different type of variables through a special index (called "Índice de Estado"). The adopted methodology included: (i) identification of existing water sources for representative demand units; (ii) selection of the most suitable indicators to represent water availability evolution for each demand unit; (iii) weighting of indicators to achieve representative numerical results of different drought levels in river basin's water systems; (iv) validation of selected indicators.

Those indicators are supposed to enable detection, with sufficient anticipation, of different drought severity levels in order to adopt adequate mitigation measures and actions, and are the basis for the implementation of the specific Drought Plans at the river basin level.

5.1. Indicator systems and transboundary agreements

It should be noted that, independently of the followed way for the definition of indicators for the Iberian Peninsula, those should be compatible and homogenized between both countries. Also, those indicators should comply with both countries agreements and conventions, and so should be in accordance with those adopted under the Albufeira Convention for the shared river basins.

In fact, the before referred (section 3.) shared rivers' guaranteed flow regimes are depending only (except for Guadiana river where an additional criteria does also apply) on referenced precipitation values. The (annual/trimester/weekly) flow values shall be only guaranteed in all non-exceptional periods, those defined when the averaged cumulated precipitation in referenced precipitation stations on a pre-defined (corresponding) period is exceeded.

Regarding the three major shared river basins, from the analysis of the correspondent Specific Drought Plans it can be signalled that concerning the Albufeira Convention referenced precipitation stations, Spain: has not used those values as operative indicators in Douro and Tejo; has used those values but mixed up with other spanish pluviometer stations, for Guadiana.

Figure 3 allows comparing, according to (CHD, 2007), the differences in the classification of the drought alert levels when only the reference (pluviometric) indicators of the Albufeira Convention

(Figure 3a) or the global operational indicators used in Spain for the Special Plans for Drought situations (Figure 3b) are considered. Having in attention the common temporal period to both series (corresponding to the one of Figure 3b), it is visible that the alert and emergency situations (that will, certainly, incorporate the definition of situations of exception for the application of the flow regime defined in the Albufeira Convention) are more significant (or occur, as in the 2004-2006 period) with the application of the indexes defined by the Convention.

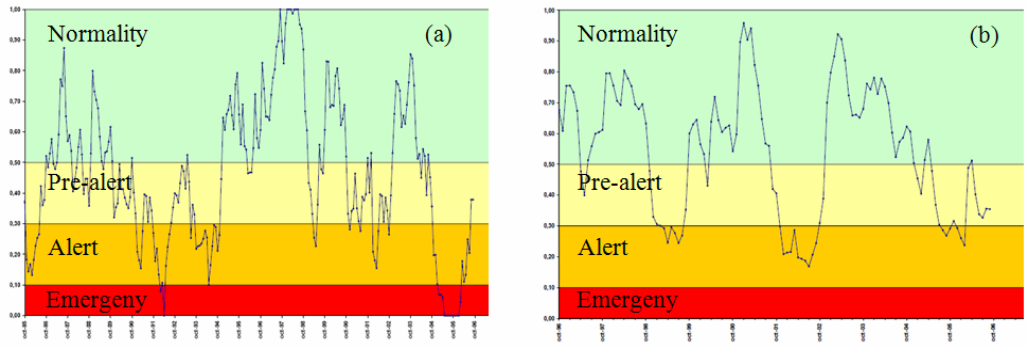


Figure 3. Douro river basin. Temporal evolution of the "Indice de Estado" and alert levels, according to: a) pluviometers considered in the Albufeira Convention; b) global operational indicators of the river basin (adapted from CHD, 2007).

This aspect will have to be surely taken into account in the future, in the context of the established objectives for the CADC in what concerns droughts, and will have considered by the two countries in terms of coordinated management of the corresponding situations.

6. IBERIAN CLIMATE CHANGE FRAME

EU is very much concerned on adapting to climatic change effects, namely on increasing the resilience of biodiversity, ecosystems and water (COM, 2009), for which WFD is relevant. In fact, and already following a previous EU Communication on Climate Change (CC) targets (COM, 2007), it is expected that the first cycle of RBMPs already integrate a first climate check on the corresponding programmes of measures and that the next generation of plans (due in 2015) should fully take CC effects and impacts into account. CC change should also be properly integrated in the implementation of the Floods Directive as well on addressing water scarcity and droughts issues.

At the Iberian Peninsula level, the two countries are at different stages in addressing climate change. In Portugal, a specific national assessment was made under a research Project SIAM – Scenarios, Impacts and Adaptation Measures in Portugal (Santos et al, 2002), ended (2^{nd} phase) 2004. The evaluation of CC effects on water resources mainly consisted on the analysis at regional scale (wider than river basin scale). Average annual and seasonal surface runoff and groundwater water recharge evolution were generated by a rainfall-runoff model and the global inputs of precipitation and temperature series were obtained from the European climate models. A National Strategy for Adaptation to CC is being prepared by the National Commission for CC and is announced to be approved in 2009.

In Spain, following the goals established by a National Adaptation Plan to CC, a first work programme on Climate Change Impacts evaluation over water resources, biodiversity and coastal areas affection followed, the first results made public (March 2008) do refer and/or show (OECC, 2008): achievement of a complete database on climate regional scenarios for this century; ongoing work on the adaptation of these scenarios for simulation; use of numerical models for natural water resources, changes on demands (agriculture and domestic supply) and affections of water storages on exploitation systems; changes on past natural water resources regime (runoff and groundwater recharge) for the entire country being evaluated (for the establishment of reference conditions); improvement on the simulation of future development scenarios for quantification of water demands for agriculture.

7. CONCLUSIONS

Portugal and Spain have carried out a big internal effort, namely in institutional terms, to adequate their water policy to WFD aimed goals and scheduled plan of actions. Nevertheless, a need for a more dynamic and coordinated implementation process is still needed.

Both countries are in delay concerning the EU scheduled WFD implementation plan, and the RBMP's end of 2009 deadline is definitively compromised for Portugal and seems at risk for Spain.

The two countries have chosen (at least for the first cycle of WFD implementation) a basin-based water management and a decision-making process framed on coordinated River (sub-) Basin Management Plans, one per country, in compliance with the Albufeira Convention provisions. Both countries decided that the Commission for Convention Appliance and Development (CADC) shall be not only the driving force for the Convention appliance but also for a common and co-operative WFD Iberian implementation process. Nevertheless, the effective role and authority of the CADC on the envisaged real coordinated and integrated water resources management of the Portuguese-Spanish river basins has to be better framed (hopefully as the Competent Coordination Water Authority).

A great effort is needed by Portugal, in internal terms, and by the two countries, at Iberian level, on drought monitoring, planning and management issues, being clear that the two countries are currently at different stages on addressing those. A common and/or homogenized set of indicators and drought alert levels should be agreed between the two countries, and those should be in accordance with the Albufeira Convention provisions and the established minimum flow regime monitoring indicators.

Those actions shall be complemented by an internal but also bilaterally coordinated strategy and effort in terms of climate change analysis, scenarios and actions concerning water resources and related ecosystems.

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FLOODS AND DROUGHTS MODELING UNDER CLIMATE CHANGE SCENARIOS USING NEURAL NETWORKS

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Abstract: The purpose of this study is to investigate the impact of climate change on floods and droughts in New- Brunswick, Canada, using downscaled climate models and neural networks. Future climate data were extracted from the Canadian Coupled General Circulation Models (CGCM2) and the Hadley Circulation Model (HadCM3) with a multilayered feed-forward neural network models to predict the local variability in river discharge and extreme events. With projected high and low flow frequency calculations, it will be possible to compare current design criteria with future scenarios that consider climate change. The simulations were computed using the generalized extreme value (GEV) distribution function, and the parameters of the distribution were estimated using L-moments method.

Key words: Climate Change, Droughts, Floods, GEV, Neural Networks.

1. INTRODUCTION

There is currently a broad scientific consensus that the global climate is changing in ways that are likely to have a profound impact on human society and the natural environment over the coming decades. Climate change and its impacts on a global scale are the focus of an intense, broad-based international research effort in the natural and social sciences. However, understanding the nature and potential consequences of climate change at regional scales remains a challenge. Moreover, it has been recognized that changes in the frequency and magnitude of extreme weather events are likely to have more substantial and widespread impacts on the environment and human activities than changes in the average climate.

A number of extreme events (with significant impacts on the environment and socio-economic activities) have been observed during the last decade, including heavy floods, severe droughts and extreme heat around the world. These have caused serious risks to ecosystems and have had severe economic consequences on sectors such as agriculture and water resources. Anticipating specific climatic impacts, is thus, as much a challenge of assessing risks and uncertainties associated with plausible scenario of climate extremes and variability as it is in predicting changes in the average climate. As such, it is important 1) to improve our ability to manage extreme climatic risks, 2) to assess the consequences of such extreme events in the coming decades, and 3) to develop new tools and design criteria to more accurately assess the impact of such extremes on water resources. Improved knowledge and assessment tools would assist policy-makers in formulating more robust policies to mitigate the impacts of climate change and to develop adaptation strategies.

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2. BACKGROUND AND DATA GATHERING

Over the next 100 years, mean surface air temperature is expected to increase by 2 to 6 °C in Atlantic Canada (Parks Canada 1999), contributing to potentially large reductions in stream flow (Hengeveld 1990; Natural Resources Canada 2002) and significant impacts on aquatic resources (Minns et al. 1995). The demand for water withdrawal from rivers (e.g. irrigation, drinking water, etc.) and maintaining adequate instream flow for the protection of aquatic resources are tenuously balanced, and represent an ongoing challenge in water resources management. Many rivers in eastern Canada have recently experienced record low flow conditions, coupled with record high water temperatures (Caissie 1999a,b; Caissie 2000), and an increase in water withdrawal. New Brunswick lies on Canada's Atlantic coast, and is bordered by ocean on its southern (Bay of Fundy), northern and eastern (Gulf of St. Lawrence) shores. Generally, average temperatures in New Brunswick range from -10 °C in January to 19 °C in July. New Brunswick receives approximately 1100 mm of precipitation annually, with 20 to 33% falling as snow. Precipitation tends to be highest in southern parts of the province.

Major rivers and many smaller streams flow from the interior highlands of New Brunswick. Rainfall, snowmelt, and groundwater all contribute to the volume of flow, producing variations from season to season and year to year. Most high flows and floods are caused by spring snowmelt. Heavy rainfall can also cause high flows and floods, especially on small streams. Low flows generally occur in late summer, when precipitation is low and evaporation is high, and in late winter, when precipitation is stored until spring in the form of ice and snow.

Daily maximum and minimum air temperature and total precipitation data from 7 meteorological stations in New Brunswick were obtained from Environment Canada (Figure 1 and Table1). Air temperature data from the National Climate Data Archive was "homogenized" at 6 of the 7 stations to remove any non-climatic inconsistencies due to station alterations including changes in site exposure, location, instrumentation, observer, observer program, or a combination of the above. Daily discharge (m^3/s) data from 7 hydrometric stations in New Brunswick (Figure 1 and Table 2) were obtained from Environment Canada's National Water Data Archive (HYDAT). Discharge series included data from 1919 to 2005, with continuous series extending 43 to 79 years at some stations.

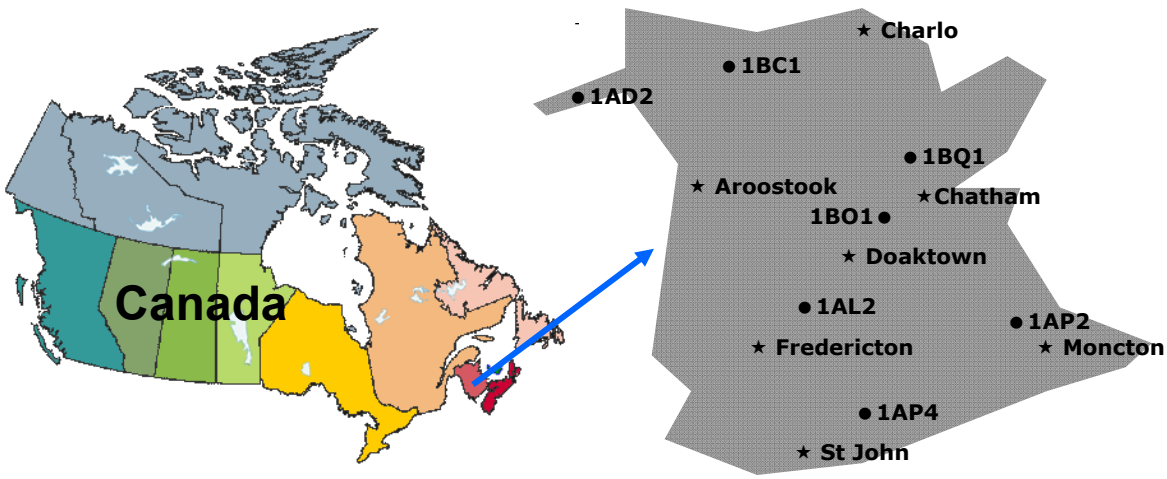


Figure 1. Location of meteorological and hydrometric stations in New Brunswick, Canada (● Hydrometric station ★ Metrological station)

Table 1. Location and length of air temperature and precipitation time series at meteorological stations in New Brunswick.

Meteorological Station	Latitude, Longitude	Air Temperature Series	Precipitation Series
Aroostook	46º 48' N; 67º 43' W	1913-1999 (87 yrs.)	1929-2005 (77 yrs.)
Charlo Airport	47º 59' N; 66º 20' W	1945-1999 (55 yrs.)	1966-2005 (40 yrs.)
Chatham Airport	47º 01' N; 65º 27' W	1895-1999 (105 yrs.)	1943-2005 (63 yrs.)
Doaktown[1]	46º 33' N; 66º 09' W	1952-1999 (48 yrs.)	1934-2005 (72 yrs.)
Fredericton Airport	45º 52' N; 66º 32' W	1895-1999 (105 yrs.)	1951-2005 (55 yrs.)
Moncton Airport	46º 06' N; 64º 47' W	1895-1999 (105 yrs.)	1939-2005 (67 yrs.)
Saint John Airport	45º 19' N; 65º 53' W	1895-1999 (105 yrs.)	1946-2005 (60 yrs.)

Table 2. Location, drainage area (km^2), and period of record at selected hydrometric stations in New Brunswick.

Hydrometric Station	Latitude, Longitude	Drainage Area (km^2)	Period of Record
Saint John R. at Fort Kent	47° 15' N, 68° 36' W	14,700	1927-05 (79 yrs.)
Nashwaak R. at Durham Bridge	46° 08' N, 66° 37' W	1,450	1962-05 (43 yrs.)
Canaan R. at East Canaan	46° 04' N, 65° 22' W	668	1926-40, 1963-05 (58 yrs.)
Kennebecasis R. at Apohaqui	45° 42' N, 65° 36' W	1,100	1962-05 (43 yrs.)
Restigouche R. below Kedgwick R.	47° 40' N, 67° 29' W	3,160	1963-05 (42 yrs.)
SW Miramichi R. at Blackville	46° 44' N, 65° 50' W	5,050	1919-32, 1962-05 (57 yrs.)
NW Miramichi R. at Trout Bk.	47° 06' N, 65° 50' W	948	1962-05 (43 yrs.)

3. METHODOLOGY

3.1 Climate data downscaling

According to the CGCM grid, western and central regions of New Brunswick fall within grid box 30X12Y, while the eastern coast falls within grid box 31X12Y. Observed NCEP (National Centers of Environmental Prediction) grid-scale predictors were interpolated to the 30X12Y and 31X12Y CGCM grid and made available by the Canadian Institute for Climate Studies (CICS) project. Local scale predictand (maximum and minimum air temperature, precipitation, and discharge) data from 1961-2100 were obtained from Environment Canada. A statistical model was used to downscale climate data. Statistical downscaling model were calibrated using daily series of maximum temperature (T_{MAX}), minimum temperature (T_{MIN}), wet-day amounts of precipitation (P). Statistical "downscaling" involves the development of significant relationships between local and large-scale climate based on coupled atmosphere ocean general circulation model (AOGCM) output. Statistical downscaling, a transfer function approach, assumes that regional climate can be determined by the large-scale climatic state and regional / local physiographic features (e.g., topography, land-sea distribution and land use) (von Storch 1995, 1999).

Statistical downscaling models were developed from daily series of maximum (T_{MAX}, equation (1)) and minimum temperature (T_{MIN}, equation (2)):

$$T_{MAX_i} = \alpha_0 + \alpha_T T_i + \alpha_{T_{i-1}} T_{i-1} + \sum_{j=3} (\alpha_X X_i)_j \quad (1)$$

$$T_{MIN_i} = \delta_0 + \delta_T T_i + \delta_{T_{i-1}} T_{i-1} + \sum_{j=3} (\delta_X X_i)_j \quad (2)$$

where T_i = mean air temperature at 2-m;
X_i = other meteorological variables selected on a per site basis
α, δ = regression constant and coefficients

and wet-day amounts of precipitation (P, equation (3)):

$$P_i = \sqrt{\mu_0 + \mu_{q_{500}} q_{500_i} + \mu_{U_s} U_s + \sum_{j=3} (\mu_X X_i)_j} \quad (3)$$

where q_{500} = specific humidity at 500 hPa;
U_s = surface zonal velocity;
X = other meteorological variables selected on a per site basis
μ = regression constant and coefficients

Downscaling models were calibrated using observed predictor variables and the observed predictand and validated with data withheld from the calibration process. Site-specific meteorological parameters were provided by Environment Canada across New Brunswick, using statistical downscaling model (SDSM) (Lines et al., 2005). Observed data for daily maximum and minimum air temperatures, and total daily precipitation were used as meteorological variables to construct suites of downscaled climate variable projections. The output from two models was obtained, namely the second generation Coupled General Circulation Model (CGCM2) and the third Hadley Centre Coupled Atmosphere-Ocean General Circulation model (HadCM3) in conjunction with the greenhouse gas + aerosol emission experiment. Results from these models will be used to provide a range of values or future discharge using Neural Network numerical models.

3.2 Neural Network Model

Future river discharge data were generated using Artificial Neural Networks (ANN). The type of ANNs used in the present study are multilayered feed-forward networks. It consists of an input layer, a hidden layer and an output layer. The learning is supervised and the training is accomplished using the Levenberg-Marquardt algorithm. The three types of architectures used are depicted in Table 3.

Table 3 – Neural network architectures

Type	Network
A	{M, T_{MIN}, T_{MAX}, Ptot} → {Qa}
B1	{M, T_{MIN}, T_{MAX}, Ptot, Qa} → {Q_{MAX}}
B2	{M, T_{MIN}, T_{MAX}, Ptot, Qa} → {Q_{MIN}}
M (1,2,..,12), T_{MIN},T_{MAX} (°C), Ptot (mm), Qa, Q_{MAX}, Q_{MIN} (m^3/s)	

It was observed that the extreme discharge events were better predicted if the average discharge was used as an input in the model. Therefore, the simulated average discharge, Qa, from the network A was used in networks B1 and B2 for subsequent predictions of high flows (Q_{MAX}) and low flows (Q_{MIN}).

As an example, future river discharge for the Fredericton site (New Brunswick) will be used in the characterization of low flow and flood events. The characteristics of these extreme events will be used to calculate the discharge of different recurrence intervals. For this location, the performance of the networks showed a R^2 of 0.80 (A) and 0.92 (for B1 and B2) for the different network architectures presented in Table 3. Also, Figure 2 show how well the ANN model simulated monthly Q_{MAX} and Q_{MIN} between 1985 and 1990 compared to observed discharge values.

A visual inspection confirms that the simulated discharges coincided fairly well with the observed discharges.

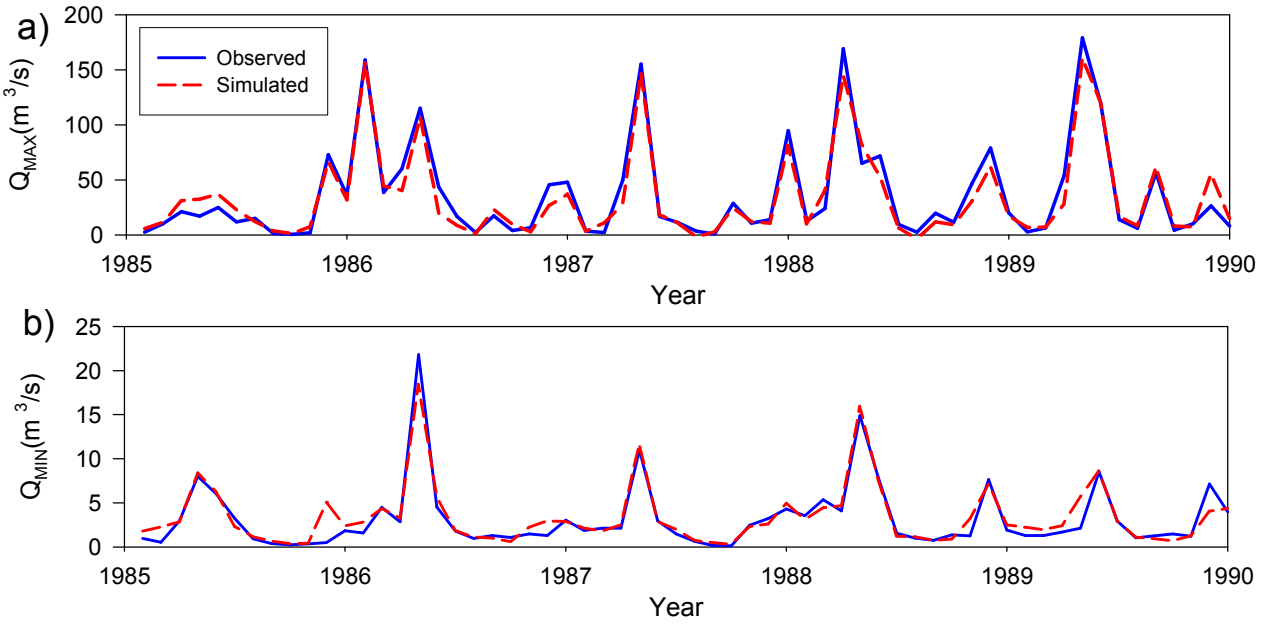


Figure2. Observed and simulated a) maximum monthly discharge b) minimum monthly discharge (using historical data)

3.3 Characterization of extreme events (high and low flows)

Stream flow data constitute the basis of information used by hydrologists to make predictions of floods and low flow events and their corresponding frequencies. The probabilistic approach is useful in the analysis due to the random nature of such events and the flexibility of this approach in characterizing extreme events. Such analysis of hydrologic data, also called a frequency analysis, can be carried out, among others, following the annual series approach or the partial duration series approach. The annual series approach is widely used in flood and low flow frequency analyses, and it will be used in this study.

3.4 Frequency of extreme events under climate change

Once that extreme event characteristics were determined (floods and low flows), a study of changes in intensity over time were carried out. For instance, the associated frequency of extreme events is very important in infrastructure design and these frequencies will most likely change under a future climate. The frequency (or return period) of high and low flow events will be calculated for the 2010-2100 period. With projected high or low flow frequency calculations, it will be possible to compare current design criteria with future scenarios that consider climate change. For instance, a 50-year high flow event today could potentially become a much more dominant high flow event in the future and thus represent a high flow event with a much lower recurrence interval. A similar situation with more severe summer droughts could also change current event frequencies. Such changes in high and/or low flow frequencies will be evaluated under climate change. The frequency of future events will ultimately impact on infrastructure design and development, flood and drought management, hydraulic and municipal water supply management as well as current statistics used by the insurance industry.

4. CASE STUDY

Future river discharge for the Fredericton site (New Brunswick) will be used in the characterization of low flow and flood events. The characteristics of these extreme events will be determined for different recurrence intervals. The annual maximum air temperature at the Fredericton site shows a significant increase for the period 2000-2100 compared to current climate conditions (1960-2000), by approximately 4 ºC. As for precipitation, the mean annual precipitation is projected to increase from approximately 1230 mm to 1670 mm within the next hundred years.

The frequency of high and low flow events was calculated based on present day data and on future climate variation. With projected high or low flow frequency calculations, it was possible to compare current design criteria with future scenarios under climate change. Therefore, the return period analysis is shown in Figure 3 for the maximum and minimum discharge. The discharge event of different recurrence intervals were computed using the generalized extreme value (GEV) distribution (F(x)). The *T* year return period for high flow is estimated by:

$$T = \frac{1}{1 - F(x)} \tag{4}$$

and the low flow by:

$$T = \frac{1}{F(x)} \tag{5}$$

The parameters were estimated by the method of L-moments and they are given in Table 4.

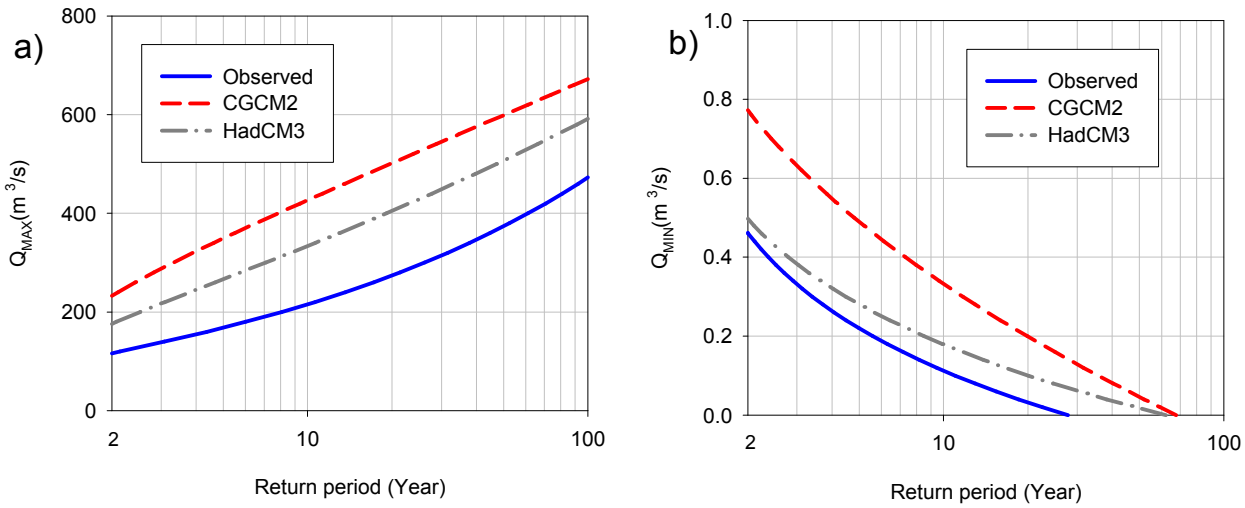


Figure 3. Return periods of observed and simulated a) maximum discharge b) minimum discharge

Table 4 Parameters of GEV used in return values

	Q_{MAX}			Q_{MIN}		
	Observed	CGCM2	HadCM3	Observed	CGCM2	HadCM3
k	0.34135	0.00632	0.12714	-0.0691	-0.46113	-0.16208
σ	33.092	102.12	70.705	0.28641	0.32474	0.25498
μ	103.51	195.27	149.49	0.35818	0.66265	0.40667

k, shape parameter σ, scale parameter μ, location parameter

For example, results showed that a 50-year high flow event could potentially become a more dominant high flow event in the future (and thus representing a flow with a much lower recurrence interval in the future). Figure 3a shown such results where a 50-year event could potentially be closer to a 20 or 10-year event in the future depending on the climate model used. The low flow analysis also shows less severe low flow, especially with the CGCM2 model under climate change (Fig. 3b). In contrast, the HadCM3 showed results that were closer to current climate low flows. The analysis show that the intensity and frequency of discharges will most likely increase in severity according both climate models (the CGCM2 showing the strongest flood increase); however, results of low flows that future climate may bring more water during periods of low flow particularly when using the CGCM2 model.

5. CONCLUSIONS

Climate change is expected to alter global temperature and precipitation patterns, exerting significant pressures on water resources. However, specific regional projections about the impact of climate change are hampered by the limited spatial resolution of global circulation models, making it difficult to determine the degree of climate change, how fast it will happen, and where it will occur. Alternatively, statistical downscaling generates local climate change projections, providing future climate scenarios on which adaptation strategies can be developed. In the 20th century, changes in climate, particularly increases in temperature, have already affected physical and biological systems in many parts of the world (McCarthy et al. 2001). In New Brunswick, climate change will undoubtedly alter the quantity and quality of water resources. Success of such industries as agriculture, forestry, and fisheries are intrinsically linked to climate, making New Brunswick particularly vulnerable to the impacts of climate change. Undoubtedly, these industries (among others) will be significantly affected by a warmer, wetter climate (in some seasons and locations) with increased water during some time of year and diminished during other times. Adaptation will be essential to maintaining their viability.

A warmer climate in New Brunswick would result in significant changes in water availability at different times within the year. A warmer climate will also contribute to warmer water temperatures in rivers, lakes, and groundwater aquifers. Warmer water temperatures may result in changes in the abundance, diversity, and distribution of aquatic species inhabiting New Brunswick streams and rivers.

Increases in annual and seasonal precipitation may increase the magnitude and frequency of flooding, particularly if the frequency of extreme precipitation events also increases, as predicted by the IPCC (Houghton et al. 2001). More frequent and intense floods can increase infrastructure damage, cause soil erosion, crop damage and others. However, increased precipitation patterns may enhance water resources (groundwater and stream flow) especially in spring and summer time.

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ASSESSING WATER SHORTAGE RISK DUE TO DROUGHT IN COMPLEX WATER SYSTEMS UNDER CLIMATE CHANGE SCENARIOS

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Abstract: Impacts of climate-induced hydrological changes are expected to exacerbate the pressure on complex water supply systems located in drought-prone areas. A strategic approach to water resources management should take into account these impacts also by including the assessment of the risk of water shortages due to drought. The paper presents a pilot implementation of the shortage risk assessment method on which is based a Strategic Water Shortage Preparedness Plan for a complex water system supplying several competing uses taking into account either current conditions or hydrological scenarios consequent to potential changes in climate. The water system chosen as case study is a typical Mediterranean complex water supply system that strongly relies on surface water regulated through reservoirs. The water supply system has been simulated considering future configuration of hydraulic infrastructures, reduced water availability at the river sections due to the impacts of climate change and measures to mitigate drought impacts. The considered scenario has been developed taking into account the assessment carried out by the three Working Groups (WGs) of the Intergovernmental Panel on Climate Change (IPCC) and the IPCC's Fourth Assessment Report (AR4) specifically tailored for considering local trends on observed data in temperatures and precipitations. A general decrease on precipitation and increase on temperatures can be inferred from the observed 42 years time series with greater precipitation reduction for the JFM quarter. Precipitation reduction causes reduction in streamflows close to 30% in some extreme cases. The risk of water shortage due to drought has been analyzed both through traditional performance indices and the probability of shortages belonging to different classes expressed as a percentage of the monthly demand. This kind of representation of results gives clear and not-ambiguous information to water managers on the impacts of the specific climate change scenario and the effectiveness of the chosen mitigation measures. Analysis of results has been carried out also with reference to reduced demands obtained by fixing thresholds considered acceptable for the different uses.

Key words: Climate change scenarios, Drought impacts, Water supply system management.

1. INTRODUCTION

The pressure on water supply systems located in drought-prone areas will likely increase due to the impacts of climate-induced changes on hydrometeorological variables. A shift from the traditional reactive approach to a proactive approach to drought becomes even more necessary within a climate change scenario due to the increased uncertainty. Tasks such as planning, monitoring, implementation of planned measures, management of emergency situations and recovery of drought damages have to be designed and analyzed taking into account a change on precipitation, streamflows and temperatures regimes. Potential climate change scenarios have to be included within the planning framework against drought at the strategic level with indirect impacts on tactical and emergency levels.

A comprehensive planning framework for coping with drought includes three different plans, namely Strategic Water Shortage Preparedness Plan, Water Supply System Management Plan and Drought Contingency Plan (Rossi et al., 2008). Such plans depend on the level of planning (Strategic, Tactical or Emergency) and involve several different institutions as depicted in figure 1 where the links between the three plans and with the drought monitoring system are represented.

The SWSPP can be considered as a "Drought management (sub) plan" to be included within the River Basin Management Plan as provided by the article 13.5 of the EU Water Framework Directive (WFD/2000/60). It fits well also with the water resources planning tools which have been proposed in the legislation of several European countries as well as of other countries out of Europe that include specific measures for drought preparedness.

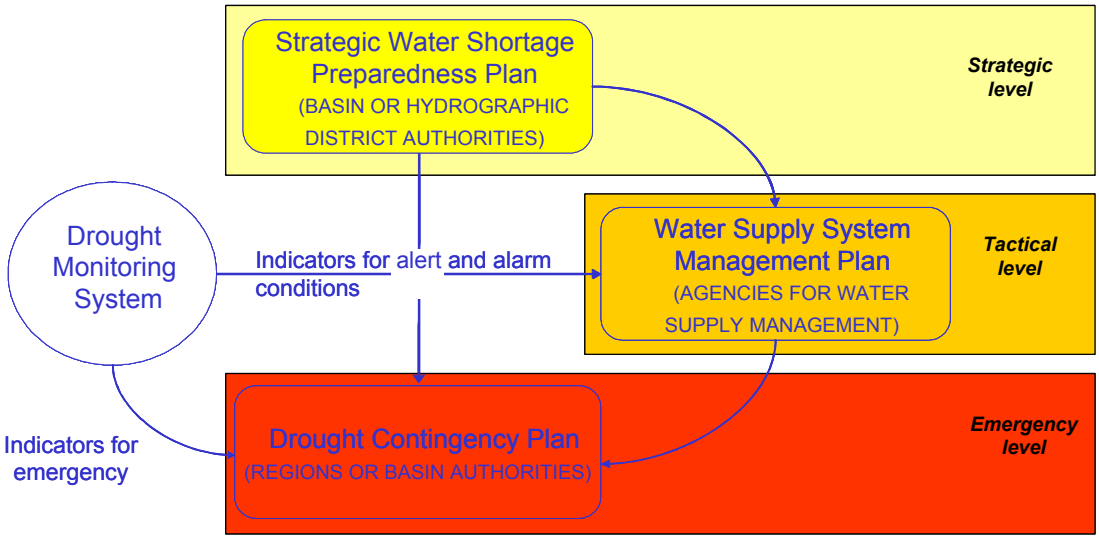


Figure 1. Planning levels for drought management and mitigation and role of drought monitoring system

Due to its strategic nature and its long time horizon of implementation, the SWSPP is where climate change scenarios should be considered in order to evaluate if its general objective of reducing the vulnerability of the water supply system to drought impacts, can be achieved also in presence of modified amounts and patterns of available water resources and demands.

A pilot SWSPP developed for a complex water supply system strongly relying on surface water is presented. In the paper, an analysis of the impacts of increased temperatures and reduced water resources defined on the basis of the 4th IPCC report is carried out in order to assess the effectiveness of the proposed mitigation measures within a climate change scenario.

2. THE AGRI-SINNI WATER SUPPLY SYSTEM

The Agri-Sinni water supply system is a complex multisource-multiuse water systems that supplies municipal, irrigation and industrial uses located in two regions of Southern Italy namely Basilicata, Apulia and marginally also in Calabria. More specifically the major part of the elements constituting the system are located in Basilicata a region less arid and less drought prone than the other two regions. For this reason, especially due to the grow of apulian municipal demands, part of the resources stored in the two principal reservoirs of the Agri-Sinni water system (Monte Cotugno and Pertusillo reservoirs) firstly devoted to irrigation have been re-allocated to municipal needs. Nowadays reservoirs and diversions built in the Basilicata region supply municipal and agricultural users of the same Basilicata region, mostly several users of the Apulia region and users located in Calabria region for a small amount.

Due to the complexity of the water supply system and its multipurpose features the interests of several stakeholders have to be taken into account during the decision process. For this reason, management of the water supply system, already very complex in normal hydrologic conditions, due to the several involved constraints, becomes particularly difficult during drought events.

To this purpose an agreement has been signed by regional governments and national representative which provide a "Coordination Committee" including stakeholders, in charge of the allocation policy of available resources among regions and users.

The system is mainly composed by four reservoirs and four diversion dams, but while some plants are fully in operation (Monte Cotugno and Pertusillo reservoirs, Agri, Sauro and Gannano diversions), others such as Marsico Nuovo reservoir and Sarmento diversion dam are partially in use in the present configuration of the system but it's likely that they will be fully in operation within a short time horizon in the future (Vita et al., 2007). For this reason, in order to take into account the possible impacts of climate changes, the system has been considered in the future configuration of plants including both Marsico Nuovo reservoir on Agri river and releases devoted to irrigation districts in Basilicata and Sarmento diversion dam diverting flows from Sarmento stream to Monte Cotugno reservoir (Figure 2).

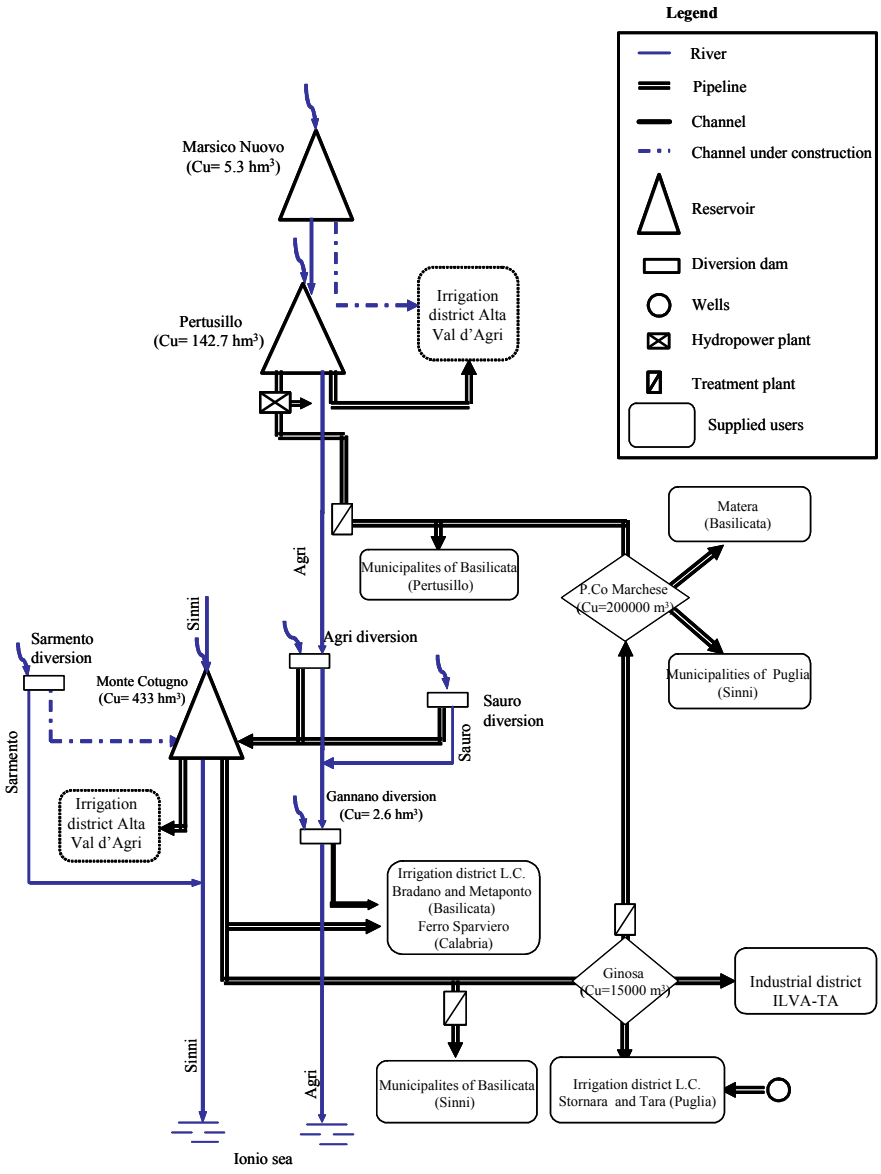


Figure 2 Scheme of the Agri-Sinni water supply system

As water demands to be supplied by the system in the present study have been considered the water requirements adopted by the "Coordination Committee" for 2007 except for the municipal demand which has been increased of 20% in order to take into account of its likely growth in the next years especially due to the increasing pressure arising from Apulia region as consequence of a reduced withdrawal from aquifers fixed by the plan for water management and quality improvement in Apulia. Ecological constraints have been considered for the simulation of the system as monthly volumes to be released as in-stream flows below systems's plants, calculated as a function of a base contribution plus a variable contribution depending on the mean flow at different months of the

year. As not-structural mitigation measure against drought impacts an assignment of monthly volumes to be stored in reservoirs in order to guarantee municipal demand satisfaction taking advantage from an interannual management of the Monte Cotugno reservoir has been fixed according to (1).

$$V_i = \sum_{k=i+1}^{N} 0.8 \cdot D_{municipal,k} \quad (1)$$

with $i = 3$ ® 9 and $N = 10$ *for Pertusillo reservoir*
with $i = 3$ ® 21 and $N = 22$ *for Monte Cotugno reservoir*

3. ADOPTED CLIMATE CHANGE SCENARIO FOR THE AREA OF THE AGRI-SINNI WATER SUPPLY SYSTEM

As a result of the assessment carried out by the three Working Groups (WGs) of the Intergovernmental Panel on Climate Change (IPCC) a Synthesis Report was edited and adopted at IPCC Plenary XXVII held at Valencia, Spain, on 12-17 November 2007. The report represents the formally agreed statement of the IPCC concerning key findings and uncertainties contained in the Working Group contributions to the Fourth Assessment Report.

It provides a synthetic and integrated view of climate change as the final part of the IPCC's Fourth Assessment Report (AR4) and has been considered as the principal guideline to elaborate a likely climate change scenario at the local scale of the Agri-Sinni water supply system.

The definition of climate change in IPCC usage refers to: "a change in the state of the climate that can be identified (e.g. using statistical tests) by changes in the mean and/or the variability of its properties, and that persists for an extended period, typically decades or longer. It refers to any change in climate over time, whether due to natural variability or as a result of human activity", (Climate Change 2007: Synthesis Report) considering climate change both natural and directly or indirectly to human induced changes on climate. As highlighted by the synthesis report, among uncertainties related to the actual modelling of the complex involved natural processes and the effects of the human induced activities, what is almost certain and commonly accepted is that: "warming of the climate system is unequivocal, as is now evident from observations of increases in global average air and ocean temperatures, widespread melting of snow and ice and rising global average sea level" (Climate Change 2007: Synthesis Report).

The report considers four families of socio-economic scenarios (A1, A2, B1 and B2) that represent different world futures in two distinct dimensions: a focus on economic versus environmental concerns, and global versus regional development patterns.

The A1 storyline assumes a world characterized by very rapid economic growth, a global population that peaks in mid-century and rapid introduction of new and more efficient technologies.

A1 is divided into three groups that describe alternative directions of technological change:

- fossil intensive (A1FI)
- non-fossil energy resources (A1T)
- balance across all sources (A1B)

B1 describes a convergent world, with the same global population as A1, but with more rapid changes in economic structures toward a service and information economy.

B2 describes a world with intermediate population and economic growth, emphasizing local solutions to economic, social, and environmental sustainability.

A2 describes a very heterogeneous world with high population growth, slow economic development and slow technological change.

Unfortunately no likelihood has been attached to any of the scenarios but the present work considers as guideline the A1B scenario with particular reference to the South Europe and Mediterranean area (SEM) as described in ch.11 related to the regional climate projections of Christensen et al. (2007). Several sub-continental regions have been identified for which area-averaged temperature and precipitation changes are presented from the coordinated set of climate model simulations archived at the Program for Climate Model Diagnosis and Intercomparison (called the multi-model data set or MMD).

For the sub-continental area SEM annual mean temperatures are likely to increase more than the global mean. Seasonally, the largest warming is likely to be in the Mediterranean area in summer and maximum summer temperatures are likely to increase more than the average in southern and central Europe. Annual precipitation is very likely to decrease in most of the Mediterranean area with a reduction of annual number of precipitation days. Risk of summer drought is likely to increase in central Europe and in the Mediterranean area. Synthetic results for SEM area and A1B scenario are reported in Table I.

On the basis of results obtained through the 21 different models included in the multi-model dataset (MMD) applied for the analysis of climate change at regional scale, the 4th IPCC report (2007) considers for Europe and the Mediterranean area (NEU):

- an increase of the mean annual temperature between 2.2 °C and 5.1 °C (MMD A1B scenario) for 2080 to 2099 with respect to the 1980 – 1999 period with highest variability for JJA quarter (observed temperature for Italy during the last 200 years show an increase of the mean annual temperature of about +1.7 °C);
- a decrease of the mean annual precipitation between -27% to -4% (MMD A1B scenario) with highest variability for JJA quarter (observed precipitation for Italy during the last 100 years show a decrease of about -5% especially during summer season).

Table I South Europe and Mediterranean (SEM) averages of temperature and precipitation projections from a set of 21 global models in the MMD for the A1B scenario. (Christensen et al., 2007)

		Temperature Response (°C)						Precipitation Response (%)						Extreme Seasons (%)		
Region[a]	Season	Min	25	50	75	Max	Tyrs	Min	25	50	75	Max	Tyrs	Warm	Wet	Dry
SEM	DJF	1.7	2.5	2.6	3.3	4.6	25	-16	-10	-6	-1	6	>100	93	3	12
	MAM	2.0	3.0	3.2	3.5	4.5	20	-24	-17	-16	-8	-2	60	98	1	31
30N,10W	JJA	2.7	3.7	4.1	5.0	6.5	15	-53	-35	-24	-14	-3	55	100	1	42
to	SON	2.3	2.8	3.3	4.0	5.2	15	-29	-15	-12	-9	-2	90	100	1	21
48N,40E	Annual	2.2	3.0	3.5	4.0	5.1	15	-27	-16	-12	-9	-4	45	100	0	46

General results included in the 4th IPCC report for the Mediterranean area have been specifically developed for the Agri-Sinni basins.

In particular on the basis of the 4th IPCC report results and observed data of precipitations and temperatures during the last 42 years by the hydrometeorological stations of the Agri-Sinni basins the following hypothesis have been adopted:

- Increase of temperature equal to +1.8 °C
- Decrease of mean annual precipitation equal to 10.0% (defined on the basis of the 4th IPCC report results) of the historic mean
- Quarterly distribution in terms of percent reduction: 19.65% for JFM; 6.25% for AMJ; 2.93% for JAS and 6.37% for OND quarters defined on the basis of trends detected on the historic time series (Table II).

Table II Quarterly historic trends on precipitation in the eight hydrometeorological stations of the Agri-Sinni basins

	OND	*JFM*	*AMJ*	*JAS*
Sinni at Cogliandrino	-0.2392	-0.3907	-0.0859	-0.0476
Sinni at Monte Cotugno	-0.1163	-0.2787	-0.0086	-0.0057
Sarmento at Monte Cotugno diversion	-0.1788	-0.5105	-0.0996	-0.0533
Agri at Marsico Nuovo	-0.0471	0.0137	-0.0455	0.0086
Agri at Monte Cotugno diversion	-0.0562	-0.2022	-0.0398	0.0056
Agri at Pertusillo	-0.0406	-0.0663	0.0186	-0.049
Agri at Gannano	-0.026	-0.2054	-0.0154	-0.0032
Sauro at Monte Cotugno diversion	0.0202	-0.0711	-0.0276	-0.011
Average	-0.0855	-0.2139	-0.0380	-0.0139

In particular, considering a fixed reduction at annual scale (10.0 %), the following expression has been used to evaluate the quarterly precipitation reduction coefficient:

$$CR_T = 1 - \left|\frac{\Delta h_T}{h_T}\right| = 1 - \left|\frac{\Delta h_A \dfrac{b_{T,st}}{\sum b_{T,st}}}{h_T}\right| \quad (2)$$

where, $b_{T,st}$ are the observed precipitation quarterly trends, Δh_A is the total annual reduction and h_T are the quarterly mean reported in Table III for the basins under study where can be noticed that quarterly historic trends are all negative with the greatest value for the JFM quarter.

Table III Quarterly precipitation reduction coefficients

Δt = 42 years	Quarter			
Δh_A [mm] = -99.14	*OND*	*JFM*	*AMJ*	*JAS*
$b_{T,st}$ (trend) [mm/year]	-0.0855	-0.2139	-0.0380	-0.0139
h_T [mm]	378.48	307.25	171.52	134.14
CR_T	0.936	**0.804**	0.938	0.971

Series of reduced precipitation and of increased temperature evaluated taking into account potential climate changes have been used to apply a simple rainfall-runoff model based on linear multiple regressions in order to obtain an estimation of the streamflows at the eight river sections of interest and evaluate actual impacts of climate changes on water resources availability. In particular the rainfall-runoff model has been developed on the basis of the two series of available records at the two reservoirs Pertusillo and M. Cotugno.

Analysis of the comparison between streamflows available at river sections in correspondence of the plants of the Agri-Sinni water supply system for historical series and taking into account for climate change shows that, on an annual average, a streamflow reduction of 24% can be expected for the eight river sections with a peak on Agri river at Gannano greater than 30% and a minimum of 17% for Sinni at Cogliandrino as can also be inferred by Figure 3. These reductions in streamflows lead in turn to reductions in volumes to be allocated as instream flow requirements.

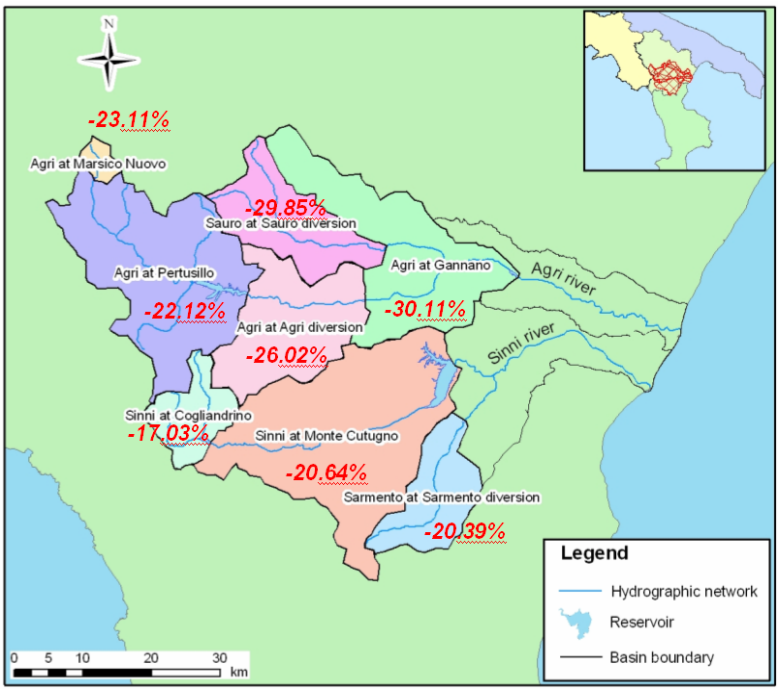


Figure 3 Spatial distribution of streamflow reduction as a consequence of the simulated climate change scenario

4. ASSESSMENT OF THE VULNERABILITY TO DROUGHT OF THE AGRI-SINNI WATER SUPPLY SYSTEM IN ITS FUTURE CONFIGURATION

The risk of water shortages for the Agri-sinni water supply system under the adopted climate change scenario has been assessed through the simulation of the system considering the previously described reduced available water resources at the river sections and increased temperatures. The raising on mean temperatures increases losses from reservoir due to evaporation especially when operating rules focused on the exploitation of the carry-over storage capacity of the reservoir are operated. This kind of operating rule focused on target storages to be stored in Monte Cotugno reservoir in order to be used for municipal supply that has the highest priority has been applied as short term measure to mitigate likely impacts of drought but at the same time could be not very efficient if stored volumes are lost due to the increase in evaporation.

Table IV shows performance indices of the water supply system calculated for municipal, irrigation and industrial uses in normal and under climate change conditions.

It should be noticed that even for municipal demand, that has the highest priority, water shortages appear more frequently with at least two more events during the 42 years of simulation. Irrigation use suffers big impacts as can be inferred and industrial use shows a longer water shortage period under climate change conditions than considering historical series.

Table IV Synthesis of results for the 42 years simulation of the Agri-Sinni water supply system also considering the adopted climate change scenario

Scenario	Use	80% quantile release		Volume based reliability	Time-based reliability related to			Def_{max}/D
					Total demand	Reduced demand		
		hm^3	% dem	Rv		Threshold	Rt	
Historical series	Municipal	290.84	99.3%	0.977	0.786	0.90	0.929	0.318
	Industrial	12.61	100.0%	0.975	0.857	0.90	0.952	0.499
	Irrigation	180.27	75.2%	0.855	0.667	0.60	0.810	0.812
Series under climate change	Municipal	263.82	90.1%	0.949	0.524	0.90	0.810	0.480
	Industrial	11.54	91.5%	0.939	0.714	0.90	0.833	0.833
	Irrigation	54.17	22.6%	0.609	0.286	0.60	0.476	0.936

From Table IV can be inferred that, despite the reduced availability of water resources at the water system plants, municipal and industrial supplies are still guaranteed with satisfying level of volume-based reliability up to 90% and acceptable levels of time-based reliability up to 80% for a total demand reduced by the 10%. On the contrary, releases to irrigation use suffer the most important impacts with a volume based reliability still acceptable (61%) but characterized by a strong variability that produce a value for the 80%-quantile release less than 25% of the annual demand. This behaviour is reported also by the time-based reliability that is reduced to less of the 30% for total demand and less than 50% for the reduced demand (with 0.6 threshold).

A comparison between probability of monthly and annual water shortages for municipal and irrigation uses for historical series and under a climate change scenario is shown in Figure 4.

Analysis of Figure 4 shows as, under the adopted climate change scenario, municipal use mainly presents an increase of the probability of water shortages both at monthly and annual scale but without a large increase on the severity of the water shortages also thanks to the target storages implemented as operating rule.

Whereas irrigation use presents severe increases both in probability and severity of water shortages especially for the months after March when target storages reduce the availability of water to be allocated for irrigation in favour of municipal producing monthly water shortages with probabilities greater than 0.5 and severity belonging to the 80-100% of the monthly demand.

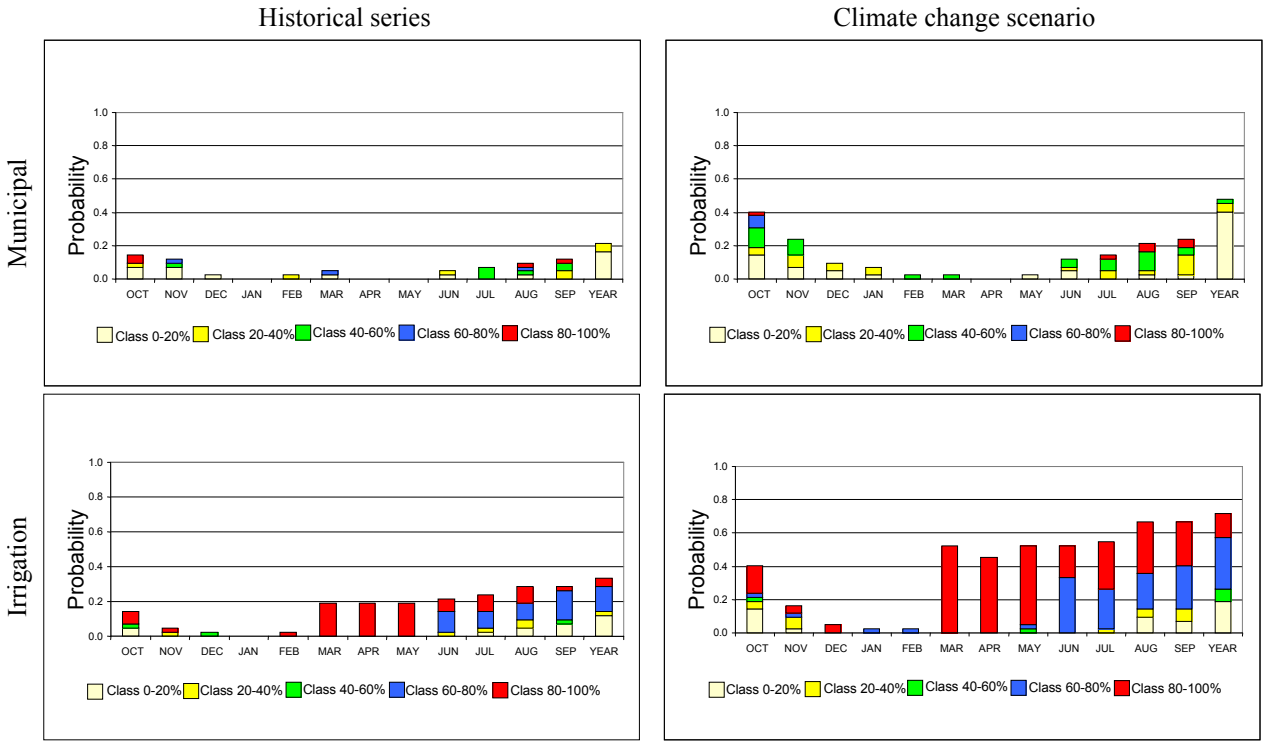


Figure 4 Comparison between probability of monthly and annual water shortages for municipal and irrigation uses considering historical series and under the adopted climate change scenario

5. CONCLUSION

A pilot implementation of a methodology for water shortage risk assessment within the SWSPP has been carried out by considering historical hydrological conditions and likely impacts of climate change. The Agri-Sinni water supply system has been simulated considering the future configuration of plants, structural and not structural mitigation measures to mitigate drought impacts and reduced water availability at the river sections due to the impacts a specific climate change scenario. This scenario has been developed taking into account the assessment carried out by the three Working Groups (WGs) of the Intergovernmental Panel on Climate Change (IPCC) and the IPCC's Fourth Assessment Report (AR4) but also considering local trends on observed data in precipitations for 42 years (1963-2004).

A general decrease on precipitation and increase on temperatures can be inferred by the observed time series with greater precipitation reduction for the JFM quarter. Less precipitation generates less streamflows at the river sections under investigation with a greater percentage of reduction close to the 30% in some extreme cases. When the impacts of climate change are studied through the simulation of the system different water shortages result for different uses both in terms of magnitude and occurrence probability. In particular for the Agri-Sinni water supply system municipal and industrial uses mainly presents an increase of the probability of water shortages both at monthly and annual scale but without a large increase on the severity, on the other hand, irrigation use presents severe increases both in probability and severity of water shortages.

ACKNOWLEDEGMENTS

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DROUGHT RISK REDUCTION IN THE DESERT STATE OF RAJASTHAM, INDIA

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Abstract: This paper highlights the situational analysis of drought in the state of Rajasthan followed by strategies undertaken for drought risk reduction. The paper presents bottom up action plan and its impact. The paper further highlights the preparedness plans that would promote a more preventive risk management approach to drought management and reduce vulnerability to water scarcity and dependence on emergency assistance from governments and international organisations.

Key words: drought, desert, water scarcity, preparedness

1. INTRODUCTION

Water scarcities, droughts and famines have impacted very large populations and for long periods in Rajasthan and western India. Rajasthan today is the largest state in India with a land area of 342,000 sq.km and a population of 55 million according to the 2001 census.

Drought in the history of Rajasthan has been a recurrent phenomenon. The following old proverb reflects on the water scarcity & its familiarity in Rajasthan

"Sat kal, satias jamana, trisath kabakacha baki tinghesan"

It means that in every 100 years in this region there are 7 years of acute drought, 27 years of scarcity, 63 manageable years and 3 normal years.

Sir Diggs La Touche who made the first regular settlement of Ajmer-Merwara in 1874 said:

"Ajmer-Merwara can never rely on two good harvests in succession."

Famines, which disfigure the history of this province, distinguish it from all other provinces of India, since there is no province or district in the country where famines occur so frequently.

The state of Rajasthan has only 1% water resource of India whereas its area is 10% & population 5% of the entire country. There are 12 districts comprising 60% of the area of this state that fall within the Great Indian Desert or Thar which also qualify it to be the most populous desert in the world. 64% of the population of the state resides in the Thar despite scanty rainfall with all its variations, timing and intensity.

The rural economy in this region is predominantly based on agriculture and animal husbandry and the livelihood of people is dependent on 'monsoonal' behavior. The production of crops is entirely a variable of the quantity and frequency of showers. Also, the availability of fodder for animals which exceed the human population in the Rajasthan is primarily dependant on the rains.

Situation Analysis:

It has been observed that rains 16-25% below normal lead to droughts and scarcity of varying proportions. Rajasthan in 2000-2003 faced the most severe drought in the last 138 years and that too for four years in succession. Drought conditions were prevalent in 31 out of the 32 districts, with 30583 villages, 33 million human population and 40 million livestock. 3.3% of the total human population of 1000 million of the country fought a losing battle against this natural disaster for four

years in succession which impacted to a great extent the vulnerable populations mainly women, especially pregnant and lactating mothers, old and infirm people, especially abled and destitute children, infants/new born babies, 982,804 small farmers, 1,331,008 marginal farmers and 1,160,296 landless laborers.

The effects of drought on population were manifested in the form of a large scale migration of the human population along with their cattle mostly from southern and western Rajasthan to the neighboring states of Gujarat, Uttar Pradesh, Punjab and Haryana in search of work. There were unconfirmed media reports of the sale of girl children in south Rajasthan and isolated starvation deaths which were not substantiated by ground realities due to the efforts of the government and other stake holders.

The drought of 2000-2001 impacted the water sources due to an overall deficit rainfall. There were genuine apprehensions of water riots and an adverse impact on the drinking water supply to major cities, towns and villages and also on the recharge of ground water aquifers, wells, tanks and other water bodies. However, this situation was addressed by ferrying water through rail, road and by identifying new water sources and augmenting the existing ones.

In this drought crops were severely affected in 19,817 villages where the damage was more than 75% and in 10766 villages it was between 50-74%. 8,950,000 hectares of the kharif (summer) crop on 116,000,000 hectares had been damaged severely resulting in a financial loss of $763 million in kharif and $400 million in rabi (winter).

An in-depth analysis of the reasons for recurrent phenomena of drought reveals that only 5 districts out of 32 in Rajasthan, receive less than 300 mm of rainfall, 7 districts receive a rainfall between 300-500 mm and the rest of the 20 districts receive more than 500 mm of average annual rainfall. During the year 2000 the pattern of rainfall showed that 6 districts, Banswara in the Udaipur Division and 5 others in the Kota Division received more than 550 mm of rainfall, despite which widespread water scarcity was prevalent in all these districts. Districts like Ganganagar and Hanumangarh even in normal years, receive less than 300 mm of average annual rainfall. They usually are not impacted by drought because of alternate and additional sources of irrigation through the canal system. In addition, other factors contributing to recurrent drought relate to inadequate & uneven distribution of rainfall especially at the critical crop growth and flowering stages. It has been seen that the average annual number of rainy days are just about 34.3 in eastern Rajasthan and only 16.8 in western Rajasthan.

The other major contributory factor for the deficient availability of water is the inadequate spread of watershed management in the region where the area already taken up under watershed development is just 1,900,000 hectares out of a total cultivable area of 24,400,000 hectares(which is just 6% of the total impact able area). The state irrigation department's assessment is that 84.6% of the rainwater is lost through surface run- off and evapo-transpiration leading to less availability of assured irrigation in major parts of the region.

The demand side management of water in rural areas reveals that communities with all the advocacy and extension programs are slowly waking up to the realities of dry land farming techniques. There is a concerted effort to use certified seeds of less water using traditional crops like moth and guar. Various initiatives have been taken to promote drought resistant species of crops and to revive the traditional livelihood patterns of people in rural areas. One major area of concern is the excessive number of unproductive cattle on account of religious sentiment but then people are now realizing their economic priorities to modify future strategies.

The financial support from government for drought and water management can be traced back to 1950, where after it alone has spent nearly $ 2000 million on drought-relief activities many of which were directed at the creation, rejuvenation & augmentation of water management structures. As an improved strategy which was initiated in the year 2000-2001 nearly $ 200 million were spent on about 100,000 drought-relief works 53,000 of which related to water conservation.

Strategies:

There are short & long term livelihood options for meeting water scarcities. The former options rely on generation of immediate employment opportunities for farm men and women by food for work programs. The works undertaken under Food for Work (FFW) should be more focused on water conservation & harvesting which in the long term would reduce the need for immediate employment opportunities every year. The wage rate which is fixed from time to time should also be linked to the CLI Consumer Living Index. It should not be dependent on the government and should be calculated on the basis of predefined formulae.

The long term livelihood options contribute to increased watershed management and other water conservation practices. Since 20 districts out of 32 receive more than 500 mm of rainfall adoption of modern and traditional water conservation and harvesting measures by construction of anicuts, 'naadis', check dams, 'johads', 'beris' and adoption of low cost and efficient irrigation practices of sprinkler and drip irrigation practices could additionally save 40 % of the water available for irrigation.

The impact of improved practices can lead to an increase in the irrigated area from 33% to as much as 80% and also has a potential of saving at least 50% of the rainwater out of a total of 84% of the remaining rainwater which is other wise lost due to surface run-off. Increase of cropping intensity from 119 % to more than 250 % is possible with the realization of this increased irrigation potential which would be accompanied by an increase in the number of working days on farms and an increased income with less risk.

Development in the agriculture sector requires that dry-land farming is encouraged in the desert districts with improved agronomic practices like spacing, use of seed cum fertilizer drill, weed management, emphasis on less water consuming crops like kair, sangri which are suitable to local conditions and are commanding a very high market value. This issue has also been raised in the recent book ' India- A Vision for 2020' by the Hon'ble President of India where the immediate need to have a second green revolution in the dry land regions of India has been very strongly underlined.

Discussion:

Bottom Up Action Plans: What is required in the immediate action plan is to develop a cost-effective strategy focused on raising rural economies for sustainable development and drought-risk reduction. There is an immediate need to create "*islands of excellence*" in 500 villages of the province in the first year to have a "*radiation impact*" on the neighboring villages in the state of Rajasthan. The idea is to mobilize communities for undertaking construction of at least one water harvesting/ conservation project in these villages. The benefits of this water project would motivate village communities to take to such works on priority over other developmental initiatives. The communities would also come forward to contribute "*shramdaan*"- *voluntary labor* after understanding the tangible returns from such works. In the first year, 500 villages could be chosen in about 25 districts of the state. These 500 villages would be selected from nearly 40,000 villages in the state and would be most vulnerable to recurrent droughts / scarcity. The information about vulnerability would be obtained from the GIS of the Regional Remote Sensing Centre, Jodhpur with whom the districts could have an arrangement for information & knowledge sharing and capacity building. The program on the success of the 1st years' experience would be extended in 10 years to cover 10,000 villages in the state. The cost involved per village would be nearly $ 10,000 only wherein 60% funds shall be spent on wages for providing immediate employment to the poorest people.

Expected Impact:

The immediate impact of such a program would be visible, perceptible and tangible returns on this investment of about $ 10,000 only per village in terms of quantifiable increase in yield per acre of crops and also in number of crops in a year, considerable increase in water availability for irrigation and substantial raise in the water table, increase in food and fodder availability, up gradation in livestock productivity and in the sale of products like milk, wool and meat. The plan

would also lead to a higher purchasing power of small and marginal farmers, improved soil conditions and quality & also these initiatives would provide immediate livelihood as wage employment.

This "*seeing is believing*" approach will have paradigm shift in public spending and the villages will utilise other rural development and *panchayat* funds for water management and conservation. The results would be evident in the form of improved rural economies for sustainable development and it would become a role-model for water management for other regions in the country. The policy impact of this strategy would be in terms of the inclusion of the concept of water management in regular development programs to build empowered communities well equipped to fight the recurrent problem of droughts / water scarcity.

Conclusion:

Top Down Plans at the state level: We should focus on a preventive, anticipatory approach to water scarcity and drought reduction, mitigation and management. We should favor preparedness over insurance, insurance over relief and incentives over regulation. We need to set research priorities based on the potential of the research results to reduce water scarcity impacts & to co-ordinate the delivery of all stakeholders/partners through co-operation and collaboration.

We should determine the needs that exist at the state and local levels to prepare for and respond to water shortage emergencies. There is a need to review all existing state laws and programs relating to drought and water management, determine differences between the needs of the users and the state legal provisions and programs designed to mitigate the impact of and response to water shortages.

In addition to the bottom up initiatives a new strategy at the province level is required with the constitution of a core team/task force/ sub-committee on water conservation & management, assessment of region-specific, agro-climatic, zone-specific, water conservation & eco-regeneration indigenous & scientific best practices in Rajasthan through review of past literature, symposiums, seminars, workshops organized by the government, NGOs & academicians. It is also suggested that need based new studies be undertaken with emphasis on the identification of region-specific CBO efforts & study of successful government programs and projects as case studies

We must try to ensure that advocacy campaigns include organization of seminars/conventions at regional levels to help prepare region-specific strategies/action plans for water conservation & management and eco-regeneration through participation of all stakeholders. Awareness generation and mass advocacy for water conservation and management can be brought about through media very effectively through the use of suitable print, electronic / internet facilities and also through interactions with educational institutions.

Preparedness plans are a very critical part of drought & water management for risk reduction and would contain three critical components:

(1) A comprehensive early warning system
(2) Risk and impact assessment procedures
(3) Mitigation and response strategies

The preparedness plans would promote a more preventive risk management approach to drought management and reduce vulnerability to water scarcity and dependence on emergency assistance from governments and international organisations. The process of developing a plan would identify vulnerable areas, population groups & economic / environmental sectors which would also seek to identify data & information gaps along with research and institutional needs.

Ultimately, preparedness plans will improve co-ordination within and between levels of government, upgrade procedures for monitoring, assessing and responding to water shortages; ensure improved information flow to primary users and affect a perceptible increase in the efficiency of resource allocation.

It is imperative that we get out of the mode of using archaic scarcity codes to handle droughts and scarcities where it's often a case of "too late, too little" and get into an arrangement which has a legal framework which defines responsibilities and role plays of stake holders,

organizations & individuals. At the same time it spells out the rights and entitlements of vulnerable populations. The state government should specify the entitlements of the vulnerable sections during droughts and scarcities. The people should not feel obligated to the government in any way. The assistance should come as a matter of right during these scarcities. The Act must have a provision to make recommendations on improving public awareness of the need for water conservation and management and on developing a co-ordinated approach to water conservation, use, management and response by governmental and non-governmental agencies, including academic, private and non-profit interests. This would require inputs from various people including representatives of the vulnerable groups where after it could come into force with the consent of the elected house in the state.

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COMPARISON OF DROUGHT INDICES IN GREECE

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Abstract: Drought is one of the major natural hazards, most often caused by a departure of the precipitation from the normal amount. Many drought indices have been developed to identify the beginning, end, spatial extent and severity of a drought, using various meteorological and hydrological parameters. One of the prospects of the farming community is to handle agricultural drought on an increased temporal and spatial resolution. The drought indices AWn and ETDIn were developed using a daily single layer water balance model that calculates daily evapotranspiration from daily measured meteorological variables, expressing soil moisture deficit. The drought indices EDI_{365} and $EDI_{28,}$ based only on daily precipitation, are also calculated to express the amount of precipitation needed for the recovery from the accumulated deficit since the beginning of drought. All these indices have a high temporal resolution (daily). On the other hand, one of the most widely used drought indices, the Standardized Precipitation Index (SPI), based only on rainfall data and accepted as a good measure of short- and long-term drought, has a coarse temporal resolution (monthly). In this study, SPI is calculated on 1-, 2-, 3-, 6-, 9-, 12- months time scale (SPI_1, SPI_2, SPI_3, SPI_6, SPI_9, SPI_{12} respectively) in 28 stations in Greece, for the period 1974-2001. These SPIs are compared with the monthly values of the four other indices, the available water index (AWn), the evapotranspiration deficit index (ETDIn) and the effective precipitation indices (EDI_{365} and EDI_{28}) that estimate droughts in a finer temporal (and spatial) resolution. SPI index is in great agreement with the finer water balance index $AW_{n,}$ (representing the effects of both precipitation and potential evapotranspiration in droughts) mainly for the short-length time scales of about three months, while for the long-length time scales between six and twelve months, SPI is in better coincidence with the effective rainfall index EDI_{365}.

Key words: available soil moisture index, effective precipitation index, evapotranspiration deficit index, standardized precipitation index

1. INTRODUCTION

Drought is a complex phenomenon, most often caused by a departure of precipitation from the normal amount. It is a creeping phenomenon that accumulates over a period of time across a vast area and the effect lingers for years even after the end of drought. Hence, the loss of life, economic impact and effects on society are spread over a long period of time, which makes drought the worst among all natural hazards. Agriculture is often the first sector to be affected by the onset of drought, due to its dependence on water resources and soil moisture reserves during various stages of crop growth. Soil moisture and crop water stress estimates are especially valuable for estimating the consequences of climatic variability on crop growth and production. Some of the hydrologic models, based upon daily data, may simulate evapotranspiration or soil water content estimates, at a finer spatial resolution. An increased temporal and spatial resolution of hydrological and meteorological parameters over a large watershed, may give the farming community, water managers and policy makers a better tool for assessing, forecasting and managing agricultural drought on a much finer scale.

Researchers usually try to simplify drought, in a drought index, which quantifies climatic anomalies in terms of intensity, duration and spatial width (Wilhite et al., 2000), in order to develop drought prediction tools, which may support timely implementation of preparedness and mitigation

measures. A large number of indices have been developed for handling drought, each of them requiring a variety of data and giving a different evaluation of the phenomenon. The majority of drought indices have a standard time scale, which does not allow the determination of drought for other shorter or longer time scales. A lot of them are complicated, based on evapotranspiration or other water balances components [Palmer Drought Severity Index (Palmer, 1965); Crop Moisture Index (Palmer, 1968); FAO Aridity Index (Doorenbos and Pruitt, 1977); Surface Water Supply Index (Shafer and Dezman, 1982); Standardized Anomaly Index (Katz and Glantz, 1986); NOAA Drought Index (Strommen et al., 1987; Titlow, 1987); Evapotranspiration Deficit Index; Soil Moisture Deficit Index (Narasimhan and Srinivasan, 2005); Reconnaissance Drought Index (Tsakiris et al., 2006)]. It must be noticed though, that indices, based only on precipitation data, have been referred as performing quite well, compared with complicated hydrological indices (Oladipo, 1985; Narasimhan and Srinivasan, 2005). Between them, the Standardized Precipitation Index (McKee et al., 1993; 1995) and the Effective Rainfall Index (Byun and Wilhite, 1996) are worth noticed.

From all the simple indices proposed to monitor and define drought, the standardized precipitation index SPI, recommended by Guttman (1999), as a primary drought index, is one of the most widely used by scientists, since only monthly precipitation data are required for its computation. It has been employed in a lot of studies for describing not only meteorological but also sometimes agricultural or hydrological drought and furthermore, its spatial and temporal distribution has been used as a tool for prediction of drought. (Lana et al., 2001; Bordi and Sutera, 2001; Keyantash et al., 2002; Lloyd-Hughes and Saunders, 2002; Quiring et al., 2003; Bonaccorso et al., 2003; Rouault and Richard, 2003; Bordi et al., 2004; Loukas and Vasiliades, 2004; Piccarreta et al., 2004; Bordi and Sutera, 2004; Tsakiris and Vangelis, 2004; Sonmez et al., 2005; Paulo et al., 2005; Vicente-Serrano and Lopez-Moreno, 2005; Vicente-Serrano, 2006; Paulo and Pereira, 2006; Mishra and Desai, 2006; Bhuiyan et al., 2006; Kim et al., 2006; Shiau, 2006; Tonkaz, 2006; Smakhtin and Hughes, 2006; Vicente-Serrano and Cuadrat-Prats, 2007; Livada and Assimakopoulos, 2007; Papaioannou et al. 2007; Shahid, 2007; Wu et al., 2007; Labedzki, 2007; Balling and Goodrich, 2007; Pandey et al., 2007; Patel et al., 2007).

Among the simple indices based on daily rainfall data, the Effective Rainfall Index (EDI) has gained recently acceptance (Kalamaras and Michalopoulou, 2004; Papaioannou et al., 2005; Smakhtin and Hughes, 2007; Akhtari et al., 2008; Pandey et al., 2008), whereas, the Evapotranspiration Deficit Index and the Soil Moisture Deficit Index, based upon water balance estimates are proposed to be used in studies aiming to estimate drought on a much finer scale in space and time (Narasimhan and Srinivasan, 2005).

The inherent complexity of drought implies that no drought index is ideal for all regions and tasks. It is often necessary to consider more than one index, investigate their sensitivity and accuracy as well as the correlation between them and finally detect how they compliment each other for the purposes of a specific research. A lot of studies, compare finer or more complex drought indices, to the SPI index, in order to monitor drought accurately (Ntale and Gan, 2003; Narasimhan and Srinivasan, 2005; Bhuiyan et al., 2006; Morid et al., 2006; Hughes and Smakhtin, 2007; Kangas and Brown, 2007; Akhtari et al., 2008; Pandey et al., 2008; Trnka et al., 2008).

In this study, daily measurements of dry and wet bulb temperatures, wind speed, actual hours of sunshine and rainfall, obtained from the National Meteorological Service for 28 sites in Greece, are employed in a single layer water balance model for obtaining the Evapotranspiration Deficit Index (ETDIn) and the Soil Moisture Deficit Index (AWn). Daily or monthly values of rainfall are also used to obtain the indices EDI_{28} and EDI_{365} or the standardized precipitation indices $SPI_{1,}$ $SPI_{2,}$ $SPI_{3,}$ $SPI_{6,}$ SPI_{9} and SPI_{12}, respectively. The calculated on a daily step drought indices AWn, ETDIn, EDI_{365} and EDI_{28} (selected to estimate droughts in a finer temporal and spatial resolution) are compared to the longer time scale widely used SPIs, in order to investigate the more accurate monitoring and definition of drought in Greece.

2. MATERIALS AND METHODS

Daily measurements of dry and wet bulb temperatures, wind speed, actual hours of sunshine and rainfall are obtained from the National Meteorological Service for 28 sites in Greece evenly spread in the country (presented in Table 1) for the period 1974-2001. Daily rainfall measurements and crop reference evapotranspiration estimates (PET) based on Penmann-Monteith equation (Allen et al., 1998) are utilized in a single-layer water balance model, for calculating the daily available soil moisture index (AW) and the daily evapotranspiration deficit index (ETDI) as the ratio (PET-AET)/PET, where AET is the actual evapotranspiration. The daily indices AW and ETDI are moreover standardized as:

$$AW_n = (AW - \overline{AW})/STDEV(AW) \quad (I)$$

$$ETDI_n = -(ETDI - \overline{ETDI})/STDEV(ETDI) \quad (II)$$

respectively, in order to depict drought severity, where $\overline{AW}$ and $\overline{ETDI}$ are the mean values of the parameters for the whole period and STDEV(AW) and STDEV(ETDI) are their corresponding standard deviations.

Using only daily rainfall data for the period 1974-2001, the effective precipitation index EDI is also calculated with a daily time step (Byun and Wilhite, 1996) for each station of the study. The first step is the calculation of EP, a function of the current day's precipitation and weighted precipitation over a defined preceding period:

$$EP = \sum_{m=1}^{N}\left[\left(\sum_{i=1}^{m} P_m\right) \Big/ m\right] \quad (III)$$

Where, N is the duration of the preceding period and P_m the precipitation of m-1 days previously. The time series of EP values is converted to deviations from the mean (DEP) and the standard deviations STDEV(DEP) of these DEP values are then estimated for each day. The EDI index is calculated as:

$$EDI = DEP / STDEV(DEP) \quad (IV)$$

This index expresses the amount of precipitation required for the recovery from the accumulated water shortage since the beginning of a drought event. In particular, the EDI_{365} and EDI_{28} indices are defined for a time period of accumulation of 365 and 28 days respectively.

The daily values of the estimated indices AW_n, $ETDI_n$, EDI_{365} and EDI_{28} were compared between them in all possible combinations. The results of linear regressions (correlation coefficient, R) and 'difference measures' (index of agreement, IA) indicated as best regressions, the ones between AW_n and $ETDI_n$ or EDI_{365} (IA ranged from 0.8 to 0.9). The comparisons between AW_n and EDI_{28} were quite satisfying (presenting IA of the order of 0.8), as well as the ones between $ETDI_n$ and EDI_{365} or EDI_{28} (IA ranged from 0.6 to 0.8). Some of the results of the above regressions and 'difference measures' especially between the daily values of the indices AW_n and EDI_{365} or EDI_{28}, are presented in table 1 for every station of the study, during the period 1975-2001. The same comparisons were extended to the (calculated as a second step) monthly values of the mentioned indices and the corresponding results are also shown in table 1. Figure 1 presents the time evolution of the daily AW_n, and EDI_{365} along with mean monthly precipitation in four stations, for selected drought time periods. A rather good compliance of the indices is apparent during the periods 1989-1992 and 1999-2001 when drought events are detected in many studies in Europe. However, it is evident that AW_n is responding even to the small changes in rainfall quicker, as compared to EDI_{365} and as a result turns out of drought while EDI_{365} still remains (Ioannina).

In order to detect the response of these more complex or finer indices in defining drought, it is proper to embody another simple drought index that has already gained acceptance by the scientific community. Thus, monthly rainfall is used for estimating the standardized precipitation index SPI according to McKee et al. (1993; 1995), for these 28 stations and the same period. The cumulative amounts of the precipitation observations are estimated (beginning from the considered month and going backwards as the specific chosen time scale) and the probability distribution of cumulative rainfall is defined by adjusting a Pearson III probability density function on the data. The last one

is transformed to a normal distribution so that mean SPI is zero and the SPI values are the standard deviations of the transformed cumulative amounts of rainfall from the mean value. McKee et al. (1993) distinguished four categories of drought (mild, moderate, severe, extreme) according to the values of the SPI index, ranging from (0 to –1), or (–1 to –1.5), or (–1.5 to –2), or (smaller than –2), respectively. SPI can track drought on multiple timescales. In this study SPI is calculated for each month, at 1-, 2-, 3-, 6-, 9- and 12-month timescales and the corresponding indices are SPI_1, SPI_2, SPI_3, SPI_6, SPI_9 and SPI_{12}, respectively.

Monthly values of the first four finer (daily) indices (AW_n, $ETDI_n$, EDI_{365}, EDI_{28}) are compared to the indices SPI. Tables 2 and 3 present the comparisons of representative SPIs (SPI_2, SPI_3, SPI_6, SPI_{12}) with the monthly values of AW_n, EDI_{28} or EDI_{365}. Analytically, results of the linear regressions and 'difference measures' (correlation coefficient R or index of agreement IA) between SPI_2 (or SPI_3) and the monthly values of AW_n and EDI_{28} (or AW_n and EDI_{365} respectively), for each station of the study are presented in table 2. Similar results for the comparisons of SPI_6 (or SPI_{12}) with the monthly values of AW_n and EDI_{365} are presented in table 3. It is indicated that the best agreements (IA of the order of 0.85-0.97) appear between SPI_6 or SPI_{12} and EDI_{365}. Each of the SPI_2 and SPI_3 appears to be well correlated with AW_n and EDI_{365} (IA of the order of 0.8-0.9). Furthermore, the time evolution of AW_n, EDI_{365} and SPI_{12} throughout the period 1975-2001, for four selected stations of the study is presented in figure 2, in order to present their response in defining drought. All three indices seem to be quite coincident through their time evolution, for the long-length time scales.

3. RESULTS AND DISCUSSION

The knowledge of the temporal distribution of water in the soil profile as a function of atmospheric, crop and soil conditions is very important for irrigation scheduling. Soil moisture and crop water stress estimates are valuable for estimating the consequences of climatic variability on crop growth and production. When daily rainfall data and daily predictions of evapotranspiration are available, the daily water balance can give information on the reliability of the rainfall in meeting crop water requirements, including the probabilities of short-term drought. When there is lack of data, either daily rainfall or monthly rainfall totals over a series of years are employed to evaluate the high risk of droughts and the effect of rainfall on agriculture.

In this article four (finer) daily indices AW_n, $ETDI_n$, EDI_{365} and EDI_{28} are estimated and then compared to the world widely used monthly SPI index, in order to examine each one's response in detecting drought severity and duration in Greece. This study shows that the water balance drought indices (AW_n and $ETDI_n$), describing more precisely drought as based on daily values of rainfall and evapotranspiration, and especially AW_n are very well correlated with simpler indices based only on daily precipitation, such as EDI_{365} and EDI_{28}. The best agreement is the one between AW_n and EDI_{365}, while $ETDI_n$ complies very well only with AW_n. In this way, it is encouraging to examine a drought phenomenon by means of such easily handled indices that remain of great time resolution.

The use of the SPI index, based on even simpler and most available monthly rainfall values, proves to be an also attractive approach in the definition of drought. This index is in great agreement with the finer water balance index AW_n, mainly for the short-length time scales of about three months, while for the long-length time scales between six and twelve months SPI is in better coincidence with the effective rainfall index EDI_{365}. $ETDI_n$ is only in reasonable agreement with the SPIs and the two effective precipitation indices, showing a satisfying correlation mainly with AW_n. The shortage of rainfall is obvious rather in advance in SPIs, as compared with the results of water balance estimations presented in both AW_n, and ETDI indices time series. The differences in the drought indices are mainly attributed to the additional effect of potential evapotranspiration, (apart from precipitation and the different time scale), and this is in accordance with the drought results reported by Narasimhan and Srinivasan, 2005.

STATION	LATITUDE	LONGITUDE	DAILY				MONTHLY			
			EDI_{365}		EDI_{28}		EDI_{365}		EDI_{28}	
			IA	R	IA	R	IA	R	IA	R
IERAPETRA	35º00	25º44	0.815	0.672	0.800	0.644	0.809	0.663	0.819	0.698
IRAKLIO	35º20	25º11	0.875	0.772	0.841	0.712	0.872	0.767	0.835	0.724
RHODES	37º24	28º05	0.834	0.701	0.759	0.585	0.833	0.700	0.753	0.592
CHANIA	35º30	24º02	0.766	0.598	0.759	0.583	0.761	0.585	0.737	0.572
THIRA	36°25	25°26	0.924	0.857	0.743	0.563	0.931	0.870	0.752	0.600
KITHIRA	36°08	23°01	0.812	0.668	0.830	0.693	0.807	0.658	0.824	0.704
MILOS	36º44	24º26	0.844	0.724	0.741	0.558	0.837	0.713	0.740	0.581
METHONI	36°50	21°42	0.796	0.646	0.824	0.688	0.790	0.638	0.833	0.724
NAXOS	37º06	25°23	0.920	0.849	0.752	0.575	0.920	0.850	0.751	0.601
AGHIALOS	39°13	22°48	0.899	0.812	0.800	0.644	0.914	0.840	0.792	0.660
SERRES	41°05	23°34	0.912	0.836	0.797	0.637	0.908	0.830	0.790	0.655
ELLINIKO	37°54	23°45	0.882	0.782	0.847	0.720	0.881	0.783	0.842	0.738
TRIPOLI	37°32	22°24	0.844	0.717	0.818	0.679	0.844	0.717	0.810	0.693
FLORINA	40°48	21°25	0.916	0.842	0.797	0.641	0.919	0.847	0.803	0.684
CHIOS	38°21	26°09	0.817	0.677	0.739	0.557	0.808	0.663	0.712	0.528
ARGOSTOLI	38°11	20°29	0.808	0.663	0.831	0.699	0.802	0.656	0.833	0.720
ANDRAVIDA	37º55	21º17	0.811	0.667	0.822	0.687	0.810	0.666	0.823	0.711
MYTILINI	39°04	26°36	0.786	0.626	0.859	0.742	0.779	0.619	0.869	0.771
MIKRA	40°31	22°58	0.922	0.852	0.831	0.694	0.926	0.862	0.826	0.719
LARISA	39º39	22º26	0.917	0.844	0.802	0.644	0.919	0.848	0.798	0.669
IOANNINA	39º40	20º51	0.830	0.696	0.802	0.656	0.828	0.694	0.805	0.684
KERKYRA	39º37	19º55	0.788	0.625	0.846	0.723	0.783	0.621	0.854	0.755
KOZANI	40°17	21°47	0.937	0.879	0.796	0.637	0.937	0.880	0.814	0.700
ALEXAND/LIS	40°51	25°26	0.890	0.797	0.823	0.682	0.888	0.795	0.824	0.709
AGRINIO	38º37	21º33	0.819	0.678	0.819	0.684	0.820	0.677	0.821	0.715
KAVALA	40°56	24°23	0.919	0.847	0.804	0.650	0.925	0.859	0.811	0.697
LAMIA	38º51	22º24	0.852	0.732	0.834	0.701	0.865	0.755	0.828	0.720
KALAMATA	37º04	22º01	0.847	0.723	0.840	0.714	0.853	0.733	0.828	0.719

Table1. Results of the linear regressions and difference measures [correlation coefficient (R), index of agreement (IA)] of the daily (or monthly) values of AW_n with the daily (or monthly) values of EDI_{365} or EDI_{28} for the period 1975-2001

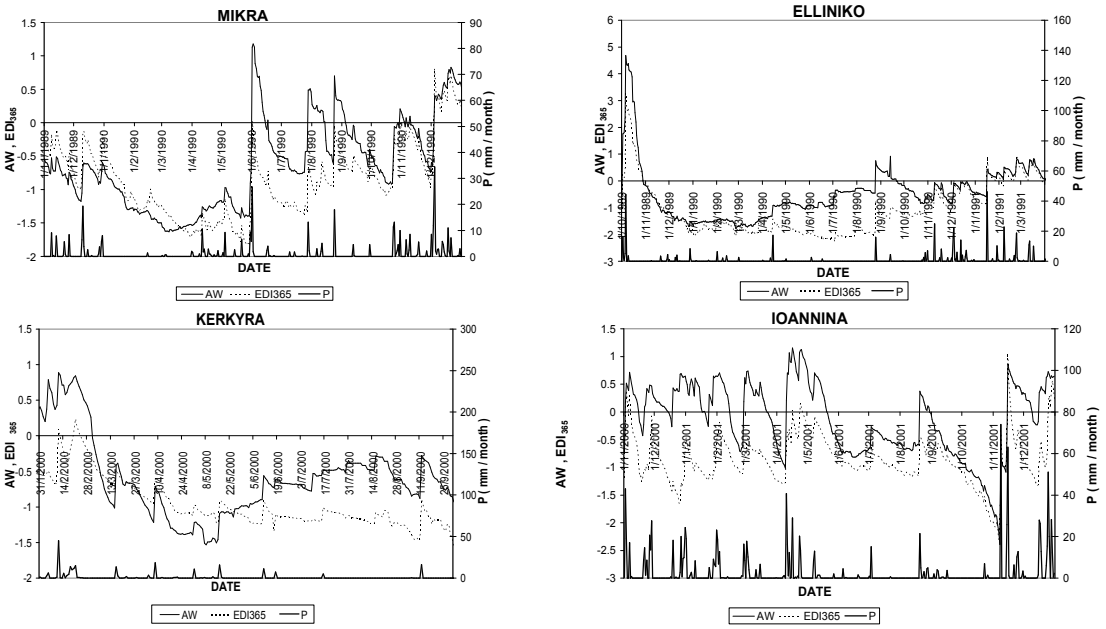


Figure1. Annual variation of Aw_n, EDI_{365} and precipitation (P) for selected time periods in four stations of the study

Table 2.Results of the linear regressions and difference measures [correlation coefficient (R), index of agreement (IA)] of SPI_2 (or SPI_3) with the monthly values of AW_n and EDI_{28} (or AW_n and EDI_{365} respectively), for the period 1975-2001

STATION	SPI_2				SPI_3			
	AW_n		EDI_{28}		AW_n		EDI_{365}	
	IA	R	IA	R	IA	R	IA	R
IERAPETRA	0.813	0.673	0.798	0.681	0.844	0.722	0.777	0.606
IRAKLIO	0.823	0.690	0.798	0.681	0.854	0.738	0.835	0.703
RHODES	0.800	0.649	0.770	0.635	0.831	0.698	0.791	0.624
CHANIA	0.774	0.610	0.815	0.714	0.804	0.655	0.788	0.616
THIRA	0.733	0.605	0.763	0.623	0.854	0.739	0.838	0.713
KITHIRA	0.837	0.711	0.804	0.691	0.855	0.741	0.794	0.633
MILOS	0.783	0.627	0.778	0.662	0.838	0.712	0.793	0.627
METHONI	0.812	0.678	0.779	0.669	0.807	0.676	0.810	0.662
NAXOS	0.808	0.665	0.800	0.692	0.872	0.769	0.829	0.696
AGHIALOS	0.834	0.707	0.849	0.794	0.868	0.767	0.861	0.752
SERRES	0.833	0.709	0.828	0.775	0.865	0.760	0.858	0.747
ELLINIKO	0.850	0.731	0.830	0.738	0.866	0.762	0.853	0.733
TRIPOLI	0.830	0.703	0.840	0.764	0.832	0.705	0.882	0.784
FLORINA	0.870	0.764	0.876	0.825	0.893	0.804	0.917	0.845
CHIOS	0.757	0.585	0.755	0.604	0.796	0.640	0.814	0.672
ARGOSTOLI	0.848	0.728	0.843	0.754	0.850	0.734	0.824	0.681
ANDRAVIDA	0.831	0.702	0.828	0.732	0.832	0.704	0.827	0.684
MYTILINI	0.845	0.726	0.825	0.723	0.832	0.708	0.827	0.682
MIKRA	0.859	0.753	0.835	0.776	0.889	0.801	0.893	0.803
LARISA	0.836	0.705	0.856	0.797	0.869	0.765	0.872	0.769
IOANNINA	0.837	0.714	0.849	0.783	0.835	0.708	0.881	0.782
KERKYRA	0.849	0.733	0.844	0.763	0.830	0.705	0.831	0.690
KOZANI	0.869	0.760	0.866	0.801	0.903	0.819	0.909	0.830
ALEXAND/LIS	0.867	0.764	0.844	0.774	0.885	0.797	0.875	0.773
AGRINIO	0.833	0.712	0.832	0.754	0.827	0.697	0.857	0.738
KAVALA	0.859	0.747	0.841	0.775	0.891	0.802	0.897	0.807
LAMIA	0.824	0.685	0.845	0.756	0.852	0.733	0.832	0.696
KALAMATA	0.841	0.724	0.816	0.729	0.839	0.720	0.841	0.713

Table 3. Results of the linear regressions and difference measures [correlation coefficient (R), index of agreement (IA)] of SPI_6 (or SPI_{12}) with the monthly values of AW_n and EDI_{365}, for the period 1975-2001

STATION	SPI_6				SPI_{12}			
	AW_n		EDI_{365}		AW_n		EDI_{365}	
	IA	R	IA	R	IA	R	IA	R
IERAPETRA	0.723	0.558	0.872	0.779	0.632	0.399	0.893	0.802
IRAKLIO	0.792	0.642	0.917	0.847	0.668	0.436	0.875	0.770
RHODES	0.706	0.512	0.895	0.805	0.649	0.425	0.891	0.801
CHANIA	0.736	0.559	0.900	0.814	0.630	0.386	0.892	0.802
THIRA	0.892	0.803	0.943	0.892	0.840	0.712	0.951	0.905
KITHIRA	0.743	0.565	0.916	0.846	0.669	0.442	0.920	0.850
MILOS	0.757	0.595	0.892	0.799	0.667	0.463	0.886	0.791
METHONI	0.715	0.527	0.918	0.849	0.658	0.426	0.911	0.833
NAXOS	0.859	0.750	0.919	0.851	0.762	0.587	0.917	0.843
AGHIALOS	0.812	0.679	0.908	0.841	0.692	0.487	0.817	0.691
SERRES	0.833	0.708	0.918	0.850	0.712	0.531	0.848	0.734
ELLINIKO	0.805	0.662	0.932	0.872	0.713	0.519	0.897	0.810
TRIPOLI	0.762	0.586	0.945	0.896	0.706	0.499	0.907	0.827
FLORINA	0.856	0.738	0.964	0.931	0.807	0.659	0.918	0.846
CHIOS	0.739	0.560	0.923	0.855	0.659	0.436	0.909	0.831
ARGOSTOLI	0.729	0.542	0.913	0.838	0.654	0.438	0.904	0.824
ANDRAVIDA	0.751	0.571	0.921	0.852	0.695	0.496	0.924	0.857
MYTILINI	0.735	0.557	0.923	0.855	0.627	0.387	0.926	0.861
MIKRA	0.849	0.733	0.948	0.901	0.733	0.545	0.860	0.747
LARISA	0.841	0.715	0.933	0.875	0.713	0.515	0.852	0.732
IOANNINA	0.741	0.559	0.949	0.904	0.585	0.339	0.858	0.743
KERKYRA	0.706	0.508	0.925	0.859	0.574	0.306	0.882	0.784
KOZANI	0.864	0.755	0.955	0.915	0.744	0.560	0.886	0.792
ALEXAND/LIS	0.805	0.657	0.935	0.878	0.721	0.518	0.896	0.805
AGRINIO	0.749	0.568	0.934	0.876	0.666	0.435	0.913	0.837
KAVALA	0.875	0.772	0.954	0.911	0.805	0.652	0.928	0.864
LAMIA	0.806	0.656	0.921	0.854	0.701	0.490	0.880	0.790
KALAMATA	0.765	0.607	0.915	0.850	0.712	0.520	0.910	0.842

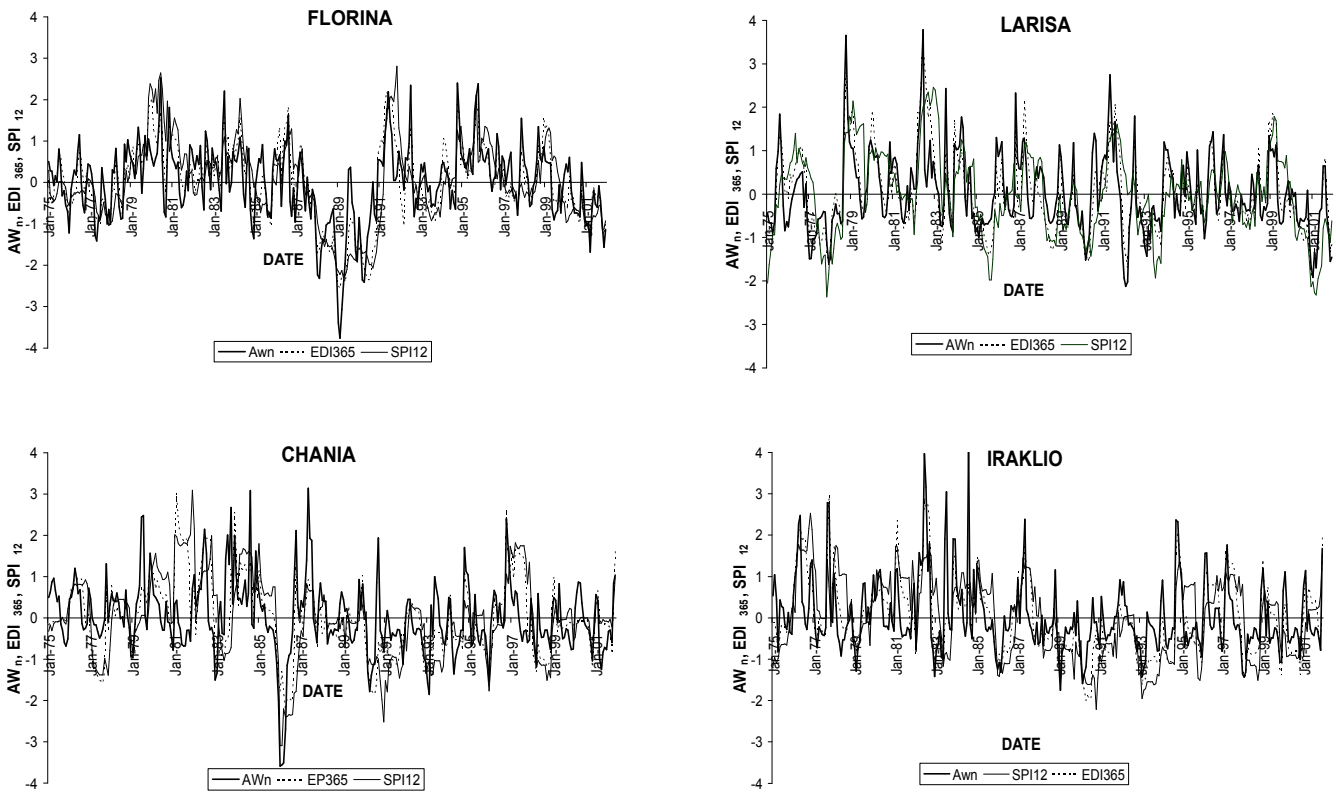


Figure 2. Time evolution of Aw_n, SPI_{12} and EDI_{365} for the period 1975-2001 in four selected stations

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THE GENERATION OF KARST FLASH FLOODS AND THE ANALYSIS OF THEIR MAIN HYDROLOGIC PARAMETERS; CASE STUDY OF THE KOILIARIS RIVER BASIN IN CRETE, GREECE

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Abstract: The objective of the present work is to determine the hydrological and meteorological parameters related to karst and non-karst flood phenomena**.** A karst flash flood is special kind of flash flood and represents one of the deadliest and costliest natural disasters worldwide. The hydrological analysis of a karst flash flood event contributes in the understanding of the runoff creation process. Karst flash flood studies, presented in the past, are based on daily data; nevertheless the accuracy of these data is not satisfactory for the detailed analysis of a flood event. For this reason the analysis of the rainfall and runoff data in hourly base is necessary. This study presents the analysis of a karst and two non karst flash flood events that took place in the Koiliaris river basin. The area of the basin has been estimated to be approximately 130 km^2. In the basin there are three telemetric hydrometric stations and two telemetric meteorological stations. These stations provide hourly real-time data which are used to determine hydrological and meteorological parameters. The main characteristics of various hydrographs specified in this study are the runoff coefficients, the lag time, the concentration time, and the basin mean maximum potential retention. The specification of these hydrometeorological parameters provides us with the necessary information in order to successfully manage and control karst flash flood events.

Key words: Flash floods, hydrographs, karst aquifer, meteorological characteristics

1. INTRODUCTION

A flash flood is defined as a flood which follows shortly, within a few hours, after a heavy or excessive rainfall event (Marechal et al., 2008). The karst flash flood is a special kind of flash flood related to the structure and hydraulic properties of karst aquifers and has been identified as one of the hazards in karst terrains. The majority of the karst flash floods appear in arid and semi-arid areas. The combination of the karst basin characteristics (sparse vegetation, thin rock soils and steep slopes) and the intensive short-term precipitation can create sharp peaks and narrow hydrographs with short time lags. The main difference between a karst flash flood and a non-karst one is the water circulation (Bonacci et al., 2006). In the Mediterranean region, where ephemeral streams are common fluvial systems, rainfall and runoff processes play a significant role to the generation of the karst and non-karst flash floods (Camarasa Belmonte and Segura Beltran, 2001). This paper presents the generation processes of kart flash floods and a detailed example of karst and non- karst flash flood events which occurred in the Koiliaris River basin in Crete.

2. STUDY AREA AND HYDROMETEOROLOGICAL NETWORK

The Koiliaris River basin is located 15 km east of the city of Chania, Crete. The basin extends from the White Mountains (Lefka Ori) to the coastline. The area of the basin has been estimated to be approximately 130 km^2. The elevations of the basin range from zero to 2120 m AMSL. The total length of the hydrological network of the Koiliaris River is 36 km. The Koiliaris River basin consists of two main tributaries, Keramiotis and Anabretis (Figure 1). The length of the river from the intersection point, where all the streams meet, to the discharge point is 3.3 km. The topography

of the area is smooth with a mild topographic slope of 12%. The geology of the basin is mainly constituted by carbonates, quaternary-neogenic deposits and flysch formations. Three telemetric hydrometric stations (H1, H2, H3) and two telemetric meteorological stations (M1, M2) have been installed in the Koiliaris River basin for flood event monitoring (Figure 1). Hydrometric station H1 is located at the tributary intersection point, while hydrometric stations H2 and H3 are strategically located at the inflow and the outflow points of a karst channel, respectively (Figure 1). Meteorological stations M1 and M2 are located at 1000 m and 400 m AMSL, respectively (Figure 1).

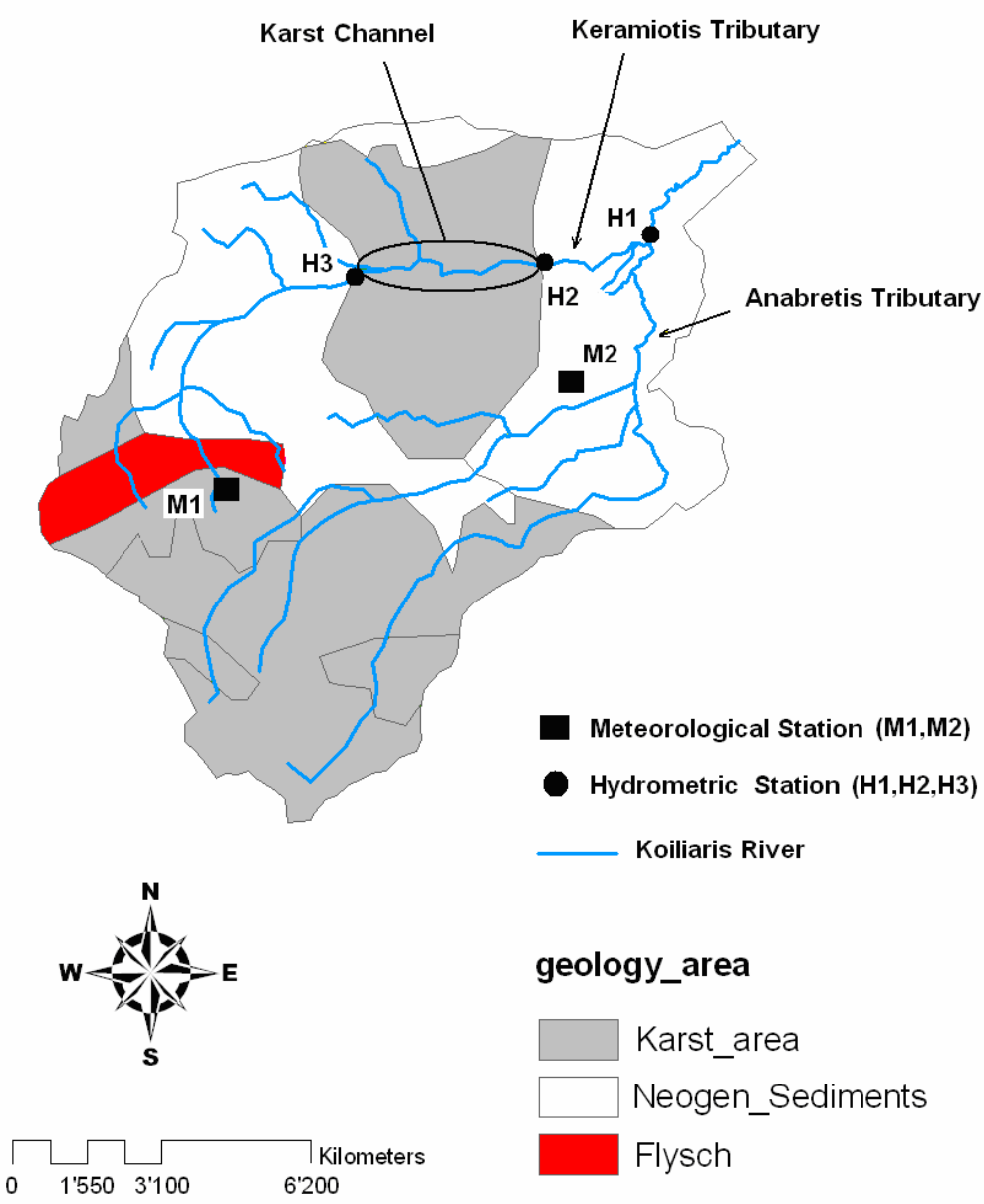


Figure 1. The geology and the hydrometeorological network of the Koiliaris River basin.

3. THE GENERATION OF KARST FLASH FLOODS

The geomorphological characteristics of the basin under study contribute to the generation of many flash floods in the area where station H3 is located. The main tributary of the Koiliaris River, Keramiotis crosses the karst area between the stations H2 and H3. In the past, many karst flash floods have been reported at the discharge point of the karst channel (station H2) which generated extreme flash floods in the downstream area (station H1).

Karst features mainly occur in carbonate rocks, limestones and dolomites (Bakalowicz, 2005). In general, karst areas are characterized by many unknown underground and surface karst features such as caves, jamas, conduits and sinkholes. Karst flash floods have specific characteristics caused by distinct conditions of water circulation and storage in the karstified medium. The main causes of karst flash flood are:

i) the high infiltration rate and small storage capacity of the karst medium,

ii) the strong interaction between the circulation of surface water and groundwater in karst areas, and,

iii) the fast groundwater flow through karst conduits.

In the Koiliaris River the karst channel is deep and narrow and encourages the process of flood wave transmission. The water losses along the karst channel are due to infiltration, evapotranspiration and transmission, not necessarily with equal importance. The initially high losses that occur along the karst channel and the subsequent appearance of karst flash flood correspond to a specific mechanism of karst flash flood generation. This mechanism takes place in three phases (states) described as follows.

A) The zero state: the state of the karst system right before the intense rainfall and after a long dry period. The groundwater level is very low and below the lower level of the surface and groundwater does not flow through the karst channels. Some of the karst channels are clogged from clay or stones and the karst springs in the area are not active.

B) The after rainfall state: the state after the beginning of intense precipitation. The groundwater level remains constant as in the zero state but the existing karst channels are full with water. The water within the karst system is under pressure and the groundwater level in the karst conduits is very high. Due to groundwater pressure, some of the clogs in the conduits are removed and intermittent springs occur on the slopes and in the lowlands. If the intensity and quantity of precipitation are large enough, overland flow can occur in some parts of the terrain. The duration of this phase is not long; it lasts from ten minutes to a few hours. During this time, large and well connected karst conduits in which fast turbulent flows exist are filled.

C) The final state: in this state, the groundwater level in the karst aquifer reaches its maximum value, contrary to the second phase where it is increased only in the karst channels. The third phase lasts longer than the second and its duration depends on the local geomorphology. In many cases, in this final stage apart from floods, landslides and mudslides may occur (Bonacci et al., 2006).

4. HYDOMETEOROLOGICAL DATA AND PARAMETERS

The Keramiotis tributary is characterized by extreme flood events of low frequency but high magnitude. The flood wave is generated in the upstream non-karst watershed and travels to the karst area. Based on this observation, two hydrometric stations (H3 and H2) were installed in order to describe the hydrodynamics of the system. The first hydrometric station (H3) is to report hydrological data of the upstream non-karst watershed and the second one (H2) for the downstream karst area (Figure 1).

The Keramiotis tributary is an ephemeral stream. In most cases, the hydrograph analysis of ephemeral streams is relatively simple since there is no baseflow. In all of the cases studied in this work, the shape of the obtained hydrograph was a typical one of a flash flood regardless of the flood duration: very sharp, with a short time base and steeply rising limbs. The analysis of the flood balance of isolated events contributes to the better understanding of the processes involved in the genesis of runoff. In the Koiliaris River basin, the information related to flash flood events is based on data collected at the three hydrometric stations and on the analysis of the obtained hydrographs. Such studies have been traditionally based on daily data. However, daily data are insufficient for a detailed analysis of the flood development. For this reason, in the present study, hourly rainfall and hydrological data were used for the karst and non-karst flash flood analysis.

The meteorological and hydrological parameters considered in this work were the cumulative value of the rainfall that generates the flood, the flood and rainfall duration, the maximum discharge value of the flood event, the runoff coefficients, the lag and concentration time, and the basin mean maximum potential retention.
The majority of these parameters are recorded directly from the stations (meteorological or hydrometric). The runoff coefficients, the lag and concentration time and the basin mean maximum potential retention are specified from the final form of the hydrographs and the geomorphological characteristics of the river basin.

The runoff coefficient (C) is related to the different land covers and hydrologic soil groups and is expressed as the ratio of event-flow volume over total rainfall:

$$C = \frac{h_R}{h_r} \quad (1)$$

where:
h_R: the runoff event (mm),
h_r : the areal precipitation (mm).

The concentration time (t_c) is most often defined as the time required for a particle of water to travel from the most hydrologically remote point in the watershed to the point of collection. The maximum runoff rate in a catchment is reached when all parts of the watershed are contributing to the outflow (USDA - SCS, 1985). The lag time is defined as the time from the centroid of rainfall excess to the time of peak runoff for a watershed (Green and Nelson, 2002).

The mean maximum potential retention (S), depends on the soil and land cover type, the slope and the management of the river basin. It also changes with time due to the change in soil moisture (Arnold et al., 1998). S is related to the curve number (CN) and is expressed as (USDA - SCS, 1986):

$$S = 254 * \left(\frac{100}{CN} - 1 \right) \quad (2)$$

The SCS method uses curve numbers determined by the different land cover and the soil types.

5. THE EFFECT OF RAINFALL ON KARST AND NON-KARST FLASH FLOODS

In small river basins, the rainfall intensity and the previous content of soil moisture are the most important parameters in the generation of karst and non-karst flash flood events (Camarasa Belmonte and Segura Beltran, 2001). Based on this, parameters that described the geomorphological characteristics of the river basin and the general characteristics of the rainfall can affect the final shape of the hydrograph peaks. These parameters as well as the output hydrological parameters are described below, for three flash flood events.

5.1 Analysis of the output hydrographs

The hydrograph analysis of the data collected at the three hydrological stations, H1, H2, and H3, is presented in this section. The analysis was performed using hourly data for three flood events that occurred during the time period 24-25/01/2008. Two of the flood events were non-karst flash floods (data from stations H1 and H3) and one was a karst flash flood (data from station H2).

A rainfall data analysis related to the three flood events was also performed using the meteorological data of station M1. Station M1 was selected for the rainfall recording since it is located in the upstream area of the Keramiotis tributary and the collected data provide for a better estimation of the water volume flowing into the river.

Figure 2 (A, B, C) shows the flood hydrographs of the three hydrometric stations and the hourly rainfall data from meteorological station M1(indicated by the blue solid line). Parts A and B of Figure 2 show the hydrographs obtained from stations H3 and H1 and represent the non-karst flash flood events, while Part C shows the hydrograph of station H2 that represents the karst flash flood event. The observed flow Q (indicated by the black line), the direct flow Q_{dir} (indicated by the red solid line), and the basic flow Q - Q_{dir} (indicated by the black dotted line) were graphically determined in all hydrographs.

Table 1 shows the rainfall characteristics of the flood events. The hydrological and meteorological parameters of the flood events, obtained from the hydrograph analysis, are presented in Tables 2, 3 and 4 for stations H1, H3, and H2, respectively. Based on Table 1, during the time period 24-25/01/2008 the total rainfall was 200 mm, the duration of the rainfall event was 24 hours, the average intensity of the rainfall was 8.33 mm/h, the standard deviation 8.67 and the coefficient of variation 75.18%.

The time base of the hydrograph of station H3 (Figure 2, Part B) was 16 hours, the rising limb was steep with a maximum discharge of 9.4 m^3/sec, reported on the 24th of January, 2008 at 21:00 and 8 hours after the beginning of the flood event. The recession limb was of almost the same duration as the rising limb with a lag time (t_p) equal to 0.3 hours and a concentration time (t_c) of 0.5 hours. The surface discharge was 15.8% of the total rainfall recorded at the meteorological station M1.

The flood wave was increased downstream (station H1), due to the flow from the two tributaries and the discharge from a spring that is located upstream and close to station H1. Thus, the non-karst flash flood event recorded at station H1 represents the whole excess flow from the entire basin. The time base of the hydrograph of station H1 (Figure 2, Part A) was 41 hours. The rising limb appears steep, with a maximum discharge of 30.5 m^3/sec reported on the 25th of January 2008 at 4:00 and 9 hours after the beginning of the flood event. The recession limb lasted for 32 hours, as a consequence of the delayed subsurface flow contributions coming from the transitional sedimentary forms, fans and piedmonts. The lag time (t_p) and the concentration time (t_c) were 11.7 και 22 hours, respectively. As shown in Table 2, 7.8% of the precipitation was converted into runoff with a delayed response.

The hydrograph of the karst flash flood event obtained at station H2 is relatively simple since there is no baseflow (Figure 2, Part C). The flood event lasted 20 hours. The shape of this hydrograph was very sharp with a short base time and steep rising limbs, a typical one for karst flashy streams. It shows a single peak with a maximum discharge of 3.1 m^3/sec on the 24th of January, 2008 at 23:00 and 5 hours after the beginning of the flood event. The lag time (t_p) was 5 hours, the concentration time (t_c) was 8.33 hours and the surface discharge was 1.3% of the total rainfall (station M1). Some other hydrological and rainfall parameters of these three flood events are presented in Tables 1, 2, 3 and 4.

The karst flash flood that took place at station H2 has a lower maximum discharge than the floods at the other two stations (H1, H3). This is due to the combination of the high permeability of the karst channel and the high value of the mean maximum potential retention of the karst area (Table 4). This results in the reduction of the flood wave travelling downstream. Based on the above analysis, the estimated concentration time (t_c) for station H1 is sufficient for the prediction and mitigation of an oncoming flood event to the downstream area of the Koiliaris River basin.

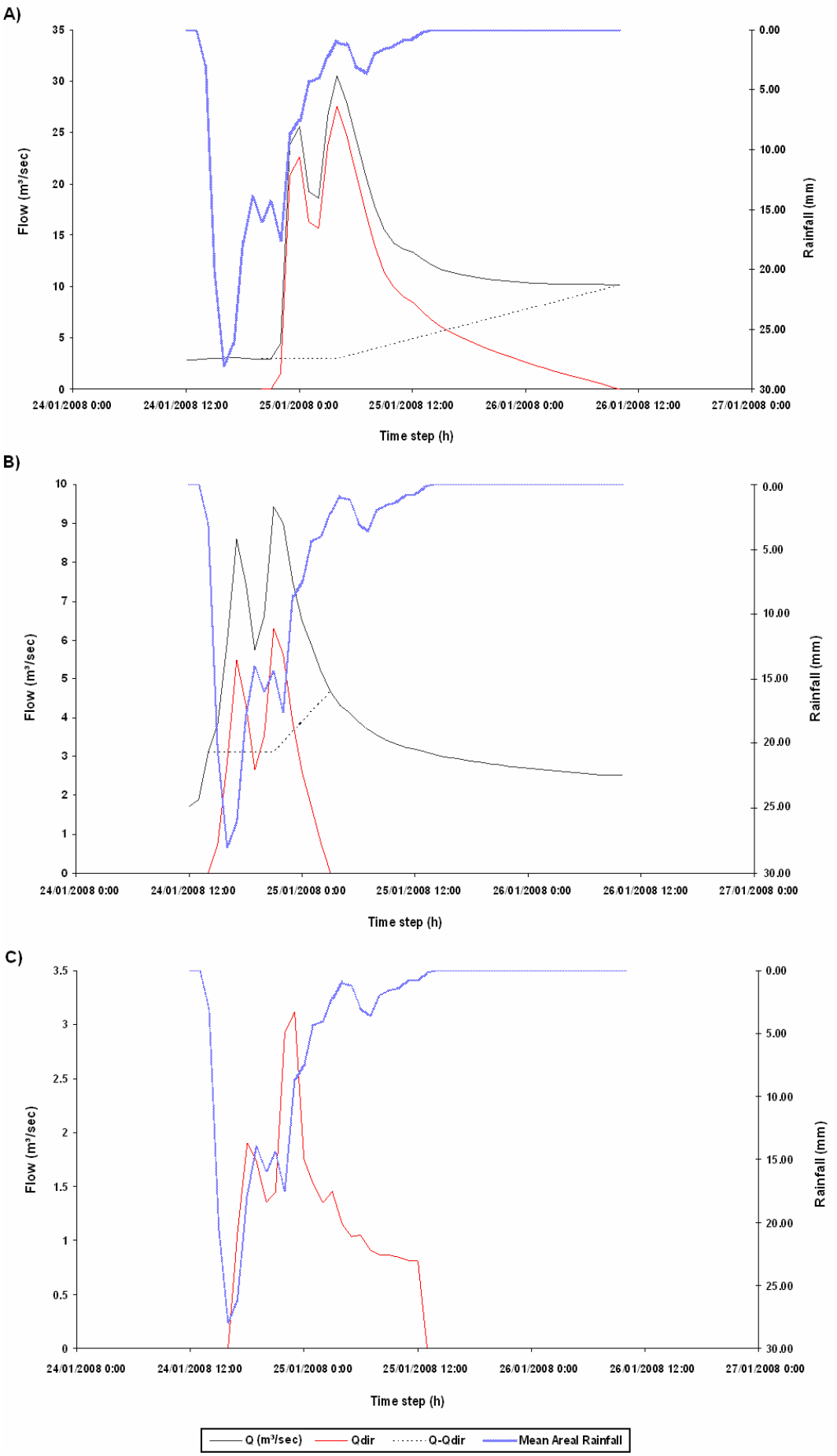


Figure 2. Hourly hydrographs for the three flood events on 24-25/1/2008 at hydrometrics stations H1, H3, H2

Table 1. The rainfall characteristics for the flood event on 24-25/1/2008

Meteorological Station	M1
Total mean rainfall (mm)	200
Duration (h)	24
Average Intensity (mm/h)	8.33
Standard deviation	8.67
Variation coefficient (%)	75.18

Table 2. The main hydrological parameters for the flood event on 24-25/1/2008 at H1

Hydrometric Station	H1
River	Koiliaris
Basin Area (km²)	130
CN	60.58
S (basin mean maximum potential retention) (mm)	165.3
Total runoff depth (mm)	15.58
Direct runoff depth (mm)	8.64
Peak discharge (m³/s)	30.5
Peak water level (m)	0.85
Time to peak discharge	25/1/2008, 4.00
Runoff coefficient for total runoff	0.078
t_c (Concentration time) (h)	22

Table 3. The main hydrological parameters for the flood event on 24-25/1/2008 at H3

Hydrometric Station	H3
Tributary	Keramiotis
Basin Area (km²)	21.15
CN	64.44
S (sub-basin mean maximum potential retention) (mm)	140.2
Total runoff depth (mm)	31.66
Direct runoff depth (mm)	6.88
Peak discharge (m³/s)	9.4
Peak water level (m)	0.8
Time to peak discharge	24/1/2008, 21.00
Runoff coefficient for total runoff	0.158
t_c (Concentration time) (h)	0.5

Table 4. The main hydrological parameters for the flood event on 24-25/1/2008 at H2

Hydrometric Station	H2
Tributary	Keramiotis
Basin Area (km²)	38.2
CN	55.52
S (sub-basin mean maximum potential retention) (mm)	203.5
Total runoff depth (mm)	2.64
Direct runoff depth (mm)	2.64
Peak discharge (m³/s)	3.1
Peak water level (m)	0.55
Time to peak discharge	24/1/2008, 23.00
Runoff coefficient for total runoff	0.013
t_c (Concentration time) (h)	8.33

6. CONCLUSIONS

In the past, the hydrological behaviour of the Koiliaris River basin has been characterized by several flash flood events. Moreover, the existence of fractured and karstified substrates is responsible for karst flash floods. In order to study the karst and non-karst flash flood events, three hydrometric stations were installed in the basin. The first station was placed at the upstream point of the Keramiotis tributary where the flash flood wave is generated, the second station was

strategically placed at the output point of the karst area, in order to record the karst flash flood event, and the last station was placed at the downstream point where flash flood events appear. In this paper, the hydrological response of the three hydrometric stations was studied for a specific flood event by using hydrograph analysis. The hydrographs are very sharp, with steep rises and short lag times in all three cases. The response time of the basin to an intense rainfall event is a few hours. According to the hydrographs, only a small part of the rainfall is converted to runoff, especially in the karst area (station H2) where the runoff coefficient was equal to 1.3%. These high transmission losses along the karst channel play a significant role to the prevention of a severe flood event at the downstream (station H1). In the absence of the karst area in the middle of the Keramiotis tributary, the flood wave would increase the possibility of catastrophic flash floods in the downstream area of the Koiliaris River. Therefore, the determination of the main hydrometeorological parameters is a useful tool for the management of flash flood events.

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PLANNING FLOOD RISK REDUCING MEASURES BASED ON COMBINED HYDRAULIC AND SOCIO-ECONOMIC IMPACT MODELLING AT A MICRO-SCALE

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Abstract: Risk management is currently shifting from the full protection against flooding towards the management of the consequences of flooding. To reduce these consequences, micro-scale analysis is compelled. In addition to the hydraulic modelling of the inundation flow, nearby land use and vulnerability of elements-at-risk are taken into consideration. This new approach requests an interdisciplinary collaboration of scientists from hydraulic and social science, as is the case for this paper. The research teams have developed a methodology to evaluate socio-economic consequences of flooding based on the analysis at the micro-scale of the flood characteristics, the exposure and the vulnerability of elements-at-risk and the adaptive capacity of society. The hydrodynamic simulations are conducted by means of 2D flow modelling run on a highly accurate Digital Surface Model - DSM with a resolution of 2 by 2 meters. The two-dimensional modelling provides high resolution flood maps detailing the distribution of water depth and flow velocity field in the floodplains, which constitute key inputs for the subsequent flood risk analysis. The methodology will be illustrated for a case study along River Ourthe, which is the main tributary of River Meuse in Belgium, for selecting and designing flood protection measure. The geographic data in this area are available at a resolution consistent with the conducted micro-scale risk analysis. They include data on social vulnerability coming from the most recent socio-economic national survey (NIS).

Key words: 2D modelling, adaptation measure, flood risk, micro scale, social impact.

1. INTRODUCTION

Flood risk management focuses on the reduction of flood risks. This means on one hand that flood probability is aimed to reduce, and on the other hand that consequences of flood risks are meant to limit. Acknowledging that full prevention against flooding is a difficult, even unfeasible, objective, water managers currently invest in managing the impacts of flooding. Two types of impacts are triggered by floods: tangible impacts and intangible impacts. Tangible impacts can be easily monetized and are merely related to material damage like damage to houses, household goods, public and industrial buildings, and agricultural goods. Intangible impacts cannot be directly expressed in monetary terms. The main category of intangible impacts are social impacts, like mental and physical health impacts, disruption in time spending, the loss of irreplaceable items, the financial disruption of a household, the difficulties in recovery and the impoverishment of the neighbourhood. Other intangible impacts are ecological impacts, typically the loss of biodiversity.
The reduction of social flood risks is the focus of this paper. Since social flood impacts are experienced at the level of the individual and the households, a micro-scale analysis is compelled by using an object oriented procedure. In addition, the new approach requests an interdisciplinary approach of scientists from hydraulic and social sciences, as is the case for this research team. A methodology is developed in order to evaluate the social consequences of flooding based on the micro-scale analysis of the three-pillar social flood risk reduction strategy. The first pillar refers to the weakening of flood characteristics. The second pillar includes the restriction of the exposure of

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vulnerable elements-at-risk. And the last pillar contains the increase of the adaptive capacity of the society.

2. ADAPTATION TO SOCIAL FLOOD RISKS

The social flood impacts are the main focus of this paper. These impacts appear at the moment that human settlement is coming into contact with flooding, and in particular when this flooding makes losses on houses, factories and public infrastructures. This occurrence may start a chain of reactions, since social impacts are interrelated to each other. Hereafter, a non exhaustive list of these potential social flood impacts that may occur during or following of a flood event is presented.
First of all, the most severe social flood impact is the loss of life, which is compared to a direct damage to the human body. Less severe, but more frequently occurring impacts are physical health (e.g. colds, skin irritation, exhaustion…) (Tapsell et al. 2002; Lamothe et al. 2005) and mental health impacts (e.g. stress, anxiety and depression …) (Tapsell and Tunstall 2001). After the flooding, the recovery process is difficult for communities that have faced the inundation. In particular for those who rent houses and those who are not covered by insurances. But for insured people as well, recovery may be stressful, because of the negotiations with insurers and loss adjusters. (Tapsell and Tunstall 2001) In addition, flood damage may have an impact on the financial situation of the household in two ways. Firstly, flood may cause in income cut due to days off from business interruption. Secondly, spending might increase due to rehabilitation after flooding (e.g. cleaning-up, repairing, health care costs…) (Twigger-Ross 2005; Tapsell and Tunstall 2001; Walker et al. 2006). People's attitude towards policymakers may make a 180 degree turn. This event may result in distrust regarding the governing authority and the loss of confidence in existing flood protection, forecasting and support provision. (Lamothe et al. 2005; Tapsell et al. 2002) However, it might be this distrust that triggers the positive behaviour of precautionary measures to protect one-selves (Grothmann and Reusswig 2006). A last social impact to mention here is the impoverishment of the neighbourhood. The dirt in the street, the damage to public infrastructure may create material impoverishment. But it is the empty houses, and the migration of people out of the flood risk area that create social impoverishment. People feel insecure and anxious about the new-comers what may affect the social cohesion of the community (Tapsell et al. 2002).
From this reviewing of the potential social flood impacts, the three pillars, or driving forces of the impacts evaluation methodology can be discerned. They are expressed in Equation 1 (based on Hilhorst 2004):

$$SI = \frac{F * E * V}{AC} \qquad (1),$$

SI : social flood impacts
F : flood characteristic
E : exposure of elements-at-risk
V : vulnerability of people
AC : adaptive capacity of the society

Firstly, the flood characteristics: the water level, the water rise, the velocity and duration of the flooding, influence social flood impacts. Secondly, exposure of elements-at-risk to flooding: people living near river banks are more likely to experience social flood impacts than people living outside the flood risk area. And thirdly, the vulnerability of the people-at-risk and the capacity that is available in the society to adapt to the floods. Certain population groups have more difficulties in coping with flooding than others. On the other hand, society has organised several measures to support people during flooding, which is called adaptive capacity. These three driving forces are clarified more in the next chapters. Thus, social flood impacts can be limited by one or more strategies. The first strategy is to weaken the flood characteristics, for instance by the construction of natural or artificial controlled flood areas, dikes and walls, green roofs and wells. These measures are most of the time promoted and implemented by water managers. In addition, exposure of elements-at-risk might be limited. These measures are related to land use and in particular implemented by spatial planners. A third strategy to reduce social flood risks is to increase the adaptive capacity of the society. Multiple measures are useful, but none of them can stand alone. For instance, flood proofing of buildings, the communication of the flood risks in order to raise awareness, financial support to reimburse the flood damage organised by private insurance

companies or by the government. In addition, an emergency plan may increase the adaptive capacity when well organised and exercised.

3. INONDATION MODELLING

As mentioned before, inundation flow characteristics are the first driving force of the social impact evaluation. These parameters are direct outputs of the hydraulic modelling. Prior to running numerical simulation, the flow boundary conditions have to be set by means of a hydrologic analysis of the catchment response. This study also leads to the associated probability of a given discharge based on the statistical analysis of the river flow time series (usually expressed in hydrology in terms of a return period in year).
It is shown that interactions between the main channel and the floodplain of rivers during flooding are important and cannot be neglected (Mcmillan and Brasington 2008). Therefore, the assumptions of one dimensional model are not acceptable in many cases involving complex floodplain geometries in particular in urbanized areas. Therefore, since very accurate Light Detection And Ranging – LiDAR topographic data, which is an airborne laser remote sensing tool characterized by a horizontal resolution of 1 by 1 meters grid size and an altitude precision of 15 centimetres, are available, the present study is based on a detailed two-dimensional flow model. This model is included in the modelling system *WOLF* developed at the University of Liège. *WOLF 2D* is based on the depth-averaged equations of volume and momentum conservation, i.e. the "shallow-water" equations (Chaudhry 1993):

- $$\frac{\partial \vec{s}}{\partial t}+\frac{\partial \vec{f}}{\partial x}+\frac{\partial \vec{g}}{\partial y}+\frac{\partial \vec{f}_{\mathrm{d}}}{\partial x}+\frac{\partial \vec{g}_{\mathrm{d}}}{\partial y}=\vec{S}_0-\vec{S}_{\mathrm{f}} \quad (2)$$

with $\vec{s}=\left[h \quad hu \quad hv\right]^T$ the vector of the conservative unknowns. $\vec{S}_0$ and $\vec{S}_{\mathrm{f}}$ designate respectively the bottom slope term and the friction term:

$$\vec{S}_0=-gh\left[0 \quad \partial z_{\mathrm{b}}/\partial x \quad \partial z_{\mathrm{b}}/\partial y\right]^{\mathrm{T}} \quad (3), \qquad \vec{S}_{\mathrm{f}}=\left[0 \quad \tau_{\mathrm{b},x}/\rho \quad \tau_{\mathrm{b},y}/\rho\right]^{\mathrm{T}} \quad (4)$$

$\vec{f}$ and $\vec{g}$ represent the advective and pressure fluxes in directions x and y, while $\vec{f}_{\mathrm{d}}$ and $\vec{g}_{\mathrm{d}}$ are the diffusive fluxes. The following notations have been used: t represents the time, x and y the space coordinates, h the water depth, u and v the depth-averaged velocity components, z_{b} the bottom elevation, ρ the density of water, τ_{bx} and τ_{by} the bottom shear stresses. The bottom friction is conventionally modelled thanks to an empirical law, such as the Manning formula. The models enable the definition of a spatially distributed roughness coefficient. The internal friction may be reproduced by different turbulence closures included in the modelling system, such as simple algebraic ones but also a complete depth-averaged k-ε model.
The space discretization is performed by means of a finite volume scheme. Variable reconstruction at cells interfaces is performed linearly, in combination with slope limiting, leading to a second-order spatial accuracy. The advective fluxes are computed by a Flux Vector Splitting (FVS) technique developed by the HACH. A Von Neumann stability analysis has demonstrated the stability of this FVS (Dewals 2006). The diffusive fluxes are legitimately evaluated by means of a centred scheme. Since the model is applied to compute steady-state solutions, the time integration is performed by means of a 3-step first order accurate Runge-Kutta algorithm, providing adequate dissipation in time. The time step is constrained by the Courant-Friedrichs-Levy (CFL) condition based on gravity waves. A semi-implicit treatment of the bottom friction term is used, without requiring additional computational costs.
The model has been extensively validated and has shown its efficiency for numerous practical applications, (Dewals et al. 2006; Dewals et al. 2008; Erpicum et al. 2009a; Erpicum et al. 2009b; Roger et al. 2009) notably flood modelling (Dewals et al. 2008)..

4. EXPOSURE ANALYSIS

The second driving force of the social flood impact evaluation is the exposure of the elements-at risk. Typically, the exposure is carried out by combining the inundation extent and land use database. In order to keep consistency between the 2D numerical flow modelling performed on a 2m Cartesian grid and the risk evaluation process, the developed tool handles several sources of highly accurate Geographic Information Systems – GIS as shown in Table 3 (Ernst et al. 2008).

Table 2. Summary of the spatial information.

Data	*Format*	*Geometric quality*	*Semantic quality*	*Features used in the risk analysis*	*Availability*
LiDAR	Raster	+	-	Accurate elevation (DSM)	Floodplain of main river
Top10v-GIS	Vector	+	+	Very rich land use data	Everywhere
PICC	Vector	+	+	Eg. cornice height, single buildings	Not in rural area
Land registry	Vector	-/+	+	Mainly economic information	Privacy conditions
Statistical data	Vector	-	+	Mainly socio-economic data	Everywhere

For increasing the reliability of the hydraulic modelling, it is performed based on a DSM which includes obstacles relevant to the flow including the "over grounded" elements-at-risk. Furthermore, the present social risk analysis is based on an object oriented process, requiring that the flood parameters need to be determined for each element-at-risk, i.e. house, industrial building…, threatened by the flood. In this context, specific features are developed in order to compute the flood parameters inside these assets. Either the average value of the flood parameters in the neighbouring cells is assigned to the element-at-risk, or the free surface and the ground level are linearly interpolated inside the asset by using a least square method and subsequently averaged (Ernst et al. 2008).

Once the elements-at-risk are identified and the flood parameters computed, the flood characteristic index expressed in Equation (5) (Coninx and Bachus 2008) can be evaluated:

$$F = 0.45*WD + 0.22*WR + 0.22*V + 0.11*D \quad (5),$$

WD : water depth score V : velocity score
WR : water rise score D : duration score

Each of these scores is binary valued based on a threshold value. For example, in the case of the water depth score, the threshold value is 0.3m. Thus, if the water depth is lower than 0,3m the variable is 0 and 1 otherwise. For the velocity, the threshold value is 2 ms^{-1}, 12 hours for the duration and considering the water speed rising, there is no numerical threshold, but the rising have to be considered as slow or sudden. Slow means that the rising is faster than people warning delay. Since the numerical simulations are run using the steady state assumption, the two last flood characteristics, i.e. water rising and duration of the flood, may not be established based on the flow modelling. But regarding the gauging station measurements, the flood events on river Ourthe can be considered as slow and long. In this context, the WR and D scores will remain respectively 0 and 1 in the following of this paper.

5. SOCIAL VULNERABILITY AND ADAPTIVE CAPACITY OF SOCIETY

5.1 Social vulnerability of people and adaptive capacity of society: concepts

The last driving forces are the vulnerability of some population groups and their capacity to adapt to flood events. They are connected to each other in the way that increased adaptive capacity will reduce the actual vulnerability of the people.

Vulnerable people are individuals that are unable to cope with the effects of flooding due to social economic, political and cultural attributes. Empirical researches (Tapsell et al. 2002; Cutter et al. 2003) shows that some people categories have more difficulties in handling the flood event (cf. § 0). The reasons are that they have a limited awareness of the risk, a lack of information and knowledge of the risk or restricted access to political power and social patterns (Aysan 1993).

Adaptive capacity refers to *mechanisms to alleviate or contain the impact* (Thywissen 2006). To be effective, these mechanisms have to be organised at society level such as social networks, economic

mechanisms (e.g. savings or available governmental budget), institutional mechanisms (e.g. collective action to reduce vulnerability), technological and non-technological mechanisms that prevent or reduce disaster impact (Brouwer et al. 2007).

5.2 Assessment methodology

5.2.1 Vulnerability of people

Theoretically, it would be best to measure vulnerability at the level of houses. However, due to privacy arguments, data is rarely available at this level. Therefore, we are limited to measure vulnerability at the level of districts, which is the lowest level of available data. Given this limitation, social vulnerability is measured by a composed index that indicates the proportion of vulnerable people in a certain area. The selected indicators are proxies of the vulnerable social groups, selected on literature review (Tapsell et al. 2002; Cutter et al. 2003).
Normalization or standardization are not necessary, because every indicator refers to data on the proportion of people and are expressed in percentages. Due to lack of scientific evidence on the interrelationships between the indicators, it is opted to weight them equally. The present analysis takes into account the ratio of (i) elderly, (ii) ill people, (iii) lone parents, (iv) foreigners, (v) financially deprived people, (vi) people living in one-storey houses. The indicators are aggregated by the geometric mean since the variables are non-comparable and ratio scale (Ebert and Welsh 2004 ; Böhringer and Jochem 2006).

$$I(Y) = \prod_{i=1}^{n} (1 - X_i)^{W_i} \quad W_i > 0 \text{ for } i = 1,...n \qquad (6),$$

$$I(S) = 1 - I(Y)$$

X = social vulnerability indicator;
$I(Y)$ = non-social vulnerability index;
$I(S)$ = social vulnerability index

Data on the vulnerability of people in case study Esneux is coming from the most recent population census, which is 2001 (NIS). Another bottleneck is that data on ill people and people living in one-storey houses is currently missing at the district level and therefore, these indicators are not yet included in this research. The indices are ranked by using fixed margins. Areas with indices between 0 – 0.06 are resilient, 0.06 – 0.13 are at risk of social vulnerability, 0.13 and 0.19 are socially vulnerable, 0.2 or more are called extremely socially vulnerable areas.

5.2.2 Adaptive capacity of the society

In contrast to vulnerability research, the search for indicators to make adaptive capacity operational is rather new and only few systematic methods seem to approach adaptive capacity. To meet this gap, the sociologists of the research team have developed an analytical framework for adaptive capacity towards floods. From literature reviewing, policy documents and in collaboration with public officers, several indicators have been identified in order to evaluate the adaptive capacity. They are classified in eight variables: technological and non-technological measures, availability and distribution of resources, institutional structure and capacity, social capital, risk spreading instruments, information, public perception. By scoring the indicators, a total adaptive capacity score is received.

6. SOCIAL FLOOD IMPACTS AND SOCIAL FLOOD RISKS

Given the intangibility, social flood impacts are hard to quantify separately. Therefore, the use of a figure that symbolises the level of severity of the social flood impacts and that enables relative comparison between areas seems to be appropriated. In this way, high priority areas for social flood risk reduction are disclosed. To estimate the social flood impacts for those people who are exposed

to floods, the aspects are combined in Equation 7 (Coninx and Bachus 2008), while Table 4 illustrates what is the meaning of low, medium or high social impact.

$$SI = 0,5 * F + 0,25 * V + 0,25 * AC \quad (7),$$

SI : Social impact V = vulnerability
F : flood characteristics AC = adaptive capacity

Table 3.Social indices value: flood characteristics, vulnerability and adaptive capacity

Flood characteristics index	Vulnerability index	Adaptive capacity index
Cf. Equation (5)	Resilient category = 0	Low adaptive capacity = 1
	Category at risk of social vulnerability = 0.33	Medium adaptive capacity = 0.5
	Category socially vulnerable = 0.66	High adaptive capacity = 0
	Category extremely socially vulnerable = 1	

Table 4. Meaning of the social flood impact levels.

Low social impact: 0-0.33	Medium social impact: 0.33-0.66	High social impact: 0.66-1
Easily recovered	Recovery takes some days to weeks	Difficult recovery, may even takes months
Short term stress	Stress may remain for some weeks	Trauma, panic attacks, anxiety, anger
Colds/coughs/headaches	Injuries, skin irritations	Hypothermia, heart problems
No evacuation necessary	Might be evacuation of vulnerable people	Evacuation of affected people
Limited loss of time	Loss of time	Loss of time is large
No migration flow	Most worried people may leave	Migration flow
No impoverishment of neighbourhood	Impoverishment may occur	Areas can be impoverished for a while

The social flood impacts are expressed as the number of people that are living in low, medium or high social flood impact areas. The aim of social flood risk management is firstly to reduce the number of affected people in general and secondly to reduce the severity of the impact.

7. CASE STUDY

7.1 Description

The case study is located in the lower part of river Ourthe, the basin of which has a catchment area of about 2900 km^2 at the downstream limit of the case study. Two towns are located along the studied part of the river Ourthe: the town of Esneux and the town of Tilff. Concerning the hydraulic modelling, the case study is located in the Ardennes massif, where the valleys are quite narrow and the water storage in the inundated floodplain is very low. Thus, the steady state assumption is valid for the river Ourthe. With the purpose of reducing the computational time, this assumption will be used for the modelled flood events. The risk analysis will be applied on an area that covers three reaches of the river Ourthe located respectively 18.5 km (reach n°1), 12.5km (reach n°2) and 10 km (reach n°3) upstream of the mouth of river Ourthe into river Meuse (Belgium). The total length of the simulated reaches is about 16 km with a computational Cartesian grid of 2 by 2 meters leading to $8{,}2.10^5$ computational cells. For this case study, the most significant factors influencing vulnerability are the high proportion of single parents and deprived people.

7.2 Adaptation measure

During the 19th century, the idea of an inland navigation canal building that should have linked the Meuse and the Rhine basins was developed. But, only some reaches of this huge engineering project were indeed constructed. The constructed parts of the canal were used to enable inland navigation and, as far as the reaches along river Ourthe are concerned, they were intensively exploited until the beginning of the 20th century.
Today, some remains of the canal, notably some locks, are still visible. Field surveys have been dedicated to take census of all the remains still visible today in order to identify the ancient course and an approximated cross section of the ancient canal. Using the collected information, the Digital Surface Model has been updated taking into account the canal rehabilitated, considered as a flood

protection measure increasing the overall cross-section of the river by 18%. Figure 1 shows cross-sections of river Ourthe just downstream of the town Tilff. Notice the protection wall (cf. Figure 1 b.), designed for a 100-year flood (876 m^3s^{-1}), the effectiveness of which is discussed below.

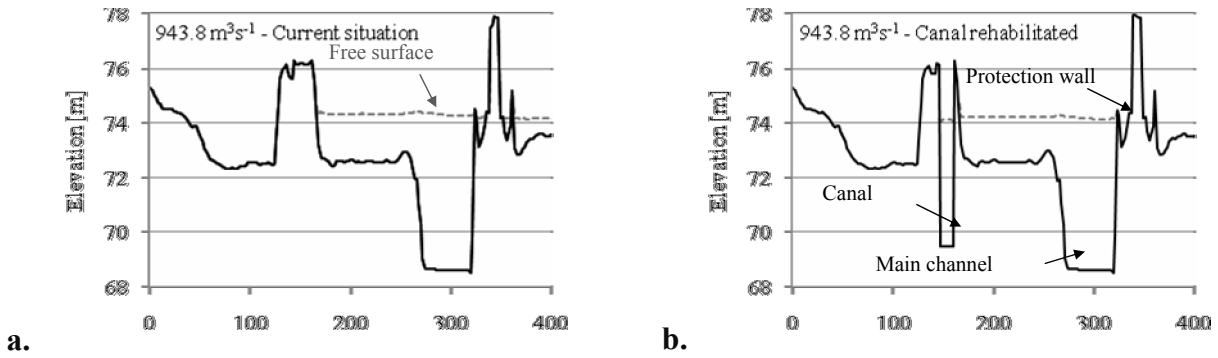


Figure 1. Cross section (a. present situation, b. canal rehabilitated).

7. 3 Risk analysis

Figure 2 a. shows the results of the social risk analysis with the rehabilitation of the ancient canal (continued curves) and the current situation (dotted curves). The light grey curves plot the number of people affected by the flood in the low social risk class, the black one is related to medium risk and there are no affected people in the higher risk class. This figure also shows the effectiveness of another non-structural adaptation measure (cf. Figure 2 b.), namely the increasing of the adaptive capacity of the people threatened by the flood. In this context, it is assumed that the adaptive capacity of people facing inundation increase from medium to high.

First, the analysis of the total number of people threatened by the flood reveals that for the interval between 876m^3s^{-1} and about 970m^3s^{-1} the effectiveness of the mitigation measure increases sharply. This trend is related to the protection wall located downstream of Tilff for the protection of a residential district. Thanks to the canal rehabilitation, this wall initially designed for a 100-year flood becomes effective up to a 250-year flood. Outside this interval, the number of people affected by flooding is reduced by no more than about 20 to 40. The number of people affected in the lowest class of social impact is rather constant; the influence of this adaptation measure is quite small.

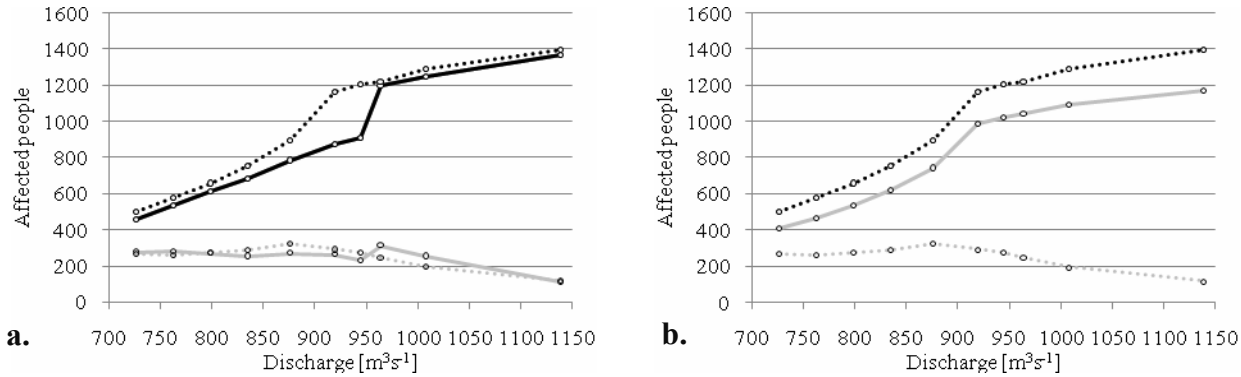


Figure 2. Number of affected people with and without adaption measure (a. rehabilitation of the ancient canal, b. increasing of the adaptive capacity).

Secondly, the graph b. in Figure 2 shows the influence of the social impact of an increase of the adaptive capacity of people threatened by flood. This measure might be put in place by perfecting a emergency planning for example. It leads to high changes in the distribution inside social impact classes. But since this non-structural measure does not influence the flow, the total number of people affected remains the same.

The general conclusion of the social risk analysis is that the effectiveness of the first mitigation measure is mainly tangible for a limited range of discharges between 876m^3s^{-1} and about 970m^3s.

The second mitigation measure seems to be very interesting thanks to the reduction of affected people in the medium social impact class. As both measures play a different part in the social impact reduction and without considering the cost of both mitigation measures, it would be interesting to combine measures in order to add the effects of the structural and non structural them, even if the cost of the rehabilitation of the ancient canal should not be underestimated within the framework of an overall cost-benefit analysis.

8. CONCLUSION

The procedure presented in this paper constitutes a genuine risk modelling chain handling the whole flow process from statistical analysis of river time series to the social risk evaluation including the flow modelling. The risk modelling system also provides practical support for selecting and designing the most appropriate protection measures.

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FLOOD DAMAGE ESTIMATION BASED ON FLOOD SIMULATION SCENARIOS AND A GIS PLATFORM

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Abstract: The objective of this paper is to improve the insight in the economic estimation of direct damage due to flooding by analysing the direct flood damage to built-up areas and agricultural areas. Depth-damage functions are constructed in a stepwise fashion that allows them to be as fined or descriptive as required. The paper presents also a tool, developed in GIS environment that uses the locally developed depth-damage functions for the estimation of direct flood damage. The usefulness of the tool is demonstrated by estimating expected flood damage over several land-use categories under specified flood simulation scenarios in a case study located in Erasinos Basin within Eastern Attica Prefecture. The visualisation of the analysis results in GIS environment allows the identification of flood-prone areas and provides an indication of the expected spatial distribution of damages for the specified flood scenarios. These findings aim to support policy- and decision- making in the context of flood risk management, spatial planning, and further economic development.

Key words: Depth-damage functions, flood damage estimation, land-use categories, GIS-based tool, Eastern Attica Prefecture

INTRODUCTION

The range of consequences or rather damages that a flood brings about includes economic, political, social, psychological, ecological and environmental damages and damages to cultural heritage. There is a substantial body of international literature that provides evidence of extensive expertise in the field of damage estimation. However, experts and academics still disagree about the methods and models to be applied.

First of all, numerous definitions of damage exist. The categorization of damage into direct (related to physical contact of water) and indirect, or into tangible (quantified in monetary terms) and intangible is commonplace, but interpretations and delineations of categories differ (Jonkman et al. 2008). Secondly, various approaches exist regarding the damage appraisal, such as financial and economic valuation based on market values (i.e. based on historical values or replacement values), while variation in the scale of analysis (micro-, meso- or macro-scale) is also found (Messner et al. 2007). Today the typical approach is the economic estimation of direct damage, mostly by applying depth-damage functions. An integrated, unifying approach is, however, missing. For consistent decision making it is desirable to have a more or less standardized approach for damage estimation at least at the higher aggregation levels, such as a river basin or a complete region.

With this in mind, this paper aims to improve the insight in the economic estimation of direct flood damage to residential and industrial built-up areas and to several agricultural land-use types. This is done based on a dataset that contains detailed information of local economic data for the area under study and historical records of flood direct damage estimates occurred in the recent past. This study aims to contribute to a stronger empirical basis of flood damage analyses.

The paper is structured in six sections. Section 2 gives a short presentation of the selected case study area. Section 3 describes in summary the derivation of the flood simulation results under specific flood scenarios. Section 4 focuses on the development of the local depth-damage functions

whereas in Section 5 the automatic calculation and visualisation of the damage estimates is illustrated by applying the developed local depth-damage functions at a GIS-based tool. Finally, some concluding remarks and recommendations are given in Section 6.

CASE STUDY

The selected case study is located within Erasinos basin (A=208 km^2), in Eastern Attica Prefecture. It comprises of two main tributaries, Ag. Georgios and Erasinos streams and a buffer zone of 1 km width alongside the streams (see Figure 1). The chosen area is characterised by rapid urban development and population rate boost and simultaneously it includes different types of landscape (woodland, cultivated land-mainly olive trees and vineyards-, wetland and areas of cultural interest). Above of all it is described as a flood-prone area due to the inadequate drainage network filled with human interventions. Figure 1 gives an overall view of Erasinos basin and the under study tributaries where the Digital Terrain Model (DTM) and the digitised Land Use map for the buffer zone are also illustrated.

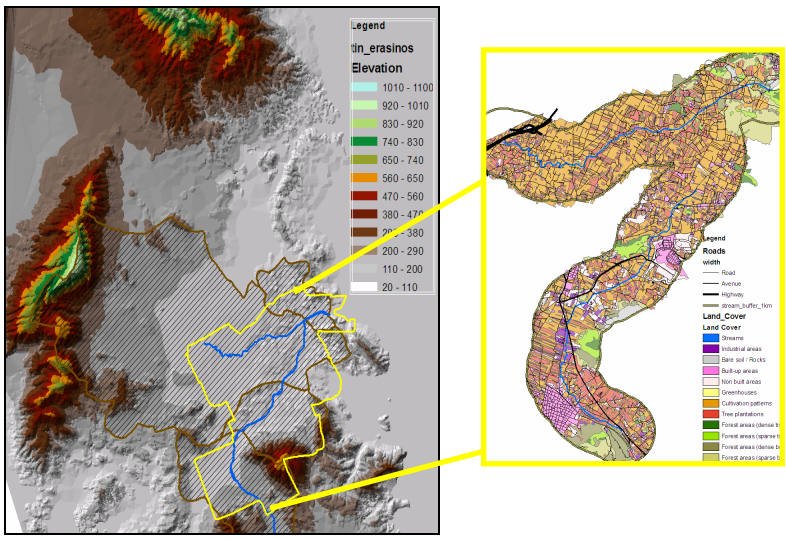


Figure 1. Digital Terrain Model and digitised Land Use map for the case study within Erasinos basin

Further details about the derivation of the DTM, the Land Use map and the socio-economic state of the area under study can be found in the technical report of DISMA (2007) and the published work of Tsakiris et al. (2008).

FLOOD SCENARIOS AND FLOOD DEPTH SIMULATION

3.1 Flood Scenarios Derivation

It is common practice to select a set of standardized flood scenarios corresponding to certain exceedance probabilities in order to perform flood damage estimation. According to the new EU Flood Directive recommendations (2007/60/EU) the flood scenarios are prepared for a 0.2-0.1 exceedance probability as a high probability scenario while medium- and low-probability floods could correspond to 0.02-0.01 and 0.002-0.001 exceedance probability and return periods of 50-100 and 500-1000 years respectively. However, these three scenarios are not sufficient to describe fully the probability range of flooding (Tsakiris et al. 2009). For a more comprehensive description some intermediate exceedance probabilities might also be of importance such as 0.04. These supplementary scenarios can be formulated in case more information on flood statistics is required (e.g. in case of rational estimation of design discharge of flood protection or river training works). It

should be mentioned that normally flood insurance involves categorization of flood-prone areas in accordance with their flooding potential. According to FEMA (1993) in the US two return periods are used for characterizing flood hazard areas, the 100-year and the 500-year return periods. Areas within the 100-year boundary are characterized as "special flood hazard areas" and areas between 100-year and 500-year boundaries are characterized as "areas of moderate flood hazard".

However since the work presented here focuses on the development of local depth-damage functions, the chosen flood scenarios should be indicative of the most usual type of flooding conditions of the area under study. The selected flood scenarios are:

- a 0.05 exceedance probability (or return period T=20yrs) as a high probability scenario based on the local history of flood events
- a 0.01 exceedance probability (or return period T=100yrs) as a medium probability scenario to indicate a 'special flood hazard area' boundary
- a 0.001 exceedance probability (or return period T=1000yrs) as a low probability scenario to indicate the worst case scenario

3.2 Hydrologic and Hydraulic modelling

The simulation of the flood-prone area along the main tributaries under the specified flood scenarios was performed in HEC-RAS software v. 3.1.3. The procedure implies that natural channels meet uniform flow conditions that the energy grade is approximately equal to the average channel bed slope, and that water surface elevations can be obtained from a normal-depth calculation. These assumptions are conservative in most natural channels. The hydrologic/hydraulic computations involved are:

- Design flood hydrograph derivation for each sub-basin by using the method of Synthetic Triangular Unit Hydrograph
- Steady flow hydraulic computations alongside Ag.Georgios and Erasinos streams (upper and lower reach) by using HEC-RAS software

After completing the river hydraulics model, HEC-RAS simulation run results were exported for processing inundation mapping in ArcMap environment by using the HEC-GeoRAS tool. Floodplain boundary and inundation depth data sets were created from exported cross-sectional water surface elevations and then they were converted to depth grid maps. This action was performed through a layer setup dialog window where the grid cell size was determined and set for all the analyses equal to 20. The definition of the grid cell size was based on the resolution of the Digital Terrain Model used. Hence the flood simulation runs were generated over a 20m x 20m cell grid under three flood scenarios: (1) T= 20yrs (2) T=100yrs and (3) T=1000yrs. Analytical description of the usage of HEC-GeoRAS tool for inundation depth mapping in ArcMap environment is given in HEC-GeoRAS user's manual (2005).

DEPTH-DAMAGE FUNCTIONS

4.1 General

The central idea in the traditional approach for direct flood damage estimation in monetary terms is the concept of depth-damage functions or loss functions. These functions relate flood depth with the extent of economic damage that usually is the maximum possible damage in the flood prone area. Depth-damage curves which were first proposed in the USA in the 1960s are currently internationally accepted as the standard approach to assessing urban flood damage (Smith 1996).

Usually, depth-damage functions include water depth as the only determinant of direct damage. However flood damage is influenced by many more factors among which are flow velocity, flood duration, sediment concentration, lead time and information content of flood warning, and the

quality of external response in a flood situation (Penning-Rowsell 1977; Smith 1996; USACE 1996; Kelman and Spence 2004). The above factors are, though, scarcely included in flood damage models. Moreover, varying temporal and spatial scales may be applied in practice when modelling flood damage and economic damage estimates may be associated with considerable uncertainty (Merz et al. 2004; Pistrika and Jonkman 2009).

In general, the estimation of direct damages involves two interrelated steps. The first is the estimation of physical damages to objects, such as buildings (Kelman and Spence 2004), while the second step is the 'pricing' of these physical damages. The procedure for the estimation of direct physical damage is visualised in Figure 2 and involves: (1) determination of flood characteristics; (2) assembling information on land use and maximum damage amounts; (3) application of depth-damage functions.

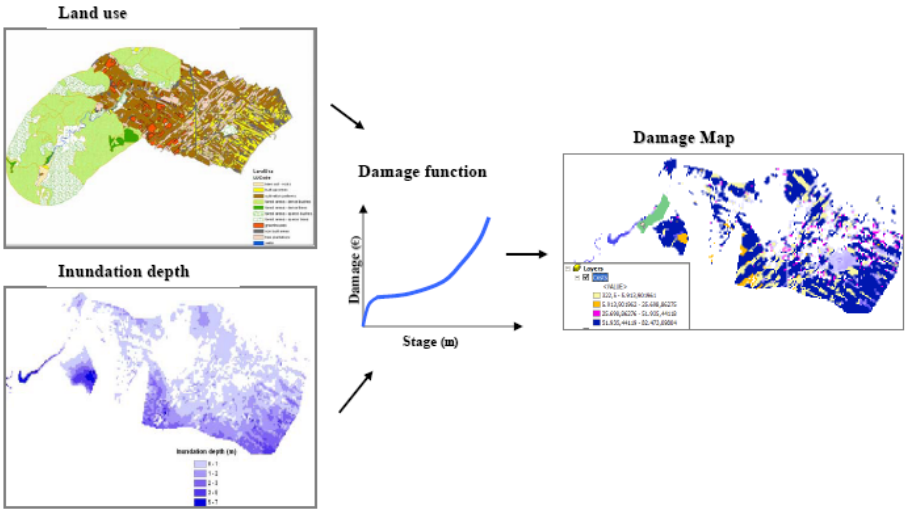


Figure 2. Schematization of the procedure for estimating the direct damage due to flooding (based on DISMA, 2007)

The following paragraph refers to the development of the local depth-damage functions and the pricing of the physical damages to the following land-use categories: (1) the built-up areas and (2) the agricultural areas. Based on international literature and by assembling local information the derived depth-damage functions developed for these land use types are presented in detail in the following paragraph.

4.2 Structuring the development of depth-damage functions

Since flood depth is the result of a flood simulation generated over a 20m x 20m cell grid, the smallest possible spatial scale of damage analysis is the 20m x 20m cell size. According to recent studies (Merz et al. 2004; Thieken et al. 2008; Pistrika and Jonkman 2009) the smaller the spatial unit is the poorer the relationship between flood depth and damage resulted. At a high level of detail and for a limited sample size of buildings the variations in building damage may be considerable. If the sample size becomes larger variations are averaged out. Consequently, in the context of the use of depth-damage functions they may be useful to estimate damage for larger areas with many buildings (Pistrika and Jonkman 2009). Keeping this in mind, the damage analysis here for built-up areas, is performed at a higher scale than the one-structure scale.

Damage is calculated as a percentage of the maximum possible damaged property value. The depreciated value of the replacement/reconstruction cost (in €/m^2) is used as the maximum possible damaged property value. The replacement/reconstruction cost values are derived from the available economic data about reimbursements due to flood damage to buildings of the Ministry of Structural Works and Environment. This study of local depth-damage functions for built-up areas took into consideration the work of Clausen (1989) and Black (1975) for estimation of potential flood damage to brick and masonry homes. Also, for the estimation to building's contents it was used the concept of the exponent 'a' to parameterise the depth-damage function according to building type.

This concept was first developed in Switzerland in a project called 'AFORISM' (1996) that involved a significant study on flood damages to buildings.

Finally a detailed survey was conducted by the author on the historical records of flood damages to buildings that occurred in the recent past in the broader area of Attica Prefecture. Three major flood events, occurred in 2002, in 2005 and in 2007 within Attica Prefecture had kept records of flood damage to buildings and thus these were taken into consideration for the development of the local depth-damage functions. It should be mentioned that the public access to these records was extremely limited due to strict laws of violation of data of private character. However studying the site inspections of the damaged buildings immediately after the event, it was concluded that the process of flood direct damage estimation was purely empirical and it resulted in a rough classification of the structural damage into two groups: (1) the inundation damage that allowed for reimbursement only for fixing the house and (2) the major structural damage / total destruction that allowed for reimbursement for reconstructing the house.

Given all the above considerations, three sets of local depth-damage functions for built-up areas were developed to estimate each of the following type of damage:

- structural damage to single-family, two-storey residence, made of concrete walls with average quality of structure
- damage to contents of single-family residence
- structural damage to industrial, one-storey building, made of concrete walls and iron roof with average quality of structure

Figure 3 shows the set of functions developed for the estimation of structural damage to single-family, two-storey residence.

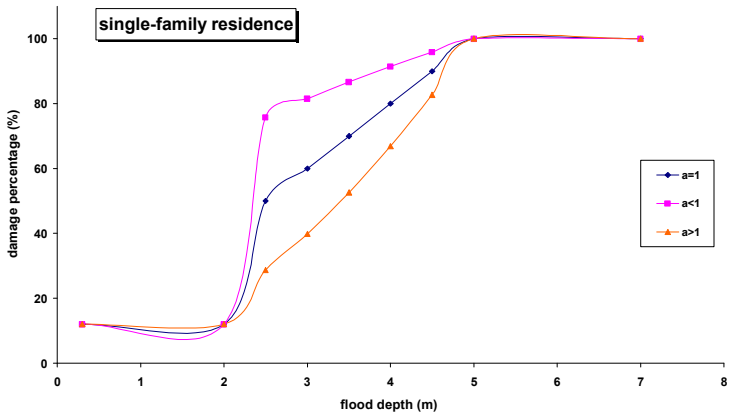


Figure 3: Depth- damage functions for structural damage estimation to single-family residence

The flood related damage to agricultural areas within the case study resulted in the development of local depth-damage functions for the following crop types:

- areas under trees (e.g. fig trees)
- olive trees
- vineyards
- garden-farming areas (complex cultivation patterns)

In general the damage functions for crops do not depend primarily on flood depth. Damage to crops depends mainly on when the flood occurs and the duration of flooding. Losses are estimated based on the area of inundation versus total area of crop land and the subsequent reduction in output, investment, and income. However, the damage functions for agricultural loss developed here do not include any variable to account for flooding duration since the area under study experienced so far only short term floods – flash floods (less than one day inundation). Also, it is assumed for all crop types that the growing season occurs during the flooding. Hence the damage estimation for agricultural loss is assumed as time independent.

The development of depth-damage functions is mostly empirical and is strongly based on flood – related damages to agriculture that happened within the area under study in the past. The flood damage data for agricultural loss were collected from the Ministry of Agricultural Development. The maximum possible economic damage refers to the replacement cost of the agriculture product and it is taken to be equal to the yield production achieved per land area multiplied by the market value of the crop type. There is no consideration for the harvest cost and other expenses and therefore the maximum yield loss in monetary values refers only to the replacement cost.

To identify the crop yield we used the crop inventory of the Ministry of Agricultural Development by calculating the average yield for the last 40 years. To identify the market value of each crop type and to capture the annual fluctuations in pricing for each crop type we used the average market value over the last five-year prices from the same inventory. Table 1 shows the maximum values of crop yield estimated for each of the aforementioned crop types.

Table 1. Maximum values of crop yield in € / m^2

Land Use Category	Market value of crop (€ / kg)	Crop yield (kg / 1000 m^2)	Market value of crop (€ / m^2)
Areas under trees	2.07	150	0.25
Olive trees	2.24	58	0.13
Vineyards	0.50	1145	0.57
Garden farming areas	0.65	2000	1.30

A GIS-BASED TOOL FOR FLOOD DIRECT DAMAGE

5.1 Theoretical Background

A GIS-based tool for direct flood damage estimation was developed by the Centre for the Assessment of Natural Hazards and Proactive Planning of National Technical University of Athens under the auspices of the EU Programme INTERREG IIIC-Sud Initiative and the Regional Operation Framework of NOÉ Programme-sub project DISMA. The tool is an ESRI ArcMap extension that provides flood damage estimates in monetary terms for pre-specified types of land uses, given a set of ArcGIS raster layers that contain the required information.

The tool achieves to automate the calculation of direct flood damage estimation by applying the aforementioned depth-damage functions. Depending on known land-use categories, such functions will return the estimated monetary damage per area unit (e.g. one square metre). The monetary estimation in M€ comes as a result of a damage percentage that depends on a given flood depth, over the total maximum possible damage in monetary units for the type of the given area. In addition the depth-damage functions are imported in a stepwise fashion through a simple and intuitive interface. Hence the tool can be used to define new (improved) or alter existing damage functions so as to allow for convenient future development.

The main output of the tool is an estimated break-down of damages, depending on the input flood-map and land-use layers that the user has previously chosen. Furthermore a visual projection of the estimated damages in ArcMap environment is feasible so that the user can easily identify the areas where the most damage is estimated to occur. Further details about the technical characteristics of the tool can be found in the technical report of sub-project DISMA (DISMA 2007).

5.2 Implementation and Results

The derivation of inundation depth maps under specified flood scenarios allowed the estimation of expected flood damage over the area covered by the Digital Terrain Model extent. For every

flood scenario the inundation depth map is integrated with the land use map in ArcMap environment and by applying the damage estimation tool (described in the previous paragraph) the direct flood damage is computed in M€ over the land-use categories for which depth-damage functions were derived.

Table 2 gives the estimated break-down of damages under every flood scenario for the area under study. It is observed that the cost for flood damage only to the structure of residential built-up areas covers about 75-80% of the total cost estimated over all land-use categories under each flood scenario. Hence structuring the depth-damage function for residential built-up areas is crucial for the reliability of the damage results. Furthermore the damage figures of Table 2 verify the fact that the smaller the exceedance probability gets the higher the expected flood damage becomes.

Finally the visualisation of the analysis results in ArcMap environment provides an indication of the expected spatial distribution of damages and thus it allows the identification of flood-prone areas. This finding aims to support policy- and decision- making in the context of flood risk management and spatial planning. For instance Figure 4 illustrates partially the visual projection of the damage analysis results for residential built up areas (both structural damage and content) under the flood scenario of T=1000yrs.

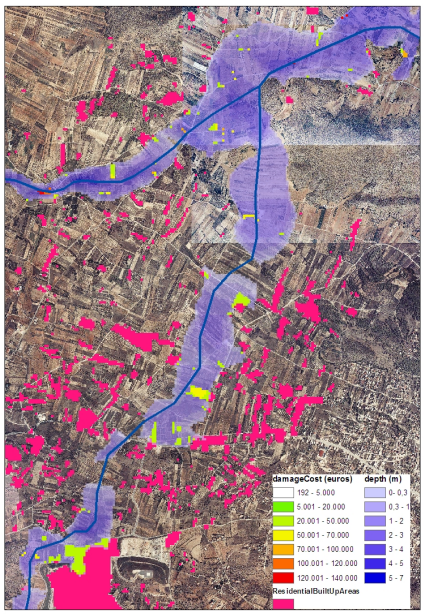


Table 2. Flood damage figures in M€ for every land-use category under every flood scenario

Land Use Category	Flood scenario 1 (T=20 yrs)	Flood scenario 2 (T=100 yrs)	Flood scenario 3 (T=1000 yrs)
Residential built-up areas (structural)	9.96	11.70	19.54
Residential built-up areas (content)	0.94	1.16	1.92
Industrial built-up areas	0.32	0.38	0.46
Areas under trees	0.01	0.02	0.05
Olive trees	0	0	0.01
Vineyards	0.08	0.14	0.32
Garden farming areas	2.11	2.24	2.66
Total (in M€)	**13.42**	**15.64**	**24.96**

Figure 4. Visualisation of damage analysis results for scenario T=1000yrs

CONCLUSIONS AND RECOMMENDATIONS

In this paper the relationship between flood depth and economic damage to built-up areas and agricultural areas has been investigated for the flooding of Erasinos basin under specified flood scenarios. The study develops local depth-damage functions for the above mentioned land-use categories in an attempt to contribute to achieving a standard approach for expected flood damage estimation. In the first part of this paper the flood depth simulation is presented and analyzed. The second part focuses on the analyses of local depth-damage functions for a) built-up areas and b) agricultural areas. Depth-damage functions are constructed in a stepwise fashion that allows them to be as fined or descriptive as required. This is done based on a dataset that contains detailed local information for the area under study and historical records of flood direct damage estimates occurred in the recent past. The main findings can be summarized as follows:

- Structural damage to residential built-up areas is distinguished empirically into two groups: (1) the inundation damage for flood depth less than two meters and (2) major structural/total destruction for flood depth more than four meters. For flood depth values between two to four

meters, building resistance is crucial and therefore the economic estimation of damage presupposes site inspection of the damaged building.
- Damage to contents of residential built-up areas is parameterized by the exponent 'a' so as to account for luxury, average or poor type of residence.
- Analysis results showed that flood damage to built-up areas covers nearly 80% of the total damage estimation.
- Inundation depth may not be a crucial factor for the economic estimation of physical damage to agricultural areas due to flooding.
- The spatial level of detail of the analysis is a determining factor for the correlation between predictions and observations. At a high level of detail and for a limited sample size of buildings the variations in building damage may be considerable. If the sample size becomes larger variations are averaged out. This also implies that any depth-damage functions should not be used at a too high level of spatial detail, e.g. for individual structures.

Based on this study the following recommendations are made:
- Further investigation of the adequate spatial aggregation level of damage analysis in relation to the variations in the data that were used for the derivation of the stage damage functions.
- It is recommended to collect further information about the variation in building types, structures, and materials. For damage to building structure a division of the damage data according to building types (timber structure, masonry, concrete buildings etc.) may lead to better results.

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FLOODS IN MEDIUM WATERSHEDS OF MOUNTAIN REGIONS

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Abstract: The paper analyses the characteristics of floods occurred in medium watersheds of the mountain regions: causes, effects and structural/non-structural measures aimed at reducing the floods negative effects. Floods represent the most frequent disaster, which can occur on different scales – from large rivers to small rivers – affecting the state of the environment, including here economic damage, ecological lack of balance and human casualties. Current statistics show an increase in the number of natural disasters (especially storms and floods) occurred in 2007, which was not, in fact, a different year from this point of view. Romania is one of the most exposed countries to natural disasters, especially to floods, which have lately caused significant damage. Thus, only in 2005, floods caused damage of over 1.5 billion EUROS. The paper also presents a case study – the flood occurred in March 2007 in Iza river watershed, located in the North of Romania, in the mountain region.

Key words: causes, effects, floods, watershed, measures.

1. INTRODUCTION

During the last years, despite the progresses we made in the field of scientific research and risk forecast, we cannot but notice a worldwide increase in the frequency of occurrence of catastrophic hydrological events, resulting in serious material damage and the loss of human lives. These undesired phenomena are often seen to be a consequence of natural events (magnetic storms, solar eruptions, El Niño phenomena) undoubtedly coupled with reckless human interventions, which led to changes of balance within the elements of the natural environment (massive land/woods clearing, without reforestation; the expansion of urban agglomeration, having as a result the waterproofing of an increasing number of areas; the building of dams, the watercourse regulation and the building of dykes; the drainage works, the excessive exploitation of water resources and, last but not least, the burning of combustible and the occurrence of the green house effect).

In order to prevent these catastrophic hydrological phenomena, experts are trying to determine, as exactly as possible, the relations between the natural and the antrophical elements within hydrographical basins, the ways in which they interact, as well as the way in which they are influencing overland and ground water flow. (Cocoş 2007)

Experts are also trying to develop complex models of simulation of the dangerous hydrological phenomena triggering mechanisms of and their effects on the environment.

Romania is one of most exposed countries to natural catastrophes, especially to floods, which have caused substantial damage during the last years. What currently occurs on the territory of Romania, and we are referring here to the two categories of hydrological phenomena – floods and droughts – is, on the one hand, the consequence of the global climatic changes on the regional and planetary level, and, on the second, of the antrophical intervention on the specific landscape.

The studies carried out by the ***Intergovernmental*** Panel on ***Climate Change*** and the European Environment Agency have shown that the air average temperature on the Terra surface has grown by 0,6°C, during the last 100 years, that this growth has been three times more rapid after 1975, on

the one hand, and that the quantity of atmospheric precipitations has grown by 10-20%, on territory of Romania (just like in other regions of Europe), on the other.

In Romania, due to these global modifications, the increase in the number of floods in small and medium watersheds has been caused by the uncontrolled deforestations carried out after 1990, which determine a rapid overland flow on the hillslopes, soil erosion and landslide intensification, minor streambed silting and by houses and another construction in areas with a high flooding risk.

2. CAUSES OF FLOODING

An important characteristic of floods in the small and medium watersheds of the mountain regions is that they occur quite fast, without allowing one the possibility to forecast them in useful time, unlike the floods in the plain regions, where one can have a rather good forecast about the water levels and discharges, respectively the flood wave propagation times.

Flood formation in small and medium watersheds is conditioned by the processes occurred on the hillslopes and on the deep erosion formations (ravines, torrents), while floods occurring in the downstream of close sections of these watersheds are not only conditioned by the maximum water discharge generated by the heavy rainfall, but also by the local transport capacity of the streambed in the areas of the localities and of the social and economic objectives.

A. **The flood formation triggering factors** are: heavy rainfall of lesser duration than the overland flow concentration time; short concentration time (a few hours); small dimension of the watershed.

B. **The elements favouring the occurrence of floods** are the geographical and the physical characteristics of the watershed and the hydrographical network: watershed area; watershed form; hillslope slopes; ***longitudinal slope*** of the main ***stream bed***; density of the drainage network; afforestation level; land use; soil texture, respectively geology, in cases where the rock is at surface; soil column water storage capacity. The geographical and physical characteristics influence: the overland flow dimension (production function); the velocity of trickling concentration in the hydrographical network (transfer function); flood wave motion in the downstream (propagation function). The production function is strongly dependent on the soil texture and the land use (here included the afforestation level), which eventually determine the storage capacity of the unsaturated areas. The hillslope slopes and the average slope of the watershed, its form, the drainage network density, the hydrographical network component slopes influence the propagation and the transfer functions.

C. Flood aggravating factors:

- natural factors: initial humidity of the watershed soil; friability of rocks; existence of deep erosion formations (ravines, torrents)
- antrophical factors: absence of soil erosion control and torrent correction measures; excessive deforestations, associated with the non-observance of norms on forest cutting or wood waste storage; inadequate agricultural practices; building of constructions and storing places in the immediate vicinity of the water course banks.

The absence of soil erosion control and torrent arrangement works contribute to the growth of torrentiality and solid (sediment) transport. Irrational deforestations and inadequate agriculture practices (absence of terraces, plough-land in the direction of the biggest slope etc.) result in the growth of runoff factors, with direct consequences on both the volume and the maximum discharge level of the flood, and the sediment transport. (Drobot 2007)

In order to determine the overland flow levels, in time, there have been developed more one- and two-dimensional flow models, which take into consideration, to different extents, the above mentioned characteristics. The application and the solving of these model equations are usually based on a series of simplification hypotheses, due to the large number of the involved parameters and the diversity of the ***laws*** governing ***their variation***. For the same reasons, it has not been possible, by now, to develop a three - dimensional model, which could render with higher accuracy this phenomenon. (Beilicci 2006)

An easy-to-apply model, merely in the case of small hydrographical basins (under 10 km^2), is the one which calculates the overland water levels generated by precipitations along the following relation: (Bâcov et al. 1988)

$$Q = i \cdot 0{,}001 \cdot S \cdot 10000 \cdot k/(24 \cdot 3600) \quad (1)$$

where Q – overland flow level (m^3/s); S – watershed area (ha); i – rain intensity (mm/day); k - runoff factor, defined by the relation:

$$k = h_S/h < 1 \quad (2)$$

in which h_s is the overland flow depth, generated by a heavy rainfall of h height.

The overland flow formula (1) can also be applied to hydrographical basins of 10...100 km^2, where the concentration time exceeds 60 minutes, but it supposes the introduction of a reducing coefficient (m), due to the decrease in rain intensity $i^{(1-m)}$, where m = 0.3.....0.5 for mountain and high hill areas and m = 0.5....0.7 for lower hill areas.

For larger watersheds, showing a higher density of the hydrographical network, it is necessary to resort to a division into smaller basins, relating to each valley (ravines, torrents, water courses), and to introduce the corresponding precipitations of different associating times and, subsequently, to compose the resulting hydrographs according to each basin, taking into consideration the fact that the heavy rainfalls do not cover the entire surface of the watershed, and that they do not have the same intensity everywhere. (Lazăr 2007)

3. EFECTS OF FLOODING

Floods occurring in downstream of the close sections of small and medium watersheds are not only determined by the maximum water discharge resulting from the overland flow on hillslopes after a heavy rainfall, but also by the streambed transport capacity. In mountain depressions or at the end of mountain areas, where there have been developed human settlements, the river longitudinal slope is rather reduced, which determines the decrease in the flow velocity as compared with the upstream hydrographical network.

The deposition of sediments in these areas is worsening the flow conditions and increasing the water levels. This phenomenon is much intensified in flooding periods, characterized by important water and sediment discharges. The evacuation of the flood discharges is much difficult to carry out when the streambed transport capacity in bridge / footbridge / under or over crossing areas is diminished because the flow is blocked by the floating and dragging materials. In the upstream of these obstacles, a true lake is formed, a backwater slope, whose level may eventually reach a dangerous level. The increased level of the water behind the obstacle causes the flooding of the neighbouring populated areas; if the floating material created dam suddenly collapses, the resulting flood wave will have a great destructive potential in the downstream areas.

The sediment transport capacity of the river is also diminished in river elbow areas and crossection reducing points or in case of ice agglomerations. As a result, in the case of small and medium watersheds in which there have been formed human settlements, the maintenance of the streambed transit capacity of the river plays an important role in the prevention of floods. In this respect, all the constructions built in the immediately vicinity of river banks (sawmills, stables, household annexes, other constructions) or the materials deposited in such areas should be removed in order to avoid the worsening of the flowing of high waters.

At the same time, an important role in the occurrence of floods and the resulting damage in the populated areas play the methods of forest exploitation. After 1990, only massive wood has been presenting interest, while the other parts of the trees have been abandoned on the hillslopes or even in the brook beds. These wastes are trained during heavy rainfalls and they accumulate in the sections of bridges, elbows, crossection reduction points, or form a storage of 5 – 10 m height at the

confluence points. The storage of wood material recently exploited and not removed from the vicinity of river beds favours their transportation during flooding periods. Got in the valley, these trees exercise a very strong destructive effect on the downstream area or block, together with the wastes resulting from the forest exploitations, the flow sections of bridges and footbridges. (Drobot 2007)

We should also mention the danger presented by floods with respect to people and animals. An important characteristic of flows in the mountain riverbeds is that the water course usually presents a large longitudinal slope, the flow regime is a critical one, of a rather high flow velocity, and it is well known that a shallow flow (of 40 – 50 cm depth), yet of a sufficient high velocity, may endanger people's life, who once caught in the torrents are impossible or extremely difficult to save, also due to the floating materials which hit and hurt them.

4. MEASURES FOR THE PREVENTION / REDUCTION OF FLOOD EFFECTS

Preventive measures (for prevention, protection and preparation) are focused on the prevention/reduction of the flood generated potential damage through:

- Avoidance of the construction of houses, social, cultural and/or economic objectives in the potentially flooding zones, by mentioning in the land use planning documentations the data regarding the previous flood effects; adaptation of future developments to the flood risk conditions;
- Promotion of adequate use of land and forest and agricultural land;
- Achievement of structurally protection measures, inclusively in the areas of bridges and footbridges;
- Achievement of structural protection measures (control of minor streambed uses, elaboration of watershed plans for flood risk reduction and of measure plans, introduction of insurance systems etc.);
- Detailed identification, geographical delimitation of areas of natural flood risk on the territory of the administrative units, introduction of these areas in the general locality plans and provision in the locality planning regulations of the specific measures aimed at preventing and diminishing the flood risk, construction building and land use;
- Implementation of flood forecast, warning and alarm systems;
- Maintenance of the existent infrastructure aimed at assuring protection against floods and river streambeds;
- Execution of protection works against river streambed erosion in the areas of the existing bridges and footbridges;
- Communication with the population and its education as regards flood risk and the means of action in emergency situations. (Government Decision 2005)

During floods, the following expeditious measures are to be taken:

- Detection of potential flood wave and flood formation;
- Forecast of flood wave evolution and propagation along the water courses;
- Warning of the authorities and the population about the extension, severity and the occurrence time of floods;
- Organization of the authorities and the population and response actions to emergency situations;
- Assurance of (material, financial and human) resources at the county level for effective intervention;
- Activation of the operational institutions, mobilization of resources etc. (Government Decision 2005)

After the flood, the following measures should be taken:

- Help of the disaster-affected population, allowing satisfaction of their immediate necessities and their return to normal life;

- Reconstruction of the distressed buildings, infrastructure and those parts of the protection systems against floods;
- Review of the flood management activities in order to improve the intervention planning process, to cope with future similar events in the affected areas, as well as other areas. (Government Decision 2005).

In Romania, there are currently implemented or to be implemented three important projects aimed at reducing the flood impact on the environment:

1. SIMIN – Integrated Meteorological Information System – has been achieved and comprises four meteorological and synoptical stations located in the country so that they assure the representativity of the meteorological information all over the country. It has an infrastructure formed of meteorological radars, aerological observers, weather forecast centres, pluviometric stations, a ***meteorological satellite data collection*** and processing centre, lightning detection stations and a system of dissemination of the meteorological products.

2. DESWAT – Destructive Water Abatement and Control of Water Disaster – whose main objectives are to improve the forecast capacity, precision and speed, to implement the SIMIN project facilities, to evaluate the potential damage – costs, in case of floods.

3. WATMAN – Water Management – is a project aimed at implementing the national strategy of water management in case of disasters, elaborated by the Romanian Ministry of Environment and Water Management. It has six main objectives: achievement of the water management system for the water resource management infrastructure, implementation of the alarm system in floodable zones, design of a quick intervention system in flood and pollution periods, modernization of the communication system in the field of water management, creation of a decision support systems and integration of the meteorological system with the hydrological system and the water resource management system.

5. CONCLUSIONS

Considering the prevailing frequency of occurrence of floods in the medium surface watersheds of the mountain regions (at least in Romania), for areas of high flooding risk it is necessary that we make researches and studies that allow us to know the causes, the evolution and the effects of these phenomena on the environment. These researches require the achievement or the updating of topographical, geomorfological, climate and pedological studies regarding soil erosion, overland flow and other degradation, hydrological, hydrogeological, geotechnical processes, vegetation related and social / economic studies.

On the basis of these studies we can elaborate forecast models of flood evolution and effects on the examined areas. The development stages of such model can be the following:

- Elaboration of the aforementioned studies;
- Modulation of overland flow and soil erosion processes on hillslopes through one of the already known forecast models / programs (WEPP; RUSLE; CREAMS etc.), according to different hypotheses of land management and occurrence of heavy rainfall;
- Modulation of water and sediment / solid flow in hydrographic network beds (with DUFLOW; SMS; HEC or other programs) to forecast water discharge and levels in different sections along the water courses.

Based on the obtained results, we can establish economic and technical effective measures of flood control (soil erosion and overland flow control measures, hydrotechnical measures, structural and non-structural measures), which take into account the present situation and the perspectives of the areas and harmonize the interests of all the important relevant factors (human factor, environment, economic factor etc.).

We should also take into consideration the following aspects:

1. The use of GIS systems for forecast of flood in watersheds – is indispensable in the definition or the calculation of primary elements which occur in the different relations or mathematical models for the determination of flood maximum discharge or hydrograph.
2. The diversification of the site data acquisition and processing methods for increased precision of researches regarding the quantitative and qualitative modifications of the hydrographical space. Such methods are photogrametry and remote sensing, and they contribute to the permanent updating of data bases.
3. The permanent development of the used models of flood forecast, the creation of a complex forecast model, which satisfy all the conditions of a complex model.
4. The creation of multi-disciplinary teams for the modulation of hydrological, pedological and soil erosion processes, because the development of a model of the complex processes occurring on the hillslopes requires a compromise between the processes which are to be incorporated in the model and those which are known from the available data.
5. The necessity to develop a three - dimensional overland flow model, which should render more accurately the overland flow physical phenomenon and based on which we would realise the physical fundamental hydrologic model of the watershed.
6. The creation of representative experimental watersheds (small watersheds) which would allow us to study the causes, the ways of manifestation, and the effects of overland flows; at the same time, we should have the possibility to test the efficiency of the different prevention / reduction measures of floods effects. These experimental basins should be endowed with sufficiently dense networks of observance points of the evolution of such processes (solid and liquid discharges, water levels, evolution of water quality). (Beilicci 2006)

6. CASE STUDY – EXAMPLE OF A FLOOD IN A MEDIUM WATERSHED OF A MOUNTAIN REGION

Maramureş county (figure 1) is situated in the North-West of Romania, between 47020'00" and 48000'15" parallels, North latitude and between 22052'30" and 25007'30" meridians, East longitude; being neighboured by Satu Mare, Sălaj, Cluj, Bistriţa - Năsăud and Suceava counties. In the North it is bordered by the frontier line between Romania and Ukraine, Tisa river forming a natural borderline of a 62 km length.

Iza river drains the Depression of Maramureş, flowing parallel to the Vişeu river, but with a left asymmetry of the basin. It has an 83 km length and a 1303 km^2 watershed area, whereof 406 km^2 return to its main affluent Mara. The predominant land use is that of forest and pasture, with massive deforestations during the last years, without corresponding afforestation.

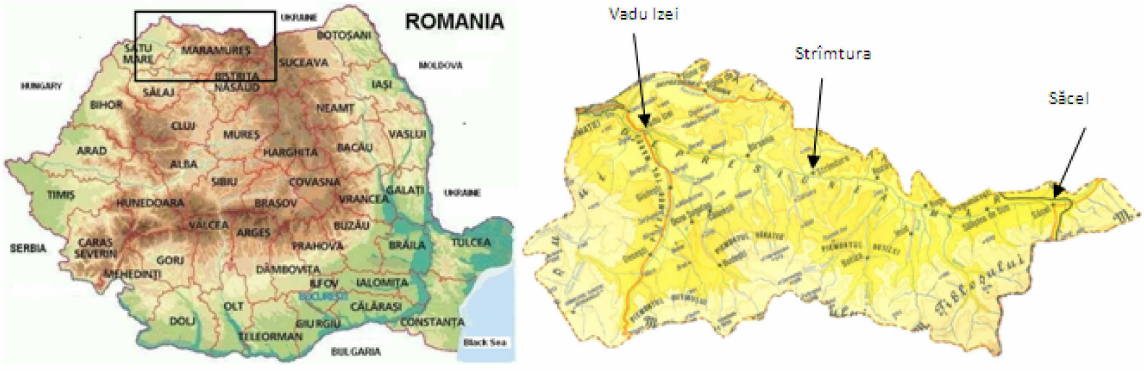


Figure 1. Iza river watershed

The largest flood waves are usually formed during the months of March – May, due to the simultaneous effects of heavy rainfalls and snow melting. Such flood wave occurred in March 2007.

The rainfall had an uneven extension in space, and, in order to determine the habitat, we used the records obtained from the three hydrometric stations situated on the river Iza banks: Săcel, Strîmtura and Vadu Izei (Săcel in the upstream and Vadu Izei in the downstream). In figures 2, 3 and 4 we can see the daily variation of the precipitations in 2007, and the flood hydrograph occurred on 4 March 2007 (water level hydrograph). (Dunca 2008)

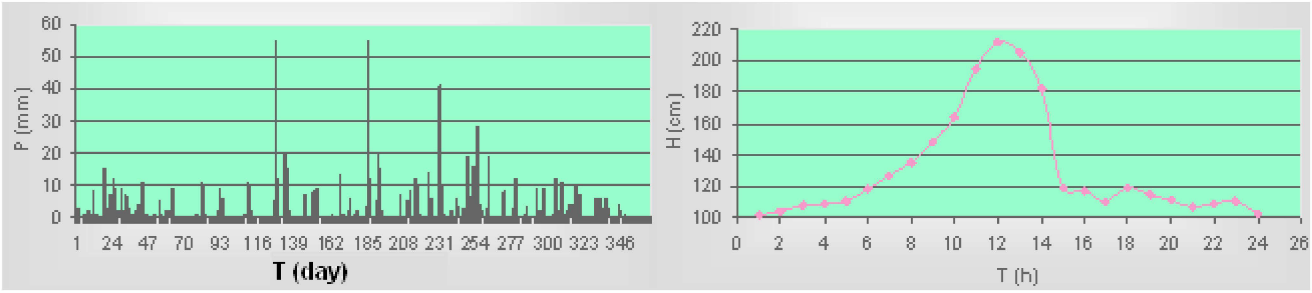


Figure 2. Daily variation of precipitations in 2007 and flood hydrograph on 4 March 2007 at the Săcel station

In Săcel village, the maximum water level of Iza river was of 211 cm, and the maximum water discharge 80 m^3/s, the total duration of the flood wave – 23 hours.

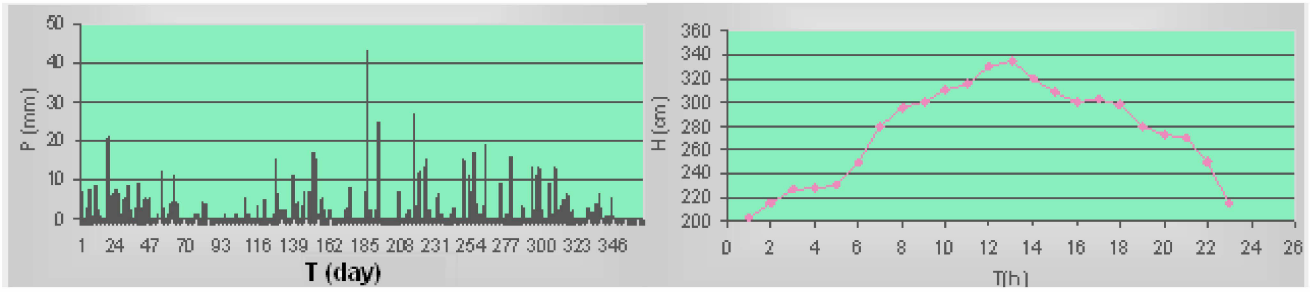


Figure 3. Daily variation of precipitations in 2007 and flood hydrograph on 4 March 2007 at the Strîmtura station

In Strîmtura village, the maximum water level of Iza river was of 335 cm, and the maximum water discharge 358 m^3/s, the total duration of the flood wave – 23 hours.

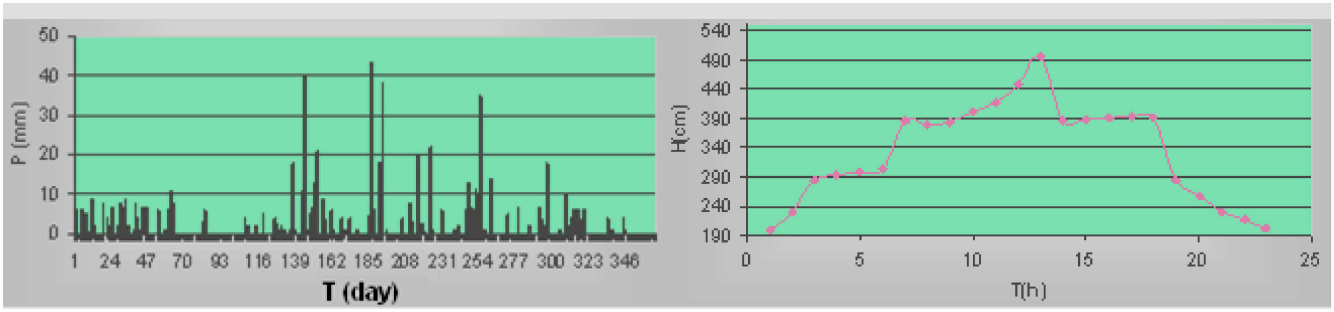


Figure 4. Daily variation of precipitations in 2007 and flood hydrograph on 4 March 2007 at the Vadu Izei station

In Vadu Izei village, the maximum water level of Iza river was of 496 cm, and the maximum water discharge 597 m^3/s, the total duration of the flood wave – 23 hours. Iza river has a multiannual average water discharge of 15,9 m^3/s.

The water dangerous levels were surpassed by 90 ....140 cm. The occurred flood took a large quantity of materials from the upstream or the river banks or from the inhabitants' households in the

vicinity of the river. The flood wave transported a 4.500 m^3 volume of sediments. By adding the volume of the brutish drag sediments, we obtained an average soil erosion rate of 6,55 m^3/an/ha in 2007.

Abundant precipitations caused significant floods in Săcel, Strîmtura, Vadu Izei villages, as well as in the other localities situated in the valley of Iza river. Other areas of county were also menaced by the fury of the waters. Until 18.00 o'clock, on 4 March, the most affected locality was Botiza village, where approximately 50 families were evacuated after their households had been awashed and the traffic on the county road was blocked. Around 18.30 o'clock, big problems also occurred in Săcel and Strîmtura villages (figure 5), where the water level increased by more than 100 cm in less than 10 minutes. Also, in Vadu Izei, the river went out of its streambed. This affected dozens of land hectares, broke roads and destroyed houses. In Maramureş, houses are usually built along the water courses, and when there comes a large flood wave, people are forced to leave their houses and flee the path of the water. Fortunately, there were no human casualties.

In order to prevent the future occurrence of such floods, there has been proposed the achievement of the following works in Iza river watershed: reforestations; soil erosion control works aimed at reducing the volume of overland flow and sediments; calibration and re-calibration of natural streambed; bank protection works; dykes and weirs. In Maramureş county, thanks to the FLOOD PREVENTION project, there were set up and are going to be put into service 16 hydrometric stations, 4 pluviometric stations with automatic transmission and 14 alarm stations. In the frame of same project work also hydrometeorological radar located on Igniş Mount. Within the DESWAT projects, there were set up 5 hydrometric stations with automatic transmission. The implementation of these information systems creates the possibility of warning the population in useful time about an imminent danger, which will result in mitigating damage and, undoubtedly, completely avoiding the loss of human lives. (Prefectura Maramureş 2008)



Figure 5. Flood on March 2007, Strîmtura villagge

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ESTIMATING ECONOMIC DAMAGE FROM FLOODS FOR THE RATIONALIZATION OF FLOOD PROTECTION DESIGN

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Abstract: The problem of rational flood design has gained priority an all recent studies in EU countries in the framework of the implementation of the Directive on floods 2007/60. The paper focuses on the construction of loss functions for estimating the economic damages in the flood plain under each scenario of a certain exceedance probability. Using simplified linear land cross sections of the inundated area loss functions are constructed for estimating the anticipated average annual economic damage that is the annualized flood risk. This figure is then compared with the annualized cost of flood protection works leading to the rationalization of flood protection works design.

Key words: Floods, economic damage, loss functions, annualized risk flood protection works, rational design.

1. INTRODUCTION

Floods are natural phenomena causing adverse conditions in the flood-prone areas by extensive inundation. The consequences of floods, which may be direct or indirect, include loss of lives, economic damages, damages on the environment and the cultural heritage. They can also influence the life and activities of large populations within and outside the flooded zone.

The objective of this paper is to devise a simplified methodology for the estimation of direct economic damages caused by river floods and the formulation of a procedure leading to the optimal design of flood protection works.

Floods are considered as extreme stochastic phenomena described by certain probability density functions. The magnitude of each flood scenario may be expressed by the economic consequences in the flood-prone area through a simple deterministic relationship based on one or two independent variables, such as the inundation depth or the flood wave velocity. (CANAH, 2007; Pistrika and Tsakiris)

In this paper the consequences of each scenario are then transformed to anticipated annualized risk values (including both the level of damages and their probability of occurrence).(Tsakiris, 2007)

Finally annualized flood risk values (R) are compared with the annualized cost of flood protection works (C). The minimum sum of both values (R+C) results in the optimal design of flood protection works.

The area in which the methodology was applied is the central part of the stream of Rafina (Attica- Greece) which is currently under study.

Rafina river basin is located in Attica-Greece. It has an area of 138 km2 and the main stream is 15 km long. Recently it was interconnected with Kifissos river basin through a tunnel of carrying capacity of 143 m3/s (T=50 years). Due to this additional discharge and the increasing pace of urbanization caused by the new airport of Athens El. Venizelos, the flood risk has increased substantially.

The proposed rational procedure is in line with the new EU-flood directive 2007/60, which emphasizes the role of rationality in flood protection measures from planning to construction. (EU, 2007)

2. FLOOD SCENARIO VS INUNDATION AREA

The study is focusing on the estimation of the anticipated mean annual direct economic damages caused by river floods in the flood plain. The proposed procedure consists of the following steps:

i. Estimation of flood discharge of certain return periods following an extreme type probability function.
ii. Estimation of inundated area and depth for each scenario (of a certain return period).
iii. Linking each flood scenario with the economic damages.
iv. Annualization of flood economic damages.

In this section the first two steps of the procedure will be analyzed whereas the steps (ii) and (iv) will be discussed in the next paragraph. The procedure will be better explained following the data of the application of the Rafina stream flood plain in Attica - Greece.

As known, annual maximum discharges may be represented by a certain extreme probability distribution e.g. EVI (Gumbel) or other suitable probability distribution. Using such a distribution, we first estimate its parameters and then we are able to determine the discharge (Qmax) which corresponds to a certain probability scenario that is a certain return period. In the case of the application for the purpose of this study, the scenarios selected correspond to return periods 2, 5, 10, 25, 50, 100 and 1000 years, respectively. The flood discharges of these return periods were calculated as 194, 295, 373, 467, 542, 614 and 876 m3/s, respectively. These discharges enter into the flat main section of the stream for a length of 11 km, which crosses the flood plain of the stream. Since no discharge data were available, the above discharges were generated by using rainfall data of the meteorological stations of the area which were transformed into stream flow using the software package HEC-HMS. (US Army Corps of Engineers, 2000)

In order to estimate the inundation area of both sides of the 11 km reach, several simplifications concerning the terrain of the riverine areas were assumed. Namely constant traverse slopes of 2 % were adopted creating a triangular cross-section. The inundation area, its width and the mean depth of inundation are presented in Table 1 as resulted by the application of another software package HEC-HMS.

Table 1. The characteristics of the inundation area for each flood scenario

Scenario	Return Period T (years)	Inundation Area W*L (10^3*m^2)	Width W (m)	Mean Depth h_{mean} (m)
1	2	1747	159	0.80
2	5	2046	186	0.93
3	10	2233	203	1.02
4	25	2431	221	1.11
5	50	2563	233	1.17
6	100	2695	245	1.23
7	1000	3080	280	1.40

It should be noted that the above figures refer to the do nothing scenario. That is no rectification work is considered. In case of designing a cross-section with a certain return period it is expected that all flood scenarios below the design scenario will produce zero inundation area. Only the flood scenarios with returns periods higher than the design scenario will produce inundation areas and further economic consequences as can be seen in Fig. 1.

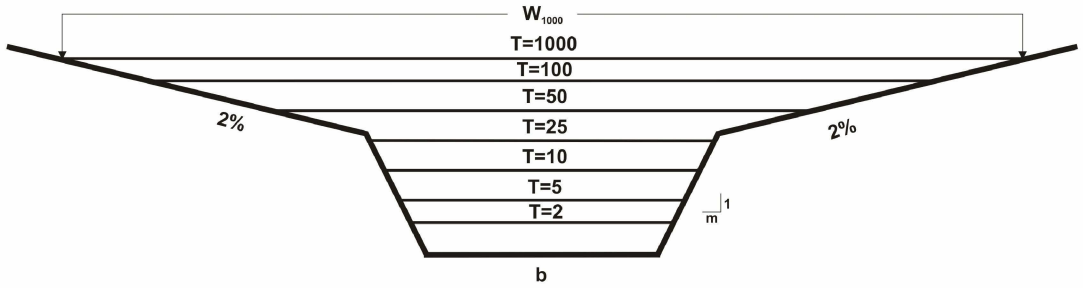


Fig. 1: Determination of Inundation areas

3. ANTICIPATED ECONOMIC DAMAGES

For the calculation of economic damage the maximum potential consequences (MPC) are firstly determined (Zentelis, 2001). The average MPC depends a) on the percentage of land uses, b) on the cost of reconstruction (CR) of existing items at risk and c) on the percentage of the cost of reconstruction which is anticipated to be needed due to the distraction caused by floods.

For the case study the average MPC was calculated as in the Table 2.

From the table above, the MPC is determined as 231,2 €/ha of land. Due to other anticipated damages to vehicles, tractors, etc., the total MPC becomes 254 €/ha. Due to the uncertainly of the assumptions adopted, it is logical to consider a range of MPC 200-300 €/ha.

However, these maximum economic damages, so called maximum potential consequences correspond to high inundation depths or alternatively to mean depths created by floods of very high return periods. In this paper, the critical inundation depth, above which MPC occurs, was postulated as 1 m. For inundation depths below, it is assumed that the damages are proportional to the inundation depth. That is, for mean inundation depth 0.60 m the anticipated economic damages are equal to the 60% of MPC.

Table 2: Calculation of the average maximum potential consequences (MPC).

	Percentage of land covered (%)	**Cost of reconstruction €/ha**	**Percentage of CR needed (%)**	**MPC**
Urban & industrial buildings	10	20000	10	200
Agricultural areas	84	60	50	25.2
Common utility areas (e.g. roads)	6	200	50	6
			Total	231.2 €/ha

Based on the above, the economic damages for each flood scenario (in which the return period exceeded the return period of the design scenario) are calculated by the following equation:

$$Z = MPC \cdot W \cdot F \cdot L \quad (1)$$

in which, Z is the economic damage for the entire inundation area for each flood scenario

W is the width of the inundation area

F is the ratio of mean depth over critical depth (e.g.1 m)

L is the total length of the reach of the stream (e.g.11 km)

For each design scenario the anticipated annualized economic damages are calculated as the sum of products of the mean economic damage for each probability class. The probability class is obtained by subtracting the cumulative exceedance probability of each couple of flood scenarios. If for, every example, we wish to obtain the probability class (Pi+1-Pi) between flood scenarios with T=5 and

T=10 years respectively, this is calculated as Pi+1-Pi = $\frac{1}{5}-\frac{1}{10}=0.10$. Since in the application example we use 7 flood scenarios, 6 probability classes are formed.

The final anticipated annual economic damage, which is in fact the annualized flood risk, is calculated for each design scenario by the following equation:

$$R(Z)=\sum_{i=1}^{n}\left(\frac{Z_i+Z_{i+1}}{2}\right)\cdot\left(P_{i+1}-P_i\right) \qquad (2)$$

in which Zi is the anticipated economic damage for the i-th flood scenario having exceedance probability Pi= $\frac{1}{T_i}$.

In the application example the annualized risk obtained for each design scenario using the latter equation, appears in Table 3.

Table 3: Annualized risk in M€/y for each design scenario.

Design Scenario (T years)	Do nothing	2	5	10	25	50	100	1000
Annualized Risk [R(Z), M€/y]	35.41	8.49	4.20	1.91	0.84	0.40	0.20	0.00

4. ANNUALIZED COST OF FLOOD PROTECTION SYSTEMS

The cost of engineering work comprises of the initial cost of construction and the annual cost of operation and maintenance. Since these cost categories are of different nature, discounting is needed for transforming initial costs (P) to annual costs (A) according to the well known “capital recovery” equation:

$$A=P\cdot\frac{i(1+i)^n}{(1+i)^n-1} \qquad (3)$$

in which i is the interest rate and
n is the economic life of the works in years.

By presenting all engineering and maintenance costs in annual figures it is possible to compare or add them with the annual anticipated economic losses caused by floods. Therefore, the optimal design can be the alternative which corresponds to the minimum sum of both annual cost of the engineering works and the annual anticipated economic damages (annualized risk) which are associated with this level of design.

It should be noted that for the transformation of initial costs to annual values, the economic life of the works and the interest rate for the cash flow operations should be determined based on reliable information from the country and region of the flood plain. For the application example, both the interest rate and the annual maintenance cost were represented by a unique composite interest rate equal to 4.5%, whereas the economic life of the protection works was assumed equal to 50 years.

For the application example the annual cost of construction and maintenance for each design scenario is presented in Table 4.

Table 4: Annual cost of protection works of each design scenario

Design Scenario	2	5	10	25	50	100	1000
Annual Cost of Protection Works	1.42	1.94	2.25	2.42	2.63	2.89	3.66

5. ANTICIPATED ECONOMIC DAMAGES

Finally the sums of annualized risk and the annual cost of protection works are obtained. The optimal solution corresponds to the minimum sum. For the application example this minimum sum is derived for the design return period of 50 years as can be seen in Figure 2.

It should be mentioned here that the result obtained is based on a number of assumptions most of which seem very uncertain. Therefore a type of sensitivity analysis is needed for investigating the range of expected values of the optimal design scenario. From a thorough analysis of all the factors involved two were examined namely the maximum potential consequences and the interest rate.

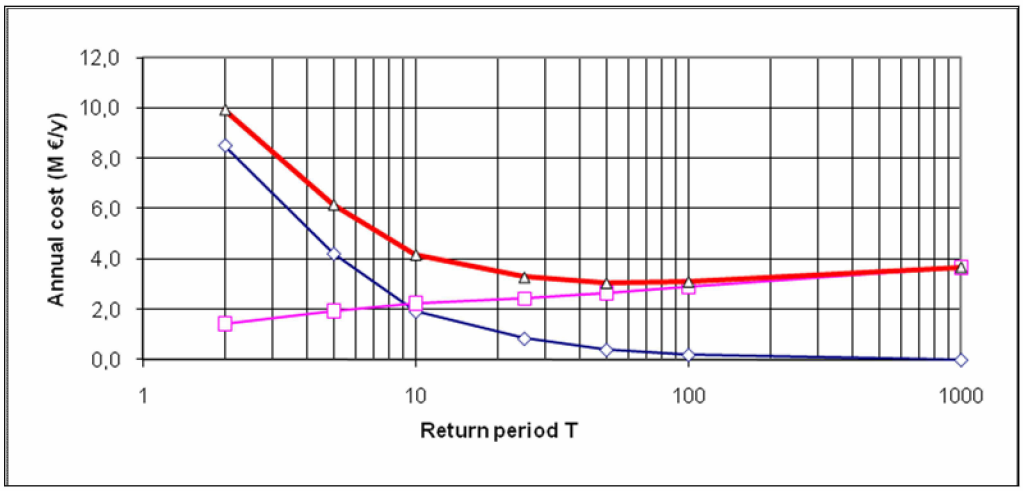


Figure 2: Determination of the optimal design

From this simple sensitivity analysis it was concluded that as the maximum potential consequences increase, the optimal design return period increases. As can be seen Table 5. In this table several values of maximum potential consequences are used to determine the minimum total annual cost, and the selected return period of the design flood discharge.

On the opposite as the interest rate increases the optimal return period decreases. However the results are not as sensitive as to the maximum potential consequences.

Table 5: Sensitivity of return period of design flood discharge in relation to the maximum potential consequences

Maximum potential Consequences (k€/ha)	Minimum Total Annual Cost (M€/y)	Return Period of Design Flood (y)
100	2.83	50
200	3.03	50
300	3.19	100
500	3.38	100
1000	3.66	1000

6. CONCLUSIONS

Apart from the reconnaissance and quick assessment of the economic consequences from flood events, achieved with the methodology presented in this study, some other important conclusions were derived:

1. The optimal design of river rectification works is selected for the stream which is analyzed. For the most probable future conditions the optimal design discharge is found to correspond to the 50-year flood.
2. As it was expected, the increase of the mean value of the maximum potential consequences from floods in the area leads to design floods of greater return periods.
3. Greater interest rates lead to more expensive flood protection works and therefore they lead to design floods of smaller return periods. However, the optimal solution is not sensitive towards the interest rate change.

ACKNOWLEDGEMENTS

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CYPRUS IN WATER CRISIS; THE ANATOMY

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Abstract: A water crisis like any crisis is not the result of a single event but as a consequence of a series of events initiated and executed by nature, the water managers and water professionals, the government itself and its ministers, the Parliament and the political parties, the NGO's, Government Authorities and Departments, the local and regional authorities (governmental or municipal), the consumers in general and the society and finally by act of war and international criminal acts of war. Cyprus prepared its water master plans and started its implementation in the early 70's. The water master plan provided for the satisfaction of the projected water demand up to the year 2005 from natural resources (including wastewater recycling) and from there on unconventional water resources should be sought. The master plan implementation was substantially completed by the 1990's, when it was discovered that the available water resources were by some 40% below the 1970's estimates, the water demand was much more than any updated estimates made between the 1970 and 1990 and the wastewater recycling projects were no implemented. It was in the 1990's when the Government decided to change the water development and management policy introducing desalination to augment the water available and to accelerate the implementation of the domestic wastewater recycling, introducing at the same time more strict water demand management measures. This paper will discuss why the water resources were reduced, why the water demand rose so rapidly and although at Governmental level decisions were taken to promote a revised water policy in which desalination and recycle wastewater were to play a vital role in avoiding a water crisis, not enough was done to avoid the water crisis that took place I the years 2007-2009.

Key words: Climate change; water shortage; water scarcity; integrated water management; water demand management; recycled water; seawater desalination.

1. PHYSIOGRAPHY, CLIMATE, ECONOMY AND LAND USE OF CYPRUS

Cyprus is an island in the North Eastern end of the Mediterranean Sea, with an area around 9251 square kilometers. The main features of the island are the big agricultural plain between the two mountain regions (Pentadaktylos in the north and Troodos in the centre) and the coastal areas situated around the island at elevations below 300 meters. Most of the precipitation falls on the Troodos Mountain, where the water resources originate and flow through the rivers to the plain and to the coastal areas. Most of the rivers, which flow only in winter, originate on the Troodos Mountain.

The climate is semi arid with an average annual precipitation around 461 mm, mainly falling in the winter and spring months and geographically varying from 250mm in the plain areas to 1200mm on the top of the high mountains. The evapo-transpiration varies from 1722mm/year in the low lying areas to 1243 mm/year in the areas above 300 meters, 75%, of which occurs in the months May to October (See Figure 1).

Cyprus is inhabited by more than 1.1 million people out of whom only 802,000 people are legal residents (the rest illegal colonizers and the occupying Turkish Army brought by Turkey after invading Cyprus in 1974) of which 68% live in urban areas and the remaining in the rural areas. Today's economic activity is centered on the services which contribute more than 75.6% to the GNP, the industry contributing 20% and the Agriculture contributing 4.4%. About 715,000 people

live in the Government controlled areas, where the remaining are living in the areas under occupation by the Turkish army, with 40,000 Turkish troops fully equipped.

From the total area of 9251 km2, some 45.830% of the area is agricultural land, 7.46% is Carob land, 18.96% is forest land, 8.94% is Scrub land, 11.89% is barren land and 6.92% is build up areas. From the Agricultural land 90% is dry or rained land, 6% is cultivated with annual crops (potatoes, onion, and vegetables) and the remaining 4% is cultivated with permanent crops mainly grapes, citrus, deciduous and olives. Agriculture consumes about 75% of the water resources.

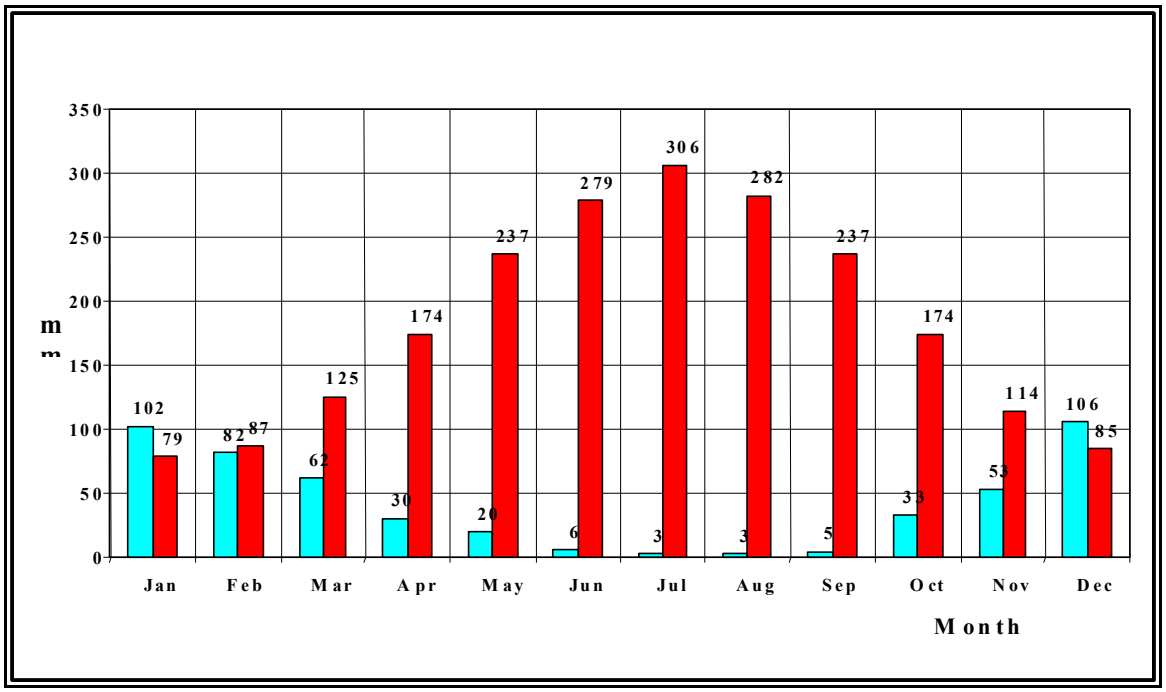


Figure 1. Average long term average monthly precipitation and evaporation.

2. WATER RESOURCES AND INDICES

The water resources of Cyprus (conventional and unconventional) in the areas under Government control, are very limited amounting to 406 MCM (or 568 m3/capita), made up from 235 MCM surface water, 135 MCM from groundwater, 30 MCM from desalination of seawater and 6 MCM from treated recycled domestic effluents. The water availability is 568 m3 per capita per year, much below the 1500 m3 per capita per year fixed by the United Nation as a minimum for unhindered development (See Figure 2 and Table 1).

Table 1. Estimated water resources by source for area under Government controlled.

No	Description	Unit	Quant	Remarks
1	Renewable water resources	Km3	0.370	All internal
1.1	Surface	Km3/yr	0.235	Excl. 48 MCM losses to sea
1.2	Groundwater	Km3/yr	0.135	Excl. 70 MCM losses to sea
2	Non Renewable water resources	Km3/yr	0.036	
2.1	Recycled domestic effluents	Km3/yr	0.006	
2.2	Desalination	Km3/yr	0.030	13 MCM in 1997 &17 in 2001
3	Total water resources	Km3/yr	0.406	

The water resources indices of Cyprus are relatively out of proportions indicating that drought cannot be effectively mitigated because the water availability index is just 568m3/capita (the United Nations minimum for unhindered development is 1500 m3/capita), the Competition index is

relatively high 1761 inhabitants per MCM per year, the Regularization index is relatively high 88%, which means that no additional water can be developed and the Independence index is 100% which means that no external water resources exist.

3. WATER MASTER PLAN

Cyprus prepared a water master for the development of its natural water resources to satisfy the projected water demand for a sustainable development. The Water Master Plan was prepared in the period 1968-1973 and was based on meteorological and hydrological data of the period 1900-1970 and the projections for water demand were based on the prevailing economic, social and environmental conditions at the time and the general development policy decisions. The Water Master Plan, which was completed in 1973 was envisaging for the development of all water natural resources surface and ground and the recycling of the wastewater of the Limassol town, for the satisfaction of the projected water demand with natural and recycled water up to the years 2000 to2005. From there on the water balance sheet appeared with a water deficit, for which non conventional water resources (seawater desalination or water imports) should be sought and made available to the water cycle. The Master Plan provided for the implementation of five major water projects namely, a) the Paphos Irrigation Project, b) the Khrysochou Project, c) the Morphou Tylliria Project, d) the Vasilikos-Pentaskhinos Project and e) the Southern Conveyor Project as shown on Figure 3. In 1976 after the Turkish invasion the Pitsilia Integrated Rural Development Project was added to the list and implemented in the years 1977-1982.

The implementation of the Water Master Plant started i973 and was completed in the late 1990's with five projects constructed except the Morphou Tylliria Project which was not implemented because of the Turkish invasion and occupation of the project area.

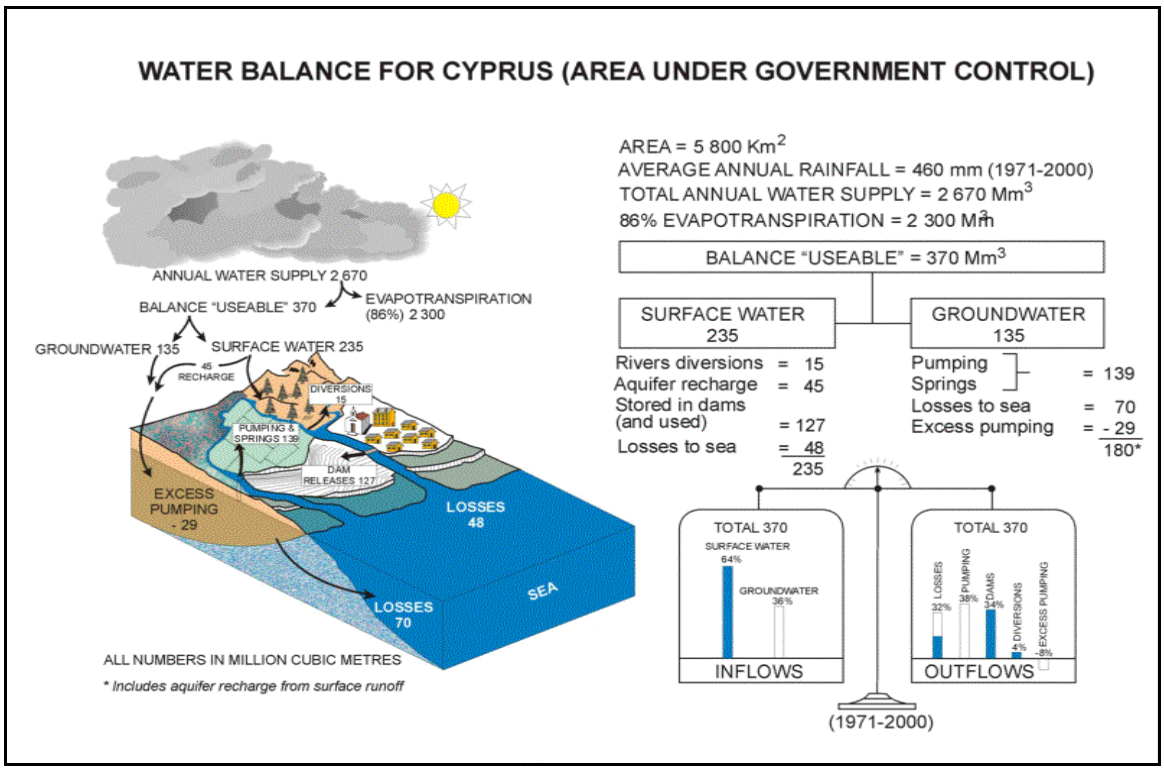


Figure 2. Hydrological cycle for Cyprus.

Table 2. Water resources indicators for Cyprus.

No	Description	Unit	Parameter	Remarks
1	Population	000´s	715	People in the Goverment Control area
2	Water wealth indicators			
2.1	Water resources index	M3/cap/yr	568	
2.2	Competition index	Inh/hm/yr	1761	
3	Constraints on exploitability			
3.1	Regularization index	%	91	Requlrized Quantity=171
3.2	Natural steadiness index	%	42	
3.3	Independence index	%	100	
3.4	Freedom of action index	%	0-100	

Definitions for data on Table 2
Water resources index = ratio of total renewable resources divided by the total population
Competition index = ratio of total population on total natural resources
Regularization index = ratio of total renewable resources divided by the total natural resources
Natural steadiness index = ratio of natural regular resources on the total renewable resources, here equal to the groundwater flow+ desalination and recycled
Independent index = ratio of natural internal runoff on the total renewable resources.
Freedom of action index = ratio of outflow to the sea or flow evaporated on total resources.



Figure 3. Major water projects.

4. WATER SHORTAGES AND WATER CRISIS

From the meteorological data analysis it appears that Cyprus is facing repeated meteorological droughts, of varying severity with precipitation reduction up to 50% of the average precipitation and duration periods 1-5 years. Because of these phenomena the big dams within the major projects were planned and constructed with capacities 2-2.5 times the average river flow, thus safeguarding over annual storage.

Although Cyprus suffered from severe droughts in the 1980's it was not until 1992 after the completion of the four major projects that it was realized that the available water resources from

the completed projects were not enough to satisfy the water demand. From the analysis of the measured inflows to the dams and the rate of growth of the water demand it was realized that the water balance sheet presented deficits that were due to lower water yields from the completed projects (surface and groundwater). It was also realized that the water demand growth was comparatively very high the one envisaged in the water master plan, which increased further the water shortage.

Since 1992 Cyprus is suffering from repeated water shortages, which resulted to (a) cuts of water supply for irrigation and domestic uses, (b) the change of the water policy t(c) he introduction of the seawater desalination, (d) import of water, (e) acceleration of wastewater recycling projects, (f) implementation of water saving and water demand measures etc.

This paper analyses the reasons why water supply was reduced, why water demand growth was higher than anticipated, why the water management was not carried out in accordance with the yields of the various water sources and why water development projects including seawater desalination projects were not implemented on time.

5. REASONS FOR REDUCTION OF WATER AVAILABILITY AND PROJECT WATER AVAILABILITIES

The water master plan which was prepared in 1973 was based on the meteorological data of the period 1900-1970, with an average annual precipitation around 541 mm per year. From 1972 right after completing the Water Master Plan a climate change started with the precipitation steadily decreasing and the temperature increasing steadily. Statistical analysis of the precipitation records available over the period of the hydrological years 1916/1917-1999/2000 shows a step change around 1970, with the mean precipitation of the period 1972-2000, reduced to 461 mm a fact that has as a consequence the significant reduction in the water available on the island. The decrease in the mean annual inflow to the dams referred to as design estimates, varies between 24% and 58%, with an average of around 40%, which means that Cyprus has roughly 40% less surface water than the amount estimated in the 1973 Water Master Plan. On the other hand the groundwater resources in Cyprus are overexploited by about 40% of sustainable extraction, resulting to depletion of aquifers increasing the risk of non adequate supply both for domestic and irrigation uses. This was verified by a Study carried out by the Water Development Department and the FAO in 2002 and published in a report named "Re-Assessment of the Water Resources and Demand of the Island of Cyprus".

As a result of the climate change and precipitation reduction the surface and groundwater resources are less on the average by 40% of those considered in the Cyprus Water Master Plan. For example the yield of the Integrated Southern Conveyor Project was estimated around 100 MCM per year which is now only around 60 MCM and decreasing, creating a deficit in the water balance of 40 MCM.

6. REASONS FOR WATER DEMAND GROWTH AT HIGHER RATES THAN THOSE OF THE MASTER PLAN

The 1973 Water Master Plan was prepare taking into consideration the planning parameters at the time, which were based on the social, economic, and environmental environments and the visions of the politicians at the time. The rate of growth of domestic and irrigation water demand was based on natural population growth rates and on population and labor availabilities. However after the commencement of construction of the water master plan projects certain events took place, which overturned all the above assumptions which either increase abruptly the water demand and/or

increase the rate of growth of water demand, resulting very quickly to the ceiling figures of water availability. The events that took place are the following.

a) Turkish invasion and ethnic cleansing: In 1974 Turkey, a NATO country of 70 million, invaded Cyprus and occupied 37% of its area, forcing some 200,000 people (of Greek origin) to leave their homes and properties and to seek refuge in the Government control areas in the south part of the island. At the same time it forced some 40,000 people (of Turkish origin), leaving in the areas under the Government Control areas to move to the area under occupation (north part of the island), as a pat of its plan to create two areas, one totally made of Greeks and another made of Turkish Cypriots and Turks from the mainland. The population in the Government controlled area increased by 160,000 people and the net population that was added to the water supply systems whose resources were from the Government control areas increased by 80,000 or by 20% increasing the domestic water demand by 25% and the irrigation by 5% assuming that around 10% of the refugee people had as their main occupation irrigated agriculture (people from Morphou, Lapithos, Karavas, Mesaoria etc). Assuming that the use of water to domestic and irrigation is 30% and 70% then the total increase in water demand is estimated at 9.5% of the total demand (6% from domestic increase and 3.5% from irrigation).

b) Accelerated tourist development: After the Turkish invasion and in its efforts to create more jobs for the 200,000 refugees, the Government accelerated the development of the tourist industry, which needed comparatively increased quantities of water. The tourist visit numbers increased by more than twice increasing from less than one million visitors to two and a half million visitors. Assuming that the stay of one visitor was around 11 days the 1.5 million additional visitors amounted to 45,000 equivalent people with high water demand, around 1.5 times the average local residents. This corresponded to a domestic water demand increase of 11% of the domestic water demand or 3.3% of the total water demand.

c) Abrupt increase of population due to influx of foreign labor force: Additional increase of the domestic water demand was caused by the increase of the population by approximately 5-10% by the influx of foreign labor force from the far and Middle East countries and most recently from countries of Eastern Europe. The additional water demand beyond the water master plan is calculated at 5% of the Domestic water demand or 1.5% of the total demand.

d) New style of development with increased water demand: Beginning in 2000 the housing development in Cyprus and the leaving style of the local and permanent tourist population had a turn with the design and construction of modern houses/villas equipped with swimming pools, irrigated grasses areas and other amenities with relatively high water demand. This added additional water demand to the existing water demand.

7. WATER DEFICITS IN THE YEARS 1990-2000

Based on the above the water demand increase beyond the water master plan estimates and provisions, amounted to 15% made of 9.5% from the Turkish invasion, 3.3% due to tourist development, 1.5 % due to the influx of foreign workers and 0.7% due to change of life style of the local permanent population. This combined with the decrease of the available water resources by approximately 40%, created a very high deficit with adverse effects on the economic and social life and on the environment. Figure 4 shows the normal water demand the actual water supply and the deficit faced during the years 1987-1999.

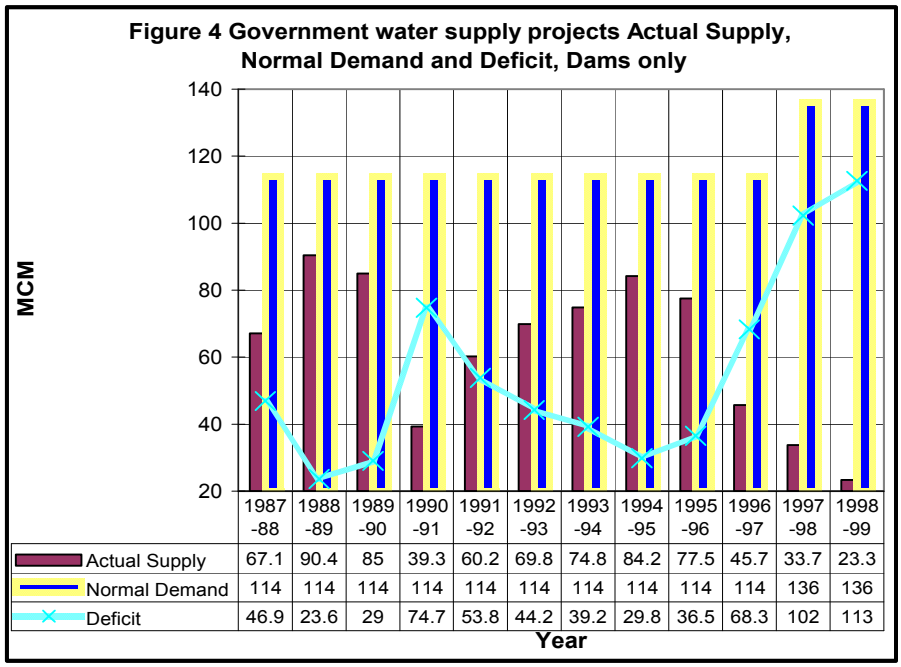


Figure 4. Government water supply projects – actual supply, normal demand and deficit; dams only. The deficit varies from 23.6 MCM to 113 MCM or from 20.7% to 83% of the normal demand. The high deficit from the dams in the period 1998-1998 and 1998-99 was mitigated by the Dhekelia Desalination Plant. The average deficit is estimated at 55 MCM or 48% of the total normal demand compared to the total theoretically estimated at 54.3%.

8. CONSIDERATIONS ON THE REDUCTION OF AVAILABLE WATER RESOURCES AND THE INCREASE OF WATER DEMAND

The changes which occurred on the water demand due to Turkish invasion, the tourist and foreign labor influx were known to the Government and to the water managers but what was not known were the climate change and its effect on the water resources availability. The reduced precipitation recorded during the period 1972-1990 was attributed to erratic climate behavior and there was not serious suspicion that there as a climate change. It was early in the 1990's after the completion of all major dams and when actual inflow data were collected for approximately 6-10 years that the water managers started to realize that the inflow to the dams was not corresponding to what was originally estimated and that the inflow was by 40% less than that originally estimated and considered for the water balance. With the drought of 1991 the water managers realized that the major dams by then completed was not able to meet the water demand and a proposal to introduced seawater desalination was submitted to the Government and Tenders were invited. Unfortunately for various reasons no tender award was made and the desalination project was abandoned until 1995 when a new tender was issued and awarded and the first desalination project was put into operation in 1997. In 1998 in a report to the Ministry of Agriculture Natural Resources and the Environment the Water Development Department presented the new water balance which had a deficit of 40% compared to the original water yield estimates, warned that deficits were increasing due to continuous increase of domestic water demand and proposed a series of measures including the construction of desalination plants in Larnaca, Limassol and Paphos. Based on the Report the Council of Ministers took certain decisions restating its water development policy setting priority to the development of the non conventional water such as desalination and wastewater recycling, intensifying water saving measures, implementing water auditing etc. Among the actions taken was a study carried out by the Water Development Department and the Food and Agriculture Organization on the Re-assessment of Water Resources of Cyprus which was completed in 2002. The Study concluded, from the statistical analysis of the precipitation, that there is a decrease in the mean annual flow to the dams

on the average by 40%, meaning that the island has roughly 40% less water resources than had been estimated based on the pre-1970 records. The study gave also the new annual water Balance and the water demand on which further actions should have been taken.

9. WATER CRISIS AT ITS PEAK AND CAUSES

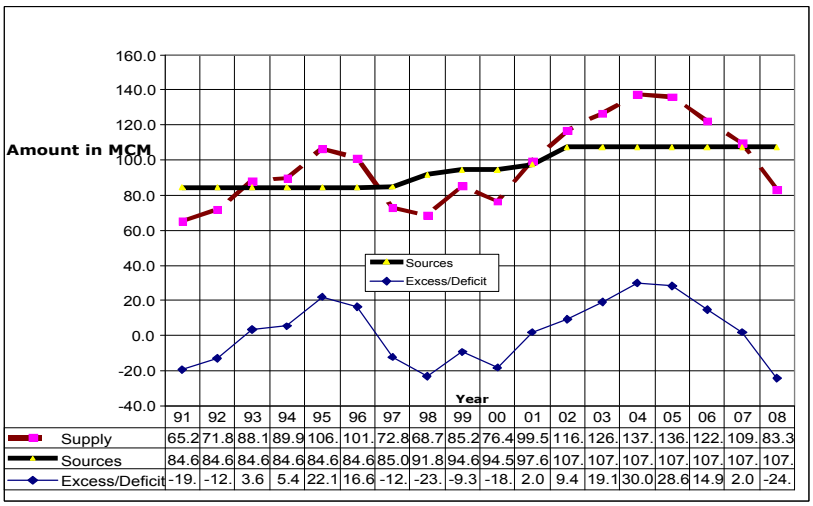
Although those responsible for the water development and water management had all the information on the available water resources since 1997 and verified in 2002, the situation was led to a water crisis in 2007 reaching a peak in 2008. During 2008 the total supply from the Government Projects amounted only 83.3 MCM or 60.8%, to the year supplied a year before which was a year without restrictions. The domestic water supply was reduced by 11.4 MCM or 15.4 % compared to 2007, and the supply of irrigation water was reduced by more than 50%. The situation was very critical and water was a limiting parameter to almost all major economic activities since water cuts were applied to the tourist and potable sectors and to the irrigation industry at levels where survival was questionable. The water cuts could have been worse if the emergency measures including the increase of the desalination plants production by 25% and the import of water from Greece, and the construction of the mobile desalination plant in Limassol of 20,000 m3/day were not successfully implemented. The impacts on the quality of life and the economic activity although not yet evaluated were huge just taking into account that for the import of water from Greece the cost amounted to approximately 50 million Euro for 8 MCM, irrigation water supplied amounted only to 50% and the population was forced to incur extra expenditure for the drilling of boreholes, the purchase of extra storage tanks, pumps and motor, purchase and installation of water saving equipment, etc. All these were caused because of the following actions or no actions taken by the state:

a) No timely implementation of the Domestic Effluent Recycling Projects: The domestic effluents recycling plan provides for the recycling of all domestic effluents and introduction into the master plan water balance or in other words to be used in places where natural quality water is used and free equivalent amounts of water for domestic use. The Nicosia, Larnaca and Limassol sewage treatment and recycling projects are developing very slowly depriving the projects of valuable quantities of water.

b) No timely implementation of the seawater desalination plan. The Government in 1999 decided to install one Seawater desalination plant in Limassol to be operated by 2002, and another desalination plant in Paphos to be operated by the year 2005, with a total capacity of 20 MCM. The implementation of the Desalination program never materialized because of interventions from the following:
   - The Environmentalists: They objected to the construction and operation of seawater desalination projects on the grounds of environmental criteria (emissions to atmosphere and marine environment degradation) claiming also that the water resources of Cyprus are enough to cover the present demand, and that further reduction of losses and wasteful use of water will make up any shortages of water. These assumptions and conclusions are totally wrong since the water resources of Cyprus are very limited, unaccounted water is not totally losses but water consumed and not accounted and environmental impacts of not having water are worse than those from the desalination.
   - The Local People Objections: Since the desalination Plants are sited in coastal areas where land has a high value the affected people and communities objected strongly to have desalination plants next to their properties claming that it will affect the environment and value of their properties. Unfortunately the Governments did not have the will and did not take any initiatives to persuade these communities and people that their concerns are not real. These led to long time postponement or stoppage of the desalination program.
   - The politician's ignorance on water matters and desalination issues: The political ignorance on water matters was one of the obstacles that impeded the implementation of

the seawater desalination projects. Politicians from all fractions of the political life objected, each for his own reasons, to the implementation of the desalination program ignoring in principle the adverse effect of water shortages. Desalination was used as a political tool to limit the capabilities of the government or of the specific minister or to help individuals or groups of individuals, or to promote the interests of specific group of people, ignoring the adverse effect that was caused on the population in general.

- Government officials with total ignorance of the subject use their influence to postpone or even to abandon the program. It is very sad that government departments or offices, without any responsibility on the water issue, without even trying to understand the subject matter, used their influence to stop or postpone the desalination program, on the grounds of saving money or postulating that the money spent could be saved. Of course when the crisis was at its peak they use their independent position to criticize bitterly those that were unable to deliver enough quantities of water.
- Political indecisiveness and intervention lead to bad management of the available water resources: It is know by everybody that water is limited and insufficient to cover the needs of the population both for domestic and irrigation needs. It is also true that for Cyprus with the semi dry climate and with a big coefficient of variation of water inflow to the dams, water has to be saved when it is abundant, for use during water shortage periods and that pumping of water reservoirs and aquifers should be equal to the safe yield of the source. These are very few principles on which the management of the water stored in the dams and recharged to the aquifers should be based, pumping water from the sources, on an annual basis, quantities equal to the safe yields. With the 2002 study, which proved that the available water resources were by 40% less than those calculated with the pre-1970 precipitation data and on which the irrigation allocations were decided since these quantities were not available a decision should be taken to reduce the irrigation allocation until the desalination projects were constructed. With the indecisiveness of the Government, because of political cost, the previous decisions were valid and the Water Development Department was instructed during the periods of high inflows to the dams to supply water according to the original allocations or even higher. This resulted to the deletion of the surface reservoirs and the reduction of the boreholes yields leading to a water crisis during low precipitation years. Figure 5 shows the total annual water supply from the Government Water Projects and the total safe yields of the Government Projects during the period 1991-2008.



	91	92	93	94	95	96	97	98	99	00	01	02	03	04	05	06	07	08
Supply	65.2	71.8	88.1	89.9	106.	101.	72.8	68.7	85.2	76.4	99.5	116.	126.	137.	136.	122.	109.	83.3
Sources	84.6	84.6	84.6	84.6	84.6	84.6	85.0	91.8	94.6	94.5	97.6	107.	107.	107.	107.	107.	107.	107.
Excess/Deficit	-19.	-12.	3.6	5.4	22.1	16.6	-12.	-23.	-9.3	-18.	2.0	9.4	19.1	30.0	28.6	14.9	2.0	-24.

Figure 5. Total Supply from Government projects and total safe yield of all.

From the figure the following is noted:

1. The Government Projects total safe yield increased from 84.6 MCM in 1991 to 107 MCM in the year 2008, mainly due to the introduction of Desalinated water (30 MCM) and the reduction of the annual yield from boreholes and from the surface reservoirs.

2. The total supply of water from the government projects on an annual basis varied around the sources safe yield with some years highly above the safe yield and other years below the safe yield, depending on the availability of the water.
3. During the period 1993-1996 the supply was in excess of the average yield in total by 31 MCM or on the average by 7.5 MCM /year.
4. Similarly in the period 2002 to 2006 the supply was in excess of the average sources yield with an average of 20.4 MCM per year, which means that a quantity of 102 MCM was supplied in excess of the source average yield.

The irrational management of the limited resources led to the great water crisis that was faced in the years following the excess water supply and particularly to the water crisis in the years 2007 and 2008 and now in 2009 where the water to be supplied will be only 85% of the domestic needs and 50-60% of the normal irrigation demand.

10. CONCLUSIONS

Although a water master plan was developed and implemented on time, a number of events caused rapid increase of the domestic and irrigation water demand, mainly due to the Turkish invasion, the tourist development, the influx of foreign workers and the change of the living style. The total abrupt increase which was in addition to the provisions in the master plan was of the order of 15 of the total demand. From the point of view of the sources these were reduced by 40% because of climatic changes. The changes to the water balance sheet were realized by the Water Development Department early in the 1990's and the Government was informed accordingly with proposals how to augment the available quantities and restore the balance sheet. Although the Governments periodically understood and took the right decisions these were not implemented on time, which upset the water balance. The worse was that the Government and state officials continue to behave as if there was no problem supplying water quantities when available above the safe yields of the sources and introducing water cuts when water availability was less than the demand, creating conditions of water scarcity and acute water shortages. A simple mathematical exercise would reveal that if the planned desalination plants in Limassol and in Paphos were put into operation on time the water sources would increase by 20 MCM per year bringing a balance to the water resources balance sheet at the right time avoiding the acute water crisis of 2007, 2008 and 2009 and minimizing the economic, social and environmental impacts. The water resources management should be carried out with due diligence and with a view that Cyprus is a drought prone country. The groundwater resources must be gradually restored and the surface water resources must be managed according to their availability on a long term basis, supplying annually quantities not exceeding the safe annual yield.

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WATER SCARCITY, WASTEWATER REUSE AND THE ECONOMIC VALUE OF WATER

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Abstract: Wastewater reuse is one of the most promising measures to stabilize water availability in arid and semi-arid areas. Wastewater reuse projects can be made more effective and widespread by applying economic approaches. Under specific conditions farmers, cities and the environment can gain from water reclamation and intersectoral water transfers. In particular, farmers should be motivated by adequate economic incentives to reuse treated wastewater rather than by charging them with the full cost of water reclamation. It has been often suggested to charge farmers with the true value of water at catchment areas, so that the application of treated wastewater will pay for them. However, such a pricing policy may discourage farmers to replace conventional resources with reclaimed water. Furthermore, multi-stakeholders platforms at watershed levels appear to be preferable over compulsory regulations in achieving economically efficient intersectoral water transfers. Water agencies and providers of treated wastewater should involve farmers in structured negotiations to argue them into the benefits of water reclamation. In agriculture, cost savings due to reduced use of chemical fertilizers, increases in yields and revenues due to nutrient-rich reclaimed water and improved water supply are the dominant positive economic impacts. From the viewpoint of cities, the additional supply of freshwater conserved by farmers engaged in reuse can lead to benefits, including reduced costs associated with groundwater overexploitation, transmission of remote water resources or seawater desalination. If the expenditures needed for wastewater reuse and intersectoral water transfer are small relative to the benefits, water exchange between the agricultural community and cities will be worthwhile. Scarce water resources can be used more effectively and further impairments to nature due to water shortage will be prevented. The economic value of water for non-agricultural uses plays a crucial role. The additional freshwater volumes conserved, evaluated with domestic water prices, may indicate the benefits of improvements in municipal water supply. Non-monetary benefits, such as restoration of groundwater tables, prevention of wastewater recharge into rivers and conservation of eco-systems are to be taken additionally into consideration.

Key words: Agriculture, intersectoral water transfer, irrigation, wastewater reuse, water value.

1. INTRODUCTION

A more efficient water management, including wastewater reuse and water exchange between agriculture and cities, can contribute to tackle water scarcity and to alleviate follow-up problems. Wastewater reuse can be also a solution in areas, where farming in small sized parcels prevails like in rapidly growing developing countries (Lens et al. 2001; Jimenez and Asano 2008). By application of wastewater reuse in agriculture overexploitation of aquifers and surface waters can be avoided. Food supply can be improved by increased agricultural productivity and, where freshwater resources are released, the water availability in municipalities can be enhanced. Higher yields of food crops due to wastewater reuse mean improved food availability. Without access to resources such as wastewater many poor families would not be able to meet their nutritional needs (WHO 2006).

As competition for water from other sectors intensifies, irrigation will be increasingly under pressure to reallocate water for higher value uses. The number of regions rises where water availability limits food production and where intersectoral competition for water increases with urbanization and economic development (Svendsen and Turral 2007). Around the cities in arid and semi-arid regions, competition with other sectors often makes water the main factor that limits agricultural and urban development (Bahri 2008).

According to studies in Middle Eastern and North African countries, the cost of increasing the provision of good quality water, ranging from 1 US$ to 6 US$ per cubic meter, can exceed

significantly the cost of wastewater treatment. Using reclaimed water can substitute good quality water for less quality demanding uses, and allows a reduction of local groundwater over-drafting and seawater intrusion in coastal areas. For instance, in order to meet water demands, reclaimed water volumes must increase more than 4 times by the year 2010 in Jordan. Some countries in the Middle East are planning to reuse 50 to 70% of their total wastewater volume (Bahri 2008). According to other studies the price of recycled water used for irrigation appears consistently lower than those of potable water (Lazarova and Bahri 2008).

In those cases, the economic and environmental benefits of freshwater release would justify the expenses for water reclamation. Non-application of water reclamation can lead to considerable losses in terms of societal welfare forgone. In assessing the benefits of intersectoral water transfer the value of high-quality water plays a key role. This value increases with tightening water scarcity.

2. INTERSECTORAL WATER TRANSFER

2.1 Benefits and costs

Water reclamation in irrigated agriculture associated with release of freshwater for cities can result in a wide range of ecological and economic benefits. They include:

- Prevention of yield losses and mitigation of food shortage.
- Increase in crop yields due to improved water supply and nutrient contents in wastewater.
- Additional incomes for farmers due to cost savings in water abstraction and fertilizing, and increase in sales revenue.
- Reduction of discharge of effluents into rivers and less overexploitation of water bodies.
- Conservation of aquatic ecosystems (such as wetlands) resulting from recharge of aquifers and rivers with reclaimed water.
- Additional water supply for cities by reallocation of water from irrigated agriculture to cities.

It is also important to be aware on the costs of water reclamation:

- Operation and investment cost of water infrastructure needed to treat effluents and to convey reclaimed and released water.
- Costs of onsite facilities such as adapted irrigation techniques and water ponds.
- Costs of water quality monitoring and administrating the maintenance of different water systems.
- Environmental damage such a contamination of water bodies and soil degradation.
- Health risks for farm workers and consumers.

The costs are to be compared with the benefits to find out whether water reclamation will be feasible and to determine the best reallocation strategies. Different calculation methods can be used. One of them computes the net benefits of alternative strategies by discounting the costs and value added at different years over the life-span of a water reclamation project. Another method calculates the annualized capital cost of water infrastructure and the operational costs per year. The second method allows comparing the reclamation cost with water prices. If the cost is significantly lower than the current water price, wastewater reuse in irrigation and water reallocation to cities would pay. In such a situation, the readiness to pay for potable water exceeds the cost of additional water supply. The same applies to other supply options such as seawater desalination which is often more costly than water reclamation (Spulber and Sabbaghi 1998; FAO 2006).

Not all the costs and benefits can be evaluated directly in economic terms. Thus, the assessment of water reclamation projects considers sometimes only the costs and expenditures prevented. However, the benefits from intersectoral water should be assessed and that can be done by evaluating water with its value also for non-agricultural uses. The value of potable water is composed of the cost of infrastructure, and the environmental and scarcity costs. If water prices

ignore the latter, there is a risk of resource overuse. Further value determinants are the willingness and ability of users to pay for high quality water. The scarcity cost depends on water shortage and demand. More often increasing water scarcity is being attributed to climate change and rapid economic growth, particular in tiger states.

Figure 1 illustrates in simplified terms the relationship between water shortage, scarcity cost and water use. The curve indicates the willingness to pay for potable water and shows increases in water use with declining tariff *T*, the sum of unitary infrastructure cost *c* and scarcity cost *v*. If water pricing would be based on infrastructure cost *c* only, water use (B) can go beyond the current resource availability *A* and may cause overexploitation of *B* - *A*. If water shortage aggravates (shifting the bold line to the left) the scarcity value of water will rise which should be considered in water pricing. The same applies to an increase of water demand due to, for instance, growing population (shifting the curve to the right).

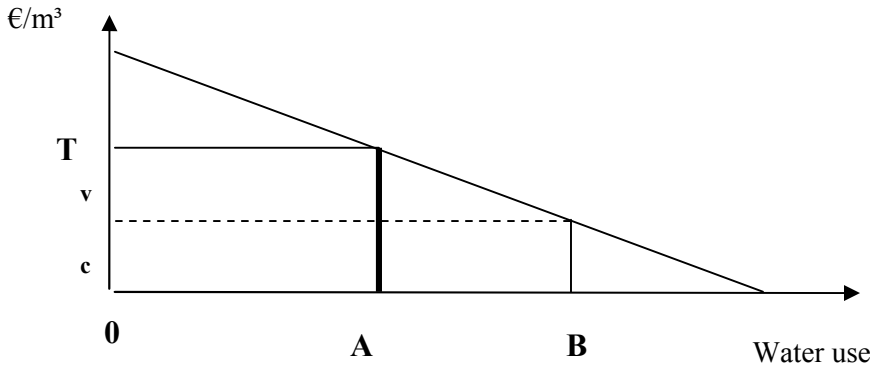


Figure 1. Scarcity value of water

The net benefit of intersectoral water transfer can be associated with the value of potable water, i.e. the value added for citizens will rise with increasing water scarcity. Box 1 gives an overview on key factors that influence directly the economical feasibility of water transfer projects.

Box 1. Costs and benefits of intersectoral water transfer

Costs	Capital and operational cost • Upgraded wastewater treatment • Reclaimed water distribution in irrigated agriculture • Freshwater conveyance to cities
Benefits	Value added to farmers • Less use of fertilizers • Reduced abstraction of conventional water • Yield losses prevented • Increased yields Value added to citizens • Increased supply of potable water • Less resource overexploitation and pollution

2.2 Governance of intersectoral water transfer

Wastewater reuse in irrigated agriculture can contribute essentially to alleviate water shortage problems. Governments are challenged to foster wastewater reuse projects, especially if market-based initiatives do not provide sufficient incentives to encourage farmers to replace freshwater with treated wastewater. Due to high charges wastewater use is sometimes not an attractive option

for farmers. Charges that aim to recover fully the cost of wastewater treatment and distribution networks discourage farmers to convert to reclaimed water. Institutional barriers (such as patrimonial water rights) and reservations against wastewater use can also hamper the conversion from conventional resources to reclaimed water.

In situations, where wastewater reuse and intersectoral water transfer could mitigate considerably water shortage, governments may enforce farmers by compulsory regulations. For instance, the State of California created laws that prescribe the use of treated wastewater instead of freshwater (Asano et al. 2007). However, due to the complicated regulatory proceedings the laws have been applied in only a few cases. Alternatively, authorities may require the use of treated wastewater as a condition for granting or renewal of freshwater abstraction rights. By rigid regulations, however, the conflicts with authorities may become hardening and can reduce the readiness of farmers to cooperate. Often authorities do not have sufficient information on the water volumes farmers extract from natural resources.

Governments may implement water pricing policies instead. Charging farmers with the water prices other water users have to pay, such as citizens or industries, may induce to convert to reclaimed water, if its price is less than cost reductions in using conventional resources. Such cost reductions may embrace not only less charges for freshwater but also the cost savings in water abstraction and fertilizing. Moreover, such pricing policies may stimulate farmers to apply water-saving irrigation technologies, by which freshwater can be released. The revenue from such charges could serve as a financial resource for funding the cost of water reclamation (Abu-Madi et al. 2007). On the other hand, such a policy may endanger the existence of farmers especially in economically weaker regions. Under certain conditions, cost savings might not offset the inconveniences associated with crop restrictions and health security measures required.

Negotiations with farmers are often preferred by authorities (FAO 2007a, b). The aim is to convince farmers on the economic and environmental advantages resulting from water reclamation and reallocation. Farmers are often not aware of these advantages. Significant increases in societal welfare can be achieved by more effective tackling water shortage problems. Treated wastewater may be offered to farmers at a discount, for free or even by compensation payments. Farmers can be disposed more easily to use treated wastewater by providing economic incentives, so that they will get better off. For instance, a farmer may sell all or part of his water rights to nearby growing city or he/she may provide a given quantity of freshwater only in drought periods and the city may compensate him/her for lost crops.

When a city enters into bargaining with farmers it is important for each of the parties to know the room to negotiate. A farmer may expect minimum income improvements from replacing freshwater with treated wastewater. Farmers` decision may be influenced by:

- Cost savings S in abstracting conventional resources and fertilizing
- Increases in sales revenue R due to more reliable water provision allowing increases in yields and preventing yield losses in drought periods
- Charge r and volume RW of reclaimed water used
- Compensation payment p per cubic meter obtained for released freshwater volume FW.

To ensure income improvements the sum of cost savings S, increase in revenue R and compensation payments p x FW must exceed the cost of reclaimed water r x RW. From the viewpoint of cities, providing treated wastewater to farmers, the following factors may influence the economic and financial feasibility of water exchange:

- Value u and volume FW of freshwater released by farmers
- Charge r and volume RW of treated wastewater delivered to farmers
- Compensation rate p and volume of freshwater FW released
- Cost of intersectoral water transfer Q for upgrading wastewater treatment and conveying reclaimed and released water.

The cities would get better off if the benefit (u x FW) from water reallocation *plus* the revenue (r x RW) from charging farmers exceed the *sum* of compensation payments (p x FW) and cost of

water exchange *Q*. Obviously, there are lower and upper boundaries that limit the range for charging and compensating farmers.

Compensation payments can be zero if the cost savings and increase in sales revenue lead to considerable income improvements in agriculture, even though farmers may contribute to the cost of intersectoral water transfer. The charge on reclaimed water can be zero, provided the benefits of released water is sufficiently high and the cost of water exchange *Q* relatively low. Then the value of freshwater *u* exceeds the unitary cost of intersectoral water exchange (*Q : FW*). This means that the value of improved water supply justifies the cost of water reclamation and freshwater reallocation. Both parties can gain: Farmers pay less for reclaimed water than they benefit from wastewater application. Cities gain if their outlays are less than the benefit obtained from improved water supply.

Further costs and benefits should be taken into account, even though it is difficult to evaluate them in economic terms. Value surrogates may help, such as the saved pumping cost of groundwater over-drafting or the costs needed to restore impaired natural habitats. Such values are often dependent on the extent to which the costs of remedial actions will be accepted. Cost savings in drinking water purification may reflect the benefits from reduced discharges into water bodies.

Water reclamation projects can require huge investments which are often not affordable for municipalities. In such cases, governmental financial aids are needed to construct wastewater treatment plants, storages or pipeline networks (Lazarova and Bahri 2008). Otherwise economically efficient and sustainable water reclamation projects would not be realized. Farmers should be additionally supported for applying water-saving irrigation techniques to promote the release of freshwater. Where water reallocation will be beneficial both for farmers, cities and the environment, granting governmental subsidies will be justified in the long run.

As mentioned, the correct evaluation of water for non-agricultural uses is crucial for proving the economic efficiency of water reclamation and intersectoral water transfer. However, this approach does not mean that consumers are to be charged with high tariffs regardless of their economic situation. Water pricing policies have to consider that in many countries poor people cannot afford to purchase potable water as its full cost. Governmental subsidies or cross-subsidies between wealthier and poorer water users are required.

3. APPLICATIONS

As pointed out above, the economic value of freshwater for cities can be much higher than in agriculture. By resource reallocation substantial water can be freed and used in higher value uses (Molle and Berkhoff 2006). As a recent FAO study has proved, under certain conditions the benefit of intersectoral water transfer can exceed significantly its cost (Heinz et al. 2008). This means that investments in wastewater reuse facilities pay back and are profitable in the long run. In such cases, the economic benefits of freshwater release would justify clearly the expenses for more water reclamation.

The economic and financial feasibility of water reuse projects is a major prerequisite for an efficient implementation of water transfer strategies. The use of treated wastewater for irrigation can raise the productivity in agriculture. Production costs can be reduced and crop yields can rise. However, depending on to which extent farmers are committed to contribute to the costs of wastewater reuse projects income losses may occur. Here, the cost of water reclamation in relation the economic benefits generated in agriculture and the charges for using treated wastewater for irrigation are the crucial influencing factors.

Further applications of wastewater reuse are landscape irrigation, groundwater recharge and the replacement of freshwater demands of industries. As Spulber and Sabbaghi (1998) pointed out, groundwater recharge with reclaimed municipal wastewater can be used for the restoration of groundwater levels, the protection of underground freshwater in coastal aquifers against seawater intrusion and for the storage of surface water, including treated effluents. Especially the latter application can be important for irrigated agriculture where the demand for reclaimed water is

confined to certain seasons and water loss from evaporation should be prevented. A typical example for the use of reclaimed water as a hydraulic barrier to solve the seawater intrusion problem can be found at the Llobregat lower delta in Spain. The recently upgraded wastewater treatment plant generates treated effluents that is also used to supply the ecological flow of the lower part of the Llobregat river, to provide water for agricultural irrigation and to supply water to wetlands in the river deltaic areas (Sala et al. 2008).

A well-known case is the Mezquital Valley (named also as Tula Valley), where the Mexico City's untreated wastewater is used by 73,600 farmers to irrigate 76,000 ha. Due to the improved water availability and the nutrient content of the wastewater the productivity is much higher than at rain-fed agricultural land (Jiménez 2008a). However, there are health risks, such as increased morbidity in children and fears concerning the health of consumers. Therefore, the construction of further treatment plants at Mexico City is planned to remove pathogens while partly preserving nutrients. It is proposed to use parts of the aquifers at Mezquital Valley to supply Mexico City (which is faced with rapidly increasing water demand) either by infiltrating the reused wastewater to the aquifers or by extracting groundwater where wastewater has no or less influence (Jiménez 2008b).

Investigations on existing water reuse projects have found out that the prices of reclaimed water are in the most cases lower than those of potable water. They range from zero to 25% of potable water rates (Lazarova and Bahri 2008). This indicates that the value of freshwater which can be potentially released due to more wastewater reuse in agriculture exceeds significantly the cost of water reclamation, even though current domestic water tariffs often do not reflect the full cost of potable water. In particular, many of the major cities in Mediterranean countries have such tariffs in contrast to cities in countries with abundant water supplies. On the other hand, the added value of irrigation for food and horticulture production can be relatively high, depending on the market value of agricultural products. According to a study dealing with wastewater reuse in Morocco, the economic benefits in irrigated agriculture are the results, firstly, of the low prices of reclaimed water (0.05 €/m³) relative to the charges for groundwater use (0.07 €/m³) and, secondly, of the increased crop yield (200 to 500%) Both impacts lead to an increase in farmers' revenues and incomes in the range of 200-500 €/ha (Lazarova and Bahri 2008). Consequently, both farmers and citizens can gain from fostering wastewater reuse and intersectoral water transfer.

As Bahri (2008) reports, in Jordan by the year 2020, when the population is projected to be about 9.9 million, the expected available volume of treated wastewater is estimated to be 265 M m³/yr, which is about 25% of the total water available for irrigation. Currently the major proportion of reclaimed water is discharged to wadis and reaches reservoirs used for irrigation. The irrigated area is around 13,300 ha. In the future, there could potentially be about 43,100 ha irrigated with treated effluent if all the water was to be used for irrigation purposes. In the central Jordan Valley, 56% of farmers use drip irrigation; the remainder use surface irrigation methods. To encourage reuse, the government made it mandatory for all new wastewater treatment projects. Reclaimed waters are used for recharge of a salty fossil aquifer near the site in Kufrinjeh and a shallow, partly saline, aquifer in Aqaba. Farmers using treated effluent mixed with freshwater in rivers, reservoirs and canals pay up to 45 Fils/m³ (around 0.05 €/m³), the same price paid for freshwater. Since the tariff for distributing the water does not cover the operation and management costs of reclaimed water, the Jordan Valley Authority (JVA) envisages increasing tariffs for conventional irrigation water in the Northern Jordan Valley, while tariffs for blended water in the Southern Valley would remain at the current level in order to compensate farmers for the higher salinity of the reclaimed water. The JVA provides free irrigation water to farmers during winter for salt leaching. Farmers cannot sell or lease their water rights. It is remarkable that the water quality standards for reuse have been made recently more stringent. The Jordan Ministry of Water and Irrigation is conducting a detailed master planning project and has adopted a new overall water strategy, including wastewater management. Of importance are measures such as upgrading the existing treatment plants and direct water reuse to replace freshwater sources in agriculture, industry and landscape. Special emphasis is placed on the replacement of groundwater used currently for irrigation with reclaimed water from

new water reclamation plants (Bahri 2008). Future research activities may explore the economic values added for the agricultural community, households in villages and citizens at Jordan Valley resulting from extended wastewater reuse as part of an integrated water resource management (IWRM) (SMART 2009).

4. CONCLUSIONS

In arid and semi-arid areas farmers face often losses in yields and income due to *water* shortage and unreliable water supply. The yields could be increased by using the nutrient content of wastewater. Expenditures for artificial fertilizers and water abstraction could be saved. Augmenting the water availability by wastewater reuse can increase yields, farmers' income and the agricultural productivity. With population growth and accelerated urbanization, improvement of living standards, development of irrigation, industry and tourism, the problems of water scarcity will intensify. At the same time the volume of urban wastewater is increasing.

From the viewpoint of citizens high water costs and prices are associated often with water shortage, especially where remote resources are to be used, dams must be constructed and seawater desalination is needed. The release of freshwater by farmers can improve the water availability not only for domestic users but also for other purposes (industries, tourism and aquatic environment).

The economic benefits from water exchange between agriculture and cities can be assessed by evaluating the damage that would be caused otherwise by resource overuse and pollution. An additional indicator is the value added obtained by different water users.

Thanks to the benefits generated by water reallocation cities can provide treated wastewater to farmers without charging them with its full cost or for free, and under certain conditions, even by compensating them. Both farmers and cities can gain (win-win situation). Positive impacts such as restoration of aquifers, conservation of aquatic ecosystems and less seawater intrusion at coastal areas are further benefits. Particularly in developing countries with rapid urbanization more municipalities may be encouraged to reclaim water instead to discharge treated or untreated wastewater into rivers or to the sea.

Bargaining and water trading between different stakeholders at local level are preferable to compulsory commitments to achieve economically efficient wastewater reuse projects. Water agencies should enter into negotiations with farmers to convince them on the economic advantages resulting from using reclaimed water. The effectiveness of water pricing as a political tool for demand management in agriculture appears to be limited.

The net benefit from intersectoral water transfer can be considerable if the value added for farmers and citizens exceeds significantly the costs of water reclamation and freshwater reallocation. The benefits depend on the increase in agricultural productivity, the costs of municipal water services, the water scarcity and the readiness and ability of citizens to pay for high quality water. When water reclamation improves water availability, the value of freshwater might be gradually depreciated as consequence of decreased costs of water services and reduced water scarcity. Thus, wastewater reuse projects should not focus only on water reallocation from agriculture to cities but should be embedded in an integrated water resource management (IWRM) at river basins.

In regions with small and far-scattered villages, decentralized wastewater reuse systems may be preferable to large central wastewater treatment plants. This applies especially to economically weaker regions as in developing countries. Typical decentralized water reclamation is characterised by onsite treatment and wastewater reuse for irrigation and other domestic uses. In contrast, at centralized concepts treatments plants are often located remotely from the areas of reuse and require costly conveyance systems. One of the advantages of decentralized wastewater reuse is that they can, as 'closed cycles', be adapted to individual homes and to local agro-hydrological conditions with low financial requirements. Further cost and benefit analyses are needed to evaluate the economic values added for farmers and municipalities that result from water reclamation and intersectoral water transfers.

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HYDROLOGICAL DROUGHT DETECTION USING A WATER BALANCE DERIVED DROUGHT INDEX AND METEOROLOGICAL DROUGHT INDICES

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Abstract: This study estimates hydrological drought characteristics using a water balance derived drought index and meteorological drought indices in Pinios river basin, Thessaly, Greece. Thessaly is an agricultural plain region surrounded by mountains and is traversed by Pinios River, which has a basin area of about 9500 Km^2. Fourteen (14) sub-watersheds of Pinios river basin were delineated according to the major tributaries of Pinios river using GIS. For the assessment of hydrological drought, because none of the sub-watersheds have flow gauge stations at their outlets, the following procedure was followed: A 6-parameter monthly conceptual water balance model (UTHBAL model), which uses as inputs areal precipitation, temperature and potential evapotranspiration, was calibrated with the available observed runoff data to optimize model parameters. Monthly precipitation from 66 stations and temperature data from 26 stations were used for the estimation of the basin-wide precipitation and temperature. Then, the water balance model was applied regionally to simulate runoff from the 14 major sub-watersheds of Pinios River basin for the period October 1960 - September 2002. The synthetic runoff was normalized through Box-Cox transformation and standardized to the mean runoff to produce the water balance derived drought index for the assessment of hydrological drought. The basin-wide precipitation and temperature data for the 14 sub-watersheds were used for the calculation of meteorological drought indices. The Standardized Precipitation Index (SPI) at multiple time scales and four indices of the Palmer method (i.e. PDSI, WPLM, PHDI and the Palmer moisture anomaly Z-index) were used as indicators of meteorological drought severity. The hydrological drought index was statistically compared with the meteorological drought indices for the study watersheds. The results showed that the water balance derived drought index is a good indicator of hydrological drought in all watersheds, since is capable to quantify drought severity and duration identified by meteorological drought indices.

Key words: drought indices, drought severity, hydrological drought, meteorological drought, runoff.

1. INTRODUCTION

Droughts are the world's costliest natural disasters, causing an average $6-$8 billion in global damages annually and collectively affecting more people than any other form of natural disaster (Wilhite, 2000). Given the consequences and pervasiveness of drought, it is important to assess drought severity, but precise quantification of drought is a difficult geophysical endeavor. Several studies have been made to assess and quantify different aspects of droughts, such as spatial differences in droughts (Loukas and Vasiliades, 2004), drought prediction with the use of atmospheric circulation indices (Lloyd-Hughes and Saunders, 2002), and drought hazard mitigation (WMO, 2000). However, more efforts have been made to develop drought indices, which allow an earlier identification of droughts, their severity and areal extent. During the twentieth century, several drought indices were developed, based on different variables and parameters (Heim, 2002; Keyantash and Dracup, 2002.). Drought indices are very important for monitoring droughts in time and space, and drought early warning systems are based primarily on the information that drought indices provide (Svoboda et al., 2002).

Hydrological systems respond to precipitation changes as a function of the time scales determined by the different frequencies of the hydrologic/climatic variables. Peters et al. (2005)

have shown that the effects of the meteorological droughts in groundwater are: a) the decrease of the frequency of drought events and b) a shift in the drought distribution. Therefore, drought indices used must be associated to a specific timescale to be operative for management purposes in each of the different usable water sources. McKee et al. (1993) indicated that the time period from the arrival of precipitation until usable water resources (i.e. soil moisture, groundwater, snowpack, river discharges, and reservoir storages) differs considerably. Thus, the time scale over which precipitation deficits accumulate becomes extremely important and functionally separates different types of drought: meteorological, hydrological, water resources, agricultural, socioeconomic, etc. Recent studies have used meteorological drought indices as indicators of hydrological and water resources variables, like soil moisture, surface runoff and reservoir storage. Vasiliades and Loukas (2009) presented the assessment of Palmer drought indices as indicators of surface runoff and soil moisture drought at seven (7) basins ranging from 133 to 6591 km^2. They found that weighted PDSI is a good indicator of surface runoff drought, whereas the Palmer Z-anomaly index represents satisfactorily soil moisture drought. Vicente-Serrano and Lopez-Moreno (2005) evaluated the Standardized Precipitation Index (SPI) of various time scales as indicator of runoff and reservoir storage in a 2181 km^2 mountainous basin in Spain. They provided empirical evidence that surface runoff respond to short SPI time scales (1-4 months), whereas the reservoir storages respond to longer time scales (7-10 months).

The objective of this study is to monitor hydrological drought severity using a water balance derived drought index in subwatersheds of Pinios river basin, Thessaly, Greece. These sub-watersheds, which were delineated according to the major tributaries of Pinios river using GIS, show high complexity of hydrological processes with a marked seasonal and interannual variability. Furthermore, two well-known meteorological drought index families, the SPI at multiple timescales and the Palmer drought indices, were estimated for comparison with the water balance derived drought index in depicting drought patterns.

2. STUDY AREA

Thessaly is located in central Greece and is a plain region surrounded by high mountains. The Thessaly plain with an area of about 4000 km^2 is one of the most productive agricultural regions of Greece. The climate is continental at the western and central side of Thessaly and Mediterranean at the eastern side. Winters are cold and wet and summers are hot and dry with large temperature difference between the two seasons. Mean annual precipitation over Thessaly region is about 700 mm and it is distributed unevenly in space and time. The mean annual precipitation varies from about 400 mm at the central plain area to more than 1850 mm at the western mountain peaks. Generally, rainfall is rare from June to August. The mountain areas receive significant amounts of snow during the winter months and transient snowpacks develop. Thessaly experienced severe, extreme and persistent droughts during the periods from mid to late 1970s, from late 1980s to early 1990s and the first years of 2000s (Vasiliades and Loukas, 2009). These three drought periods were quite remarkable and affected large areas. During these three periods the monthly and annual precipitation was significantly bellow normal in Thessaly. The prolonged and significant decrease of monthly and annual precipitation has a dramatic impact on natural vegetation, agricultural production and the water resources of the region (Loukas et al., 2007).

Fourteen (14) subwatersheds of Pinios river basin were delineated according to the major tributaries of Pinios river using GIS (Fig. 1). Processed monthly precipitation data from a large network of precipitation stations (Fig. 1) for the period October 1960 to September 2002 were used. The monthly areal precipitation of each subwatershed was estimated by the Thiessen polygon method modified by the precipitation gradient using the stations, which are within or in the vicinity of the watershed. Mean monthly basin-wide temperature was estimated using the mean monthly temperature data from 27 meteorological stations (Fig. 1) with the method of temperature gradient. One temperature gradient was used for all study areas. Monthly areal potential evapotranspiration has been calculated using the Thornthwaite method. Finally, monthly discharge data from eight (8)

water-gauging stations were used for the calibration of the hydrological model to estimate model parameters. Table 1 presents the main characteristics of the study watersheds.

Table 2. Characteristics of the study watersheds

Subwatershed	Area (km^2)	Minimum Elevation (m)	Maximum Elevation (m)	Mean Elevation (m)
Enipeas	1079.31	100	1626	393.9
Farsaliotis	792.61	100	1008	227.3
Litheos	361.36	100	1190	317.9
Mesdani	1116.22	116	2129	790.7
Neoxori	308.22	100	1409	452.4
Pamisos	233.14	100	1990	583.6
Pinios	1293.76	1	1905	275.9
Portaikos	308.44	117	1852	632.7
Sofaditis	734.00	100	1618	469.1
Theopetra	142.89	100	757	213.5
Kalentzis	502.81	100	1428	315.7
Pliouris	274.79	100	1004	157.7
Titarisios	1925.94	100	2728	610.5
Kousbasaniotis	530.03	100	711	192.7

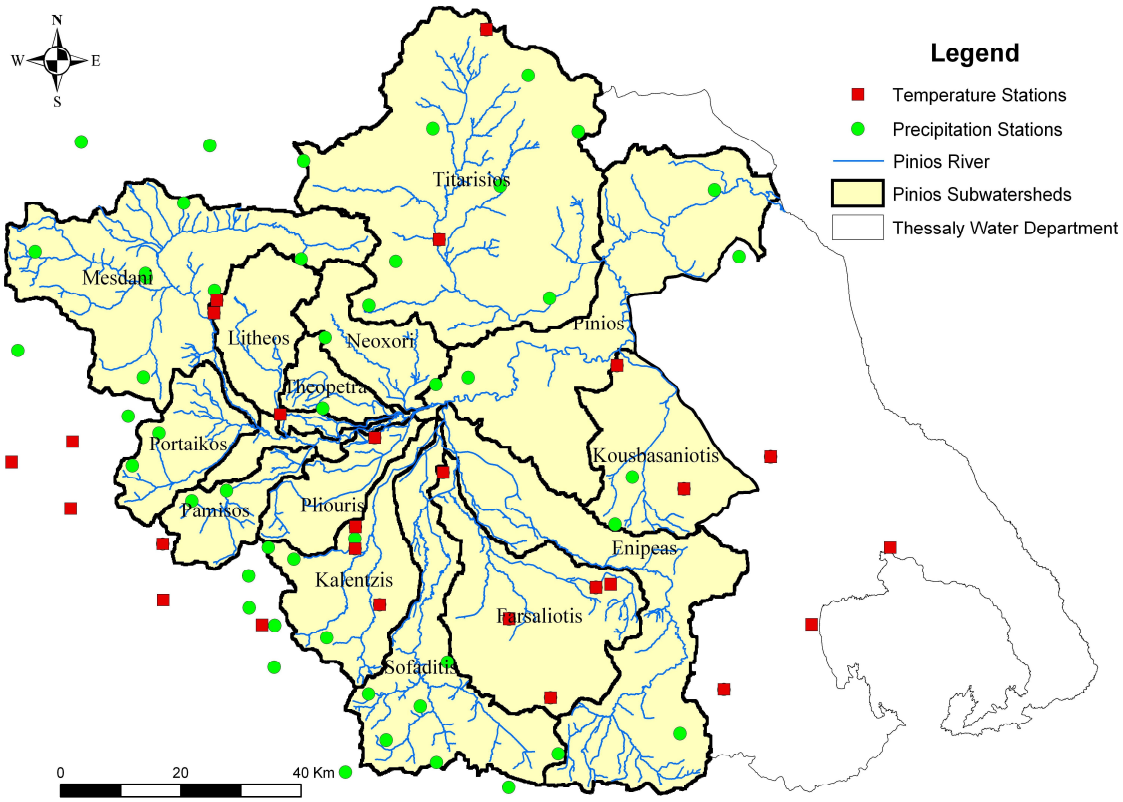


Figure 1. Study area and database.

3. METHODOLOGY

This study evaluates hydrological drought detection using a water balance derived drought index. The hydrological drought severity was evaluated from the standardized outputs of the monthly UTHBAL conceptual water balance model. The use of simulated discharges was mandatory due to the absence of discharge observations. Standardized simulated river discharges were compared with the two well-known meteorological drought index families, the SPI at multiple timescales and the Palmer drought indices. For comparison the Pearson correlation coefficient was used and the characteristics (i.e. severity and duration) of the identified drought episodes were evaluated and compared. In the next paragraphs the water balance derived drought index and the meteorological drought indices are presented.

3.1 Water Balance Derived Drought Index

Most of the study subwatersheds have not discharge observations at their outlets. For this reason, the monthly conceptual water balance UTHBAL model (Loukas et al., 2007) has been used to reconstruct and produce simulated runoff data. The UTHBAL model allocates the watershed runoff into three components, the surface runoff, the interflow runoff and the baseflow runoff using a soil moisture mechanism with the first priority of the balancing being the fulfillment of actual evapotranspiration. The model separates the total precipitation into rainfall and snowfall, because the correct division of precipitation is essential for accurate runoff simulation. The rain-snow percentage is estimated using a logistic relationship based on mean monthly temperature and the snowmelt is calculated estimated using the simple degree-day method. A complete mathematical formulation of the UTHBAL model could be found in a recent paper (Loukas et al., 2007). The UTHBAL model requires monthly values of mean temperature, precipitation, and potential evapotranspiration and produces values for actual evapotranspiration, soil moisture, groundwater and surface runoff. The input timeseries were estimated using the methods presented in the previous paragraphs.

The UTHBAL model has six parameters to be optimized in order to estimate watershed runoff. In this study, model parameter values were taken from previous application of UTHBAL in selected watersheds of Pinios river basin (Loukas et al., 2007, Vasiliades and Loukas, 2009) and applied regionally to estimate runoff in the study subwatersheds. The produced synthetic runoff timeseries were used for the estimation of hydrological drought. The synthetic were normalized, using the Box-Cox transformation, and standardized. The Box-Cox transformation takes the form:

$$Y = \begin{cases} \dfrac{X^{\lambda} - 1}{\lambda}, & \lambda \neq 0 \\ \ln(X), & \lambda = 0 \end{cases} \quad (1)$$

and the transformed values were standardized using:

$$Z_{WBI} = \frac{Y - \overline{Y}}{s_Y} \quad (2)$$

where, X are the values of the original timeseries of surface runoff, Y are the values of the transformed timeseries, λ is a parameter for which the values of the transformed timeseries (Y) are normally distributed, Z_{WBI} are the values of the standardized timeseries, $\overline{Y}$ is the mean value of the transformed timeseries and s_Y is the standard deviation of the transformed timeseries.

The transformation and standardization have been performed on monthly basis, meaning that the monthly values have been analyzed individually and separately for each month of the year. The transformed and standardized runoff timeseries were used as indicators of hydrological drought severity. Once standardized the strength of the anomaly is classified as set out in Table 2. This table also contains the corresponding probabilities of occurrence of each severity arising naturally from the Normal probability density function. Thus, at a given location for an individual month, moderate dry periods ($Z_{WBI} \leq -1$) have an occurrence probability of 15.9%, whereas extreme dry periods ($Z_{WBI} \leq -2$) have an event probability of 2.3%. Extreme values in the Z_{WBI} will, by definition, occur with the same frequency at all locations. Negative Z_{WBI} values indicate droughts and positive Z_{WBI} values denote wet weather conditions (Table 2).

Table 2. Drought classification by Z_{WBI} values and corresponding event probabilities

Drought Index value	Category	Probability (%)
2.00 or more	Extremely wet	2.3
1.50 to 1.99	Severely wet	4.4
1.00 to 1.49	Moderately wet	9.2
-0.99 to 0.99	Near normal	68.2
-1.49 to -1.00	Moderately dry	9.2
-1.99 to -1.50	Severely dry	4.4
-2 or less	Extremely dry	2.3

3.2 Meteorological Drought Indices

In the present study two well-known meteorological drought indices, Standardized Precipitation Index (SPI) and Palmer drought indices, were employed in the analysis to evaluate the water balance derived drought index. The Standardized Precipitation Index (McKee et al., 1993) has been developed for defining and monitoring droughts. The main advantage of the SPI is that can be calculated for multiple time-scales. This is very important because the timescale over which precipitation deficits accumulate functionally separates different types of drought and, therefore, allows to quantify the natural lags between precipitation and other water usable sources such as river discharge, soil moisture and reservoir storage. Computation of the SPI involves fitting a Gamma probability density function to a given frequency distribution of precipitation totals for a station, area or a watershed. The cumulative probability is then transformed to the standard normal random variable z with mean equal to zero and variance of one, which is the value of the SPI (Table 2). The Palmer drought indices are: 1) The Palmer Drought Severity Index (PDSI), 2) The Palmer Hydrological Drought Index (PHDI), a hydrological drought index used to assess long-term moisture supply, 3) the Palmer Moisture Anomaly Z-Index (Palmer Z-index), a measure of the departure from normal of the moisture climate for that month. This index can respond to a month of above-normal precipitation, even during periods of drought and 4) the Modified Palmer Drought Severity Index (WPLM) as proposed by the National Weather Service Climate Analysis Center for operational meteorological purposes (Heddinghaus and Sabol, 1991). The modification incorporates a weighted average of the wet and dry index terms, using the probability as the weighting factor. The WPLM and PDSI will have the same value during an established drought or wet spell (i.e., when the probability is 100%), but they will have different values during transition periods, with the WPLM having a more gradual transition from one spell to the next. Although Palmer indices are referred as meteorological drought indices, however, the calculation procedure considers precipitation, evapotranspiration, and soil moisture conditions, which are determinants of hydrological drought, i.e. the period during which the actual water supply is less than the minimum water supply necessary for normal operations in a particular region. The Palmer indices measure the departure of the moisture supply from normal conditions. Moisture supply is calculated from the water balance of a two-layer soil model using monthly mean precipitation and temperature data as well as the local available soil water content. From the input data, all the basic water balance terms, namely evapotranspiration, soil recharge, runoff, and moisture loss from the surface layer are estimated. A detailed calculation procedure of the original Palmer indices (i.e. PDSI, PHDI and Palmer Z-index) could be found in the original paper (Palmer, 1965) and for the WPLM in the study that has been originally proposed (Heddinghaus and Sabol, 1991). In this study, areal monthly precipitation accumulations were used for the estimation of the monthly SPI for 1-month, 3-month, 6-month, 9-month, 12-month, and 24-month timescales. Timeseries of monthly areal precipitation and temperature were used for the calculation of Palmers' indices. The estimation of the available water capacity was taken from the UTHBAL model for all study subwatersheds. Finally, because the Palmer indices have been criticized for their arbitrary classes and their spatial comparability (Vasiliades and Loukas, 2009), all Palmer indices have been standardized to the normal distribution in order to be comparable with the study drought indices. Hence, the strength of the anomaly for all calculated drought indices is classified as set out in Table 2.

4. RESULTS

Table 3 shows the Pearson correlation coefficients between the water balance derived drought index (Z_{WBI}), and the meteorological drought indices. For river discharges correlations are positive, but there are important differences with regard to examined meteorological drought indices. Higher correlations have been obtained with the 3-month SPI and 6-month SPI (average R = 0.80 and 0.79, respectively, for all watersheds) but with large ranges (Table 3) followed by WPLM (average R = 0.78 for all watersheds, ranging from 0.64 to 0.85) and PDSI (Table 3) for all study watersheds.

Graphical examination of temporal evolution and scattergraphs confirm that the Z_{WBI} matches better the 3-month SPI and 6-month SPI for all study watersheds except Kousbasaniotis subwatershed which Z_{WBI} corresponds better to larger timescales (12-month SPI and 24-month SPI). A possible reason could be the small slopes of this watershed. Graphical comparison of water balance derived drought index with Palmer indices show that WPLM closely matches the Z_{WBI} index. The monthly patterns follow the results of the continuous correlation analysis. High correlations (R>0.8) are found for autumn and early winter months. The highest correlations were obtained in December and January. Contrarily, the worst correlations were obtained for spring and summer months. This noticeable seasonality in the strength of correlations is, probably, the result of the hydrological behavior of the study watersheds. A possible reason could be the seasonal variability of precipitation and the accumulation of snow and its consequent melting. The snow accumulation and the snowmelt are not accounted in the calculation of the meteorological drought indices but they are taken into consideration in the water balance derived drought index.

Table 3. Correlation between Z_{WBI} and meteorological drought indices (values in italic show the best meteorological drought index with the Z_{WBI})

Sub-watershed	SPI-1	SPI-3	SPI-6	SPI-9	SPI-12	SPI-24	PDSI	PHDI	WPLM	Z-index
Enipeas	0.47	*0.86*	0.81	0.67	0.58	0.38	0.78	0.73	*0.80*	0.71
Farsaliotis	0.44	*0.84*	0.80	0.68	0.57	0.37	0.80	0.73	*0.81*	0.70
Kalentzis	0.35	0.76	*0.85*	0.80	0.75	0.60	0.80	0.80	*0.85*	0.58
Kousbasaniotis	0.21	0.46	0.65	0.73	*0.77*	0.77	0.63	0.63	*0.64*	0.35
Litheos	0.52	*0.91*	0.82	0.70	0.61	0.44	0.76	0.70	*0.78*	0.73
Mesdani	0.59	*0.86*	0.70	0.56	0.49	0.38	0.76	0.70	*0.81*	0.71
Neoxori	0.46	*0.86*	0.84	0.76	0.69	0.49	0.79	0.75	*0.82*	0.70
Pamisos	0.49	*0.81*	0.74	0.62	0.55	0.44	0.76	0.75	*0.80*	0.63
Pinios	0.45	*0.83*	0.81	0.68	0.60	0.41	0.77	0.72	*0.78*	0.68
Pliouris	0.49	0.79	*0.92*	0.89	0.85	0.64	0.82	0.77	*0.81*	0.75
Portaikos	0.62	*0.82*	0.64	0.52	0.46	0.37	0.73	0.69	*0.78*	0.72
Sofaditis	0.41	*0.76*	0.73	0.64	0.56	0.39	0.67	0.60	*0.67*	0.62
Theopetra	0.49	*0.87*	0.85	0.76	0.69	0.52	0.80	0.76	*0.82*	0.71
Titarisios	0.46	0.80	*0.81*	0.74	0.64	0.51	0.77	0.73	*0.80*	0.67

Drought episodes have been identified for the study drought indices. Table 4 presents the identified drought episodes (drought index value $\leq$-1) and their respective drought classes for the period Oct 1960 - Sep 2002 for the water balance drought index and the best meteorological drought index identified by the previous analyses (one timescale for SPI and WPLM for Palmer indices). Average drought episode duration is also shown. These results show that the water balance derived drought index is capable to quantify drought episodes and duration identified by the meteorological drought indices. Overall, frequent moderate and severe droughts in Pinios river subwatersheds are identified and have small duration. However, extreme drought episodes are present in all subwatersheds except Pamisos watershed and have larger durations. For example in Enipeas catchment the extreme drought episode identified by Z_{WBI} in the hydrologic year 1976-1977 has duration of 9 months (Dec76-Aug77), drought severity of -2.75 and maximum drought peak of -3.68. The same event has identified as extreme by SPI-3 and WPLM with 7 (Dec76-Jun77) and 20 (Jan77-Aug78) months duration, respectively. Similar results are observed in quantifying drought episodes at the other Pinios river subwatersheds. Drought duration identified by Z_{WBI} is quite similar with the SPI identified duration whereas the Palmer indices overestimate drought event duration. Hence, SPI is more capable than WPLM in depicting hydrological droughts. This conclusion is based on the responsiveness of the SPI to emerging precipitation deficits at shorter time scales (e.g. 3 months). Thus, the SPI recognizes moisture deficits more rapidly than the PDSI, which has a response timescale of approximately 8-12 months (Heim, 2002).

Table 4. Identified drought episodes for the study drought indices in all Pinios river subwatersheds

Subwatershed	Drought Index	Moderately dry Episodes	Severely dry Episodes	Extremely dry Episodes	Total Dry Episodes	Average drought duration (months)
Enipeas	SPI-3	20	9	3	32	2.4
	WPLM	14	1	1	16	4.2
	Z_{WBI}	15	8	2	25	2.4
Farsaliotis	SPI-3	20	9	1	30	2.7
	WPLM	16	1	1	18	4.3
	Z_{WBI}	17	5	3	25	2.8
Kalentzis	SPI-6	13	7	1	21	3.6
	WPLM	10	4	0	14	6.9
	Z_{WBI}	6	8	2	16	4.1
Kousbasaniotis	SPI-12	9	2	0	11	6.9
	WPLM	9	1	0	10	7.6
	Z_{WBI}	7	0	1	8	8.1
Litheos	SPI-3	17	12	1	30	2.8
	WPLM	12	2	1	15	4.7
	Z_{WBI}	16	8	1	25	3.0
Mesdani	SPI-3	16	12	4	32	2.4
	WPLM	15	6	0	21	3.7
	Z_{WBI}	21	8	1	30	2.7
Neoxori	SPI-3	22	10	3	35	2.3
	WPLM	8	3	0	11	7.7
	Z_{WBI}	12	5	1	18	4.1
Pamisos	SPI-3	21	16	1	38	2.1
	WPLM	16	3	0	19	4.1
	Z_{WBI}	27	9	0	36	2.4
Pinios	SPI-3	22	12	0	34	2.5
	WPLM	12	3	0	15	5.8
	Z_{WBI}	20	5	1	26	2.9
Pliouris	SPI-6	11	6	1	18	4.7
	WPLM	7	2	0	9	12.4
	Z_{WBI}	12	8	1	21	3.7
Portaikos	SPI-3	26	10	3	39	2.1
	WPLM	16	4	0	20	3.7
	Z_{WBI}	29	7	3	39	1.9
Sofaditis	SPI-3	23	11	1	35	2.2
	WPLM	8	3	1	12	5.9
	Z_{WBI}	15	5	2	22	2.3
Theopetra	SPI-3	21	10	2	33	2.5
	WPLM	8	1	1	10	9.5
	Z_{WBI}	14	6	2	22	3.4
Titarisios	SPI-6	12	7	1	20	3.7
	WPLM	13	1	1	15	5.0
	Z_{WBI}	17	3	2	22	3.4

5. CONCLUDING REMARKS

This study evaluated hydrological droughts using a water balance derived drought index. The hydrological drought severity was evaluated from the standardized outputs of the monthly UTHBAL conceptual water balance model in major subwatersheds of Pinios river basin with

varying geomorphologic characteristics and high complexity of hydrological processes with a marked seasonal and interannual variability. The results show that frequent moderate and severe hydrological drought episodes are present in all Pinios river subwatersheds The water balance derived drought index was compared with two well-known meteorological drought index families, the SPI at multiple time scales and the Palmer four indices, in depicting hydrological droughts. SPI time scales of 3-month and 6-month and the Weighted Palmer Index (WPLM) represent better hydrological droughts for all study watersheds irrespectively to their area, geophysical, and hydroclimatic characteristics since are satisfactorily correlated with the water balance index. The assessment of hydrological drought severity was quite successful with all examined hydrological and meteorological drought indices. However, the response of meteorological drought indices to the identified historical drought episodes was quite variable. Different results were obtained in the evaluation of hydrological drought duration whereas SPI simulates satisfactorily hydrological drought duration and the Palmer drought indices fail to represent satisfactorily drought episode duration. The SPI is simpler than the PDSI in calculation and is more spatially consistent. It can be used along with the water balance derived drought index in risk and decision analyses, if the user knows exactly what operational timescale is needed. The choice of indices for drought monitoring in a specific area should eventually be based on the quantity of climate data available and on the ability of the index to consistently detect spatial and temporal variations during a drought event. The above findings could be useful for forecasting and monitoring hydrological drought severity and in developing a drought preparedness plan in the region.

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A MULTICRITERIA PREPAREDNESS PLANNING FOR EACING WATER SCARCITY

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Abstract: The study presents a multicriteria method incorporating a fuzzy set approach and the 0/1 programming for selecting the most appropriate solution for facing water shortage in a water system under a set of objectives and constraints. The proposed method includes also the right of veto which can be utilised under certain extreme conditions or for securing commensurate solutions. The proposed method was applied for devising the technical component of the rational preparedness plan against long term water scarcity in the island of Naxos, Cyclades (Greece).

Key words: Fuzzy sets, multicriteria methods, water resources management, water scarcity, preparedness plan

1. INTRODUCTION

The Strategic Preparedness Plan for facing droughts and water scarcity involves many interest groups and a significant number of criteria so that future actions will be efficient and socially and environmentally acceptable. The technical component of such a Preparedness Plan should be based on a flexible, transparent and comprehensive multicriteria framework so that decision makers, stakeholders and the public could be convinced for a fair and sound analysis of all viable alternatives.

One widely used family of multicriteria methods is the family of *Outranking* methods. These methods focus on pair wise comparison of alternatives while they are applied to discrete choice problems (Roy 1996). The outranking approach differs from the value function approach in that there is no underlying aggregative value function. The output of an analysis is not a value of each alternative, but an outranking relation on the set of alternatives. The most widely used outranking methods are the Electre family methods and the Promethee family methods. Although outranking methods are more complicated than others applied to MCDM problems, they are robust tools to address the conflicting nature, the uncertainty and the different nature of the criteria, which appear very often through the multicriteria analysis.

On the other hand, it is known that the islands (e.g. Naxos, Cyclades) are characterized by small river basins and small groundwater aquifers. Consequently, the final solution may consist of a set of various different water development scenarios even if the entire region is not significantly large. This set of actions composes the final solution. In this paper a new methodology was proposed to deal with this complex multicriteria planning problem by using a fuzzy outranking process assisted by the 0/1 linear programming. The methodology is applied for planning against water scarcity in the island of Naxos, Cyclades.

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In short the proposed methodology consists of two phases. In the first face water development scenaria are compared using a multicriteria approach whereas in the second phase 0/1 programming is used for reaching the final decision. The two phases are described in the following paragraphs.

2. APPLYING FUZZY LOGIC TO OUTRANKING PROCESS

In general, the outranking methods in MCDA are based on binary comparisons (outranking relations), while incorporating many fuzzy logic concepts. In the aid of the fuzzy logic and sets, the *strict preference* (P) and *indifference* (I) are defined as fuzzy sets in order to express the granularity of the preference. Furthermore, the *strict preference* and *indifference* are founded as a function of *outranking relation* (S)*, which is a fuzzy binary relation*.

S: the outranking relation. The statement aSb means "a is not worse than b" (i.e a is at least as good as b). The assertion aS_jb means only that either the score $a_j - b_j$ is positive or the difference $b_j - a_j$ is too small to justify a preference in favour of b on criterion j.

Finally, the construction of an overall relation H (where H may be denoted either the strict preference or the indifference or the outranking relation) always incorporates the comprehensive concordance and non-discordance relation in formulating a comprehensive preference relation. Both the concordance coalition and the non-discordance coalition are defined axiomatically by using the fuzzy aggregators (e.g. Perny and Roy 1992, Fodor et al. 1998, Tsakiris et al., 2009). Therefore, the following proposition may be proposed in order to achieve the overall preference relation:

A proposition of the type aHb holds if and only if the coalition of attributes or criteria in agreement with this proposition is strong enough and if there is no significant coalition discordant with it. This principle can be expressed in the following logical equation:

$$H(a,b) \equiv (C_H(a,b) \wedge N(D_H(a,b))). \tag{1}$$

in which:

$C_H(a,b)$: The concordance principle begins by asking to what degree each criterion or attribute or voter agrees with the statement $H \in \{S,P,I\}$ for the pair (a, b). These answers are aggregated to obtain an overall index C_{aHb} measuring the overall agreement with the proposition aHb (over all criteria). The resulting relation C_{aHb} is a valued comprehensive concordance relation.

$D_H(a,b)$: The comprehensive discordance relation measures the degree to which there exists at least one discordant criterion (respect of the minority)

N: a strict fuzzy complement

Usually the fuzzy intersections are used to express the $\wedge$ operation. The way of construction the overall concordance coalition and the overall non- discordance coalition is presented elsewhere (Tsakiris and Spiliotis, 2008).

The principle of construction of the overall H reration (Eq.1) is named as concordance and non-discordance principle. According to this principle, the overall evaluations of the alternatives take into consideration both the right of majority together with the right of respect of the minority (veto-right).

In this paper the overall outranking relation S_j applied in the Electre III method, the min fuzzy intersection and the classical complement were selected to express the overall outranking relation (Eq.1) between two alternatives aSb:

$$S(a,b) = min\{(C_S(a,b),(1 - D_S(a,b))\} \tag{2}$$

It should be noted that the use of min intersection instead of other fuzzy intersections such as the algebraic product, gives a less compensative character in the decision.

Let A the set of all alternatives. To simplify the decision process the use of scoring function was proposed in this study to utilize the overall outranking relations (Eq.2). The scoring functions evaluate the extent to which each alternative dominates over all the other elements in A and/or it is not dominated by anyone of the elements belonging to A. In this process various scoring functions could be adopted. In this paper the net flow was adopted, which is similar to Promethee II method (Brans and Vincke 1985), due to the fact that it incorporates both the sense of dominant and domination (Fodor et al. 1998):

$$v(\alpha, A, S) = 1/|A| \sum_{b \in A} \left[S(\alpha, b) - S(b, \alpha) \right] \tag{3}$$

in which:
A: is the set of all alternatives
$v(\alpha, A, S)$: the scoring function of the alternative α with respect to the outranking relation S.
S(α, b): the outranking relation which indicates that the alternative α is not worse than alternative b
b: an arbitrary alternative which belongs to set of all alternatives A.

The applications of the scoring function in the outranking relations provide more general and comprehensive consideration than the Promethee method since the Promethee method does not incorporate the sense of veto (non- discordance principle). Furthermore the proposed method can be expanded to incorporate fuzzy data, the use of several fuzzy aggregators in the multicriteria process and the use of other scoring functions instead of the one presented by Eq.3.

Moreover, the direct comparison of various final solutions with the application of multicriteria method is of no practical value since each final solution is comprised of a number of actions with each of which having different level of acceptance when compared to the other alternatives that belong to the same final solution.

3. 0/1 PROGRAMMING FORMULATION

In the second phase, a 0/1 programming method was used to devise *a final set of actions* (final solution) which potentially affect water supply and demand provided that the objective function was generated according to the proposed multicriteria analysis.

Furthermore, with the use of mutually exclusive constraints, the time scheduling of the proposed alternatives was achieved. It is proposed that each action or a set of actions at a different time period is a different alternative compared with the same action or set of actions at another time period. Actually the same set of actions at different time periods could result in different cost and socio-economic and environmental impacts.

Let the set of actions i at time horizon j: α_{ij}

Then:

$$X_{ij} = \begin{cases} 1 \text{ if } \alpha_{ij} \text{ is selected} \\ 0 \text{ if } \alpha_{ij} \text{ is not selected} \end{cases} \tag{4}$$

The multicriteria problems solved using the scoring function method (Eq.3) consist of a finite set A of i×j possible alternatives that are evaluated on k criteria. Therefore, the comprehensive criterion is developed to correspond to each of the k criteria by using the scoring function:

$$max \left\{ \sum_{j=1}^{J} \sum_{i=1}^{N} v(\alpha_{ij}, A) X_{ij} \right\} \tag{5}$$

In selecting the combinations of options that will efficiently meet competing socio-economic and environmental objectives however, limitations on resources such as available budget, geographical dispersion requirements and others, had to be considered. These constraints were expressed in terms of the decision variables X_{ij}.

$$\sum_{i \in I(d)} V_{ij} X_{ij} \geq D_j \qquad \text{(Demand constraints)} \qquad (6)$$

in which

$I(d)$ is the set of actions, which can fulfill the water demand D

V_{ij} the amount of water which is avialable by the action i within year j.

D_j the total amount of water demand which is requested within year j.

The division of the water supply system to the natural water system (e.g. watershed or groundwater) is proposed, in order to express the constraint of water availability.

$$\sum_{i \in I(w)} V_{ij} X_{ij} \leq WA_j \qquad \text{(Water availability constraints)} \qquad (7)$$

in which

$I(w)$ is the set of alternatives, which belong to the water system w.

V_{ij} the amount of water that is offered by the action i within year j.

WA_j the total amount of water availability which could be developed from the water system within year j.

Furthermore, the use of mutually exclusive alternatives could enhance the proposed multicriteria tool. For example, the following constraint may be adopted in order to ensure that each project i can be constructed only once.

$$\sum_{j} X_{ij} = 1 \qquad \text{(Time incompatibility)} \qquad (8)$$

Similar equations can be used to express the constraints of the geographical dispersion or the geographical incompatibility can be used in order to express the incompatibility between alternatives that contain others (Tsakiris and Spiliotis, 2008).

4. THE WATER SYSTEM

Naxos is one of the major islands of Cyclades with an area of 428 Km^2. Naxos is 108 miles from Piraeus. The climate of the island can be characterized as semi-arid. Administratively the island of Naxos is divided in two Municipalities (Naxos and Drymalias). The population of the island, according to the census 2001, is estimated 17646 inhabitants. The main economic activities of the island are mostly related to the municipality of Naxos. From 1981 until today a rapid development of tourism has been observed.

The increasing water demands together with the absence of vital water resources development projects cause a long standing water scarcity, which creates serious impacts both on the society and the environment. Consequently, a competition between the agricultural and the touristic use of water holds impeding economic development in both sectors. (Epp. et al., 2007).

For the purpose of the analysis of this paper the island is divided into two parts because of its different terrain, hydrogeological, economic development growth, water consumption and administrative patterns

(Fig.1). The analysis which follows applies to the western part of the island belonging mainly to the municipality of Naxos.

A set of alternatives (water development actions) addressing the municipal, touristic and the agricultural demands were developed, including the reduction of the agricultural demand, the construction of a dam or off-stream reservoirs and the utilisation of groundwater for time horizons 2011 and 2021.

The selection of the location of reservoirs is based on the water potential of the corresponding basins and the technical viability of the proposed alternatives (Kaplanidis 1984 & Paritsis 2001). Therefore, the proposed actions of exploitation the surface water was selected taking into consideration the precipitation pattern, the hydrogeological background, the slopes and the size of each basin. The streamflow data were calculated using a simple rainfall- runoff model which was applied to a number of small river basins. It should be mentioned that where the marble backgrounds appear then the water balance is not sufficiently reliable. For these reasons the main surface water development actions are nested to the North – Eastern part of the Naxos Island (Kaplanidis 1984). The rainfall – runoff model used is a simple water balance model which uses monthly data and is described in detail elsewhere (Tsakiris and Spiliotis, 2008).

The construction of the off-stream reservoirs at Faneromeni, Eggares, Saint Thaleleos and Kinidaros have been investigated in the past. For this purpose, the mean streamflow volume of the months from October to March were obtained. The normal distribution was selected for this purpose. The proposed off-stream reservoirs are planned for municipal and touristic use.

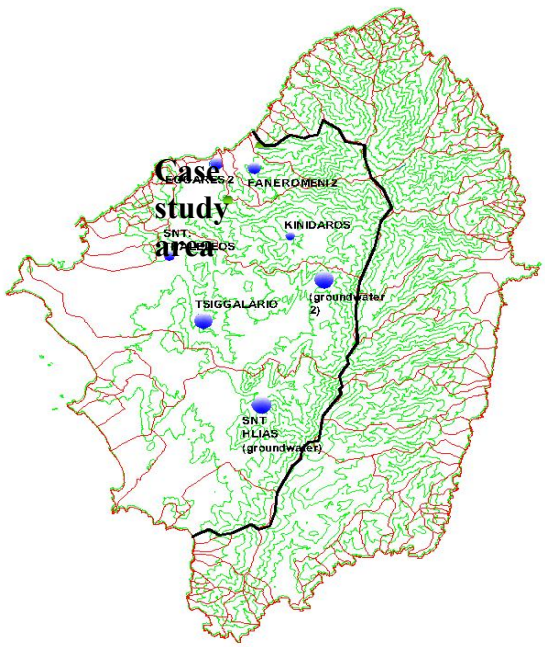


Figure 1: The proposed study area in the island of Naxos.

A reservoir –balance model was developed with the use of Matlab R2008a in order to analyze the response of the existing dam/reservoir of Faneromeni and the proposed dam/reservoir of Tsiggalario. Moreover, an ecological discharge was considered to protect the downstream ecosystem. A critical point of view is the incorporation of the risk for fulfilling the water demands. Having considered that a very low probability of failure is acceptable for the municipal demand, the operation of the reservoir of Faneromeni was simulated and finally the safe available quantity of water from Faneromeni was taken as the volume with a very low probability of failure. It is shown that for the conditions of the Agean islands, the available surface water potential is only 50 – 70% of the mean annual runoff even if the capacity of the dam is substantially bigger than the mean annual streamflow at the location of the dam (Nalbantis and Tsakiris, 2006). Similarly we examine

the available water potential of Tsiaggalario dam for agricultural use supposing that the acceptable probability of failure is 0.10.

Table 1: Estimation of the score of alternatives constructed until 2011 (j=1)

	Criteria					**Additional Available annual volume of water (m^3)**	**Multicriteria Ranking**
Alternatives 2011	*alternatives*	Water development (veto)	Economic	Environmental	Water efficiency		
		W1=0.4	W2=0.225	W3=0.225	W4=0.15		
Dam of Faneromeni		existing	existing	existing	existing	available 1,100,000 (municipal)	-
Off-stream reservoir of Eggares		existing	existing	existing	existing	available 500,000 (agricultural) 150,000 (municipal)	-
Groundwater 2		existing	existing	existing	existing	available 350-450,00 (municipal)	-
Reduction of demand	X_{11}	0.000	1.000	1.000	1.000	1,000,000 (agriculture)	-0.313
Dam of Tsiggalario	X_{12}	1.000	0.000	0.300	0.600	2,100,000 (agriculture)	0.287
Eggares+Eggares 2 +Faneromeni	X_{13}	0.791	0.780	0.400	0.400	500,000 (municipal)	-0.151
Faneromeni+ Faneromeni 2 +Eggares	X_{14}	0.791	0.780	0.600	0.500	500000 (municipal)	-0.017
Faneromeni+ Faneromeni 2 +Eggares+Eggares_2	X_{15}	0.940	0.559	0.600	0.500	1,000,000 (municipal)	0.296
Kinidaros+Eggares+ Faneromeni	X_{16}	0.672	0.917	0.400	0.300	100,000 (municipal)	-0.294
Kinidaros+Eggares+ Eggares_2+Faneromeni	X_{17}	0.821	0.696	0.400	0.300	600000 (municipal)	-0.160
Kinidaros+Eggares+ Eggares_2+Faneromeni +Faneromeni_2	X_{18}	0.970	0.476	0.300	0.300	1,100,000 (municipal)	0.120
Snt Hlias	X_{19}	0.900	0.900	0.900	0.600	500,000	0.569
Snt Thaleleos	X_{10}	0.133	0.853	0.400	0.800	400,000	-0.539

Table 2: Estimation of the score of alternatives constructed until 2021 (j=2)

	Criteria					**Additional Available annual volume of water (m^3)**	**Multicriteria Ranking (continued)**
120 Alternatives 2021	alternatives	Water development (veto)	Economic	Environmental	Water efficiency		
		W_1=0.4	W_2=0.225	W_3=0.225	W_4=0.15		
Dam of Faneromeni		existing	existing	existing	existing	*available* 1,100,000 (municipal)	-
Off-stream reservoir of Eggares		existing	existing	existing	existing	available 500,000 (agricultural) 150,000 (municipal)	-
Groundwater 2		existing	existing	existing	existing	available 350- 450,000 (municipal)	-
Reduction of demand	X_{21}	0.000	1.000	1.000	1.000	1,000,000 (agriculture)	-0.313
Dam of Tsiggalario	X_{22}	1.000	0.324	0.300	0.600	2,100,000 (agriculture)	0.309
Eggares+Eggares 2 +Faneromeni	X_{23}	0.791	0.851	0.400	0.400	500,000 (municipal)	-0.078
Faneromeni+ Faneromeni 2 +Eggares	X_{24}	0.791	0.851	0.600	0.500	500000 (municipal)	0.056
Faneromeni+ Faneromeni 2 +Eggares+Eggares_2	X_{25}	0.940	0.702	0.600	0.500	1,000,000 (municipal)	0.351
Kinidaros+Eggares+ Faneromeni	X_{26}	0.672	0.944	0.400	0.300	100,000 (municipal)	-0.275
Kinidaros+Eggares+ Eggares_2+Faneromeni	X_{27}	0.821	0.795	0.400	0.300	600000 (municipal)	-0.098
Kinidaros+Eggares+ Eggares_2+Faneromeni +Faneromeni_2	X_{28}	0.970	0.646	0.300	0.300	1,100,000 (municipal)	0.168
Snt Hlias	X_{29}	0.900	0.920	0.900	0.600	500,000	0.585
Snt Thaleleos	X_{20}	0.133	0.901	0.400	0.800	400,000	-0.504

According to the hydrological considerations of the Naxos island a set of groundwater development works is also proposed at the location of Saint Hlias in the eastern part of Naxos where a significant marble background exists (Paritsis 2001). The selection of the location of Saint

Hlias for the groundwater exploitation add two additional advantages. Firstly, the groundwater reservoirs are less sensitive to droughts and climatic changes over the years and secondly all the proposed surface water development works are located in the northern part of the island and are far away from the area of water consumption.

A set of criteria was selected in order to describe several aspects in the decision-making phase. The alternatives and the evaluation matrix of the criteria selected are presented in Table 1 and 2. The right of veto was applied to the criterion of water development providing a more social - regional aspect in the problem of muticriteria ranking. Another program in Matlab was developed to produce the multicriteria ranking based on fuzzy sets and logic (Eq.1 – 3) incorporating the use of the right of veto.

The decision variables were selected to be 0-1 numbers (Eq.4). Several constraints were applied to prevent the simultaneously selection of mutually exclusive alternatives (Eq.8), while the objective function was based on the proposed multicriteria process (Eq.5). The constraints of water availability (Eq. 7) and water demand (Eq. 6) in 2011 and 2021 were also considered.

A critical point in this analysis was the water demand constraints. Especially, concerning the agricultural demand, several considerations may be adopted.

Therefore, the construction of Tsiggalario dam to fulfill the agricultural demands and the utilization of the Saint Ilias aquifer for fulfilling the municipal demands, were selected for the horizon of 2011. The construction of two off - stream reservoirs was additionally proposed for the horizon of 2021. The final solution (total set of actions) could be characterized as a rational and balanced solution.

After this reconnaissance selection, an analytical simulation may follow in order to study the system with higher accuracy and more the precise description of the design parameters.

The use of veto right does not have as consequence the annihilation of the compensative evaluation. As an example, although the alternatives x_{15} and x_{18} have a different evaluation of criteria finally the x_{15} alternative has a higher value of scoring function since it has a higher evaluation from the majority of criteria. In fact the addition of small off- stream reservoir of Kinidaros in the system of Faneromeni- Eggares basins would offer small aid of available water with a decrease towards the economic criterion as well as the environmental and the water efficiency criteria.

A very important decision is the choice of criteria they could practice veto. It was concluded that the use of the veto - right differentiates drastically the multicriteria analysis. It differentiates the objective function and consecutively, the final decision. As an example the choice of significant reduction of agricultural consumption of water does not predominate over the construction of the dam of Tsiggalario due to the evaluation of the water development criterion in which veto right is exerted in the mutual binary evaluation process.

It was also observed, that the final decision is relatively stable even if changes in the constraints are imposed. The final decision could be altered in the case of significant change of the forecast of the municipal water consumption.

5. CONCLUSION

A fuzzy multicriteria methodology was presented as the mechanism for long term decisions in the Preparedness Plans of a region to face water scarcity. The proposed methodology is comprised of two steps: the fuzzy outranking method and the 0/1 programming.

The methodology incorporates the analysis with respect to concordance and non- discordance principle (right of veto) using fuzzy data as it is common when Preparedness Plans are devised.

One important innovation of the proposed methodology lies in the fact that time – dependent alternatives can be analyzed comprising of various simultaneous actions in a number of adjacent elements (small watersheds) of the entire water system of the region under study.

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INDUSTRIALIZATION, THE MAIN FACTOR OF SHORTAGE AND POLLUTION IN THE KAROON RIVER, SW IRAN

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Abstract: Karoon is the largest river in Iran, Which covers about 30 percent of the entire surface water resources of the country. This waterway originates from the central parts of the Zagros Mountains (SW Iran) and after 890 km journey joins the Persian Gulf. This river system supplies the water demands of 16 cities, several villages, 300000 hectares of agricultural lands and several hydropower plants. Meanwhile, the Karoon River plays a key role in development of the region, where the ancient civilizations of Iran flourished and bricked up about 4000 years ago. Unfortunately, in recent decades inter-basin water transfer and repeated sever periods of drought along with an unsatisfied demand for water due to overpopulation and industrialization have resulted in considerable decrease in flow discharge of the river. Furthermore, the discharge of untreated urban wastewater and industrial effluent, the drainage from agricultural lands, and the uncontrolled discharge of solid wastes directly into the Karoon River and its tributaries have considerably degraded the water quality of the river. These have caused a gloomy future for numerous industries and more than four million inhabitants of the region. Therefore, the related organizations and all of the water users altogether are required to design some applicable policies for reforming this unfavourable conditions and deliverance the river.

Key words: Pollution, Water resources management

1. INTRODUCTION

The case study is focused on the Karoon River in Ahvaz city, Khuzestan province, Southwestern Iran (figure 1). This region situated between latitudes 31º, 19´ N and longitudes 48º, 42´ E. It has a dry and hot climate with annual precipitation and evaporation about 250 mm and 1900 mm, respectively (Rouhipour, 2006). The temperature fluctuates between 0-50ºC in different seasons. Ahvaz is the center of the province and its population is about 1.5 million people, according to preliminary data from the decennial census conducted in late 2006. From geological point of view, it has been covered with siliciclastic rocks and evaporites of Fars group (Miocene-Pliocene) and Quaternary alluviums. (Motiei, 1993).

The Karoon waterway with an average annual discharge of 643 m^3 sec^{-1} flows through this county. Khuzestan is one of the most important provinces of the country with a high potential for agricultural and industrial development. This has contributed considerably to the pollution of the environment. In addition, near one-third of Iran's total surface water of 94 billion m^3 is flowing into the Khuzestan, but ironically, most of its inhabitants (more than four million people) in central and southern portions face with the problem of water supply (Afkhami et al., 2007). All these factors, either by themselves or in interaction, place an increasing pressure on available water resources, so producing serious dysfunctions in the water ecosystem and harmful effects on human health as well.

It is obvious that, Water as a basic factor in economic and social development, should always be taken into consideration in national and regional planning. Therefore, studying the issues concerning its pollution sources and their management under the present development conditions are an urgent and unavoidable affair. The purpose of this research is to evaluate the quantity and quality of the Karoon River water, in Ahvaz city during 40 recent years. To achieve the intended objective, in the first step all of the statistical data (e.g., annually flow discharge and TDS average) were gathered by referring to the responsible organizations. After that, the main effective agents on the Karoon River such as different factories and dams were considered. Finally, to amend the existing situation some reforming suggestions are made.

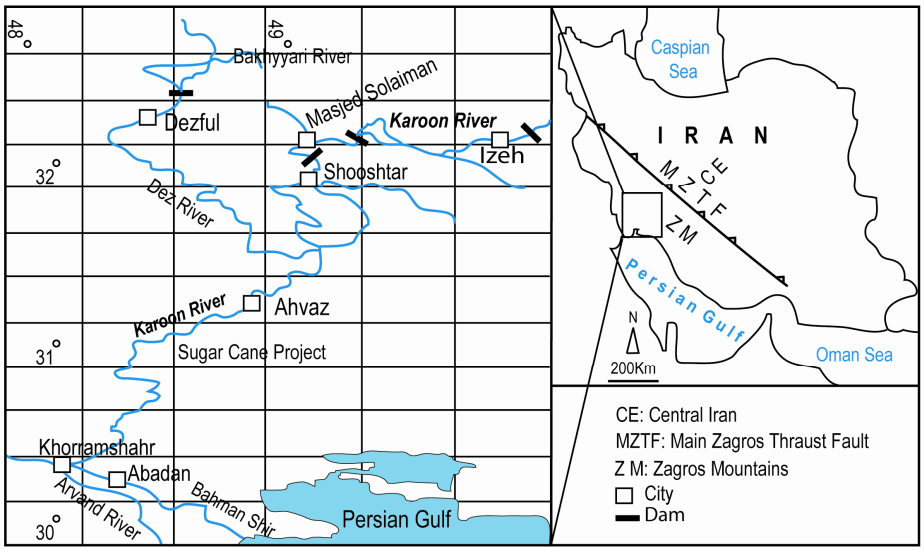


Figure 1. The Karoon River system in Khuzestan province, SW Iran.

Discussion and results

Iran is one of the few countries in the world with a long and extensive background on establishing ancient civilizations, which the water scarcity and precipitation have never caused to freeze the civilization and humanization formation. This is because ancients have realized the importance of an organization to supervise the irrigation and water conveyance affairs. As the result, they have always exploited technology and precise engineering to control water, and utilize it in the best way to flourish the economical, cultural and social statuses of the country. Although there are records of the water-establishments in some of the historical documents, rarely did it mention to the supervision organizations of those establishments. The most important establishments and water conveyance networks have been recognized in southern part of the territory. One of these historical water establishments is located in Khuzestan province where the ancient civilizations of Iran flourished and bricked up about 4000 years ago. This historical complex consists of some dyke, bridge, canals, castle, weir and watermills located on the Karoon banks in the city of Shooshtar (figures 1 and 2).



Figure 2. Weir and watermills (now waterfalls) at the Karoon River upstream, Shooshtar (related to 4000 years ago).

As mentioned above, the problem of water supply has been a constant preoccupation in Iran since the beginning of the country's history. In addition, the socio-economic developments in Iran over

the last few decades have brought about considerable changes in natural and human environmental conditions, particularly in population structure, life pattern, power sources, industrial and agricultural production and transport. One of the most considerable effects due to these is overpollution and water deficiency in the Karoon River.

Karoon River with the length of 890 Km and catchments area of 67000 Km^2 is the longest river in the country, which flows along many industrial and agricultural zones. The Karoon River begins high in the Zagros Mountains and flows in a general westerly direction through a descending series of anticline ridges and synclinal valleys before emerging on to the Khuzestan Plain. The Karoon River passes through about 16 cities, and continues southward to join Tigris and Euphrates to form a large shipping waterway called Arvand River (Shiva, 2002). Finally, it conveys to the Persian Gulf. Figure 1 shows the location of the Karoon drainage area. The width of this river in along path is different. In mountainous part is about 25-40 m and in plain near Ahvaz city 250-400 m and around Khorramshahr it reaches 350-700 m (Javaheri et al., 2008; Musavi-Jahromi and Golabi, 2008). The depth of Karoon River in mountainous area ranges about 4-5 m and around Ahvaz reaches 5-7 m.

As can be seen from table 1, the maximum river width in the research zone was 940 m in 1955 and 710 m in 2006. The average width of the river along the research area was 301 m for the year 1955, 246 m in 2006 and the minimum values of river width were 62, and 84 m for the years 1955 and 2006, respectively. It seems that the Karoon river width and depth has been clearly decreased with time during the considered periods (figures 3 and 4).

Table 1. The geometric parameters of the Karoon River, 1955-2006 (Javaheri et al., 2008 with some changes).

Year / Width	1955	1964	1990	1991	1998	2003	2006
Max.	939.8 (m)	828.5	741.1	862.7	689.7	757.3	710
Min.	62	88.4	107.2	109.3	103.9	94.7	83.6
Ave.	300.8	275.6	288.8	313.5	261.8	265.7	245.7



Figure 3. The Karoon River in 1952 (photo by Green, Allen C., 2 Feb. 1952).

Satellite images and direct site visit show that many small and large islands have been formed along this river, mostly at the north and south of Ahvaz city during the recent years (figures 4 and 5). These islands not only have reduced the conveyance capacity of the river but also increased the flood risk around the city. The reduction of water flow at downstream and deposit trapping by numerous bridges in recent years are responsible for sedimentation these islands.



Figure 4. The Karoon River in 22 Nov. 2008 (the same location in fig. 4).



Figure 5. Satellite image showing the Karoon River and islands (from www. Wikimapia. Com, 15 Nov. 2008).

The results made show that the flow discharge and quality parameters of the Karoon River have been considerably degraded as well. Figures 6 shows that in 1968 the TDS and discharge of the river were 1137 mg/l and 750 m^3/s respectively. However, these amounts have been changed to 1555 mg/l and 378 m^3/s in 2008.

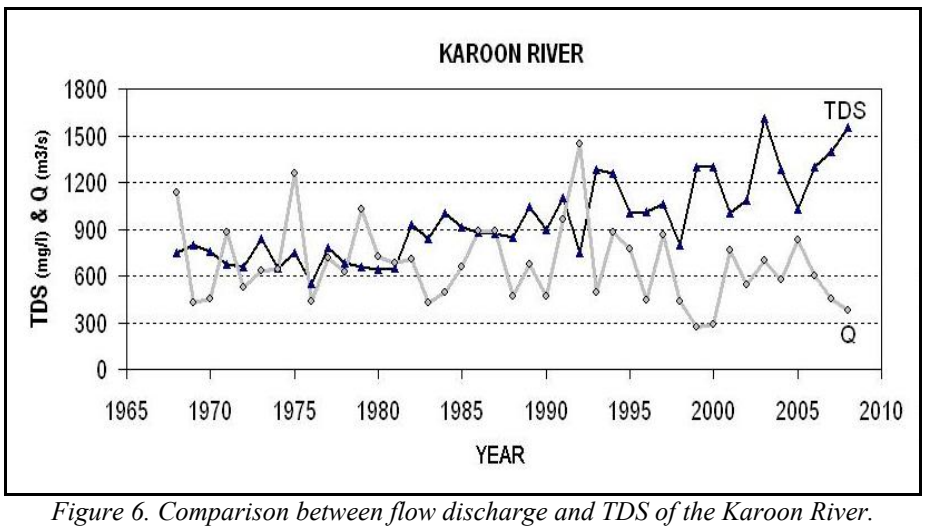


Figure 6. Comparison between flow discharge and TDS of the Karoon River.

Apart from these, other quality indices such as over salinity and high concentration of heavy metals, sulphate, nitrate and chloride have resulted in, the Karoon River to meet with very sensitive and urgent conditions. For instance, Digomanoline et al., (2004) have reported that the amounts of some heavy metals (e.g., Ni, Cr and Cu) in the waterway are much more than the respective standard. This can significantly affect the development of the region and cause a gloomy future for more than four million population of the county. Moreover, our future generations are to face a serious water shortage during the coming decades. Pollution of water resources will certainly make this situation worse.

Conclusions and suggestions

Making a comparison between water quality variables of the Karoon River (e.g., TDS>1550 mg/l) and recommended standards for water show that water in this river is very far from normal conditions. In other words, it has been deteriorating at an alarming rate due to the discharge of untreated municipal wastewater (Figure 7), agricultural drainage and untreated industrial effluents. Even, with the existing conditions, it is not suitable for agriculture.

The main agents responsible for the Karoon River pollution and water deficiency in recent few decades has been as follows:

1-Water transfer to the neighbouring catchments.

2-Repeated sever unwanted droughts and irregular distribution of 255 mm annual precipitation.

3-Constraction of several huge dams and hydropower plants.

4-Dry and hot climate and high temperature resulting in sever evaporation, specially from the dams reservoir.

5-Increasing water demands due to overpopulation and change of life pattern (about 1 million inhabitants of the province in 1956 to 4.5 million in 2006).

6-Agro-industrial development without ecological and environment attentions (1370 active plants with 10 billion m^3/yr water consumption and 2.4 billion m^3/yr uncontrolled discharge into the environment).

7-The Persian Gulf saline water intrusion has resulted in enormous problems for the date palms in downstream of the river (Abadan and Khorramshahr cities). These are the biggest palmgroves in the world.

8-Evaporite deposits outcropping at an extensive area resulting in water and soil over salinity.

9-Low efficiency irrigation systems.



Figure 7. The discharge of untreated domestic wastewater into the Karoon River.

The results show that among the mentioned factors, agricultural and industrial activities are the main consumers and pollutants of the Karoon water. Future development of the region can cause more crucial than today. Therefore, the related organizations and all of the water users altogether

are required to design some applicable policies for reforming this unfavourable conditions and deliverance the river. The following suggestions are finally made:

1-To perform an adequate management on demand and saving water resources.

2-Education in relation to water resources importance at all levels from the water project designer to the water user**.**

3-Industries interference with water pollution should be taken under the control and further advice.

4-Considering the role of groundwaters in water supply.

5-wastewater collection and treatment.

6-To prevent high water wastage by treatment of old water supply and irrigation systems

7-Optimum usage of fertilizers and pesticides

8-To prevent water evaporation from dams reservoir by floating solid bodies

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MODELING DISTRIBUTION SYSTEM WATER QUALITY WITH HYDROSIM MODEL

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Abstract: Simulation of the water quality in water distribution networks is an internationally accepted tool for the plan and the design of new systems and the evaluation of old ones in order to keep the drinking water clean and wholesome. Directive 98/83/EC has established specifications for the basic and preventive parameters and also, for the priority dangerous substances in drinking water. This Directive explicitly mentions that the parameter values should be maintained at the point of human consumption and not at the entry point of the network (water intake). The current valid technical specifications in Greece (Presidential Decree 696/1974) do not contain any prevision for simulating the hydraulic behavior of water distribution systems and neither for the drinking water quality analysis. Thus, while there is a long list of qualitative parameter values for drinking water, there is no institutional provision for the investigation during the design of a drinking water distribution network for the effect of the network design philosophy (network shape, simple or multiple water intake etc.) on the qualitative characteristics of the drinking water. It is well known that water quality in distribution networks is continuously changing. Therefore, the only way that these values can be examined and adjusted in advance is the simulation of the water quality in distribution networks for real or hypothetical conditions (scenarios) utilizing dynamically varying modeling programs. For this purpose the software package HYDROSIM was developed which is coded in Visual Studio .NET 2003 utilizing the source code of EPANET 2 which is a free software of Environmental Protection Agency (E.P.A.). The purpose of this research paper is the investigation of the effect of the design philosophy of a water distribution network on the qualitative characteristics of the drinking water and suggestions on improving the current specifications for the design of water networks in Greece.

Key words: chlorine decay, water age, distribution system, drinking water, water quality model.

1. INTRODUCTION

The objective of any Water Distribution System (WDS) is to provide water to the consumer in the proper quantity and pressure but also in the proper quality in terms of flavor, odor, appearance and sanitary security. Consequently each drinking water company has to maintain a very good water quality in the whole distribution system. This goal is not easily obtainable due to the geometric complexity of the network, the complexity of the network connections, the different functional regulatory systems, the temporal and spatial variations on the water demand and the reactions between the various substances contained in the water and also, the reactions between the water and the internal wall of the pipes (Wu 2006).

In practice, the combination of conventional methods of treatment and disinfection of drinking water gives very good results as far as it's quality (USEPA 2001). Traditionally chlorine is used as a disinfectant (USEPA 2001; Munavalli and Kumar 2003). However, the disinfection of drinking water with chlorine, while reducing the risk of pathogenic infections, may be a chemical threat to human health due to disinfection residuals and their Disinfection By-Products-(DBPs) (Sadiq and Rodriguez 2004). Specifically, chlorine presents some problems that make its management difficult. The Regulatory agencies establish stingent standards for the residual chlorine in the water and in any place in the network so that consumers are protected from pathogenic infections. However, chlorine decays due to decomposition in natural water with time and therefore enough quantity of

disinfectant should be added to the water system but not at a high concentration in order to avoid problems in taste, odor and creation of DBPs.

For the continuous monitoring of water quality, sampling is carried out in appropriate places which plays an important role in minimizing the risk of supplying unsatisfactory water quality for public health. However they provide only a limited picture of water quality, because the sampling is carried out at specific points, and can not be used for predicting future conditions or the execution of «what if» analysis because of its limited coverage and high cost (Wu 2006).

The complex dynamic behavior of water quality and the various physical, chemical and microbiological processes taking place simultaneously in a WDS have induced scientists to use simulation models of water quality for the investigation of temporal and spatial evolution of disinfectant substances in WDS (Clark et al. 1993; Rossman et al. 1993; Boulos et al. 1995; Islam and Chaudhry 1998; Fernandes 2002).

The purpose of this work is to investigate the effect of the design of a water supply network in the quality of drinking water and suggest improvements on the current standards for drinking water network studies in Greece.

2 THE CURRENT LEGISLATION IN GREECE

The standing technical specifications in Greece for studies of water supply systems were formed in 1974 (Preferential Decree 696/74, Articles 204 - 207) which is 35 years ago without any improvement or correction. During this period a lot of progress was made in water network design, computers, hygiene of drinking water, etc. Furthermore, these specifications, do not follow international standards and practices, as opposed to the institutional framework for water quality for human consumption, since they do not refer to the obligation of creating the operation model of the WDS and make no reference whatsoever concerning the investigation of system reliability, nor in examining the characteristics of the network (geometric form, water intake locations, hours of pumping, etc.) to ensure good drinking water quality.

Regarding human consumption water quality Drinking Water Directive 98/83/EC (DWD) is incorporated in national law by virtue of Joint Ministerial Decisions (JMD) 2600/21-06-2001 and 38295/22-03-2007. The above legislation instruments follow a parametric approach that determines the standards for the most common substances that can be found in drinking water and are based on the examination and follow-up of the conformity in the numerical models of quality of water based on the samples that they are taken with a specific process (frequency, places of sampling etc.) The inspections aim at ensuring that the network water is safe and wholesome. These instruments establish three levels of control that is to say: (a) check monitoring, the aim of which is to provide, in a regular basis, information on the organoleptic and microbiological quality of the water supplied for human consumption and information on the effectiveness of drinking water treatment (particularly of disinfection), where it is used, in order to determine whether or not water intended for human consumption complies with the relevant parametric values laid down in DWD, (b) the audit monitoring, the aim of which is to provide the information necessary to determine whether or not all of the DWD's parametric values are being complied with and (c) the additional monitoring, which is realized in special cases for specific pathogenic bacteria and micro-organisms.

However, even though the Greek legislation in force clearly defines the quality limits of drinking water and the way of inspection, in practice there are a lot of shortcomings in its application, due to: (a) lack of suitable organization and personnel of the water supply undertakings, (b) shortage of economic resources for the confrontation of expenses for water treatment and inspection and maintenance of water supply systems, (c) lack of alternative water supply sources, which forces the drinking water companies (Municipal Water and Sewerage Companies etc.) to supply the network with water which does not always meet the standards in force, (d) lack of infrastructures for the required samplings and analyses according to the national and international models etc.

With Administrative Instructions (Circular) /39511/06-04-2005 of Ministry of Health and Social Solidarity the places of sampling and laboratory tests are specified at predetermined points of the

distribution network from the water intake to the consumer faucet according to the corresponding follow-up program in which the sampling points should be reported. More specifically, for the sampling points it specifies that a percentage should be from stationary places (pump rooms, storage tanks and places with preexisting problems) and the rests should be taken accidentally from busy points (hospitals, schools, apartments etc.) and places with a high probability of pollution danger (branches, vacuum points etc.). The Joint Minesterial Decision (JMD) 5673/1958 establishes: (a) as basic method of disinfection of drinking water the chlorination (Article 2 (1)), (b) the imported quantity of chlorine in the network should be such that the residual of free chlorine is 0.20 mg/L (Article 3 (1)) and (c) the chlorination should be carried out so that its action is at least 20min from its introduction in the network till the uptake of water from the consumer (Article 3 (4)). It is marked that the newers JMD 2600/21-06-2001 and 38295/22-03-2007 does not specify limits for the residue of free chlorine.

3. HYDROSIM SOFTWARE

Using a well-designed hydraulic and water quality model may be an important approach for simulation of hydraulic and dynamic behavior of water quality for all elements of a water network, since the qualitative parameters of water in a drinking water network are time dependent, since water supply and pressures are constantly changing. Thus, the only way to examine in advance and regulate with safety these parameters is the simulation of the water quality of the network using dynamic models (Liou and Kroon 1987; Clark et al. 1993; Rossman et al .1993; Boulos et al.1995; Islam and Chaudry 1998; Fernades 2002). The use of these models in a real WDS makes it possible to predict the concentrations of disinfectant substances in each point. In this work HydroSim (Vantas 2007) was applied, which has developed in a Visual Studio. NET 2003 and for the calculations the source code of EPANET2 version 2.00.10 (EPANET 2 Programmer's Toolkit) was incorporated, which is written in ANSI C, was developed by USEPA and is software whose source code is distributed free (public domain software) (Rossman 2000). In EPANET2 for hydraulic calculations followed the method of Todini and Pilati (1987), while the analysis of water quality, method TDM (Lagrangian Time - Driven Method) (Liou and Kroon 1987). HydroSim performes dynamic simulation of hydraulic behavior of water and change the quality of a pipeline under pressure, which may consist of pipes, junctions, pumps, valves, valves and storage tanks or locations without water to limit the size or the complexity of the network. HydroSim simulates, also, the flow of water in each pipe, the pressure at each node, the amount of water in tanks and qualitative characteristics (age, origin and concentration of substances) for a predetermined period of time and steps.

4. APPLICATION

To investigate the effect of the design of a water supply network on the quality of drinking water, the network of Figure 1 was used which in the past was studied by Clark et al. (1995), Vasconcelos et al. (1997) and it is used as an sample problem for the software EPANET (Rossman 2000). This network has a very oblong shape, consisting of 92 nodes, 2 water supplies (river and lake), 3 tanks, 2 pumping stations, 117 pipelines and should provide at least 3 atm water pressure. In the network the consumption was considered to vary with time periodically during the day. For most nodes the consumption of water was considered to be for domestic use. As model of daily consumption was considered the model of a typical Greek city (Lamia) (Vantas 2007).

The Greek legislation in force does not specify maximum and typical values for the residual chlorine, but indicates that its value should be such that no problems should exist, a concentration of chlorine equal to 0. 20 mg / L was selected at water entry points and at least 0.05 mg/L residual chlorine (NWQMS 2004), within the network, in order to achieve disinfection. The Kinematic model chosen was first-order with a rate coefficient of K_b = - 0.5 hr^{-1} and was assumed that no reactions take place in the pipes, so the concentration of residual chlorine is written (Rossman

2000): $C = C_o \exp(-0.5\, t)$, where C = concentration $[M/L^3]$ at time t [T], C_o = concentration $[M/L^3]$ at time t_o. Mixing was considered to take place instantaneously and completely in the storage tank (full mixing model) (Rossman et al. 2003). The time step for hydraulic solution was 1 hr and for simulating the water quality was 5 min.

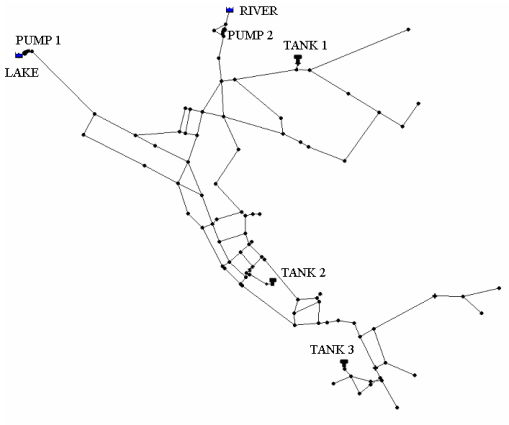


Figure 1. Network schematic

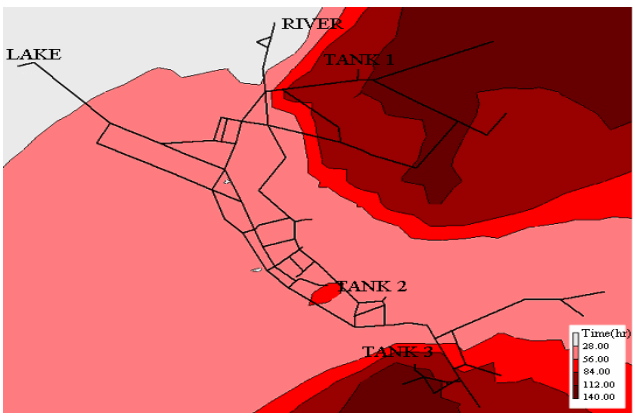
4.1 Regulation of initial chlorine dose at the water entry points

From the simulation of the network of Figure 1, it was found that it works without problems from a hydraulic point of view. In order to investigate the effects of the initial dose of chlorine to the water entry points to the system (Pump 1 and Pump 2) and the age of residual chlorine at the ends of the network, successive simulations were carried out for different initial doses. Residual chlorine (Table 1) at the ends of the network and the age of the water in the network and in the storage tanks were determined (Figure 2)

Table 1.Initial and residual chlorine

Initial chlorine (mg/L)	0.2	0.3	0.4	0.45	0.50
Minimum chlorine residual (mg/L)	0.02	0.03	0.04	0.05	0.06

It can be deduced from Table 1 that for a minimum value 0.05 mg / L residual chlorine at the ends of the network the initial dose of chlorine at entry points of the network has to be 0.45 mg / L which is almost twice the value of 0.20 mg / L set as the maximum permissible limit. On the other hand if the initial dose is 0.20 mg / L, then the minimum value in residual chlorine is 0.01 mg / L or 5 times less than the minimum allowable. Figure 2 shows that the water remains in the network long enough and particularly there are areas (dark brown) in which the age of the water exceeds 7 days.



Σχήμα 2. Values of water age in the network under examination

The substantial large values of the age of the water in the network are due to the long time of storage in the water tanks. The minimum and maximum values observed for the age of water in the tanks after 1000 hr time of simulation, is for the Tank 1 from 2.9 to 4.6 days, for the Tank 2 and Tank 3 from 1.0 to 3.1 days and 2.5 to 5.3 days, respectively. Note that the large resident time of water in tanks degrades water quality, because it favours the creation of disinfection by-products and does not allow retention of residual chlorine at sufficient levels for adequate disinfection.

4.2 Possible solutions to the problems

In order to maintain the residual chlorine within the permissible limits 0.05-0.20 mg / L and taking into account the specific characteristics of the network, such as the oblong shape, the location and number of tanks, the locations of water intakes and the number and the hours of pumping operation we should to reduce: (a) the age of water in tanks and in the network and (b) the excessive chlorination.

a. *Water age reduction in the tanks and the network*

To confront with this problem, different maximum water levels (MWL) were given in Tank 2 for which Pump 2 stops to operate and the age of the water was calculated in all three tanks (Figure 3). Observing Figure 3 we can see that the MWL in the Tank 2 for which the water in all three tanks have the lowest age is 3.75 m. Comparing the age of water obtained for the three tanks for water level 3.75 m at Tank 2 regarding to an original water level of 6 m, it was found that reduces the age of the water by 21%, 48% and 38% for tanks 1, 2 and 3 respectively and therefore the initial choice of a water level of 6 m in the Tank 2 was the worst as far as the age of water in tanks.

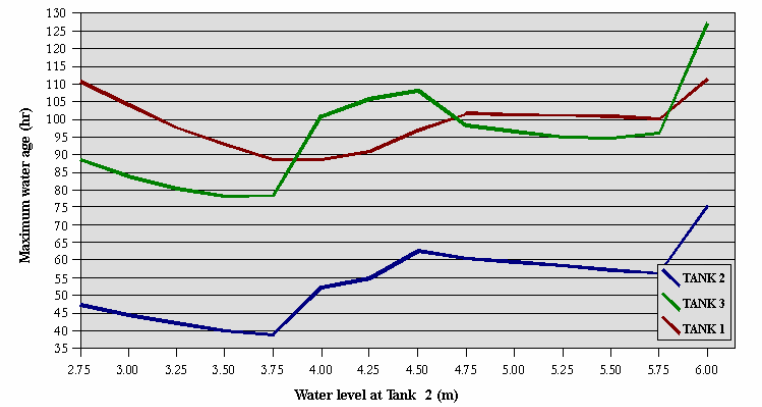


Figure 3. Water age in Tanks 1, 2, 3 as a function of water level in Tank 2.

Then the operation of Pump 1 (15hr) remained stable, but we changed the start time from 0.00 am, which was the original design, to 3.00 am and then to 6.00 am. It was found that changes to the start time of pump 1 have no influence on water age in the tanks.

In a similar way for different values of MWL in Tank 1 and M.W.L of 3.75 m in Tank 2 (previous case) the age of water was determined in the other tanks and concluded that when MWL in Tank 1: (a) is from 8 to 10 m, age does not affect the water in any of the tanks, (b) is less than 8 m, the age of water in the Tank 1 increases rapidly and for values <3 m water is effectively stationary (too many years in storage tank), (c) is from 6 to 8 m age in the water Tank3 increases up to 140 hr and lower prices of M.W.L stabilize it at 100 hr. Finally removing Tank1 shows that the age of the water in tanks 2 and 3 is 24 and 96 hr respectively.

Then M.W.L was held constant in Tanks 1 and 2 at 10 and 3.75 m (previous cases) respectively, and the age of the water was determined in the tanks for various values of MWL of Tank3. It was found that when MWL in the Tank3: (a) is from 7 to 12 m, no changes occur in the age of water in all tanks, (b) is from 4 to 7 m, the age of the water increases rapidly in the Tank3 and for values less than 4 m the water in Tank3 is practically stationary and (c) varies throughout the range of the

above values, the age of the water in tanks 1 and 2 remains constant. Finally, removing Tank3 shows that the age of the water tanks1 and 2 are 93 and 38 hr respectively

Apart from the above cases the following cases were examined briefly: Case I - Elimination of Tank 1 and Tank 2: Significant reduction of the water age of Tank 2 (24 hr), and at the ends of the network (41 hr). Case II - Increase of the volume of Tank 2: The alleged removal of Tank 1 and Tank 3 in which total volume of water stored is 7.030 m^3 means that to restore the balance of water we should increase the volume of water stored in the Tank 2 from 4.910 m^3 to 11.940 m^3. Choosing a MWL in Tank 2 equal to 3.75 m causes a significant reduction of the age of water in this tank (34 hr) and at the ends of the network (41 hr).

b, Reduction of excessive chlorination:

The following cases were examined for better chlorination of the system:

(a) Regulating of Pump 2 so MWL at Tank 2 is 3.75m: It was assumed that the introduction of chlorine to the system is to pump stations 1 and 2 and the initial dose of chlorine is 0.20, 0.25 and 0.30 mg / L. It was found that the residual chlorine at the ends of the network is 0.03, 0.05 and 0.07 mg / L respectively. Therefore, to achieve a minimum residual chlorine of 0.05 mg / L to 0.25 mg / L of chlorine should be added to the network head chloride, which is more than the permitted (0.20 mg / L), which will affect consumers who are in the region of water intake and pump stations 1 and 2. (b) Adding 0.20 mg / L of chlorine in pumping stations 1 and 2 and 0.15 mg / L in Tank1 and Tank 2. .This case was found to produce allowed residual chlorine levels (0.05 and 0.20 mg / L) (Figure 4). (c)Removal of Tank1 and Tank 3 and proportional increase of the volume of water tank2 in order to meet the total water volume stored and chlorine input to the network in positions of pumps1and 2 in a quantity of 0.20 mg / L. It appears that the amount of chlorine throughout the network is within the allowable limits (0.05 - 0.20 mg / L) (Figure 5).

Although several assumptions and simplifications for the network under examination were made it was found to have a fairly complex operation and the results of the changes that were made could not be predicted from the beginning. It is obvious that the process described refers to the design stage of a water supply network and after its construction the process should be repeated for the calibration of the parameters. .

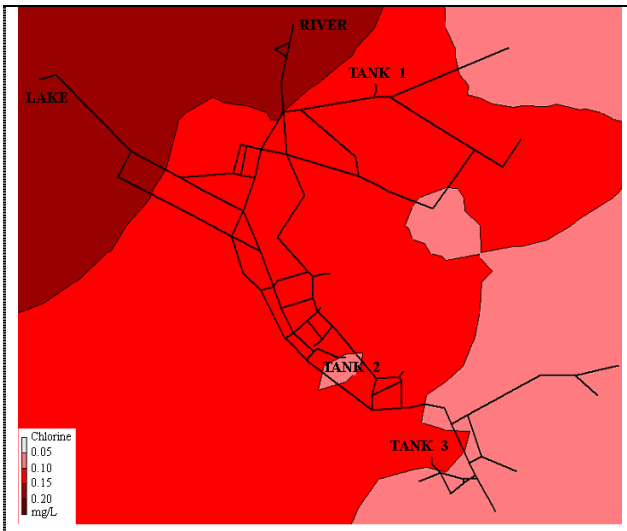
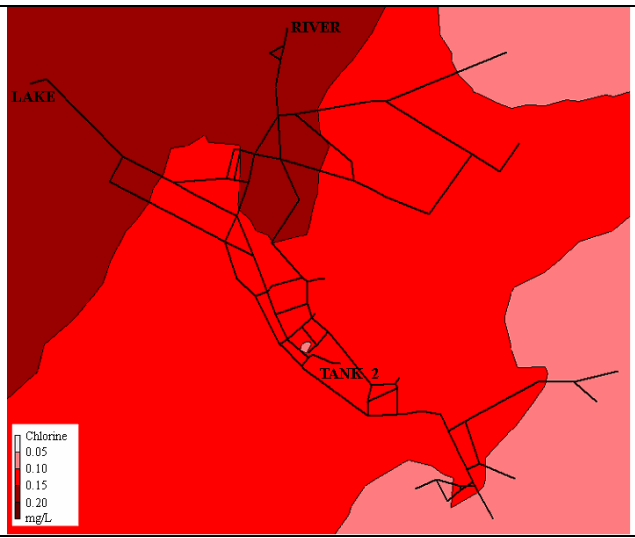


Figure 4. Residual chlorine for an initial dose of 0.20 mg/L in Pump2 and Tank 1 and Tank3



Σχήμα 5 Residual chlorine for an initial dose 0,20 mg/L in Pump 2 and in and Tank 1 and Tank 3 and icrease of the volume storage of Tank 2

5. CONCLUSIONS

Models that predict water quality in distribution networks can ensure safe lifelong basis consumption of drinking water at the point of consumption with continuous monitoring of the water quality and simultaneous regulation of disinfection. The use of models for simulation of water

quality can be used to: (a) to assist water agencies with limited funds to manage their water networks. (b) To identify causes of problems in water quality and find managerial solutions that can be easily employed (regulation of pumping operation, setting the level of tanks, etc.) and thus have low cost. (c) Reduce disinfection by-products by reducing the age of the water in network and minimize the use of disinfectants without further pre-treatment of the water to remove organic substances. (d) Create emergency scenarios, such as contamination of the network, and investigate the optimal treatment.

It was found that for the optimum water disinfection of a drinking water network we should :(a) Minimize the initial dose of chlorine. (b) Reduce as much as possible the age of the water in the network and tanks. (c) Make appropriate choices for the positions of chlorination equipment, in order to achieve reduction of the initial dose at the head of the network, uniform distribution of residual chlorine concentration and minimum use of disinfectants. (d) Regulate the operation of pumps (duration, starting and stopping times) and carefully determine the active volume of the tanks (to avoid or minimize water stagnation in the tanks).

As far as Greece is concerned the main problem is the absence of technical specifications for the creation of models for new or existing drinking water networks, since the Presidential Decree 696/74 does not contain any specifications. It is proposed, therefore, to adopt new technical standards for studies concerning: (a) new drinking water networks (b) expansion of existing networks and (c) reviewing of existing networks, taking into account the progress of science in water network design, computers, hygiene of drinking water, GIS etc. These standards should, together with the hydraulic study of the system, provide a simulation study of water quality, disinfectants substances, age of the water in the network and tanks for the different consumption conditions at the beginning and at the end of life of the project, but also at intermediate intervals (eg every ten years) and finally a study of the reliability of the network (pipelines, pumping stations, etc.).

While the existing institutional framework for the quality of drinking water is considered generally satisfactory we have the opinion that it should: (a) Increase the number of unwanted substances and provide parametric values for them since the list of priority substances in Decision 2455 / 2001/EC contains 24 new toxic substances. (b) Establish maximum and minimum concentration of residual chlorine in a water network and avoid the use of the term "no problems" which is vague. Finally, it is proposed that the responsible stakeholders (Municipalities, D.E.Y.A. etc.) see after the creation of models for their networks connected with GIS, as experience has shown that several D.E.Y.A. in our country, given the age the networks, do not know the exact geometric and hydraulic characteristics of their networks and therefore face different management problems.

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CFD INVESTIGATION OF PARTICLE DEPOSITION AND RESUSPENSION IN A DRINKING WATER DISTRIBUTION SYSTEM

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Abstract: A comprehensive 3D numerical investigation of hydrodynamics of particles flowing through a Al-Diwanyia water pipe-network distribution system in Iraq has been modeled. The multiphase mixture model available in Fluent 6.3 is used in this study. In this numerical simulation four different particles have been used as secondary phases to calculate real multiphase effect in which inter-particle interaction has been accounted. The deposition of particles, along the periphery of the wall around pipe has been investigated. The effect of bend and fluid velocity on particle deposition has also been investigated. The maximum particle deposition is at the bottom wall in the pipe before entering the bends conversely, downstream of bends the maximum deposition is not at the bottom - as seen upstream of the bends - rather it occurs at an angle of 60^0 toward the inner side from the bottom. The larger particles clearly showed deposition near the bottom of the wall except downstream of the bends. As expected, the smaller particles showed less tendency of deposition and this is more pronounced at higher velocity. This numerical investigation showed good agreement with the experimental results.

Key words: Computational Fluid Dynamics (CFD), pipe distribution system, mixture model , particle deposition, resuspension.

1. INTRODUCTION

Drinking water distribution networks are supposed to transport only dissolved matter rather than suspended solid particles. However, it is not possible to make the drinking water free from any suspended solid particles. The ability to determine the origins of these particles varies between different water supply systems, with possible sources being from catchments, treatment processes, biofilm growth within the water supply pipes, and corrosion products a s shown in "Figure 1" (Jayaratne et al. 2004).

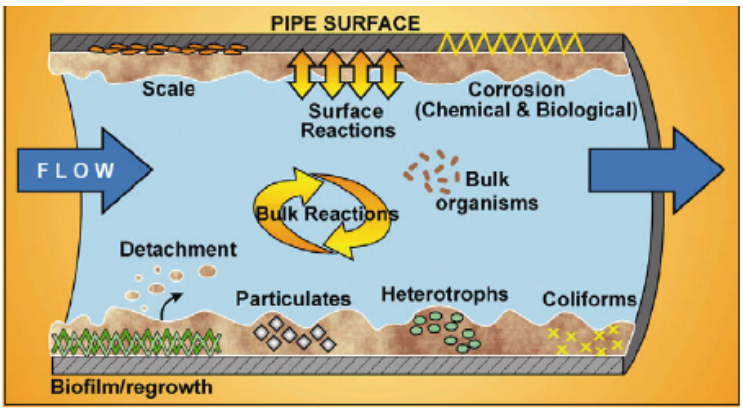


Figure 1. Distribution System Interactions that Affect Water Quality.

Turbidity, which is, in general, the measure of particles content in water, is the main cause of discolouration of water. The distribution systems in Iraq receive less than 10 NTU of turbidity,

which is more than international standard (5 NTU). However, when the level of turbidity exceeds 5 NTU, the discolouration of the water is visible to naked eyes. According to IWQMS (International Water Quality Management Strategy) the water becomes discoloured just after the average background datum 1.4 NTU, but a major discoloured water event was defined as having a maximum turbidity exceeding 2.6 NTU under normal circumstances. This very small amount of particles can accumulate and settle down at the bottom wall or any pockets like dead ends, T-junctions, bends of the pipe networks, or a combination of these. During any abnormal flow (e.g. during wash out, pipe burst, etc.), these particles re-entrain into the system at a time under high turbulence, which create turbidity. There are also some other causes of turbidity spikes, which have not yet been identified. Complains come often when these distribution networks experienced turbidity spikes at anywhere of the systems due to numerous causes. These spikes travel along the pipe networks and into households resulting in dirty water. To drink such discoloured water caused by the turbidity spikes is not safe. Many researches have attempted to understand the possible deterioration of water quality once it enters a distribution system (Anderson and Russell 1970; Laurinat, et al., 1985, David, et al., 1987, and Thomson, 2003).

An improvement in our understanding of the complex hydrodynamic behavior of suspended and/or deposited particles involved in these distribution pipe networks requires mathematical and physical models. Computational Fluid Dynamics (CFD), along with an analytical turbulent model is one of the most popular mathematical techniques, as it has the ability to predict the behavior of complex flows for such multiphase flow applications. A CFD investigation was carried out to predict the hydrodynamic behavior of turbid particles flowing through a pipe network. This paper will discuss the distribution of sediments in the drinking water distribution system in Al-Diwanyia in Iraq. The main interest is the behavior (deposition and resuspension) of the particles coming from the Al-Diwanyia Water Treatment Plant.



Figure 2. Schematic drawing of pipe a test loop (Grainger, et al., 2003).

2. MATERIALS AND METHODS

The CFD model uses the characteristics of sediment as input; this means that velocities at which the sediment suspends, resuspends and/or settles have to be determined. This was performed by obtaining samples of particulates from the water distribution systems of Al-Diwanyia in Iraq. The samples in Australia were analyzed using a pipe test-loop and a water tunnel at CMIT (CSIRO Manufacturing & Infrastructure Technology) (Grainger, et al., 2003). The rig consisted of a test pipe with a diameter of 100 mm; a schematic drawing of the test pipe and boundary conditions are shown in "Figure 2" and "Table 1" respectively. The test rig was used to model the validation and determine the flow velocity of the water when the sediment starts to settle and the sediment velocity with which the sediment settles.

Table 1 Physical and hydraulic characteristics of the system used for CFD simulation.

Pipe loop length (m)	150
Pipe length in Al-Diwanyia	3200
Diameter of the pipe D (m)	0.1
Total volume of water (m^3)	0.322
No. of phases	4
V_F of each secondary phase	342 ppm
Particle density (kgm^{-3})	1640
Particle sizes (μm)	1- 100
Average water velocities (ms^{-1})	0.05-0.50

3. MATHEMATICAL MODEL

GOVERNING EQUATION

The multiphase mixture model of Fluent 6.2 (Fluent Inc., 2006) used in this study solves the continuity and momentum equations for the mixture. The volume fraction equations are solved for the secondary phases. The model also solves the well-known algebraic expressions for the relative velocities for the secondary phases (Fluent Inc., 2006).

Continuity Equation for the Mixture

The term "mixture" can be defined by the combination of all the primary and secondary phases. The continuity equation for the mixture is

$$\frac{\partial}{\partial t}(\rho_m) + \nabla \cdot (\rho_m \vec{v}_m) = 0 \tag{1}$$

Where $\bar{v}_m$ is the mass-averaged velocity of the mixture, which is:

$$\vec{v}_m = \frac{\sum_{k=1}^{n} \alpha_k \rho_k \vec{v}_k}{\rho_m} \tag{2}$$

And ρm is the mixture density:

$$\rho_m = \sum_{k=1}^{n} \alpha_k \rho_k \tag{3}$$

αk is the volume fraction of phase k.

Momentum Equation for the Mixture

The momentum equation for the mixture can be obtained by totaling the individual momentum equations for all the phases. It can be expressed as:

$$\frac{\partial}{\partial t}(\rho_m \vec{v}_m) + \nabla \cdot (\rho_m \vec{v}_m \vec{v}_m) = -\nabla p + \nabla \cdot \left[\mu_m \left(\nabla \vec{v}_m + \nabla \vec{v}_m^{\mathrm{T}}\right)\right] + \rho_m \vec{g} + \vec{F} + \nabla \cdot \left(\sum_{k=1}^{n} \alpha_k \rho_k \vec{v}_{\mathrm{dr},k} \vec{v}_{\mathrm{dr},k}\right) \tag{4}$$

Where n is the number of phases, $\bar{F}$ is the body force, and μ_m is the viscosity of the mixture:

$$\mu_m = \sum_{k=1}^{n} \alpha_k \mu_k \tag{5}$$

$\vec{v}_{dr,k}$ is the drift velocity for the secondary phase k:

$$\vec{v}_{\mathrm{dr},k} = \vec{v}_k - \vec{v}_m \tag{6}$$

Relative (Slip) Velocity and the Drift Velocity

The relative velocity (also referred to as the "slip" velocity) is defined as the velocity of the secondary phase (p) relative to the velocity of the primary phase (q):

$$\vec{v}_{pq} = \vec{v}_p - \vec{v}_q \tag{7}$$

The drift velocity and the relative velocity ($\vec{v}_{qp}$) are connected by the following expression:

$$\vec{v}_{\mathrm{dr},p} = \vec{v}_{pq} - \sum_{k=1}^{n} c_k \vec{v}_{qk} \tag{8}$$

The basic assumption of the algebraic slip mixture model is that in order to prescribe an algebraic relation for the relative velocity, a local equilibrium between the phases should be reached over short spatial length scales. The form of the relative velocity is given by

$$\vec{v}_{pq} = \frac{\tau_p}{f_{\mathrm{drag}}} \frac{(\rho_p - \rho_m)}{\rho_p} \vec{a} \tag{9}$$

Where $\vec{a}$ is the secondary-phase particle's acceleration and τp is the particulate relaxation time. Following Manninen, et al., 1996 τp is of the form:

$$\tau_p = \frac{\rho_p d_p^2}{18 \mu_q} \tag{10}$$

Where dp is the diameter of the particles of the secondary phases p, and the drag function fdrag is taken from Schiller and Naumann, 1935:

$$f_{\mathrm{drag}} = \begin{cases} 1 + 0.15\,\mathrm{Re}^{0.687} & \mathrm{Re} \le 1000 \\ 0.0183\,\mathrm{Re} & \mathrm{Re} > 1000 \end{cases} \tag{11}$$

And the acceleration $\vec{a}$ is of the form (12)

$$\vec{a} = \vec{g} - (\vec{v}_m \cdot \nabla)\vec{v}_m - \frac{\partial \vec{v}_m}{\partial t}$$

The simplest algebraic slip formulation is the so-called drift flux model, in which the acceleration of the particle is given by gravity and/or a centrifugal force, and the particulate relaxation time is modified to take into account the presence of other particles.

Volume Fraction Equation for the Secondary Phases

From the continuity equation for the secondary phase p, the volume fraction equation for the secondary phase p can be obtained:

$$\frac{\partial}{\partial t}(\alpha_p \rho_p) + \nabla \cdot (\alpha_p \rho_p \vec{v}_m) = -\nabla \cdot (\alpha_p \rho_p \vec{v}_{\mathrm{dr},p}) + \sum_{q=1}^{n} (\dot{m}_{qp} - \dot{m}_{pq}) \tag{13}$$

Turbulence Viscosity (The Spalart-Allmaras Model)

Instead of μ_m (equation 5) the turbulent viscosity, μt, is computed from

$$\mu_t = \rho_m \tilde{\nu} f_{v1} \tag{14}$$

Where the viscous damping function, f_{v1}, is given by

$$f_{v1} = \frac{\chi^3}{\chi^3 + C_{v1}^3} \tag{15}$$

Where Cv1 = 7.1 and

$$\chi \equiv \frac{\tilde{\nu}}{\nu} \tag{16}$$

3. VALIDATION OF MODEL

In this paper the results have been validated with the experimental result conducted by Grainger, et al., 2003. At CSIRO Grainger, et al., 2003, demonstrated an experiment for particle distribution and deposition in a test loop. In order to compare the experiment results (Grainger, et al., 2003), we have used the same geometric and boundary conditions. "Figure 3" represents the cumulative particle volume fraction as a function of the heights across the pipe at a certain location in the loop for both the experiment and the CFD results.

Nevertheless, the trend is similar, but the experimental results show a marginally lower volume fraction. This is because of the shortcomings of the measuring instruments that were used in the experiment.

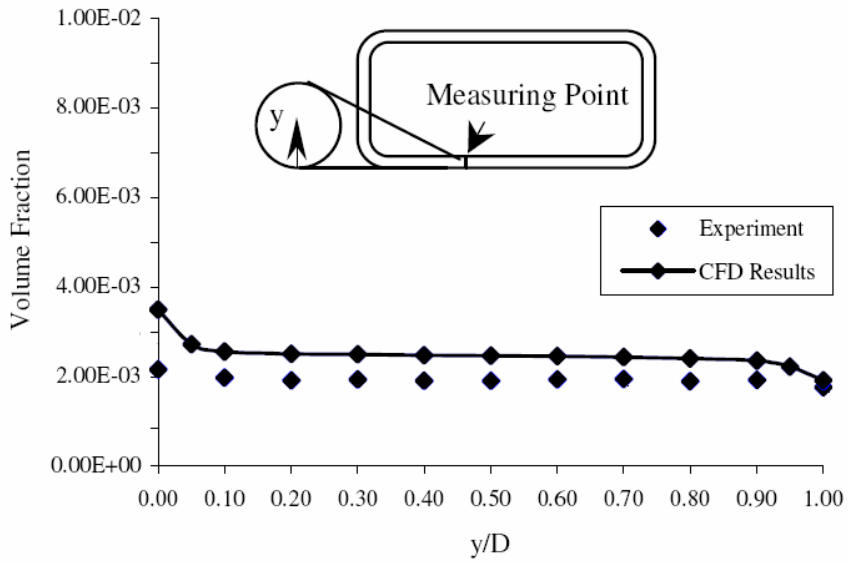


Figure 3. Comparison of the CFD results and experimental data for the velocity 0.43 m.s-1 at the center of the pipe.

4. RESULT AND DISSUASION

Figures 4 show the relative concentration plotted along the pipe for different height of 0.25D, 0.5D, 0.75D, and 1D from the bottom wall of the pipe. Relative concentration is a dimensionless parameter, which represents the ratio of local particle concentration to that of bottom of the pipe wall. Figures 4 shows more homogeneous distribution of particles at different depths in the bend region. Due to high steam line curvature and associated centrifugal force the fluid at different depths gets well mixed resulting in homogeneous distribution of particles near bend. Down stream of the bend, the streamline curvature and associated centrifugal force disappears and particles start

to segregate to different concentration at different depths. This segregation or stratification is more pronounced at lower velocity. At higher velocity the particles do not get enough time to segregate before they reach the next bend. Higher turbulence at higher velocity also contributes to homogeneity of the particles around bends.

Figures 5 shows typical circumferential distributions of particles volume fraction for the different velocities (0.07 and 0.5 ms^{-1}) for up and downstream location of bends (1st, 3rd and 4th). The angle 0^0 starts at the top wall and angle 1800 is the bottom wall of the pipe. The upstream profiles exhibit a distinctive variation with the maximum deposition at the bottom of the pipe.

The peak deposition at the bottom wall is high when the velocity is low. This can be easily explained as particles disperse at higher velocity and can be found higher concentration across the cross-section of the pipe due to high turbulence. However, the trends of the particle deposition at upstream of bends "Figures 5" are not symmetrical along vertical plane. This is because of the particles entrainment in between bends is governed by the particle distribution of previous upstream bend, which is different from straight pipe flow.

As shown in Figure 5 at downstream of the bend the maximum deposition does not occur at the bottom (180^0) rather it occurs at on the inner wall of the bend. The location of the peak deposition is situated at an angular displacement of 60^0 with respect to the bottom of the wall. The fluid is subjected to centrifugal force as it flows around the bend. Particles having higher specific gravity get segregation and deposit in the bend region. This deposition is pronounced near the outer wall of the bend entry and inner wall of the bend exit. However, due to the influence of high particle distribution at the inner wall upstream bend exit, the inner wall of the downstream bend entry will receive relatively higher load of particles. In Figure 6 the volume fraction of particles plotted as a function of circumferential angles for different planes (0^0, 22.5^0, 45^0, 67.5^0, and 90^0) of 4th bend "Figure 2" for different velocities (0.07 and 0.5 ms^{-1}). 0^0 and 90^0 planes are known also as up and down stream planes respectively. The peak deposition "Figure 6" moves from bottom (180^0) to 240^0 (60^0 inner wall from bottom) as it goes from upstream plane (0^0) to downstream plane (90^0). For a higher velocity "Figure 6", the particle diffusivity increases as a result of fluid diffusivity, which resulted in less particles deposition at the bottom inner wall.

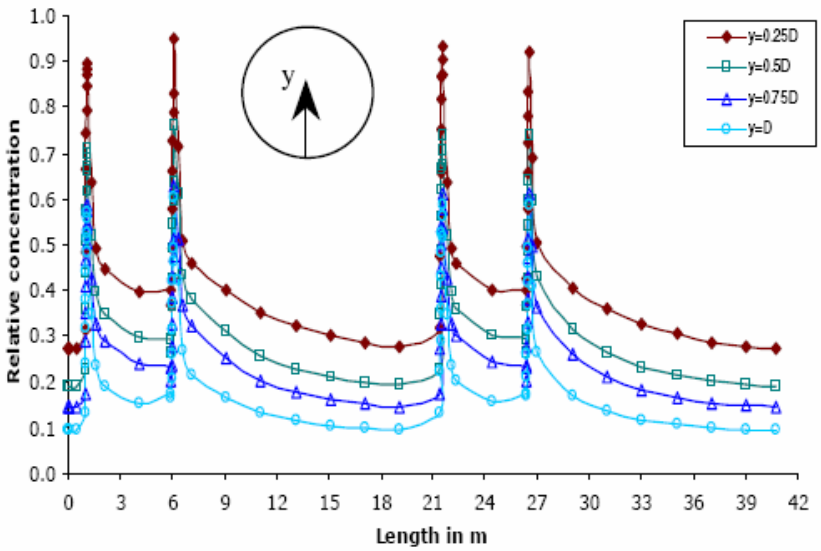


Figure 4a. Relative concentration of particles for different depths along the pipe at 0.07 ms^{-1}.

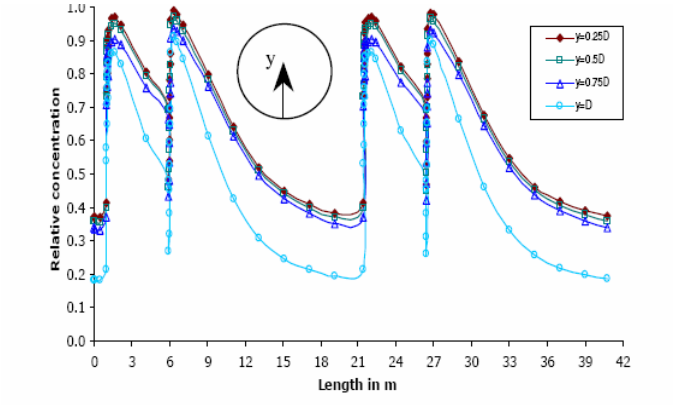


Figure 4b. Relative concentration of particles for different depths along the pipe at 0.5 ms^{-1}.

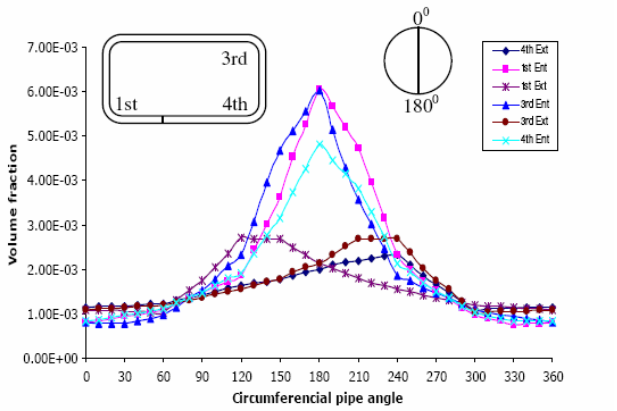


Figure 5a. Cumulative particle deposition (CFD) as a function of circumferential pipe angles at three different up and down stream of bends at 0.07 ms^{-1}.

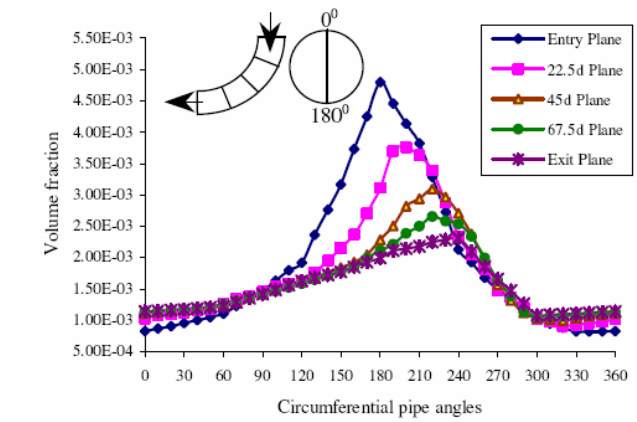


Figure 5b. Cumulative particle deposition (CFD) as a function of circumferential pipe angles at three different up and down stream of bends at 0.5 ms^{-1}.

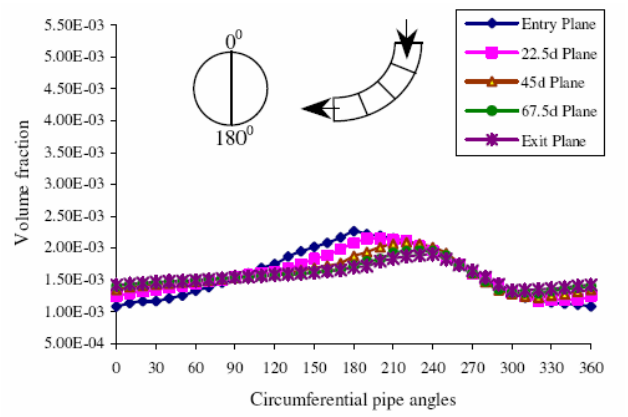


Figure 6a. Particle deposition (CFD) as a function of circumferential pipe angles at different planes of 4th bend at 0.07 ms-1

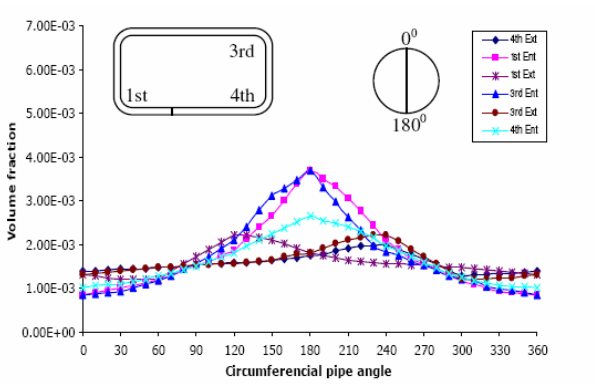


Figure 6b. Particle deposition (CFD) as a function of circumferential pipe angles at different planes of 4th bend at 0.5 ms^{-1}.

5. CONCLUSION

Particle deposition and suspension in pipe flow have been studied with a 3D numerical multiphase mixture model available in Fluent 6.3. The model solves continuity and momentum equations for the mixture and volume fraction equations for the secondary phases. Considering the forces include gravitational and drag force it is investigated how this deposition depends on the particle diameter, density, the height of the pipe, and the velocity of fluid. The deposition of heavier particles at the bottom wall of the pipe is greater at low velocity rather than high velocity but light particles remain suspended in fluid across the pipe circumference. Particles of small sizes have a greater concentration at the bottom wall of the pipe for low rather than high velocity, whereas there is a greater concentration of particles at the bottom wall of the pipe for higher rather than lower velocity for larger particles.

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WATER NETWORK ASSESSMENT AND RELIABILITY ANALYSIS BY USE OF SURVIVAL ANALYSIS

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Abstract: Sustainable management of urban water distribution networks should include not only new methods for monitoring, repairing or replacing aging infrastructure, but more importantly expanded methods for modelling deteriorating infrastructure so as to proactively assess the risk of failure and devise replace-or-repair strategies. The paper presents a framework for devising such a proactive risk-based integrity-monitoring strategy for the management of urban water distribution networks. The framework presented is based on a combination of artificial neural network, parametric and nonparametric survival analysis and it is utilized in the estimation of time-to-failure metrics for pipe networks.

Key words: Water distribution networks; decision support system; risk analysis; survival analysis.

1. INTRODUCTION

Each year, thousands of kilometers of pipes across the globe are upgraded or replaced in an attempt to maintain the uninterrupted transport of water. The goal is the safeguard of the health of urban populations, the need to increase the reliability of the pipe networks and the service provided to people, as well as socioeconomic factors in relation to the cost of operations and maintenance of piping networks.

The fact, though, remains that existing water distribution networks are increasingly at risk due to numerous factors and sustainable management of such networks should include not only new methods for monitoring, repairing or replacing aging infrastructure, but also and more importantly should include expanded methods for modelling deteriorating infrastructure conditions and for proactively devising strategies to keep the networks in operation. In essence, water distribution agencies are nowadays faced with the increasingly more complex task to intelligently and efficiently assess (or model) the condition of a pipe network and manage the network in ways that maximize its reliability and minimize its operational and management costs. At the outset of this balancing act of operating costs and reliability is one of the most important dilemmas facing water distribution agencies: should an organization repair or replace ageing and/or deteriorating water mains and, in either case, what should the sequence of any such repairs be as part of a long-term network rehabilitation strategy?

The manuscript reports on the investigation of possible risk-of-failure parameters and outlines a multi-criteria decision support system (DSS) for the modelling of the behaviour of water distribution networks based on survival analysis techniques. The analysis is based on a dataset covering five years of pipe-breakage incidents.

2. STATE OF KNOWLEDGE

Research to-date has helped identify a number of potential risk factors that contribute to pipe breaks, ranging from time-invariant to time-dependent factors. Pipe diameter and pipe material are

examples of the former category, while pipe age and previously observed breaks are examples of the latter category.

In general, a rational decision to replace a pipe or not is often made by evaluating two alternatives: (1) to replace the pipe now, thus incurring the replacement cost and any future costs associated with the new pipe, or (2) to retain the existing pipe, thus saving any replacement costs but incurring higher risks for failure, increasing costs of repair, disruption of service and damages.

Most of the studies in the past have attempted to identify statistical relationships between water main break rates and influential risk factors such as a pipe's age, diameter and material, the corrosiveness of the soil, the operating pressure and temperature, possible external loads (including highway traffic) and recorded history of pipe breaks. And most such studies have shown a relationship between failure rates and the time of failure (age of pipes), with some studies suggesting methods to optimize a pipe's replacement time. Shamir and Howard (1979) reported an exponential relationship and Clark (1982) developed a linear multivariate equation to characterize the time from pipe installation to the first break and a multivariate exponential equation to determine the breakage rate after the first break. A study by Andreou et al. (1987) suggested a probabilistic approach consisting of a proportional hazards model to predict failure at an early age, and a Poisson-type model for the later stages, and further asserted that stratification of data (based on specific parameters) would increase the accuracy of the model. A non-homogeneous Poisson distribution model was later proposed by Goulter and Kazemi (1988) to predict the probability of subsequent breaks given that at least one break had already occurred. Finally, Kleiner et al. (1999) developed a framework to assess future rehabilitation needs using limited and incomplete data on pipe conditions.

Prasad et al. (2003) outlined a multi-objective genetic algorithm method and introduced a new reliability measure, called network resilience. The measure tried to provide surplus head above the minimum allowable head at nodes and reliable loops with practicable pipe diameters. Kanakoudis and Tolikas (2004) addressed the question 'what should system managers prefer? Systems that never fail or systems that remain safe during a failure?' by presenting a method that hierarchically analyses the possible preventive maintenance actions in a water system. The whole attempt was based on indices that assess the performance level of the system, before any action is taken, and accomplished through a technico-economical analysis that takes into account the costs associated to the repair or replacement of failing system components. Frequently used methods for the risk analyses of water supply systems were presented by Tuhovcak et al. (2006). The methods address the identification of qualitative and quantitative risks posed by the individual system components, the evaluation methods and interpretation of results, with an emphasis on the Hazard Analysis and the Critical Control Points (HACCP) method. Park (2008) presented a method to assess and track changes in the hazard functions between water main breaks by using the proportional hazards model. The method provides a systematic framework for analyzing changes of the hazard functions as more breaks occur, and a for identifying a critical point at which the hazard function changes to a different functional form.

Recently, a simulation model was applied to an inventory of water mains in New York City to analyze replacement strategies, and Vanrenterghem et al. (2004) reported on models for the structural degradation of urban water distribution systems based on data from New York City. Additional work on the same case study was reported by Aslani (2003) and Christodoulou et. al. (2003). The knowledge gained by the New York City case study was furthered and reported upon by Christodoulou et al. (2007) in a developed framework for integrated GIS-based management, risk assessment and prioritization of water leakage actions.

3. NONPARAMETRIC SURVIVAL ANALYSIS

In past case studies, the analysis of the risk for failure had been reduced to fitting failure times to parametric models (most notably Weibull probability distributions) and examining such models through the stratification of data and the examination of the various risk factors one at a time. Such

models assume that a pipe's failure time can be approximated by parametric equations relating risk factors to observed outputs, expressed in mathematical forms. Should one assume that such relationships between risk factors and outputs are not expressed mathematically by means of known distributions, then the estimation of the underlying multivariate probability distribution function becomes of paramount importance to the evaluation of the expected probability of failure at different time intervals.

Survival analysis is a branch of statistics dealing with deterioration and failure over time (for example, death in biological organisms and failure in mechanical systems) and involves the modelling of the time-to-event. That is, it is the study of the elapsed time between an initiating event and a terminal event. In the case of piping networks such initiating events can be the installation of a pipe, a water-leak observation or the start of a pipe treatment, and cases of terminal events can be a relapse of a previous leak, a fix or a failure. The method is based on estimating the reliability of a system and its lifetime subject to multiple risk factors, and aims to provide answers on the population fraction that survives past an expected lifetime, the effect of the various risk factors on the system's lifetime and on the probability of survival, and the expected mean time to failure (Hosmer et al. 2008, Lee and Wang 2003, Hintze 2006). The data values used in the analysis are a mixture of complete and censored observations. In the former case a terminal event is thought to have occurred while in the latter case a terminal event has not occurred. A terminal event is assumed to occur just once for each subject.

Should the elapsed time until the occurrence of a specified event be denoted by T, then the probability distribution of T may be specified using one of the following basic functions: (1). a probability density function, *f(t)*, denoting the probability that an event occurs at time t; (2) a cumulative distribution function, *F(t)*, denoting the probability that an individual survives until time t; (3) a survival function, *S(t)*, denoting the probability that an individual survives beyond time t; (4) a hazard rate, *h(T)*, denoting the probability that an individual at time T experiences the event in the next instant; and (5) a cumulative hazard function, *H(T)*, being the integral of *h(t)* from 0 to T. Once one of these functions has been specified, the others may be derived using the mathematical relationships presented below (Eq. 1 - 5).

$$F(t) = \int_0^t f(x)dx \tag{1}$$

$$S(t) = \int_T^\infty f(x)dx = 1 - F(T) \tag{2}$$

$$S(t) = \exp\left[-\int_0^T h(x)dx\right] = \exp[-H(T)] \tag{3}$$

$$h(T) = f(T)/S(T) \tag{4}$$

$$H(T) = \int_0^T h(x)dx = -\ln[S(T)] \tag{5}$$

The survival function is usually the primary quantity of interest and it is estimated by use of the nonparametric Kaplan-Meier curve, while the hazard rate (also known as the conditional failure rate in reliability) is a metric which may be used for identifying the appropriate probability distribution of a particular mechanism (Hintze 2006). The recommended nonparametric estimator of the survival distribution, *S(T)*, is the Kaplan-Meier (Eq. 6) also known as the product-limit estimator.

$$\hat{S}(t) = \begin{cases} 1 & \text{if } T_{min} > T \\ \prod_{A \le T_i \le T} [1 - \frac{d_i}{r_i}] & \text{if } T_{min} \le T \end{cases} \tag{6}$$

where, d_t is the number of failures occurring in time T_t; D_t is the set of these failures; R_t is the set of all individuals that are at risk immediately before time T_t; r_t is the number of the individuals that are at risk immediately before time T_t; T_i is the termination time (failure event) for the i^{th} event; and A is

the minimum time considered below which failures are not considered (that is the case when data is left-truncated). A plot of the Kaplan-Meier estimate of the survival function is a series of horizontal steps of declining magnitude which, when a large enough sample is taken, approaches the true survival function for that population. The value of the survival function between successive distinct sampled observations is assumed to be constant and when no censoring occurs the Kaplan-Meier curve is equivalent to the empirical distribution. As already mentioned, an important advantage of the Kaplan-Meier estimator is that the method can take into account censored (i.e. missing) data.

It is often useful, though, to be able to compare the hazard rates of two groups. This is most often accomplished by employing the hazard ratio (*HR*), a metric that assesses the effect of an explanatory variable on the hazard or risk of an event (thus being an estimate of relative risk). If, for example, the hazard function of two variables is given by a regression model of the form

$$h(t,x,\beta) = h_0(t)r(x,\beta) \tag{7}$$

with $h_0(t)$ characterizing how the hazard function changes as a function of survival time, and $r(x,\beta)$ characterizing how the hazard changes as a function of the covariates of the two data sets, then the hazard ratio of the two data sets with covariate values of x_0 and x_1 is given by

$$HR(t,x_1,x_0) = \frac{h(t,x_1,\beta)}{h(t,x_0,\beta)} = \frac{h_0(t)r(x_1,\beta)}{h_0(t)r(x_0,\beta)} = \frac{r(x_1,\beta)}{r(x_0,\beta)} \tag{8}$$

The hazard ratio, thus, depends only on the function $r(x,\beta)$ and not the baseline hazard function, *h0(t)*. Proportional hazards regression models include the Cox semi-parametric proportional hazards model, and the exponential, Gompertz and Weibull parametric models. In Cox's proportional hazard model (Cox 1972), $r(x,\beta) = exp(x\beta)$, the hazard function is

$$h(t,x,\beta) = h_0(t)e^{x\beta} \tag{9}$$

and the hazard ratio is

$$HR(t,x_1,x_0) = e^{\beta(x_1-x_0)} \tag{10}$$

4. CASE STUDY

The water distribution network used as a case study is the network of the city of Limassol, Cyprus. The network is over 50 years of age, serves approximately 170,000 residents through approximately 64,000 consumer meters in an area of 70 sq.km., consists of about 800km of pipes and carries an annual volume of potable water of about 14 million cu.m. (about €7.0 million in value).

The dataset used in the study spans a 5-year period (2002-2007) and about 2,000 break incidents. The data analysis includes five risk factors: the number of observed previous breaks (*NOPB*), the pipe category (*Category*), length (*L*) and diameter (*D*), and the traffic load in the vicinity of a pipe (*TRAF*). These risk factors were analyzed by means of both numerical methods (artificial neural networks, ANN) and analytical methods. At first, a four-layer back-propagation ANN, consisting of 1 input, 2 hidden and 1 output layer, was employed in identifying the knowledge inherent in the historical data (Christodoulou et al. 2007). The ANN was further utilized for identifying patterns in the interactions of these risk factors (ANN inputs), their possible contribution to a pipe's failure and to the associated estimated life-cycle (the and ANN outputs). The analysis indicated that the order of significance (in descending order) of the aforementioned risk factors was *NOPB*, *Category*, *L*, *D* and *TRAF*, with the first four risk factors significantly outweighing the *TRAF* factor. In addition to the aforementioned risk factors, data was also collected on the type of pipe in which water loss was observed (*PipeType*: water main or house connection), and the presumed reason for

failure (*IncidentType*: pipe deterioration, corrosion, interference by others, tree roots, connection hose, other).

Further to the ANN analysis, the incident data was analyzed by means of both Poisson regression (Table 1, Fig. 1) and Cox regression (Table 2) by which the *PresumedPipeAge* output was fitted to models of the basic variables *NOPB*, *Category*, *PipeType*, *IncidentType* and *D*. For the purpose of data stratification, the factors *D* and *NOPB* were grouped in subcategories *DiameterRange* and *NOPBRange* respectively. *DiameterRange* takes the values 'Small' for diameters of 0.00-25.4mm, 'Medium' for 25.4-127.0mm and 'Large' for values greater than 127.0mm. *NOPBRange* takes the values 'Small' for 0-4 incidents, 'Medium' for 4-8 and 'Large' for more than 8 incidents. As the residual errors from the first analysis indicate (Fig. 1) the Poisson regression model (Table 1) is a good fit. The Cox regression, though, with the added information on the factors' proportional hazards provides a more powerful analysis tool. Of particular interest is the 'risk ratio' metric (Table 2) showing the value of $exp(b_i)$, which indicates the ratio of two hazards whose only difference is that x_i is increased by one unit.

$$e^{\beta_i} = \frac{h(T|x_i = \alpha + 1)}{h(T|x_i = \alpha)} \tag{11}$$

Table 1. Poisson regression analysis of risk factors NOPB ,Category, PipeType, IncidentType and DiameterRange

Variable	Coeff.	Error	Chi^2 (*)	Level (**)	Confidence Limit	
					Lower 95%	Upper 95%
Intercept	9.37716	0.00429	4773051.70	0.0000	9.36875	9.38558
(Category = 'DI')	0.05252	0.00744	49.87	0.0000	0.03794	0.06710
(Category = 'GI')	0.02445	0.00391	39.09	0.0000	0.01679	0.03212
(Category = 'MDPE (BLACK)')	0.02654	0.00389	46.43	0.0000	0.01890	0.03417
(Category = 'MDPE (BLUE)')	0.02876	0.00405	50.47	0.0000	0.02083	0.03670
(DiameterRange = 'MEDIUM')	0.00723	0.00187	14.98	0.0001	0.00357	0.01090
(DiameterRange = 'SMALL')	0.01223	0.00302	16.36	0.0001	0.00630	0.01816
(IncidentType = 'CORROSION')	-0.01275	0.00175	52.81	0.0000	-0.01619	-0.00931
(IncidentType = 'INTERF. BY OTHERS')	-0.03334	0.00186	320.35	0.0000	-0.03699	-0.02968
(IncidentType = 'OTHER')	-0.00878	0.00235	13.94	0.0002	-0.01339	-0.00417
(IncidentType = 'PIPE DETERIORATION')	-0.01628	0.00150	117.08	0.0000	-0.01923	-0.01333
(IncidentType = 'TREE ROOTS')	-0.00738	0.00183	16.18	0.0001	-0.01097	-0.00378
NOPB	0.00115	0.00003	1715.40	0.0000	0.00110	0.00121
(PipeType='WATER MAIN')	0.02870	0.00411	48.77	0.0000	0.02064	0.03675
Dispersion Phi	21.1626					

(*) Wald's Chi^2 (Ho:b=0) is the one degree of freedom chi-square statistic for testing the null hypothesis that $\beta_i = 0$ against the alternative that $\beta_i <> 0$.

(**) Prob. Level: The probability of obtaining a chi-square value greater than the above. This is the significance level of the test. If this value is less than some predefined alpha level, say 0.05, the variable is said to be statistically significant.

For example, in the dataset used and based on the analysis results (Table 2) if the number of observed previous breaks within the subgroup '*NOPBRange=medium*' is increased by one then the hazard rate increases by 20.49%. The *NOPB* risk ratio shows a 1.6% increase in the hazard rate for each additional incident, but this covers the entire data range from 0 to 8+ incidents which is overweighed by the number of incidents in the range 0-3 and it is, thus, not representative of the whole data population.

Table 2. Cox regression analysis of risk factors NOPB, Category, PipeType, IncidentType and Diameter Range.

Variable	Regression Coefficient (B_i) (*)	Standard Error of B_i	Risk Ratio (**)
B1 (Category = 'DI')	0.779548	1.320206	2.1805
B2 (Category = 'GI')	0.133945	0.676191	1.1433
B3 (Category='MDPE (BLACK)')	0.214512	0.666312	1.2393
B4 (Category='MDPE (BLUE)')	0.204553	0.688775	1.2270

Variable	Regression Coefficient (B_i) (*)	Standard Error of B_i	Risk Ratio (**)
B5 (DiameterRange='MEDIUM')	-0.040946	0.427816	0.9599
B6 (DiameterRange='SMALL')	-0.528034	0.859742	0.5898
B7 (IncidentType='CORROSION')	0.161740	0.412117	1.1756
B8 (IncidentType='INTERF. BY OTHERS')	0.242169	0.423415	1.2740
B9 (IncidentType='OTHER')	-0.241108	0.527373	0.7858
B10 (IncidentType='PIPE DETERIORATION')	0.112606	0.359604	1.1192
B11 (IncidentType='TREE ROOTS')	-0.057751	0.428092	0.9439
B12 NOPB	0.015908	0.009615	1.0160
B13 (NOPBRange='MEDIUM')	0.186366	0.221957	1.2049
B14 (NOPBRange='SMALL')	-0.198185	0.218228	0.8202
B15 (PipeType='WATER MAIN')	-0.391782	0.722911	0.6759

(*) This is the estimate of the regression coefficient, β_i. It is the amount that the log of the hazard rate changes when x_i is increased by one unit.

(**) This is the value of $\exp(\beta_i)$. It is the ratio of two hazards whose only difference is that x_i is increased by one unit.

Survival plots and hazard rate plots are then developed for each different data group: material type (Fig. 1), incident type (Fig. 2), diameter range (Fig. 3).

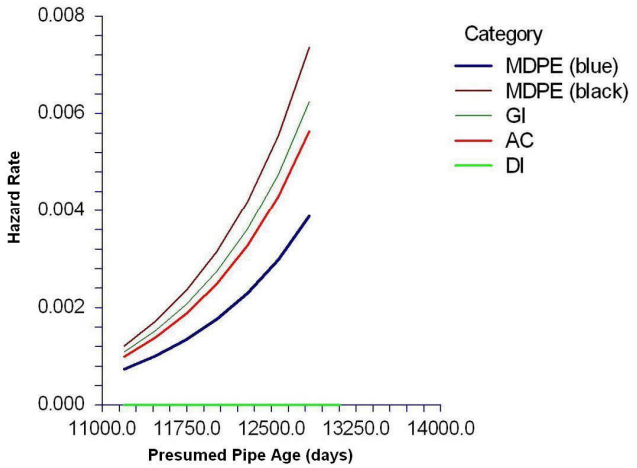


Figure 1. Hazard rate vs. presumed pipe age (in days) for data stratification based on material type.

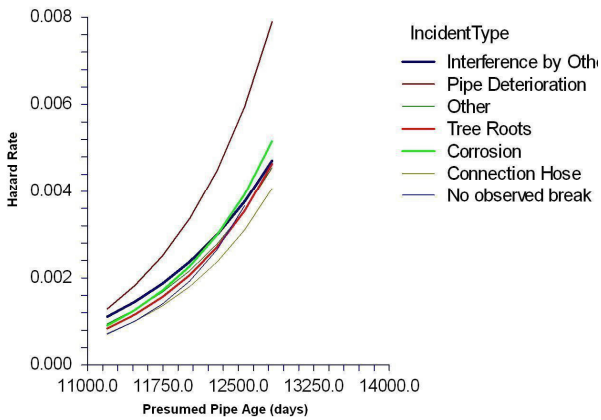


Figure 2. Hazard rate vs. presumed pipe age (in days) for data stratification based on incident type.

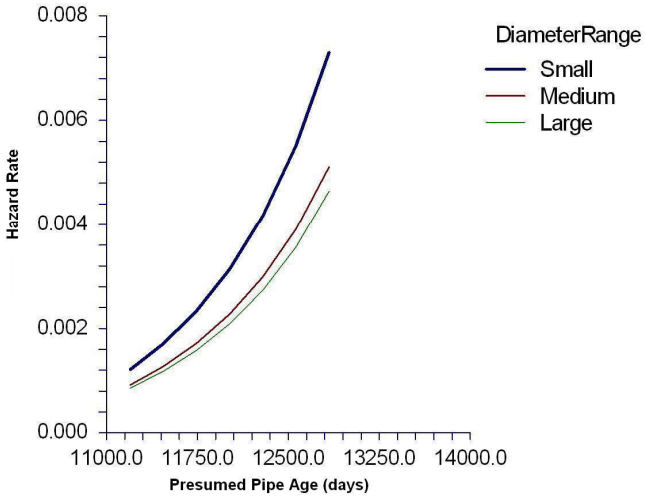


Figure 3. Hazard rate vs. presumed pipe age (in days) for data stratification based on diameter range.

From the figures, it can be deduced that:

- The hazard rate of the black-colour medium density polyethylene (MDPE) pipes is higher than galvanized (GI), asbestos cement (AC) and MDPE-blue pipes. The data sample on ductile iron (DI) pipes is too small to produce statistically significant results and should therefore be ignored.
- The hazard rate related to pipe deterioration greatly outpaces the hazard rate of the other incident types (corrosion, interference by others, tree roots, connection hose, other). What starts as a relatively small deviation at the start of the observed failure time (at about 30 years) soon doubles (0.008 vs 0.004) at the upper end of the expected lifetime (at t~35 years). And while the hazard rate for incidents attributed to tree roots, corrosion, connection hose or other reasons increases almost uniformly over time, the deterioration-related hazard rate accelerates over time (from 0.001 to 0.008). This is an indication that aging pipes should be replaced at smaller time-intervals (at about 11,000 days ~30 years), otherwise the risk of failure increases substantially. Also of interest is the observation that the hazard rate related to tree roots accelerates in time and surpasses the rate of increase of corrosion-related, interference-related and connection-hose incidents. This is an indication that pipes in the vicinity of trees should be monitored more closely and/or replaced in smaller time intervals than other pipes.
- Medium and large diameter pipes have approximately the same hazard rate over time but small-diameter pipes have an increasing hazard rate. This is an indication that small-diameter pipes should be replaced at smaller time intervals than medium and large-diameter pipes. The observation may be attributed to the fact that the pipe network in study consists primarily of small-diameter pipes (neighbourhood and house connection pipes) that operate under relatively high pressures and therefore are subject to increased risk of failure.

5. CONCLUSIONS

The study identifies several risk factors and their significance to failure, based on a five-year dataset (about 2,000 waterloss incidents). Factors such as pipe deterioration, material type, pipe diameter, number of observed previous breaks and tree roots. The survival analysis, though, and the data stratifications employed provide an additional insight to these risk factors, furnishing the pipe network managers with a powerful decision support tool for ‘repair or replace’ strategies. Furthermore, the nonparametric nature of the Kaplan-Meier estimator does not necessitate an a-

priori knowledge of multi-factored probability distribution functions for the estimation of the pipes' probability of failure, thus simplifying the analysis. A limitation of the method is its reliance on fairly large datasets in terms of time-span, or on reasonably accurate estimation of the lifetimes of failed pipes.

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WATER SUPPLY IN SLOVAKIA

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Abstract: The development of the Slovak public water supplies has fallen behind other European countries. In 2007, only 86,3% of the population was connected to public water systems, which supplied them with drinking water. Paper deals with issues relating to water resources, water utilization, water quality and protection, water supply systems. The major objective of the Slovak Republic (SR) as a member of the European Union (EU) includes adopting and implementing EU legislation with respect to the Slovak environment. This is a highly demanding task, in particular for those Slovak authorities and organizations responsible for meeting these requirements. The task is especially formidable since, besides resolving all major current problems, they are also expected to outline the future perspectives of Slovak water management.

Key words: water management, water protection, water resources, water supply.

1. INTRODUCTION

The Slovak Republic (territory 49 014 km^2; population 5,4 million) is situated in the temperate climate zone of the Northern hemisphere with regularly alternating seasons. About 38 % of the country is forested. Based on measurements of the average annual air temperature, the warmest part of the country is the area of Štúrovo in the south (10,4 ^{o}C); while the peaks of the High Tatras in the north, in particular the Lomnický Peak (– 3,7 ^{o}C), are considered the coldest. The average amount of precipitation is around 760 mm, with the Danubian Lowland being the driest (below 550 mm) and the High Tatras the most humid part of the country (above 2 000 mm).

Since the Slovak Republic is situated on the watershed of the Black Sea (96 %) and Baltic Sea (4 %), which is the territory where the majority of European rivers rise (excluding the Danube), the regularity of hydrological regime, in particular the high and medium level discharges in the springtime are of great importance.

2. WATER RESOURCES

2.1. Surface Water

In dry periods, the capacity of natural surface water sources amounts to approximately 90,3 m^3.s $^{-1}$. When subtracting ecological discharges (minimum discharge needed for supporting and maintaining aquatic life and other functions of a stream), only 36,5 m^3.s^{-1} is available for utilization (excluding the Danube, Morava and Tisa rivers). Water reservoirs across Slovakia allow increasing the discharges in dry periods by 53,8 m^3.s^{-1}, thus increasing utilizable discharges to 90,3 m^3.s^{-1}.

At present, there are 54 water reservoirs across Slovakia (with the overall capacity exceeding 1 million m^3) with the gross controllable capacity of 1 890 million m^3. The capacity of these reservoirs allows for the interception of about 14 % of the annual mean discharge from our

country's territory, as well as the increase of low discharges in dry periods by about 55,5 $m^3.s^{-1}$ (above discharge Q355d 80 $m^3.s^{-1}$). Thus the total increased discharge in rivers initiating within the territory of Slovakia reaches approximately 135,5 $m^3.s^{-1}$ ($Q_{355}d$ increased by 69 %). Water withdrawal, currently amounting to 39,0 $m^3.s^{-1}$, is equal to about 29 % of the discharges during dry periods and to 10 % of the mean discharge. The water consumption in the territory of SR, varies between 4,8 to 9,0 $m^3.s^{-1}$ and decreases during an average year by 1,5 to 2,3 %. The previously mentioned water reservoirs include 8 tank reservoirs. Their major purpose is to ensure large-scale drinking water supplies for Northern, Central and Eastern parts of Slovakia. The capacity of this reservoirs is approximately 4,0 $m^3.s^{-1}$

For the utilization of surface water in Slovakia in 2007 see Table 1.

Table 1 Use of surface water in Slovakia in 2007 in $l.s^{-1}$

Public Supply	Industry	Irrigation	Other agriculture	Total
1676,9	9564,7	504,2	42,9	11284,6

Source: SHMI

2.2 Ground Water

Ground water in the territory of SR is considered extremely important because it represents the major source of drinking water. Although the hydrogeological conditions of accumulation, circulation and production of ground water are generally favorable, the uneven distribution of ground water resources and the fact that there are some hydrogeological structures practically without useable ground water supplies, represent a disadvantage.

The Eastern Slovak region has significantly lower documented volumes of useable ground water (17 % of the total). The remaining 27 % represents the resources of the Central Slovak region.

According to the Slovak Hydrometeorology Institute's (SHMI) documentation from December 2006, the natural ground water resources in the territory of Slovakia represent 146,720 $m^3.s^{-1}$, out of which 76,748 $l.s^{-1}$, i.e. 52%, are documented usable sources. (Ministry of Environment of the Slovak Republic. 2007).

Usable ground water resources are defined as those ground waters that can be withdrawn from the subsurface by technical means, while keeping the natural balance of the environment. (Ecological limits).

The established utilizable volumes of ground water are broken down into two basic categories.

- Sources and supplies approved by the Committee for Available Groundwater Quantity Classification (AGQC). They are classified into categories A, B, C, C1 and C2 according to the degree of the quality of verification and the level of knowledge of supplies and classification principles by Slovak Technical Standard (STS) 75 72 21. In 2006 the individual categories of the usable ground water supplies were: category A 851,0 $l.s^{-1}$, category B 2 126,6 $l.s^{-1}$, category C 2 877,9 $l.s^{-1}$, category C1 26 584,1 $l.s^{-1}$, category C2 12 969,4 $l.s^{-1}$and total are 45 409,0 $l.s^{-1}$
- Categories A and B include ground water sources already used as drinking water supplies; categories C1 and C2 include sources ready to be used as drinking water supplies after water treatment.
- Sources and supplies of ground water that have not been approved by AGQC. Data on their volume are documented and broken down into three categories: I, II, III, according to the degree to which they were studied and the reliability of gathered data. In 2006, the amount of useable ground water resources not approved by AGQC was: 31 390,3 $l.s^{-1}$. For the utilization of ground water in Slovakia in 2006 see Table 2.

Table2. Use of ground water in Slovakia in 2007, in $l.s^{-1}$

Public Supply	Food Production	Other Industry	Agriculture and Live-stock Farming	Cultivation of Plants	Community Purposes	Other	Total Use
8836,1	295,6	852,3	275,8	95	340,2	970,2	11665,2

Source: SHMI

3. SURFACE AND GROUND WATER QUALITY

3.1. Surface Water Quality

The quality of surface water in the Slovak Republic is assessed by summarizing results of classification as defined by the STS 75 7221 (*Surface Water Quality Classification*), using follow major indicator groups (A-Microbiological and biological indicators, B-Physical and chemical indicators, C-Radiological indicators). Depending on the water quality, the streams are classified into categories of cleanliness I-V Group. The oxygen regime indicators of the Slovak streams have developed favorably. Since 1990, the percentage of the worst quality category V has rapidly decreased at sampling sites. In 2006, 9,8 % of the sampling sites were classified as category V. This improvement can also be expressed in kilometers. According to the oxygen regime indicators, in 2006, there were 224,4 km of streams polluted enough to receive the category V rating (6,33% of the total assessed length); in 1995, it was only 160 km (4,6% of the total assessed length), while the overall length of monitored streams was about the same. The 630 km of streams falling under the undesirable category IV in 2006 (19,2 % of the sampling sites) decreased to 530 km in 1995 (12 % of the sampling sites).

From the negative indicators, biological and microbiological indicator groups D and E should be mentioned, since the results of water quality assessment by the indicators of these groups have been classified as categories V and IV in the majority of monitored sampling sites (15,63% - D and 77,94% - E in 2006). In 2007 the quality of Slovak surface water was monitored in 199 sampling sites.

3.2 Ground Water Quality

In 2006, the ground water quality was monitored in 26 areas that are considered significant in terms of water management, mainly in river alluvia and in Mesozoic and neovolcanic complexes, primarily throughout the basic network of SHMI stations, supplemented by wells and utilized as well as non-utilized springs. The overall monitoring network consists of 368 monitoring sites. In 2006 there were 334 monitoring sites including 219 wells of the basic monitoring network, 27 used survey wells, 17 not used survey wells, 46 used springs and 25 not used springs. The samples of groundwater were taken once a year. The Rye Island (Danube Lowland) represents a separate section of the Slovak ground water quality-monitoring network. In 2006, the ground water quality was monitored in 34 monitoring stations in 4 areas there, with a monitoring frequency of four times a year for basic monitoring and twice a year for additional monitoring.

The assessment of the results of analyses was performed using comparison of limit values of indicators from Regulation No. 354/2006 on Requirements Quality of Water Intended for Human Consumption and Control of Quality of Water Intended for Human Consumption.

The ongoing monitoring has proved that in monitored areas problems are emerging relating to negative oxidation-reduction conditions in ground water. This has been proven by frequently increased concentrations of Fe, Mn and NH_4.

The prevailing type of land use in the monitored areas (urban or/and agricultural areas) is reflected in the relatively frequent increases in the parameters of oxidized and reduced forms of

nitrogen in the water. The most frequently recorded concentrations of trace elements included mostly aluminum. However, together with pollution caused by other indicators, it occurred only locally.

3.3. Drinking Water Quality

In 2006 as many as 13,334 drinking water samples from sampling sites in water distribution network were analyzed in laboratories of water companies. In these laboratories 366,397 analyses were done concerning particular parameters of drinking water quality.

The most numerous were parameters affecting sensorial properties of drinking water where 133,602 analyses were done as well as microbiological and biological water quality parameters where 126,168 samples were analyzed. In 2006 the portion of drinking water analyses meeting hygienic limits reached the value 99.44 %. The STS fully corresponds with Drinking Water European Standard.

Number of samples meeting the requirements for drinking water quality concerning all parameters reached the value 91.18 % in 2006. Hygienic limits in drinking water supply system were not met mostly individually by the following parameters:

- Microbiological and biological parameters:
  - Escherichia coli, coliform bacteria, enterococci, living micro-organisms, cultivated micro-organisms at 22 ºC and 36 ºC
- Inorganic and physical-chemical parameters:
  - Antimony, arsenic, nitrates, colour, manganese, turbidity, iron
- Radiological parameters:
  - Total volume alpha-activity, radon 222 volume activity
- Disinfectants (chlorine, chlorine dioxide, chloramines, ozone) and their by-products:
  - THM_3, chlorites, chlorates, bromides etc.

The positive fact in physical-chemical parameters in 2006 is that no case of exceeding the limits for heavy metals and specific organic substances occurred within the drinking water quality monitoring.

4. WATER RESOURCE PROTECTION

Water resource protection should be viewed as an integrated protection of quality and quantity of surface and ground water, including natural curative springs and mineral waters. The major determinant in terms of water resource quality protection is the water pollution source, having either direct or indirect impact on water resources. Water quantity protection is based on increasing the accumulation capacity of watersheds and aquifers and on controlling the observance of calculated values for the volumes of withdrawn water. This has done by establishing ground water utilization limits (ecological limits), as well as binding minimal discharges (ecological discharges - MQECO) in the streams, in conformity with the principles of surface water management in the respective watersheds.

Both aspects of water protection (quantitative and qualitative) are subordinated to a territorial water protection system, in particular in such source areas that are considered significant from the perspective of water management. The system consists of three types of protection: general protection, resulting from the Act on Waters No. 364/2004; broader regional protection, implemented by means of protected water management areas; and special protection with increased severity – a specific protection of water sources utilized for drinking purposes implemented by means of hygienic protection zones.

Broader regional water protection is implemented by means of protected water management areas (PWMA, also called "Protected areas of natural water accumulation"). Currently there are 10 PWMAs declared in SR covering a territory of 6 942 km^2.

5. PUBLIC WATER SUPPLIES

The total number of inhabitants supplied with drinking water from public water supply network increased in 2006 compared to the previous year by 59.3 thousand to 4,653.4 thousand inhabitants that is 86.3 % out of the total number of population of the Slovak Republic. Increase of supplied inhabitants in 2006 represented 1.0 of percentage point. The level of public water supply network development is regionally unbalanced. The largest proportion of supplied population is in the Bratislava Region and higher proportion than the total average is also in Trenčín, Žilina and Trnava Region. The public water supply network development in Banská Bystrica, Košice and Prešov regions falls behind the total average. Much more differentiated condition concerning drinking water supply occurs in particular districts where the proportion of supplied inhabitants is moving up from approximately 60 % up to the saturation level in some regions.

Development of the total number of inhabitants and the number of inhabitants supplied with drinking water is in the Table 3.

Table 3 Total populations and population supplied with drinking water from the public water supply systems

	1995	1996	1997	1998	1999
Total (number of) population in thousands	5363,7	5373,9	5383,2	5394,4	5395,3
Population supplied by WSW, thousands	4256,8	4290,4	4351,6	4410,1	4447,8
Proportion (%)	79,4	79,8	80,8	81,8	82,4

	2000	2003	2004	2005	2006
Total (number of) population in thousands	5,400.6	5,380.1	5,384.8	5,386.7	5,390.4
Population supplied by WSW, thousands	4,479.2	4,535.1	4,569.1	4,594.1	4,653.4
Proportion (%)	82.9	84.3	84.9	85.3	86.3

Source: Water Research Institute

In the facilities of water companies, local authorities and other subjects there was produced 334.3 mil. m^3 of drinking water in 2006, that means the decrease by 3.7 mil. m^3 compared to 2005. Table 4 indicates that the decrease of invoiced water continued – in 2006 it decreased by 7.0 mil. m^3 in total. The amount of invoiced water represented 66.5 % out of the total amount of water intended for supply.

Since 1990, water consumption in Slovakia has been constantly decreasing. This phenomenon is caused partially by decreasing industrial production, but also by the gradual decrease in public water consumption due to increasing drinking water prices. The specific demand for drinking water has decreased since 1990 by almost 45 %; the current consumption value is 89,5 $l.person^{-1}.day^{-1}$. Compared to EU countries, which consume approximately 145,0 $l.person^{-1}.day^{-1}$, the specific water demand in Slovakia decreases year after year. It is alarming situation mostly because the present high costs which is fluctuate depends of the water works (14 mains companies) from 0,65 to 1,00 € for drinking water lead to construction of own sources of drinking water with the quality far behind the hygienic standards.

The consumption of drinking water is falling deep below the average, leaving us dangerously close to approaching the minimal hygienic standards. This trend also applies to other water use

categories, since the demand of industrial and other users – compared to 1990 – has decreased by 30,2 %.

Based on analyses, the specific household water demand should reach the bottom line (80,0 l.person^{-1}.day^{-1}) by 2010; and consequently, the overall water use is predicted to decrease to 165 l.person^{-1}.day^{-1}. The average specific water demand for households in 2030 is predicted to reach 145 l.person^{-1}.day^{-1}, and the overall specific water demand 220 l.person^{-1}.day^{-1}.

The overall volume of non-metered water has decreased to 114,2 million m^3 - that is 25 % of the total water production; more than 83% of that represents losses in the distribution network (27,2% of produced water). Measures should be adopted and taken to decrease the losses in the piping system to an acceptable rate, corresponding to European standards.

Construction of public water supply network led to an increase in number of technical facilities and structures. Compared to 2005 the total length of water supply systems in Slovakia (water companies, local authorities and other subjects) increased by 638 km up to the total length 26,357 km. Data on water supply and water supply network development are listed in Table 4.

Table 4 Water delivery and trends in development of water supply systems in WSW administration

No.	Indicator	Unit	Year					
			1999	2004	2005	2006	2007	2008 exp.
1.	Population supplied by water supply systems (WSS)	thousand	4 014,4	4569,1	4594,1	4653,4	4 690	4 720
2.	Surface water resource capacity	l.s^{-1}	29 308	33 855	33 848	33 546	33 500	33 500
3.	Distribution network length	km	22 950	25 313	25 719	26 357	26 400	26 700
4.	Ground water resource capacity	l.s^{-1}	24 396	28 413	27 921	27 713	27 650	27 650
5.	Water produced in WSW facilities	mil.m^3	402,5	353,2	338,0	334,3	327,4	324,8
	from ground water	mil.m^3	336,0	298,6	287,6	280,6	278,0	275,0
6.	Total water produced	mil.m^3	402,8	356,6	343,1	339,2	331,6	329,1
7.	Metered water - total	mil.m^3	286,5	241,6	232,0	225,0	224,8	224,1
	- households	mil.m^3	185,9	166,2	159,1	152,1	152,2	152,0
8.	Non-metered water	mil.m^3	116,3	115,0	111,1	114,2	106,8	105,0
	of which: losses in the piping network	mil.m^3	96,8	93,2	93,2	94,2	91,9	90,0
9.	Specific water consumption	l.per.day^{-1}	126,9	99,7	94,9	89,5	88,9	88,2

Source: Water Research Institute

5. CONCLUSION

By the Slovak constitution streams, ground water, mineral water, and natural curative springs are in the ownership of the Slovak Republic. Therefore, in harmony with the needs of society and with the principles of the European Water Chart, the more intensive water utilization and the higher the level of “civilization” of society, the more responsibly and diligently the waters have to be protected, regulated and replenished. Therefore, it is of primary importance that both the surface water and ground water are administered by organizations fully ensuring the direct application of

state interests, in particular in the rational utilization of water, water protection and the fair distribution of these irreplaceable natural resources.

The Slovak state water management policy for the future has to be based on the state water management objectives, which include in legislations of Ministry of Soil Management of the SR, Ministry of Environment of the SR and Ministry of Health of the SR, which include in:

- Providing sufficient volume and quality of drinking water for inhabitants and all consumers
- Providing sufficient volume of adequate-quality supply of water for industry, agriculture, power generation and other purposes and services with a high level.
- Overall treatment of the volume of utilized and polluted water before discharging it back to its natural environment.
- Achieving a high level of environmental protection within the context of sustainable development.
- Achieving an adequate level of flood prevention in residential and industrial territorial units and areas of transport infrastructure and intensive agricultural production.
- Achieving of water resources and facilities protecting against damages caused by drought.

The ultimate goal of Slovakia is to reach level in water supplying and connection to sewerage in well-developed countries in the European Union. This goes hand in hand with assuming the responsibilities relating to the harmonization of water management policy with that set by the European Union legislation, which is enforced as an inseparable part of the right to a healthy and clean environment.

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EVALUATING THE PERFORMANCE OF AN URBAN WATER DISTRIBUTION NETWORK: TIPS AND TRICKS FOR TOUBLESHOOTING

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Abstract: The whole world is experiencing water shortage and climate change conditions more than ever, driving all the people involved (governments, water utilities, stakeholders, etc.) to manage water resources more efficiently. Water losses in urban water distribution networks due to leaks and breaks are almost one third of the water supposed to be supplied to the citizens. Water Utilities must take radical measures in order to significantly reduce the levels of Non-Revenue Water (NRW). Therefore, the IWA standard international water balance and some of the related performance indicators (PIs) are being calculated in this paper. The Larisa city case in Greece was chosen as a case study in order to identify the problems faced during the implementation process and find out ways to tackle them. A city in Greece was chosen especially because the country is facing water shortage problems and Greek Water Utilities miss many of the most important data for the implementation of the water balance and the calculation of the PIs.

Key words: performance indicators, IWA standard international water balance, water losses.

1. INTRODUCTION

An urgent need for a common international terminology is apparent the last decades since there is a wide diversity of definitions used for water balance calculations internationally or even within the same country. This made the IWA Task Forces on Water Losses and Performance Indicators to develop an international "best practice" standard approach for water balance and PIs calculations using best practices around the world (Farley et al., 2003). In Greece, this need is even more urgent since the country is facing water shortage problems. As the Water Utilities in Greece are far behind in implementing indicators to measure the water being lost, the authors are currently running research projects helping them to design and implement effective NRW reduction strategies.

The Larisa city case is presented in this paper focusing on the tips and tricks used in order to tackle the problems faced during the implementation of the project. Local, national and international experience is being used to this end. For each one of these factors a sensitivity analysis took place to check their influence in the water balance outcome and the levels of the chosen PIs.

2. CALCULATING THE WATER BALANCE OF A WATER NETWORK

The IWA Task Forces on Water Losses and Performance Indicators produced an international "best practice" standard approach for water balance calculations (fig.1), with definitions of all terms involved (Farley and Trow, 2003):

- *System input volume (SIV)* is the annual volume input to that part of the water supply system.
- *Authorised consumption* is the annual volume of metered and/or non-metered water taken by registered customers, the water supplier and others who are implicitly or explicitly authorised to do so. It includes water exported, leaks and overflows after the point of customer metering.

- *Non-revenue water (NRW)* is the difference between system input volume and billed authorised consumption. NRW consists of: a) unbilled authorised consumption and b) water losses. *Unbilled authorised consumption* includes items such as fire fighting and training, flushing of mains and sewers, cleaning of suppliers' storage tanks, filling of water tankers, water taken from hydrants, street cleaning, watering of municipal gardens, public fountains, frost protection, building water etc. *Water losses* are the difference between system input volume and authorised consumption and consists of apparent and real losses.
- *Apparent losses* consist of unauthorised consumption (theft and illegal use) and all types of metering inaccuracies [a) random errors due to accounting procedures (misread meters, incorrect estimates for stopped meters, improper calculations etc.), b) systematic errors due to under-registration or over-registration of customer meters].
- *Real losses* are the annual volumes lost through all types of leaks, bursts and overflows on mains, service reservoirs and service connections, up to the point of customer metering.

<table>
<tr><td rowspan="6">Annual System Input Volume</td><td rowspan="3">Authorised Consumption</td><td rowspan="2">Billed Authorised Consumption</td><td>Billed Metered Consumption</td><td rowspan="2">Revenue Water</td></tr>
<tr><td>Billed Unmetered Consumption</td></tr>
<tr><td rowspan="2">Unbilled Authorised Consumption</td><td>Unbilled Metered Consumption</td><td rowspan="4">Non - Revenue Water</td></tr>
<tr><td>Unbilled Unmetered Consumption</td></tr>
<tr><td rowspan="3">Water Losses</td><td rowspan="2">Apparent Losses</td><td>Unauthorised Consumption</td></tr>
<tr><td>Customer Meter Inaccuracies and Data Handling Errors</td></tr>
<tr><td colspan="2">Real Losses</td></tr>
</table>

Figure 1 IWA Water Balance (Farley and Trow, 2003)

Table 1 IWA Performance Indicators (Alegre et al., 2006)

Indicators	Number	Indicators	Number	Indicators	Number
Water Resources (WR)	**4**	**Operational Indicators (Op)**	**44**	**Economic & Financial Indicators (FI)**	**47**
Personnel (Pe)	**26**	Inspection & maintenance	6	Revenue	3
Total Personnel	2	Instrumentation calibration	5	Cost	3
Personnel per main function	7	Vehicle availability	1	Composition of running costs per type	5
Technical services personnel per activity	6	Electrical & signal transmission equipment inspection	3	Composition of running costs per main function of the water undertaking	5
Personnel qualification	3	Rehabilitation	7	Composition of running costs per technical function activity	6
Personnel training	3	Operational water losses	7	Composition of capital costs	2
Personnel health & safety	4	Failure	6	Investment	3
Overtime work	1	Water metering	4	Average water charges	2
Quality of service (QS)	**34**	Water quality monitoring	5	Efficiency	9
Service coverage	5	**Physical Indicators (Ph)**	**15**	Leverage	2
Public taps & standpipes	4	Water treatment	1	Liquiduty	1
Pressure & continuity of supply	8	Water storage	2	Profitability	4
Quality of supplied water	5	Pumping	4	Economic waterlosses	2
Service connection & meter installation & repair	3	Valve, hydrant and meter availability	6		
Consumer complaints	9	Automation & control	2		

Table 2 Comments on the appropriateness of the different PIs' expressions (Alegre et al. 2006, Liemberger et al. 2007)

PI	Group	Measure	Condition	Comment
Non-Revenue Water	Financial	Volume of NRW as % SIV		influenced by non-fixed parameters
		NRW % by cost		more appropriate
Apparent Losses	Operational	Volume of AL as % SIV		poor indiactor
		Volume of AL as % Water Billed		poor indiactor
		m3/service connection/day		more appropriate
		litres/service connection/day		more appropriate
		litres/metered property/day		more appropriate
		% of water supplied		more appropriate
		% of authorised consumption		more appropriate
Real Losses	Operational	Volume of Real Losses as % SIV		influenced byconsumption
		per billed account or per property		multiple properties
		litres/service connection/day	**service connections>20/km mains**	more appropriate
		m3/km mains/day	**service connections<20/km mains**	more appropriate
		litres/service connection/day when pressurised	**intermittent supply**	intermittent supply
Water Losses	Operational	m3/service connection / year		
UARL		m^3/km of mains/day/metre of pressure		depends on service connections
		in litres/service connection/day/metre of pressure	**service connections>20/km mains**	more appropriate

The PIs evaluate the efficiency and effectiveness of the water services. IWA published a manual of best practice (Alegre et al., 2006) including 170 different PIs arranged in the following groups (table 1). PIs range from basic ones that provide a general management overview of efficiency and

effectiveness to detailed PIs which deal with specific elements of operational management (table 1). Trying to finger out which one of the PIs is the most suitable, a literature review concerning comments on the PIs takes place resulting in table 2.

3. IWA WATER BALANCE FOR LARISA DISTRIBUTION NETWORK

The city of Larisa is located in the eastern part of the Greek mainland, being the capital city of the Thessaly Region. According to the latest census, the water distribution network serves 192,000 inhabitants, through 628,081 km of pipes and 72,000 meters (data of 2006). The pipes are made of cast iron, asbestos cement, steel, PVC and PE (fig. 2). The municipal Water Utility (DEYAL) runs the entire water supply system. The water supplies come from three groups of water bores. The water is being stored in two water tanks from where it is supplied through water mains (free flow) in the two pressure zones of the network. DEYAL keeps detailed data records regarding the water supplied, consumed and metered (fig. 2). In 2006 the network was divided in 4 pressure zones with the operating pressure ranging between 2.5 atm and 6 atm, and an average value of 3.6 atm.

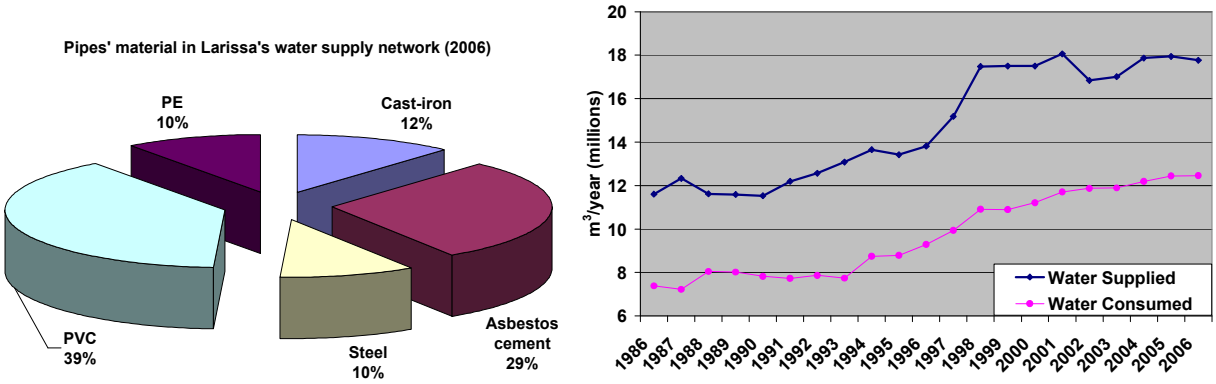


Figure 2 Pipes material, water supply and consumption for Larisa city (1986-2006)

3.1 Results of the IWA Water Balance calculations for the Larisa case

Based on the data records kept by DEYAL, the system's water balance was estimated for six consecutive years (2001-2006) (fig.3 for 2001 and 2006). There were several assumptions involved being analyzed in the tips and tricks section: (a) the minimum water use being charged according to the billing system used was not taken into account - the meters' readings were used; (b) the unbilled authorised consumption was considered to be equal to the unbilled metered consumption; (c) the unauthorised consumption was calculated equal to 1% of SIV; (d) the metering errors were estimated to be 5% of SIV; and (e) the average water tariff was put in the model (1.37 €/m^3 for the years 2001-2003 and 1.45 €/m^3 since 2004) in order to calculate the NRW values (fig.4).

<table>
<tr><td rowspan="5">Annual System Input Volume
18,059,662 m3/year</td><td rowspan="3">Authorised Consumption
11,705,246 m3/year</td><td>Billed Authorised Consumption
10,977,845 m3/year</td><td>Billed Metered Consumption
10,977,845 m3/year</td><td>Revenue Water
10,977,845 m3/year</td></tr>
<tr><td></td><td>Billed Unmetered Consumption (0 m3/year)</td><td rowspan="4">Non - Revenue Water
7,081,817 m3/year</td></tr>
<tr><td>Unbilled Authorised Consumption
727,401 m3/year</td><td>Unbilled Metered Consumption
727,401 m3/year</td></tr>
<tr><td rowspan="2">Water Losses
6,354,416 m3/year</td><td></td><td>Unbilled Unmetered Consumption (0 m3/year)</td></tr>
<tr><td>Apparent Losses
1,083,580 m3/year</td><td>Unauthorised Consumption (180,597 m3/year)
Customer Meter Inaccuracies and Data Handling Errors
902,983 m3/year</td></tr>
<tr><td colspan="3">Real Losses (5,270,836 m3/year)</td></tr>
</table>

<table>
<tr><td rowspan="4">Annual System Input Volume
17,770,139 m3/year</td><td rowspan="3">Authorised Consumption
12,458,122 m3/year</td><td>Billed Authorised Consumption
11,687,062 m3/year</td><td>Billed Metered Consumption
11,687,062 m3/year</td><td>Revenue Water
11,687,062 m3/year</td></tr>
<tr><td></td><td>Billed Unmetered Consumption (0 m3/year)</td><td rowspan="3">Non - Revenue Water
6,083,077 m3/year</td></tr>
<tr><td>Unbilled Authorised Consumption
771,060 m3/year</td><td>Unbilled Metered Consumption
771,060 m3/year</td></tr>
<tr><td>Water Losses</td><td>Apparent Losses</td><td>Unbilled Unmetered Consumption (0 m3/year)
Unauthorised Consumption (177,701 m3/year)</td></tr>
</table>

	5,312,017 m3/year	1,066,208 m3/year	**Customer Meter Inaccuracies and Data Handling Errors** 888,507 m3/year	
		Real Losses (4,245,809 m3/year)		

Figure 3 Larisa's IWA water balance for 2001 and 2006

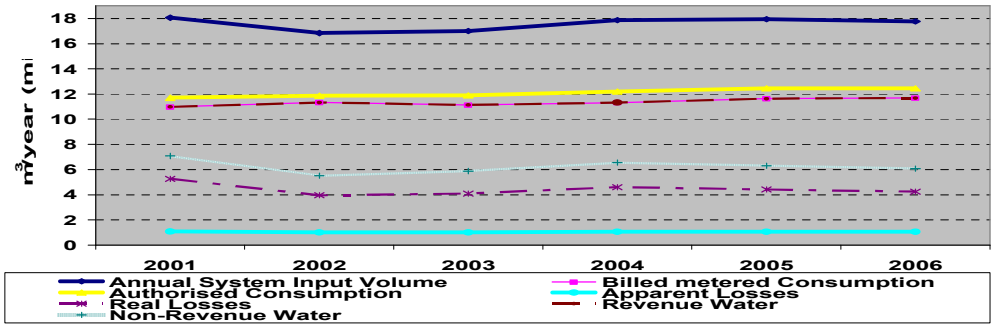


Figure 4 Water data for Larisa water supply network

4. TIPS AND TRICKS – SENSITIVITY ANALYSIS

Many problems had to be tackled trying to figure out the IWA water balance for the Larisa case. Due to the fact that some data was either unreliable or even worse missing regarding specific parameters involved in the calculations, targeted sensitivity analysis controls take place in order to estimate how much these data shortages affect the reliability of the results of the whole research. Therefore, tips and tricks were used in order to estimate the water balance and the related PIs.

Billed Metered Consumption: DEYAL uses a bimonthly billing system. There is a minimum charge of water use ($20m^3$)even if it is not actually being used, considered necessary in order DEYAL's operational expenses to be covered. If the actual water use is greater than this minimum amount of water, then the real consumption is being charged. Therefore in order to perform the water balance calculations correctly, the real consumption was used as billed metered consumption.

Unauthorised Consumption: As DEYAL does not keep any records regarding the unbilled un-metered consumption this amount was assumed to be zero in the water balance. Also, DEYAL does not keep any records regarding the unauthorized consumption, the illegal connections, bypasses, theft, etc. claiming that there are no proofs or strong evidence regarding water theft, illegal connections, or by-passes incidents in the network. Based in the international literature, this amount was assumed to be 1% of SIV (Farley et al., 2003; Kanakoudis and Charavgis, 2005). During the relative sensitivity analysis, the unauthorised consumption was assumed to range between zero and 1% of the SIV (fig. 5). The results showed that the Apparent Losses decrease as the unauthorised consumption gets closer to zero while the Real Losses increase (fig. 5).

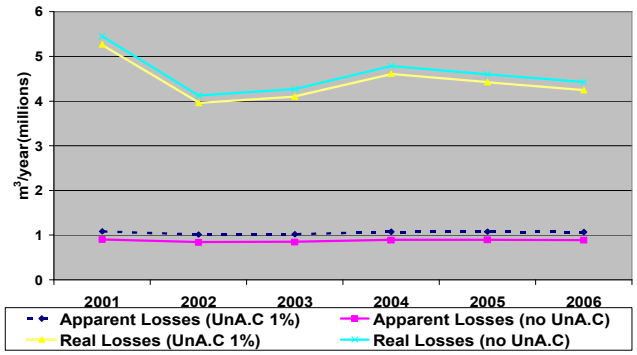


Figure 5 Apparent & Real Losses when Unauthorised Cons. is either SIV's 1 or 0%

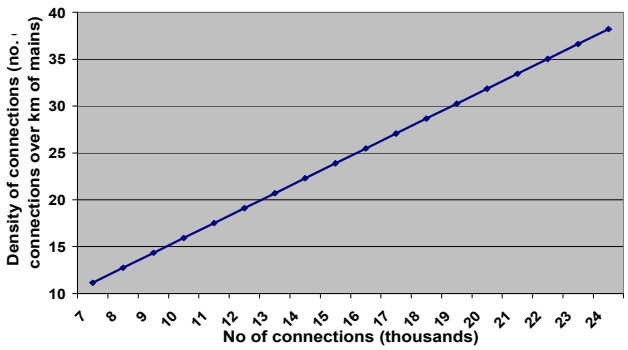


Figure 6 Density of connections depending on the number of connections

Customer Meter Inaccuracies and data handling errors: Water losses due to meter errors, under-registration etc. are also not being registered by DEYAL. Therefore based on the acceptable

metering error level (3%) given by the meters' manufacturer and thus accepted by DEYAL, the water losses related to meter errors, under-registration etc., was considered as 5% of the SIV. This is quite an optimistic approach for a Greek city, as in the case of Athens, capital of Greece, research studies estimated that the meter errors are 15% of the SIV (Kanakoudis and Charavgis, 2005) while at the same time the local Water Utility claims that this error is not more that 9% of the authorized consumption (Georgiadis and Kanellopoulou, 2008). Once more a sensitivity analysis took place in order to check the PIs when the meter errors related water losses level to range between 3-15% of SIV. Results showed that by increasing the meter errors related water losses level by 1% of SIV every year, the Apparent Losses increase and the Real Losses decrease.

Number of Connections: The number of connections is another key local factor faced in order to calculate the UARL and the Infrastructure Leakage Index (ILI). In Larisa the density of service connections varies a lot depending on the specific part of the city. There are areas where each building is being served by more than 15 water meters and other areas where it is served only by one or two water meters. Sometimes one connection serves two or even three buildings at the same time. The number of connections has never been actually metered in Larisa and based on the fact that there are 72,000 water meters installed, a sensitivity analysis took place assuming that the number of connections range between 7,000 to 24,000 connections (representing 9.7% to 33.3% of the total meters) (fig. 6).

UARL: To calculate the UARL, the length of the service pipes needs to be known. DEYAL didn't provide such data. This length varies a lot among the different parts of the city, from 1 or 2 meters (in the old buildings where the meters are located near the pavement) to 10 meters (in the new buildings where the customers' meters are located in the back). Therefore the length of the service pipes was assumed to be about 7 meters followed by a sensitivity analysis that showed that this length does not really have an impact on the value of the UARL.

Infrastructure Leakage Index (ILI): The ratio of the Current Annual Real Losses (CARL) to the UARL is the Infrastructure Leakage Index (ILI). The ILI measures how effectively the infrastructure activities (repairs, active leakage control and pipeline/assets management) are being managed at current operating pressure. In order to calculate the ILI, the number of connections has to be known. As DEYAL does not have a clear view of this number, a sensitivity analysis took place in order to check the impact of the number of connections to the UARL, Real Losses (fig.7) and ILI values (fig.8). The year 2006 was used as the base scenario. The results showed that as the number of connections decreases, the Real Losses per connection value increases rapidly, while the levels of UARL and ILI increase very slowly and quickly respectively (fig.7;8). A sensitivity analysis revealed that the ILI value does not significantly change when the length of the service connection pipe ranges between 1m and 10m.

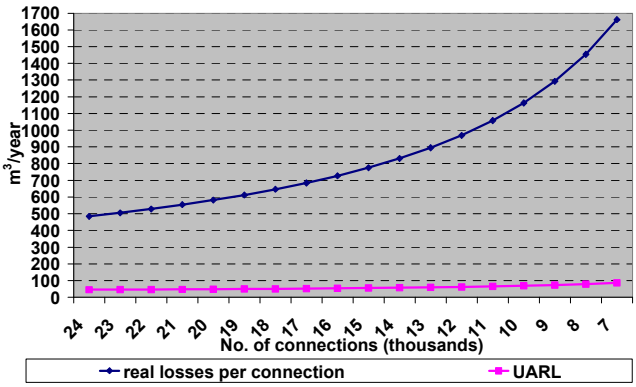


Figure 7 Real Losses per connection and UARL for different number of connections

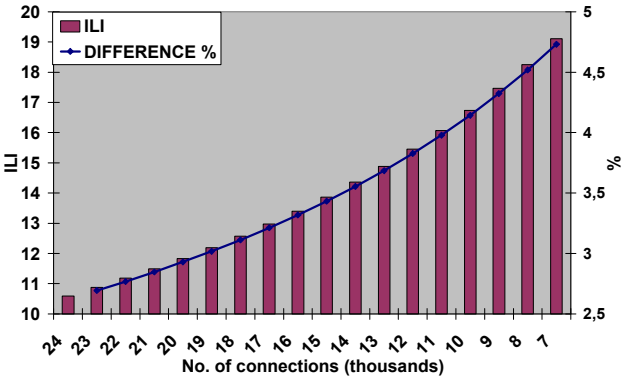


Figure 8 ILI (values and its % differences) for different number of connections

Pressure: ILI values are estimated in the current operating pressure. If the pressure value change then the system has a different ILI value. In the Larisa case the current operating pressure is not stable. DEYAL knows the pressure limits the network is operating. As already mentioned since 2006 the network is divided in 4 distinct pressure zones with the operating pressure ranging

between 2.5 atm and 6 atm. In order to check the impact of the pressure in the values of ILI for different number of connections, a sensitivity analysis took place with the pressure ranging between 31m and 59m (fig.9). The results showed that ILI values decrease as pressure values increase for the same number of connections (fig.9). These calculations do not take into account the differentiation (increase) on the CARL when the pressure values are increasing.

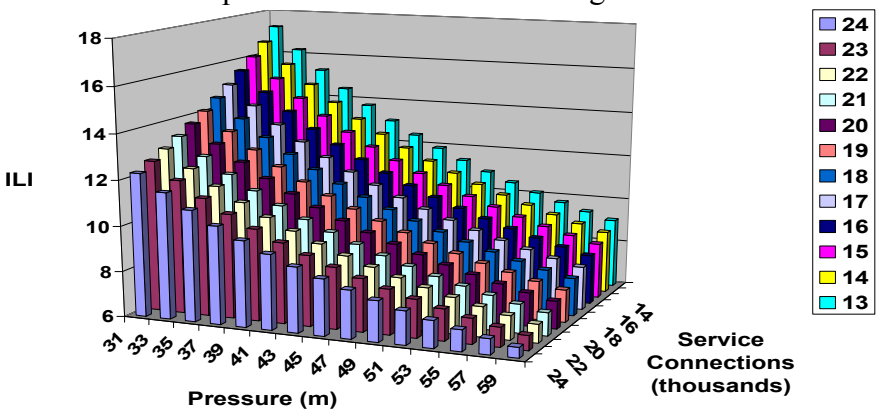


Figure 9 ILI values for different values of operating pressure and number of connections

5. RESULTS OF THE PIS CALCULATIONS FOR LARISA CASE

Table 3 presents the mathematical formulas (Alegre et al., 2006), used to calculate the basic PIs for the case study water distribution network. Figure 10 presents the water losses volumes and their respective values for the period 2001-2006. Water and Real Losses per connection were calculated for different number of connections (13,000-24,000). For connections less than 13,000 the PIs do not really mean anything since the connections' density become less than 20 (fig.6;11). The value of the Apparent Losses per SIV is constant since the water losses related to illegal connections and meters errors were assumed to be 1% and 5% of the SIV respectively. Thus, in order to evaluate the Apparent Losses level, their expressions as % of Authorised Consumption and in lt/connection/day (fig. 12) were used. The highest values are reported for 2001, dropping to 8.5 and 8.6 for 2002 and 2003 respectively and raising to 8.8 in 2004 (fig.12). In 2005 and 2006 the values are dropping again. When the Apparent Losses are measured in lt/connection/day it is obvious that their value increases as the number of connections decreases (fig.12).

Table 3 Performance indicators calculated and their mathematical formulas

Indicators		Formula
Op23	Water losses per connection	(A15*365/H1)/C24
Op24	Water losses per mains length	(A15/H1)/C8
Op25	Apparent losses per connection	[A18/(A3-A5-A7)]*100
Op26	Apparent losses per system volume	A18/A3
Op27	Real losses per connection	(A19*1000)/(C24*H2/24)
Op28	Real losses per mains length	(A19*1000)/(C8*H2/24)
Op29	Infrastructure leakage index	Op27/[18*C8/(C24+0,8+0,025*C25)/(D34/10)]
Fi46	Non-revenue water by volume	(A21/A3)*100

Indicator	Variable	Units	Indicator	Variable	Units
A3	System input volume	m^3	C8	Mains length	Km
A5	Exported raw water	m^3	C24	Number of service connections	-
A7	Exported treated water	m^3	C25	Average service connection length	M
A15	Water losses	m^3	D34	Average operating pressure	KPa
A18	Apparent losses	m^3	H1	The assessment period	Days
A19	Real losses	m^3	H2	The time system is pressurized	Hours
A21	Non-revenue water	m^3			

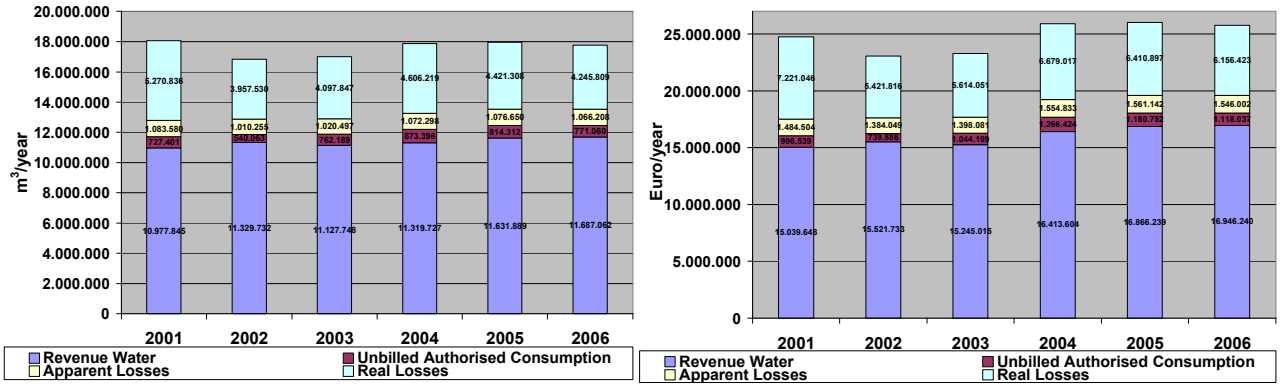


Figure 10 Water Losses volume and water losses value for Larisa water distribution network

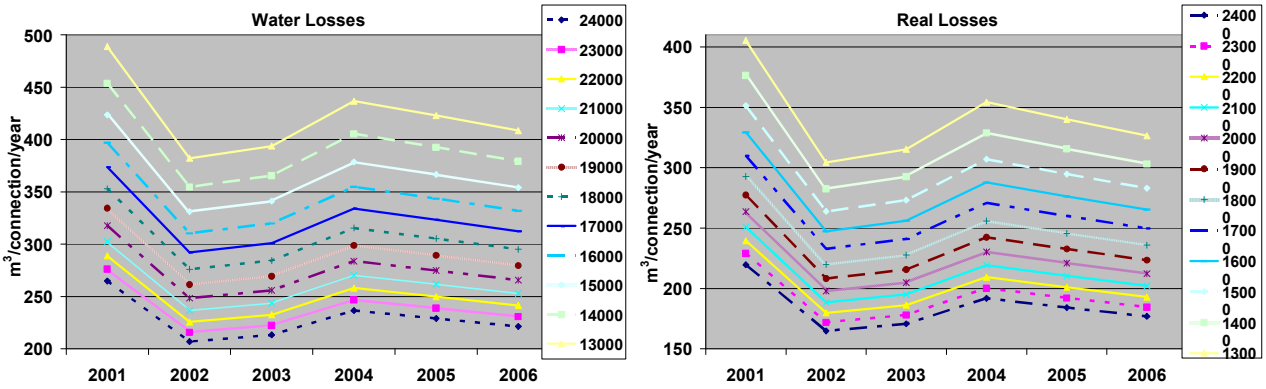


Figure 11 Water and Real Losses per connection for different number of connections

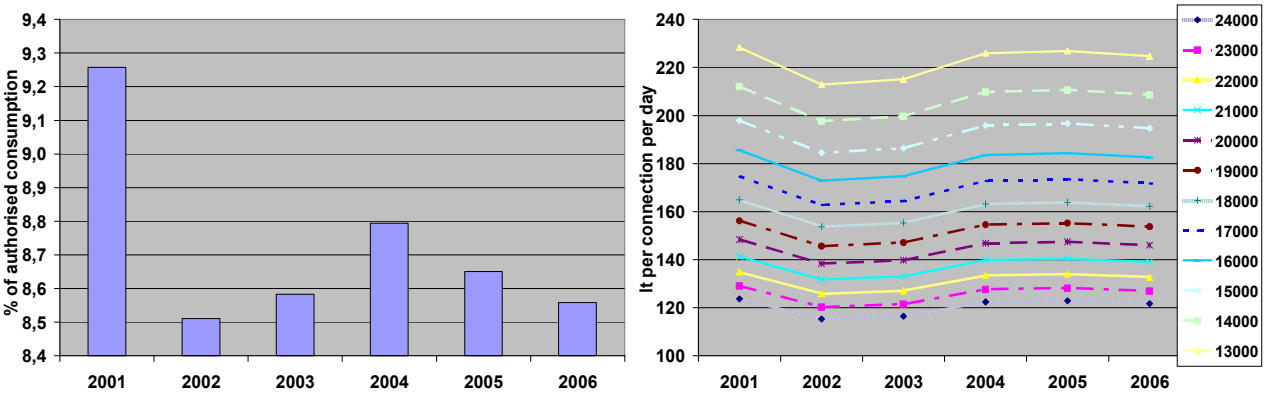


Figure 12 Apparent Losses as % of Authorised Consumption and in lt/connection/day

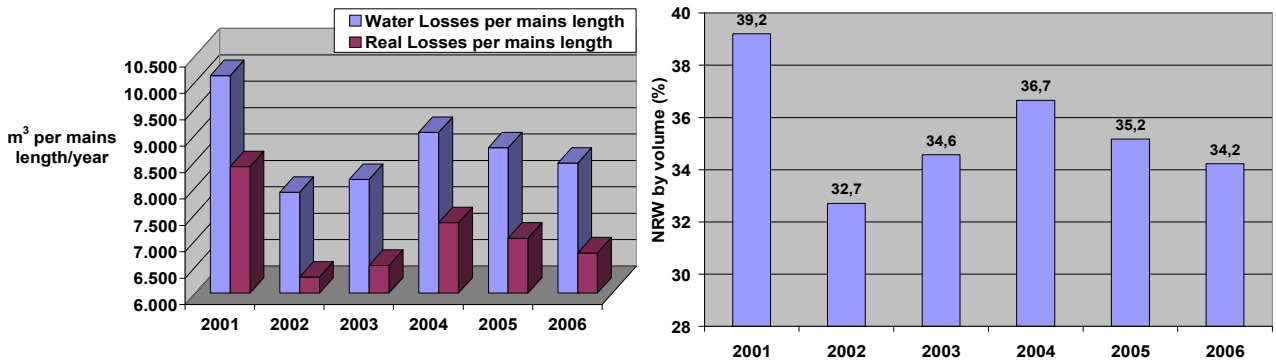


Figure 13 Water and Real Losses per mains length and NRW by Volume (%)

Figure 13 presents the calculated Water and Real Losses per mains length. A radical decrease is observed in 2002 and 2003, followed by a small increase in 2004 and then the values are decreasing again in 2005 and 2006. NRW was calculated as % of the SIV (fig.13) and the tendency of its

values agrees with the one of Water and Real Losses per mains length. Finally the value of ILI is calculated for 2001 until 2006 for different number of connections (fig. 14). The total number of connections proved to be very crucial for the calculation of the ILI levels.

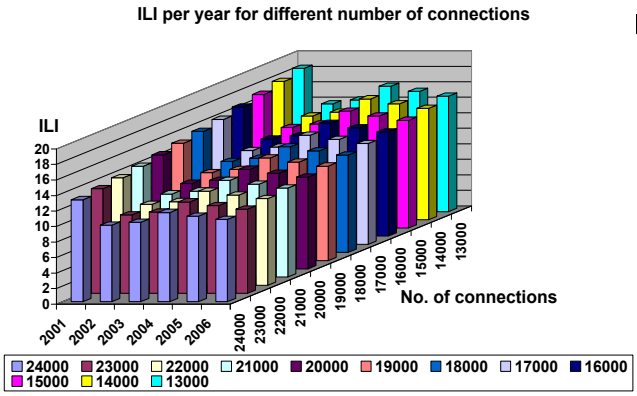


Figure 14 ILI per year for different number of connections for Larisa water distribution network

Physical Loss Assessment Matrix

Technical Performance Category		ILI	Litres/connection/day (when the system is pressurised) at an average pressure of:				
			10 m	20 m	30 m	40 m	50 m
Developed Country Situation	A	1 - 2		< 50	< 75	< 100	< 125
	B	2 - 4		50-100	75-150	100-200	125-250
	C	4 - 8		100-200	150-300	200-400	250-500
	D	> 8		> 200	> 300	> 400	> 500
Developing Country Situation	A	1 - 4	< 50	< 100	< 150	< 200	< 250
	B	4 - 8	50-100	100-200	150-300	200-400	250-500
	C	8 - 16	100-200	200-400	300-600	400-800	500-1000
	D	> 16	> 200	> 400	> 600	> 800	> 1000

Figure 15 Physical Loss Assessment Matrix (Liemberger et al., 2007)

6. CONCLUSIONS

The IWA standard water balance is a diagnostic approach giving to the Water Utility the chance to correctly evaluate its system and design an effective water loss strategy. Nevertheless, some problems usually arise. The purpose of this study was to identify these problems in the case of Larisa and find out ways to solve them. Therefore a number of tricks were used. In Larisa, as what is usually the problem in Greece, the main problem had to do with lacking data. DEYAL is not keen on keeping detailed data apart from the one related to the water being supplied and billed, and has never implemented a project in order to calculate the water losses level related to the illegal connections, the meter under-registration, the malfunction incidents, and the number of customer connections. Thus, a sensitivity analysis took place for each one of these parameters to check how the lacking data affects the PIs' levels. The calculated ILI's values vary a lot depending on the number of connections but they do not change with the length of the service connection pipe. Although ILI takes different values depending on the number of connections, they are all greater than 8 classifying the network in group D, meaning that it needs a leakage strategy to be implemented as soon as possible. Generally ILI's values are dropping in the last few years, probably due to the DEYAL's efforts to better manage the water network by installing pressure reduction valves (PRVs) in 2004.

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A RISK ANALYSIS FRAMEWORK FOR EVALUATING FAILURE OF WATER MAINS

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Abstract: This paper presents analysis results of degradation modeling for water distribution systems in urban environments. The system is based on data from New York City (1982-2002) and the application of parametric and non-parametric survival analysis. Artificial Neural Network techniques are also employed for the identification of the underlying risk factors and their relevance to water main breaks (about 12 factors were reviewed, including age of pipe, diameter, material, location, previous breaks, etc.). The statistical models used were calibrated for different groups of pipes and a combination of parametric (Weibull distributions) and non-parametric models (Kaplan-Meier survival curves) were utilized to investigate the suitability of them to the estimation of pipe degradation and the forecasted "time to next failure (break)". The research aims the development of a framework of risk analysis models for the degradation of water distribution systems.

Key words: Water pipes, risk analysis, risk of failure**.**

1. INTRODUCTION

The problem of water losses in urban distribution networks has been one of the biggest infrastructure problems facing city and municipal authorities and a major obstacle towards a sustainable management of national and urban resources. Existing water distribution systems are increasingly at risk due to numerous factors (both internal and external to the distribution networks), yet important advances in the development of expert systems for utility system risk assessment have been confined to immediate, pipe-related characteristics (e.g. age, geometry, operating pressure, leakage history) (US Environmental Protection Agency, 2006) while new advances in remote monitoring have yet to be embraced by the water utility industry.

In the case of water mains, such studies range from investigations of failure mechanisms, to statistical analysis of historical data and to risk-analysis frameworks that can add to the state-of-knowledge of the urban utilities with respect to their distribution network and the underlying risk factors associated with its deterioration. The studies usually attempt to identify statistical relationships between water main break rates and influential risk factors such as a pipe's age (Date Of Birth, DOB), diameter (D), length of pipe (L) and material (M), the corrosivity of the soil (H2O), the operating pressure and temperature, possible external loads (including highway traffic (T)) and prior pipe breaks (Number Of Previous Break, NOPB).

2. PAST STUDIES

Most studies in literature show a relationship between failure rates and time of failure (age of pipes), and some of them suggest a methodology to optimize the replacement time of pipes. Shamir

and Howard (1979) reported an exponential relationship; Walski and Pellicia (1982) applied such exponential relationship to a data set stratified by material of pipe, and Clark (1982) developed a linear multivariate equation to characterize the time fiom pipe installation to the first break and a multivariate exponential equation to determine the breakage rate after the first break. A study by Andreou et al. (1987a,b) suggested a probabilistic approach consisting of a proportional hazards model to predict failure at an early age, and a Poisson-type model for the later stages, and further asserted that stratification of data (based on specific parameters) would increase the accuracy of the model. A non-homogeneous Poisson distribution model was later proposed by Goulter and Kazemi (1988) to predict the probability of subsequent breaks given that at least one break had already occurred. Finally, Kleiner et al. (1998), and Kleiner and Rajani (1999) developed a framework to assess future rehabilitation needs using limited and incomplete data on pipe conditions.

More recently, a simulation model was applied to an inventory of water mains in New York City to analyze replacement strategies, and Vanrenterghem-Raven, Eisenbeis, Juran, and Christodoulou (2004) developed models for the structural degradation of urban water distribution systems based on data from New York City. These studies usually attempt to identify statistical relationships between water main break rates and influential risk factors such as a pipe's age, diameter and material, the corrosiveness of the soil, the operating pressure and temperature, possible external loads (including highway traffic) and recorded history of pipe breaks. The knowledge gained by the New York City case-study was furthered and reported upon by (Christodoulou 2003, 2006, 2007) in a developed framework for integrated GIS-based management, risk assessment and prioritization of water leakage actions.

3. RISK ANALYSIS USING ARTIFICIAL NEURAL NETWORKS

Part of the current study is based on data collected from New York City's Department of Environmental Protection (NYCDEP) and covers an urban area in the district of Queens (New York City) that includes sub-areas with high urban stress (dense network of highways and subways, elevated and underground) as well as quiet residential zones. The pipe network in the modeled area has a total length of 227 miles (about 365 km) and consists of pipes of varying diameter (from 10 to 182 cm) the majority of which (51.4% of total length) having a diameter smaller than 20 cm and only a small percent (5.5%) having diameter over 90 cm (almost all of these pipes are made of steel). In terms of material, 52.6% of pipes are in cast iron, 15.8% in lined cast iron, 15.8% in ductile iron and 5% in steel (STL). It should also be noted that the majority of the pipes (52.6%) are more than 70 years old and only 17.2% were laid within the last 30 years.

The historical data spans a 20-year period (1982–2002) and about 500 pipe breaks in a population of about 6600 statistical individuals (breaks or not). The data included a number of presumed risk factors, for which information was collected either at the time of the break event (such as age and diameter), or as an aftermath (such as vicinity to highways, subway system, residential and industrial areas). The latter was necessitated by the fact that either the data analysis was done retroactively (going back in time) or the site conditions changed over time (for example the land use, or the traffic changed), thus rendering impossible data collection at a single point in time.

The analysis was performed by means of a self adjusted artificial neural network (ANN) utilizing the back probagation algorithm (Figure 1). Neural networks are particularly effective for pattern recognition in multivariate non-linear problems and sets with incomplete data. The output of the ANN pattern recognition algorithm resulted in ranking of risk factors that reinforces past studies, and helps focus the statistical and stochastic study of the problem based on a smaller number of risk factors.

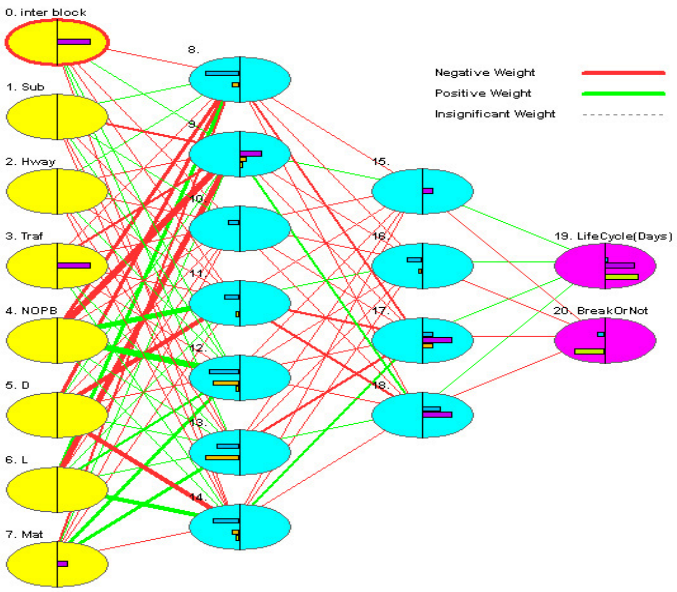


Figure 1: Artificial neural network architecture for risk analysis of water mains.

Fuzzy logic (FL) is derived from fuzzy set theory and deals with reasoning that is approximate rather than precisely deduced from classical predicate logic. Fuzzy truth (contrary to the absolute/crisp truth derived by classical predicate logic) represents membership in vaguely defined sets, not likelihood of some event or condition. The technique allows for set membership values to range (inclusively) between 0 and 1, and in its linguistic form allows for imprecise concepts such as "low", "low-medium", "medium" and "high".

Artificial neural networks (ANN) are similar to fuzzy logic in many respects but different in the way they handle knowledge. ANN are ideal for the representation of non-linear relationships among input and output parameters, and they are developed through a systematic training and knowledge acquisition based on past data sets (training sets). A generic neurofuzzy system usually consists of four levels of computation and analysis: an input level, a fuzzification level with membership functions, an inference level with rules that process the input data and the fuzzy membership functions, and a defuzzification level that transforms the computational results to readable outputs.

At the second level (the fuzzification layer) the input factors X_i are fuzzified according to predefined membership functions. Values between zero and one characterize fuzzy members which only partially belong to the fuzzy set of interest. The third layer (the inference layer) is where the knowledge-processing and knowledge-extraction occurs based on the application of the predefined fuzzy rules. The rules are, in essence, hypothetical scenarios of combinations of inputs and the assumed output (based on the experts' knowledge) and they attempt to capture the knowledge of the experts as they deal with the hypothetical input/output scenarios.

The third layer (the inference layer) is where the knowledge-processing and knowledge-extraction occurs based on the application of the predefined fuzzy rules. These are rules of the traditional "IF-THEN" form which are produced based on the collective experience and expertise of "experts" in the subject matter.

In the case of the study reported herein and the analysis of the presumed risk-of-failure factors a linguistic (or numeric) value is assigned for every input factor describing the factor's appraised condition at the time of evaluation. Then, depending on the values deduced by the artificial neural network and the expert's appraisal the neurofuzzy system forms the second part (THEN) of the inference rules. In this second part the outputs of the artificial neural network are evaluated, weighed and ranked in three different linguistic values, subject to the combination of presupposed conditions.

The ANN results (Figure 2) were then reinforced by both Poisson regression (Table 1) and multimodel regression analysis (Table 2) by which the "Time To Failure" output was fitted to linear models of one to six variables (all other factors were automatically pruned as insignificant by the regression algorithms).

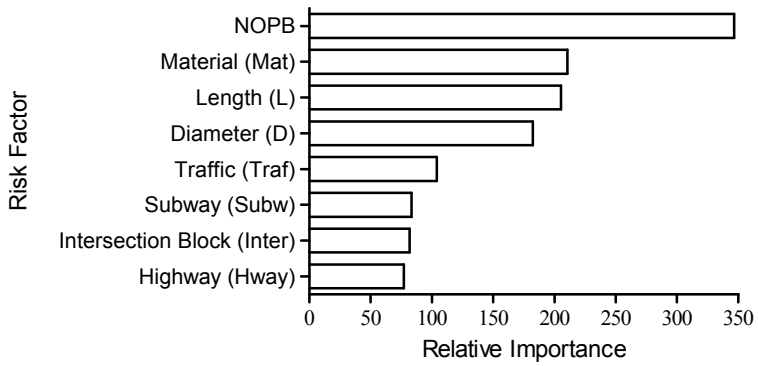


Figure 2: Artificial neural network architecture for risk analysis of water mains.

4. MULTIVARIATE RISK ANALYSIS USING NON-PARAMETRIC METHODS (KAPLAN-MEIER SURVIVAL CURVES)

In past case studies the problem of risk analysis of water distribution systems had been reduced to fitting failure times to parametric models (most notably Weibull probability distributions) and examining such models through stratification of data and examination of risk factors one at a time. Such models assume that the failure time of pipes can be approximated by parametric equations relating risk factors to observed outputs, expressed in mathematical forms. Should one assume that such relationships between risk factors and outputs are not expressed mathematically by means of known distributions, then the estimation of the underlying multivariate probability distribution function becomes of paramount importance to the evaluation of the expected probability of failure at different time intervals.

Table 1: Poisson regression analysis of risk factors

Model Size	R-Squared	Root MSE	Cp	Variables in Best Model	Legend
1	0.354236	1854.855	14.057252	E	A (Intersection Block)
2	0.723061	1330.625	5.743929	AE	B (Subway)
3	0.863724	1043.587	3.810636	ACE	C (NOPB)
4	0.954942	692.909	3.259952	ABCE	D (Diameter (in))
5	0.964232	756.104	5.000168	BCDEF	E (Length (ft))
6	0.964238	1069.202	7.000000	ABCDEF	F (Material Type)

Model size is the number of independent variables in the model. Model size will range from 1 to p.

R-Squared is the ratio of the variation explained by the model to the total variation in the dependent variable. R-Squared ranges from zero to one. The larger the R-Squared, the better the model.

This is the square root of the mean square error. The smaller this value is, the better the model.

Cp is Mallow's statistic. The optimum model will have a Cp value close to p+1, where p is the number of independent variables. A Cp greater than (p+1) indicates that the regression model is overspecified (contains too many variables and stands a chance of having collinearity problems). On the other hand, a model with a Cp less than (p+1) indicates that the regression model is underspecified (at least one important independent variable has been omitted). The formula for the Cp statistic is as follows, where k is the maximum number of independent variables available:

$$C_p = \left[\frac{MSE_p}{MSE_k}\right](n-p-1)-[n-2(p+1)]$$

Pipe failures can be thought of as a multivariate Bayesian classification problem, for which a probability distribution function is required for each class (risk factor). In practice, it is often difficult to determine the probability distribution function with high accuracy, since there may exist too few training vectors, and the data may be incomplete or partially inaccurate. In such cases where a probability distribution function is to be developed from sparse, real-world data sets, other means are required to approximate the function.

The most common non-parametric estimator of the survival function is called the Kaplan-Meier product limit estimator. This estimator is defined as follows, in the range of time values for which there are data.

$$\hat{S}(t) = \prod_{t_1 \le t} \left[1 - \frac{d_i}{Y_i} \right] \quad \text{if } t_1 \le t \quad \text{and } \hat{S}(t) = 1 \quad \text{if } t < t_1 \tag{1}$$

Table 2: Multi-model regression analysis of risk factors

Model Size	R-Squared	Root MSE	Cp	Variables in Best Model	Legend
1	0.354236	1854.855	14.057252	E	A (Intersection Block)
2	0.723061	1330.625	5.743929	AE	B (Subway)
3	0.863724	1043.587	3.810636	ACE	C (NOPB)
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6	0.964238	1069.202	7.000000	ABCDEF	F (Material Type)
Model size is the number of independent variables in the model. Model size will range from 1 to p.					
R-Squared is the ratio of the variation explained by the model to the total variation in the dependent variable. R-Squared ranges from zero to one. The larger the R-Squared, the better the model.					
This is the square root of the mean square error. The smaller this value is, the better the model.					
Cp is Mallow's statistic. The optimum model will have a Cp value close to p+1, where p is the number of independent variables. A Cp greater than (p+1) indicates that the regression model is overspecified (contains too many variables and stands a chance of having collinearity problems). On the other hand, a model with a Cp less than (p+1) indicates that the regression model is underspecified (at least one important independent variable has been omitted). The formula for the Cp statistic is as follows, where k is the maximum number of independent variables available: $C_p = \left[\frac{MSE_p}{MSE_k} \right](n-p-1) - [n-2(p+1)]$					

In the above equation, d_i represents the number of deaths at time t_i and Y_i represents the number of individuals who are at risk at time t_i. The product limit estimator may be used to estimate the cumulative hazard function $H(t)$ using the relationship

$$\hat{H}(t) = -\log\left[\hat{S}(t) \right] \tag{2}$$

The hazard rate can be estimated by using Epanechnikov's kernel smoothing. In its general form, approximation of the distribution function can be defined as follows: Given a series of data points for target Y and predictors $X_1, X_2, \ldots, X_n$, find a function F such that $Y = F(X_1, X_2, \ldots, X_n) + \varepsilon$ so that, based on the observed underlying joint probability distribution $F(X_1, X_2, \ldots, X_n)$ = Expected Value of Y given the values $X_1, X_2, \ldots, Xn$

Kernel smoothing constructs the function F by estimating the underlying empirical joint probability density of the Y and $X_1, X_2, \ldots, X_n$ and then estimates the most likely (i.e. the expected value) of Y. The joint probability density function is estimated by constructing a series of estimators based on known density functions (called kernels).

The technique is made more efficient by transforming the input data set into a smaller dimensional space, similar to a method called Sliced Inverse Regression. Gaussian kernels are useful for unbounded data sets, and Epanechnikov kernels are useful for bounded data sets (the Epanechnikov kernel may behave poorly for data that is outof- bounds). The results of the regression using Epanechnikov's smoothing kernel are shown in Figure 3**.**

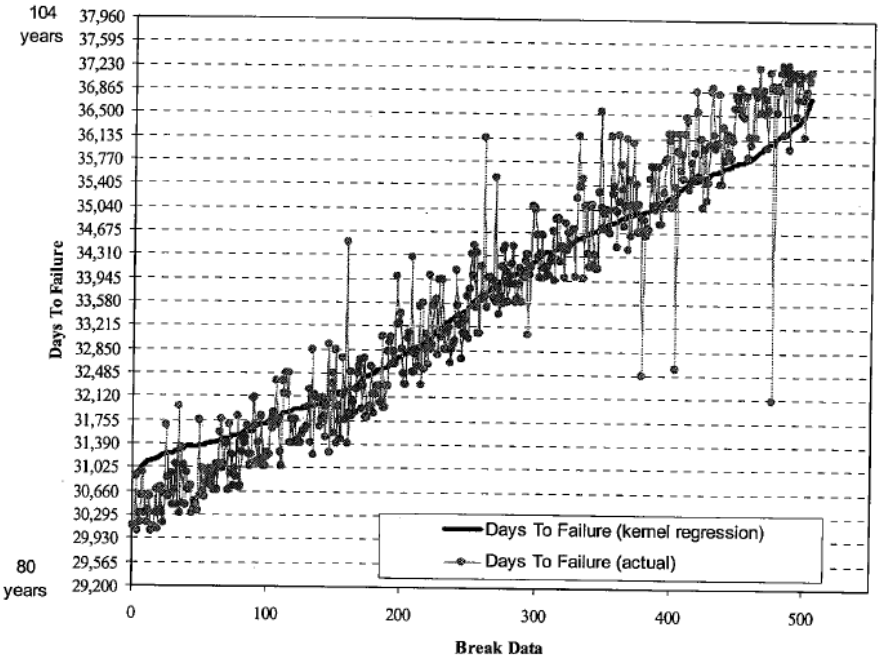


Figure 3: Multidimenional regression of the "Time-To-Failure" output using Epanechnikov's smoothing kernel.

5. CONCLUSIONS

In summary, the combination of classical survival analysis with neurofuzy systems seems to be a powerful tool in analyzing time-related data and identifying patterns in historical datasets. In the case of the New York City dataset, the ANN implementation provides insight into the risk factors for pipe failures, while the survival analysis and the Epanechnikov kernel provides the means for estimating time-to-failure events. The method is currently implemented in datasets from Cyprus with equally good results.

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REDUCTION OF APPARENT LOSSES USING UNMEASURED FLOW REDUCER (UFR) – CASE STUDIES

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Abstract: The demand for water is increasing due to population growth and technology development whereas water is becoming a scarce resource. There is an urgent need to find innovative solutions for water loss in water supply systems. Most water meters don't measure low flow rates (less than 12 liters per hour). The **Unmeasured Flow Reducer (UFR)** main objective is to reduce the apparent losses by changing the flow regime through the water meter at low flow rates. This reduces the volume of unmeasured water at low flow rates and enables the measurement of leaks by a domestic water meter. This paper shall review water meter accuracy curve error, reduction of apparent losses using the UFR - function and case studies conducted around the world.

Key words: Apparent losses, UFR, water meter accuracy curve error.

1. INTRODUCTION

Apparent losses (often referred to as non physical, paper losses or commercial losses) are in many cases the most expensive water losses that a water system will encounter. The water meter is the cash register of the water utility and therefore any losses occurring from the meter or the handling and processing of the data there after will be lost at sales revenue value per unit of measure (Thornton and Rizzo. 2002).

There are four major reasons for apparent losses: meter accuracy curve error, data transfer errors, data analysis errors and unauthorized consumption.

Water meter accuracy error is considered to be a significant component of the apparent losses in a water system. The error curve of the water meter (figure 1) can affect the unregistered water volume. Every meter type has its own rate of decay of the accuracy with time. The meter may be influenced by external parameters such as the quality of water (scale, sand etc.) and suspended particles.

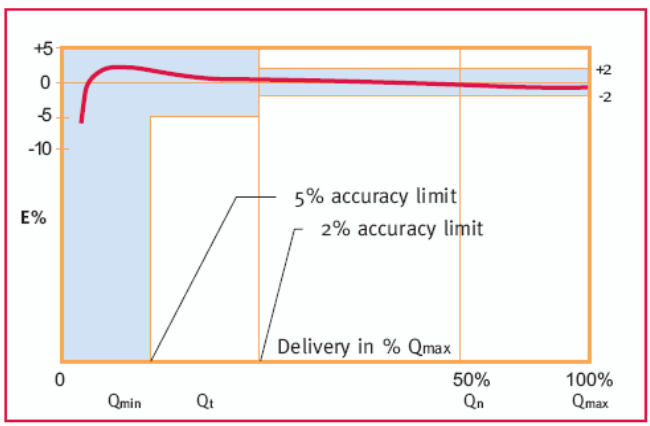


Figure 1. Error curve of a water meter.

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ISO standard (ISO 4064-1, Second edition, 1993) defines the performance of any type of water meter by looking at 4 main points along a flow range: Q_{min}, Q_t, Q_n, Q_{max}. It doesn't mention anything regarding the starting flow (usually one third of Q_{min}), which the water meter starts to measure. Q_{min} is the minimum accurate flow, accuracy of ±5%. Q_t is the transitional flow, at which the meter will be stable in performance, accuracy of ±2%. Q_n is the meter's nominal flow. Q_{max} is the maximum flow; it is determined as being twice the nominal flow. Figure 1 shows the ideal scenario for a brand new meter (Multi-jet class B), older meters will have an accuracy curve that is shifted outwards from the ideal ±2% accuracy band. The variability at low flows is much greater than at medium and high flows, the error curve is steep until reaching the minimum flowrate. Consequently the uncertainty about the real performance of meters at low flows will always be greater than at higher flows (Arregui el. 2006).

2. DOMESTIC WATER METERS

Most domestic water meters are usually of the Velocity type: single jet, multi-jet; Positive-displacement: oscillating piston and nutating disc.

Velocity meters are affected by flow distortions or dimensional changes in the meter that may interfere with the internal velocity of the flow passing through the instrument. Velocity meters are also sensitive to any increment in the drag torque on the sensor element especially at low flows. An incorrect mounting position of the water meter increases the friction of the moving parts. The effect is only noticeable at low flows for which this term is relevant in the effective drag torque. If the mounting position is not correct, according to the manufacturer recommendations, it may lead to a higher degradation rate of the meter. The wear of the moving parts increases the error at low flows (figure 2). Depositions may cause over registration, at medium-high flows and under registration at low flows. However, on the long term, depositions grow so large that they can prevent the impeller from rotating, temporarily or permanently, causing a severe under registration of the meter (Arregui el. 2005).

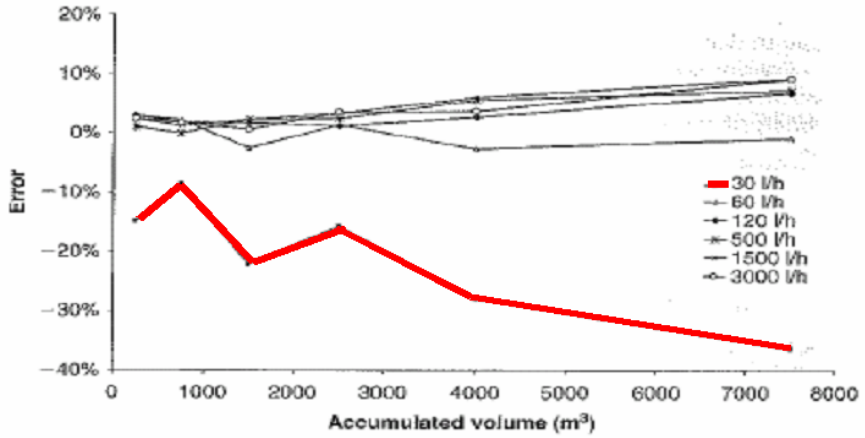


Figure 2. Multi-jet error (nominal flow 1.5 m^3/hr) on accumulated volume (Arregui el. 2006, p.21).

Positive-displacement meters are mainly used for domestic uses due to excellent sensitivity to low flow rates and high accuracy across a wide range of flow rates. A known volume of liquid in an enclosed compartment moves with the flow of water, and the flow rate is calculated based on the number of times the compartment is filled and emptied. A reduction in the actual volume of each compartment is impossible, for this the meter will stop. Therefore, over-registration will not occur and the error curve will become more negative (under-registration) (figure 2). Consequently, these meters tend to under-register actual water usage when they become older and excessively worn.

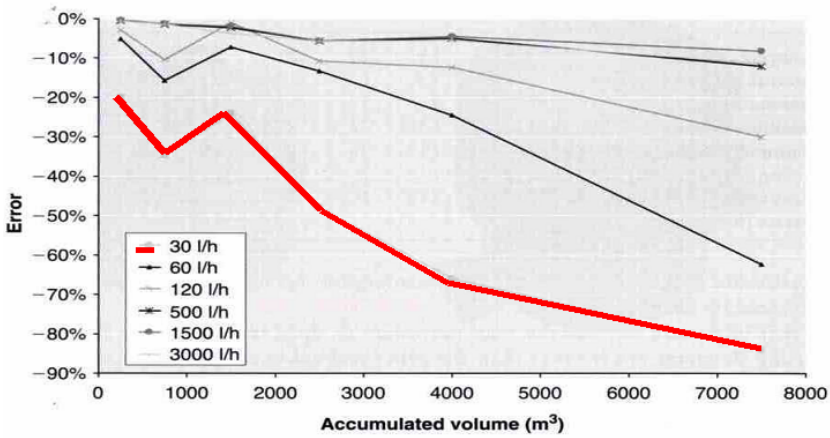


Figure 3. Evolution of the metering error obtained from a sample of volumetric meters (1/2") (Arregui el. 2006, p.28).

3. ROOF TANKS

Water consumption at low flows is the most difficult to measure. As it can be seen in any accuracy curve of any water meter, measuring errors are much larger in this range. Under these working conditions the energy transfer from the fluid to the sensing element is very small and any increase in friction, caused by any factor, may stop the impeller or the piston at lower flowrates. For that reason, the under-registration expected from a meter installed in a user that consumes a great amount of water at low flows is very high. Furthermore, considering that the accuracy of a water meter decays at a faster rate at low flows, this under-registration will increase more rapidly than in other type of users (Arregui el. 2006).

Figure 4 indicates the effect of a roof tank ballvalve on the flow profile of a normal domestic household. An indirect plumbing system usually consists of the household kitchen fed straight off the inlet mains pressure whilst the rest of the household is fed via gravity from a roof tank. It is the slow closure of the roof tank ballvalve that induces flows that are lower than the starting flow of the water meter, as shown in Figure 4 above. The larger the surface area of the roof tank or the higher the starting flow Q_s of the meter, the larger will be the resulting meter under-registration (Rizzo and S. John, 2006).

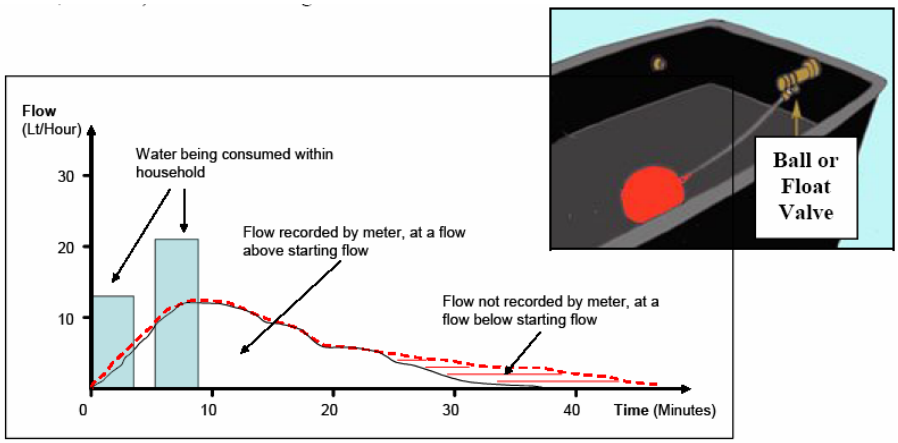


Figure 4. The effect of a roof tank on meter under-registration (Rizzo and S. John, 2006).

Figure 5 indicates the result of repeated tests carried out on standard $1m^3$ roof tanks. Data logging was carried out on water meters positioned upstream and downstream of the roof tank. The logic to these tests was that, whilst the water meter installed upstream to the tank would substantially under-register due to the ballvalve effect, the water meter downstream of the tank would work fairly accurately. This would be because most of the outflows from the tank would be above Q_s or even above Q_{min}. Results confirmed that the downstream readings were over 6% higher

than the upstream readings, indicating that the upstream meter was under-registering by over 6% of daily consumptions due to the ballvalve effect. Replacing the new upstream meter with aged meters (having greater wear) produced under-registration figures ranging from the initial 6% to up to 95%. All tests were carried out using Class D (Q_n = 1.0m^3/Hr) volumetric water meters (Rizzo and S. John, 2006).

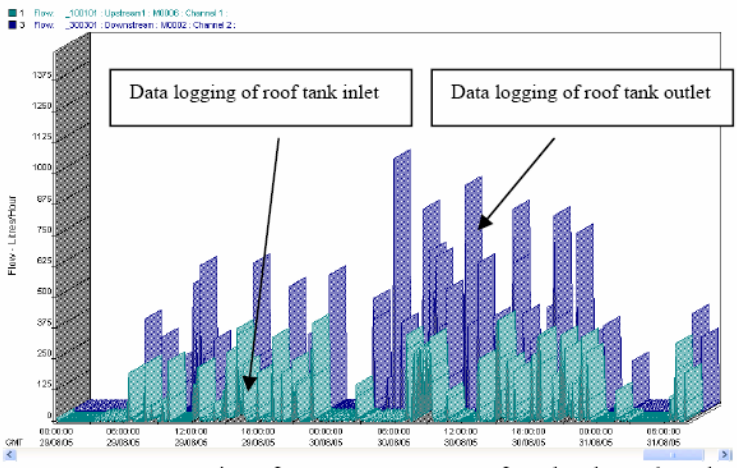


Figure 5. Data logging of water meters at roof tank inlet and outlet (Rizzo and S. John, 2006).

4. UFR - UNMEASURED-FLOW REDUCER

The UFR (figure 6) works by changing the way that the water flows through the water meter at low flow rates. At low flow rates there is not enough energy in the flow to activate the water meter register. The UFR begins to operate at very low flow rates and creates batches of flow that the water meter can measure (figure 7). Due to the change in the mode of water flow to batches, the UFR enables the water meter to measure low flow rates.

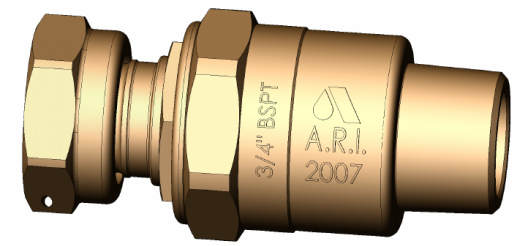


Figure 6. UFR - Unmeasured-Flow Reducer.

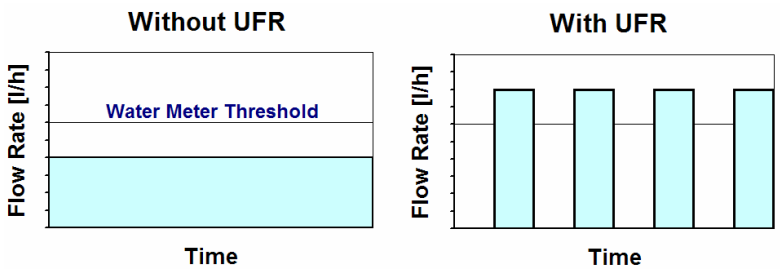


Figure 7. UFR – Principle of Operation.

The UFR regulates the water flow so that there is no water flow at all part of the time, while the rest of the time, the flow is high enough to be measured. Changes in the flow discipline at low rates allow the existing water meter to measure at all water flow rates passing through it. When the flow rate increases over the water meter's measuring threshold, the UFR remains permanently open, so

that it does not interfere with measurements. The UFR also acts as a non-return valve and prevents backflow (it closes when the downstream pressure and upstream pressure are equal).

The UFR is a differential non-return valve, designed in such a way that the pressure difference required to open it is more than that required to keep it open. The pressure difference to open the UFR is 0.4 bar, whilst the pressure difference to keep it open is 0.1 bar. When a leak develops (a flow rate below the measurement threshold of the water meter) the downstream pressure drops. When the downstream pressure drops below 0.4 bar of that of the upstream pressure, the UFR opens and allows for a flow rate above that of the measurement threshold (figure 8). The free flow of water through the UFR equalizes the pressure across the UFR and allows it to close (figure 9). The continuing leak downstream to the UFR will make this operation repeat itself over and over again. Every time the UFR opens, a quantity of water passes through the water meter at a flow rate above the measurement threshold of the water meter and so the flow is measured.

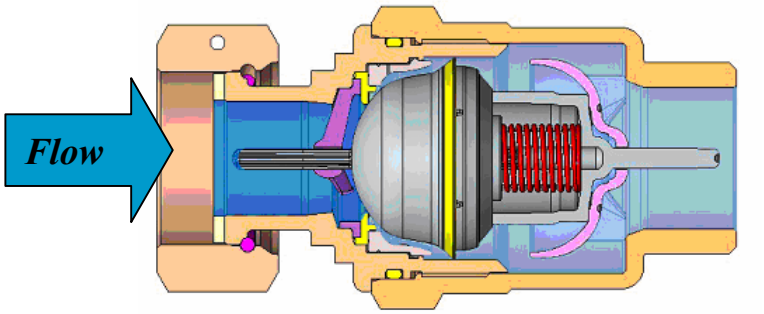


Figure 8. UFR closed: downstream pressure decreases because of leakage.

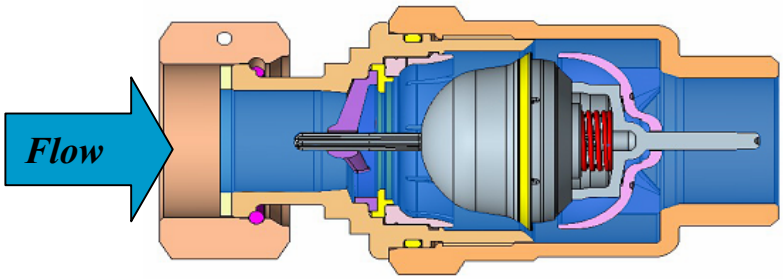


Figure 9. UFR opens: downstream pressure equals that of upstream.

5. INTERNATIONAL CASE STUDIES

The main purpose of the case studies was to answer the following questions:

1. Can a water meter measure all flow rates?
2. Can the UFR reduce the unmeasured flow?
3. Is the contribution of the UFR to the flow registration of the water meter significant?

A statistical test is conducted to find out if there is unmeasured flow passing through the water meter at low flow rates. The test procedure is as follows:

1. Verify that the water meter leak detector is stationary.
2. Close the shut off valve, before or after the water meter. (If the valve is old or faulty it might not seal, in that case this procedure is not reliable enough to determine if there is unmeasured flow or not).
3. Wait for about 60 seconds (during this time water would have drained from the household pipe work, or as a leakage or as a low flow rate flow, for instance through a float valve into a water tank).
4. Open the shut off valve while watching the leak detector in the water meter carefully. If there is a leakage in the household, the volume, equal to that of the drained water, will flow

with enough energy to activate the water meter, and it will be seen on the leak detector. If there is no leakage, the leak detector will remain stationary.

6. PILOT FIELD TEST METHODOLOGY

In order to determine the contribution of the UFR the following steps are taken:

1. Selection of a DMA (district metered area) consisting of reasonable number of meter connections with a single feeding source with a main master meter.
2. Establishing the current water loss without UFR by comparing the reading of the main meter with the sum of the domestic meters. The readings should be taken at long enough intervals and there should be no significant differences in the time the readings are taken in order to reduce differences resulting from additional consumption during a period of time readings were taken. AMR system that enables collecting the readings from all meters at the same time is much more preferable.
3. Installation of UFR - and re-measuring the difference between the readings of the main meter and the sum of the domestic meters.

6.1 Case Study 1 – Israel, Jerusalem, Ein Karem

In March 2005, 120 UFRs and 360 UFRs were installed in two separate DMAs in Ein Karem, Jerusalem. The water meters in these DMAs are multi jet, Qn 2.5 class B (figure 10).



Figure 10. UFR installed, Jerusalem-Ein Karem.

The under registration percentage was recorded prior to and after the installation of the UFRs and is a comparison of the sum of the domestic water meter readings to that of the main water meter of the DMA. Table 1 summarizes the results of this study. The average contribution of the UFR was 8.5%.

Table 1. Study results with and without UFR

Location	No. of Consumers	With or Without UFR	Period of Time [months]	Under-Registration [percent]
Ein Karem First DMA	120	Without UFR	8	16%
		With UFR	6	6.1%
		Contribution of UFR		9.9%
Ein Karem Second DMA	360	Without UFR	8	26%
		With UFR	6	18.8%
		Contribution of UFR		7.2%
		Average Contribution of UFR		**8.50%**

6.2 Case Study 2 – Cyprus, Larnaka

This case study was conducted in Larnaka, Cyprus from October 2006 to December 2006. The DMA contains a variety of households: private houses, buildings and houses with roof tanks. The water meters in this DMA are as shown in figure 11.

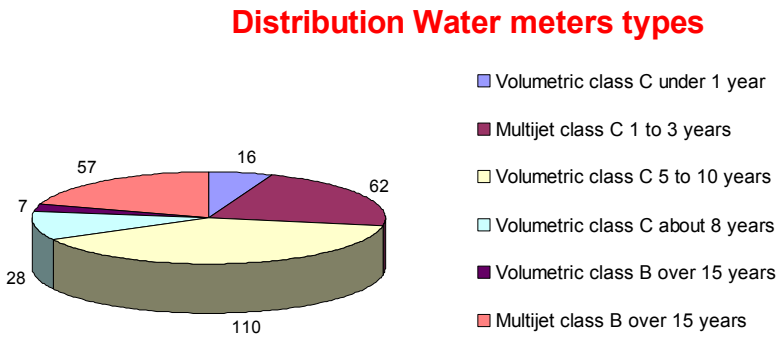


Figure 11. Distribution of Water meters types in the DMA.

Weekly readings were taken before and after the installation of the UFRs. According to table 2 the contribution of the UFR was 9.93%.

Table 2. Larnaka results with and without UFR

	With UFR's	Without UFR's	DIFFERENCE
Area 1 consumer meters Total (Consumption)	3242.84	3066.48	
Area 1 bulk meter total (Demand)	3556.00	3667.00	
Not registered water in M^3	313.16	600.52	287.36
Not registered water as a percentage of the total **Demand**	8.81%	16.38%	7.57%
Not registered water as a percentage of the total **Consumption**	9.66%	19.58%	9.93%

6.3 Case Study 3 – Malta

The national water utility in Malta identified a small zone for pilot study purposes. The zone was chosen in accordance with the ages of the water meters in the zone, allowing for a normal distribution of meter ages with an average of five years in age. Poezija zone has 26 domestic consumers with volumetric class D (Qn=1 m3/hr) water meters and roof tanks on the buildings (figure 12). The UFR units increased the metered volume of water by a substantial 5.5% to 6% of the water supplied to the zone (Table 3).



Figure 12. The pilot zone (left) where the UFR (right) was installed in Malta.

Table 3. Effect of UFR's on Water Meter Under-Registration

Test No.	Global % Under-registration Vs. Master Meter		% Overall improvement
	Without UFR's	With UFR's	
1	18.1	12.1	6
2	26.7	21.2	5.5
3	28	22.2	5.8

7. CONCLUSIONS

Leakages and other unmeasured water flows at low flow rates were found in many of the households tested. The UFR succeeded in the reduction of unmeasured flow and was found very effective in reducing apparent losses. The UFR had a significant effect on the flow registration of the water meter. The UFR increased the water meter registration by 5% to 10%.

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TOWARD EFFICIENT, EQUITABLE AND SUSTAINABLE MUNICIPAL WATER SUPPLIES FOR DOMESTIC PURPOSES IN THE WEST BANK: A CONTINGENT VALUATION ANALYSIS

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Abstract: This paper is an empirical work dealing with municipal water services in the West Bank where the emphasis is put on the analysis of water management. Toward that end, the study used Contingent Valuation Method (CVM) to estimate the Total Economic Value (TEV) of domestic water uses. The application of this approach consists of the construction a hypothetical market. The dichotomous choice with follow-up format question was utilized in developing the CV questionnaire; this elicitation technique was used to model Willingness to Pay (WTP) utilizing Tobit econometric model, binary-Logit econometric model and Ordinary Least Squares (OLS). The estimated coefficients of the econometric models were utilized to determine the relationship between WTP and socioeconomic variables, Log-likelihood and adjusted coefficient of determination (adjusted R^2). Furthermore, the marginal impacts of the coefficients of Tobit model were used to measure the elasticities of the coefficients of the explanatory variables. The CVM was applied to a stratified random sample of 520 households in the Ramallah and Al-Bireh governorate including urban and rural areas, and refugee camps. The face to face (in-person) interview survey was adopted. To assess the performance of water pricing policy, a questionnaire survey was directed to the personnel staff of water institutions. This method was administrated to a stratified random sample of 117 employees in the water institutions. Descriptive statistics enhanced by non-parametric tests were used. The main findings of this study were measured the WTP with key socioeconomic variables suggested by economic theory and CV studies including income, age, gender, location, educational level, employment status, family size, water consumption, and use of water filters. Also, the pricing policy of municipal water services was examined.

Key words: Contingent valuation method, total economic value, water management, West Bank, willingness to pay.

1. INTRODUCTION

Over the past four decades the Palestinian territories of the West Bank and Gaza Strip (WBGS) have suffered from water scarcity (Elmusa 1993; PWA 2005). The crisis of water scarcity is only one in the WBGS, poor quality of water supply and the absence of proper assessment make a profound impact on the Palestinian economy (Bellisari 1994). Supply of and demand for waters in the West Bank (WB) are in an increasing deficit while the consumption of waters is growing, and the amount of available water could not bridge the gap of shortage (PWA 2005).

The overall objective of the study is to undertake an empirical work dealing with the municipal water for domestic purposes in the WB. The study will examine water management. This study, however, is an important and necessary because:

Pricing water resources based on the marginal value of the product will lead to efficient allocation of water (Henry and Bowen 1982). The mean Willingness to Pay (WTP) is appropriate when the efficiency criterion of welfare economics is adopted (Mendonca and Tilton 2000). The study, therefore, undertake a Contingent Valuation Method (CVM) to evaluate the economic efficiency of domestic water uses as an essential element to formulate efficient allocation mechanisms to meet the households' needs of both, sufficient water quantity and acceptable water quality. Additionally, the CVM is regarded as the main method for the valuation of environmental goods including use and non-use values (Mitchell and Carson 1989).

In Palestine water is scarce and vulnerable. Also, water has an economic value only in case its supply is scarce relative to its demand, since if water is unlimited, it will be a free good (Ward and Michelsen 2002). The concept of water as an economic good is simple, like any other good and it has a value to users who are willing to pay for it hence the consumers will use water as long as the benefits due to use of an additional cubic meter exceed the costs so incurred (Briscoe 1996).

Several methods for pricing and allocating water have arisen because marginal cost pricing is hard to implement in practice (Johansson et al. 2002). The study however seeks to formulate a proper water pricing policy based on the WTP principle as background information to be applied in practice.

2. CONCEPTUAL FRAMEWORK

2.1 The Concept of Total Economic Value (TEV)

The concept behind the TEV is that any good or service is constituted of different attributes, some of which are concrete and easily measured, while others may be more difficult to quantify. However, the TEV is broken-down into two main values, those are: use values and non-use values presented below (Marksoo 2007; Rogers et al. 2002).

(a) Use value: this value can be subdivided into direct use value or indirect use value. Direct use value is consumptive, extractive, or structural use value, derives from goods which can extracted, consumed, or directly enjoyed, for instance, direct uses of water include drinking. The indirect use values occur from the natural functioning of ecosystems, for example, an indirect use of water receives is characterized by its fewer benefits those are not traded in any market and are sometimes referred to as un-priced benefits to the water users.

(b) Non-use value: this value could be not easy to describe or to evaluate; it is derived without direct use of a resource. Non-use value can be named as intrinsic value or passive value. The non-use value, however, can be subdivided into existence, bequest, and option value. Existence value measures the WTP for a resource for some moral, altruistic, or other reason which is not related to current or future use. Bequest value is the value a habitant places on the ability to conserve a resource so that it can be used by future generations. The third part of nonuse value is option value; the concept of option value refers to the value placed on a resource's future use.

2.2 Water Management

The Dublin principles were an attempt to concisely state the main issues and thrust of water management, especially, in the developing countries. The four Dublin principles presented below have been universally adopted as main principles for integrated water resources management (IWRM) (Rogers et al. 2002; Marksoo 2007).

- Fresh water is a finite, vulnerable, essential resource to sustain life, development and the environment.
- Water resources development and management should be based on a participatory approach, involving all relevant stakeholders at all levels such as users, planners, and policy makers.
- Women play a central part in the provision, management and water safeguarding.
- Water has an economic value in all of its competing uses.

2.3 The Contingent Valuation Method (CVM)

The CV technique has been used for about twenty years to estimate passive use values. For over the last five years there has been a dramatic increase in the number of academic publications and presentations related to this technique (Mitchell and Carson 1989). Also, CVM is the most common method for valuing non-market amenities (Arrow et al. 1993).

The Contingent Valuation Method is an example of a hypothetical direct valuation technique non-market valuation requiring the active involvement of respondents. Due to a hypothetical policy change that will cause some environmental impacts, the benefits and costs are evaluated through questionnaire surveys that detect the WTP of consumers to accept or avoid, the policy change in question. The questions maybe open-ended or close-ended, with take frames such as dichotomous choice (leave-it-or-take-it). These values are summed for users and non-users alike, and the net benefits to society estimated. It will be shown subsequently that careful use of the CVM can elicit either use or non-use values for an amenity. Econometric analysis of survey results is generally utilized to derive mean values of WTP bids and to estimate the determinants of respondents' familiar with the asset being valued (Lipton et al.; 1998; O'Doherty 2001; Haab and McConnell 2003).

3. METHODOLOGY OF RESEARCH

3.1 Analysis Techniques and Econometric Models

Descriptive statistics such as frequencies, mean, standard deviation, cross-tabulation, correlation, and one-sample t-test, enhanced by non-parametric tests such as Mann–Whitney U test and Kruskal-Wallis K test were used (the statistical package SPSS was run to test these statistics).

The willingness to pay question format utilized in the CV questionnaire offers the possibility to use a variety of estimating techniques. The discrete choice with follow-up question used to model WTP by using Tobit econometric model, binary-Logit econometric model, and ordinary least squares (OLS) can be utilized (Cameron and Trivedi 2005; Wooldridge 2001). However, Tobit is recommended to use for several reasons presented beneath (Tobin 1958; Cameron and Trivedi 2005).

$$WTP = \begin{cases} WTP^{Tobit} & if\ WTP^{Tobit} > 0 \\ 0 & if\ WTP^{Tobit} \le 0 \end{cases}$$

where, WTP^{Tobit} is unobserved continuous dependent variable, or the unobserved true WTP. Therefore, the Tobit model is an appropriate to avoid endogeneity bias; the empirical WTP model is a simultaneous equations instrumental variables model. The model of WTP is a Tobit regression and the quality or quantity model is an OLS's regression:

$$WTP_i = \alpha' X_{1i} + \beta \hat{q}_1 + \varepsilon_{1i}$$
$$q_i = \gamma' X_{2i} + \varepsilon_{2i}$$
$$\rho = correlation[\varepsilon_{1i}, \varepsilon_{2i}]$$

where, $\hat{q}_1$ is the estimated variable from the used model for water quantity or water quality. The estimation method is full information maximum likelihood allowing for correlation in the normally distributed disturbance terms ρ, the test for the exogeniety of $\hat{q}_1$ is a t-test for $\rho = 0$

The variables in the X_{2i} vector but not in the X_{1i} vector are the identifying variables. These variables should have high explanatory power in the instrumenting equation and low correlation with WTP and the disturbance term. Coefficient values estimated using Tobit technique may differ substantially in both, magnitude and sign, from those estimated by using ordinary least squares. The marginal effect of an autonomous variable, say m, on $E(WTP)$ is $\frac{dE(WTP)}{dm} = b\Phi(Z)$, where Z is evaluated at the mean of all variables including quality or quantity in accord with the desired dependent variable. Since, $0 < \Phi(Z) < 1$, the marginal impact will always be smaller in absolute value than the coefficient estimate (E-Views software econometric package was utilized).

3.2 CV Design and Survey Administration

In developing countries the use of CVM to measure the WTP for social projects is well accepted and widely utilized in many different circumstances, but some biases may exist when using this technique. Those biases are due to the hypothetical nature of this method. Careful survey design is necessary to avoid such problems (Whittington 1998; Whittington 2002).

This empirical study was undertaken a CV technique to measure the TEV of domestic water services including both, use values and non-use values.

The scenario used for face-to-face (in-person) interviews in this study administrated to value the water quantity and water quality in Ramallah governorate. The dichotomous choice is followed by an open-ended follow-up question, for instance, "what is the maximum that you are willing to pay …?" was used in the survey questionnaire of CV (Freeman 1993; Arrow et al. 1993; Mitchell and Carson 1989; Haab and McConnell 2003).

A stratified random sample was utilized to ensure a representative sample of households, chosen from the governorate of Ramallah for dividing the households into three main sub-groups. Three divisions (strata) and a representative sample for each stratum were randomly selected: (a) household heads who live in the cities of Ramallah and Al-Bireh; (b) household heads of rural areas; and (c) household heads who live in the refugee camps. After dividing the study population into the appropriate strata, a simple random sample was taken across each stratum.

The usable sample is composed of 520 household's heads, 1.5% of the households census for each stratum was taken to befit the size of the population, since the sample was administrated to the household's heads in the cities, villages, and refugee camps.

3.3 Administration of Water Institutions Survey

The study was also undertaken a second survey which was directed to the employees of water institutions for evaluating the pricing policy adopted from the water institutions. A stratified random sample is to ensure a representative sample of all management personnel, was chosen in the local councils and Jerusalem Water Undertaking (JWU). An important element of the features that we might want our sample is to show a proportional representation of different positions in which employees work. Specifically, the target sample was divided into three levels (strata): (1) senior management; (2) management department; and (3) Finance department. Since, the target sample was directed to the JWU, the municipalities of Ramallah and Al-Bireh, and fourteen Local councils in Ramallah governorate, which are the large village local councils. Due to dividing the personnel into three appropriate strata, a simple random sample was taken through each stratum.

A stratified sample of 117 employees in the water institutions was selected. The usable sample was targeted employees who have practical experience with municipal water, 70% of personnel were taken for senior managements, 70% of personnel were taken for management departments, and 70% of personnel for finance departments.

4. EMPIRICAL ANALYSIS AND FINDINGS

The CV survey was used for in-person interviews. The survey was interviewed the household heads (100% of the CV sample). The study used CVM to measure the use and non-use values of domestic water services. As shown in table 1 and table 2, the tobit model used the open-ended WTP variable as a dependent variable, which included positive and true zero responses and estimated a mean WTP (Jones et al. 2008). The logit model used the binary WTP variable (0, 1) as a dependent variable (Ojeda et al. 2008).

As table 1 indicates, the mean WTP value of the tobit model of 518 respondents in the CV survey was obtained as New Israeli Shekel (NIS) 31.4. This result implies that the respondents interviewed are voluntary willing, on average, to pay NIS 31.4 to have sufficient and reliable water supply for the households (use values). The log-likelihood is -1895.051 according to tobit model and is -319.707 according to logit model. Both gave a negative answer, as a desired value for maximum likelihood estimation to seek the probability distribution that makes the observed data most likely (Myung 2003). According to the linear model the adjusted coefficient of determination (adjusted R^2) is 0.11, which is acceptable result for CVM studies (Imandoust and Gadam 2007).

Table 2. Bid curve analysis: Estimated WTP model – The use values of municipal water

	Expected sign of coefficient	Logit-Estimated coefficient	Logit-Z-value (P-value)	Tobit-Estimated marginal effect	Tobit-Z-value (P-value)
Constant		1.478	2.617 (0.008)	21.871	1.654 (0.098)
Age	+/-	0.094	0.842 (0.399)	4.274	1.579 (0.114)
Gender	+/-	0.124	0.556 (0.577)	4.668	0.858 (0.390)
Location - Urban	+	0.047	0.201 (0.840)	4.206	0.744 (0.456)
Location - Rural	-	-0.253	-1.167 (0.243)	-10.928	-2.118 (0.034**)
Time-period	+	-0.011	-1.506 (0.131)	-0.293	-1.553 (0.120)
Water consumption	-	-0.009	-2.037 (0.041**)	-0.258	-2.145 (0.031**)
Income	+	0.0001	2.273 (0.023**)	0.006	4.471 (0.000*)
Education	+	0.047	0.460 (0.645)	-0.307	-0.120 (0.904)
Employment Status	+	-0.785	-2.077 (0.037**)	-13.008	-1.559 (0.118)
Gainfully employed	+	-0.277	-1.872 (0.061***)	-4.810	-1.326 (0.184)
Household size	-	-0.049	-1.140 (0.253)	-1.311	-1.178 (0.238)
The use of water filters	+	0.924	4.216 (0.000*)	19.233	3.881 (0.000*)
Log-likelihood = -1895.051 (Tobit model)					
Observations = 518					
Mean WTP = NIS 31.4					
Adjusted R^2 = 0.11, R^2 = 0.13					

*Significant at the 0.01 level, **Significant at the 0.05 level, ***Significant at the 0.10 level of confidence.

The results revealed that the variable water consumption, the use of water filters and income have significant impact on WTP, since income and the use of water filter have a positive impact, while consumption has a negative effect on WTP (the more I consume the less I am willing to pay), which seems to be consistent with the findings in other studies and the economic theory. The rural areas have a negative and significant impact on WTP; this implies that the rural respondents are unlikely to be willing to pay (WTP) for improved municipal water supply (Piper 1998; Farolfi et al. 2007; Zhongmin 2003). But the variable employment status and gainfully employed "the family members who are gainfully employed" have significant and negative impact on WTP, which is to be contradictory to the findings in other studies (Mendonca and Tilton 2000; Jones et al. 2008). The variable urban location, time-period "how long the respondent has lived in the region?", educational level, and household size have insignificant impact on WTP, which is to be inconsistent with the findings in other studies that have a significant impact on WTP (Kaoru 1993; Mendonca and Tilton 2000; Mbata 2006; Ojeda et al. 2008; Zhongmin 2003). The variable age and gender also have insignificant impact on WTP.

On the other hand, tobit model shows the marginal effect "elasticity" of income is 0.006 and less than 1, as a result, the respondent's WTP change for income change is likely to be very low and

limited. Also, according to Kruskal-Wallis K test the mean rank reveals respondents who live in the refugee camps are likely to be WTP more than rural and urban areas, although urban and rural respondents have a level of income, which is higher than the income level of the refugee camps' respondents. This result was mainly because of: the networks and reliability of water supply in the refugee camps are bad compared with urban and rural areas.

The data also showed that the respondents encounter a crisis of water in terms of both, quality and quantity. As the majority of respondents revealed that the government should proceed to solve the problem of both, water quantity and quality in parallel.

As table 2 indicates, the mean WTP value of the tobit model of 518 respondents in the CV survey was obtained as NIS 20.8. This result implies that the respondents interviewed are voluntary willing, on average, to pay NIS 20.8 to insure water for future generations (non-use values). The log-likelihood is negative, as a desired value for maximum likelihood estimation (Myung 2003). The R^2 is 0.092, which is accepted result for CVM (Imandoust and Gadam 2007).

Table 2. Bid curve analysis: Estimated WTP model – The non-use values of municipal water

	Expected sign of coefficient	Logit-Estimated coefficient	Logit-Z-value (P-value)	Tobit-Estimated marginal effect	Tobit-Z-value (P-value)
Constant		0.248	0.459 (0.645)	-8.434	-0.718 (0.472)
Age	+/-	0.040	0.377 (0.705)	2.683	1.124 (0.260)
Gender	+/-	0.136	0.621 (0.534)	2.604	0.544 (0.586)
Location - Urban	+	0.469	2.01 (0.044**)	8.852	1.828 (0.067***)
Location - Rural	-	-0.458	-2.157 (0.03**)	-2.057	-2.057 (0.039**)
Time-period	+	0.004	-0.596 (0.550)	0.060	0.364 (0.715)
Water consumption	-	-0.0148	-2.915 (0.003*)	-0.265	-2.489 (0.012**)
Income	+	0.000046	0.938 (0.347)	0.002	2.265 (0.023**)
Education	+	0.162	1.607 (0.107)	3.164	1.453 (0.146)
Employment Status	+	-0.201	-0.567 (0.570)	-2.480	-0.320 (0.748)
Gainfully employed	+	-0.157	-1.092 (0.274)	-1.233	-0.387 (0.698)
Household size	-	-0.045	-1.044 (0.296)	-0.863	-0.900 (0.367)
The use of water filters	+	0.823	3.920 (0.000*)	15.739	3.698 (0.000*)
Log-likelihood = -1641.168 (Tobit model)					
Observations = 518					
Mean WTP = NIS 20.8					
Adjusted R^2 = 0.072, R^2 = 0.092					

*Significant at the 0.01 level, **Significant at the 0.05 level, ***Significant at the 0.10 level of confidence.

The outcomes also revealed that the variable water consumption, the use of water filter and income have significant impact on WTP, since income and the use of water filter have positive impact while consumption has a negative effect on WTP. The variable location showed that the rural respondents have a negative and significant impact, while urban respondents have significant but positive impact on WTP; this implies that the urban respondents are likely to be WTP for insured water for next generations, but the rural respondents are unlikely to be WTP. These results are consistent with the findings in other studies (Piper 1998; Zhongmin 2003; Farolfi et al. 2007). The variable educational level, time period "how long the respondent has lived in the region?", employment status, gainfully employed "the family members who are gainfully employed" and household size have insignificant impact on WTP , which seems to be inconsistent with the findings in other studies (Mendonca and Tilton 2000; Gopalakrishnan 2003; Mbata 2006; Ojeda et al. 2008). The variable age and gender also have insignificant impact on WTP.

The study also revealed that the majority of respondents are unlikely to deal with water as an economic good that has an economic value. However, the study findings showed that there is a contradiction regarding this issue. According to one sample t-test, although the most of the respondents revealed that the total revenue of water selling should recover the total costs and encouraged imposing financial penalties or fines on households who have higher level of water consumption compared with other households, there are more than a half of the respondents prefer to get on water in a low price, which is insufficient to cover the average costs of water.

On the other hand, in the course of the study a second survey was undertaken. This survey consisted of interviews of the employees of water institutions (100% of the sample). It was used to evaluate the pricing policy adopted in the water institutions. However, according to one sample t-test, the adopted pricing policy in the water institutions is designed to achieve cost recovery, in other words, to recover at-least the minimum level of average costs per unit of water sold. In contrast, the water institutions don't take the negative externalities resulted from water uses such as infections or water degradation into consideration through the pricing policy. Also, the water institutions, especially, the JWU adopt a graduate pricing system in harmony with all consumer social categories to facilitate payment of water bills (water prices increases proportionally with increase of consumed water quantities).

5. CONCLUSIONS

This study is one of the few that has utilized CVM for improved water quality and secured sufficient water supply (use values), since a CVM survey was also used to obtain estimates of WTP for sufficient water for future generations (non-use values) in the WB.

Households are voluntarily WTP, but only NIS 31.4 monthly an insignificant amount in increased water costs for reliable water supplies that based on the coefficients of the explanatory variables explained below, since the mean WTP was NIS 31.4 per month of each household. Additionally, they are voluntarily WTP, but only insignificant amount for reliable water supply for future generations, since the mean WTP was NIS 20.8 monthly of each household-head. These WTP estimates are likely to increase the respondent's current monthly water bill (Mendonca and Tilton 2000).

Regressions' outcomes also show that respondents who have a high level of income and/or use water filter in the households are likely to be WTP for both: improved water quality and reliable water quantity (use values) and also for water conservation for future generations (non-use values).

Households who have high level of water consumptions are unlikely to be WTP for use and non-use values. Rural respondents were unlikely to be WTP for use and non-use values of domestic water services, while urban respondents are likely to be WTP for non-use values. According to Kruskal-Wallis K test, there is a gap among cities, villages, and refugee camps in terms of WTP, since respondents of urban areas and refugee camps are likely to be WTP. Moreover, the percentage change of WTP with respect to income change was very low.

Households who are gainfully employed are unlikely to be WTP for non-use values. On the other hand, the variable age, gender, education, time period "how long the respondent has lived in the region?", and household size have insignificant impact on WTP.

In general, as a result of water scarcity and the unique situation of Palestine the households could accept to deal with water as an economic good, but they dislike affording more than what they are paying currently. However, it seems the respondents are unlikely to deal with water as an economic good at present.

This study also revealed that the main water institutions adopt already graduate pricing system for water and interested to attain the minimum level of average costs at-least, but they neglect to consider the environmental costs as a part of the total costs of water.

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ANT COLONY OPTIMIZATION FOR LEVEL OF SERVICE IMPROVEMENTS IN PIPING NETWORKS

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Abstract: The paper presents a methodology for optimal flow routing in piping networks by imitating the natural selection processes utilized by real-life ants in search of the shortest path to a food source. Ant Colony Optimization is a population-based, artificial multi-agent, general-search technique for the solution of difficult combinatorial problems with its theoretical roots based on the behaviour of real-ant colonies. The fundamental mathematical background of the ACO method is outlined and a suggested possible implementation strategy is described for solving for shortest paths in water piping networks. The ACO methodology should be of interest to both researchers and practitioners since it provides an alternative method to routing optimizations, with a wide range of applications. The described ACO virtual multi-agent approach is supplemented by a sample case study as well as algorithms for the optimization of urban water distribution networks.

Key words: Water distribution networks; ant colony optimization; pipe routing; optimization.

1. INTRODUCTION

Ongoing globalization and urbanization has brought upon urban water distribution network managers an increasing need to better understand and manage the pipe networks servicing them so as to increase the networks' efficiency and reliability and to alleviate the strain on the usage of water resources. The increase in demand for water resources and reliable service is even more pressing in developing countries and countries with limited resources.

In the case of urban water distribution networks in areas of extended droughts and limited water resources, the problem is even more complex due to the inability of the networks' owners to easily and cost-effectively replace utilized or lost resources, the lack of alternatives, and the pressing need to provide these resources to the public. As a result, utilities in charge of managing such water distribution networks are nowadays faced with the increasingly more complex task to intelligently and efficiently manage such networks in ways that maximize a system's reliability and minimize its operational and management costs. In these cases, optimal pipe routing (under either normal operational conditions or emergency rerouting conditions), life-cycle costing and maintenance strategies become of paramount importance to the utilities as they seek ways to increase system reliability and quality of service while minimizing costs of operation.

Central to this balancing act of operating costs and reliability are two very important questions facing water distribution organizations: (1) what is the best way to route a network so as to minimize installation, operational and maintenance costs and increase its reliability, and (2) how can one evaluate the risk of failure for a piping network, and the proper course of action in terms of repairing or replacing a failure-prone or unreliable pipe segments? The latter of the two aforementioned problems has been addressed and reported upon extensively in literature (Andreou et. al. 1987, Aslani 2003, Vanrenterghem 2003, Christodoulou et. al. 2006, Christodoulou 2007)

and several frameworks have been suggested. The work presented in this paper addresses the former of the two aforementioned problems, and to the extent of optimizing the routes (pipe segments and valves) and operational costs of a network and for subsequently evaluating the node-to-node risk of failure and resulting impacts of failure. The goal is to seek methodologies that would allow water distribution network managers to arrive at speedier and easier evaluations of the reliability of a pipe network (or a pipe subnet) and through that at more efficient designs and management of urban piping networks. The ability to quickly assess possible pipe paths and the probability of failure from a given node to another node of interest within the system will allow the pipe network's managers to increase the reliability-of-service within the network and to better react in the event of a catastrophic event to quickly resume operations and service-flow within the nodes of interest.

Traditional pipe routing techniques exhibit functional and computational limitations in their approach. For one, water utilities seem to route their piping networks in a "seemingly arbitrary manner" in which pipes are laid down a major road and then branched out to side streets based on consumer demand and the rate of urbanization. Secondly, the process makes no provision for longest (or shortest) paths from any node to any other node in the network. Consider, for example, the need to identify the longest and shortest paths to a desired location in the pipe network, given that at the time of evaluation the water distribution system is experiencing a failure at another location in the network. Which sequence of pipe segments and valves need to be opened or closed to redirect the flow of water to the specific location (network node) at minimum cost and time? Additionally, what is the number of customers affected by each possible rerouting scenario and how can one optimize the choice of rerouting paths? Furthermore, how can the network's combined risk of failure be evaluated and minimized given the possible network topologies resulting from the possible pipe routings?

A possible improvement to traditional network routing practices may come in the form of intelligent path-traversing algorithms, and in particular artificial agent technologies. One such methodology is the Ant Colony Optimization (ACO) metaheuristic that imitates the collective behaviour of real-life ant colonies; a behaviour characterized by a collective knowledge-processing system that relies on knowledge acquired by individual members separately as each one "walks" a possible path. The information acquired by the individual members (trail-laying), processed and then memorized by the colony (trail-following) when in search for path solutions to a destination eventually converges to an optimal solution to the path-traversal problem; a process very similar to pipe routing and network optimization.

In computer implementations of the ACO algorithms, artificial ants are agents and solution-construction procedures that stochastically build solutions by considering (1) artificial pheromone trails which change dynamically at run time to reflect the agents' acquired search experience, and (2) heuristic information on the problem/network being solved.

2. ANT COLONY OPTIMIZATION

Ant Colony Optimization (ACO) is a population-based general search technique, proposed by Dorigo (1992, 1996), for the solution of difficult combinatorial problems. The method is inspired by the foraging behaviour exhibited by real ant colonies and the essential characteristic of ACO algorithms is "*the combination of a priori information about the structure of a promising solution with a posteriori information about the structure of a previously obtained good solution*" (Maniezo et al. 2004).

Why Ant Colony Optimization, and how can it be applied to pipe routing? The answer to this question lays in the similarities that ACO and CPM exhibit in their approach to network mapping and to longest-path or shortest-path calculations. It has been observed that real ant colonies exhibit a

behaviour that is suitable to optimal network traversing and thus, by induction, to pipe routing. This behaviour is characterized by an ant colony's ability to map a network and the possible paths within it, and to find the shortest path between a food source and an ant nest through the use of chemical aids (much similar to corporate learning and memorization). The process of network optimization by real-rife ants is based on the ants depositing a chemical substance (termed "pheromone") while traversing possible paths and forming "pheromone trails" which can then be followed by other ants in the colony. When choosing their way through the possible path routes, ants can sense the deposited pheromone and follow, with a stronger tendency, the paths which are marked by stronger pheromone concentrations. In essence, while an isolated ant moves essentially at random, an ant encountering a previously traversed path and pheromone-laid trail can detect such path and therefore decide with higher probability to follow the path and subsequently reinforce it with the ant's own pheromone.

The collective behaviour is therefore characterized by a positive (reinforcing) feedback loop where the probability with which each ant chooses the path to follow increases with the number of ants having chosen the same path in the preceding steps. The final result is the relative quick convergence of the path-traversing process to the shortest path.

Since the original work by Dorigo (1991, 1992) a number of ACO algorithms has been developed, the most known of which being the Ant Colony System (ACS), the Elitist Ant System, the MAX-MIN Ant System, a rank-based version of Ant System, and the Best-Worst Ant System. The common framework for all ACO applications was proposed posteriori to be the ACO metaheuristic (Dorigo et al. 1999) while the generic problem topology and generic behavior of the artificial ants was outlined by Stützle et al. (2002). Within this framework, artificial ants are seen as stochastic solution procedures, acting as agents. These agents have short-term individual memory that helps them identify available decision options when reaching a decision node (forward pass), and additive corporate memory when the agents' collective experience is combined to form an optimal route (backward pass). A flowchart adaptation of an ACO algorithm is shown in Figure 1.Construction of the solution by the artificial ants is biased by the pheromone trails (which change at run-time), the heuristic information on the problem instance and the ants' private memory.

3. ANT COLONY OPTIMIZATION AND ROUTING OF PIPE NETWORKS

Since the underlying topology and path-traversing mechanism in ACO exhibits many similarities to urban water distribution systems (UWDS), the concepts and methodology employed by the ACO metaheuristic can find a direct application in pipe routing optimization. If one substitutes the search for shortest path (ACO) to the search for shortest or longest path (UWDS) and treats ACO ants, states, connections and cost function to UWDS's water flow, operational state, pipes/valves and customers serviced respectively then the ACO metaheuristic can be employed in solving for the shortest (or longest) path in connected, acyclic graphs (such as pipe networks). Furthermore, by considering different nodal states and ant types then the search for shortest path can be accompanied with calculations for the shortest path from a node to any node, plus level-of-service improvements.

For a given network topology (urban water distribution system) defined by a graph $\mathbf{G} = (N, A)$, with N being the set of nodes (valves) and A being the set of arcs (pipes) connecting the subject nodes, the proposed ACO-based procedure for finding the shortest path(s) between a set of chosen nodes N_1 and N_2 can be summarized by the following steps:

1. Initialize all arcs with small amounts of pheromone, τ_0. This value can be an inverse line-distance between the nodes N_1 and N_2, or the inverse line-distance of the subject arc.
2. An artificial ant is launched from node N_1 (the start node) pseudo-randomly walking from a node to a successor node via the connecting arcs until it reaches either the end-node (N_2) or a dead end. When at a given node, the artificial ant's selection of an arc to follow is probabilistic, based on a stochastic assignment of each i^{th} arc's likelihood of selection, as defined by

$$p_i = \frac{\tau_i \eta_i^{\beta}}{\sum_i \tau_i \eta_i^{\beta}} \tag{1}$$

   In the above equation, τ_i is the pheromone concentration on the i^{th} arc, η_i is an a priori available heuristic value for the i^{th} arc and β_i is a parameter determining the relative influence of the heuristic information. The value of η_i can be defined either as the inverse of the length of the arc, or the inverse of the length of the arc plus the line-distance between the subject node and N_2. It should be noted that previously visited arcs are excluded from the selection (to enable complete "tree spanning" and avoid "memorization"). The selection is further assisted by the consideration of a randomly generated number, $0 \le q \le 1$, which is compared to a predefined value, q_0, specific to the network topology. If $q \le q_0$ then the arc with the highest value p_i is selected. Otherwise, a random selection of an arc is used based on the distribution defined by the equation for p_i.
3. Upon crossing each i^{th} arc during the aforementioned solution-constructing phase a local pheromone update rule is applied to update the level of pheromone concentration at the given arc. The updated pheromone level is defined by

$$\tau_i = (1-\rho)\tau_i + \rho\tau_0 \tag{2}$$

   where ρ is another network topology parameter ($0 \le \rho \le 1$). As already noted, the goal of the local updating rule is to enable exploration of more path/route variations by making already traversed arcs less likely to be chosen again during the randomization of the arc selection process.
4. Steps (2) and (3) are repeated for all ants in the ant colony and the most successful ant (i.e. the one whose path defines the solution) is used to globally update the network's pheromone trails. The global update rule is defined by

$$\tau_i = (1-\alpha)\tau_i + \alpha\tau_L \tag{3}$$

   where α is yet another network topology parameter ($0 \le \alpha \le 1$) whose value determines the level of evaporation of pheromone concentrations. The factor τ_L is a value inversely proportional to the path length of the best solution in case of an arc visited by the best ant or zero for all other ants. The global update rule can be applied by either the "global-best" or the "iteration-best" ant. In the first case, the ant to perform the update is the one that obtained the best solution (found the shortest path in the network) during the entire optimization process. In the second case the update is performed by the ant reaching the best solution during each iteration of the algorithm.
5. Steps (2) - (4) are repeated for either a fixed number of iterations or until a predefined condition is met, and upon termination of the algorithm the pheromone trail in the graph

$\mathcal{G} = (N, A)$ is used to determine the solution (the arcs with highest pheromone concentration form the shortest path of the network).

It is important to note that since the desired optimization is on the shortest path of the network the calculations performed are on the negative values of the pipe lengths. This is necessitated by the fact that the aforementioned ACO algorithm maximizes the total distance between the start and end nodes. Therefore, should one desires the shortest distance then (s)he should operate on the negative values of the pipe lengths, keeping the rest of the algorithm calculations unchanged.

4. CASE STUDY NETWORK

The network topology in study (Figure 3) is a hypothetical simplified urban water distribution network that has 3 inputs (could be thought of as water reservoirs or inflow pipes), 6 main valves and 17 water distribution mains for a total length of 749 pipe-segments (each segment is actually 6 meters in length, but for the purpose of simplicity and generality the length representation will be left in generic terms).

In ACO terms, the case-study network has a topology of 10 nodes and 17 node connections and of assumed network parameters $q_0 = 0.3$, $\beta = 1.0$, $\rho = 0.5$, $\alpha = 0.5$, and $C = 50$. Of the 10 network nodes included in the assumed network topology, 3 are "ant nests" (i.e. nodes with no predecessor nodes), 6 are regular nodes and 1 is a "food source" (i.e. a node with no successor nodes). It is also noted that the network in study is based on unidirectional nodal connections (an acyclic graph) so as to emulate real-life gravity-based water distribution networks, and that the network construction is based on an assumed maximum number of three successor connections per node, to simplify the manual calculations and subsequent verification by standard forward and backward-pass procedures (a Critical Path Method algorithm is used). A two-dimensional annotated view of the network topology is shown in Figure 1, also including the node-to-node lengths of all pipe segments in the network.

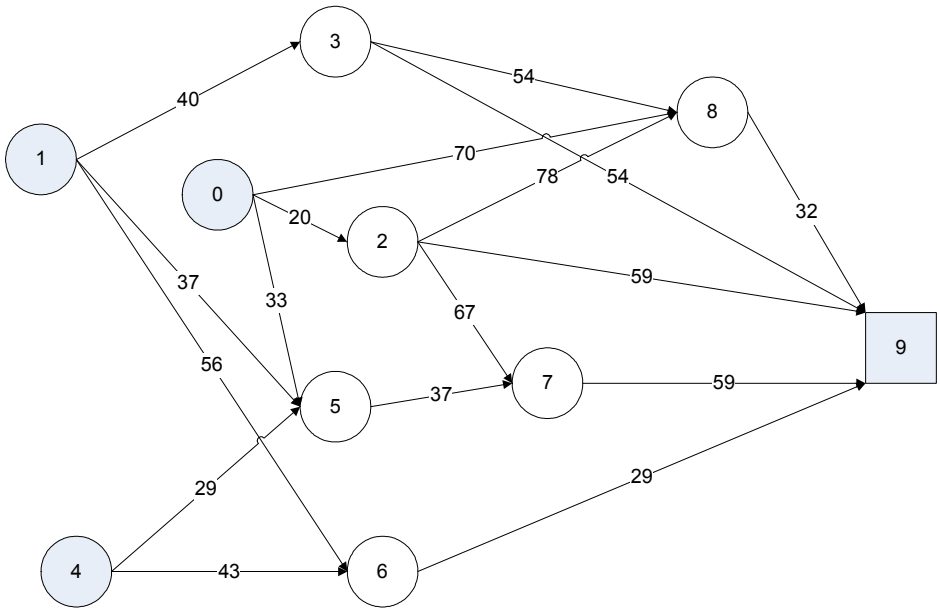


Figure 1. Annotated ACO topology of case-study piping network.

Let us consider the following scenario: given that a pipe breaks at the outskirts of our subnet (just before one of nodes '0', '1', or '4'), what is the best way to reroute water flow so as to guarantee service at node '9'?". This is, in essence, a node-to-node shortest-path problem that requires evaluation of all possible paths from start-nodes '0', '1' and '4' to end-node '9'. The solution path includes pipes '4-6' and '6-9' for a shortest-path length of 43+29= 72 units.

Let us now turn our attention to the ACO algorithm. Even though different solution states are reached at the end of each "ant run", convergence to the correct solution is eventually achieved (in fewer than 10 iterations actually). Since the method is "intelligently" iterative (each successive iteration is stochastically dependant on previously acquired knowledge about the network topology) the shortest path may change several times before converging and stabilizing to the correct solution.

The final ACO solution results are shown in Figure 2 and tabulated in Table 1. During each iteration ("ant-run") the algorithm generates the associated pheromone concentration levels for each arc, deduces the criticality of each connection and through that identifies the resulting shortest path (thus the critical pipe segments). The solution obtained by the ACO algorithm is tabulated in Table 1 as "FinalPheromone" values. The algorithm considers these values during the solution phase and decides which pipe segments on a continuous path are critical (column "On Shortest Path?" in Table 1).

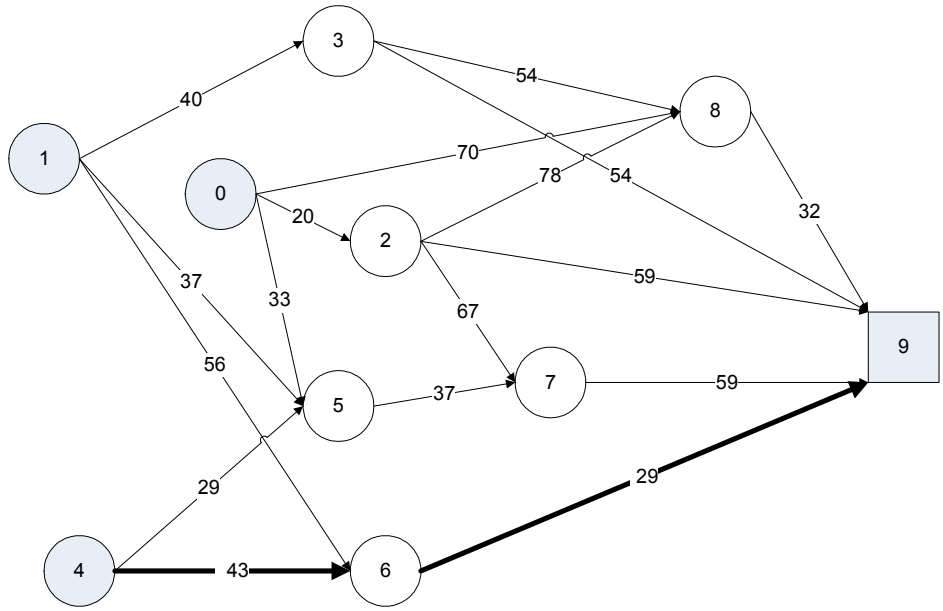


Figure 2. ACO solution of the case-study network topology.

Table 1. Solution of the case-study network topology using ACO.

Start Node	End Node	Arc Length	Pheromone Level				On Shortest Path ?
			Original	After 50 iterations	After 100 iterations	After 200 iterations	
0	8	70	0.01429	3.69E-15	0.00E+00	0	-
0	5	33	0.03030	8.88E-16	0.00E+00	0	-
0	2	20	0.05000	1.66E-02	1.43E-02	1.29E-02	-
1	5	37	0.02703	8.88E-16	0.00E+00	0	-
1	3	40	0.02500	1.15E-15	0.00E+00	0	-
1	6	56	0.01786	2.96E-04	2.51E-03	6.37E-04	-
2	7	67	0.01493	8.88E-16	0.00E+00	0	-
2	9	78	0.01282	4.55E-16	0.00E+00	0	-
2	8	59	0.01695	5.64E-03	4.85E-03	4.38E-03	-
3	9	54	0.01852	8.88E-16	0.00E+00	0	-
3	8	54	0.01852	9.70E-16	0.00E+00	0	-
4	6	43	0.02326	1.39E-02	1.39E-02	1.39E-02	Yes
4	5	29	0.03448	1.36E-04	3.37E-05	5.42E-03	-
5	7	37	0.02703	1.07E-04	2.64E-05	4.25E-03	-
6	9	29	0.03448	1.31E-02	7.08E-03	1.22E-02	Yes
7	9	11	0.09091	3.59E-04	8.89E-05	1.43E-02	-
8	9	32	0.03125	8.88E-16	0.00E+00	0	-

The end-result (Table 1) is the convergence of the ACO-based process to segments '0-2', '1-6', '2-8', '4-6', '4-5', '5-7', '5-9', '6-9' and '7-9' as important (high pheromone concentration). The algorithm then sifts through these pipe segments to identify the shortest continuous path from

network-start to network-end and flags activities ‘4-6’ and ‘8-9’ as critical, with a calculated shortest path length of 72 units, in agreement with the solution obtained by traditional methods.

The ACO method for shortest or longest-path can be extended to account for impact calculations and for level-of-service improvements. If the pipe lengths are replaced with the number of consumers serviced by each pipe segment then the impact of each pipe burst can be linked to the number of consumers affected. Then, the ACO algorithm can be used to find the longest path in terms of customers affected and severity of event, and the shortest path to reroute the water flow so as to minimize the number of customers affected.

4. CONCLUSIONS

The ACO artificial agent seems to provide a powerful means to performing network optimization through shortest path calculations, since it is able to efficiently construct shortest-path solutions in acyclic (unidirectional) network topologies. Despite the seemingly iterative approach of the ACO algorithm, the method exhibits quick convergence to the final solution (as shown in the examined case study) and thus low computational time. The ACO methodology can further be modified and extended to account for reliability and level of service calculations, and node-to-node optimizations. The former can be achieved by considering the probability of failure for each pipe segment and including it in the utility function to be optimized. The latter can be achieved by setting the start node of the node-to-node sequence of interest as “ant nest” and the end node as “food source” and reconstructing the solution path (shortest path) while all other nodes are set as plain network nodes.

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MANAGEMENT STRATEGIES TO TACKLE WATER LEAKAGE IN URBAN WATER DISTRIBUTION SYSTEMS

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Abstract: Water shortage and the future threat posed by changing climatic conditions has intensified the need for the development of appropriate water management approaches, which aim in keeping a balance between water supply and demand.

Losses from water distribution systems must be of concern to every water utility, especially in areas of our planet where water is found in very limited quantities. It is therefore imperative that water utilities apply simple and effective methodologies in accounting for water losses from their transmission and distribution systems. The Water Loss Task Force (WLTF) of the International Water Association (IWA) has established a water audit method, which traces water from its source right through the system and derives at the end the revenue and non-revenue component, in other words is a methodology for water accountability and an integrated approach to water loss control (Yates, 2005).

The Water Board recognised at a very early stage the importance and significance of establishing a proper water audit system and has over the years developed its infrastructure in such a way in order to be able to account efficiently and accurately for all water produced. Reduction and control of water loss was achieved through the application of a holistic strategy based on the approach developed by the WLTF of the IWA. Integral part of this approach is the establishment and operation of DMAs coupled with pressure management.

Key words: District Metered Areas, Leakage Reduction, Pressure Management, Urban Distribution Systems.

1. WATER LOSS CONTROL

1.1 General

Efficient and effective water loss control should be recognised as a first priority for improving potable water supply. Decision makers at all levels in water utilities must understand that any water loss control strategy in order to be effective must be a continuous activity based on a long term strategy and should form an integral part of the utility's vision. The success of the strategy will inevitably depend on the commitment and dedication at all levels within the utility and of course on the adoption of appropriate strategies and techniques.

The benefits of a water loss control strategy could be summarised as follows:

- Saving a precious and valuable resource.
- Increasing the efficiency of existing systems.
- Delaying huge infrastructure investments.
- Increasing the life expectancy of the systems.
- Increasing the revenues for the water utility.
- Reducing energy requirements.
- Improving the Carbon Footprint of the utility.

1.2 The Methodology

The IWA Water Loss Task Force (WLTF) has played a principal role in the advancement of water loss control strategies, methodologies and procedures world wide and in the development of

world class tools and techniques to reduce losses and increase water accountability in revenue generation. Methodologies have been developed by the WLTF in the following areas of urban water networks management which were tested in many Utilities world wide with very positive results in reducing Non-Revenue Water: These methodologies and practices may be summarised under the following broad headings:

- Accurate and comprehensive metering
- Water balance
- Apparent losses
- Real losses

1.3 Real Losses

The IWA Water Loss Task Force has been promoting for a number of years the four leakage control strategies to reduce Real Losses from urban water distribution systems, namely:

1. Active leakage control
2. Pressure management
3. Speed and quality of repairs
4. Targeted renewal of infrastructure

These have to be balanced in order to achieve the most cost effective leakage programme which reduces leakage to an economically, environmentally and socially acceptable level (Figure 1).This approach is well tested and has been applied around the globe with extremely positive results for utilities.

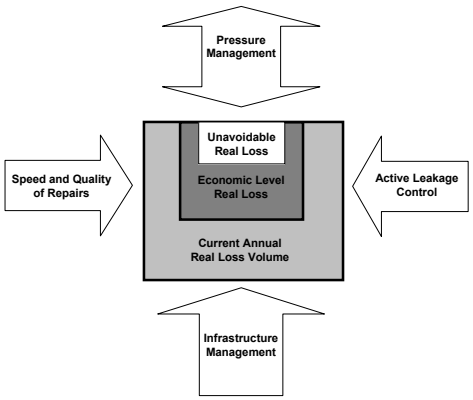


Figure 1. Four Leakage Control Strategies

The above WLTF methodologies and strategies have a global application and represent what could be termed “best practice” in the area of water loss control. They have been implemented successfully in countries in Europe, North and South America, the Caribbean, Australia, New Zealand, Pacific Islands, South Africa and South East Asia.

As a first step in the reduction of leakage from water distribution networks it is recommend that pressures in the network are examined and optimised in such a manner that any excess pressure is removed. Leakage is a function of pressure and any reduction in pressure would result in a corresponding reduction in leakage. Dividing the network in discrete areas called District Metered Areas (DMAs) provides an efficient and effective way of managing pressures and controlling leakage thus reducing the Real Losses to an economically acceptable level.

2. DISTRICT METERED AREAS

2.1 Definition

A District Metered Area (DMA) is defined as an area of the supply network having ideally about 2000 properties supplied preferably from a single entry point which is metered (water entering and leaving) and pressure controlled.

2.2 Objective

The main objective of establishing a DMA is to reduce real losses to an economically acceptable level and to maintain this level through the application of proactive strategies, such as Active Leakage Control.

2.3 Advantages

There are a number of advantages in setting up DMAs, namely:

- The areas of the network are smaller, more manageable.
- The application of active leakage control is easier.
- Leaks are identified quicker based on MNF monitoring.
- Run time of leaks is much shorter.
- Better pressure optimization.
- Lower water losses.
- Financial savings.

2.4 Disadvantages

Minor problems may be encountered in the formation of DMAs which of course with proper planning and design can be resolved. These problems are:

- Water quality problems associated with "dead ends" in the network.
- Customer complaints due to reduction in water pressure in the system.

3. DMA LITERATURE REVIEW

Farley and Trow, 2003, referring to the DMA concept stated that "the technique of leakage monitoring is considered to be the major contributor to cost-effective and efficient leakage management. It is a methodology that can be applied to any network."

Dividing the distribution network into a number of leakage monitoring Zones or DMAs has been proved to be an efficient and effective technique for monitoring and controlling water loss. In Manual no. 4 of Managing and Reducing Losses from Water Distribution Systems, 2002, is stated that "*dividing a distribution system into sectors such as PMZs and DMAs, which are then individually managed and monitored, enables*:

- *Continuous monitoring and recording of the total quantity of water entering each sector.*
- *Leak detection teams to target sectors and minimise the length of time leaks run for, if total supply is inexplicably high in particular sectors.*
- *Water quality incidents, for example dirty or discoloured water, to be contained within one sector and not spread throughout the network."*

The application of the DMA concept has universal application. Irrespective of geographical location or distribution network size it has been established that this technique is reliable for

leakage reduction and monitoring. However, it is important in order to maintain results to achieve long-term sustainability of the DMA concept.

MacDonald and Yates, 2005, stated that "*the experience of DMA implementation at Halifax Regional Water Commission has been nothing but success".*

Sturm and Thornton, 2005, reported that on the basis of their research project findings on nine North American Utilities with the goal to assess how international best practices can be transferred to North America, DMA techniques are applicable to Utilities in North America.

On the issue of sustainability, especially in developing countries, Loveday and Dixon, 2005, stated that "*DMA implementation is a key component of NRW reduction activity and their maintenance must be seen as a long-term commitment".*

4. REAL WATER LOSSES

The IWA recommended definition of Real Water Losses, sometimes referred to as "physical losses", is the annual volume of water lost from the system through all types of leaks and overflows from reservoirs and from bursts in mains and service connections up to the point of customer metering (Fanner, 2004). The Water Board of Lemesos has placed great importance in reducing the real losses and this is reflected in its adopted strategy over the years.

The development of the system infrastructure took place in a much organised fashion with new areas of supply being incorporated into their respective pressure zones, which are strictly governed by contours. Each pressure zone is subdivided into DMAs, having a single metered source with physical discontinuity of pipe network between DMA boundaries.

The DMAs vary in size from 50 properties to 7000 although the average size being approximately 3000 properties. Distribution main diameters within the DMAs vary between 100mm and 250mm and where possible, interconnecting ring systems within the DMAs have been formed to minimize head loss at peak demands.

The Water Board has maintained records of its operational activities since 1963, which include production of water from sources, distribution through district meters and consumption from consumer meters. Meter readings at water sources (boreholes and treatment plant) are connected via a SCADA telemetry system to the control room. This enables continuous monitoring of the water source outputs and accurate recording of flows. Likewise storage reservoir outlet meters are monitored on SCADA providing the same ability to observe trends as well as to record daily, weekly, monthly and yearly totals.

As all the trunk mains, made of ductile iron, are purely dedicated to transferring water from sources to the storage reservoirs, it is possible to carry out a water balance between production meters and storage reservoirs outlet meters. The results show that on a yearly basis the difference between the production meters and reservoir outlet meters is less than 1% which is considered to be negligible and is attributed to meter registration errors.

Distribution of water to the DMAs is effected through dedicated ductile iron mains from the storage reservoirs. Each pressure zone has its own dedicated storage reservoir supplying the DMAs within the specific pressure zone. Each DMA has a single feeding point, which is metered. With this arrangement it is possible to carry out a water balance between the storage reservoir outlet meters and the DMA meters. The results show that on a yearly basis the difference is about 2%, which is attributed to meter inaccuracies.

Therefore it could safely be assumed that all real water losses are within the DMAs.

5. DMA REDESIGN

Since 1993 leakage management was effected through data logging, pressure reduction and leak location. Data logging for flow and pressure was carried out using ten "Radcom" data loggers. Logging was typically done for a period of 7 days and the loggers were then removed, the data

downloaded, and deployed again to monitor other DMAs. In this way each DMA was monitored 3 to 4 times a year for a period of one week each time. It was also recognised that there was potential for pressure reduction and in 8 out of a total of 27 DMAs, Pressure Reducing Valves (PRVs) were installed. Initially leak location was done using step testing and sounding. Since 1999 the Water Board employs state of the art technology for leak identification and location including, acoustic loggers, correlators and ground microphones.

The above methodology, however, presented several shortcomings. The data available from the periodic logging were insufficient to enable proper monitoring and control. Awareness, Location and Repair (ALR) time of leaks was extremely long and it was very difficult to prioritise intervention activities. The deployment and removal of loggers was time consuming and labour intensive.

It was obvious that there was a need for an immediate review of the Water Board's leakage management strategy. After careful consideration and examination of the available techniques, methodologies and technologies it was decided that in order to achieve further reduction of real losses from the distribution network it was imperative that a proactive approach in evaluating the efficiency of the network needed to be adopted based on continuous monitoring of flow and pressure in all DMAs.

Therefore, it was considered important to first carefully examine the size of the DMAs in the pressure zones in an effort to further reduce the real losses from the network and provide better and more effective active leakage control.

The key factors for good DMA design (Water Loss Task Force, 2004) formed the basis of the redesign. These were:

- minimum variation in ground level across the DMA,
- easily identified boundaries that are robust,
- area meters correctly sized and located,
- single entry point into the DMA,
- discrete DMA boundaries,
- pressure optimised to maintain standard of service to customers,
- degree of difficultly in working in urban area.

The variation in ground levels across the supply area was examined and particular attention was given to the influence of the pressure within the DMAs. Main highways and physical features, such as streams were chosen to form discrete boundaries between DMAs. A single entry point into the district was chosen where a meter chamber was constructed to house the district meter, a pressure reducing valve and a pressure sensor. It must be stressed that the implementation of the redesign was not an easy task due to the difficulties and restrictions imposed in executing works in built up areas. These works involved inter alia, the construction of new district meter chambers, laying new lengths of pipeline and installation of new telemetry system for continuous monitoring of flow and pressure.

The redesign process yielded DMAs of smaller, more manageable size with physical pipework discontinuity between DMAs. In order to verify that all interconnecting pipes between DMAs were located and isolated, a zero pressure test was carried out which involved closing the valve at the inlet to the DMA thus isolating the DMA and observing that the pressure within the DMA dropped immediately indicating that all interconnecting pipes were isolated. This test was usually carried out between 02:00 and 04:00 in the morning in order not to inconvenience consumers.

It is essential, for the effective operation of DMAs, to establish a reliable continuous monitoring system in order to apply best practice DMA management which involves the analysis of DMA night flow referred to as the Minimum Night Flow (MNF) in order to assess leakage. For this purpose each district meter is equipped with a programmable controller which is powered in most cases by solar energy panels providing a cheap and effective solution, approximately €1800 per station. Most importantly the operating cost of such a solution is extremely low, €14/month. The continuous monitoring of the district meters combines information technology and telecommunication networks to transfer the data via the World Wide Web.

6. PRESSURE REDUCTION

Continuous flow monitoring began immediately upon completion of the redesign works in each DMA. This enabled the establishment of the flow pattern for the DMA providing essential information such as maximum and average daily flows as well as minimum night flows. Data required to establish legitimate customer night use and background leakage in each DMA were collected. Having available this information the Burst and Background Estimates (BABE) component approach to leakage was used to analyse the Minimum Night Flow (MNF).

Management of pressure is a key factor in an effective leakage management policy. This has long been recognised by the Water Board and the ultimate goal is for all DMAs to be equipped with PRVs to reduce pressure where possible and to control and stabilise pressure in DMAs where pressure reduction is not practicable.

Measurements of pressures within the DMAs were carried out to establish operating pressures at the low, medium and high points of the DMA as well as the Average Zone Night Pressure (AZNP) for each DMA. Furthermore, the pressure measurements were critically examined with the aim to reduce pressure as much as possible whilst maintaining the minimum standard of service to the consumers. As a rule a minimum standard of service of 2 bars at the highest point in the DMA at maximum demand was considered. This of course had to be reconsidered in some cases where there were high rise buildings which use the network's pressure to get the water to their roof tanks. In these cases the Water Board will subsidise the installation of ground tanks and pumping systems in order to pump the water to the roof tanks of the high rise buildings thus enabling further pressure reduction to be effected.

The reduction of pressure in the DMAs was effected (Figure 2) over a period of a few hours and amazingly enough no complaints were received from consumers except in some cases where people were using the mains pressure to water their lawns. It was explained to them that they can no longer rely on the mains pressure to water their lawns and they were assisted by the Water Board to install small irrigation pumping systems for garden irrigation. Pressure measurements were repeated in each DMA after pressure reduction was effected to ensure that there were no potential problems and that the minimum standard of service was maintained.

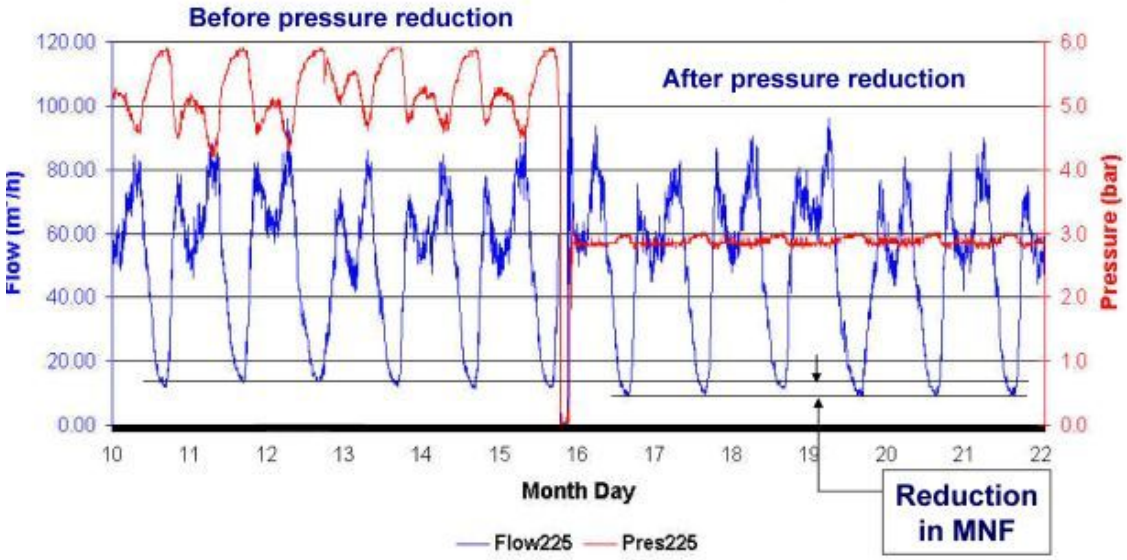


Figure 2. Pressure reduction

7. RESULTS

7.1 Background and Locatable Losses

Having implemented the DMA redesign works, data was collected and BABE calculations were carried out in order to determine background and locatable losses for each DMA. A similar calculation was carried out after applying pressure reduction in each DMA. It must be stressed that the values used for actual MNF were field values measured over a period of approximate one month before and one month after the application of the pressure reduction so that the values of the flows before and after are comparable and are not influenced by seasonal variations.

Pressure reduction was effected in all DMAs without attempting to locate and fix the locatable losses in these DMAs. The application of the pressure reduction resulted in a reduction of 38,1% in the background losses and in a reduction of 38,8% in locatable losses.

It is worthwhile noting that due to pressure reduction there was an immediate reduction of 25% in the volume of water required for whole of Sector 2 which meant a saving of 220.000 m^3 per annum valued at €170.000.

The leakage – pressure relationship $(L_1/L_0) = (P_1/P_0)^{N1}$ is valid for calculating the possible leakage reduction in a water distribution system for a given pressure reduction. The exponent "N1" is specific to the distribution network depending on the type of material that the pipework is made of. In order to establish the "N1" values for the Water Board's network the ratios P_1/P_0 and L_1/L_0 were calculated and the "N1" exponent derived for each DMA. The leakage ratio included both background and locatable losses.

The values of N_1 vary between 0,64 and 2,83 with an average value of 1,47. These figures are of the same order of magnitude as figures reported by others (Lambert, 2001), which reinforce the use of the leakage – pressure relationship $(L_1/L_0) = (P_1/P_0)^{N1}$. The distribution mains in all of the DMA's under consideration is a mixture of asbestos cement and uPVC with MDPE communication pipes.

7.2 New Burst Frequency

Records were kept of reported leaks before and after pressure reduction and an analysis of these shows a reduction of leaks both for distribution mains and communication pipes. The results shown in Table 1 cover a period of seven months before pressure reduction and the corresponding seven months after pressure reduction.

Table 1. Reported leaks

Description	No. of leaks reported		Reduction of leaks
	Before	After	
Distribution mains	49	27	45%
Communication pipes	296	178	40%

The weighted overall Average Zone Night Pressure for Sector 2 was 52,5m before the reduction and 35,8m after. This meant that there was an overall pressure reduction of 32%. Therefore, for a 32% overall pressure reduction there was an overall reduction in new leaks on mains, fittings and communication pipes of 41% which compares favourably with a similar case in Australia reported by Lambert, 2001, where a 40% reduction in one sector of a city reduced frequencies of all new leaks on mains, services and fittings in that sector by 55%. Of course there many other factors

affecting burst frequency of mains such as: weather conditions, accidental damages, etc. (Farley and Trow, 2003).

It was estimated that based on the above percentage reduction in new bursts the Water Board saved in labour and material costs that would have incurred for locating and repairing these bursts approximately €100.000 per annum.

8. CONCLUSIONS

The Water Board of Lemesos operates a well-organised supply and distribution system with permanent pressure zones and District Metered Areas thus providing a solid foundation on which an effective leakage control policy has been developed.

The DMA redesign and the application of pressure reduction has produced favourable results with both background leakage and locatable losses being reduced by approximately 38%. Furthermore the frequency of reported leaks was reduced by approximately 41%. The overall pressure reduction for the fifteen DMAs under consideration was of the order of 32%.

The target of the Water Board of Lemesos is to reduce the NRW to about 10% of the system input volume, which is considered to be the economic level of leakage. The Water Board demand forecasts indicate an increase of approximately 30% by the year 2020 and the leakage reduction will go some way towards offsetting this increase in demand as well as a provide considerable cost saving.

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APPLIED PRESSURE MANAGEMENT TECHNIQUES FOR CONTROLLING LEAKAGE IN ATHENS, GREECE

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Abstract: The purpose of this paper is to present the results, experiences and conclusions obtained so far after the application of advanced pressure management techniques *(flow or time modulation)* in the complex water supply network of Athens, within the overall effort aiming to efficiently provide water to the consumers and to control leakage.
The operation of the Athens water supply network which has a total length of 9.000 km, has always been based on pressure management. Therefore, there are 220 pressure zones from which 140 are supplied through pressure reducing valves. Until few years ago, no specific actions were taken regarding leakage control. After the completion of several projects, EYDAP SA decided to apply advanced pressure management techniques by installing hydraulic or electronic control devices to further control the pressure reducing valves. Two years after implementing these techniques in selected zones, the level of service to customers has been upgraded due to the **minimization of bursts** *(in most cases almost "elimination")* and the **efficient** provision of water *(with adequate pressure during the day and the min required during the night)*, eliminating the consumers complaints. Simultaneously, a drop in the minimum night flow (MNF) was recorded in most zones and a reduction in the mean daily consumption was also achieved. The payback period in all cases was estimated to be a few months only.

Key words: flow or time modulation, leakage control, pressure management

1. INTRODUCTION

The Water Supply and Sewerage Corporation of Athens (EYDAP SA) is the largest water utility in Greece, servicing a population of over 4 million people, with a supply network of 9.000 km and a sewerage network of 6.000 km. EYDAP SA is responsible for the collection, transfer and treatment of raw water, the supply of high quality potable water to the Greater Athens Area, the distribution and operation of the water supply network, as well as for the collection, treatment and disposal of sewage.

Water is supplied to the consumers through 220 pressure zones due to varied ground level in the Greater Athens Area (from +0 m up to +600 with a mean elevation of the 4 treatment plants equal to +211). Therefore, there are 45 reservoirs in the network, approx. 550 pressure reducing valves (PRVs) of diameters 50mm up to 600mm and 70 pumping stations of total installed power equal to 30.000 HP. The mean daily consumption amounts to 1.150.000 m^3/day, varying from 950.000 up to 1.450.000 m^3/day during summer. The water is provided to the customers through 1.950.000 water meters (diameters 5/8'' up to 6'').

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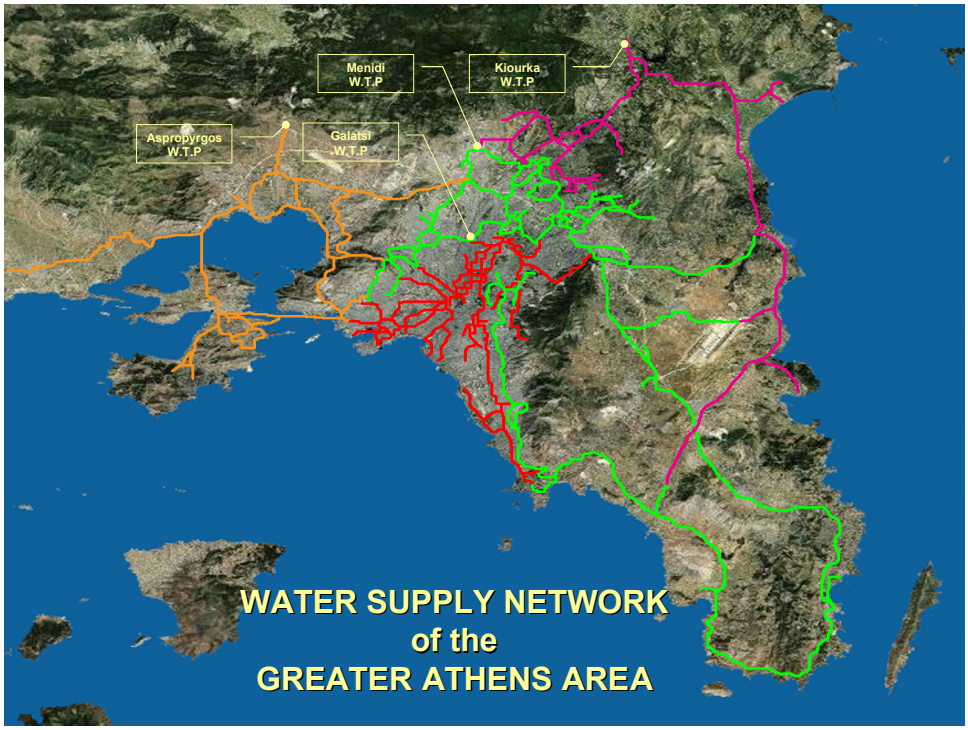


Figure 1. Water Supply Network of EYDAP

Regarding Non-Revenue Water (NRW), it is currently estimated to be as high as 20% from which a mean percentage of 8% corresponds to under-registration of water meters (estimation based on statistical analysis) and the rest 12% to real losses. In the early 1990s, the NRW was estimated to be equal to 40%. Since then, actions were taken regarding the water meters and every year approximately 120,000 water meters are being replaced (multi-jet, Class C). However, the mean age of the water meters is over 10 years. Recently, a project of replacing bulk meters (2'' up to 6'' diameter) which provide water to industries and municipalities, started (these correspond to 30% of the total consumption). A recording and telemetry system using GSM technology is also installed in every site in order to observe operation and to use data for billing through internet. The expected increase in revenue water is estimated to be 3,5 % after the completion of the project (2010).

Until few years ago, no actions were taken by EYDAP SA, regarding real losses. Leakages are usually detected following customers' complaints *(e.g. appearance of water in basements, low unexpected pressure etc).*

2. ACTIONS TAKEN REGARDING LEAKAGE CONTROL

In 2001, five pilot projects which covered in total of1.000 km of the network (200 km each) were completed including all required works for the upgrading of its operation and leakage control: model simulation, construction of new pipelines, creation of district metering areas (DMAs) of limited size (up to 10km each), active leakage control and repair of leaks. The total cost of upgrading and leakage control came out to be approx. 5.870 €/km of network.

The main results of great importance which contributed to further actions' planning were:

- Leakage was important only in certain parts of the network (in 2 out of 5 pilot areas, areas which were very close to the sea). Usually, due to the soil conditions of the Greater Athens Area, water from unreported leakages and bursts is evident on the surface or in basements shortly after their occurrence. Therefore, it is financially preferable to allocate the zones which have high levels of leakage and then apply active leakage control - ALC *(considerable cost of ALC followed by repair).*
- For EYDAP' s complex network, the existing pressure zones should be considered as district metering areas (DMAs) irrespective of their magnitude, in order to determine and

control leakage. It turned out to be almost impossible to create and maintain DMAs of limited size due to the complexity of the network and to the heavy traffic especially in build up areas.

- Flow and pressure had to be monitored at the supply points of the pressure zones on a continuous basis.

In 2004 (just before the Olympic Games), a recording and telemetry system using GSM technology was installed in all operating pressure reducing valves (270 PRVs) in order to monitor pressures on a daily basis. This measure proved to be a critical and important step for the upgrading of the operation of the network and leakage control.

After observing the PRVs operation on a daily basise for some time, it came out that a large number of PRVs had to be rehabilitated because of their bad performance leading to high frequency of bursts especially during the night, as well as to high levels of unreported leakage. This rehabilitation procedure which almost lasted for 2 years resulted in assuring fixed outlet pressure 24 hours per day and guaranteeing the minimum level of service at the critical points of the zones at peak demand. Since the completion of the PRVs rehabilitation, the number of new bursts caused by high pressures during minimum demand (night) has dramatically dropped.

Therefore, having the experience and the accomplishments of all previous works executed during the last years, EYDAP SA decided that the next step in order to control leakage and burst frequencies in problematic pressure zones, was to apply advanced pressure management techniques which provide better control of pressure, reducing unnecessary or excess pressures with low cost of investment compared to the cost for ALC and repair.

3. APPLICATION OF ADVANCED PRESSURE MANAGEMENT TECHNIQUES

A pilot project of a total cost of 360.000 €, plus VAT, started in 2006 (lasted for 2 years) in which all available technologies regarding modulation, were used and tested for their efficiency (hydraulic flow modulation, fully modulation controllers).

The project was applied in 14 pressure zones in different parts of the network, where water is provided through **30 PRVs** of diameters ranging from 100mm up to 500mm. All available technologies regarding modulation were decided to be tested, such as:

❖ PRVs of diameters ranging from **100mm up to 200mm** were replaced with new PRVs with hydraulic flow modulation (DOROT Control Valves – HYMOD)



Figure 2. PRV with hydraulic modulation



Figure 3. PRV installation in chamber

System advantages:

- Reduction of bursts' frequency.
- No power supply requirements – completely hydraulic operation.
- No extra moving parts in addition to those of a standard fixed outlet PRV.
- Fast, real time response to demand changes.

- Easy modulation of pressure-flow profile for reasons of optimization without the need of orifice plate replacement.

❖ PRVs of diameters from **250mm up to 500mm** were upgraded and <u>fully modulating controllers</u> (Palmer Environmental) were installed.



Figure 4. PRV with electronic modulation



Figure 5. PRV installation in chamber

System advantages:

- Continuously variable outlet pressure profiling (enabling time or flow modulation).
- Outlet pressure can be set remotely – *no need to visit site.*
- Optimized profile according to demand or time *(e.g. summer/winter).*
- Fully compatible with Hydraulic Actuator.
- No external power supply requirements. Powered by an internal battery with an expected operational life of over five years

The work was divided into two stages:

<u>*STAGE 1*</u> included all works regarding the upgrading of PRVs, such as installation of filters, flow meters, new PRVs up to 200mm in diameter, construction of by-passes, rehabilitation of larger in diameter PRVs and finally installation of controllers.

<u>*STAGE 2*</u> included all works needed to transform the pressure zones to DMAs, as well as the operation of the zone by using advanced pressure management practices. During this stage, the problems which had to be faced and solved were:

- Some pressure zones because of their magnitude and the multiple feeds through PRVs, had to be redesigned and their boundaries had to be redefined. A mean sized DMA has a total length of pipelines equal to 35 km and about 14.500 water meters (customers) equivalent to 4.800 connections depending on the area of the network.
- Each pressure zone had to become a District Metering Area (DMA) by checking the tightness of boundary valves by acoustic means and by carrying out pressure drop tests. Most common problems encountered, were:
    - large number of boundary valves *(usually more than 30 valves in each DMA).*
    - 30-40% of boundary valves required rehabilitation *(valves which were inoperable and had to be replaced, valves which had to be located and exposed, valves under cars, valves with leaks, valves which were half open for different purposes…)*
    - In limited cases, some “unknown” connections between neighboring zones were discovered after pressure drop tests failures.

- After proving a DMA, inox caps constructed by EYDAP's technical department, were placed on the boundary valves in order to differentiate them from the rest of the valves and to eliminate the risk of being operated by mistake.

- Meanwhile, the hydraulic operation inside the zone had to be checked in order to efficiently and adequately provide water to consumers. Critical points had to be determined. After placing data loggers inside the zone in order to check the hydraulic operation, it was found *(in most zones)* that there were parts of the network where pressure was unexpectedly low. Therefore, the valves in that area had to be checked *(under all the above mentioned problems)* and several times the construction of new "smart" connections between pipelines in order to upgrade the hydraulic operation was inevitable.
- Once a DMA was proven, **modulation** - *time or flow* - was applied and the results were evaluated.

All the above works were carried out by EYDAP's small numbered technical personnel in addition to their daily workload regarding the operation and maintenance of the total network.

4. RESULTS

Where possible (regarding the type of pressure control device installed), both time and flow modulation were tested for their effectiveness in efficiently and adequately providing water to the consumers, as well as in controlling leakage.

The main results are:

- Wherever advanced pressure management techniques were applied, the level of service to customers has been upgraded because of:
  - **the minimization – even the elimination of new bursts** which were usually caused until now by excess night pressure *(frequent occurrences of bursts followed by repair works usually annoy the consumers who complain),* contributing to the life extension of the network.
  - the efficient provision of water with adequate pressure during the day and the minimum required pressure during the night throughout the zone.
- Both time and flow modulation have an impact in decreasing leakage. In all DMAs where these techniques were applied, a drop in the minimum night flow (MNF) was observed (max observed drop was 9 l/sec). The magnitude of the drop in MNF mainly depended on the material of the pipelines of the network *(leakage in plastic and A/C pipes is more influenced by pressure, compared to metal pipes).*

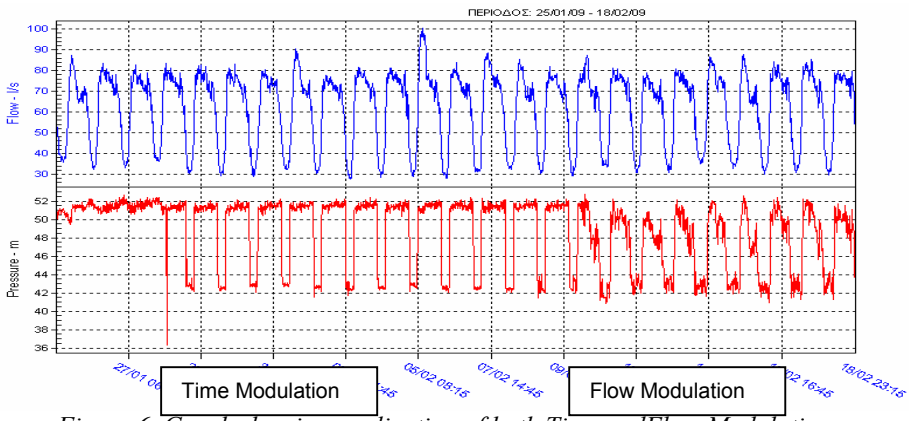


Figure 6. Graph showing application of both Time andFlow Modulation

- A reduction of the mean daily consumption recorded by the flow meters at the supply points of the DMAs after the application of advanced pressure management techniques was

observed. In the case of time modulation, the reduction was estimated to range from 2% up to 4,5%, in comparison to flow modulation where the saving of water is higher reaching 8,5%.

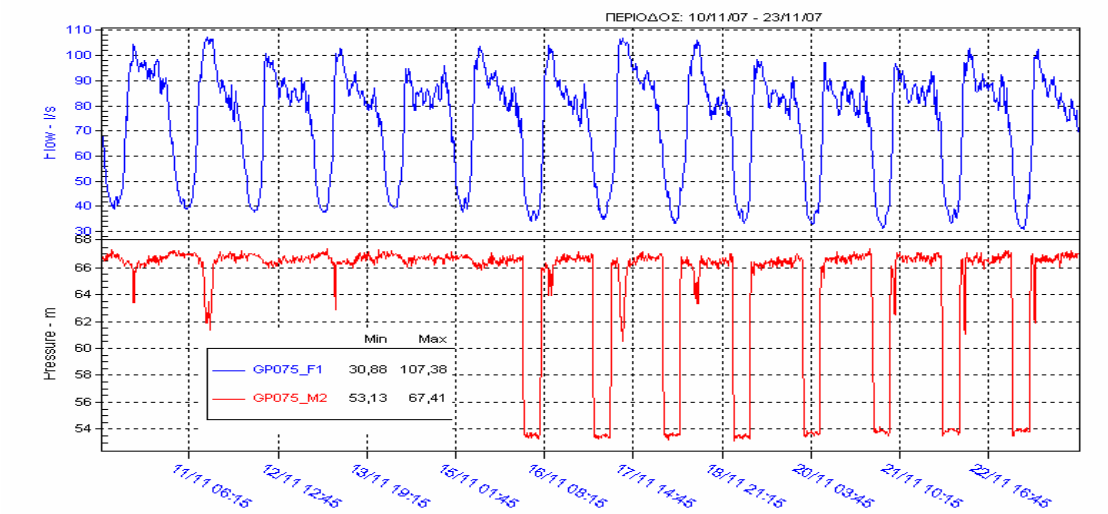


Figure 7. Graph showing application of Time Modulation

- For EYDAP's network, time modulation is more suitable in zones where there are hilly areas inside the zone at higher elevations than the supply points (PRVs) and with high rise buildings with multiple floors requiring continuously high pressure during the day. EYDAP is obliged to provide water at a minimum pressure of 2 bar at ground level, however due to high rise buildings in the urban areas, the operational pressure - if possible - is not permitted to drop under 3 bar during the day hours. Therefore, in such cases and in order to avoid complaints from the consumers who seem to be extremely sensitive to any pressure changes, it is desirable to maintain constant pressure during the day hours and decrease the pressure during the night.

- Time modulation **can** be applied in large pressure zones where water is supplied by multiple feeds under the constraint of not having the output pipelines of the PRVs direct intercommunication. For example, one of the pressure zones which was included in this project, is very large having a length of about 250 km of old network (*mainly AC and cast iron*), however it was selected because of frequent bursts due to night pressure as well as due to the age and material of the pipelines of the network. Initially, it was fed by 6 PRVs (2x400mm, 1x300mm, 2x250mm and 1x200mm). After logging pressure in the zone and conducting upgrading works, the zone is currently fed only through the 2x400mm PRVs where time modulating devices are installed and are successfully operating resulting in a reduction of the mean daily consumption equal to 5,6 %. The 200mm PRV is set at a very low pressure point and is only supplying water during peak demand (the flow is also recorded).

- Flow modulation seems to be more efficient in most cases, eliminating pressure shocks to the zone since the pressure is continuously controlled based on the demand (smooth change over time). Since the application of flow modulation in DMAs, no new bursts were recorded because of night pressure. In DMAs where time modulation was applied to control pressure, there were a few cases of new bursts which occurred during the increase of pressure from low to high value by the controller.

- The payback period in all cases was estimated to be a few months only *(3-4 months depending on the size of the PRV).*

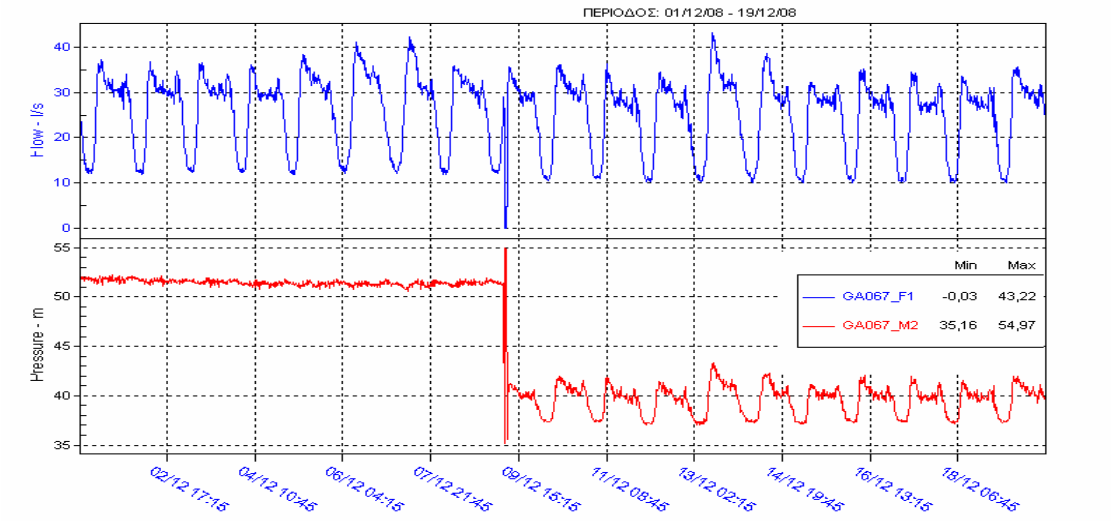


Figure 8. Graph showing application of Flow Modulation

- Due to the above mentioned positive results, EYDAP SA is now expanding the application of advanced pressure management techniques in other zones supplied by PRVs, after accomplishing all necessary works in order to consider the existing pressure zones as DMAs. However, it is time consuming to obtain and maintain the DMAs.

- For the pressure zones where the percentage of leakage is high, further actions will be taken by EYDAP SA and active leakage control is considered to be the next action which will be taken.

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ACTIVE LEAKAGE CONTROL IN LOW PRESSURE AREAS – IT CAN BE DONE

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Abstract: This paper highlights the problems that can occur when working in a low pressure water distribution system. This then poses the question. In countries that traditionally have high pressure within the distribution what is considered 'low pressure'? Australia for instance low pressure may mean 30m of water pressure, USA 20m, UK 15m and Central Europe 10m. This paper shows why low pressure in a distribution system can cause problems when conducting ALC and a case study where ALC was successfully completed in a busy city with an average water pressure of 6m. Although this is deemed to be low pressure there are many cities in the world that has an average pressure of 3m and lower, in these cities it may be considered that the distribution main is not always pressurised and therefore acoustic leakage detection cannot be done.

Key words: Acoustic, Active Leakage Control, Correlator, Pressure, Noise Logger.

1. RELATIONSHIP OF LEAKAGE AND PRESSURE

Water when leaving a pipe creates 'energy' and it is this energy that is transferred through the water and the pipe wall by the means of 'acoustic' leak noise. Dependant on the material of the pipe this noise either dissipates over a long distance or is absorbed very quickly into the pipe wall. The effect being, for metallic or hard walled pipes the sound transfer is extremely good and for non metallic or soft walled pipes extremely bad.

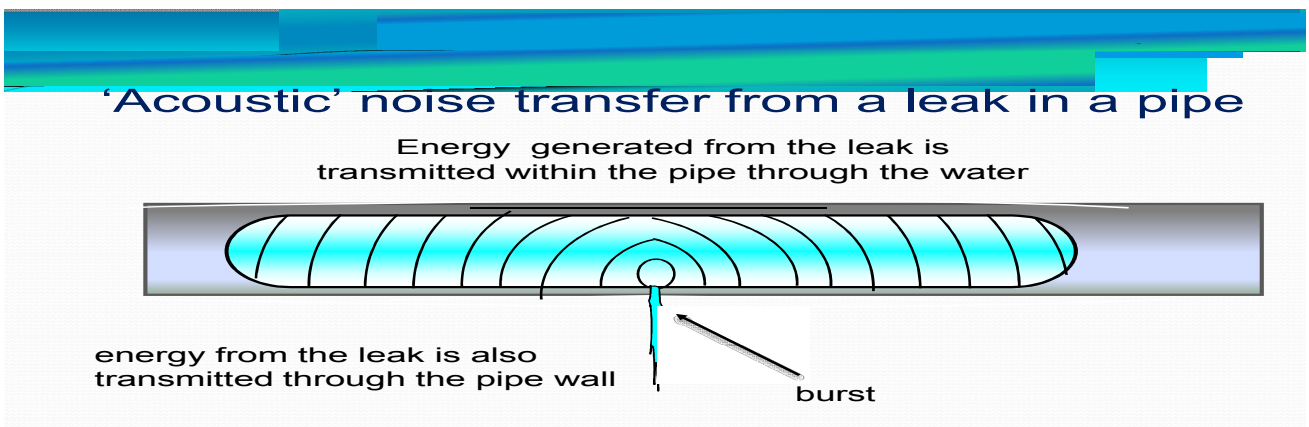


Figure 1. Acoustic noise transfer in a pipe

Another consideration that has to be considered as well as that of pipe material is the pressure within the pipe and the size of orifice.

high pressure + small orifice = good noise / high energy
high pressure + large orifice = medium noise / medium energy
poor pressure + large orifice = poor noise / little energy

2. APPROACHES FOR SUCCESSFUL ACTIVE LEAKAGE CONTROL

The approach now commonly promoted is that of LLP:

- **L**ocalise
- **L**ocate
- **P**inpoint

The methods of active leak control below are restricted depending on the pipe material and acoustic noise transversal over distance.

2.1 Localise

A method of narrowing down the location of the leak by various means to include but not restricted to the list below.

- *Step Testing* – narrowing down the area by means of isolating parts of the distribution system.
- *Acoustic noise logging* – deployment of noise loggers that identify constant noise during the preset hours that indicates a leak may be present. These may be left permanently deployed or adopt the 'lift & shift' technique.
- *All fittings survey* – listening on all customer stop taps and mains fittings with a manual or electronic listening device to identify leaks on mains and property side
- *Mains fittings only survey* - listen on all mains fittings with manual or electronic listening stick to identify leak mainly burst water mains.
- *District Metered Area*- a methodology where flows into an area are monitored and after allowances are deducted for known domestic and commercial usage, the remaining volume can be classified as 'losses' (containing both real and apparent) this volume is converted to known performance indicators which enables the areas to be ranked from best to worst performing for Active Leakage Control (ALC) prioritisation.

2.2 Locate

A method of locating the region of the potential leak to within + or - 300mm

- *Correlator* – a method of locating the position of the leak using acoustic listening equipment, this equipment can correlate over 1000m of pipe work (material dependent) and can locate a leak within + or - 500mm even less providing the correct data entered. This depends on 3 critical entries which is pipe length, pipe diameter and pipe material if any of these are not known to the precise measurement then the leak location measurement has more errors i.e. + or - 5m or even greater.
- *Correlating noise loggers* – a method of deploying acoustic devices that will indentify if a leak may be present (or not) and also give an indication as to the location of the suspected leak. These units can be set to listen either during the hours of night time or at any preset period of the day/night. Accuracy is dependent on the data entered and if information is not known then it is normally + or – 2m. (non correlating noise loggers will only identify if a leak may be present)

2.3 Pinpoint

A method where the precise location of the leak is identified prior to excavations taking place. If the operator is skilled and using the equipment correctly then pinpointing is normally to within + or – 200mm. Factors affecting the ability to locate the leak from the surface, are, the ability for the noise to transfer from the leak through the ground material surrounding the pipe to the surface, the

other being the operating pressure within the pipe work and the size or type of leak that has occurred.

- *Manual listening stick* – where the ability of the operator is relied upon to pinpoint the exact location of the leak. If this operator has any hearing difficulties then the ability to locate a leak accurately is greatly reduced.
- *Electronic listening stick* – a listening stick with an electronic measurement device incorporated, this being either a needle display, a digital readout in form of a bar chart or a numerical display. When using such a device the reliance of the operator's ability to hear is greatly reduced, for this reason it is always advisable to use this type of device.

3. CASE STUDY HANOI - VIETNAM

The initial water balance in the five pilot areas indicated very high levels of losses. It was uncertain if any of the leaks could be detected given the low pressure (6.6 m system wide average pressure). Stuart Hamilton jointly with a team of HWBC staff carried out leak detection between 23:00 and 04:00 during a period in October 2007 and February 2008.

The following equipment was used:

- Listening Stick
- Stethoscope
- Ground Microphone
- Leak Noise Correlator
- Advanced Noise Loggers

3.1 Pressure Monitoring Program

The pressure throughout the Hanoi distribution system can be considered low therefore it was important to understand if ALC could be done. Hanoi has carried out a large-scale pressure monitoring program on around 140 points on transmission, distribution and service lines to understand the pressure situation.

The weighted average pressures observed were:

- Transmission lines: 9.0 m
- Distribution lines: 7.4 m
- Service lines: 6.0 m

The system-wide weighted average was calculated to be 6.6 meter, +/- 10%.

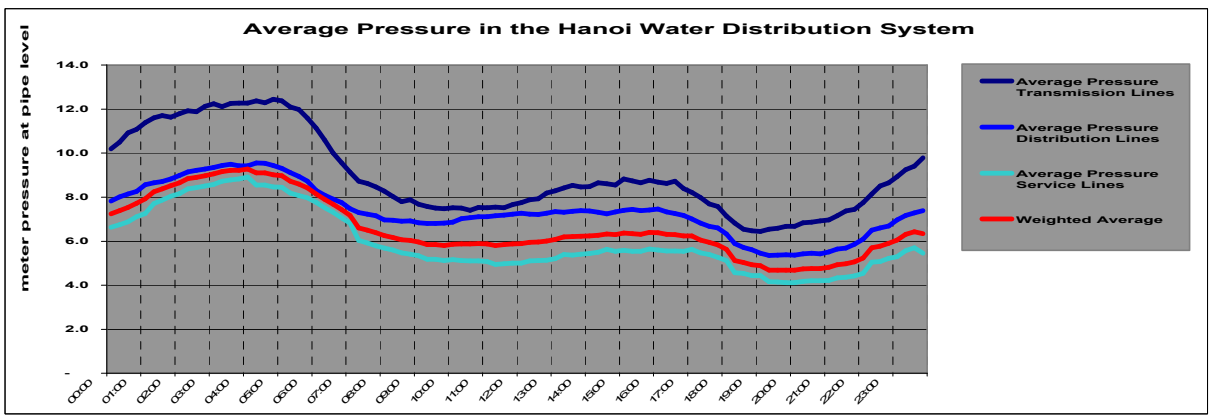


Figure 2 Pressures in Hanoi

Average pressure ‘Transmission Line’ highest
Average pressure ‘Distribution Line’ 2nd highest
Average pressure ‘Service line’ Lowest
Weighted average between ‘Distribution line’ and ‘Service line’

3.2 NRW Assessment in Five Pilot Areas

Pilot Areas were selected in different parts of the distribution system – one area in each of the HWBC's Enterprises.

For each of the 5 pilot areas data was collected which comprised of length of distribution mains, service lines and number of service connections. Flow and pressure data was collected from each of these 5 pilot areas over a 48hr period. The period was restricted to this time as low pressure was being experienced in adjoining systems preventing a permanent DMA from being established or extending the measurement period.

Summary of findings during leakage survey

Different approaches were used to ascertain if acoustic equipment can be used in low pressure areas of different service pipe and mains materials. Decisions for the methodology of ALC were originally made from studying the distribution system from the plans, it was quickly realised that the ALC methodology had to be different for each area worked in to obtain best results from the equipment; this was mainly dependent on pressure available, material of pipe work, background noise level and availability of fittings.

In this paper information is available for pilot areas 1, 2 and 5.

Area 1

The leakage model indicated that there might be up to 5 Equivalent Service Pipe Bursts (ESPBs). (After adjustment for pressure)

For area 1 it was decided that all fitting should be listened on for audible leak noise however it was quickly realised that there was a distinct lack of visible service connections to enable ALC to be satisfactory completed, it can be assumed that a number of small leaks may have been missed.

The noise transfer from the leaks within the distribution system was very poor and can be accredited to the service pipe and mains material which was in certain areas found to be polythene; the quantity of polythene pipe in this area is unknown.

A leaking service was located by listening on the ground close to the leak and was clearly auditable however when listening on the water meter direct .5m away from the leak position the leak noise was barely audible, when this leak was excavated it was found that the service pipe was black polythene.

It was decided that the only feasible way of detecting all leaks in a situation like in Area 1 will be to carry out a comprehensive ground microphone leak detection survey listening to all distribution mains and service connections keeping the distances between contact points as short as possible.

Any other acoustic methodology may be unsuccessful due to the distinct lack of fittings be it either distribution mains or service connections.

Area 2

The leakage model indicated that there might be up to 19 Equivalent Service Pipe Bursts (ESPBs). (After adjustment for pressure).

This was deemed as the oldest and the most problematic of the 5 chosen areas this being for its age, service connection material, mains material and usage.

The methodology of ALC in this area was to listen on all services with a listening device of both electronic amplified and non electronic. This method was chosen as the deemed approach as there is only a small number of mains fitting available and a large number of suspected leaks in the area, it was not known at this early stage if the leaks were present on service connections or distribution mains.

Noise loggers were deployed on the valves that were present within the distribution system and with success as they indicated an area where a leak may be present, this area was investigated and a leak was located and repaired. A greater success would have been possible if there were more valves present.

A large number of leaks were found in area 2 between that of the communication pipe and supply pipe. The majority of the property/private (supply pipe) side leaks located were that of open taps (cannot turn off), leaking joints, homemade repairs or just general deterioration of the galvanized iron pipe work.

Nearly all the leaks that were found on the communication pipe work were that of deterioration of the galvanized iron pipe work or on joints. See picture 1.



Picture 1. Galvanized Iron leak Hanoi 2008



Picture 2. Reinstatement Hanoi 2008

It was noted that on some occasions when repairs had been completed the quality of the backfill/reinstatement was not always a consideration and in some instances sharp stones were placed upon the new polythene pipe work, this is a pipe failure waiting to happen. See Picture 2.

General findings

The length of the communication pipe is some instances exceeded 30m + (from the main to the 1st meter connection) of which the water company holds responsibility; it is on these lengths that illegal connections are easy to make and un-metered water would be taken into a property.

Although all the properties are metered many of these are hidden in positions that were impossible to locate unless the house holder shown the location, some of these meters may not have been read for a considerable period of time.

Property leaks were not being repaired to a satisfactory standard and the common approach is to bandage the leak with either cloth or tape to prevent the leak from spraying.

Area 5

The leakage model indicated that there might be up to 5 Equivalent Service Pipe Bursts (ESPBs). (After adjustment for pressure). It was decided that noise logging would be the best method of ALC in this area.

The noise loggers were deployed and left in situ and downloaded each morning to view the reports of where leaks may be present. On each download occasion a leak position was clearly indicated on the sluice valve closest to the water meter, this was thought to be the meter causing the noise and was ignored on the 1st night reading. The 2nd night this was much the same and was investigated during the early hours of the morning to see if a leak is present or not.

A leak position was located with the use of the electronic listening device and the ground was marked for excavation. The following day the ground was excavated and a leak was located and

repaired on a 63mm polythene service/main, several excavations had been made in this vicinity in the past as water was on occasions seen to be running from the ground causing problems.



Picture 3. 63mm burst



Picture 4. 63mm after repair

The position was logged again with the noise loggers and the valve was shown to be quiet and no other leaks were present, this was confirmed on the flow data obtained.

No other ALC method was used in this area i.e. sounding of services

Leaks located

A total of 21 leaks on service connections were reported and repaired. In addition to that, an even larger number of smaller leaks before and after the customer meter have been found some of these were so minor that repairing of them was deemed un-economical but 12 were deemed large enough to report for repairing.

Quantification of leaks has been tried but with the low level of confidence in the flow or DMA data this may not be totally accurate

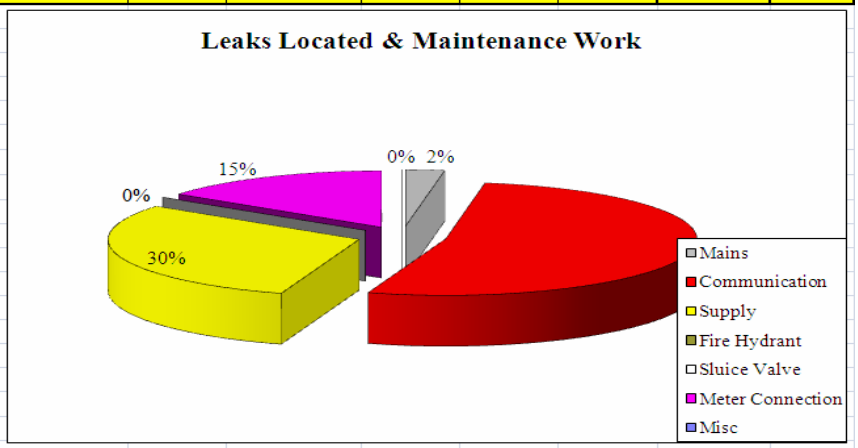
However, the large number of leaks found in three small areas of the Hanoi water distribution system (representing less than 0.5 % of the system) was prove enough that there is indeed a leakage problem.

Compliance Zone		All performance			Name					
DMA					Date	2007				
No of Properties					Sheet No	1 of 1				

Distance Km	Mains Fittings	Property Connection	Properties	Mains	Communication	Supply	Fire Hydrant	Sluice Valve	Meter Connection	Misc
7.5	59	2812	3412	1	21	12	0	0	6	0

KPI Start		0
KPI End		0
Water Saved (l/s)		0
Total Cost of Survey		£
Cost of Repairs		£

Total hours	84
Noise Logging / Inspection/ Misc	40
Total hours Leak Detection	44
Total leaks	40
Props per leak	85.30
Man hours per leak	1.10
Fittings per man hour	34.18
Connections per Leak	71.78
Kilometers per leak	0.19



Communication pipe is from the main to the stop tap or meter

Supply is from the stop tap or meter to the house

Graph 1. Overall performance data Hanoi pilot areas study 2007-08

4. CONCLUSIONS

Leak Detection – Experience Made

The most important conclusion of the trial leak detection campaign is that it is possible to detect leaks in the Hanoi water distribution network using traditional methods and equipment. It should be noted that the trials were conducted by a fully trained engineer and it cannot be assumed that the same results could be obtained by a non trained member of staff.

This trail has demonstrated that when using common technology and an experienced engineer leakage detection can be achieved in areas where the night time pressure does not reach 10m.

However experiences from this trial shows that not all techniques work in every situation and that the correct method of approach should be used to gain maximum benefit.

Visual Inspection

In the areas with poor infrastructure condition the majority of leaks that were not visual during the day time become visible during night hours (between midnight and 04:00). They might either be visible on the surface or by looking into chambers of all kind of underground utilities.



Picture 5. Visual leak during night time hours

Regular detailed night inspection by experienced and motivated staff would help to reduce the backlog of undetected leaks.

Non-electronic Listening Devices

Simple and cheap non-electronic devices i.e. listening stick or the stethoscope, are very useful to do sounding on service connections. This type of equipment can be used by the experienced and non experienced staff when looking for leaks, this simple listening device can also be used when sounding the surface for the pinpointing of a leak as the services in Hanoi are sometimes only .1m in depth.

It must be considered at this stage that the human hearing ranges from 20htz to 20,000htz at a young age and decreases as age sets in, noises from leaks can be as low as 1htz preventing the human ear from being able to hear the leak noise.

Electronic Listening Stick

Electronic technology much like then one used for this trail is very easy to use for both the experienced and non experienced engineer. The equipment used in Hanoi has a numerical display which indicated the level of noise heard removing the criteria to pinpoint using the human ear.

Ground Microphone

The ground microphone is normally mainly used for leak pinpointing – this means to finally decide the exact location of the leak (for excavation) after it has been localised by any of the other methods

Leak Noise Correlator

Leak noise correlation on soft walled pipes i.e. non metallic is generally not as effective as on hard walled pipes i.e. metallic pipes. Due to this problem many areas in the Hanoi distribution network, where there are (i) plastic pipes and (ii) a lack of access points to the network the use of correlators will be limited.

A successful correlation was experienced in Hanoi where the correlator was used to locate a leak on a 200mm PVC main over short distances maximum of 60m at 10m pressure during the early hours of the morning 2am – 4am, it was found that the leak was on an unknown illegal connection to a house and workshop.

Leak Noise Loggers

The presumably biggest surprise of the leak detection trial was that leak noise loggers worked well despite the extraordinarily low pressures. However, a serious problem for the large scale use of noise loggers is the lack access points to the network .

If access points are made available, i.e. fittings in chambers, valves underground hydrants etc, and then the wide use of noise loggers may become the preferred use of ALC. Permanently installed leak noise loggers must be considered, new technology now allows for remote reading of the loggers giving a 24hr report on the condition of the distribution system and the possibility of any leaks being present.

Correlating Leak Noise Loggers

The success of the correlating function of the correlating noise loggers is still questionable in the Hanoi situation. This is down to the type of leak that is currently being experienced which is the majority of communication and supply pipes and connections, until all these common leaks are dealt with basic noise logging function may be the most operable.

References

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REDUCTION OF WATER LOSSES CAUSED BY PRESSURE TRANSIENTS USING AIR VALVES

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Abstract: The demand for water is increasing due to population growth and technology development whereas water is becoming a scarce resource. The concern for water losses has grown to a global concern and there is an urgent need to find innovative solutions for water loss in water supply systems. Many water utilities ascribe their high seasonal burst frequencies to pressure transients. Pressure transients cause cracks and unseen small breaks in buried pipelines, pipe fittings and accessories and also can cause severe pipe ruptures and pipe bursts. These damages result in leakage and can bring to financial losses and to water shortage at times of scarcity. Control of pressure transients can greatly reduce the formation of these damages and consequent leakage. Recent studies have shown that presence of air pockets of certain sizes can intensify the strength of upsurges and produce a burst in the pipe. Entrapped air appears not only during the filling or draining of the pipes but also during the normal operation of the system. Entrapped air reduces the transportation capacity of the pipe producing additional head losses and thus energy losses. Absence of air can cause collapse of the pipe due to sub-atmospheric (negative) pressures. In some cases, the absence of an air cushion can increase the damages of pressure transients. Air valves are the most cost effective tools for the control of minor transients and in many cases for the control of major transients. They are used to ensure the release of the entrapped air accumulated during the normal operation of the system and the filling of pipes. They are also installed to suppress sub-atmospheric pressures by admitting air rapidly to avoid cavitation and subsequent column separation during drainage and system shutdown. By reducing negative pressure transients, air valves reduce positive pressure transients. Furthermore, well sized, quality combination air valves prevent the formation of vapor cavities and, thus, prevent extreme upsurges. Case studies conducted in Russia showed that air valves were found very effective in reducing water losses (pipe burst and leakage) by 15% and increasing the energy saving by more than 20%.

Key words: Water loss, pressure transients, air valves.

1. INTRODUCTION

Pipeline systems are usually designed assuming homogeneous construction and structural integrity of the pipes and accessories, assuring leak free, efficient operation. In real life, though, this is not always the situation. Sometimes, even a brand new pipeline may not be homogeneous, and some segments of pipe wall may not be uniformly thick and/or uniformly strong all around the pipe's circumference. This is especially true for aging pipes and pipes operated under severe and/or corrosive environments. Actually, many pipelines suffer from cracks and faulty joints, seals, and gaskets. Most of these gaps go unnoticed until they result in severe pipe ruptures and pipe bursts. In the Professional and Technical Resources section of the American Water Works Association's website, in a chapter titled "Apparent and Real Losses", it is stated that: " Many drinking water utilities around the world respond to leaks only after they receive a report of water erupting from a street or a complaint from a costumer about a damp basement" (AWWA 2008). The transition from a small gap to a noticeable pipe rupture or burst, may take many years, while the pipeline leaks. Leakage of conveyed liquids out of the pipeline can bring to financial losses and to water shortage at times of scarcity. In potable water transmission, loss of often-expensive treated water results in loss of revenue to the Water Company or municipality. This loss, at times of drought, can result in shortage of drinking water.

2. WATER LOSS

With increasing global concern for water losses, the International Water Association (IWA) and the American Water Works Association (AWWA), like other prominent international organizations, appointed special committees of experts to review the problems related to water losses and recommend courses of action and best practices. The IWA, with the participation of the AWWA, developed an international methodology for managing water losses. In the methodology, water supply is divided into two officially named categories, Revenue Water and Non- Revenue Water, where Non Revenue Water includes unbilled authorized consumption and water losses. Water losses are divided into apparent losses and real losses. The category of Real losses includes a sub-category of "Unavoidable annual real losses", UARL. These are considered economically acceptable losses, since it is concluded that the benefits of their repair do not justify their cost. In the official IWA format audit for the city of Philadelphia, PA for the 2002 fiscal year, of the total 995,834 m3/d of water supplied, 261,981 m3/d, 26.3%, were classified as Real losses, and 20,064 m3/d, 2%, were classified as UARL (Kunkel 2003). In rural Pennsylvania, as in other rural areas in the U.S., the situation is even graver. John Hood, the executive director of the Pennsylvania Rural Water Association (PRWA) states that small systems can be loosing as much as 50% to 70% of their water. Susan Obleski of the Susquehanna River Basin Commission expressed her concern that up to 70% of losses in water distribution systems can be attributed to leaks (Burt 2007). The Water Loss Control Committee (WLCC) report and the work done by the IWA and the AWWA don't actively deal with control or prevention of water losses. It does touch on one important active measure. When developing the district metered area (DMA) concept for monitoring purposes, it was apparent that pressure has a great influence on the intensity of existing leaks. This lead to investigating pressure management strategies using DMAs, such as lowering pressure at times of low consumption and raising it only at high consumption (Kunkel 2003). As more and more people become concerned about water losses, municipalities and utilities become more involved in surveying the extent of the problem and in taking actual steps in reducing these losses, but, none of them deal with the prevention or limitation of the leak causing gaps in the pipelines.

3. THE CAUSES FOR PIPELINE DAMAGES

There are many causes for pipeline damages resulting in leakage. There are natural causes, such as soil movement, soil erosion, extreme temperature changes resulting in damaging expansion and contraction of pipelines, freezing of water inside the pipeline exerting internal pressure due to the expansion of ice, etc. Granular materials and aggressive hydraulic regimes (high velocity flow, etc.) can bring to physical erosion of pipeline walls. Vehicles, digging equipment (backhoe, etc.), and other implements of everyday activities can cause external physical damage. Aggressive biological, chemical, and electrical environments cause corrosion and erosion, weakening, pitting, fissuring, and/or breaching in pipe walls. Potable water containing dissolved oxydants, such as oxygen and chlorine (dissolved oxygen is always required in potable water and a minimum residual chlorine concentration is, also, usually required). When in contact with metallic iron, these oxydants present a driving force for active corrosion. It is claimed that corrosion rate is probably limited by the rate at which oxygen (that comes out of solution) is provided to the surface (AWWS 2002). Hydraulic pressure transients (surges) present one of the most prevalent causes for pipe damage, including pipe bursts, pipe collapse, pipe cracking and pitting, leakage and intrusion. Damages brought about by hydraulic transients are even more prevalent and more severe, when the pipeline or pipeline sections are already weakened by events or processes mentioned above.

4. HYDRAULIC PRESSURE TRANSIENTS (SURGE)

Pressure transients can be described as waves possessing both, positive and negative amplitudes. These transients are caused by sudden extreme changes in flow velocity brought about by events such as pipe bursts, sudden changes in demand, sudden pump start-ups and shut-offs, opening and closing of fire hydrants, quick closing and opening of in-line isolating valves, flushing and draining operations, fire flow, feed tank draining, and other similar events.

The transients that generally attract most concern are the extreme upsurges that cause severe damages to pump stations or to pipelines. Customarily, transient analysis focused almost entirely on preventing disastrous failure at pump stations and their connecting distribution pipelines. Less attention was given to transient occurring in distribution systems and minor pipelines. Surge control was focused mainly on extreme upsurges and on preventing pipe bursts by reducing maximum pressures. Not enough emphasis was given to negative pressure transients. Pipe collapse was attributed to "vacuum", which was not always recognized as a hydraulic transient event or a down-surge. Great emphasis is given to pressure surges at pump stations because of economic consequences of these extreme transient events. Pumps and other equipment in the pump station are expensive, as are consequent damages they may incur under severe transients. But, there is another reason why some water system operators focus their attention to the pump station - the misconception that a surge event is restricted to its source. If a surge event was initiated by a pump trip, for instance, some operators believe surge will affect only the pump station. This, of course, is a fallacy. Pressure transient waves, with their positive and negative amplitudes, propagate from their source (pump, closed valve, etc') to the water transmission system. Some of these waves can reflect off different obstacles along the pipeline, such as fittings, joints and accessories, and propagate back to the source. When two or more waves of similar amplitude (positive or negative) meet, the intensity of the resulting pressure (positive or negative) at the point of encounter is equal to the sum of the pressures of the individual waves. So, it is possible, for the intensity of transients at locations along the pipeline, to be greater than at their source.

5. WATER COLUMN SEPARATION, DOWN-SURGE, AND VAPOR CAVITY

Most pressure transient events are accompanied by water column separation. When a pump shuts off, or when an isolating valve is closed rapidly, water supply down stream stops, but the water continues to flow away from the pump or valve, driven by inertia. The water flowing away from the pump or valve is referred to as a "water column" and this phenomenon is called "water column separation". Water column separation also occurs when instantaneous demand is greater than supply and when there is a sudden flow velocity increase, such as at pipe burst or pipe drainage.

Already in 1900, Joukowsky, who proposed the law for instantaneous water hammer in a simple pipe system, expressed in the Joukowski Equation ($\Delta H = aV_0 / g$), was the first to observe and understand column separation (Bergant 2006).

At water column separation, if nothing replaces the parting water column, leaving a void behind, pressure drops and a down-surge occurs at the point of separation. When pressure drops below vapor pressure of the liquid, some of the liquid vaporizes, filling the void with vapor. The void is now called a "vapor cavity". A complete water column separation is not a prerequisite for vapor cavity formation. When negative amplitudes of pressure waves meet, and their total intensity is greater than the local head, resulting in a local pressure drop below the vapor pressure of the liquid, a gap and vapor cavity will form. The gap does not have to occupy the full diameter of the pipe, and it can be located anywhere along the pipeline (Bergant 2006). As mentioned before, a transient event includes upsurges and down-surges. Which occurs first, depends on what initiated the transient event. If the transient event is initiated by a pump shut-off or by a quick closing valve (looking down-stream from the valve), the transient event will commence with a down-surge. If we look upstream from a quick closing valve, the transient event will commence with an upsurge, as

the water column slams against the closing valve. After the formation of a vapor cavity, when the water column and/or an upsurge transient wave return, raising pressure back to above vapor pressure, the vapor changes phase back to liquid. This is called “vapor cavity collapse”. A hydraulic pressure transient event that includes the formation and collapse of a vapor cavity is called “cavitation”.

Hydraulic transients in pipeline systems can be divided into two distinct flow regimes, the water hammer regime, where cavitation does not occur, and the cavitation regime, where cavitation does occur. The classical water hammer equations are not valid in regions of cavitation (Bergant 2006). In a cavitatin regime, when cavitation occurs, the consequent upsurge is much higher than in a regular upsurge, resulting in a water hammer regime. Joukowsky did not use the term “vapor cavity”, but he realized that in a transient event commencing with the slamming of a water column against a closed valve (gate), the second upsurge is greater than the initial upsurge. In Joukowsky’s words: “The condition, that water column is separated from the gate, prolongs the duration of the reduced pressure and makes the second impact stronger than the first, because it takes place with the velocity, at which the liquid column speeds into the rarefied void.” (Bergant 2006).

6. PIPE DAMAGE FROM PRESSURE TRANSIENTS

If a pipe is new and its walls are of uniform thickness and strength, at a very extreme down-surge, the pipe could collapse to look like an open channel (see Figure 1, 2).



Figure 1. A steel pipeline, newly laid above ground (in India), collapsed when a drainage valve was opened for flushing, too fast.



Figure 2. Pipe collapse in Venezuela due to down-surge.

In many cases, a pipe’s walls are not uniformly thick and strong around its circumference. Because of manufacturing faults or because of corrosion, erosion, wear, etc’, quite often, certain areas on the pipe wall’s circumference are thinner and weaker than the rest of the circumference. At

water column separation and/or down-surge, the weaker area of the pipe wall will buckle inwards. At column return and/or upsurge, the same weak area will buckle outwards. Pressure transients will usually recur again and again in the same points in the system. With recurring down-surges and upsurges, resulting in inward and outward buckling, the weaker areas on the pipeline will crack longitudinally, along the weak section of the pipe. After continuously recurring transient events, and this could take days, months, or years, the pipe will eventually rupture. These transients do not have to be extreme. Very slight transients can, over long periods, or even shorter periods of time, induce this pipe deterioration process.

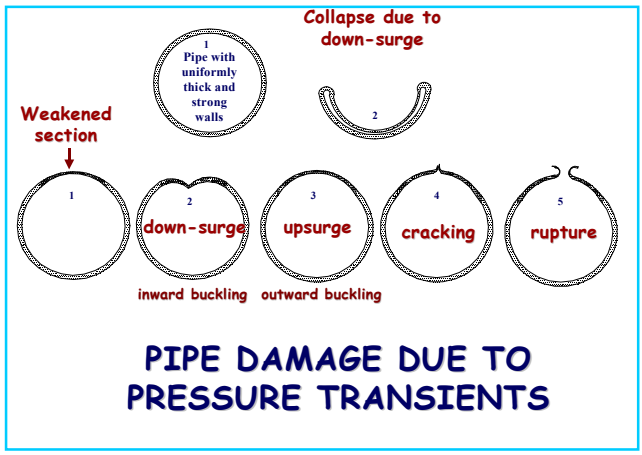


Figure 3. Pipe reaction to pressure transients.



Figure 4. 48" water main that ruptured at 5th Avenue and 19th Street in New York City, at 2 AM, January 2nd, 1998, the night following New Year's Day (DEP).

7. TOOLS FOR RESTRAINING AND CONTROLLING PRESSURE TRANSIENTS

There are many types of transient restraining and/or control tools available today, from the classic elevated tanks and flywheels to the most advanced pressure control valves. Some of the more common tools used today are flywheels, surge control valves, surge anticipators, surge towers, surge tanks, air vessels, air chambers, surge shafts, air valves, and more.

But, most of these tools have restrictions in their applications. Some, like the flywheel are restricted to use at the pump station, some require external power (electricity, compressors, etc'), some, like the hydraulically controlled surge control valves, require a clean water source, some, like the pressure anticipating valve, require a drainage receptor, some are very large and cannot be installed in all required locations in the distribution system, most of them are expensive or very expensive. Some of the solutions above are overkill for minor transient control. Air valves are the most cost effective tools for protection against damage from most of the common everyday pressure

transient events, combined with other surge protection tools, in order to enable reduction in size and/or cost (air vessels, surge tanks, etc').

8. AIR VALVES FOR REDUCING AND CONTROLLING PRESSURE TRANSIENTS

There are three major types of air valves but all are sometimes, wrongly called "air release valves". AWWA standard C512-07 distinguishes between the three types of air valves, calling them: Air-release valve, Air/vacuum valve, and Combination air valve (AWWA 2008).

The air/vacuum valve, sometimes called "large orifice air valve" or "kinetic air valve", has a large orifice that discharges air at pipe filling and at water column return (following water column separation), and through which air enters the pipeline at pipe draining, at water column separation, and at sudden, extreme pressure drops. Because of the size of the orifice, this type of air valve will not open when the pipeline is full and pressurized. It will only open when internal pipe pressure is lower than the external pressure. Because of its high capacity for air intake, this type of air valve is essential for down-surge control, and is sometimes called "vacuum breaker".

The air-release valve, sometimes called "small orifice air valve", or "automatic air valve", has a small orifice that is capable to open even when the pipeline and air valve are under pressure. The air-release valve releases small volumes of air that accumulate in its body, thus preventing the accumulation of air bubbles to air pockets in the pipeline. When properly located and assembled on pipeline, air release valves can prevent any air from accumulating.

The combination air valve, sometimes called "dual purpose air valve" or "double orifice air valve", combines the functions of the air/vacuum valve and the air release valve either in one body or in two bodies connected together, with a single pipe connection. This is the most commonly used type of air valve since in most cases both functions are required in the same location.

In addition to the vacuum breaking aspect of the air/vacuum valve and the air/vacuum component of the combination air valve, and their protection against pipe collapse and siphonage, there are other aspects to the air/vacuum operation that may be somewhat less familiar. At water column separation the air/vacuum orifice performs two major, very important, functions. Firstly, it allows a large amount of air into the pipeline, very quickly, in the region of separation. This air replaces the displaced water, preventing the formation of a void cavity. Secondly, the air/vacuum orifice exposes the region of separation to the atmosphere, discontinuing the pressure drop, and balancing pressure to atmospheric pressure. In order for the air/vacuum orifice to open, there must be a pressure drop, however, both air/vacuum functions above limit the intensity and the duration of the pressure drop (down-surge), and prevent harmful cavitation.

Fleming et al. (2006) survey different studies and concluded that even when air valves are not necessarily located in all critical locations, their presence reduce the number of negative nodes in the distribution system greatly. In one system, installation of just five air valves reduced negative nodes in the system by 40%.

Since a pressure transient is a wave, propagating down and up a pipeline with negative and positive amplitudes, by restraining the negative amplitudes (down-surges), the air/vacuum orifice also aids in curbing the positive amplitudes (upsurges). Furthermore, since cavitation amplifies upsurges to intensities much greater than normal Joukowsky surge, prevention or restraint of cavitation reduces or prevents consequent upsurges to a far greater degree.

Lingireddy et al. showed that reducing the size of the air discharge orifice can reduce or even eliminate (when sized correctly) the "air slam" or upsurge. Referring to two simulations performed, they report: "Both examples show that an outflow orifice smaller than the inflow orifice is desirable to alleviate undue secondary pressure surges due to final release of air." Figure 5 shows the results of surge analyses performed for one air valve with a 4 inch discharge orifice and a second with a 0.5 inch discharge orifice.

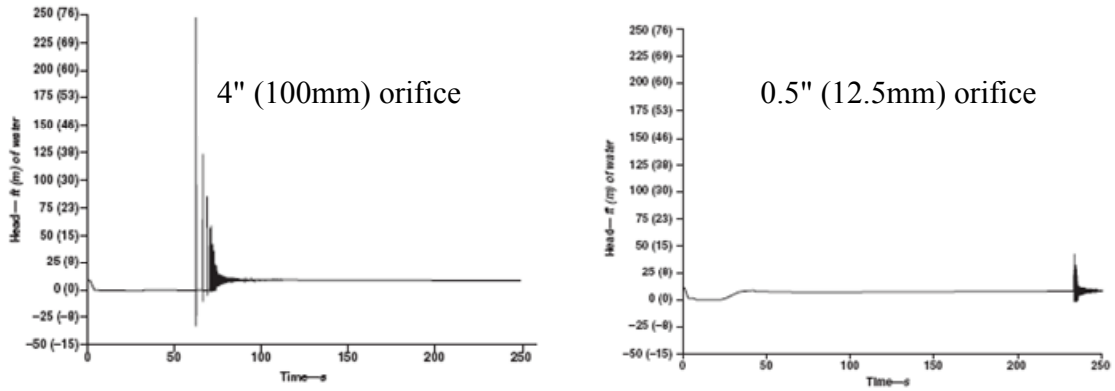


Figure 5. Surge analyses for secondary pressure surges due to final air release through 4 inch and 0.5 inch air discharge orifices (Lingireddy 2004).

Dr. Lingireddy's conclusions, based on lab tests, calculation and analyses, are that the use of air valves with large air intake capacities and much smaller air discharge capacities is highly desirable for ensuring effective and efficient surge control. Modern air valves with non-slam and three-stage accessories that utilize the principles expressed in Dr. Lingireddy's article have been readily available for a number of years. These accessories enable the use of nominal air valves for efficient down-surge control, and still allow non-slam, surge-suppressing closure. One such air valve is a three-stage, non-slam combination air valve that has a nominal air intake orifice for down-surge protection, and a throttling disc with a much smaller orifice, that throttles air discharge at a predetermined differential pressure across the valve. The throttling disc throttles air discharge, causing the remaining, slowly depleting air pocket, to slow down the returning water column and to dampen returning positive transient waves (upsurge pressure waves), by absorbing much of the energy, like a shock absorber. At final air valve closure, because the velocity of the returning water column is much lower, ΔV in the Joukowsky Equation is smaller, slam is prevented, and upsurge is suppressed (Zloczower 2007). Air valve technology has improved greatly in the past few years. An air valve with an aerodynamic rectangular float and an independent rolling seal mechanism was developed. It has slanted, rectangular orifices that combine the air/vacuum and air release components, allowing the breaking of the air flow 90 degrees to the vertical (not directly opposite to the inlet). All these features were designed for resisting premature closure, yet providing a more gradual, softer, slam preventing closure. A completely revolutionary dynamic combination air valve appeared in the market, with a float-less air/vacuum component. The freedom from dependability on a float for the operation of the air/vacuum component, affords this dynamic combination air valve a number of advantages, such as a short profile for large diameter air valves, a smaller body and lighter weight, and freedom from premature closure (the air valve remains open until all the air is exhausted). Besides being an air valve, with all the advantages afforded a regular combination air valves, the dynamic air valve also acts similar to a surge relief valve or a surge anticipating valve, in that it relieves the secondary upsurge (returning water column) by releasing a controlled amount of water.

9. CONCLUSIONS

This paper highlights international concern for water losses due to leakage of water from pipelines. The main cause of leakage are pipe breaks, cracks, dislocated and damaged seals, gaskets, joints, fittings, etc', generally referred to as "gaps". It was established that pressure transients, both major and minor, are a major cause for the creation of new gaps and for the expansion of existing gaps. It was further established that internal corrosion, enhanced by the presence of air pockets,

weaken pipelines, and is a major contributor to the creation of new gaps and for the expansion of existing gaps. In addition to steady state pressure in pipelines, the upsurges of transient events are a major driving force for leakage. Cavitation (the formation and collapse of vapor cavities) was shown to greatly amplify and intensify pressure transients. Amplified pressure transients quicken the creation of gaps, create larger gaps, and greatly enlarge existing gaps. Amplified upsurges greatly strengthen leakage



Figure 6. 1: A 6" dynamic combination air valve for water; 2: Non-slam, three-stage combination air valves for water; 3: A 3" stainless steel non-slam combination air valve for wastewater.

It is shown that today, a wide range of modern, innovative air valves are readily available. This paper sights studies, analyses, and advanced experiments that prove that air valves are reliable and efficient tools for the alleviation and restraining of pressure transients. Air valves discharge existing air pockets and prevent the accumulation of new air pockets, thus preventing air enhanced corrosion and pressure transient intensification. It was shown that air valves prevent cavitation and the consequent amplification of pressure transients. Air valves prevent or curtail the creation of new gaps and the enlargement of existing gaps, thus preventing or restraining leakage.

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WIRELESS SENSOR NETWORKS FOR WATER LOSS DETECTION

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Abstract: Cyprus is facing a growing water shortage problem, therefore water losses associated with inefficiencies of the underground water distribution system are not only causing revenue losses for the local water boards but also significantly negatively affect the national water reserves. This paper presents an outline of two research projects, related to each other, "WATERSENSE" and "URBAN WATER DISTRIBUTION MODELLING, SIMULATION AND OPTIMIZATION OF LEAKAGE DETECTION VIA SENSING TECHNOLOGIES" (UWDM). These projects aim the development of an integrated management system for water distribution networks that combines analytical and neurofuzzy decision support systems, geographical information systems and wireless sensor networks. The methodology focuses on the sustainable management of water pipe networks through real-time data acquisition and processing of sensor signals collected from a distributed ad-hoc wireless sensor network.

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Key words: water losses, network management, optimization, sensing.

1. INTRODUCTION

According to the International Water Association's Water-Loss Task Force the four basic leakage management activities for leakage reduction are (i) pressure management, (ii) active leakage control, (iii) speed and quality of repairs and (iv) pipeline asset management, maintenance and renewal (Lambert and McKenzie 2002).

The main problem faced by the owners of urban water distribution networks is the deterioration of their networks because of aging and the consecutive repairs of the network. Their primal target is to manage leakage losses, maintenance of pipelines and networks, and renewal of networks. That's why life-cycle costing and maintenance strategies are very useful and of great importance to the agencies.

In order to be able to monitor an urban water distribution network for localization and quantification of leaks, parameters such as water flow, pressure, soil and air moisture, soil and air temperature, rain measurer and acoustic noise loggers should be collected directly from sensors embedded in strategic places within the network. The wide area and complexity of an urban water pipe system, the cost related to the deployment of a wired sensor data acquisition system as well as the time which is needed for such systems to be deployed, have for years prohibited the investigation of multi sensor systems. Recent advances in wireless sensor networks (WSN) (Akyildiz et al. 2002) provide researchers with an easily deployable, scalable, flexible and relatively inexpensive approach for real time distributed data acquisition and monitoring. Ad-hoc wireless sensor networks are self-organized networks composed of a large number of sensor nodes that interact with their environment and communicate in a wireless fashion. Their target is to transfer their processed data to a remote base station. The utilization of a WSN to implement the underground monitoring system proved to be benefited from rapid and flexible deployment. The WATERSENSE decision-support and management system is based on the continuous processing of

the received WSN signals, forms an integral part of a leakage management strategy and providing a useful decision making tool.

2. STATE OF KNOWLEDGE

2.1 Leakage detection technologies

Evolution of water distribution system leak detection technologies has been very fast during the last two decades (figure 1). The basic technologies used nowadays are:

- Ground Penetrating Radar,
- Optical Sensors,
- Moisture Sensors,
- Sound Sensors,
- Pressure Sensors,
- Robotic Cameras.

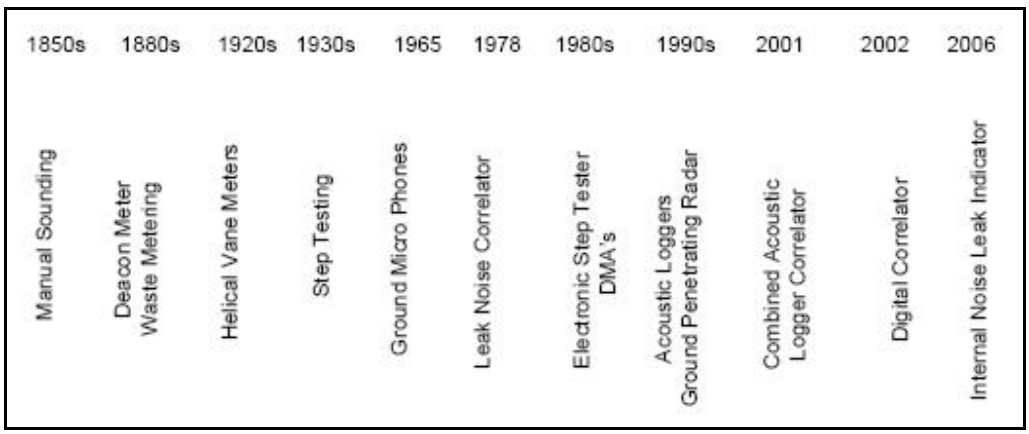


Figure 1. Evolution of leakage detection technologies. (from Pilcher et al. 2007)

Further to the economical benefits offered by the several leakage detection technologies, such technologies also help in: (Lahlou 2001)

- Improving the management of water distribution networks,
- Reducing losses of water,
- Reducing network damage and deterioration,
- Reducing the risk of infection in the water system,
- Improving the relation between customer and local water distribution services.

2.2 Assessment & Deterioration Modelling

To-date a number of studies have been undertaken on infrastructure assessment and deterioration modeling with respect to presumed risk factors. The studies usually attempt to identify statistical relationships between water main break rates and influential risk factors such as a pipe's age, diameter and material, the corrosiveness of the soil, the operating pressure and temperature, possible external loads (including highway traffic) and recorded history of pipe breaks.

Most studies in literature show a relationship between failure rates and time of failure (age of pipes), and some of them suggest a methodology to optimize the replacement time of pipes. Shamir and Howard (Shamir and Howard 1979) reported an exponential relationship, and Clark et. al. (Clark et al. 1982) developed a linear multivariate equation to characterize the time from pipe installation to the first break and a multivariate exponential equation to determine the breakage rate

after the first break. A study by Andreou et al. (Andreou et al. 1987) suggested a probabilistic approach consisting of a proportional hazards model to predict failure at an early age, and a Poisson-type model for the later stages, and further asserted that stratification of data (based on specific parameters) would increase the accuracy of the model. A non-homogeneous Poisson distribution model was later proposed by Goulter and Kazemi (Goulter and Kazemi 1988) to predict the probability of subsequent breaks given that at least one break had already occurred. Finally, Kleiner et al. (Kleiner et al. 1998, Kleiner 1999) developed a framework to assess future rehabilitation needs using limited and incomplete data on pipe conditions. More recently, a simulation model was applied to an inventory of water mains in New York City to analyze replacement strategies, and Vanrenterghem (Vanrenterghem 2003) developed models for the structural degradation of urban water distribution systems based on data from New York City. Additional work on the same case study was reported by Aslani (Aslani 2003) and Christodoulou et. al. (Christodoulou et al. 2003). The knowledge gained by the New York City case study was furthered and reported upon by Christodoulou et al. (Christodoulou et al. 2006, Christodoulou et al. 2007) in a developed framework for integrated GIS-based management, risk assessment and prioritization of water leakage actions.

Almost all of these studies, though, rely heavily on post-processing of leakage data, and very little has been done on wireless sensor technologies for real-time monitoring of the infrastructure.

3. WIRELESS SENSOR NETWORK SYSTEM ARCHITECTURE

3.1 Nodes Network

The goal of the current study was to investigate various sensing technologies and implement them in wireless sensor networks, as a start of an integrated water resources management tool (IWRM). To that extent, moisture, sound and pressure sensors were utilized and interconnected using WSN. At first, an experimental setup was used at the premises of the University of Cyprus (WATERSENSE lab).

The WATERSENSE lab consists of four of wireless nodes placed at specific locations along the experimental water distribution system model, to collect and reliably transmit sensor data to a base-station which is located within the lab area. Figure 2 represents the wireless sensor network system concept.

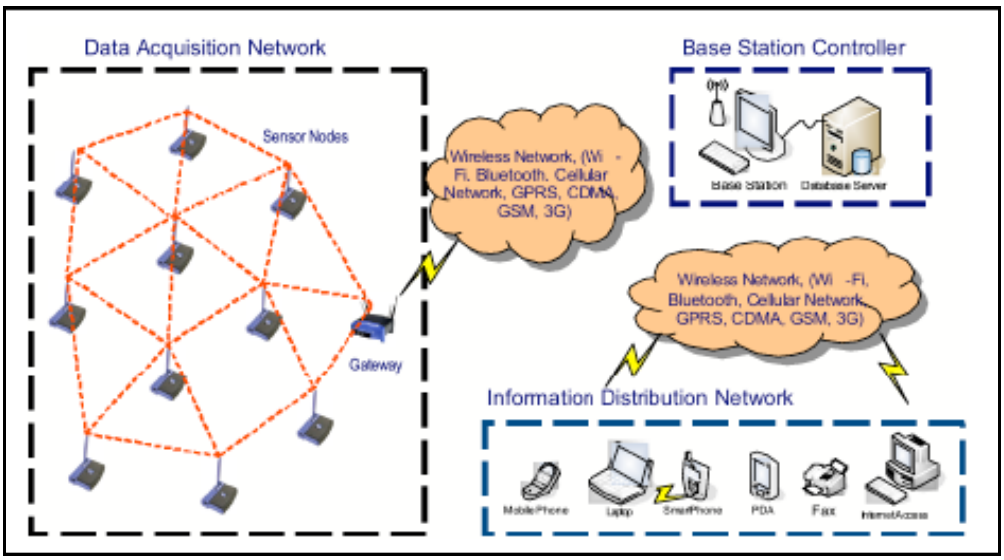


Figure 2. WATERSENSE Network Architecture.

The system is based on the Mica2 433 MHz motes (Mica) enclosed in waterproof package for outdoor monitoring. Soil volumetric water content is measured by the Decagon Ech$_2$o dielectric

sensors (Decagon) attached to sensor nodes using an MDA300 data acquisition board (Mica). Soil Moisture is measured from various locations at 0.5m and 1.0m below the level of the water pipe network. Water flow is measured using the widely deployed KENT V100 (PSM) meters (Elster) while pressure transducers from Honeywell were modified so as to be integrated to the MDA300 data acquisition boards. Finally, Zonescan 800 leak noise logger from Gutermann is used for leak noise detection.

The TinyOS operating system installed at each mote was used as the software platform for data acquisition, processing and efficient low-power communication (TinyOS). The communication of the motes to the base station provides the functionality equivalent to a data logger where the data acquired from all sensors are stored in a specifically designed database. The motes communicate with the Stargate (Hunaidi and Chu 1999) gateway which is responsible for sending the acquired data to the remote base-station via a GPRS link. The Stargate gateway is also used to update the configuration of each individual sensor mote giving the ability to modify the configuration of the whole WSN if needed. Parameters that can be adjusted include data-acquisition sampling rate, sampling duration, period of sampling and sleep time. The majority of the parameters measured by WATERSENSE such as pressure, flow and moisture entail low data acquisition and transmission rates while acoustic noise/vibration data acquisition need significantly higher data rates. For this reason, the WSN is designed in such a way so as to be easily adapted to the needs of the monitoring and management system.

Originally the WSN will operate in a low rate function acquiring data from all sensors except the acoustic sensors. When the system indicates a probability of a leak then it triggers the acoustic sensors which are used for the localization of the leak. Standard correlation algorithms Hunaidi and Chu 1999) are used for the locating the leaks within the infrastructure water network.

The WISENSE wireless sensor platform is battery operated, so special interest is given towards the design of an optimal power management and energy conservation system. Research is currently concentrated on two directions; (a) the use of solar power to drive the Stargate gateway so as to be also deployable in remote locations and b) the incorporation of techniques to reduce the power consumption of the motes. For the latter, programmable sleep-time operation is used in which the motes wake–up at specific time slots, collect the required data from the sensors, transmit the data to the gateway and then go back to sleep. In addition data compression techniques will be studied so as to increase performance and decrease power consumption. The pilot deployment will serve as a test-bed for extensive testing of various operation scenarios so as to optimize the energy consumption at each mote. The final system is expected to be able to integrate battery and solar-operated motes and gateways according to the physical location of the sensors in the urban water distribution network.

3.2 Data Collection and representation

All data collected and stored on the Base Station Controller, through a Gateway Station. Data are classified on a data base and uploaded on a web page, specifically designed for WATERSENSE project. This page is accessible from all over the world and offers to its users the opportunity to check, on real time, the wireless nodes and in addition the condition of the water pipe network.

The webpage shows information for all inspection points together or separately. Users have access to check the graphic daily, monthly and yearly data representation, related with each sensor. Data such as soil temperature, air temperature, soil moisture, air moisture, pressure of the pipe network, water flow, and rain fall volume are represented. Figure 3 represent the WATERSENSE webpage layout.

3.3 Decision Support System

A multiparameter decision system based on artificial neural network and fuzzy logic is responsible for detecting and localizing leaks and other anomalies in water pipes and for producing early notifications and suggestions, which are then distributed to field engineers and maintenance technicians. Deterioration modeling and detection of leaks in water-pipe infrastructure is certainly a very complex task due to the non-linear behavior of the system. In order to overcome this, WATERSENSE is based on the processing of multi-parameter data such as water pressure and flow, acoustic/vibration signals, soil moisture and rainfall readings. Processing of acoustic/vibration signals in combination with soil moisture readings are used for detecting leaks while the analysis of pressure transients and flow enables early detection and localization of larger leaks.

Even though the project entails a large-scale deployment of the system in the city of Lemesos's water distribution system, the research team at the moment is concentrated on the development and field validation of a small-scale prototype deployment which resembles the operation of the town's water system. The purpose of the prototype deployment is to investigate the communication challenges imposed by the environment and to evaluate the system in terms of durability of sensors and sensor nodes under real operational conditions.



Figure 3. WATERSENSE Webpage Layout.

4. INTEGRATED WATER LEAKAGE DETECTION

The proposed integrated pipeline asset management system for the monitoring, rehabilitation and life-cycle costing of urban water distribution networks is in essence a knowledge-based system complemented with wireless sensors, analytical and numerical analysis tools and supplemented with a geographical information system (GIS). At the outset, the management system collects, analyzes, lumps and visually displays risk assessment metrics for every network component (or street level) so that the network manager user knows the risk of failure (i.e. water loss) for each area in the city (Fig. 4).

The proposed system encompasses:

- Wireless sensors for the collection of operating network parameters (pressure, flow, humidity, noise).
- Data on system characteristics (such as pipe diameter, length, material, installation date, zoning, etc.).

- Historical data on pipe break incidents (date of incident, response time and cost to repair/replace, number of previously observed breaks, reason for and classification of break incident, etc.).
- A statistical analysis tool for the analysis of pipe break incidents.
- An artificial neural network component for data pattern identification.
- A fuzzy logic processor, for the development of fuzzy logic rules describing the behaviour of the network.
- A risk assessment module (primarily a survival analysis module).
- A geographical information system (GIS) for visualization.
- A life cycle costing module for the aggregation of costs by area and pipe.
- A prioritization-of-work module.
- A data query and reporting system for the retrieval of needed information.

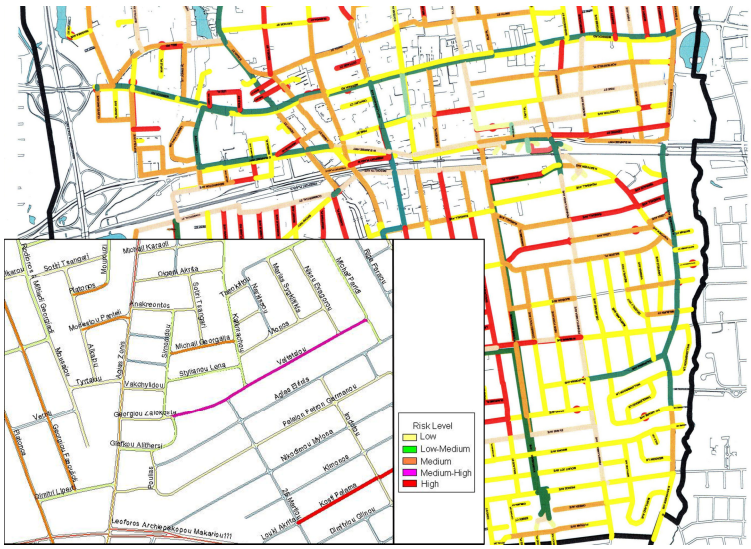


Figure 4. Example of GIS mapping of the risk- of- failure metric.

Historical data analysis is based on a number of analytical and numerical tools (such as statistical analysis, artificial neural networks and neurofuzzy systems) and investigates the possible contribution of a number of presumed risk factors to a “break or not?” outcome, ranking these factors according to their relative importance and contribution to the “Break or Not” forecasted output.

The analysis identified the most important risk factors to be the number of previously observed breaks (NOPB), the material type (MAT), the length (L) and the diameter (D) of each pipe (Christodoulou et. al. 2006).

5. DEPLOYMENT NETWORK

5.1 Strovolos Lab

The pilot deployment of the water distribution network is located at the Civil Engineering department’s laboratories in Strovolos. The simulator consisted of two pipe lines (one plastic and one galvanized) of approximate 70m length and 4 inches wide, which are connected to each other at their ends. The system has a reservoir and a water pressure system. Along the two pipe lines, mechanical check points (pressure, flow, noise) and inspection nodes (sensors) have been installed. Figure 5 is a picture of the pilot deployment at Strovolos labs.

5.2 Lemesos Water Board

The water distribution network for which the project is currently under development and implementation (Water Board of Lemesos) is over 50 years of age and serves approximately 170,000 residents through approximately 64,000 consumer meters in an area of 70 km^2. The annual volume of potable water distributed through the network of pipes, of approximate length 795km, is about 13.7x106 m^3 and of value €7.0 million. The network is divided into pressure zones, each subdivided into District Metered Areas (DMAs) having a single metered source with physical discontinuity of pipe network between DMA boundaries. The DMAs vary in size from 50 properties to 7000 although the average size being approximately 3000 properties. Distribution main diameters within the DMAs vary between 100mm and 250mm and where possible, interconnecting ring systems within the DMAs have been formed to minimize head loss at peak demands. The network owner (the Water Board of Lemesos) has maintained records of its operational activities since 1963, which include production of water from sources, distribution through district meters and consumption from consumer meters. Meter readings at water sources (boreholes and treatment plant) are connected via a SCADA telemetry system to the control room. This enables continuous monitoring of the water source outputs and accurate recording of flows. Likewise storage reservoir outlet meters are monitored on SCADA providing the same ability to observe trends as well as to record daily, weekly, monthly and yearly totals. The continuous monitoring of the DMA meters combines information technology and telecommunication networks to transfer the data via the World Wide Web. The historical data gathered in the programmable controller of each DMA is sent by the controller to an email account. Operating software installed in the dedicated computer at the Water Board's control room connects to this email account twice a week and downloads the data, which are first sorted according to the DMA and then are used to update existing reports.



Figure 5. Pilot deployment at Strovolos Labs.

6. CONCLUSIONS

The manuscript describes the framework of the WATERSENSE and UWDM projects; research projects aiming the development of an integrated management system for water distribution networks. The projects are partially funded by the Cyprus Research Promotion Foundation. The

WATERSENSE project has recently been completed and the UWDM is currently at the first year of its implementation. A pilot WSN deployed in a controlled environment (laboratory) is to serve as a test-bed for various communications and sensor signal-processing techniques that aim the improvement of the performance and the reduction of the power consumption of the network. The research findings will be important for the development of the final realworld deployment in the water distribution network of the city of Lemesos at the end of the second year of the project.

To-date, the research team has completed development of a GIS-based decision-support system (DSS) that maps the risk of failure for each segment of the pipe network based on rolling historical data, and analyzed by neurofuzzy systems. The developed DSS, which is currently beta-tested by the Water Board of Lemesos, is to be amended by the wireless sensor network for real-time processing and modeling of the underlying pipe network operational parameters (humidity, noise, pressure, flow) so as to increase the efficiency of the overall risk assessment and pipe management system and, most importantly, to reduce the amount of water losses in the network.

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Dams/Reservoirs

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DYNAMIC ANALYSIS OF INTERACTION OF GRAVITY DAM AND HYPER RESERVOIR IN FREQUENCY DOMAIN

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Abstract: Modeling of dam reservoir by finite element method have some problems. The most important problem is non-ability in modeling of whole of reservoir in finite element. That should cut the model in sufficient distance of dam's body and with applying a suitable boundary condition of wave absorbent in this area , we can prevent from returning the waves to the dam. In the research, it has discussed how the reservoir can model by ANSYS Software, And it has introduced the element which provides boundary condition of reservoir wave absorbent and it acts as Sommerfield condition and it has explained the way of its application. Also, the other present wave absorbent option in the Software that help to analysis, has introduced and provided the results obtained from its use in harmonic analysis in dam and reservoir.

Key words: finite element, dam reservoir modeling, Dam -Reservoir Interaction.

1. INTRODUCTION

After damages resulted from earthquake at early years of 70Am decades in some large dams, many of researches attended to seismic analysis concern of concrete dams by considering interaction effects in dam, reservoir and foundation stone, the concern has caused significant advances in related field and providing many strategies in due to accurate modeling of the action in this complex set. In this area, dynamic interaction analysis, because of more importance, has adopted significant volume of carried out researches [1,2,3,4], to doing the analysis in these 2 fields, there are different numeric ways such as finite element and boundary element [1,5,6], a often. The reservoir that placed in back of the dam has high length, usually, there assumed semi-infinite environment and seeks special strategies for analysis. It should be noted that one of the suitable ways for analysis of these environments, is application of semi-infinite elements or hyper elements. The elements have one, two dimensional natures for two or other dimensions problems; they model the materials action in meshed direction approximately and in orthogonal direction (that the element has a length) accurately and non-numerically. But, while these fluid semi-infinite elements have frequency-dependence natures, use of them in time-domain analysis is impossible. In ANSYS Software there are not the element that able provide HE (Hyper Element) condition for the system, wave absorption element from the Software. Have performance like Sommerfield condition that we will introduce Sommerfield condition in chapter2 and in chapter 3 we will provide wave absorbent elements of ANSYS Software.

2. MAKE THE RELATIONSHIP BETWEEN DAM AND RESERVOIR

Based on finite element theory, the matrix equation fore dynamic response of the structure to support excitation in time domain is shown as following:

$$[M]\{\ddot{r}\}+[C]\{\dot{r}\}+[K]\{r\}=-[M]\ [J]\{a_g\} \quad (1)$$

where M:mass index matrix, C:damping index matrix, K: structure stiffness index matrix, r: relative nodal displacements vector, J: unit matrix that is responsible for transferring support acceleration vector $\mathbf{a}_g$ to degrees of freedom of the structure it should be known that if a fluid like water placed into reservoir in back dam is subjected to support excitation, and equation similar to the above relation will be dominated on it :

$$[G]\{\ddot{p}\}+[L]\{\dot{p}\}+[H]\{p\}=-[B]\ [J]\{a_g\} \quad (2)$$

where G: pseudo mass index matrix, L: pseudo damping index matrix, H: structure stiffness index matrix, P: Hydrodynamic pressures vector, B: mapping matrix that causes the nodal acceleration transformation $\mathbf{a}_g$ to pressure flux and along with, J: matrix defined as before constitutes the right and side of matrix equation of fluid. When the structure and the fluid are placed along side and affected by the same base acceleration, fluid and structure interaction problem becomes important and it causes changes in responses of the both of these parts. In this case, the mapping matrix B, in addition of transformation of support acceleration to pressure flux, has some other duties as transforming the structure accelerations to pressure flux and also transformation of dynamic pressure to applied forces to the structure. In fact, the matrix causes interactions in the structure and the fluid. In this case, two equations dominate on the structure and the fluid are placed together and fundamental relation is made:

$$\begin{bmatrix}[M] & 0\\ [B] & [G]\end{bmatrix}\begin{Bmatrix}\ddot{r}\\ \ddot{p}\end{Bmatrix}+\begin{bmatrix}[C] & 0\\ 0 & [L]\end{bmatrix}\begin{Bmatrix}\dot{r}\\ \dot{p}\end{Bmatrix}+\begin{bmatrix}[K] & -[B]^T\\ 0 & [G]\end{bmatrix}\begin{Bmatrix}r\\ p\end{Bmatrix}=\begin{Bmatrix}-[M]\,[J]\{a_g\}\\ -[B]\,[J]\{a_g\}\end{Bmatrix} \quad (3)$$

Then, above relation transfers to frequency domain and use it as finding dynamic response in the dam, with regard to structure and fluid interaction effects. It should be mentioned that as in concrete dams, we use hysteric damping concept, damping matrix [C] defines as $[C]=(\frac{2\beta}{\omega})[K]$ and the relation changes to the following one after simplicity:

$$\begin{bmatrix}-\omega^2[M]+(1+2\beta i)[K] & -[B]^T\\ -[B] & \frac{1}{\omega^2}(-\omega^2[G]+i\omega[L]+[H])\end{bmatrix}\begin{Bmatrix}r\\ p\end{Bmatrix}=\begin{Bmatrix}-[M]\,[J]\{a_g\}\\ -\frac{1}{\omega^2}[B]\,[J]\{a_g\}\end{Bmatrix} \quad (4)$$

It should be said that above relation is complete for the dam and the reservoir analysis. But, we should enter infinity effect of the reservoir that causes the waves pass and damp in relation (4), if we require to analysis of semi-infinite reservoir that its length is much more than its other dimensions, here one of the best methods to entering the effect, is use of semi-infinite elements or big elements. In 2D reservoir, semi-infinite element way an are one dimensional network of limit elements that are orthogonal to direction and in fact, it has infinite length in direction of flow, Mathematically, the big elements by its own index matrix, indicates the relationship between pressure and its flux in upper boundary of the reservoir that continues to infinite, then it provides passing the wares on the boundary. There are other strategies that we can use them in end of the reservoir to absorbing the waves, it is condition Sommerfield condition. It (equation 5) is an approximate condition being of the boundary condition is because of this fact that the relation can absorb only the waves which are one dimensional that they have right angle with it.

$$\frac{\partial p}{\partial n}=-\frac{1}{c}\frac{\partial p}{\partial t} \quad (5)$$

In that relation, in above relation p , t And n are pressure, time and normal vector perpendicular to the surface respectively, In general, the continuum field equations for a flexible gravity dam fixed at its base, along with the appropriate boundary conditions can be summarised as follows:

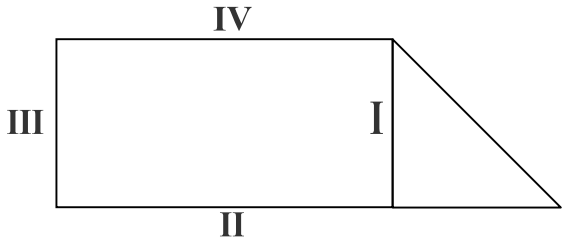


Figure 1: Reservoir Boundary conditions

$$\frac{\partial p}{\partial n} = -\rho a_g^n \qquad \text{Boundary condition I}$$

$$\frac{\partial p}{\partial n} = -\rho a_g^n - q\frac{\partial p}{\partial t} \qquad \text{Boundary condition II}$$

$$\frac{\partial p}{\partial n} = -\frac{1}{c}\dot{p} \qquad \text{Boundary condition III}$$

$$p = 0 \qquad \text{Boundary condition IV}$$

ANSYS Software uses the relations (4) to dynamic analysis when there is interaction between the dam and the reservoir to solving the equations. It should be noted that to modeling boundary condition of end of the reservoir in ANSYS Software we special element that it will explain in part 3. The element as Sommerfield condition creates the condition in end of the reservoir to absorbing the wave.

3. MODELING OF DAM AND RESERVOIR SYSTEM

To model the dam and reservoir system, we can use structural and fluid elements while the gravity dam analysis is two-dimensional, then in Software, it has used 2D elements. Therefore, in the ANSYS Software, for the dam, a PLANE82 element is used that has 8 nodes and for the reservoir, a FLUID29 element is used that has 4 nodes. When the dam and the reservoir join together, the middle node, PLANE82, remains open because it doesn't couple with the nodes from FLUID29 element. To prevent the error result from this action, freedom degrees of the nodes have coupled with upper and lower nodes. A complete model of the dam and the reservoir is shown in Figure (2). The damping is hysteric for the analysis and it is equal to 0.050. In this analysis we ignore the dam interaction with flexible foundation, and the foundation is taken to be rigid. The excitation has been applied as unit acceleration of mean 1m/s2 and it is applied to base nodes and in horizontal and right direction. The excitation is set to 20 Hertz in frequency domain and with expediential frequency step equals to 0.04. The concrete is assumed to be homogeneous and isotropic with the following basic properties: Elastic modulus Ec = 22.00 GPa , Poisson's ratio νc = 0.20 , Unit weight γc = 24.8 kN/m³. For the reservoir, the water is taken as compressible, inviscid fluid, with weight density of 9.81 kN/ m³ and pressure wave velocity of 1440.0 m/s and Input limit $MU = 0-1$. High is 200 meters and length is 600 meters means $L = 3H$ in reservoir of the dam. To model the boundary condition of wave absorbent at the end of the reservoir, we can use a FLUID129 element. The element is linear and has 2 nodes. The following four considerations are made: (1) the element was used only for a regional boundary condition that it models with FLUID129. (2) the end of the

reservoir should be the arc of the circle, (3) the center of the arc that element places on it, should place near the structure center and close to the FLUID129 element, (4) minimum distance of arc from center of structure should be as $R > \frac{L}{2} + 0.2\frac{c}{f}$, with L: structure length, C: speed of wave distribution in water and f: excitation frequency.

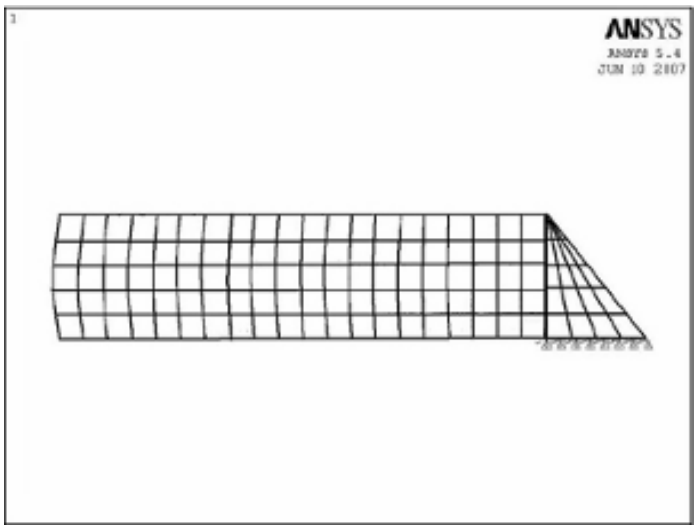


Figure 2: The reservoir and dam system model in ANSYS at horizontal excitation

In using the FLUID129 element, we should consider all of the above mentioned concerns. In cases where the dam models multi-sectionally, to applying accurate interaction, we should model the reservoir as the dam, and the arc of the end of the reservoir should divide and model equally. In this case, the end of the reservoir should be concentric and conradius. Attention should be given so as the material and modelled arcs are model as accurately as possible, otherwise the software can not do the analysis. In such event, in order to simulate the boundary condition at the end of the reservoir, an element other than FLUID29 should be used. Also, for an application of the wave absorption condition, we do not need that end of the reservoir be as the arc of the circle (in spite of essential condition for using FLUID129) and we can model the reservoir rectangle. Due to using the condition, we take MU=1 in the material properties section for water also to be 1 as the impedance to end line of the reservoir.

4. NUMERICAL RESULTS

After the dam and its reservoir with the mentioned specification were modeled in part 3, it was analyzed dynamically, with the interaction between the dam and reservoir and the interaction between dam and the foundation ignored. To show the results in the frequency domain we used transfer function graphs. In these graphs, the horizontal axis shows the excitation frequency. The axis has become non-dimensional by dividing the frequency by the first normal mode frequency that it was obtained from the modal analysis of the dam without the reservoir. The right axis indicates the response of crest to various excitations. As expected, the response is like acceleration. It should be mentioned that the responses are absolute values. Figure (3) and Figure (4) were selected as reference graphs and comparisons have been made relative to them. The response of the graphs were obtained from dynamic analysis, done by computer programming and by use of Sommerfield as boundary condition of absorbent of the wave in end of the reservoir. In Figure (5) and Figure (6) the horizontal response of the crown in the dam was analyzed for t horizontal and right excitations. The harmonic analysis of the dam was done by ANSYS Software and FLUID129 elements as boundary conditions at the end of the reservoir. By comparing Figure (5), Figure (6) with Figure (3), Figure (4) we can observe that the results from the analysis by ANSYS Software

and by using the suggested element are in good coordination to obtained results with Sommerfield condition. Furthermore, because of the presence of turbulence in the responses, the chosen element acts better than the Sommerfield condition. Figures (7-10) provide a response comparison between the ANSYS software. In these graphs, results from using FLUID129 elements are visible for two excitations with results obtained from activating the impedance option as boundary condition at the end of the reservoir. The comparisons can help in providing the way for optimizing the usage of the software in dynamic analysis in the dam and reservoir.

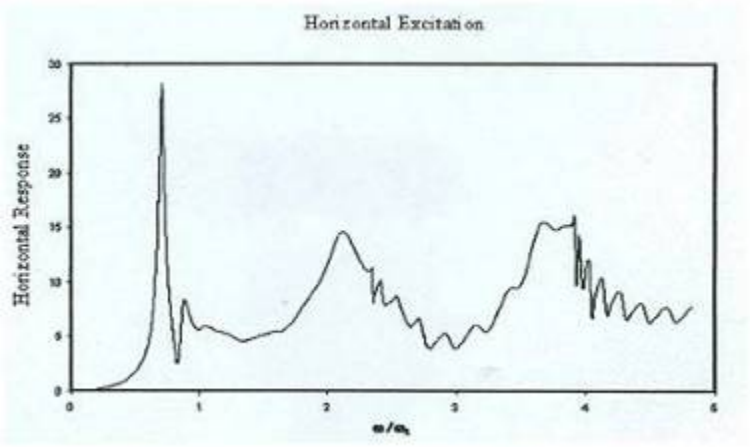


Figure 3: Horizontal the response of crest of the dam to horizontal excitation by Sommerfield condition[6]

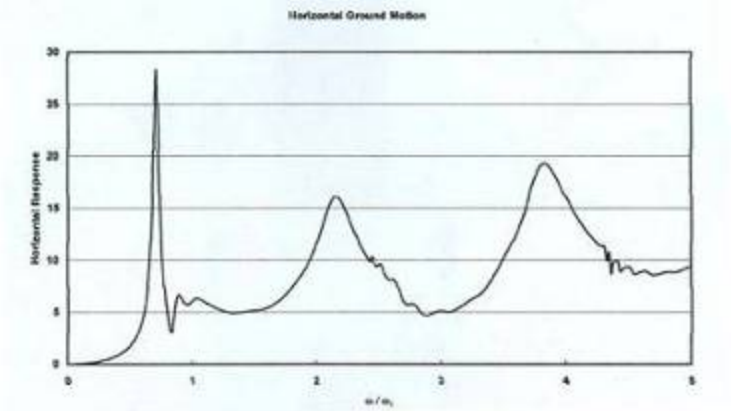


Figure 4: The response from Ansys analysis by FLUID129

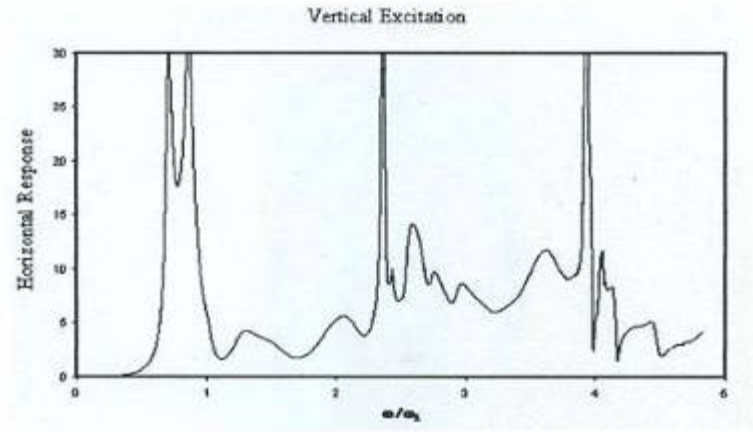


Figure 5: Horizontal the response of crest of the dam to vertical excitation by Sommerfield condition[6]

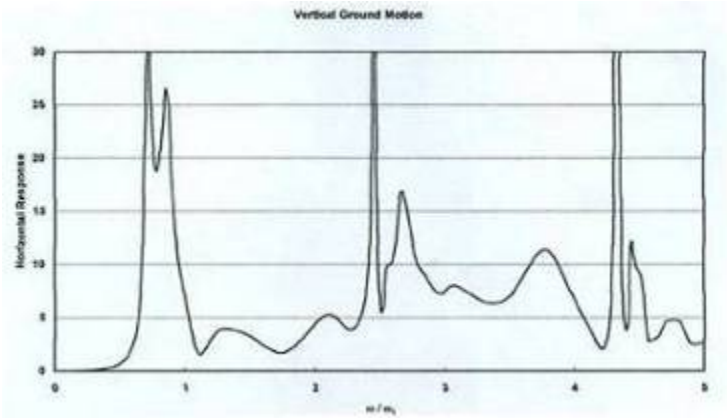


Figure 6: The response from Ansys analysis by FLUID129

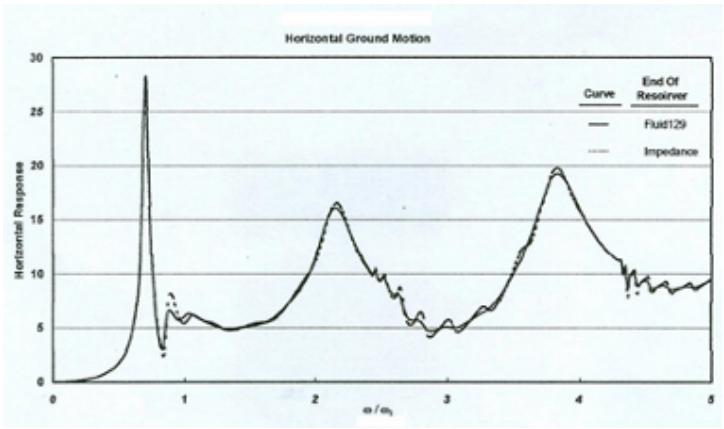


Figure 7: Horizontal the response of crest of the dam to horizontal excitation

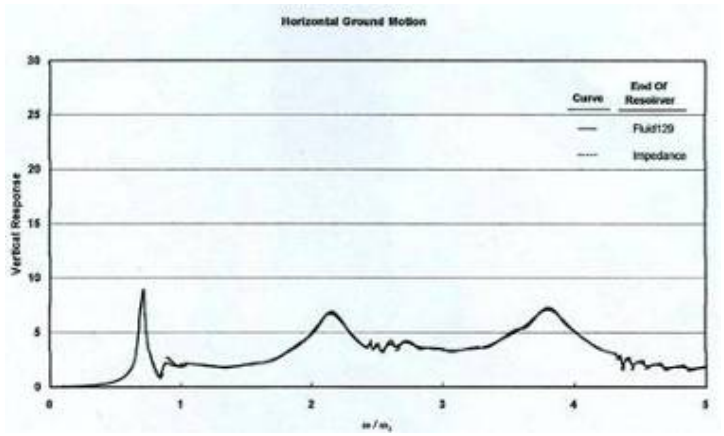


Figure 8: Vertical the response of crest of the dam to horizontal excitation

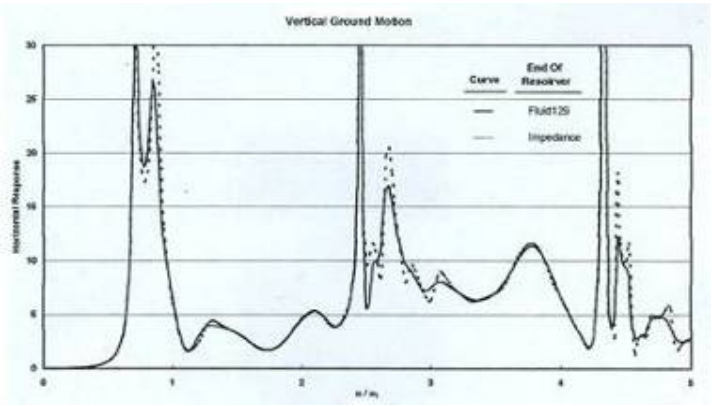


Figure 9: Horizontal the response of crest of the dam to vertical excitation

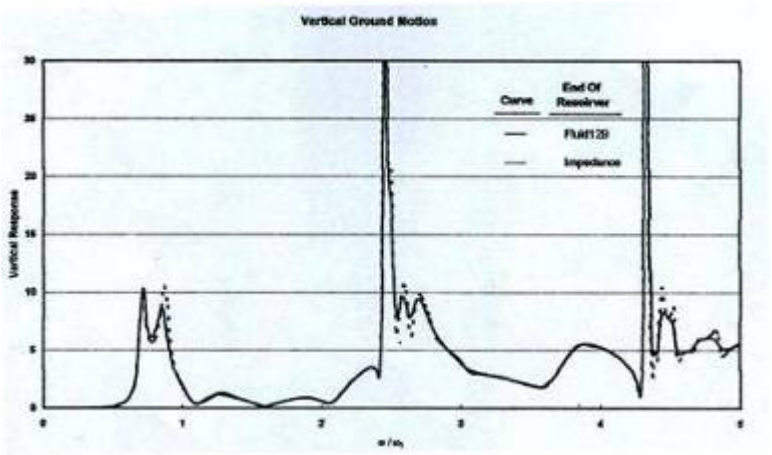


Figure 10: Vertical the response of crest of the dam to vertical excitation

5. CONCLUSIONS

The following two conclusions can be made:

1. There is conversance between analysis response resulted from Sommerfield condition and the response resulted from analysis with the ANSYS software. It indicates that we can use the suggested element in ANSYS software for the dynamic analysis of dam and reservoir interaction with high precision. Also, the small turbulence in the obtained response from the software against the Sommerfield condition, can be included as a benefit in the work with ANSYS.

2. Comparison of the responses from the use of FLUID129 elements and impedance banding as boundary condition in the ANSYS software indicates that there are a few errors between impedance method response and FLUID129 method, although the responses have more turbulence than main method but impedance loading can be a suitable option to absorbing the wave at the bottom of the reservoir

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NUMERICAL SIMULATION OF A DAM-BREAK FLOOD WAVE

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Abstract: In this paper, the propagation of a flood wave after the break of a reinforced concrete dam is simulated numerically, assuming one-dimensional unsteady flow. Two algorithms were written in Fortran language. These algorithms are based on the full hydrodynamic equations of one-dimensional unsteady flow, using the numerical schemes Lax-Wendroff and MacCormack, respectively. In order to validate the simulation results, comparison with the experimental data published in the work of S. Soares-Frazao (2007) is made. The experimental set-up consists of a water tank that simulates the reservoir of a dam, followed downstream by a horizontal dry bottom section, a triangular bottom sill with high slopes and a small pool of water at rest that ends up in a vertical diaphragm. The algorithms can deal successfully with the simulation of the triangular bottom sill and the high negative value of slope without any geometric simplification. The comparison of experimental and numerical data shows a high degree of convergence. The use of diffusion factor is favourable for the operation of numerical schemes that are unable to describe abrupt changes of hydraulic flow properties (velocity, water depth, bottom slope etc.). There is a weakness in describing the phenomenon of flood wave reflections on vertical diaphragms due to the supercritical flow conditions, in addition to the simultaneous formation of a hydraulic jump.

Key words: dam-break, experimental data, numerical schemes, numerical simulation

1. INTRODUCTION

Generally, during a dam failure, a flood wave, with destructive consequences for the downstream area, is created. The main objective of the present paper is the study of flood wave propagation after the immediate and abrupt break of a reinforced concrete dam. When a reinforced concrete dam breaks, it is destroyed immediately and completely, unlike the break of flexible dam construction (there is no failure fractures where the water can slip from).

The study of a flood wave propagation can be achieved through the numerical solution of unsteady flow equations. Generally, unsteady flow is a three-dimensional problem. In cases where flood waves propagate within a river, the water velocity component along stream is much greater than the corresponding vertical and traverse velocity components and so than the latter can be considered negligible.

Consequence of this simplification is that the problem of propagation of a flood wave can be simulated assuming one-dimensional flow (along stream direction). The equations that simulate the propagation are the Saint Venant's equations.

The propagation of flood wave after a possible dam-break and the simulation of this phenomenon is widely studied by many researchers. Determination of flooded areas and the time that is needed for a wave to reach these areas, is feasible with calculation of a propagation and determination of flow characteristics. This determination is essential for the downstream area's protection from the destructive consequences of a possible dam-break.

Various researchers have tried to simulate the above phenomenon from the early 20th century. The first investigation efforts were experimental. However, during the last decades, most of the

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researchers are using numerical simulations (Terzidis and Strelkoff 1970; Xanthopoulos and Koutitas 1976; Cunge et. al. 1980; Katopodes 1984; Bellos and Sakkas 1987; Chaudhry 1993).

2. BASIC NOTIONS

The equations that describe the one-dimensional, unsteady, open channel flow are widely known as Saint-Venant equations. These equations are based on mass and momentum or energy conservation and are valid under a number of assumptions. With suitable modifications these equations can be transformed to the form shown below:

$$\frac{\partial W}{\partial t}+\frac{\partial F}{\partial x}=D \tag{1}$$

where

$$W=\begin{vmatrix} A \\ Q \end{vmatrix} \qquad F=\begin{vmatrix} Q \\ V^2 A+gA\bar{y} \end{vmatrix} \qquad D=\begin{vmatrix} +q_l \\ gA(S_0-S_f)+q_l V_l \end{vmatrix}$$

and A is the area of water cross-section, Q is the discharge, V is the water velocity, q_l is the lateral outflow per unit length of a channel, S_0 is the slope of channel bottom, S_f is the slope of energy line, V_l is the inflow's component velocity at direction of the flow and $\bar{y}$ is the vertical distance between the centre of gravity of the cross-section and the water surface.

Equations (1) constitute a hyperbolic type system of differential equations which is appropriate for numerically solution. The main advantage of Equations (1) is that they are in a conservation law form, proved to be appropriate for describing discontinuous flow phenomena (e.g. hydraulic jump).

3. NUMERICAL SOLUTIONS

A hydrodynamic model-scheme is based on the numerical solution of unsteady open channel flow equations. This method is recommended in cases where flow's unsteadiness is intense and there is no possibility for the parallel application of other models, such as the kinematic or the diffusion model. In the kinematic model, the flow is assumed uniform and steady and in the diffusion model the flow is assumed nonuniform and steady (Ponce et. al. 1978).

For this numerical solution, the well known numerical scheme Lax-Wendroff (Lax and Wendroff 1964) is used, which is derived by substitution of the time and space differences with up and central differences, in Equations (1). This scheme includes two steps, has second class accuracy and is stable for Courant numbers smaller than 1. These two steps, as well as the solution process of the numerical scheme are described below (Equations (2) and (3), Figure 1):

'Predictor' Step

$$W_i^{k+1}=\frac{1}{2}(W_{i-1}^k+W_{i+1}^k)-\frac{\Delta t}{2\Delta x}(F_{i+1}^k-F_{i-1}^k)+D_i^k\Delta t \tag{2}$$

'Corrector' Step

$$W_i^{k+2}=W_i^k-\frac{\Delta t}{\Delta x}(F_{i+1}^{k+1}-F_{i-1}^{k+1})+2D_i^k\Delta t \tag{3}$$

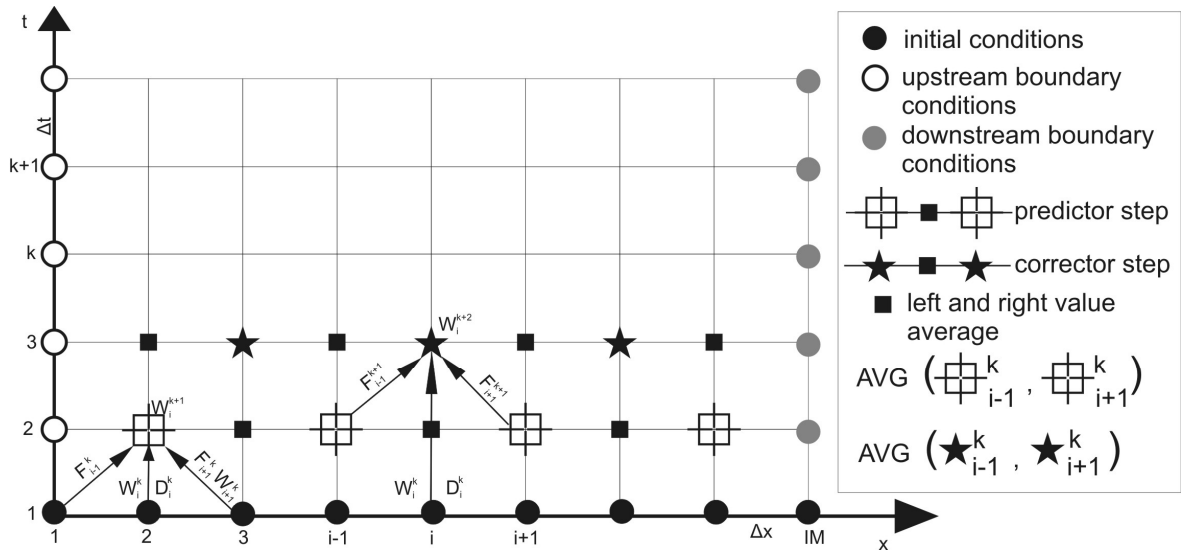


Figure 1. Lax-Wendroff numerical scheme

Considerable diffusion is observed in the Lax-Wendroff numerical scheme at the wave front. An improved development of Lax-Wendroff numerical scheme is the MacCormack numerical scheme which is more suitable for discontinuous flows. This scheme is a rational numerical scheme which includes two steps (prediction and correction), it has second class accuracy with respect to time and space, but instead of central up and down differences are used. These two steps as well as the solution process of the numerical scheme are described below (Equations (4) and (5), Figure 2):

'Predictor' Step

$$W_i^* = W_i^k - \frac{\Delta t}{\Delta x}(F_{i+1}^k - F_i^k) + D_i^k \Delta t \tag{4}$$

'Corrector' Step

$$W_i^{k+1} = \frac{1}{2}(W_i^* + W_i^k - \frac{\Delta t}{\Delta x}(F_i^* - F_{i-1}^*) + D_i^* \Delta t) \tag{5}$$

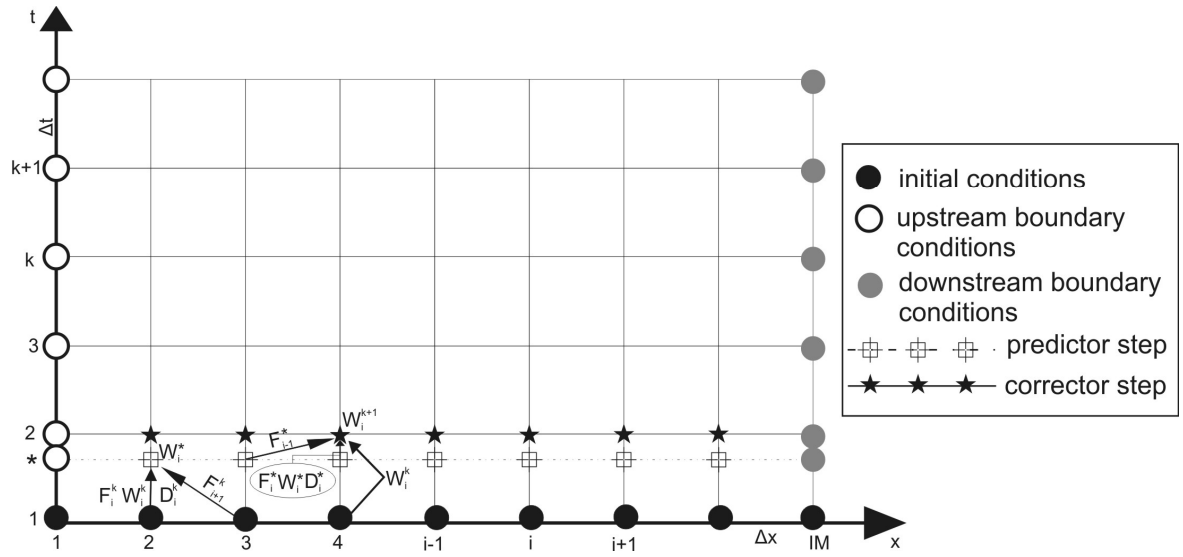


Figure 2. MacCormack numerical scheme

Both above mentioned numerical schemes are stable under Courant condition (CFL) and friction criterion. CFL condition is:

$$C = \frac{\Delta t}{\Delta x}\left|V \pm c\right| \le 1 \tag{6}$$

where c is the celerity.

Diffusion is added to the numerical schemes with a co-factor (OME) at correction step (Equations (4) and (5)) in order to solve problems of abrupt change of flow characteristics (Koutitas 1982):

$$W_i^k = OMEW_i^k + \frac{1}{2}(1-OME)\left(W_{i-1}^k + W_{i+1}^k\right) \tag{7}$$

and the added diffusion is determined (Bellos 1994)

$$(1-OME)\frac{\Delta x^2}{2\Delta t}$$

4. DESCRIPTION OF EXPERIMENTAL DATA

4.1 Experimental Set-Up

The experimental set-up consists of a channel with width b=0.50 m and length l=5.60 m, with a horizontal bottom (Figure 3). A vertical diaphragm is placed at the end of the upstream section of the channel and another one at a distance l_1=2.39 m from the first.

This diaphragm represents the dam, while the water tank represents the reservoir. The water depth in this section is y_1=0.111 m. Just downstream of the water tank there is a dry bed section, which has a length l_2=1.61 m.

Just downstream of this section, a symmetrical triangular bottom sill is placed, which has a length l_3=0.90 m and a height y_2=0.065 m. Just downstream of the triangular bottom sill there is a small water pool at rest, which has a length l_4=0.70 m and a depth y_3=0.02 m.

A vertical diaphragm is also placed at the end of channel. The purpose of this diaphragm is the prevention of water flow and the creation of a basin at rest after a dam-break. The lateral walls of the channel are made of glass and the Manning's coefficient has been estimated n=0.011.

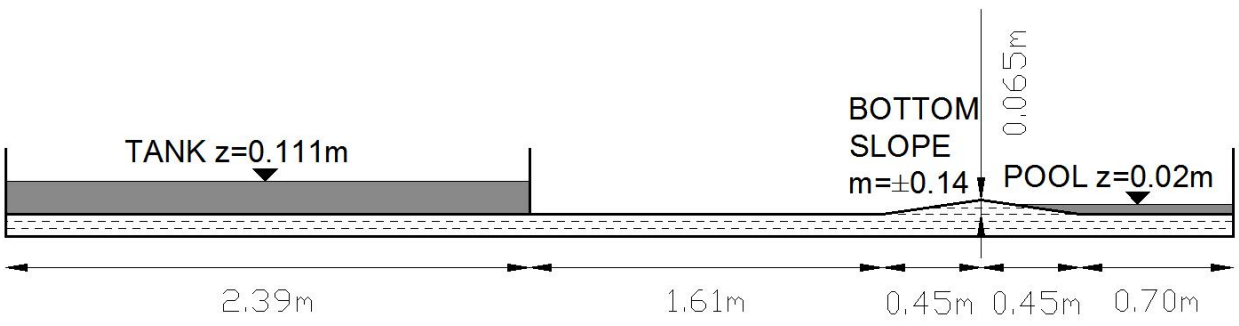


Figure 3. Experimental set-up

4.2 Experiment Implementation

The experiment starts with the abrupt opening of the diaphragm which retains the water of upstream tank and simulates the dam. The abrupt failure of a reinforced concrete dam is simulated with this process. Water flows in the dry bottom until it reaches the triangular bottom sill. The abrupt slope change creates a hydraulic jump.

Afterwards, the water overflows the triangular bottom sill and flows in the small water pool at rest, which is located downstream. The overall flow is reflected when the water front reaches the vertical diaphragm at the end of channel. This process creates waves. The upstream vertical diaphragm placement creates reflection and waves also. Pictures of five water profiles have been taken in five time steps (t=1.8 s, t=3.0 s, t=3.7 s, t=8.4 s, t=15.5 s).

5. ALGORITHM IMPLEMENTATION

5.1 Input Data

The propagation is simulated numerically, assuming unsteady state, one-dimensional flow. Two algorithms were written in Fortran computer language. These algorithms are based on the full hydrodynamic equations of one-dimensional unsteady flow, using the numerical schemes Lax-Wendroff and MacCormack, respectively.

As it is mentioned, the lateral walls of the channel are made of glass. This fact allows the assumption of a channel with infinite width. There are a lot of advantages in a channel with infinite width, in relation to the simplification of the equations, as a band of width 1 m is only studied. Hence, the input data for the width of the channel is b=1.0 m and not b=0.5 m as in the experiment. The input data for the water depth at the dry bed section is h=0.0001 m. It should be noticed that this depth has no influence on passing velocity of the wave, but it can be assumed appropriate for calculation reasons (Bellos and Sakkas 1987).

Water is prevented by the vertical diaphragms upstream and downstream of the experimental set-up, creating a pool. In order to simulate these conditions, the boundary and initial conditions must be specified.

In the upper and lower part of the channel, where vertical barriers are present, the discharge is assumed to be zero and the water depth is determined with the *method of characteristics*. The upstream and downstream boundaries were determined with the ‘back’ and ‘front’ characteristics, respectively (Terzides 1982). Finally, the optimal diffusion factor was determined to be OME=0.9.

5.2 Experimental Results

The corresponding water profiles at times t=1.8 s, t=3.0 s, t=3.7 s, t=8.4 s, t=15.5 s are illustrated in Figures 4, 5, 6, 7, 8, respectively. These profiles are derived from experimental data, the numerical simulation with the Lax-Wenrdoff (LAX) scheme and the numerical simulation with the MacCormack (MAC) scheme.

The discontinuity of experimental data is attributed to the fact that only half of the downstream section was photographed, as well as to the fact that there were beams which were used to support the lateral walls and hence the optical field was not clear enough.

Comparison of the experimental and numerical results at times t=1.8 s, t=3.0 s, t=3.7 s, t=8.4 s shows a high degree of convergence. At time t=15.5 s there is a divergence between the experimental and numerical data.

However, the numerical simulation results can be assumed to be satisfactory. The proposed algorithms can describe the overflowing of an obstacle with sufficiently high slopes, without any complicate considerations.

Moreover, the algorithms simulate with high convergence the initial time steps of the simulated flow, which is of more interest, and are more useful in predictions regarding a real dam-break and the destructive consequences in the areas downstream.

Besides, the conducted laboratory experiment represents just a study case and not a real situation. There are no obstacles (such as the vertical diaphragms) downstream of a river, that prevent the propagation of a flood wave.

For further study, the phenomenon analysed in this paper can be further investigated taking into account the effect of wave reflections in obstacles (such as the vertical diaphragms).

Moreover, the existence of an obstacle downstream of a dam (such as the triangular bottom sill) and the expected energy reduction due to the hydraulic jump formation can be investigated, in order to see if this obstacle can be assumed as a safety measure which limits the destructive consequences of a possible dam-break.

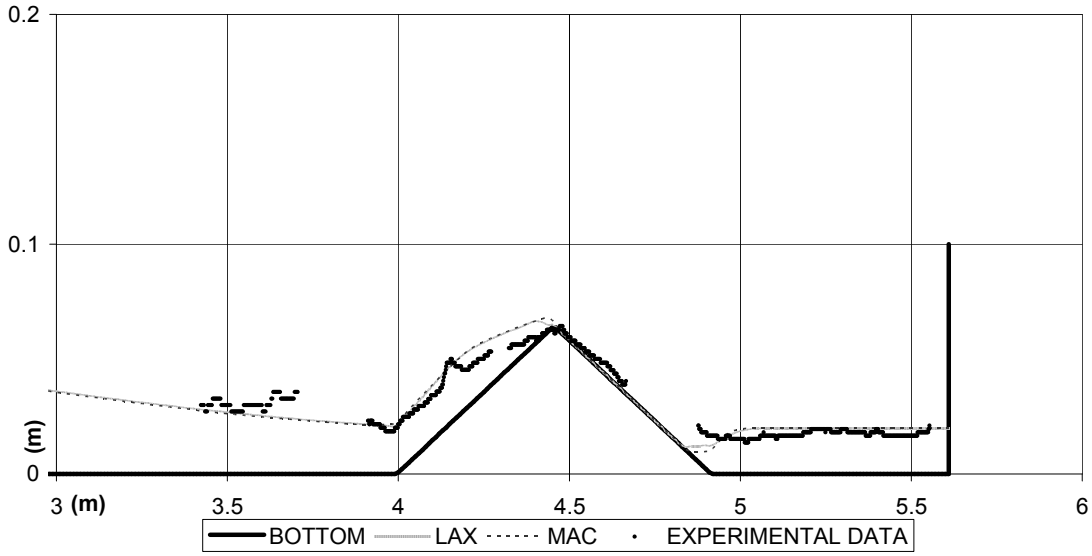


Figure 4. Water profile at time step t=1.8 s

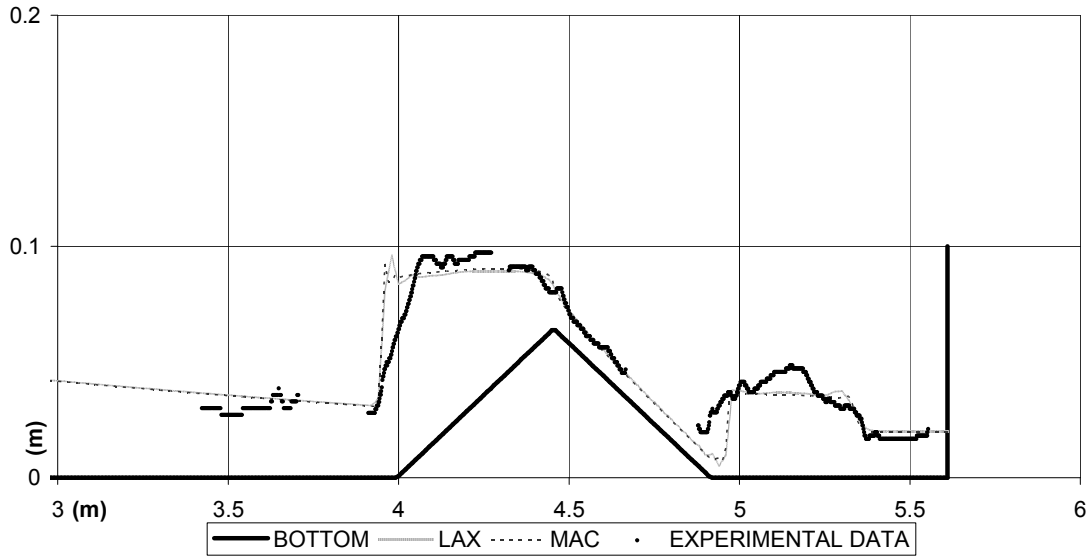


Figure 5. Water profile at time step t=3.0 s

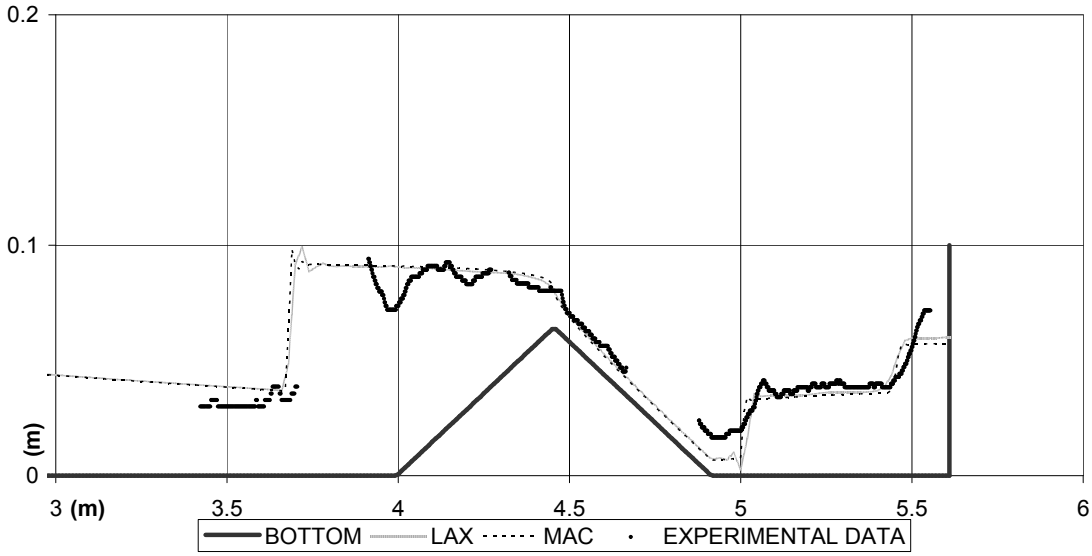


Figure 6. Water profile at time step t=3.7 s

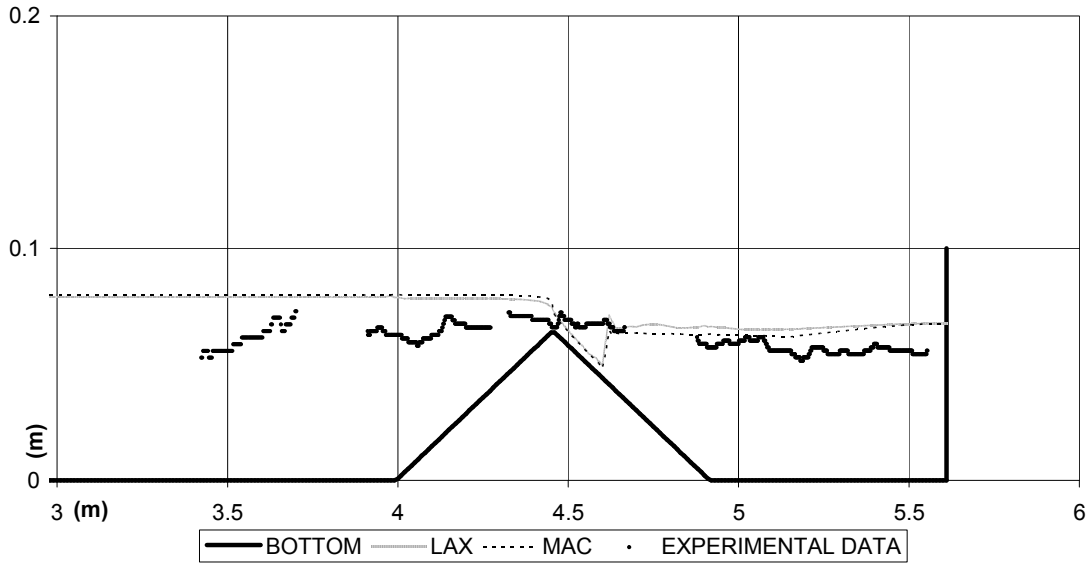


Figure 7. Water profile at time step t=8.4 s

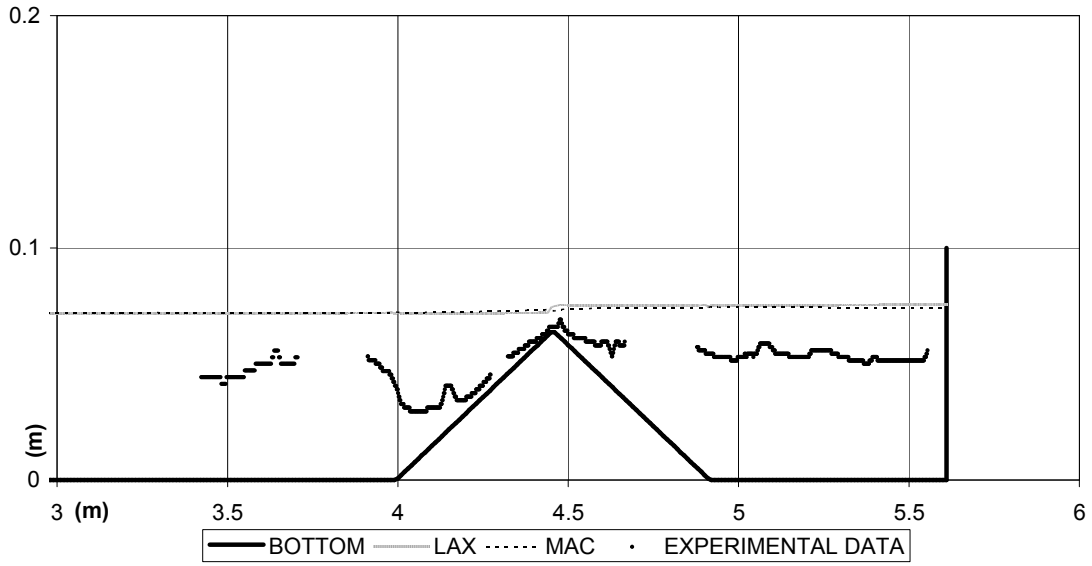


Figure 8. Water profile at time step t=15.5 s

6. CONCLUDING REMARKS

In this paper two algorithms using MacCormack and Lax-Wendroff numerical schemes were tested using experimental data published in the work of S. Soares-Frazao (2007).

Based on the results of numerical simulation carried out by the above numerical schemes it was concluded that both algorithms describe satisfactorily the flood wave propagation phenomenon which is a result of an abrupt dam-break in the field with a triangular bottom sill, without any geometric simplification for the obstacle.

The use of diffusion factors is favourable for the operation of numerical schemes that are unable to describe abrupt changes of hydraulic flow properties (velocity, water depth, bottom slope etc.).

The drawback of the proposed procedure is the difficulty in describing the phenomenon of flood wave reflections in vertical diaphragms due to the supercritical flow conditions, in addition to the simultaneous formation of a hydraulic jump.

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RELIABILITY OF THE RESERVOIRS AT THE REGION OF THE FORMER LAKE KARLA IN THESSALY IN MEETING CROP WATER REQUIREMENTS

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Abstract: For the time being, twelve surface reservoirs operate in the area of the former lake Karla in order to supply the sufficient water quantities for irrigation of the nearby cultivated areas mainly during the period June – August when the irrigation water requirements are high while the available water from other resources is negligible. In the present study, monthly climatic data for a period of 43 years (1953-1997), from the meteorological station of the National Meteorological Service (N.M.S.) in Larissa are utilised for the estimation of reservoir evaporation and the crop water requirements in order to evaluate the efficiency of reservoirs in meeting crop water requirements with reference to their technical characteristics and the size of the irrigated areas. The analysis exemplifies that the reliability among the various reservoirs of the study area is not the same. Reservoirs usually do not cover the total crop water requirements of the areas they serve. On average, they cover only part of them (48 – 83%), depending on their characteristics and the cropping pattern of the sub-area they serve.

Key words: Evaporation, evapotranspiration, FAO 56 Penman-Monteith, reliability, reservoir.

1. INTRODUCTION

The Thessaly plain, dominated by the Pinios river watershed, is the largest agricultural area in Greece. Under the present water management regime, the available water provides only 40-70% of the crop water needs over much of the area and extensive droughts occur during critical periods of the growing season (July-August) with adverse effects on crop yield and quality. In order to alleviate these effects a number of irrigation related structures have been planned and constructed. For that purpose, twelve surface reservoirs operate in the area of the former lake Karla in order to supply the sufficient water quantities for irrigation of the nearby cultivated areas.

In the frame of the research programme "Archimedes – EPEAEK II", co-funded by the European Social Fund & National Resources and realised by the Dept. of Civil Engineering of TEI of Larissa, Greece, under the title "Spatial mapping and estimation of hydrological risk, emphasising on floods and draughts in urban and rural areas of Thessaly and their environmental impacts", the above region has been selected as study area.

The research aims at estimating the reliability of the above structures, i.e. water reservoirs, in covering the essential water quantities for irrigation.

In the study area there are twelve (12) reservoirs (Land Reclamation 2005; Goumas 2006) in total that supply irrigation water into nine sub-areas, as shown in Figure 1. These reservoirs are filled up with water, through pumping, early in the spring from Pinios river. This water is then used to irrigate the local cultivations during the dry period (mainly June and July).

In the study area, sprinkler and trickle irrigation is applied on 70% of the cultivations while the rest 30%, mainly serials, are rain fed. From those irrigated areas cotton covers about 85%, while the rest is cultivated by maize, alfalfa and processed tomatoes.

Given the size of reservoirs along with the corresponding irrigated area and the distribution of crops, it is considered more than necessary to evaluate the reliability of each reservoir in meeting

the irrigation water needs, taking into account the variability of climatic conditions from year to year.

Methodologies for validating irrigation networks have been proposed in the past for specific crop species (Kotsopoulos 1989; Kotsopoulos and Svehlik 1995). This work necessitates mainly the estimation of both the evaporation of reservoirs and the evapotranspiration of crops on a monthly basis. These estimations are then utilised to calculate the water balance of each sub-region for a certain number of years.

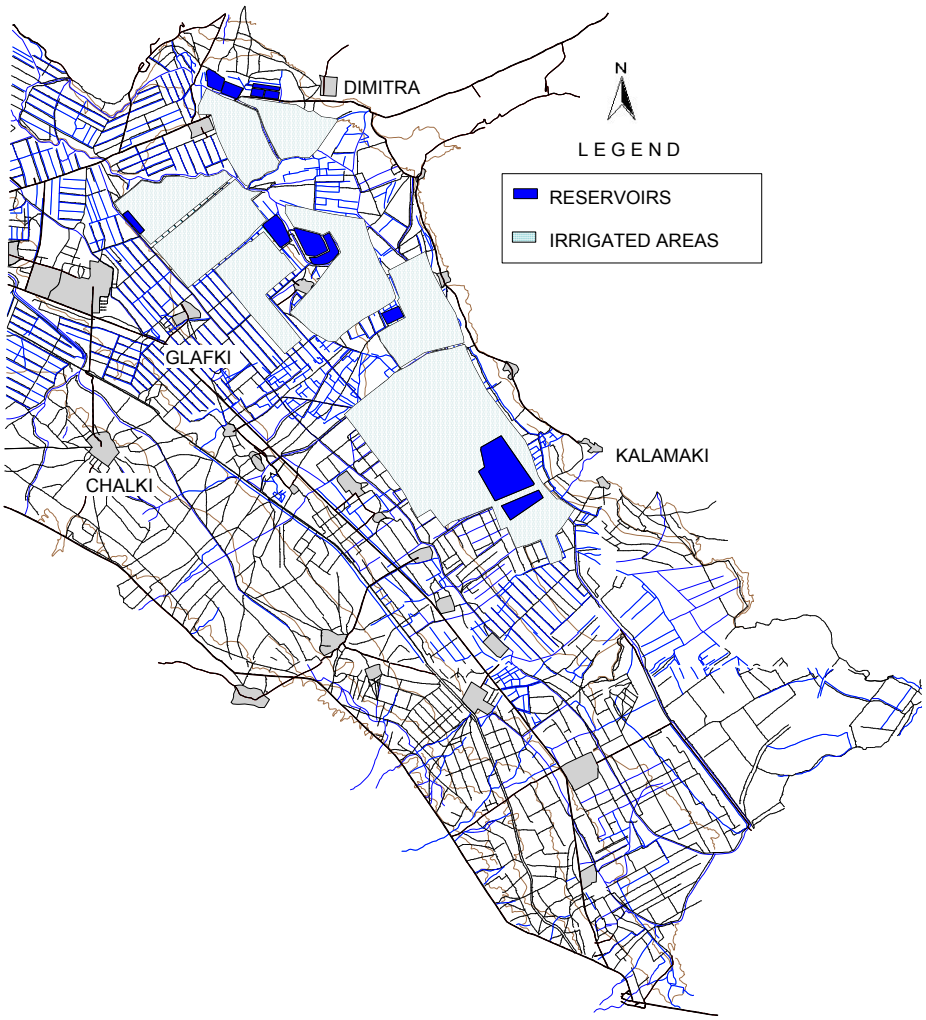


Figure 1: Reservoirs and irrigated by them areas in the region of former Lake Karla

This project considers monthly climate data taken from the meteorological station of the National Meteorological Service (N.M.S.) of Larissa along with soil data (Kalfountzos et al. 1999) and crop species data of the study area, with the aim at estimating the water balance of each sub-region which is irrigated from a certain reservoir. These estimations are then utilised to evaluate the reliability of reservoirs in order to cover sufficiently the local irrigation water needs.

2. MATERIALS AND METHODS

Each region of cultivation is characterised by a number of parameters, such as water evaporation from reservoirs, crop water needs, rainfalls and reduction of soil moisture, which have to be consider. All these features have to be taken into account in order to estimate the water shortage

during each year and consequently the risk of failure in water supply for irrigation.

The study area is close to the city of Larissa, where a meteorological station of the N.M.S. is already in operation. The greater Larissa area is suffering by warm and dry summers. This period is crucial for crop growth, demanding for irrigation great quantities of water. At the plain part of the county and especially at the region of the former Lake Karla, the prevailing temperatures along with other climatic parameters which determine the level of evaporation and evapotranspiration of crop species (solar radiation, sunshine duration, relative humidity, wind speed) are practically considered to be the same to those recorded by the N.M.S. of Larissa.

2.1 Calculation of evaporation and evapotranspiration

Nowadays, the most reliable method for the calculation of evaporation and evapotranspiration is considered the modified Penman-Monteith equation, as described in FAO-56 (Allen et al. 1998; Kotsopoulos et al. 2003).

The method is applied in two stages. Initially the evapotranspiration of the reference crop is calculated and then crop evapotranspiration is estimated using the corresponding crop coefficient as shown in the following relation:

$$ET_c = K_c \cdot ET_o \tag{2.1}$$

where ET_c is the daily evapotranspiration (mm/d), K_c the crop coefficient which is dependant on the crop development stage (Allen et al. 1998), and ET_o the evapotranspiration of the reference crop (mm/d).

Evaporation can be calculated in a similar way through the equation:

$$E = K_c \cdot ET_o \tag{2.2}$$

where E is the daily evaporation (mm/d) and K_c the surface coefficient, which, for shallow water surfaces takes the value of 1.05 (Allen et al. 1998).

The evapotranspiration of reference crop is determined through the modified Penman-Monteith method as described in FAO-56 (Allen et al. 1998; Kotsopoulos 2006) and is expressed through the equation:

$$ET_o = \frac{0.408\Delta(R_n - G) + \gamma \frac{900}{T+273} u_2 (e_s - e_a)}{\Delta + \gamma \cdot (1 + 0.34 \cdot u_2)} \tag{2.3}$$

where ET_o reference evapotranspiration (mm/d), R_n net radiation at the crop surface($MJm^{-2}d^{-1}$), G soil heat flux density ($MJm^{-2}d^{-1}$) which here can be considered negligible ($G \approx 0$), T mean air temperature at 2m height (^oC), u_2 average wind speed at the same height (ms^{-1}), e_s saturation vapour pressure (kPa), e_a actual vapour pressure (kPa), Δ slope of saturation vapour curve at temperature T (kPa $^oC^{-1}$) and γ psychrometric constant (kPa^oC^{-1}). For the evaluation of R_n, the estimation of extra terrestrial radiation (R_a) is essential along with the maximum sunshine duration (N) which can be realised through periodic functions (Kotsopoulos and Babajimopoulos 1997).

Through equations (2.1) and (2.3) along with the monthly rates of climatic parameters (temperature, sunshine duration, relative humidity and wind speed) for the years 1955-1997 taken from the N.M.S. of Larissa, the monthly rates of evapotranspiration for the above period of 43 years is calculated. These results are presented in Figure 2 for the period 1990-94.

2.2 Calculation of water balance

The validation of reservoirs' reliability of the study area necessitates the water balance of each sub-area irrigated by a certain reservoir system.

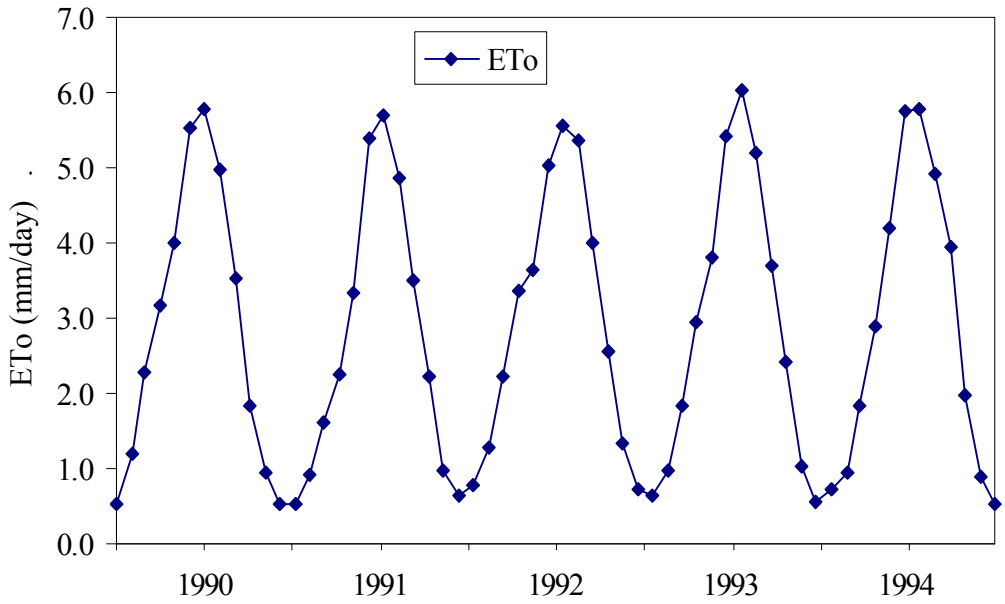


Figure 2: Calculated values of reference crop evapotranspiration (mm/d) at Larissa for the years 1990-94

The water balance is estimated per year and takes into account the technical characteristics of reservoirs (capacity, water surface) along with the evaporation rate (Kotsopoulos et al. 2006), the size of irrigation areas the cropping pattern and the crop water requirements, the rainfalls and the reduction of soil moisture.

The data used for the calculations are presented on Table 1 and refer to the technical characteristics of reservoirs (Goumas 2006) along with the characteristics of their irrigated sub-areas (Kotsopoulos et al. 2007).

Table 1: Technical characteristics of reservoirs and their irrigated sub-areas

	Technical characteristics of reservoirs			**Characteristics of the irrigated sub-areas**				
	Reservoir	**Capacity $10^6 m^3$**	**Water surface $10^3 m^2$**	**Irrigated Area $10^3 m^2$**	**Cotton %**	**Maize %**	**Alfalfa %**	**Processed Tomato %**
1	Eleftherio I, II	1.70	600	4000	85		15	
2	Glafki	2.10	550	5415	85	10	5	
3	Kalamaki I, II	8.00	2750	20000	85		15	
4	Dimitra	1.00	400	2600	85		15	
5	Platikampos II	1.45	500	4560	85	10	5	
6	Platikampos I	0.50	250	1500	85	10	5	
7	Namata I, II	2.90	983	10000	80	10	5	5
8	Omorfochori	1.25	350	5000	80	20		
9	Kastri	1.10	350	4900	85		15	

3. Results and discussion

Results refer to the average reliability of reservoirs, R_m, its standard deviation, SD_R and the variation coefficient, CV_R. These values come from a simulation period of 43 years. As reliability of a reservoir is defined, the ratio of its capacity over the total crop water requirements of the sub-area it serves. The value 1.0 for reliability (maximum value) means that water requirements of all crops irrigated in a certain sub-area are completely satisfied. These results are shown in Table 2.

Table 2: Reliability parameters of reservoirs for the study area

	Reservoir	**Average rate, R_m**	**Standard Deviation, SD_R**	**Variation Coefficient, CV_R, %**
1	Eleftherio I, II	0.832	0.113	13.6
2	Glafki	0.816	0.115	14.1
3	Kalamaki I, II	0.800	0.111	13.9
4	Dimitra	0.758	0.116	15.3
5	Platikampos II	0.667	0.105	15.8
6	Platikampos I	0.660	0.109	16.5
7	Namata I, II	0.610	0.099	16.3
8	Omorfochori	0.543	0.091	16.7
9	Kastri	0.483	0.082	17.0

For the cropping pattern assumed under the existing agricultural conditions (Table 2), the average reliability, R_m and its standard deviation, SD_R, vary significantly from area to area. Average reliability, R_m, maybe as low as ~48% (Kastri) while its maximum value slightly exceeds 83% (Eleftherio). On the contrary, the variation coefficients of reliability, CV_R, vary slightly from area to area and are in the range of 13.5-17.0%.

Taking into account the values of the above parameters and the assumptions of the Central Limit Theorem (Haan 1977; Kite 1985; Yevjevich 1982; Chatfield 1983), a general classification of those reservoirs is feasible according to their reliability under specific conditions.

The validation is typically realised with respect to the average reliability of reservoirs for various planning periods and risk levels. These reliability results are presented in Table 3.

Table 3: Estimated reliability of reservoirs for the study area

	Reservoir	**Mean R_m** for a period of 10 years and a 10% risk level	**Mean R_m** for a period of 10 years and a 5% risk level	**Mean R_m** for a period of 5 years and a 5% risk level
1	Eleftherio I, II	0.787	0.766	0.749
2	Glafki	0.769	0.749	0.732
3	Kalamaki I, II	0.755	0.735	0.718
4	Dimitra	0.711	0.690	0.673
5	Platikampos II	0.625	0.606	0.590
6	Platikampos I	0.616	0.597	0.580
7	Namata I, II	0.570	0.552	0.537
8	Omorfochori	0.506	0.490	0.476
9	Kastri	0.449	0.435	0.422

In general, the most reliable reservoirs are those of Eleftherio I & II followed by Glafki, Kalamaki I & II, Dimitra, Platikampos II, Platikampos I, Namata, Omorfochori and Kastri.

More specifically, for a planning period of 10 years and a 10% risk level, the reservoirs are classified according to their average reliability as follows: Eleftherion I & II (78.7%) followed by Glafki (76.9%), Kalamaki I & II (75.5%), Dimitra (71.1%), Platikampos II (62.5%), Platikampos I (61.6%), Namata (57.0%), Omorfochori (50.6%) and Kastri (44.9%). These results are depicted in Figure 3.

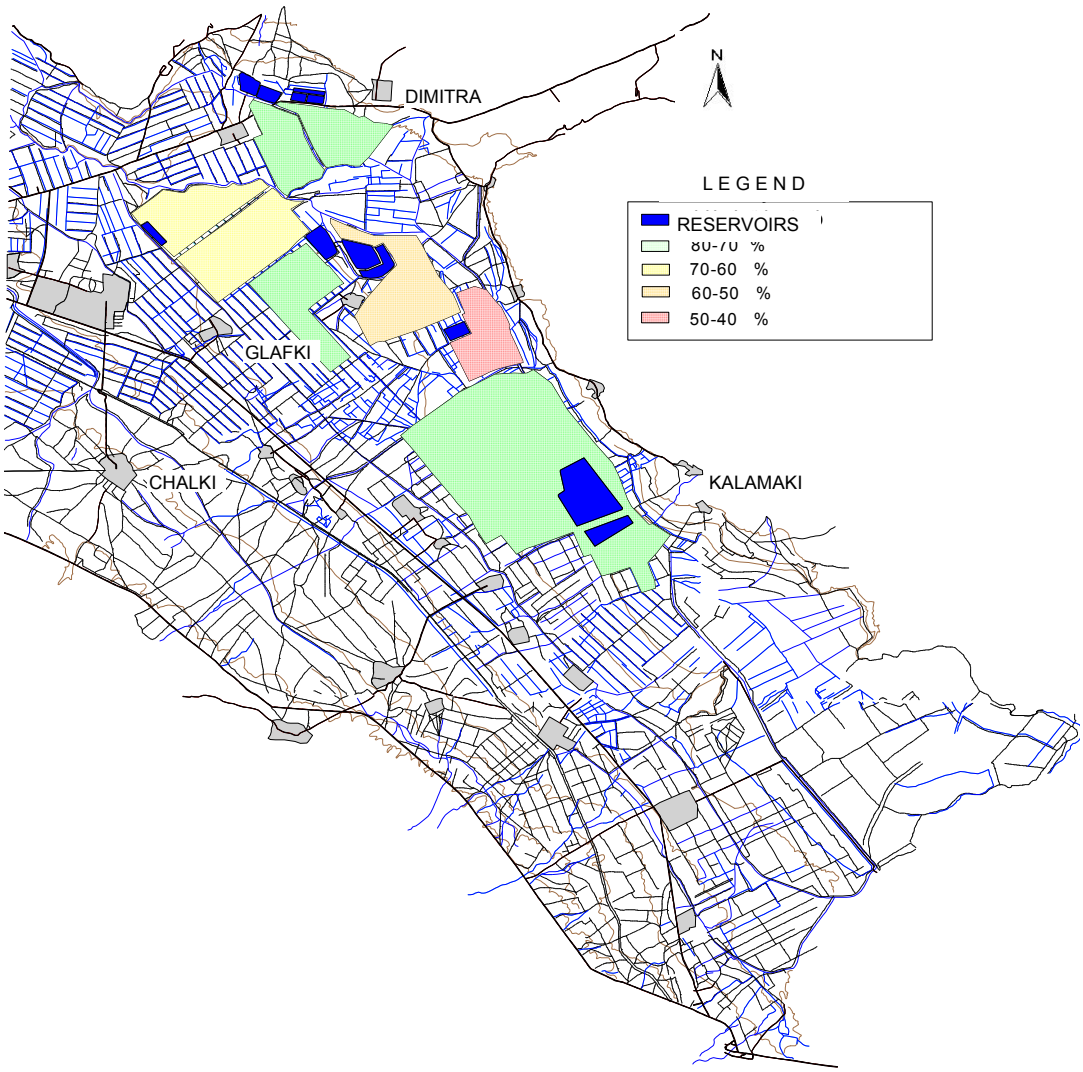


Figure 3: Average reliability of reservoirs in the region of Karla for a decade and a 10% risk level

4. Conclusions

From the calculated values of water shortage related to crop water requirements and the reservoir capacity for each part of the study area (broader area of the former lake Karla) that take into account the stochasticity of climatic conditions for a period of 43 years, the size of reservoirs along with the size of the irrigated areas and the current distribution of crop species, the following conclusions can be drawn:

a) The reliability of the reservoirs within the study area as it is related to irrigation water requirements vary significantly from place to place. Reservoirs usually do not cover the total crop water requirements of the areas they serve. On average, they cover only part of them (48 – 83%), depending on their characteristics and the cropping pattern of the sub-area they serve. The most

reliable reservoirs, according to the characteristics presented above, are those of Eleftherio I & II, Glafki, and Kalamaki I & II while the least is the one of Kastri.

b) Reliability values for the cultivated areas served by the same reservoir vary from year to year, due to the discrepancies of the climatic conditions. This variability, expressed through the variation coefficient, fluctuates between ~13.5 - 17%.

The above reliability analysis for the structures used to store water for irrigation, may be a useful tool for evaluating the efficiency of the existing reservoirs and planning the construction of new ones, so that, they meet the local irrigation water requirements.

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EVALUATION OF THE FUTURE OPERATION OF THE RESTORED LAKE KARLA RESERVOIR UNDER CLIMATE CHANGE

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Abstract: This study evaluates, through stochastic simulation, the operation of the restored Lake Karla's reservoir in Thessaly. The future timeseries of monthly precipitation and mean monthly temperature have been produced through downscaling techniques using the results of the Canadian Centre for Climate Model Analysis General Circulation Model (CGCMa2). A stochastic methodology has been developed to downscale the outputs of GCM and produce future precipitation timeseries. The methodology combines a multiple regression (MLR) model and a stochastic timeseries model for the precipitation residuals between the observed and the downscaled precipitation through the MLR model. The mean monthly temperature was downscaled by a simple MLR model. The downscaling methods have been developed for the historical base period 1960-1990, validated for the period 1990-2002, and, then, used for the generation of the future timeseries for the periods 2020-2050 and 2070-2100 for SRESA2 and SRESB2 socioeconomic scenarios. The generated meteorological timeseries have been used as input data in a monthly water balance model (UTHBAL) and a reservoir operation model which balances the inflows, outflows and the change of water storage and posses a constrain on the reservoir water level for wetland preservation. The under-construction reservoir of the Lake Karla designed to cover the large irrigation water demands. The historical and future water demands estimated using the Near Irrigation Requirement and estimates of the water efficiency of the water distribution system and irrigation methods for the existing cultivation pattern and for a management scenario which combines crop cultivation pattern change according to European Union Common Agricultural Policy and improved water efficiency of the water distribution and irrigation system. The results indicated that the Lake Karla reservoir although satisfactorily covers the present climate water demands, it fails to satisfy the future climate water demands, even for the moderate SRESB2 scenario and the improved water management scenario.
Key words: Climate change, Lake Karla reservoir, water resources management, uncertainty.

1. INTRODUCTION

Uncertainty is one of the main features of water resources systems, which causes their management particularly difficult. Conventional methods of deterministic approach have proven unable to describe effectively the processes and thus, lead to assume them as random. Sources of uncertainty could be the hydrometeorological data, water demand data, models used, etc. The stochastic methods give answers to water resources uncertainty by quantifying the uncertainty of the various components and processes of water resources management. In this study, the main source of uncertainty is assumed the one that comes from the uncertainty in the hydrometeorological data (mainly precipitation), the runoff and the impact of climate change. The parameters of stochastic models are estimated by the statistical characteristics of historical timeseries. Stochastic models are being used to produce long- length synthetic timeseries, that

reproduce the statistical structure and characteristics of the observed historical data timeseries. The use of synthetic timeseries into programs of water system management, allows further evaluation of future operation of these systems for a large scale of hydro-climatic scenarios. Uncertainty over the natural availability of water resources is thus quantified alongside the ability to output almost certain conclusions on the performance and liability of the water system.

This study aims to evaluate the future operation of the restored Lake Karla reservoir under climate change scenarios using stochastic simulation procedures. Other studies in the area have shown that climate change will affect Lake Karla watershed (Loukas, 2007b). Two time periods were selected, a medium-term period (2020-2050) and a long-term period from (2070-2100), to assess the impacts of climate change and to compare with the historical base period (1960-1990). A methodology has been developed in order to connect Global Circulation Models (GCMs) output with precipitation and temperature. The methodology uses statistical downscaling of monthly precipitation and temperature computed by the outputs of Global Circulation model Canadian Centre for Climate Modeling Analysis (CGCMa2) for emission scenarios SRES A2 and SRES B2 (IPCC 2001), combined with stochastic timeseries models for the simulation the residuals of the statistical downscaling of precipitation. The stochastically generated downscaled precipitation and temperature are, then, used as input to a hydrological model and a system for the simulation of the operation of the reservoir of the restored Lake Karla. At the end, the probability of satisfying the historical and future water demands and the probability of satisfying the lower water level of the reservoir set for wetland preservation for two agricultural management scenarios have been evaluated.

2. STUDY AREA

Lake Karla watershed is located in the southeastern corner of the plain of Thessaly (Fig. 1). It is a closed and elongated watershed with a length of 35 km and a width of 9-15 km. The watershed stretches to the north to river Pinios and mount Ossa, to the east till mount Mavrovounio and the bulge of Pilio, to the south until mount Chalkodonio and Megavouni and to the west to mount Fylleion. The Lake Karla's natural watershed extends over an area of 1663 km^2, but after the construction works of the reservoir it narrowed down to 1171 km^2.

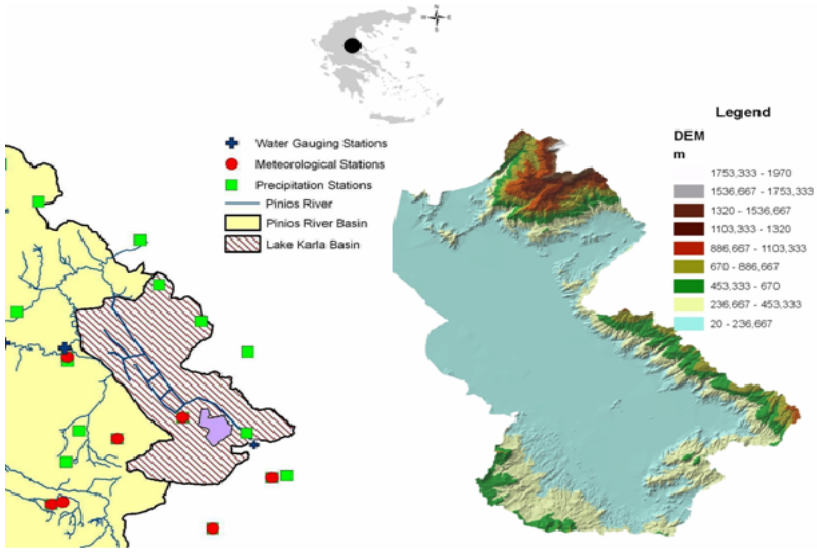


Figure 1. Study area, database and digital elevation model of Lake Karla watershed.

In the middle of the watershed lies the deepest section of the plain of Thessaly, part of which was occupied by the natural Lake Karla until 1961. Lake Karla was the main recipient of the watershed runoff and River Pinios overflowing winter flood waters. The natural Lake Karla was drained in order to reveal new agricultural fields, reduce the malaria problem and protect the area from frequent flooding. However, the drainage of Lake Karla has caused other serious environmental

problems (e.g. groundwater over pumping for irrigation, degradation of groundwater, increased soil salinity, etc). In the early 1980's the construction of a reservoir with an average area of 38 km^2 has been decided. The reservoir will receive water through pumping from Pinios River during winter months and watershed runoff. A network of channels will convey the water to the reservoir (Fig. 1). Today, the construction of water works is near completion.

The climate of the watershed is typical continental characterized by cold and wet winters and hot and dry summers. Lake Karla watershed, as a part of the wider Thessalian plain, is confronted, during the summer months, with severe water deficit problems. This is mainly caused by the domination of water consuming cotton cultivation. Examination of the water demands of the study area reveals that, almost 97% of the total water demand is for agricultural irrigation purposes (Patsonas, 2008) which, on average, reach the amount of about 160 million m^3 per year.

The basin wide precipitation has been estimated using the data from the available meteorological stations in the watershed and its vicinity and the Thiessen polygon method with elevation adjustment. The basin wide temperature was estimated using the available data and the elevation gradient method. The monthly evapotranspiration has been calculated by the Thornthwaite method using the basin wide temperature.

3. METHODOLOGY

A stochastic methodology was applied in order to generate monthly timeseries of precipitation for the stochastic simulation and the evaluation of Lake Karla reservoir operation for a historical period and future periods under climate change. The methodology is based on the combination of a multiple linear regression model (MLR) between the outputs of the GCM Canadian Centre for Climate Modeling Analysis (CGCMa2) and the observed precipitation for two socio-economic scenarios (SRES A2 and SRES B2) and a timeseries model for the stochastic generation of the residuals between observed and downscaled precipitation by the MLR model. The temperature was downscaled using an MLR model between the outputs of global circulation model Canadian Centre for Climate Change Modeling Analysis (CGCMa2) and the observed basin wide temperature. The downscaling methodologies were developed and validated against historical data (Patsonas 2008).

The generated timeseries of precipitation and the downscaled temperature, then, are used as input to a monthly water balance model in order to calculate the runoff of the watershed. The simulated watershed runoff and the withdrawals from the River Pinios were used in a simulation system of Lake Karla the reservoir operation. The agricultural water demands in the study area were estimated using the Near Irrigation Requirement (Dasdane 1974) for two agricultural management and water demand scenarios.

3.1 GCM and Downscaling

The output variables of GCM, able to show changes in temperature or precipitation, could be the wind speed, mean sea level pressure, geopotential height, geopotential thickness, and others. The variables used in this paper were chosen through parametric analysis for the region of Thessaly (Loukas *et al*, 2008) amongst the primal output variables of Global Circulation Model Canadian Centre of Climate Modeling Analysis (CGCMa2). Multi-linear regression models between GCM primal output variables (weather predictors) and observed temperature and precipitation has been developed for the period 1960-90 and validated for the period 1990-2002 (Patsonas 2008). For temperature a MLR model with 14 predictors (12 monthly dummies plus mean sea level pressure and geopotential thickness 500 hPa-1000hPa) has been proved capable to reproduce the observed temperature timeseries with increased accuracy (R^2=0.96). Consequently, the future temperature produced by the MLR model for the periods 2020-50, 2070-2100 and for the two greenhouse gas emissions socio-economic scenarios (SRES A2 and SRES B2).

A similar MLR model has been used for precipitation downscaling using 16 predictors (12 monthly dummies, mean sea level pressure, geopotential height of 500 hPa, geopotential thickness

500 hPa-1000 hPa and wind speed). The MLR model proved inadequate to simulate the observed precipitation. For this reason a stochastic timeseries model was developed to simulate the residuals between observed precipitation and precipitation estimated by the MLR model. The timeseries model is a periodic auto regressive model (i.e. a PAR (4)) and is used to generate 100 timeseries of precipitation residuals. The summation of MLR model estimated precipitation and precipitation residuals simulated well the observed precipitation (Patsonas 2008). The precipitation for the future periods was simulated by the developed methodology first by applying the MLR model and then adding to these estimates the precipitation residuals generated by the timeseries PAR (4) model, assuming that the precipitation residuals will remain unchanged in the future. The results of downscaled precipitation and temperature for the two socioeconomic scenarios, SRES A2 and SRES B2, and for the two future periods, 2020-2050 and 2070-2100, are presented in Table 1.

Table 1. Average monthly precipitation and temperature for the historical base period and the future periods for the two socioeconomic scenarios

Precipitation						
Month	**1960-90 Average**		**SRES A2 Average**		**SRES B2 Average**	
	Observed	**Simulated**	**2020-50**	**2070-100**	**2020-50**	**2070-100**
	(mm)	**(mm)**	**(mm)**	**(mm)**	**(mm)**	**(mm)**
October	72,01	72,11	67,66	61,89	69,80	66,05
November	70,12	70,09	65,05	58,26	63,06	61,42
December	70,74	70,53	66,17	58,94	66,06	63,38
January	53,66	53,26	52,41	47,39	52,57	50,27
February	48,25	48,38	45,64	40,75	46,39	43,27
March	52,95	52,77	50,21	43,20	50,23	45,55
April	38,73	39,03	35,99	33,20	36,96	34,04
May	43,35	42,92	40,57	36,01	40,74	38,26
June	25,22	26,38	24,95	21,46	25,29	23,56
July	26,29	27,75	25,81	23,19	25,37	24,29
August	17,67	18,10	16,69	15,12	16,77	15,68
September	45,70	53,47	52,14	49,82	52,31	51,28
Year	564,69	574,79	543,29	489,22	545,54	517,05
Temperature						
Month	**1960-90 Average**		**SRES A2 Average**		**SRES B2 Average**	
	Observed	**Simulated**	**2020-50**	**2070-100**	**2020-50**	**2070-100**
	(^{o}C)	**(^{o}C)**	**(^{o}C)**	**(^{o}C)**	**(^{o}C)**	**(^{o}C)**
October	15,20	14,60	15,55	16,16	15,52	15,81
November	10,24	9,85	10,72	11,40	10,74	10,98
December	5,83	5,51	6,25	7,01	6,31	6,58
January	4,56	5,67	4,85	5,53	4,87	5,17
February	6,12	8,49	12,74	13,47	20,64	21,12
March	8,95	8,46	10,26	10,98	17,13	17,54
April	12,84	12,44	13,29	13,88	13,18	13,63
May	17,33	16,83	17,67	18,23	17,62	18,02
June	21,88	20,97	22,28	23,06	22,16	22,53
July	23,78	23,25	24,26	25,06	24,27	24,59
August	23,41	22,85	23,90	24,61	23,82	24,15
September	19,76	19,37	20,19	20,87	20,12	20,38

These results show that average precipitation will decrease in the future, whereas, average temperature will rise. Future annual precipitation will decrease, on average, by about 30-40

millimeters for the period 2020-50 and by 40-60 millimeters for the period 2070-2100 compared to the average annual observed precipitation for the period 1960-1990. The average annual temperature would rise by 0.5-1^{o} Celsius in the future, although large changes have been indicated in the monthly values (Table 1).

3.2 Hydrological Modelling

The monthly watershed runoff was simulated using the UTHBAL model. UTHBAL is a monthly water balance model (Loukas *et al*, 2007a), which is able to calculate the snow accumulation and the snowmelt. The UTHBAL model uses as input the watershed precipitation, temperature and potential evapotranspiration and calculates the watershed runoff, actual evapotranspiration, the snow accumulation and snowmelt among other hydrological parameters. Lake Karla watershed has areas of high altitude and thus, snowmelt is a significant component of the watershed runoff. UTHBAL model was calibrated for the period 1960-90 and validated for the period 1990-2002 (Patsonas 2008). The calibration-validation results indicated that the model is able to accurately simulate the monthly runoff. The future watershed runoff was simulated for the periods 2020-2050 and 2070-2100 using the calculated precipitation and temperature for the two socio-economic greenhouse gas emissions scenarios (SRES A2 and SRES B2). The results indicated that, following the decline of monthly precipitation and the rise of monthly temperature, the watershed annual runoff would decrease, on average, by about 20-25 millimeters per year for the medium-term future period (2020-2050) (Table 2). The annual runoff would continue to decrease by 35 millimeters for the long term period (2070-2100). This result indicated that watershed annual runoff for the period 2070-2100 would be almost half of the watershed runoff for the historical base period (Table 2).

Table 2. Average monthly watershed runoff for the historical and future periods and for the two socioeconomic scenarios

Month	1960-90 Average		SRES A2 Average		SRES B2 Average	
	Observed (mm)	Simulated (mm)	2020-50 (mm)	2070-100 (mm)	2020-50 (mm)	2070-100 (mm)
October	3,27	3,28	2,85	2,34	2,73	2,42
November	3,67	4,29	3,46	2,66	3,29	2,84
December	5,68	6,60	4,56	3,04	4,29	3,45
January	9,14	9,99	6,79	3,95	6,43	4,77
February	10,88	11,56	6,49	3,90	5,41	4,22
March	11,12	11,55	5,96	3,55	3,46	3,04
April	8,50	8,21	5,35	3,41	3,56	3,05
May	5,35	4,65	3,83	2,99	3,40	2,95
June	3,76	3,50	3,16	2,69	2,95	2,70
July	2,90	2,95	2,79	2,44	2,66	2,46
August	2,67	2,72	2,55	2,23	2,44	2,26
September	2,80	2,61	2,44	2,13	2,34	2,16
Year	69,74	71,91	50,25	35,33	42,95	36,33

3.3 Water Demands

Agricultural water demands were estimated using the Near Irrigation Requirement method (Dastane, 1974) for the historic and future periods. Two water demands scenarios have been developed. The first scenario is the base scenario, which represents the water demands for the current cropping pattern, water distribution system and irrigation methods. The second scenario involves the replacement of the water consuming cultivation of cotton with wheat leaving of 10% of the agricultural area fallow, the improvement of the water transfer system and the limitation of water losses and the replacement of water consuming irrigation methods with drip irrigation. The change of the crop cultivation pattern conforms to the new European Union Common Agricultural

Policy. For this second scenario water losses factor was reduced to 1.38 from 1.49 for the first scenario.

The current cropping conditions, with the domination of cotton cultivation throughout the Thessaly plain, lead to large water demands during summer months. With the replacement of cotton with wheat, and the improvement of irrigation and water transfer methods, water demands become relatively smaller and more equally distributed throughout a year for the two future time periods (Fig. 2).

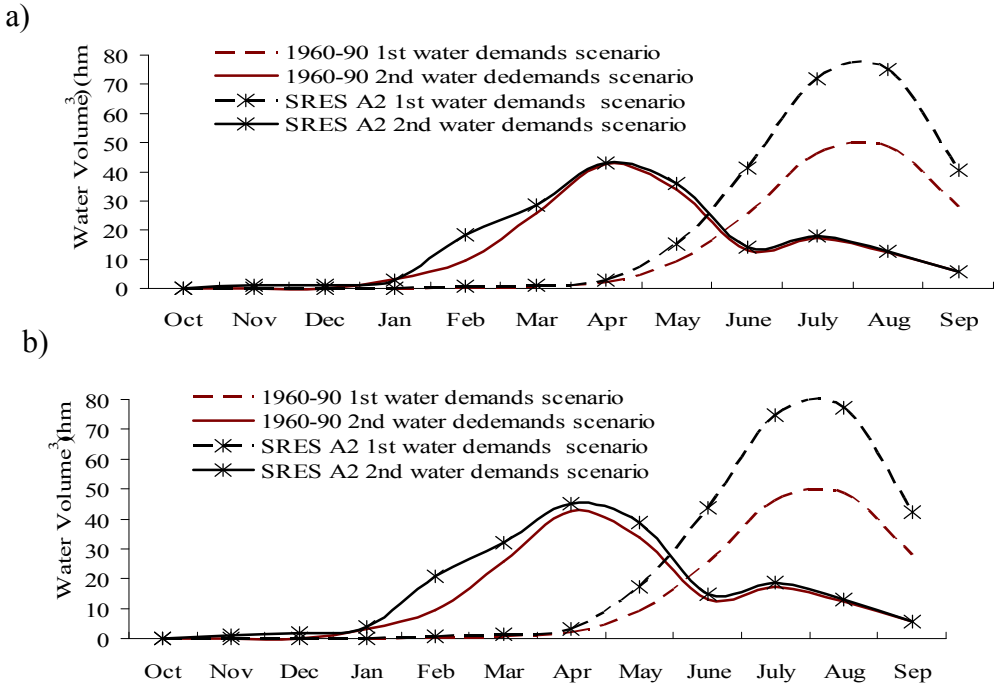


Figure 2. Average water demands for the two water demand management scenarios for the historical and future periods for SRES A2, a) 2020-2050, b) 2070-2100.

3.4 Reservoir Operation Modelling

The reservoir operation model used in this project (Loukas *et al*, 2007a) balances the inputs to reservoir or lake (watershed runoff, precipitation, water transfers to reservoir) and the outputs (water withdrawals, evaporation, deep percolation) and estimates the reservoir water volume and the overflow. The Lake Karla reservoir operation was simulated using this model and the results of the previous analysis for the historic and future time periods. The water transfer by Pinios River was estimated according to river flow at a specific cross section, the pumping and conveyance efficiency of the channel network for the historic and future periods (Patsonas 2008). The precipitation and watershed runoff were the stochastic inputs to the reservoir model, estimated as previously presented for the historical and future periods. Finally, the desired water withdrawals were equaled to the estimated agricultural water demands. Figure 3 shows typical variations of reservoir water storage. The reservoir operation was evaluated through this stochastic simulation by assessing the probability of satisfying the desired water demands (i.e. water withdrawals) and the probability of the reservoir water level being lower than the limited water level set for reservoir ecosystem preservation.

4. RESULTS

Average water demand for the first water demand management scenario and the historic period 1960-90 has been estimated around 160 million m^3 distributed in the summer months. The total

useable water volume of the reservoir is 120 million m^3, thus reservoir water level reaches the limited water level almost every year. Total water demand, for the historic period 1960-90 and the second water demand management scenario remains unchanged compared to the first scenario. However, the distribution of water demand over longer periods within a year, leads to a larger water volume stored in the reservoir within a year and the reservoir water level to reach less frequently the lowest water level for reservoir wetland preservation (Fig. 3a).

For the two future periods (i.e. 2020-50, 2070-2100), the increase of water demand could not be satisfied from the reservoir withdrawals, due to reduced water inputs (i.e. watershed runoff and River Pinios flood water) to the reservoir and as a result, the reservoir water level frequently reaches the lowest water level within a year and the water stored in the reservoir would be significantly reduced even for the second water demand scenario (Fig. 3b and 3c).

The above findings are clearer in the results of stochastic analysis and the estimation of the probability of desired water demand satisfaction from the Lake Karla reservoir and the probability of the reservoir water level been higher that the lowest water level set for wetland preservation (Tables 3 & 4). The probability of desired water demand satisfaction from the Lake Karla reservoir would decrease for the two future time periods, been lower at the end of the century (Table 3). The decrease would be smaller for the second water demand scenario because of the smaller desired water demand (Fig. 2). However, the variation of the results would be larger because of the distribution of the water demand over larger period (Fig. 2). Similar results have been found for the probability of reservoir water level been higher that the lowest water level (Table 4).

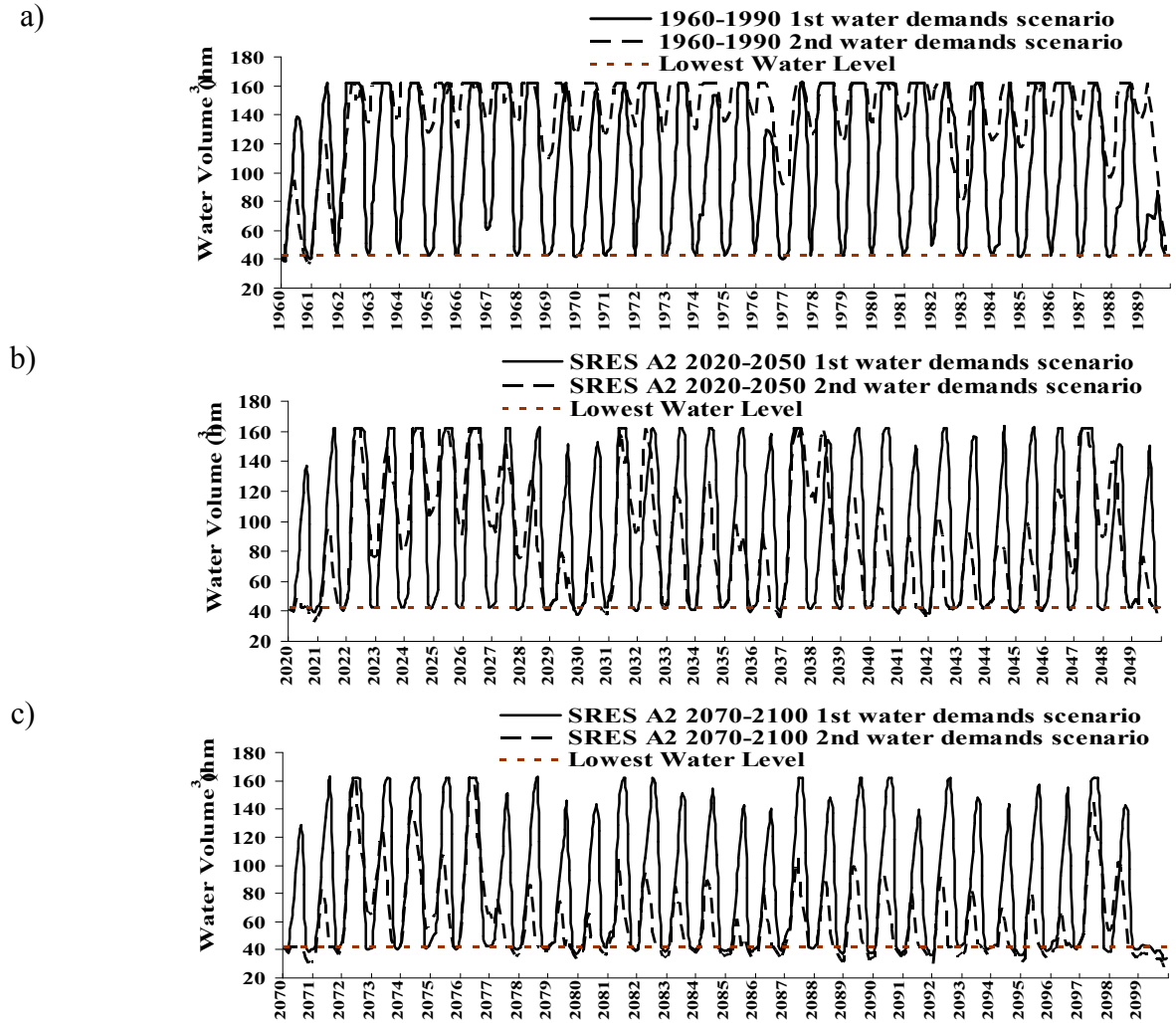


Figure 3. Typical variation of reservoir Lake Karla water storage for scenario SRES A2, (a) for the historic period, (b) 2020-2050, (c) 2070-2100 and the two water demand management scenarios

Table 3. Probability of desired water demand satisfaction for historical and future time periods and water demand scenarios

	1st scenario	2nd scenario	1st scenario	2nd scenario	1st scenario	2nd scenario
	1960-1990		2020-2050 (SRES A2)		2070-2100 (SRES A2)	
Average	0,8971	0,9851	0,7942	0,8211	0,7475	0,6002
St. Deviation	0,0044	0,0047	0,0039	0,0245	0,0079	0,0276
Max	0,9083	0,9944	0,8028	0,8611	0,7611	0,6583
Min	0,8889	0,9722	0,7861	0,7833	0,7361	0,5611
	1st scenario	2nd scenario	1st scenario	2nd scenario	1st scenario	2nd scenario
	1960-1990		2020-2050 (SRES B2)		2070-2100 (SRES B2)	
Average	0,8971	0,9851	0,7872	0,6539	0,7721	0,5580
St. Deviation	0,0044	0,0047	0,0049	0,0265	0,0050	0,0186
Max	0,9083	0,9944	0,7944	0,6861	0,7806	0,6000
Min	0,8889	0,9722	0,7806	0,6111	0,7639	0,5278

Table 4 Probability of reservoir water level been higher that the lower water level for historical and future time periods and water demand scenarios

	1st scenario	2nd scenario	1st scenario	2nd scenario	1st scenario	2nd scenario
	1960-1990		2020-2050 (SRES A2)		2070-2100 (SRES A2)	
Average	0,9577	0,9833	0,8516	0,8368	0,7411	0,5845
St. Deviation	0,0100	0,0060	0,0174	0,0331	0,0270	0,0462
Max	0,9806	0,9972	0,8750	0,8833	0,8111	0,6778
Min	0,9278	0,9667	0,8111	0,7806	0,7000	0,5083
	1st scenario	2nd scenario	1st scenario	2nd scenario	1st scenario	2nd scenario
	1960-1990		2020-2050 (SRES B2)		2070-2100 (SRES B2)	
Average	0,9577	0,9833	0,8370	0,6820	0,7926	0,5789
St. Deviation	0,0100	0,0060	0,0172	0,0386	0,0236	0,0331
Max	0,9806	0,9972	0,8694	0,7333	0,8389	0,6556
Min	0,9278	0,9667	0,8083	0,6111	0,7417	0,5222

5. CONCLUSIONS

Stochastic simulation provides a powerful decision-making tool through quantifying uncertainty in reservoir operation. In this study, the source of uncertainty is assumed to be linked only to precipitation, runoff and climate change. The results of this study indicate that the Lake Karla reservoir could satisfactory cover the irrigation water demand under current climate without endangering the preservation of the reservoir wetland. However, there would be significant probability that the reservoir could not satisfy the desired water demands in the future due to climate change, even for the improved water demand scenario. In this case, the reservoir wetland could be put in danger due to the frequent lowering of the reservoir water level below the preservation limit set. Thus, additional measures should be taken in order to either further reduce the water demands or increase the availability of water resources in the area.

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STUDY, ANALYZES AND FORECAST OF SURFACE WATER - LAKHMES DAM

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Abstract: In the present study, we tried to show the importance of the stochastic modeling of the monthly surface water of the Lakhmes dam, 36 years of observation, for the management and the exploitation of dam storage to the profile of an irrigated perimeter of 474 ha. The catchments area of the study is in the Siliana region, Northern western of Tunisia, in semi-arid zone with relief very contrasted and with cereal vocation. The rainfalls average lies between 400 and 450 millimeters .It is characterized by one wet and cold winter and a dry and hot summer. This work made the object of the bibliographical analysis, the investigations of ground and the collection and the data analysis. Thus, following the exploratory statistical analysis, we studied the normality and the stationary of the surface water series on a monthly scale. The series observed presents a non normal behavior. We highlighted a control break in the long-term behavior of the series. The series, standardized and transformed, was modeled by an auto-regression model with moving average. The tests of validation of the model selected are conclusive. This attempt of modeling of surface water by the stochastic approach seems to be interesting for a rigorous management and optimal exploitation of water resources, useful for the perimeter irrigated.

Key words: Catchments, dam, forecast, model, stationary, water inflow.

1. INTRODUCTION

The monthly water inflows to the Lakhmes dam are known historically. However, the management of the water resource requires the knowledge of the behavior and evolution of water inflow in the medium and long term. The time series analysis of the monthly water inflow is of an interest as well for management as for the development of the forecasting models.

2. CHARACTERISTICS OF THE CATCHMENTS AREA

The area catchment's of wadi Lakhmes has a surface of 162 km ². It is of form circular and present a broken relief made up of assembly lines, plates and plains.

The basin is in semi arid zone. The Lakhmes dam is frequently subjected to deficits of more or less important of water inflow. For this purpose, the dam is reinforced by two sources of water namely:

- The connection of wadi Bouabdallah, area catchment of 31 km2.
- The construction of a pumping station which drives back with the Lakhmes dam them
- water of believed starting from an other larger wadi named wadi ouzafa. The figure 1 hereafter, represents the site of the hydraulic structures in the catchment area of the lakhmess dam.

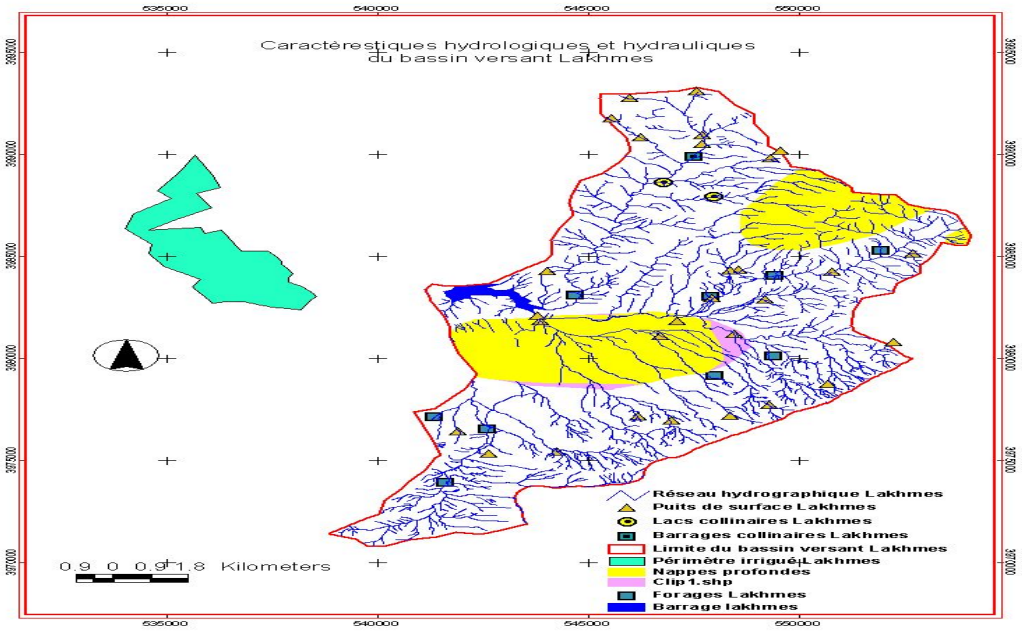


Figure 1. Watershed and sites of hydraulic structures.

3. OBJECTIVES

The main objectives of the study are:

- Analysis of the monthly water inflow of the Lakhmes dam,
- The proposal for a forecasting model.

4. METHODOLOGY

Our approach is based on an exploratory analysis of the series, the detection of segmentation and the analysis of the stationarity.

4.1 Data

The data base used in this study were obtained from the hydrological directories established with the direction of the water resources (DG/RE) and from the farm operating of the dam. The monthly water inflows with the Lakhmess dam are calculated from the elements provided by the farm operating of the dam. The equation of the assessment is given by:

$$\mathbf{A= C+P+L+F+E+D+DV} \quad (1)$$

A: water inflow, C: variation of the capacity, P: pumping in selected, L: release F: water loss, E: loss evaporation, D discharge by the spillway, FD: unsilting.

4.2 Precipitations

The average of the monthly rains is of 38.09 mm. The standard deviation is equal 39mm and the coefficient of variation is equal 0.96 mm. Figure 2 shows the evolution of the rains over one period from 1969 to 2004.

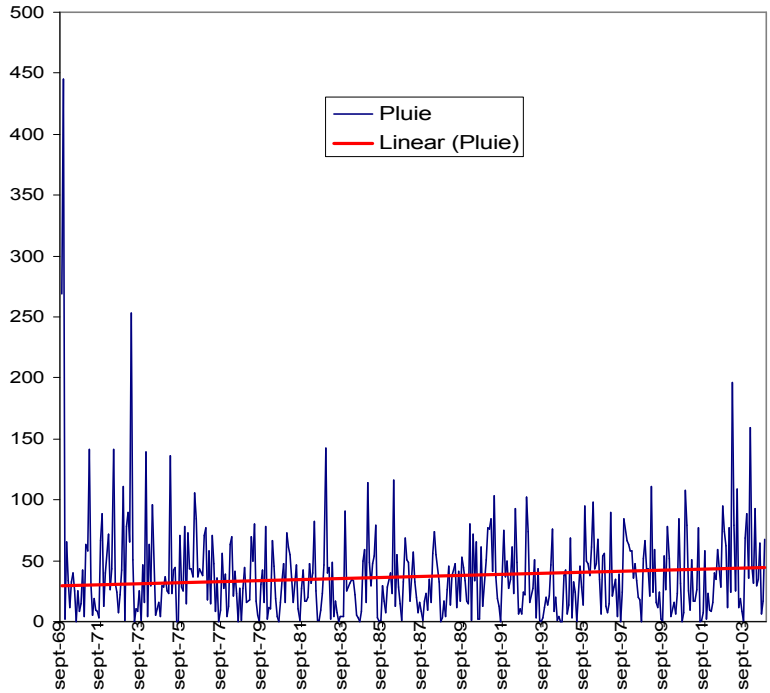


Figure 2. Evolution of rains, observed period 1969-2004.

According to the graph, we notice the appearance of the very high peaks at the beginning of the period of observation. The area of Siliana in general and lakhmess in particular is characterized by periods of rather frequent and more or less prolonged drynesses. The meteorological dryness constitutes a phenomenon scour with 63% of dry years. As for the hydrological dryness it then is accentuated than the weather dryness (Bergaoui, Med. and Al, 2001).

5. TIME SERIES OF THE MONTHLY WATER INFLOWS ANALYZES

Table 1 shows the values of the dispersion and location parameters. The coefficient of variation is high what testifies to an important irregularity of the contributions. It makes it possible to measure the degree of variability in a series and dispersion of the values compared to the average. The difference between the location parameters is important and the series of the water inflow does not follow a normal distribution.

Table 1. Dispersion and location parameters of the series water inflow

Parameter	***Available sample***	***mean***	***median***	***minimum***	***maximum***	***variance***	***standard deviation***	***CV***
values	*420*	*5.28*	*1.92*	*0.0*	*96.76*	*101.76*	*10.09*	*1.91*

5.1 Normality

Thus, to standardize the series, we proceeded by transformation. Several tests were carried out. It is mainly about the transformation of Box cox. It's given by

$$T_t = \left(\frac{A_t^{\lambda} - 1)}{\lambda}\right) \qquad \textbf{(2)}$$

At : observed time series

λ : a transformation parameter

The three main reasons for making a transformation are as follows. To stabilize the variance, to make the seasonal effect and to make the data normally distributed. For the value of λ = 0.20, the standard deviation clearly decreased (table n° 2); indeed it passed from 5.279 (observed series: SB) with 2.152 (transformed series: ST), which means that the transformation generated a damping of the irregularity of water inflow.

Table 2. Dispersion and location parameters of the series water inflow

Parameters	**Taille**	**mean**	**median**	**Variance**	***Standard deviation***	**CV**
S B	420	5.28	1.92	101.76	10.88	2.06
S T	420	4.98	0.7	5.09	2.26	0.45

The transformation of "Box Cox" allowed having a distributed series following the normal law, figure 3. Transformed series, λ = 0.20

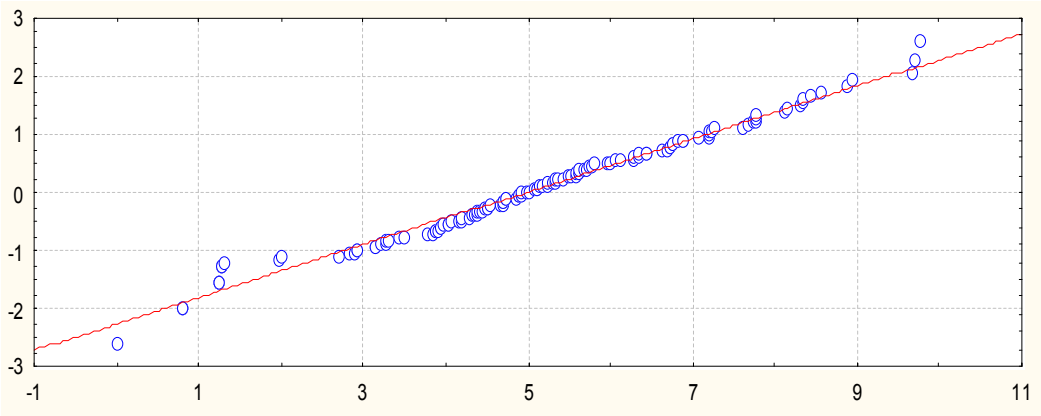


Figure 3. Normal distribution of water inflows series.

Having a series which is not stationary, we proceeded as follows to make it stationary.

5.2 Segmentation of water inflows series.

It is necessary to be ensured is what there exists or not a control break in the time series. It is a question of checking if there exists a change of the law of probability in the series; for this purpose one applied the method of segmentation of series (Hubert., and Al 1989) what is adapted in the search of multiple changes of average in the series. In our case, the segmentation of Hubert watches the existence of several segmentations which are different from/to each other. It has a significant difference between the successive averages, table n° 3. Thus and with an aim of envisaging values at the horizon twelve, we retained the going segment of the series of 1973 up to 2004. is 136 successive monthly values and having the standard deviation and weakest. It is about a series of 136 successive monthly values.

Table 3. Procedure of segmentation of the water inflow series at lakhmess dam.

beginning	**end**	**mean**	**Standard deviation**
1	2	85.750	17.041
3	42	7.510	7.090
43	44	63.700	8.910
45	51	25.300	9.691
52	78	8.930	8.810
79	81	39.000	6.521
82	280	4.278	5.377
281	283	28.533	1.882
284	**420**	**3.140**	**5.279**

To study the forecasting model of water inflow series,. We decide to retain the series of the monthly water inflow (136 values) going from April 1993 till August 2004. The calculated statistical parameters, for both series, are carried in table 4.

Table 4. Comparison of parameters.

Parametres	sries SB	Segment
Mean	5.277	3.140
Standard deviation	10.088	5.279
Coefficient of variation	1.911	1.68
Available sample	420	136

We notice that the truncated series, the segment, presents a net decrease to the levels of these parameters (average, standard deviation, coefficient of variation). The truncated series follows well a normal law.

5.3 Identification of the form of the model

The graph of the evolution of the standard deviation according to the average, for the series of water inflows, shows that there is a linear regression between the two variables. The coefficient of regression is different from zero. The seasonal movements have an amplitude proportional to the trend, therefore it is indeed a multiplicative model.

The model is given by:

$$At = Trend * C * S * \varepsilon \quad (3)$$

At: water inflow at time t
C: is the cyclical component, S: is the seasonal component
ε: is the residual component.

5.4 Stationary Time Series

Functions ACF and PACF of the Transformed series S2 (λ=0.2).

Time series is said to be stationary if there is no systemic change in mean, no systemic change in variance and if strictly period variations have been removed (Chatfield C. 1989).

The identification of the term of autocorrelation (AR) and moving average (MA) are done starting from the respective curves of the partial autocorrelation function PACF and autocorrelation function ACF. The analysis of the graphs of the ACF and PACF enabled us to conclude that, the series presents a trend which translates the general evolution of the monthly water inflow. Consequently we are going to apply the smoothing by mobile average of order 12 to estimate the values of this trend.

The functions of autocorrelation (ACF), figure 4m and partial autocorrelation (PACF),figure 5, confirm the existence of a trend and of dominant periodicity what translates the existence of a striking seasonal effect.

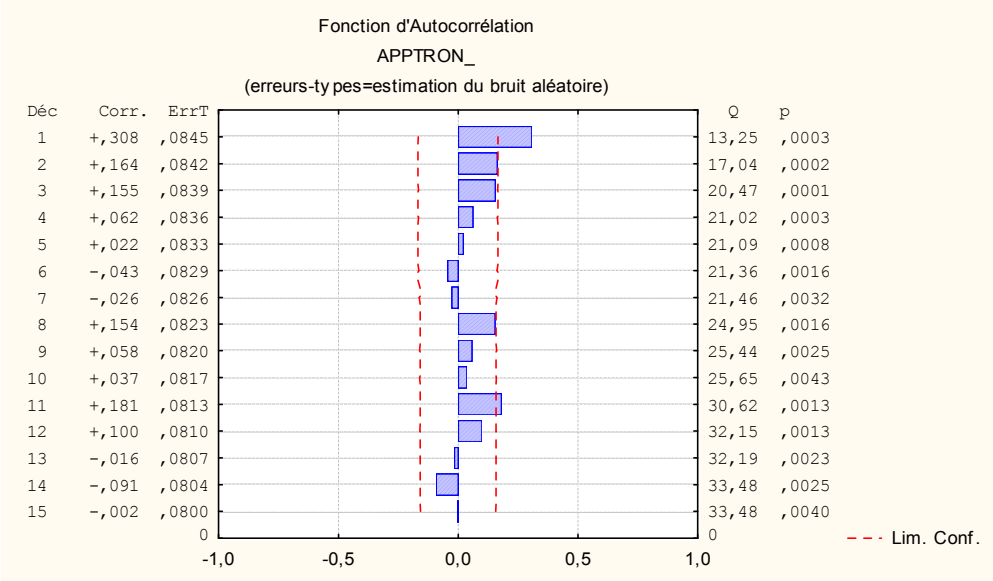


Figure 4. ACF Function.

The analysis of the ACF and the PCAF, shows us that their values of the coefficients of correlation present oscillations with a very slow diminution what proves a non-stationary series.

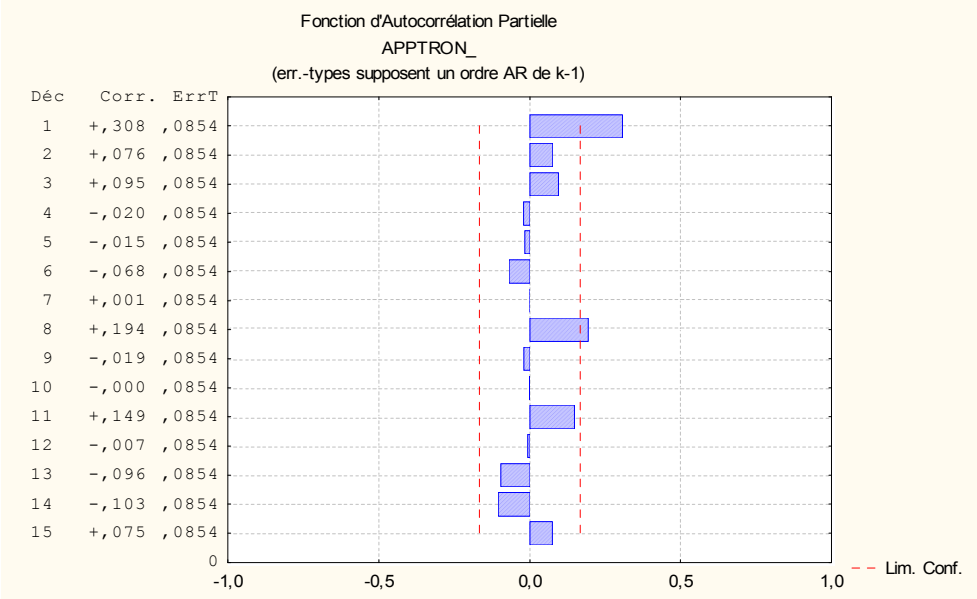


Figure 5. PACF Function.

It is a question of applying adequate transformations to make it stationary. Indeed, after standardization, one has to estimate and eliminate the two components, seasonal component and the trend. The various transformations which the series of the monthly water inflow underwent (standardization, deseasonalization, differencing first) made it possible to make it stationary. Lastly, the application of a forecasting integrated model ARIMA (1,1,1), Box and Jenkins 1976, appears to agree with the series of the water inflow. The residues are compared to a white vibration. A model is called integrated model because the stationary model is fitted to the differenced data. A special type of filtering, which is particularly useful for removing a trend, is simply to difference a given times series until it becomes stationary.

6. CONCLUSION

The statistical analysis of the series of water inflow in Lakhmes dam allowed to confirm that it is not distributed by the normal law. It presents a trend and a dominant periodicity. The series is non-stationary. The transformation of the series followed by a first differentiation appears to indicate q' a model of type ARIMA (1,1,1) can be convenient for the forecast of the monthly water inflow. Finally, in spite of the stormy rains with strong intensity, the dam Lakhmes received only one Mm3 from sediments since 1966 until 2004; what represents approximately 1.63 m3 / ha / year, that is 0.163 mm / ha / year .The rate of sedimentation, is very weak. It is explained by the watershed management. It is important to indicate that the hydrological behavior of the watershed, further to catchment management, has changed.

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MULTI-OBJECTIVE MANAGEMENT OF COASTAL AQUIFERS IN SEMI-ARID REGIONS UNDER DROUGHT CONDITIONS

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Abstract: **G**roundwater is the main water recourse in many semi-arid coastal regions and water demand, especially in summer months, can be very high. Groundwater withdrawal for meeting this demand often causes seawater intrusion and degradation of water quality of coastal aquifers. In order to satisfy demand, a combined management plan is proposed and is under consideration in the island of Santorini. The plan involves: i) desalinization (if needed) of pumped water to a potable level using reverse osmosis and ii) injection into the aquifer of biologically treated waste water. The management plan is formulated in a multi-objective optimization framework, where simultaneous minimization of economic and environmental costs is desired; subject to a constraint so that cleaned water satisfies demand. The decision variables concern the well locations and the corresponding pumping and recharging rates. The problem is solved using a computationally efficient multi-objective genetic algorithm (NSGAII). The constrained multi-objective optimization problem is transformed to an unconstrained one using a penalty function proportional to constraint violation. This extends the definition of the objective function outside the domain of feasibility. The impact of prolonged droughts on coastal aquifers is investigated by assuming various scenarios of reduced groundwater recharge. Water flow and quality in the coastal aquifer is simulated using a three-dimensional variable density finite difference model (SEAWAT). The method is initially applied to a test aquifer and the trade-off curves (Pareto fronts) are determined for each drought scenario. The trade-off curves indicate an increase on the economic and environmental cost as groundwater recharge reduces due to climate change.

Key words: Coastal aquifers, seawater intrusion, aquifer management, pumping optimization, genetic algorithms

1. INTRODUCTION

Groundwater is the main water recourse in many semi-arid coastal regions and water demand, especially in summer months, can be very high. Groundwater withdrawal for meeting this demand often causes seawater intrusion and degradation of water quality of coastal aquifers. Therefore groundwater management is of paramount importance in many coastal areas and islands. One of the main objectives is maximization of pumped water subject to constraints which protect the aquifer from seawater intrusion. A common approach is to combine numerical models, simulating the physical system, with efficient optimization algorithms to perform the optimal search. Some works are based on sharp interface models (Mantoglou 2003; Mantoglou et al. 2004; Park and Aral 2004; Karterakis et al. 2007; Mantoglou and Papantoniou, 2008), while others use 3D variable density models (Das and Datta 1999; Rao et al. 2004; Kourakos and Mantoglou 2009).

However, maximizing pumping may not a viable alternative in some cases, for example in cases where the maximum allowable amount of water does not satisfy demand. In order to tackle this problem, a method based on desalination of brackish water using reverse osmosis is proposed. Further, the possibility of improving the aquifer water quality by injecting biologically treated waste water (WWT) is investigated. Injecting treated waste water appears to have many benefits such as restoration of groundwater levels, prevention of saline water intrusion, water accumulation to balance seasonal differences in water supply, improvement of water quality and flexibility in aquifer management, etc. (Li et al. 2006). Several researchers have studied management plans based on WWT (Georgopoulou et al. 2001; Li et al. 2006; Lopez and Vurro 2008).

This study develops a pumping management methodology which is based on desalination of pumped brackish water and on recharging the aquifer with treated waste water. The management methodology is based on a multi-objective optimization (MOO) framework where minimization of two objectives related to economic and environmental costs is sought. An efficient MOO algorithm is combined with a 3D variable density numerical model predicting salinity distribution in the aquifer. Further, a sensitivity analysis is performed where the impacts of extreme droughts are investigated.

This paper is organized in 5 Sections. Section 2 presents a short description of the problem and the governing equations of flow and solute transport in coastal aquifers. Section 3 presents the management problem and outlines the optimization methodology. Section 4 presents some preliminary applications of the methodology and Section 5 summarizes the conclusions of this study.

PROBLEM DESCRIPTION AND GOVERNING EQUATIONS

Coastal aquifers in semi-arid regions are under severe exploitation and seawater intrusion is a major problem. In some cases, freshwater demand may exceed the aquifer capacity. In order to overcome this problem, a management scheme is proposed where pumped brackish water is desalinated. Further, the quality of aquifer water is improved by injecting biologically treated waste water into the aquifer. According to this WWT scheme, waste water is treated to a potable level and is injected back into the aquifer at appropriate locations. ***Figure 1*** outlines the proposed system. This plan hopefully results in a viable plan where the required demand Q_{dem} is satisfied without the need to transport water from elsewhere, while the aquifer is protected. If salt concentration of pumped water is greater than the maximum allowable concentration of potable water $c^{\max}$, pumped water is desalinated using a reverse osmosis procedure (RO).

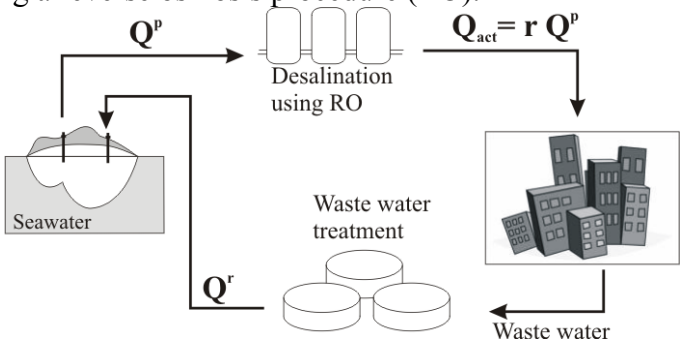


Figure 1 Outline of management plan

The supply of desalinated water Q_{act} obtained by RO, is less than the water pumped Q_p. In fact, as salt concentration in pumped water is increased, the amount of desalinated water obtained by RO is reduced. The actual amount of desalinated water obtained by RO, Q_{act}, is a function of salinity and is given by:

$$Q_{act} = \sum_{i=1}^{N_w^p} r_i \cdot Q_i^p \tag{1}$$

where Q_i^p are the pumping rates of well i, N_w^p is the number of pumping wells and r_i is a recovery ratio of RO, expressing the ratio of desalinated water volume to the brackish water volume. Vectors $\mathbf{r}$ and $\mathbf{Q}^p$ in ***Figure 1***, are given by $\mathbf{r} = \left[r_1, r_2, \ldots, r_{N_w^p} \right]$ and $\mathbf{Q}^p = \left[Q_1^p, Q_2^p, \ldots, Q_{N_w^p}^p \right]$ respectively. Depending on the technology used, the recovery ratio may differ. Here, the following empirical expression proposed by Georgopoulou et al. 2001, is used

$$r_i = \begin{cases} 1 & \text{if } c_i \le 0.5\, kg/m^3 \\ \dfrac{-9.27 \cdot c_i + 265.1}{-5.27 \cdot c_i + 322.4} & \text{if } 0.5 < c_i \le 10\, kg/m^3 \\ 0 & \text{if } c_i > 10\, kg/m^3 \end{cases} \quad (2)$$

where c_i the salt concentration in pumped water at well i.

Thus, in order to choose an appropriate pumping strategy to satisfy demand, the salt concentration in pumped water is required. Salt concentration depends on the pumping strategy through the mechanisms of salt dispersion in the aquifer. Notice that coefficients r_i in (2) are not known constants but they depend on concentrations c_i which in turn depend on flow rates Q_i^p and Q_i^r through the salt transport processes in the coastal aquifer. Therefore, in order to solve the management problem, an accurate simulation of the physical system is needed. Here, each management solution is evaluated using a 3D variable density model in order to calculate salt concentration in the aquifer and to determine the amount of clean water Q_{act} produced by the RO system.

The governing equations that describe groundwater flow in a 3D variable density regime are described in terms of two partial differential equations, the equation of mass balance of water and the equation of solute transport (see Kourakos and Mantoglou, 2009). The first is expressed as:

$$\frac{\partial (n\rho)}{\partial t} = -\nabla \cdot (\rho \mathbf{q}) + \bar{\rho} q_s \quad (3)$$

where ρ is the variable fluid density, $\mathbf{q}$ is the specific discharge vector, $\bar{\rho}$ is the density entering from a source or leaving through a sink, q_s is the volumetric flow rate of sources and sinks per unit aquifer volume, n is the aquifer porosity, and t represents time. The equation of solute transport is expressed as:

$$\frac{\partial c}{\partial t} = \nabla \cdot (\mathbf{D} \cdot \nabla c) - \nabla (\mathbf{v} c) - \frac{q_s}{n} c_s \quad (4)$$

where $\mathbf{D}$ is the hydrodynamic dispersion tensor, $\mathbf{v}$ is the fluid velocity vector, c is the solute transport concentration, c_s is the source/sink solute concentration. These two partial differential equations are coupled because of the dependence between Darcy's velocity, pressure, density and concentration. In order to solve the problem, two more equations are needed. The Darcy law of variable density flow is given by:

$$q_x = -\frac{k_x}{\mu} \frac{\partial p}{\partial x}, \quad q_y = -\frac{k_y}{\mu} \frac{\partial p}{\partial y}, \quad q_z = -\frac{k_z}{\mu} \left(\frac{\partial p}{\partial z} + \rho g \right) \quad (5)$$

where p is fluid pressure, q_x, q_y, q_z are the components of specific discharge, μ is dynamic viscosity, k_x, k_y, k_z the intrinsic permeabilities on the x, y, z axes assumed principal, and g is the gravity acceleration. The fluid density and concentration are related by the following equation:

$$\rho = \rho_o (1 + \varepsilon \cdot \bar{c}) \quad (6)$$

where ρ_o is the freshwater density, $\bar{c}$ a normalized salt concentration and ε is the density difference ratio expressed as $\varepsilon = (\rho_s - \rho_o)/\rho_o$.

The above system can be solved using numerical methods. Here, the SEAWAT code (Guo and Langevin 2002), which is based on finite difference approximations, was used. SEAWAT solves equations 4 and 5 through a synchronous time-stepping approach which cycles between solutions of flow equation and solutions of transport equation.

2. MANAGEMENT PROBLEM AND OPTIMIZATION METHOD

The decision variables of the problem concern the pumping and recharging rates and the locations of wells. There exist many alternative solutions, with different economic and environmental costs, which satisfy demand, i.e. $Q_{act} \geq Q_{dem}$. In order to choose an optimal strategy we develop a multi-objective formulation with two objectives regarding minimization of economic and environmental costs. Mathematically this is stated as:

$$\min_{\mathbf{Q}} P_{env}(\mathbf{Q}) = \sum_{i=1}^{N_w} c_i \quad \text{and} \quad \min_{\mathbf{Q}} P_{mon}(\mathbf{Q}) = \sum_{i=1}^{N_w^p} \mathsf{z}_p \cdot \left|Q_i^p\right| + \sum_{i=1}^{N_w^r} \mathsf{z}_r \cdot \left|Q_i^r\right|$$

$$\text{subject to:} \quad \sum_{i=1}^{N_w^p} r_i \cdot Q_i^p \geq Q_{dem} \qquad (7)$$

$$Q_{\min} \leq Q_i \leq Q_{\max}$$

where P_{env} and P_{mon} specify the environmental and economic costs respectively. The two costs are expressed in terms of environmental and monetary units EU and MU respectively. Concentration c_i refers to i^{th} well, $i \in [1, N_w]$, N_w is the total number of wells, N_w^r is the number of recharging wells ($N_w^p + N_w^r = N_w$), Q_i^r are the recharging rates of well i, and z_p, z_r represent the cost of P&T and WWT per unit flow respectively. The first constraint in (7) ensures that the specified demand is satisfied while the second constraint, bounds pumping and recharging rates respectively depending on the technology used. Throughout this study, negative rates correspond to pumping while positive rates to recharging. Hence $Q_{\min}$ is the maximum pumping and $Q_{\max}$ is the maximum recharging rate for each well.

Problem (7) is a constrained multi-objective optimization problem with two objectives. Notice that coefficients r_i in (7) are not known constants but they depend on concentrations c_i which in turn depend on flow rates Q_i^p and Q_i^r through the salt transport processes in the coastal aquifer. This creates a complex optimization problem requiring special consideration.

Since evolutionary algorithms cannot handle constraints, a transformation of the problem into an unconstraint one, using a penalty function proportional to the violation of constraints, is proposed. The modified optimization problem is now expressed as:

$$\min P_{env} \quad \text{and} \quad \min P_{mon} \quad \text{if } Q_{act} \geq Q_{dem}$$

$$\min (Q_{dem} - Q_{act}) \quad \text{if } Q_{act} < Q_{dem} \qquad (8)$$

The above reformulation (8) has the effect of extending the definition of the objective function outside the domain of feasibility. Note that when Q_{act} does not satisfy demand, the problem is converted to a single objective optimization problem with the goal is to find solutions that satisfy demand regardless of environmental or economic costs. The second constraint of problem (7) is handled through the genetic operators, where individuals that exceed these values are confined to the limits.

Calculation of the two objective functions requires precise knowledge of salt concentration in the pumped water. This is obtained using a 3D variable density numerical model as described above, simulating the aquifer response to different pumping and recharging strategies. Notice that simpler simulation models, such as sharp interface models, cannot be used in this application.

The multi-objective optimization problem (8) was solved using the computationally efficient algorithm NSGAII (Deb et al. 2002) which follows the typical genetic algorithm flow. First, a random initial population spanning the whole domain is generated. The population is evaluated for the two objective functions, and each solution is initially assigned a rank, based on a domination

criterion, followed by a crowding measure expressing how close the solutions are to each other. Then, a selection operator is applied to select the best individuals. The selected individuals are combined using crossover and mutation operators in order to produce the new generation. The new generation is evaluated and ranked and the evolution continues towards the set of optimal Pareto solutions until stopping criteria are met. A typical stopping criterion is a maximum number of generations.

The objective function in problem (8) is a hybrid problem. Hence the default ranking scheme of NSGAII is modified as follows. After evaluation, each individual not satisfying the first constraint of (7), is assigned a rank depending on the constraint violation such as in (8). Thus the rank depends on the deviation from the requirement to satisfy demand. The remaining population which satisfies demand is ranked based on the classical NSGAII ranking scheme.

After ranking the population, a selection method is applied in order to select the fittest individuals. In typical MOO, individuals with lower ranks are preferred. Among two individuals with equal ranks the one with the largest crowding measure is preferred. The crowding measure expresses how close the solutions are to each other, and it is used to maintain diversity of population. According to the modified selection method, there is an additional option for solutions not satisfying constraints and solutions satisfying constraints are preferred. When two individuals both violating constraints are compared, the one with smaller violation is selected. Thus the algorithm is guided towards a population which satisfies constraints.

2. APPLICATION

The proposed management and MOO methodology are applied test aquifer as shown in the cross-section of Figure . This aquifer has some related characteristics with the main aquifer in Santorini island. The left and right boundaries correspond to seawater boundaries with salt concentration of seawater, i.e. 35 kg/m^3, while the hydraulic head increases linearly with depth at a rate of $0.025\,m$ per meter depth. The bottom of the aquifer is set at a large depth of 200 m below m.s.l. This has the effect that after a long term simulation, seawater intrudes into the aquifer and the freshwater floats above heavier seawater (Figure a). Other parameters of the aquifer are given in *Table* 1. There are 4 wells installed in the aquifer and the pumps are located 50 m below m.s.l. The aquifer cross-section is descritized into 30 columns and 20 layers of dimensions $200\,m$ and $10\,m$ respectively. The width of the aquifer is set to 200 m descritized in one column, therefore all the subsequent results are actually expressed as per 200 m of aquifer. This system idealization assumes 2D flow and transport, however, on going research considers 3D flow and transport by applying discretization along the aquifer width. Calculation of concentration and hydraulic head is based on the 3D variable density model SEAWAT (Guo and Langevin 2002).

The aquifer is initially simulated for very long period without considering any pumping or recharging until concentration and heads reach steady state. This concentration is illustrated in Figure a. Then an initial single objective optimization problem is solved in order to obtain the maximum extracting capacity of the aquifer. The objective is maximization of the pumping rates subject to constrains maintaining concentration of the extracted water (by the pumps located 50 m below m.s.l.), less than the maximum allowable concentration of potable water $c^{\max}$. The time horizon of each optimization is set to $10000\,days$. Detailed description of the single objective optimization methodology and the corresponding optimization algorithms is given in Kourakos and Mantoglou 2009. The maximum extracted amount of water was found equal to $764\,m^3/day$ while the individual rates of the four wells were determined to 41, 500, 223 and 0 m^3/day respectively (from the left to right in Figure). The concentration field at the end of 10000 $days$ is illustrated in Figure b, based the above pumping schedule. Notice that the iso-concentration surface of $c^{\max} = 0.5\,kg/m^3$ approaches the withdrawal points of wells but does not cross it, since wells are protected by the constraints. Notice that although the extracted amount of water is potable, the

aquifer ends up at a deteriorated state. A new optimization is performed where the concentration field of Figure b is used as initial condition. This is a more realistic scenario in real aquifers. The maximum extracted amount of water is now equal to $197\,m^3/day$ and the individual pumping rates are 32, 66, 99 and 0 m^3/day respectively. In this scenario the allowable amount of pumped water is about 75% less than in the first scenario. This is because the aquifer started in the first scenario from a more favorable initial condition without pumping and part of water was removed from storage. Notice that the end of 10000 *days* with reduced pumping rates, the concentration field did not deteriorate, thus this pumping rate corresponds to the "safe yield" or "sustainable yield" of the aquifer. Thus the maximum amount of water that can be extracted without causing further seawater intrusion is $197\,m^3/day$.

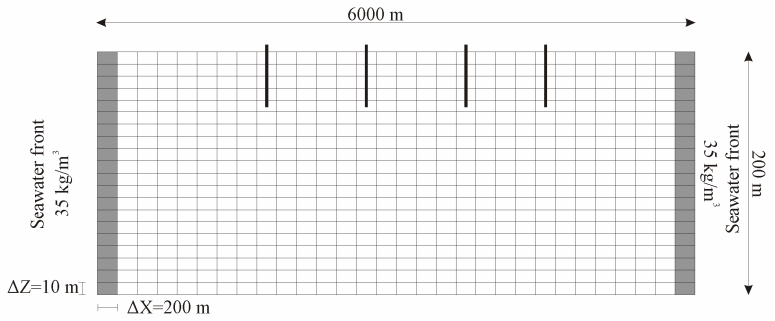


Figure 2. Aquifer description

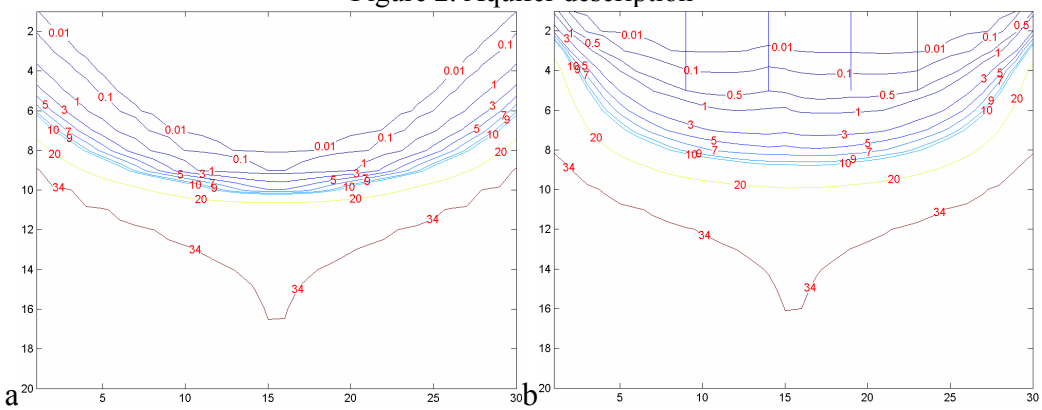


Figure 3. a) Steady state concentration without pumping, b) Concentration corresponding to pumping based on optimal strategy

Table *1 Hydraulic properties of aquifer*

Property	**values**
Horizontal hydraulic conductivity	10 m/day
Vertical hydraulic conductivity	0.1 m/day
Recharge	0.0004 m/day
Longitudinal dispersivity	20 m
Traverse dispersivity	2 m
Vertical dispersivity	0.2 m
Effective porosity	0.25

In the next application, the demand is set equal to $Q_{dem} = 350\,m^3/day$ which exceeds $197\,m^3/day$ which is the extracting capacity of the aquifer determined above. In this case it is not possible to satisfy demand without further degradation of the aquifer. The management plan and the corresponding MO optimization methodology described in Sections 2 and 3 are utilized to solve this problem. The objective is simultaneous minimization of environmental P_{env} and economic costs P_{mon}. In this initial example we assume equal pumping and recharging costs (it is assumed here that $z_p = z_r = 1$, although in reality we expect $z_p > z_r$). The other parameters of MOO problem are set to $N_w = 4$, $Q_{\min} = -300\,m^3/day$ and $Q_{\max} = 300\,m^3/day$.

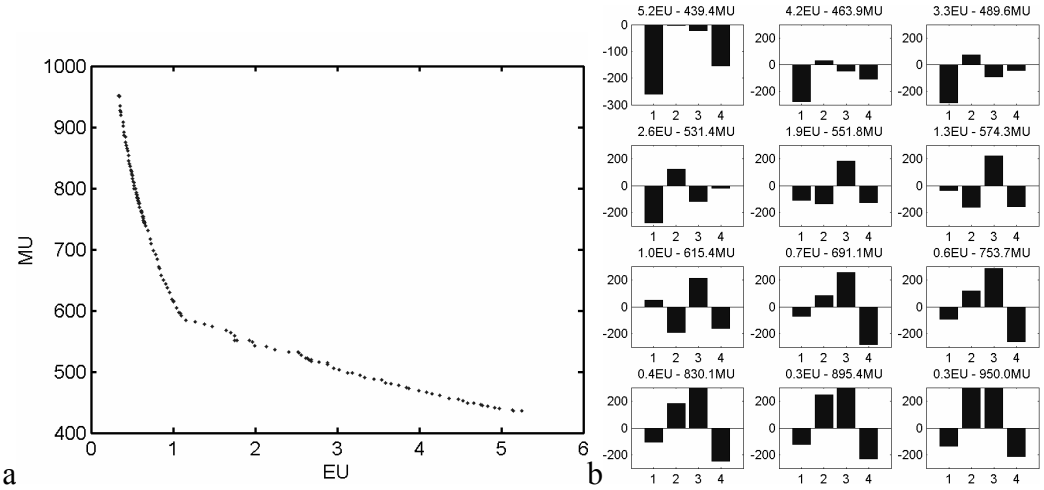


Figure 4. a)Pareto optimal set for groundwater recharge $N = 0.4\,mm/day$. b)Pumping and recharging rates for 12 Pareto optimal solutions corresponding to different costs

After 400 generations, the Pareto optimal set was stabilized and it is plotted in Figure a, while Figure b illustrates graphically the pumping and recharging rates for 12 solutions located on the Pareto optimal set corresponding to different environmental and monetary costs. The top left solution corresponds to pumping from all wells and has the largest environmental cost and the minimum economic cost. It can be seen that the most economic plan is to extract the required amount of water followed by desalination without any recharging. The bottom right plot corresponds to the most expensive solution with the minimum environmental cost. In this solution, the two wells in the center of the aquifer inject clean water with the maximum recharge rate. All other plots correspond to solutions located among these two extreme solutions. Notice that the 12 solutions, as well as all the Pareto solutions of Figure a, satisfy the demand, i.e. $Q_{act} = 350\,m^3/day$.

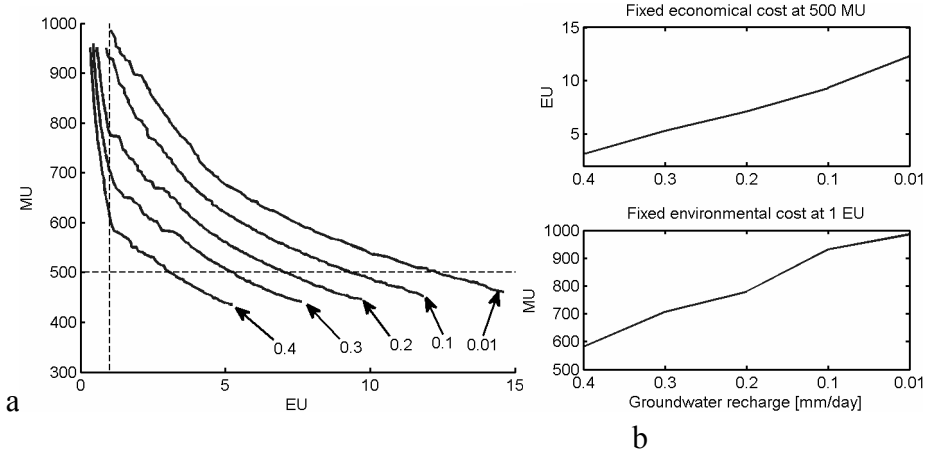


Figure 5. a) Pareto set for various scenarios of groundwater recharge, b) Increase of environmental and economic costs for reduced natural recharge

In order to study the effect of possible climate changes, we considered various scenarios of reduced groundwater recharge and obtain corresponding Pareto sets. Figurea plots the Pareto optimal sets for different drought scenarios. The groundwater recharge value corresponding to each Pareto set, is indicated in Figurea in mm/day. Notice that as groundwater recharge reduces, the economic cost must increase in order to maintain a fixed environmental cost. Conversely, as groundwater recharge reduces, the environmental cost is increased drastically in order to maintain a fixed economic cost. This is further illustrated in Figureb where the increase of environmental cost it is plotted versus groundwater recharge for a fixed economic cost of 500 MU. Similarly the economic cost increases with reduced recharge for a fixed environmental cost of 1 EU. It can be seen that as groundwater recharge is reduced, it becomes very expensive to maintain a high level of quality in the aquifer.

3. CONCLUSIONS

This study investigates a groundwater management plan and develops an optimization method for coastal aquifers under drought conditions. The main goal is to provide a specified amount of freshwater for use. Since in most semi-arid regions the extracting capacity of the coastal aquifers is not sufficient to meet demand a management plan is proposed which involves: i) desalination (if needed) of pumped water to a potable level using reverse osmosis and ii) injection into the aquifer of biologically treated waste water.

The problem is formulated into a multi-objective framework where simultaneous minimization of two objectives (economic and environmental cost) is desired; subject to constraint that the available water should satisfy demand. The decision variables correspond to the locations and pumping rates of the wells. In order to solve the constraint MOO problem using GA, we propose a transformation method to an unconstraint problem where the penalty is proportional to constraint violation. This reformulation extends the definition of the objective function outside the domain of feasibility. In addition a modification on the ranking, selection, and crossover process is proposed to handle the modified unconstraint MOO problem.

Using the proposed optimization methodology, various scenarios of possible reduction of natural groundwater recharge due to climate change were examined. For each scenario, a Pareto optimal set is derived. The comparisons among the Pareto sets indicate that maintaining a certain level of high quality in the aquifer becomes very expensive as groundwater recharge is reduced. On the other hand maintaining the management cost low, causes severe degradation of aquifer quality.

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GROUNDWATER VULNERABILITY ASSESSMENT IN KITI AQUIFER, CYPRUS

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Abstract: Approximately 70% of the population of the earth lives in coastal areas and the majority of these people depend on coastal aquifers for freshwater. For this reason the coastal aquifers are vulnerable to pollution and are now recognized as a crucial arena for future progress towards sustainability in world. This paper deals with the assessment of groundwater vulnerability of Kiti aquifer, South Cyprus. The Kiti aquifer, covering an area of about of 30 km^2, is situated in the southern part of Cyprus. The elevation is 20 m above sea level. The area is characterized by semi-arid climatic conditions. The groundwater resources are related to the Pleistocene coastal plain and the recent Tremithos river valley deposits. The concept of the groundwater vulnerability is based on the assumption that the physical environment may provide some degree of protection to groundwater against human activities. The DRASTIC method is applied to evaluate aquifer vulnerability. The method was developed by the United State Environmental Protection Agency (EPA) as a technique for assessing groundwater pollution potential. The acronym DRASTIC corresponds to the initial of the included seven (7) parameters: Depth (D), Recharge (R), Aquifer media (A), Soil media (S), Topography (T), Impact of the vadose zone media (I), hydraulic Conductivity of the aquifer (C). Determination of the DRASTIC index (DI) involves multiplying each parameter weight by its site rating and summing the total. Based on DI values a groundwater vulnerability map was illustrated, using a Geographical Information System (GIS). The highest vulnerability values in the Kiti area, covering a large part, are associated with shallow aquifers without great depth of the vadose zone. The results provide important information and the vulnerability map could be used from local authorities and decision makers for groundwater resources management and protection zoning.

Key words: Aquifer, Cyprus, DRASTIC method, Kiti, Pollution, Vulnerability, Water quality

1. INTRODUCTION

Groundwater is actually under strong human pressures in the Mediterranean basin. Such pressures include changes in land use, urbanization, intensive agriculture and water demand increase and can cause severe degradation of the quality and quantity of groundwater resources (UNESCO, 1998; Civita, 1994; Polemio, 2005; Polemio et al., 2008).

The most efficient measure to fight against groundwater pollution is to make a plan for integral prevention. The new Directive 2006/118/EC on the protection of groundwater against pollution and deterioration, developed under the Water Framework Directive 2000/60/EC, sets out criteria to assess the chemical status of groundwater bodies. Furthermore, the aforementioned Directives have forced the committing member states to ensure good chemical and ecological status of groundwater. Standing to this description of the problem it seems necessary to individuate and implement a water system management strategy, in order to guarantee sustainability of water uses and safeguard of ecosystems depending on water resources (Foster & Hirata, 1998; Gianneli et al., 2007).

Vulnerability refers to the sensitivity of an aquifer system to deterioration due to an external action (Al-Zabet, 2002). This paper deals with the groundwater vulnerability in the alluvial aquifer of Kiti aquifer, South Cyprus. Firstly, the results from previous hydrogeological studies, including development of aquifers, hydraulic parameters and groundwater quality are presented. The vulnerability map was illustrated using the DRASTIC method in a GIS context (Vrba & Zaporozec, 1994; Polemio & Ricchetti, 2001; Panagopoulos et al., 2005; Gemitzi et al., 2006). This scientific

work was carried out in the framework of an INTERREG III B ARCHIMED program, funded by the European Union (Voudouris et al., 2007).

2. DESCRIPTION OF STUDY AREA

The study area is the river Tremithos basin (Kiti aquifer), covering an area of about of 30 km^2 and is situated in the southern part of Cyprus (Fig. 1). This area extends from the border of the Kiti Community as far as the coast, and is both a residential and a rural area (Fig. 1). The most important economic activities of this area are tourism and agriculture. The elevation is 20 m above sea level.

The area is characterized by semi-arid climatic conditions. A simplified geological map is shown in Fig. 2. The study area is characterized by many pollution sources acting on groundwater quality.

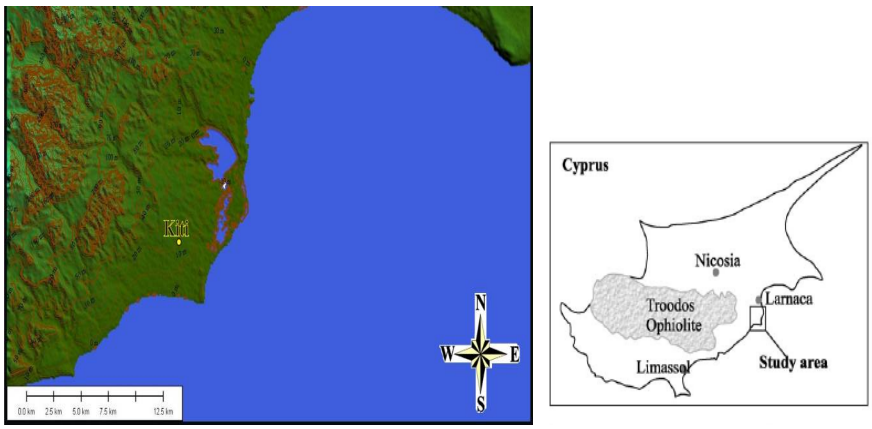


Figure 1. Topographic map of Kiti area.

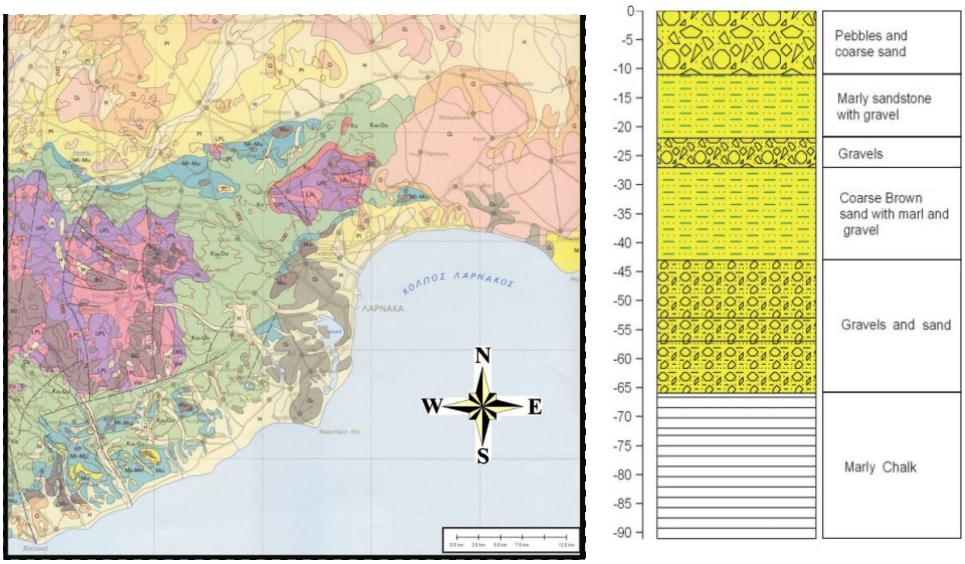


Figure 2. Geological map of Kiti area and a typical geological profile (Geological Survey Department).

The groundwater resources are related to the Pleistocene coastal plain and the recent Tremithos river valley deposits. The thickness of the deposits varies from 45 m near the mouth of the Tremithos river to about 20 m near the Kiti village (MANR, 1982).

Figure 3 shows an N-S geological cross section through the Kiti aquifer system. The aquifer system is layered, consisting of an upper unconfined alluvial aquifer separated from a lower confined carbonate aquifer by Pliocene marls (Milnes & Renard, 2004).

Over exploitation during the last decades has led to seawater intrusion. The water level recovered in the 90s but is still below sea table during the pumping season (Fig. 4). Groundwater abstraction was an average of 3x10^6 m^3/yr up to 1981, but was reduced to 1.3x10^6 m^3/yr in recent years (Milnes

& Renard, 2004). The saturated thickness is 2-3 m. The transmissivity values range between 6-135 m^2/d, as deduced from pumping tests (MANR, 1982). The permeability values range from 2 to 80 m/d. The groundwater flow has generally followed the topography of the area, moving towards the sea.

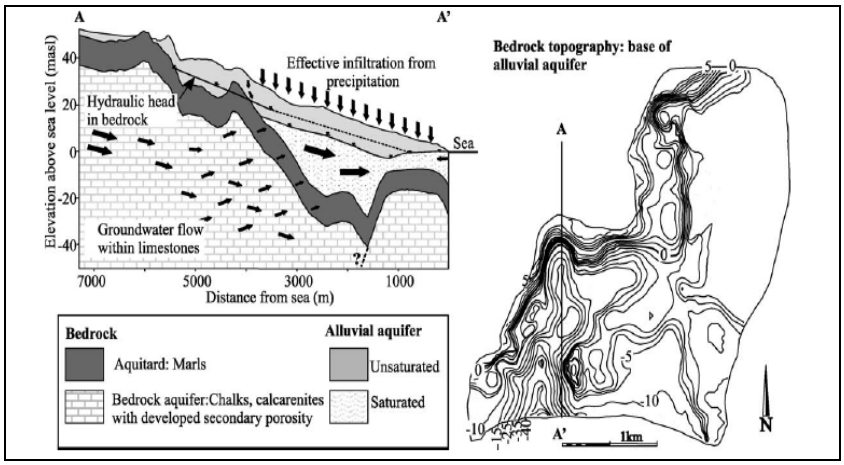


Figure 3. Geological cross section through the Kiti aquifer (Adapted from Milnes & Renard, 2004).

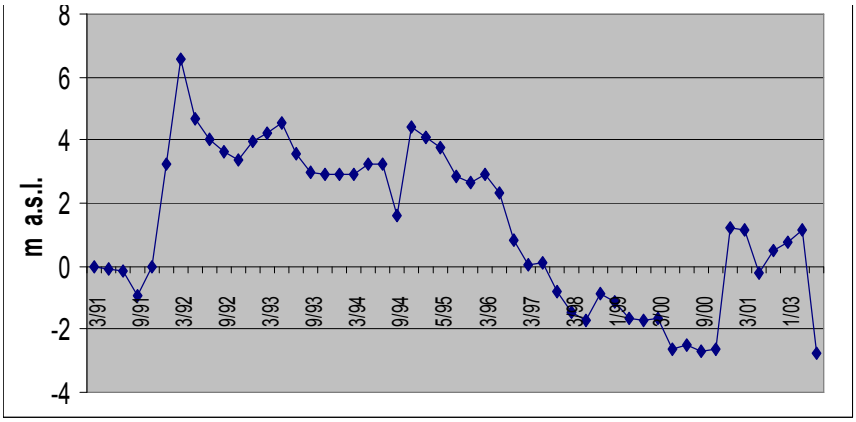


Figure 4. Fluctuation of groundwater level in m a.s.l.

3. GROUNDWATER VULNERABILITY ASSESSMENT

The concept of the groundwater vulnerability is based on the assumption that the physical environment may provide some degree of protection to groundwater against human activities. In other words, vulnerability represents the degree of weakness of one aquifer system against to pollution. In this study DRASTIC method was applied in order to evaluate the vulnerability, which belongs to rating methods for assessing groundwater vulnerability (Gogu & Dessargues, 2000).

The acronym DRASTIC (Aller et al., 1987; Al-Adamat et al., 2003; Kazakis et al., 2008) corresponds to the initial of the included seven (7) parameters: **D**epth, **R**echarge, **A**quifer media, **S**oil media, **T**opography, **I**mpact of the vadose zone media, hydraulic **C**onductivity of the aquifer.

Determination of the DRASTIC index involves multiplying each parameter weight by its site rating and summing the total. The equation for the DRASTIC Index (DI) is (Voudouris, 2009):

$$DI = \sum_{j=1}^{7} r_j.w_j \text{ or } DI=\text{DrDw + RrRw + ArAw + SrSw + TrTw + IrIw + CrCw} \quad (1)$$

where: D, R, A, S, T, I, C are the parameters, **r** is the rating for the study area and **w** is the importance weight for the parameter.

Each parameter including in the index must have a numeric value assigned between 1 and 10. The higher the index, the greater the groundwater pollution potential or greater aquifer vulnerability (Secunda et al., 1998). Parameters used by aforementioned method are fundamentally derivable

from monitoring gauges, hydrogeological field surveys including water level measurements, pumping tests and soil analyses, as well as aero-photo and remote sensing studies (Fig. 1).

4. RESULTS

Model parameters

Depth to groundwater (D)

The variable D (**D**epth to the water table) was extracted from the piezometric maps. The depth to groundwater in the alluvial aquifer of Kiti basin ranges from less than 4.5 to more than 18 m below ground surface (Fig. 5). In general, the deeper the water levels are, the longer the pollutant takes to reach the groundwater table (Voudouris & Mandilaras, 2004).

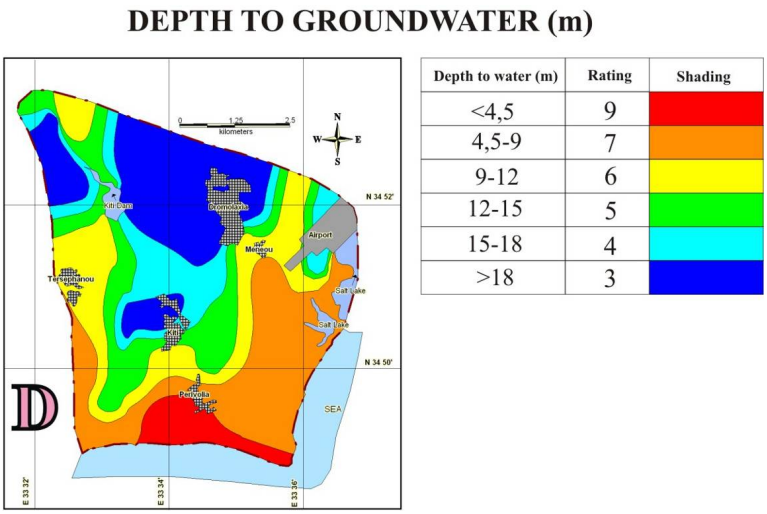


Figure 5. Depth to groundwater rating map.

Net recharge (infiltration) (R)

Net recharge is the total quantity of water which infiltrates from the ground surface to the aquifer on an annual basis. Leaching and transport of pollutants from the surface due to the rainfall that infiltrates through the vadose zone is a very important mechanism. The variable R was calculated from rainfall data and coefficients of infiltration of geological formations. The net recharge in the study area ranges between 35-85 mm/yr and the thematic map is shown in Fig. 6.

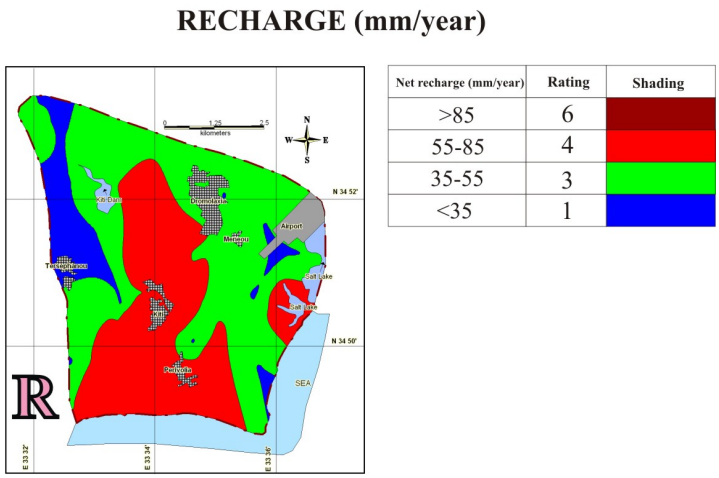


Figure 6. Net annual recharge.

Aquifer material (A)

The evaluation of variable A (Aquifer material) was based on data reports from the geological map and drilling data. Based on these data, the aquifer media was classified as (Fig. 7):

1) Gravel, 2) Sand and Gravel, 3) Sandstone 4) Limestone.

In general, the larger the grain size, the higher the permeability and the lower the attenuation capacity are.

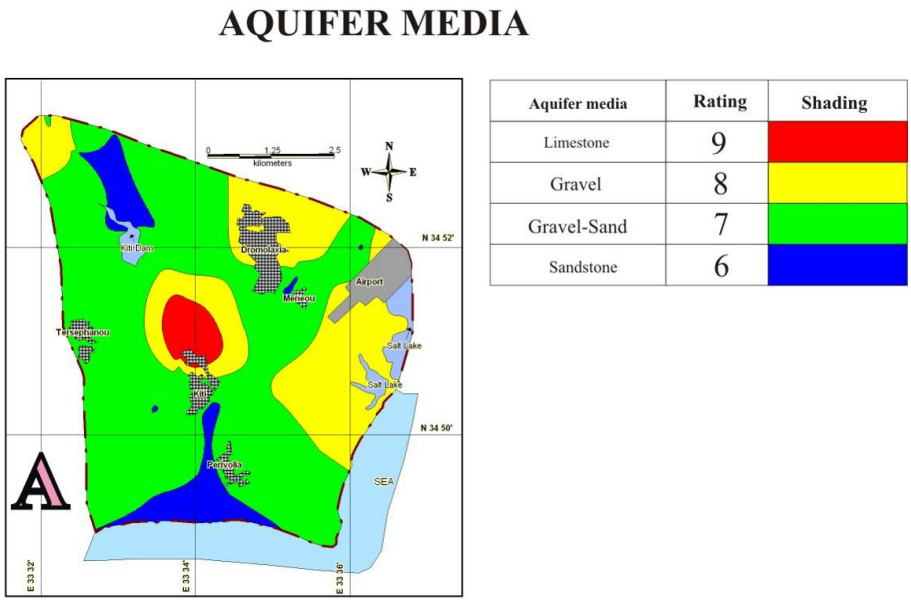


Figure 7. Aquifer media rating map.

Soil media (S)

Soil type plays an important role in the amount of recharge to the aquifer. The presence of fine material decreases infiltration, and therefore, pollution potential.

The variable S (***S***oil type) was obtained from soil classification maps. The predominant soil types are (Fig. 8): Fine textured, medium textured, coarse textured, thin layer of soil, no soil.

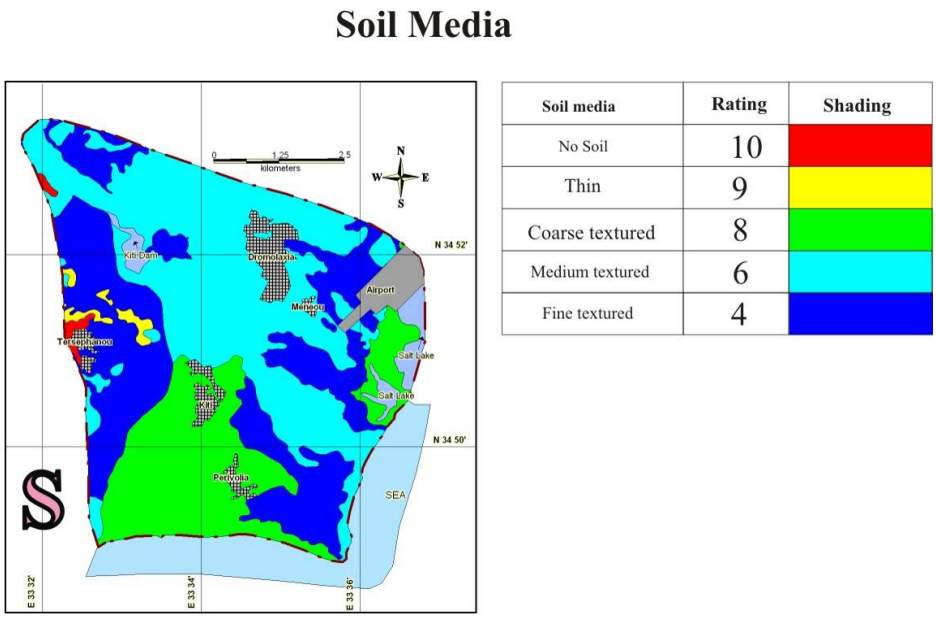


Figure 8. Soil media rating map.

Topography (T)

The variable T (Topography) was obtained from elevation points, using the triangulation method in ARC/INFO system (Fig. 9). It controls the likelihood of a pollutant to be migrated by runoff or to remain on the ground to infiltrate.

The topography of the Kiti area is for the most part a flat plain, slope being generally low (<2%), whereas a small area in the northern part of the basin reaches almost 10%.

Impact of the vadose zone (I)

The vadose zone is the zone of subsoil above the water table, which is unsaturated. The characteristics of the vadose zone determine the pathway and concentration of a pollutant. The attenuation degree in this zone is associated with the structure, the mineral composition and the organic matter content.

The evaluation of variable **I** was based on data reports from the drilling data (Fig. 10). The vadose zone in the study area is composed of unconsolidated gravel, sand and clay materials except in the northwestern part where the vadose zone is composed of conglomerates.

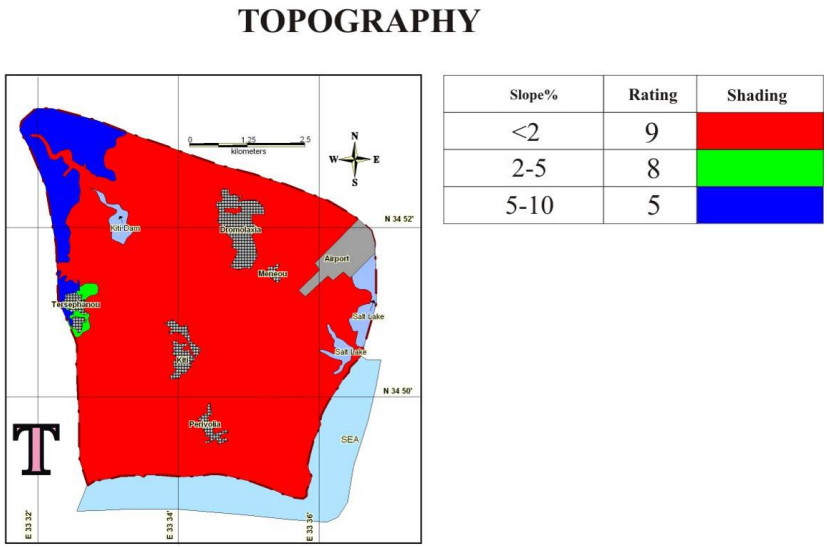


Figure 9. Slop rating map.

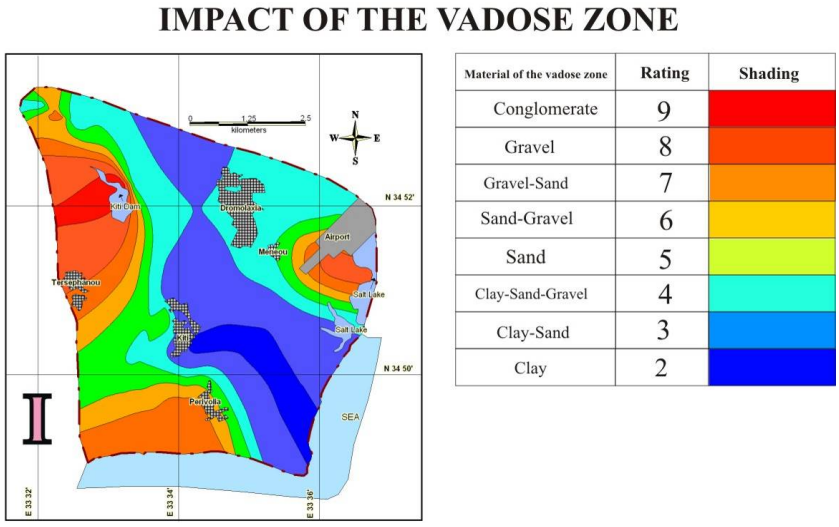


Figure 10. Vadose zone rating map.

Hydraulic conductivity (C)

The value of hydraulic conductivity is controlled by the properties of the aquifer and determines the rate of groundwater movement in the saturated zone. The groundwater velocity will increase, as the hydraulic conductivity increases, so the pollutants will also move faster, thus increasing vulnerability. The variable C was calculated from pumping test data and ranges between 2-80 m/day (Fig. 11).

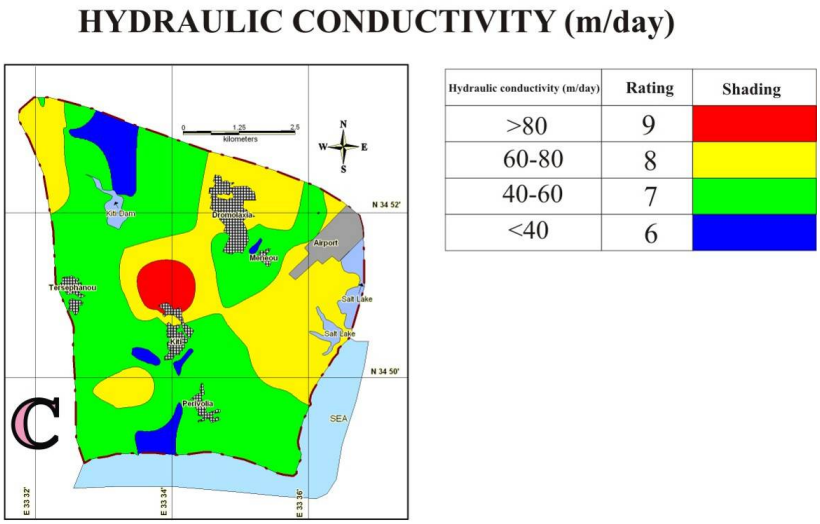


Figure 11. Hydraulic conductivity rating map.

DRASTIC index vulnerability

The vulnerability map of the Kiti area is shown in Fig. 12. The highest vulnerability values in the Kiti area, covering a large part, are associated with shallow aquifers without great depth of the vadose zone. For this reason a set of measures should be applied in order to protect the groundwater quality. Medium and low values of vulnerability are located in the north central part of the area (Dromolaxia village) in which the aquifer has great depth of vadose zone with layers of clay and silt and great depth to groundwater level.

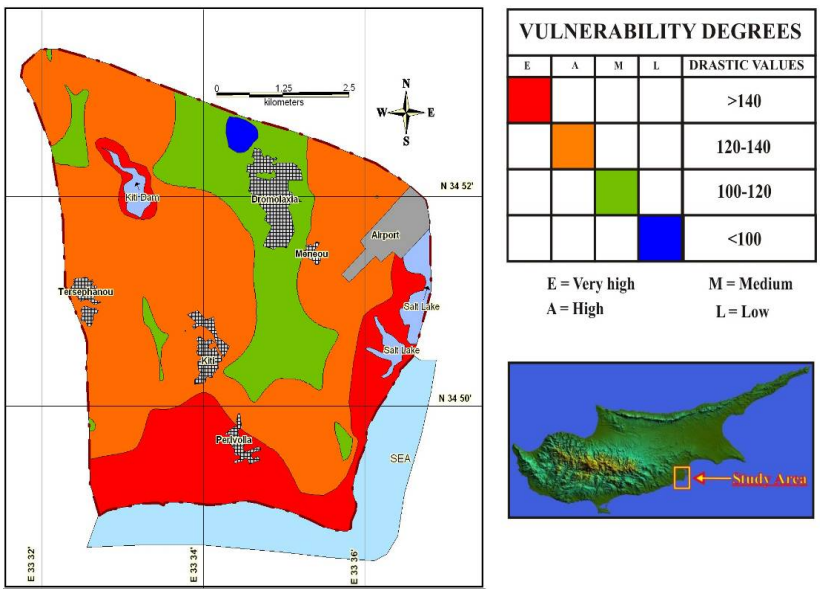


Figure 12. Vulnerability map of the Kiti aquifer.

CONCLUSIONS

The Kiti aquifer is under many pressures, due to urbanization, tourism and agriculture activities. The study area is characterized by many pollution sources acting on groundwater quality. The DRASTIC model was used to assess the groundwater vulnerability to pollution in Kiti aquifer. Low and very low values of vulnerability are located in the northern part of the study area. Highest vulnerability values are recorded in a large part of the aquifer and are associated with shallow aquifers without great depth of the vadose zone.

Regional assessment of groundwater vulnerability is a useful tool for groundwater resources management and protection zoning. The results provide important information and the vulnerability maps could be used by local authorities and decision makers.

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SIMULATION OF SEAWATER INTRUSION IN COASTAL AQUIFERS: SOME TYPICAL RESPONSES (PLAIN OF GROMBALIA NORTHEAST OF TUNISIA)

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Abstract: The underground waters represent an important resource exploited for human consumption and for agricultural and industrial use. Those waters are usually contaminated by pollutants of different natures: biological, chemical, or physical. Prevention against the aquifer pollution constitutes an important stage, to which scientists are providing with making an increasing effort, especially by studying the aquifer vulnerability. Seawater intrusion in coastal aquifers is generally three dimensional (3-D) in nature. This paper presents some typical example simulations of 3-D seawater intrusion process for a specified hypothetical study area. The simulation results presented in this paper are based on the density-dependent miscible flow and transport modelling approach for simulation of seawater intrusion in coastal aquifers. A nonlinear optimization-based simulation methodology was used in this study. The phreatic aquifer of Grombalia (prefecture of Nabeul, northeastern coast of Tunisia), which occupies an area of 363 km2, has a great economical importance, because it is used for irrigation and domestic consumption. The area of the aquifer is essentially occupied by agricultural zones, characterised by increasing use of chemical fertilizers. Various steady state simulations are performed for a specified study area. Response evaluations consider the effects of vertical recharge on seawater intrusion, effects of boundary conditions, and effects of spatially varying pumping from the aquifer. The 3-D simulations demonstrate the viability of using a planned strategy of spatially varying withdrawals from the aquifer to manage seawater intrusion. It is demonstrated that series of pumps near the ocean-face boundary induce a hydraulic head distribution that can be effectively used for controlling seawater intrusion

Key words: Seawater intrusion; coastal aquifers; Grombalia, Tunisia.

1. INTRODUCTION

Ground water contributes worldwide about 20% of people's fresh water. Despite this relatively small proportion its role is important for two reasons: On the one hand, ground water is well suited for the supply of drinking water due to its usually high quality. On the other hand, ground water basins are important long-term storage reservoirs, which in semi-arid and arid countries often constitute the only perennial water resource. Fresh water is mostly bound to the continents, of which: 68.9 vol.% is fixed in ice shields and glaciers, 29.9 vol.% stored as groundwater, 0.9 vol.% to soil moisture and atmospheric water vapour and 0.3 vol.% appears as surface water. The storage capacity is evident if one compares the volumes of surface and ground water resources. Globally the volume of fresh water resources in rivers and lakes is about 100,000 km^3. With about 8,300,000 km^3, the volume of ground water is two orders of magnitude larger (Jaroslav et al., 2006).

A special consequence of ground water level decline is shown in coastal areas. Due to the density difference between fresh water on the land side and salt water on the sea side a salt water wedge develops, which progresses inland until the pressure equilibrium at the salt-fresh water interface is reached. Every perturbation of this equilibrium by reducing the fresh water flow will lead to a further progression of the salt water wedge inland, until it eventually reaches and destroys the pumping wells. In Tunisia Sea water intrusion is notorious along the coasts, saline water from salt lakes (Chotts), in the vicinity of which ground water is pumped, can be attracted to the wells. A ground water level decline inland can lead to the rise of saline water from underlying aquifers.

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Seawater intrusion can be caused by over-pumping in sensitive portions of coastal aquifers. It often results in the degradation of water quality within the aquifer and may ultimately require costly remedial measures.

2. SEAWATER INTRUSION & GROUND WATER MODELING

2.1 Approaches to modeling seawater intrusion

The occurrence of seawater intrusion in coastal aquifers can be visualised with a simplified conceptual model (Figure 1). The transition zone is a zone of mixing between the freshwater and the saltwater. Under equilibrium conditions, the interface will remain fixed, and freshwater will discharge along the seepage face. In coastal aquifers, recharge is predominantly due to lateral inflow, whereas in island aquifers, it is due to vertical recharge. These conceptual models are analysed in different ways. Coastal aquifers are usually modelled in the vertical plane. Island systems are also analysed in the vertical plane when the thickness of the freshwater lens is a significant fraction of the horizontal width of the lens, a situation common in atoll islands.

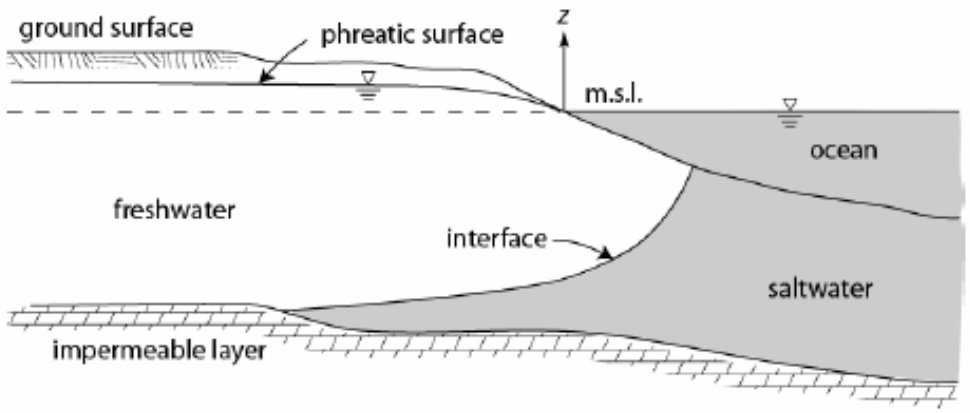


Figure 1. Schematic sketch of saltwater intrusion, sharp interface model.

There are several methods for dealing with seawater intrusion as two-phase, immiscible problem. These methods are generally known as interface methods. Under very dynamic conditions, or in case where the freshwater lens is relatively thin compared with the transition zone, it may be necessary to adopt miscible flow models. Reilly and Goodman (1985) presented a historical perspective of quantitative analysis of the saltwater freshwater relationship in groundwater systems. Custodio (1992) reported that both approaches produce good results if applied to the right circumstance. Pinder and Stothoff (1988) pointed out that the sharp interface assumption is appropriate for steady state conditions. Additionally, if conditions of variable recharge and/or significant pumping exist, the interface will move in response to the non equilibrium conditions. For this problem, the hydraulic head and fluid velocity in both the fresh and salt portions of the aquifer must be considered. Since these quantities are unknown, the problem becomes one of coupling differential equations for motion of the freshwater and saltwater, and solving for the unknown position of the interface between them. In fact when the sharp interface approximation is inappropriate owing to the large thickness of the transition zone relative to the freshwater lens, it is advisable to treat the freshwater and saltwater as miscible. There are two elements to the problem: 1) a single fluid phase with variable density; and 2) a solute representing the total dissolved solids in seawater, which affects the fluid density. As a result, there are four unknown dependent variables: fluid specific discharge, fluid pressure, salt concentration, and fluid density, requiring the solution of four equations (Frind and Mangata, 1985). Often the most disruptive influence is the effect of pumping activities. Withdrawal of water from the freshwater lens via pumping wells creates saltwater upconning, which is the migration of the fresh-salt interface towards a well in response to pumping withdrawals. However, Bear (1979) has given some examples of real aquifers

in which experimental measurements have shown the salt concentration to vary sharply at a determined location, clearly establishing a region of low salt concentration (freshwater) and a region of high salt concentration (saltwater).

2.2 Numerical methods for groundwater management

Numerical modeling has emerged over the past 40 years as one of the primary tools that hydrologists use to understand groundwater flow and saltwater movement in coastal aquifers. Numerical models are mathematical representations (or approximations) of groundwater systems in which the important physical processes that occur in the systems are represented by mathematical equations. The governing equations are solved by mathematical numerical techniques (such as finite-difference, finite-element, boundary finite element, or finite volume methods) that are implemented in computer codes. The primary benefit of numerical modeling is that it provides a means to represent, in a simplified way, the key features of what are often complex systems in a form that allows for analysis of past, present, and future groundwater flow and saltwater movement in coastal aquifers. Such analysis is often impractical, or impossible, to perform by field studies alone. Numerical models have been developed to simulate groundwater flow solely or groundwater flow coupled to solute transport (the movement of chemical species through an aquifer). For a number of reasons, numerical models that simulate groundwater flow and solute transport are more difficult to develop and to solve than those that simulate groundwater flow alone.

Coastal aquifers are particularly difficult to simulate numerically because the density of the water and the concentrations of chemical species dissolved in the water can vary substantially throughout the modeled area. Numerical models based on the sharp interface approximation assumption.

Two and Three-dimensional, dynamic numerical models handle more complex situations under realistic representation of the mechanisms involved and account for variability in physical situations. Three-dimensional models are more realistic and yields better insight. They are particularly effective in handling irregularities in coastal aquifer geometry, water use; heterogeneity in aquifer formations; temporal variations in boundary conditions; and spatial variations in salt concentrations. Therefore, these models can provide greater detail and accurate results. The only drawback is related to the intensive computational cost (memory and CPU) and the amount of data needed to feed up these models.

Currently, several solute transport models suitable for the simulation of seawater intrusion and up-conning of saline water beneath pumping sites are available. These include SUTRA (Voss, 1984), which is a two-dimensional, vertical cross-sectional model; SEAWAT (Guo, et al., 1998), CODESA3D (Gambolati, et al., 1999), FEFLOW (Diersch, et al., 2004) are examples of three-dimensional models. These models provide solution of two simultaneous, non-linear, partial differential equations that describe the conservation of “mass of fluid” and “conservation of mass of solute” in porous media; as described above.

SUTRA: (which is named from the acronym Saturated Unsaturated TRAnsport) was published by the United States Geological Survey (Voss, 1984). The equations are solved by a combination of finite element and integrated finite difference methods. Although SUTRA is a two-dimensional model, a three-dimensional element is provided since the thickness of the two-dimensional region may vary over the solution domain. The coordinate system may be either Cartesian or radial which makes it possible to simulate phenomena such as saline upcoming beneath a pumped well. SUTRA permits sources, sinks and boundary conditions of fluid and salinity to vary both spatially and with time.

SEAWAT: The SEAWAT program was developed to simulate three-dimensional, variable-density, transient ground-water flow in porous media. The source code for SEAWAT was developed by combining MODFLOW and MT3DMS into a single program that solves the coupled flow and solute-transport equations. The SEAWAT code follows a modular structure, and thus, new capabilities can be added with only minor modifications to the main program. SEAWAT reads and

writes standard MODFLOW and MT3DMS data sets, although some extra input may be required for some SEAWAT simulations. This means that many of the existing pre- and post-processors can be used to create input data sets and analyze simulation results. The original SEAWAT code was written by Guo and Bennett (Guo, et al., 1998) to simulate ground water flow and salt water intrusion in coastal environments. SEAWAT uses a modified version of MODFLOW (McDonald, et al., 1988)] to solve the variable density, ground water flow equation and MT3D (Zheng, 1990) to solve the solute-transport equation. To minimize complexity and runtimes, the SEAWAT code uses a one-step lag between solutions of flow and transport.

Groundwater Vistas (GV) is a sophisticated Windows graphical user interface for 3-D groundwater flow & transport modelling. GV couples a powerful model design system with comprehensive graphical analysis tools.

CODESA3D: The numerical model, CODESA-3D (COupled variable DEnsity and Saturation 3-Dimensional model), is described in detail in Gambolati and al. (Gambolati, et al.,1999). The CODESA-3D code is born from the integration and extension of parent codes developed, during the last ten years, by the Department of Applied Mathematics of the University of Padova in collaboration with the Environment Group of CRS4 (Center for Advanced Studies, Research and Development).The numerical model is mathematical aspects are briefly reviewed here for purposes of introducing some basic concepts and the important parameters involved in the simulations to be subsequently presented. It is a three-dimensional finite element simulator for flow coupled to solute transport in variably saturated porous media on unstructured domains (Gambolati, et al.,1999) and (Lecca, 2000). Sophisticated algorithms are used to handle the coupling and non linearity inherent in the seawater intrusion model and the non linearities in the unsaturated flow component. In addition, for computational efficiency dynamic time stepping is implemented to ensure that the code uses large time steps where possible, based on whether the convergence of the iterative coupled solution at a given time is fast (time step is increased), intermediate (time step is unaltered), or slow (time step is reduced).

FEFLOW: (Finite Element subsurface FLOW system) (Diersch, 2004) is a computer program for simulating groundwater flow, mass transfer and heat transfer in porous media. The program uses finite element analysis to solve the groundwater flow equation of both saturated and unsaturated conditions as well as mass and heat transport, including fluid density effects and chemical kinetics for multi-component reaction systems. Feflow is a finite element groundwater modeling commercial code developed by WASY AG (Germany). The code solves groundwater flow, solute transport, density dependent solute transport, coupled heat and flow, saturated and unsaturated flow. The code solves groundwater flow, solute transport, density dependent solute transport, coupled heat and flow, saturated and unsaturated flow. Different types of licenses are available at different prices depending on the type of problems that the user needs to solve. The cheapest version includes only 2D groundwater flow, while the most expensive includes 3D flow, transport, and heat processes. The code includes different types of elements that can be combined. This allows for example to model 2D planar structures such as a fault plane within a porous matrix or linear elements such as a drain or a stream in which the flow equation may be different than within the porous matrix. The code solves both steady state and transient equations.

Other numerical models have been also developed in USA such as SHARP, which is a quasi-three- dimensional, numerical finite-difference model to simulate freshwater and saltwater flow separated by a sharp interface in layered coastal aquifer systems; and FEMWATER which is a 3D finite element, saturated / unsaturated, density driven flow and transport model.

3. CASE-STUDY AREA: GROMBALIA AQUIFER

A ground water model is used to study the sustainability of ground water abstraction. A numerical model that treats density-dependent variably saturated flow and miscible salt transport is used to investigate the occurrence of seawater intrusion in the Grombalia coastal plain of

northeastern Tunisia. We examine the effects of and interplay between pumping, artificial recharge, soil/aquifer properties, and the unsaturated zone.

The study area, which is located about forty kilometers, South of Tunis (Figure 2), is an ancient bay widely open onto the Northwest and communicating with the Bay of Tunis.The Grombalia aquifer is naturally bounded northward by the Mediterranean Sea and Weastward by the Golfe of Tunis. At the south, it's limited by the mountain jebel Reba El Aïn and at the east by the jebel Abderrahmen. The climate of the studied area is Mediterranean type. The yearly middle rain, for the period 1990-2005, is 459 mm (station of Grombalia). The yearly middle temperature is 18°C. The annual evaporation of the studied area, calculated according to the Piche method, is 1731.5 mm (Gaaloul et al, 2005).

The Grombalia aquifer is a plain, which belongs to a multiplayer, three level groundwater system composed of a shallow table, a semi deep table and a deep table, the whole of which covers an area of 719 km², out of which 407 km² belong to the Quaternary period. The Quaternary, is composed of alternations of permeable layers sandblasters and of marneuses layers comparatively impervious, constitutes the reservoir of underground waters giving birth to a complex aquifer system: a shallow aquifer, a semi deep aquifer and a deep aquifer, exploited until 40 m, 60 m and 200 m, respectively.

The hydrographic network is formed essentially by the oueds El Bey, Bzikh, Ellouza, Ejjorf, Ejjdida, Bourbi, and Sidi Ettoumi. This network plays a principal role of the alimentation of the aquifer of the upstream (oueds Bey and Jedida), and of the drainage to the in downstream (oueds El Bey and Soltane).

From the exploitation viewpoint, the Grombalia aquifer is exploited by 8430 surface wells having inferior depths to 40 m, witch 6 667 surface wells are equipped of pumps and drawing yearly 90Mm³, witch attend 93 Mm³ in 2006. The recharge is practical in three places situated between Menzel Bou Zelfa and Béni Khalled. Theses sites of recharge are composed by 5 basins in El Amrine near Menzel Bou Zelfa and 28 basins in Sidi Alaya and some infiltrations basins in El Gobba near Béni Khalled.

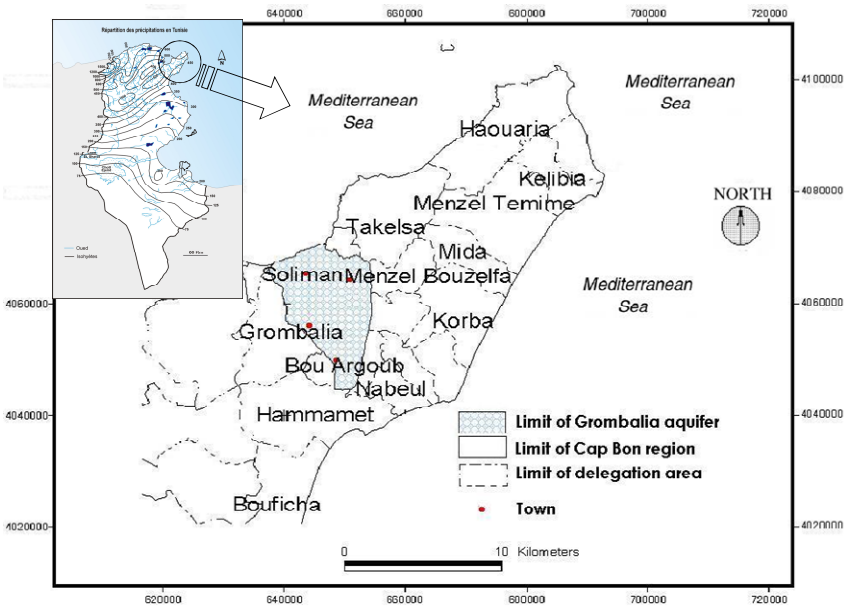


Figure 2. Geographic distribution of the Grombalia aquifer in the Cap Bon

Groundwater flow models are used to calculate the rate and direction of movement of groundwater through aquifers and confining units in the subsurface. These calculations are referred to as simulations. The simulation of groundwater flow requires a thorough understanding of the hydrogeologic characteristics of the site.

The hydrogeologic investigation should include a complete characterization of the following:

- ✓ Subsurface extent and thickness of aquifers and confining units (hydrogeologic framework).

- ✓ Hydrologic boundaries (also referred to as boundary conditions), which control the rate and direction of movement of groundwater.
- ✓ Hydraulic properties of the aquifers and confining units.
- ✓ A description of the horizontal and vertical distribution of hydraulic head throughout the modeled area for beginning (initial conditions), equiliblium (steady-state conditions) and transitional conditions wh en hydraulic head may vary with time (transient conditions).
- ✓ Distribution and magnitude of groundwater recharge, pumping or injection of groundwater, leakage to or from surface-water bodies. These stresses may be constant (unvarying with time) or may change with time (transient).

The outputs from the model simulations (Gaaloul, et al., 2006) are the hydraulic heads and groundwater flow rates which are in equilibrium with the hydrogeologic conditions (hydrogeologic framework, hydrologic boundaries, initial and transient conditions, hydraulic propel1ies, and sources or sinks) defined for the modeled area. Figure 3 shows the simulated flow field for a hypothetical site at which pumping from a well creates changes in the groundwater flow field.

Through the process of model calibration and verification, the values of the different hydrogeologic conditions are varied to reduce any disparity between the model simulations and field data, and to improve the accuracy of the model. The model can also be used to simulate possible future changes to hydraulic head or groundwater flow rates as a result of future changes in stresses on the aquifer system. Surface and groundwater and Transport models simulate the movement and chemical alteration of contaminants as they move with groundwater through the subsurface. Surface and groundwater and transport models require the development of a calibrated groundwater flow model or, at a minimum, an accurate determination of the velocity and direction of groundwater flow, which has been based on field data. The model simulates the following processes: Movement of contaminants by advection and diffusion, Spread and dilution of contaminants by dispersion, Removal or release of contaminants by sorption or desorption of contaminants onto or from subsurface sediment or rock, or Chemical alteration of the contaminant by chemical reactions which may be controlled by biological processes or physical chemical reactions.

The outputs from the model simulations are the contaminant concentrations, which are in equilibrium with the groundwater flow system, and the geochemical conditions (described above) defined for the modeled area. Figure 3 shows the simulated migration of a contaminant at a hypothetical site.

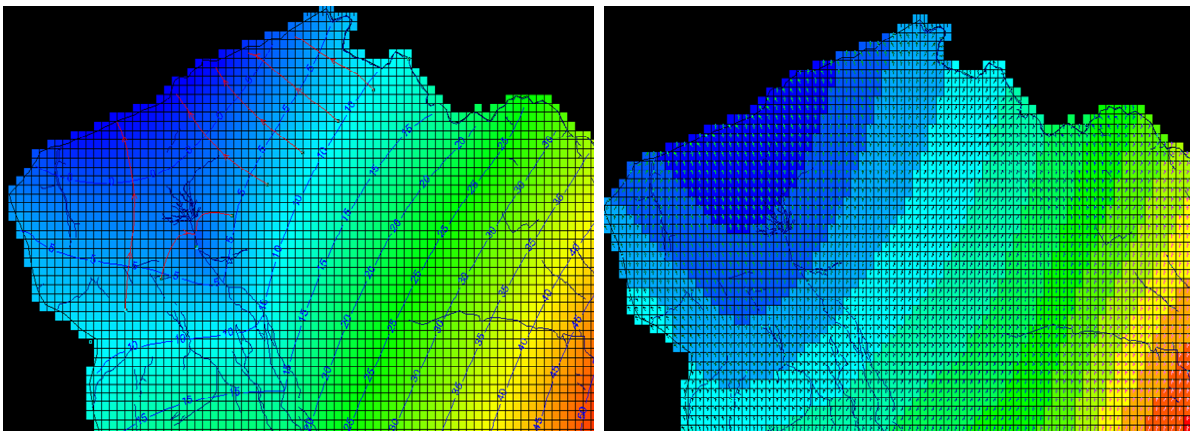


Figure 3. Simulated flow in Grombalia aquifer

As with groundwater flow models, surface-groundwater and transport models should be calibrated and verified by adjusting values of the different hydrogeologic or geochemical conditions to reduce any disparity between the model simulations and field data. This process may result in a re-evaluation of the model used for simulating groundwater flow if the adjustment of values of geochemical data does not result in an acceptable model simulation. Predictive simulations may be made with a surface-groundwater and transport model to predict the expected concentrations of contaminants in groundwater as a result of implementation of a remedial action. The simulated

containment of a contaminant plume is shown in Figure 3. Monitoring of hydraulic heads and groundwater chemistry will be required to support predictive simulations.

Model calibration consists of changing values of model input parameters in an attempt to match field conditions within some acceptable criteria (Figure 4). This requires that field condi1ions at a si1e be properly characterized. Lack of proper si1e characterization may result in a model that is calibrated to a set of conditions, which are not representative of actual field conditions. The calibration process typically involves calibrating to steady state and transient condi1ions.

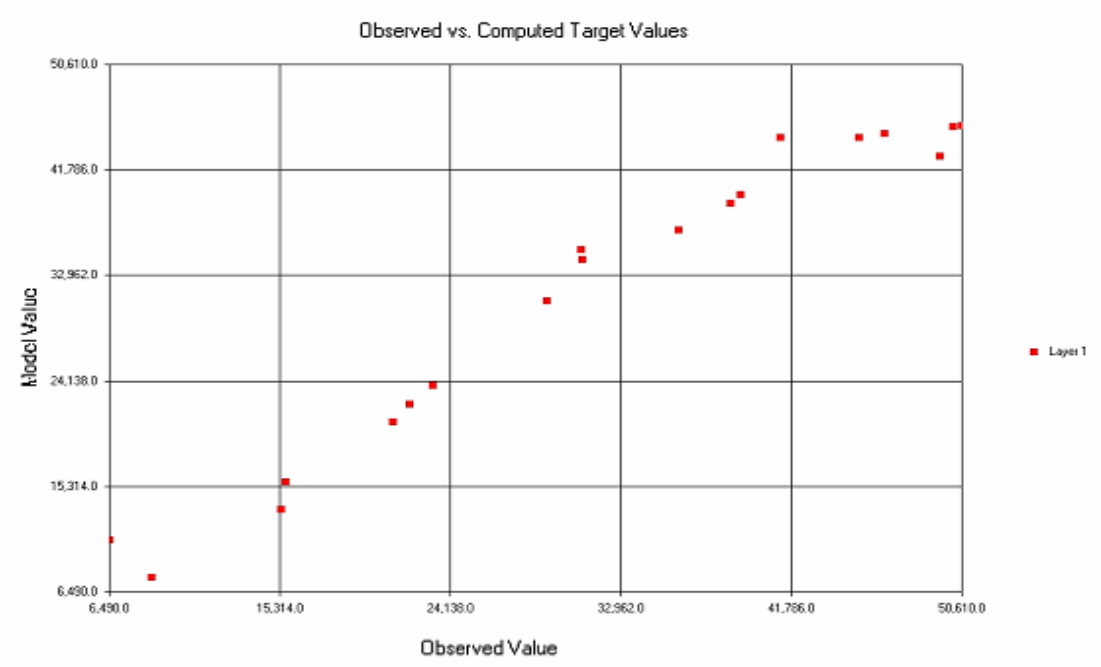


Figure 4. Comparison between measured and computed hydraulic heads.

With steady-state simulations, there are no observed changes in hydraulic head or contaminant concentration with lime for the field conditions being modeled. Transient simulations involve the change in hydraulic head or contaminant concentration with lime (e.g. aquifer test, an aquifer stressed by a well-field, or a migrating contaminant plume).

These simulations are needed to narrow the range of variabili1y in model input data since there are numerous choices of model input data values which may result in similar steady-state simulations. Models may be calibrated without simulating steady-state flow conditions, but not without some difficulty.

4. CONCLUSIONS

The use of groundwater models is prevalent in the field of environmental hydrogeology. Groundwater-flow and surface-groundwater and transport models have been applied to investigate a wide variety of hydrogeologic conditions. Groundwater flow models are used to calculate the rate and direction of movement of groundwater through aquifers and confining units in the subsurface. Surface-groundwater and transport models estimate the concentration of a chemical in groundwater beginning at its point of introduction to the environment to locations downgradient of the source. Surface-groundwater and transport models require the development of a calibrated groundwater flow model or, at a minimum, an accurate determination of the velocity and direction of groundwater flow that has been based on field data.

Models are conceptual descriptions or approximations that describe physical systems using mathematical equations-they are not exact descriptions of physical systems or processes. The applicability or usefulness of a model depends on how closely the mathematical equations approximate the physical system being modeled. In order to evaluate the applicability or usefulness

of a mode!, it is necessary to have a thorough understanding of the physical system and the assumptions embedded in the derivation of the mathematical equations.

These equations are based on certain simplifying assumptions which typically involve: the direction of flow, geometry of the aquifer, the heterogeneity or anisotropy of sediments or bedrock within the aquifer, the contaminant transport mechanisms, and chemical reactions.It is because of these assumptions, and the many uncertainties in the values of data required by the model, that a model must be viewed as an approximation and not an exact duplication of field conditions. The collection of site-specific data during the remedial action is referred to as "performance monitoring." Performance monitoring is required as a means of physically measuring the actual behavior of the hydrogeologic system and demonstrating compliance with environmental statutes.

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SMALL DAMS, PRE-RESERVOIRS: PROACTIVE, SUSTAINABLE MEASURES FOR ADDRESSING WATER SHORTAGE, FLOOD RISK, WATER POLLUTION AND COASTAL EROSION

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Abstract: Areas facing drought/ water shortage problems have adopted policies of constructing dams in order to store the water for later use. River flow in such areas carries with it significant quantities of sediments (the result of soil erosion), tree branches (the result of the drought) and polluting substances(especially in the case of a fire). This multi-phase river flow is associated with various problems such as bridge openings being blocked causing floods upstream of the bridge, collection of sediments in the receiving dams, dam storage capacity reduction and natural sediment cycle arrest. As a result the river delta and the adjacent coastal areas are starved from sediment supply, the water stored in the dams is polluted with pollutants carried by the river flow.

Measures have been taken and are being taken to address individual problems:

- Dams are built to address the water shortage problem,
- Bridges/ culverts/ water drainage systems are designed to accommodate the increasing flood events – it is not practical to design for obstructed openings,
- Coastal defences are constructed to address the problem of coastal erosion.

In a holistic approach, the above problems are addressed simultaneously and pro-active, sustainable measures developed in a recent research program are proposed. Small dams/ pre-reservoirs are constructed at the entrance of the river to the reservoir and upstream of potential flood areas. These are low elevation dams capable of storing the water carried by small rainfall events (return period of about one year). River water (together with sediments, any branches and pollution) is arrested and temporarily stored within the basin/ pond upstream of the structure (which is normally empty). After the storm, water, practically free from sediments and pollution, is allowed to enter the main dam reservoir and is diverted for irrigation or domestic supply, or continues its flow to the sea at a controlled rate. This controlled rate of flow reduces the risk of flooding downstream. Sediments collected in the small dam basins are sorted and suitable material is transported to the coastal area, thus restoring the natural sediment cycle. Any pollution (result from fire or accidents) is treated before discharge to the environment.

The small dams, pre-reservoirs are considered as sustainable, pro-active measures that reduce the risk of flooding, address the risk of water pollution, contribute towards the alleviation of the drought problem and restore the sediment cycle and the supply of sediments to the coastal environment.

Key words: Dams, Erosion, Flood, Pre-reservoir

1. INTRODUCTION

Water shortage is an acute problem especially in places experiencing frequent drought periods. A common solution in increasing the supply side of the water budget is to construct dams which create reservoirs that store flood water mainly in winter time. The stored water can be used for irrigation and domestic supply.

Apart from their main function of storing water for human use, dams also have a positive effect in flood protection. When the dams are not full, they impound flood water and protect downstream areas from flood damage. Flood protection may thus be ensured if a policy decision is taken to always maintain extra storage in the reservoirs. This policy may easily be implemented by either controlling the maximum water level in the reservoirs or by installing suitable gates at existing spillways which can store flood water by maintaining the same level of safety in the case of a major flood event.

The river flow entering reservoirs carries with it sediments, which are the result of land erosion. These sediments are stored together with the water in the reservoirs. Sediments trapped in dams have two major adverse effects:

- They reduce the storage capacity of the dams
- They starve the river mouths and the coastal environment at large from their natural supply of sediments.

The arrest of sediment supply causes the retreat of the river deltas and erosion of neighbouring beaches. Measures to overcome the erosion problem include the retreat of the area, the construction of coastal defences and beach nourishment. All these measures are associated with significant financial, social and environmental costs.

A problem highlighted by the forest fires in Greece in the summer of 2007, is the potential pollution of the water stored in the reservoirs. Especially in dams constructed downstream of urban, industrial or agricultural land, the risk of pollution of the water in the reservoirs with toxic material, heavy metals, pesticides or other hazardous substances, is high. Water treatment plants built for the treatment of fresh reservoir water have serious limitations in coping with many pollutants.

A holistic approach is presented in this paper which addresses water shortage, flood risk, water pollution and coastal erosion simultaneously.

2. DAMS IN CYPRUS AND WATER SHORTAGE

The climate of Cyprus is semi-arid, with mild and relatively wet winters and hot, dry summers. Rainfall is variable having a mean intensity of the order of 500 mm per year (figure 1). Continuous droughts are becoming more frequent, whilst in the past 30-35 years the mean rainfall has reduced significantly. There are no rivers with perennial flow in Cyprus but there are numerous small seasonal rivers, which flow mainly in winter and sometimes in early spring

The population of the island, in the areas under the control of the Republic of Cyprus is about 750 thousand people, of which about two thirds (2/3) live in urban areas. Urbanisation took place over a very short period of time, the urban population being doubled in just 30 years (1960 to 1990).

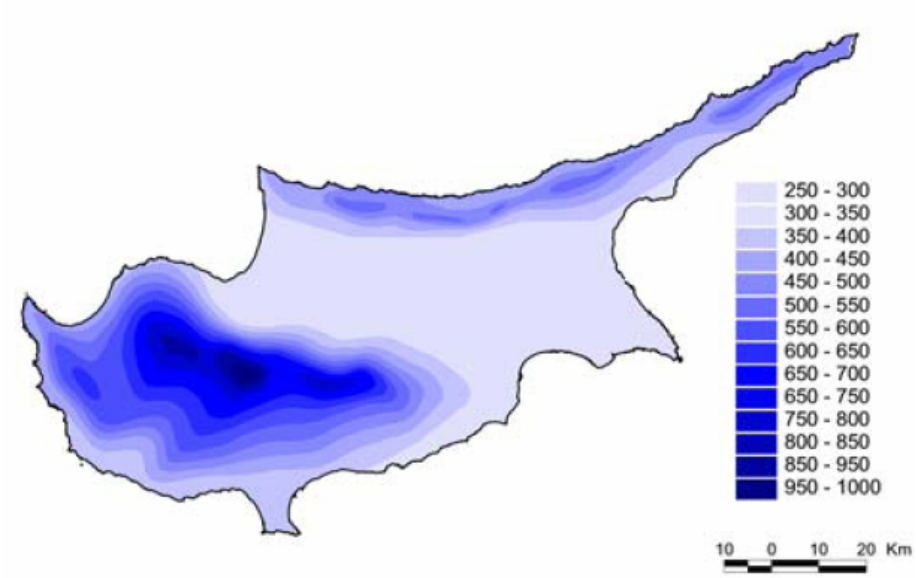


Figure 1. Mean Annual Precipitation

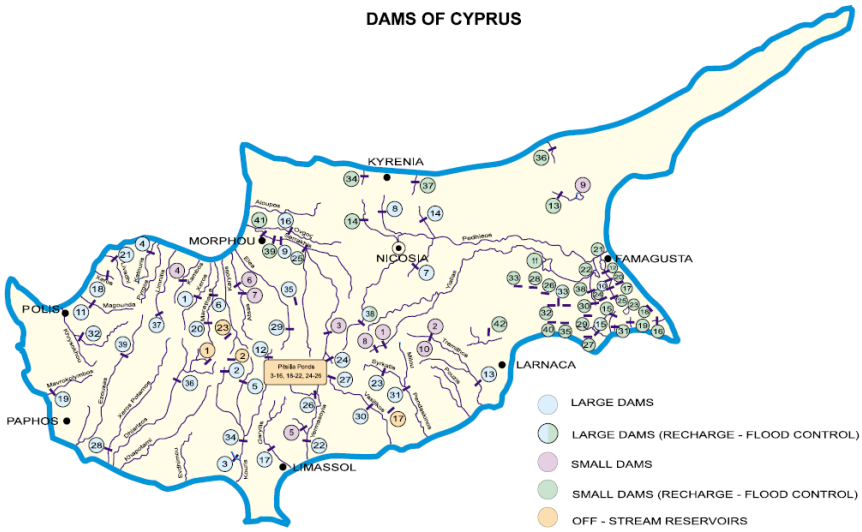


Figure 2. The Dams of Cyprus

Water shortage is a major problem. In the early sixties, the Republic of Cyprus in order to address this problem, adopted the slogan "Not a drop of water to be wasted to the sea". A series of dams has been constructed in the past 40-45 years, and today almost all (seasonal) rivers are dammed. The rivers and the dams of Cyprus are shown on figure 2. It is noted that Cyprus today has the highest concentration of dams in Europe.

The majority of the large dams are located close to the river delta in order to maximise the size of the catchment area and hence maximise the potential quantity of stored water. An example of such a dam is shown in figure 3.

The dams have been designed to accommodate biannual flows and hence they rarely fill up. Hydro potential in Cyprus is insignificant and therefore the primary role of dams is to store water. The use of water is carefully managed and monitored. In exceptional cases some dams overflow but water rarely reaches the sea, as substantial quantities are required to replenish the depleted aquifers between the dams and the sea.



Figure 3: Dam of Asprokremmos Damming the Xeros River, Paphos area

3. FLOOD PROTECTION

Although Cyprus is well known for the water shortage problem droughts were never associated with loss of life. However violent flood events throughout history claimed many lives and produced major catastrophes.

Only recently two people lost their lives due to a flash flood during a year characterized as a drought year. On a Monday night, 30th October 2006, a sudden rainfall caused the death of a

married Cypriot couple in Kissonerga area, Paphos, in western Cyprus. The couple was travelling in a car and attempted to cross a small bridge which started to overflow by flood river waters. The car was dragged by the flow and it was later found without passengers a short distance downstream. After a search by rescue services, relatives and other volunteers the wife was found 16 hours later buried in sediments. The search continued intensively during the following days, and although the length and width of the river are limited the body of the husband was not found until 8 days later buried deeper in sediments.

The river is a very small one, having a catchment area of only $7^1/_2$ km^2. The amount of sediments carried by such a small river is noted. Drought periods increase the risk of land erosion. Furthermore, vegetation dies and branches from trees, leaves, or even trunks fall on the ground to be carried by rain water. Bridges, culverts or natural contractions of the river are blocked by this vegetation causing flooding upstream of these places.

One of the most catastrophic flood events that practically destroyed the city of Nicosia in 1330 is described by Leontios Macheras (ca1430) as follows:

"On the 10th of November of 1330 after Christ a large devastation was done from rain in the night and the river of Chora came flooding down so great and uprooted many trees and brought them down to Chora , blocked the bridge of Sineskardhos and drowned a lot of people". Other sources estimate the number of drowned people between 3000 to 8000.

Downstream of dams such events are not likely to happen if the dams are not full. These structures are most effective flood protection measures if their water level is controlled. During "good" rainy seasons, when the dams are almost full, water should be allowed to flow downstream in a controlled manner so that there is sufficient space in the reservoir to store flood water from a major rainfall event.

4. EROSION ON LAND

4.1 Land Erosion worldwide

The rate of land erosion varies significantly from place to place. The rate of erosion depends on:

- The climate in the catchment area
- The geology of the catchment area
- The topography of the catchment area
- The human activity in the catchment area

Erosion requires two mechanisms to act simultaneously:

1. Water flow capable to transport sediments
2. Sediments on the land surface that can be transported

If there is rainfall but the surface is rocky or with vegetation then there no erosion. Similarly, if there is abundance of sediments but no rainfall to carry the sediments away, then again there is no erosion (White 2001).

The world average rate of erosion is of the order of 65 m^3/km^2/year, or 65mm per 1000 years (Walling, 1996). The largest erosion rates are encountered in China and the Middle East and the least in South America and North Africa.

4.2 Land Erosion in Cyprus

Land erosion in Cyprus is significant due to the following factors:

- The rain is seasonal and hence vegetation is limited. Rainfall which is sporadic and intense erodes the susceptible soil surface.
- The geology of a large part of Troodos mountains is ophiolitic which is easily erodible

- The topography is characterised by steep slopes which are erodible
- Human activity is most intense both in urbanisation and in agricultural use

All these factors contribute towards the erosion of sediments and their transport by storm water and deposition in the reservoirs.

The problem of sediments trapped in the reservoirs was identified from the start of the dam construction programme. Evacuation of the sediments from the reservoirs using dredging or sluice gates were evaluated and found as non viable due to the large cost and due to the loss of precious water respectively. The solution adopted at the time was to build the dams larger than required, allowing sufficient dead storage.

The sediment yield in four dams is presented in table 1 (Konteatis, 1974).

Table 2. Sediments in Dams in Cyprus

DAM	Capacity M m^3	Catchment Area km^2	Erosion Rate of Catchment Area m^3/ yr/km^2	Sediment yield/ Year m^3/ yr
Asprokremmos	52.4	224	940	210,560
Evretou	24	91	480	43,680
Kourris	115	308	910	280,280
Kiti	1.6	130	590	16,800

5. COASTAL EROSION

5.1 Observations of Erosion

The geomorphology of the island has been influenced by the sediments carried by its rivers. Archaeological and historical evidence is widespread about coastal cities on river mouths now located several kilometres inland (such as Engomi,3 km), lakes being formed behind offshore islands due to sediments deposited by rivers on either side of the island (i.e.Akrotiri lake) and beaches formed from sand carried by rivers (Morel, 1960).



Figure 4. Kouris River Delta showing the recess of the coastline over a period of 20 years.

Since the damming of the rivers, the sediment supply to the coastal zone ceased. The river deltas stopped expanding and in a number of cases the deltas are receding at significant rates, as illustrated in figure 4. The coastline at the Chrysochou river dammed by Evretou dam, receded by 50m in the period 1973 to 1993.

5.2 Erosion Protection Works

In order to combat the problem of coastal erosion a number of breakwaters have been constructed over the past years, such as the ones in Yermasoyeia, Limassol (figure 5). These structures cure the symptoms but not the cause of the problem.



Figure 5. Yermasoyeia Dam and River Delta. Breakwaters as a means of arresting coastal erosion.

The problem of coastal erosion has been studied extensively over the past 30 years. Main causes of the erosion have been identified as the damming of rivers, beach material mining (now stopped), and construction of illegal coastal structures, coupled with the prevailing natural potential sediment transport due to wave action.

Beach nourishment has been evaluated and rejected as a non-viable solution due to the lack of sediments. The adopted strategy is the construction of sea defence works, such as beach revetments and breakwaters (offshore detached, groynes, etc) in urban/ developed areas and the retreat of the coastline in rural/ non-developed areas.

6. POLLUTION RISK

Two further problems that are becoming increasingly important are the quality of the water entering the dams and the quality of the sediments on the beach. Recent forest fires have highlighted the risk of polluting the water stored in the dams. Although the pollution from burned trees might not be so damaging, the risk of pollution due to the burning of industrial, commercial or even housing units might pollute the water in the dams. Although the water from the dams is treated before its distribution for domestic consumption, not all pollutants can be easily removed from the water.

Further risks of pollution include accidents such as road accidents adjacent to the reservoirs, fertilisers and pesticides being carried by surface or ground water.

7. PRE-RESERVOIR

A new proposal was developed (Toumazis et al., 2008) which addresses holistically the problems of water shortage, flood protection, water quality and coastal erosion. This proposal refers to a small dam, called pre-reservoir with the following characteristics:

- It is constructed:
  - Near the entrance of the rivers in the dam reservoirs
  - Upstream of culverts/ bridges in flood risk areas
- Storm water is temporarily stored
- The water quality is checked after the storm:

- If the quality is acceptable, then the water is allowed to enter the reservoir through a pipeline
- If it is not acceptable, then the appropriate measures are taken (e.g. collection and treatment, diversion downstream)

- The sediments are collected once a year and their quality is tested:
  - If their quality is acceptable, sieving takes place and suitable material is transported and deposited at the river mouth/ beach.
  - If their quality is not acceptable, then suitable measures are taken (e.g. treatment, burial etc).

The flow chart of the pre-reservoir operation is shown in figure 6 and a typical cross-section and plan are shown in figure 7.

The river, being seasonal, is normally not flowing and the pre-reservoir is normally empty and the valve draining the water to the main reservoir is normally closed. The water, sediments and vegetation are trapped in the pre-reservoir. If the water is not adversely polluted then the valve will be opened and the water will continue its flow towards the reservoir or the stream/ river.

Once a year, during the dry summer season, the sediments, which are dry, and the organic matter will be relocated outside the pre-reservoir using earth moving equipment. Conventional sieves will be used to segregate the sediments and the organic matter. Organic material will be transported for use in fields for agricultural purposes. Material suitable for the coastal environment will be transported to the beach using road vehicles.

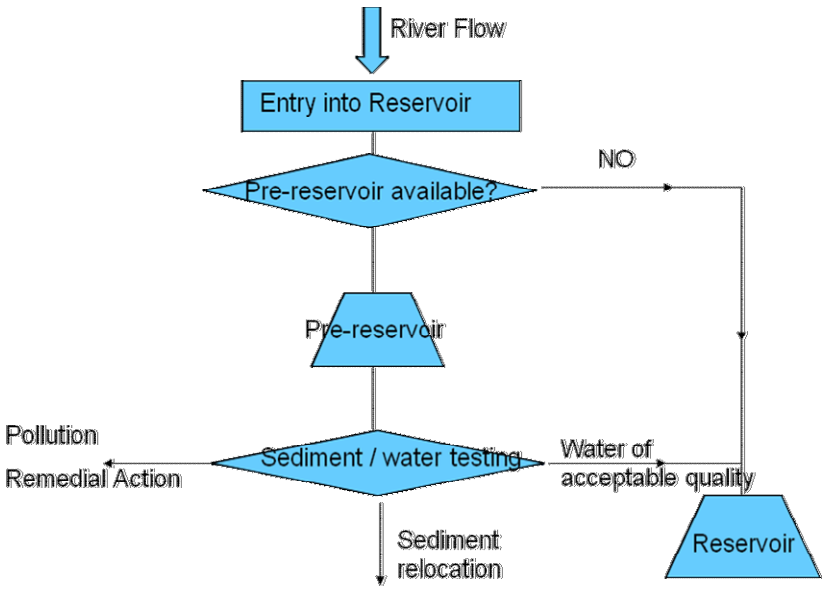


Figure 6. Flow Chart of pre-reservoir operation

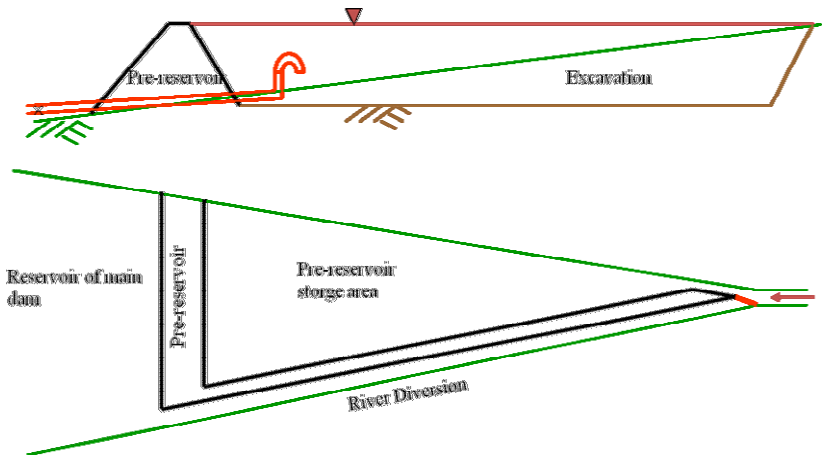


Figure 7. Plan and Cross Section of Pre-reservoir

In the unlikely/ extreme event of an accident such as fire, the polluted storm water will not be allowed to enter the reservoir. It will be either collected or treated (e.g. in case of oil pollution) or be diverted outside the reservoir (e.g. ashes from wood fire).

Similarly, contaminated sediments, unsuitable for bathing beaches, will not be transported to the coast. These sediments will be treated in accordance with the extent of their pollution.

The sediments collected in the pre-reservoirs and transported to the coast will be used for beach nourishment in eroding river deltas and beaches. This is a precious source of sediments and the management of these sediments needs particular attention. Nature would have deposited these sediments, together with the fine suspended sediments that inevitably end up in the main reservoirs and the organic matter (tree branches, leaves, etc) that are placed in the fields, at the river delta. The wave action would sort the sediments and would also cause the rounding/ smoothing of the edges of the material.

8. CONCLUSIONS

Small dams/ pre-reservoirs are constructed at the entrance of the river within the dam basin and upstream of potential flood areas. These are low elevation dams capable to store the water carried by small rainfall events. River water (together with sediments, any branches and pollution) is arrested and temporarily stored within the basin/ pond upstream of the structure (which is normally empty). After the storm, water practically free from sediments and pollution, is allowed to enter the main dam reservoir from where this is diverted for irrigation or water supply, or continues its flow to the sea at a controlled rate. This controlled rate of flow reduces the risk of flooding downstream. Sediments and organic material collected in the small dam basins are sorted and suitable material is transported to the coastal area, thus restoring the natural sediment cycle. Any pollution (result from fire or accidents) is treated before discharge to the environment.

The small dams, pre-reservoirs are considered as sustainable, pro-active measures that reduce the risk of flooding, address the risk of water pollution, contribute towards the alleviation of the drought problem and restore the sediment cycle and the supply of sediments to the coastal environment.

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RIVER AND COASTAL ISSUES IN MALAYSIA: THE ROLES OF ENVIRONMENTAL NGOs IN THE NORTHERN REGION

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Abstract: Environmental NGOs are volunteer organisations that are independent from the government and attempt to contribute towards the good of society and the environment. River and coastal managements which are important to human habitations, ecological systems and the sustainability of the environment are among their main concentrations. The environmental NGOs have equipped themselves adequately and served in a civil society role. If there is any issue to be fought for, they play a vital role in providing solutions and critical opinion by collaborating with other NGOs, via having press conferences, submitting memorandums, organising public signature drives and campaigns, lobbying the government and to some extent, filing cases in court. In some instances, their concerted efforts have managed to convince the state authorities and advocated for policy change or implementation. At times however, they seem to antagonise and be at loggerheads with government policy and actions. The numbers of NGOs in Malaysia are increasing. They have become the power that has to be reckoned with by policy makers. This paper gives an overview of four active NGOs in the Malaysian northern region that deal with river and coastal managements and analyses some of their roles in providing solutions to environmental and social issues.

Key words: coastal management, environmental, NGO, river management, water resources.

1. INTRODUCTION

Deforestation, the agricultural industry and industralisation have threatened rivers, coastlines, ecological systems and human habitations. Various non-governmental organisations (NGOs) in Malaysia have initiated a more active role in helping to conserve and restore natural resources. These non-profit organisations, which are managed by volunteers, are known as environmental non-governmental organisations (ENGOs). The development of ENGOs depicts the development of public awareness on the environment. Sonnenfeld and Mol believe that the emergence of ENGOs is to engage in sustainable development.i This paper gives an overview of the roles of the ENGOs located in the northern region of Malaysia – encompassing the states of Perak, Penang, Kedah and Perlis – in dealing with the current issues related to river and coastal managements.

The roles and activities of ENGOs in the northern region of Malaysia are not clearly known by the public. Why do we need to know their roles? By understanding the roles of ENGOs, one may be aware that they contribute to the common good that facilitates ethical life in protecting the environment and providing space for more debate in the public sphere.ii It is appropriate to the democracy as agreed by Daisy:

> Non-governmental organisations (NGOs) play a vital role in the shaping and implementation of participatory democracy. They should become partners in the implementation of Agenda 21, i.e. to promote the participation, review and evaluation of NGOs for implementation of Agenda 21 and to recognise these activities in the UN reporting system. Their 'experience, expertise and capacity' should be recognised . . . iii

The World Bank's description of the roles and activities of NGOs includes the following:

> …to relieve suffering, promote the interests of the poor, protect the environment, provide basic social services, or undertake community development.iv

2. ENVIRONMENTAL NGOs (ENGOs) IN MALAYSIA

Malaysian environmental NGOs (MENGOs)v which were formed in 2001 under the Danish International Development Assistance (DANIDA) comprise a group of 19 ENGOs.vi The mission of the MENGOs aims to be an independent platform of Malaysian NGOs that is committed towards enhancing the environmental sustainability agenda at the local, national and international levels.vii According to Ramakrishna, every MENGO has its own way of dealing with environmental issues.viii MNS and WWF-Malaysia, for example, give talks about the lifestyle of sustainable wildlife and have formed Environmental Clubs at schools.

The river – which is our most precious natural resource – has always been the concern of these ENGOs. Based on the research that we conducted in Malaysia in 2008, 50 percent of the NGOs registered under the MENGOs have concentrated mainly on the river while 30 percent of them concentrate mainly on the sea/marine resources. In the next section of this paper, we will give a broad overview of four members of the MENGOs which deal with river and coastal managements in the northern region of Malaysia encompassing Water Watch Penang (WWP), the Malaysian Nature Society (MNS), Friends of Ulu Muda II (FoUMII) and Sahabat Alam Malaysia (SAM).

2.1 Water Watch Penang (WWP)

Water Watch Penang (WWP) is a non-profit organisation set up in November 1997 as part of the Sustainable Penang Initiative. WWP was established after concerns over Penang's water quality and availability, as well as flooding problems.ix Working in partnership with the Penang Water Supply Corporation and various authorities, WWP's objectives are to create awareness for water conservation amongst the general public, to do research on water issues, to provide water education in schools and to encourage the practice of a water saving society towards the sustainable development of water resources.x Chan, who is the President of WWP, states that:

> Currently, Water Watch Penang (WWP) is probably the only water NGO in Malaysia, although other NGOs such as the Malaysian Nature Society (MNS), the Environmental Protection Society Malaysia (EPSM), Sahabat Alam Malaysia (SAM) and to a lesser extent consumers associations such as the Consumers Association of Penang (CAP) and the Consumers Association of Selangor (CAS) have from time to time dealt with water issues. xi

2.1 (a) Major Issues

The major issue faced by Malaysia is the impressionn that Malaysia has no shortage of water supply. This opinion is based on the yearly heavy rains of more than 2,000 mm and frequent flooding. However, this myth has not been proved true because Malaysia had water crises in

Melaka in 1991 and in Kuala Lumpur and Selangor in 1998, which led to suffering and economic losses.xii Furthermore, the domestic use of water in Malaysia has increased. According to Chan:

> If we consider the fact that the International Standard for water use recommended by the United Nations is 200 LPD, then Malaysians now are guilty of wasting 100 LPD. Malaysians living in the urban areas waste more than 300 LPD, i.e. an amount that can be used to sustain six persons in Sudan. …. As a comparison, an average Indian (in India) uses only 100 LPD and a Sudanese uses even less, about 50 LPD.xiii

The next major issue is that the total water catchment has reduced due to uncontrolled logging and deforestations. Deforestations can cause serious irreversible consequences on both their immediate environments as well as the downstream areas, often with a cumulative effect; glaring disasters related to deforestations in Malaysia include the collapse of the Highland Towers in 1993, the landslides of the Karak Highway in 1995, the mudslide in Pos Dipang in 1996xiv and the landslides of Bukit Antarabangsa in 2008.xv Penang has experienced water depletion due to deforestations and hill land developments in Penang Hill, Paya Terubong and other hills as well as water hazards such as flash flooding.xvi

2.1 (b) WWP's Contributions to the Issues

Regarding these issues, WWP has taken some initiatives in protecting the water catchments in Penang such as writing articles in newspapers, educating people and school children about the importance of water conservation, contributing its expertise in national forums and workshops on water and helping the government draw up plans and programmes for water conservation. Examples of its projects are "River Walk – Caring for Our Rivers Campaign", "Clinic for Penangites on the Importance of Looking after Rivers", "Adopt a River Project" and "Water Conservation and Educational Programme" with the Penang Water Supply Corporation.xvii

2.2 The Malaysian Nature Society (MNS)

The Malaysian Nature Society (MNS) is the oldest NGO in Malaysia which was incepted in 1940. The objectives of MNS are to promote the study, appreciation, conservation and protection of Malaysia's natural heritage focusing on sustainable development and biological diversity.xviii Although it has various activities related to the environment, river conservation is one of its concerns. Amongst its many activities related to river resources was a water forum in 2000xix which focused on raising awareness among local communities, targeting resident associations and other grass roots organisations.xx

2.2(a) Major Issues

In 2006, the forest reserves of Belum and Temenggor, in the state of Perak, suffered from an onslaught of logging activities. The logging reached the foot of the mountain ranges and caused steep slope and high erosions. Situated at the border of Thailand and Peninsular Malaysia, these 130 million-year old forests are said to be older than the Amazon and the Congo. They cover 300,000 hectares in area and are catchments for major rivers. They are also an irreplaceable haven for numerous plant species that support over 270 species of birds and 100 species of mammals, including the endangered Malayan tiger, Sun-bear, Seladang (Gaur), Sumatran rhinoceros, Malayan Tapir and the globally threatened Plain-pouched Hornbill.xxi The Belum-Temenggor forest has world-class wildernesses, with beautiful waterfalls and has some of the largest virgin forests which are home to many tribes of indigenous people, the Orang Asli. The outstanding value of the forest is a great asset to the country. Once logged, the ecological systems will be fragmented and damaged beyond repair, generating huge losses and costs for the nation to compensate.

2.2(b) MNS's Contributions to the Issues

Recognising the importance of the Belum and Temenggor forest reserves, MNS ran a vigorous campaign in 2006 to save them from the onslaught of logging activities. In the campaign, MNS strongly encouraged the Perak State Government to preserve the integrity of the Belum-Temenggor forest. MNS sought the participation of people in the Belum-Temengor campaign for six months and the campaign obtained more than 80,000 signatures. To continue the initiative for the campaign, general programmes for education and awareness initiatives have been carried out.xxii Eventually, after a year of initiatives, the Perak State Government gazetted Belum as a State Park on 3 May 2007 with no further logging activities.xxiii

2.3 Kedah's Friends of ULU Muda II (FoUMII)

Kedah's Friends of ULU Muda II (FoUMII) is a movement comprising 17 green groups which are listed under the MENGOs.xxiv This movement was formed in response to a controversial issue of helicopter logging by the Kedah State Government.

2.3 (a) Major Issues

In 2002, the Kedah State Government proposed the helicopter logging of the Ulu Muda Forest Reserve in the north-eastern corner of Kedah which covers 162,931 hectares.xxv In order to take the logs out of the forest readily, it may need to build an extensive road network 10-24 metres wide totalling 404 kilometres which would result in the clearing of more than four million trees. Such a damaging activity would no doubt seriously threaten the economic values of the Ulu Muda water catchment areas, may cause soil erosions, flash floods and sedimentations of dams.xxvi Cutting the trees within the watershed of Sungai Muda will seriously affect the water supply for the people of the northern region. According to Mohd Nizam, the Forest Reserve supplies 80 percent water resources for both domestic and industrial use in Penang, Kedah and Perlis.xxvii

2.3 (b) FoUMII's Contributions to the Issues

In dealing with the issue, FoUMII organised a media conference in Alor Star, Kedah, on 10 July 2008, urging the Kedah State Government to halt the logging activities. The media conference stressed that logging would not only destroy the forest ecosystem, but also destroy the livelihood of 63,000 rice farmers in the Muda Agriculture Development Authority (MADA) which covers the area of Perlis, Kedah and Penang, so aptly known as the Rice Bowl of Malaysia. Mohd Nizam recommended that the Kedah Government gazetted the Ulu Muda Forest Reserve as a state or national park, set up a Ulu Muda Trust Fund to protect the catchment areas and urged the Federal Government to compensate the Kedah Government for not logging in the area. FoUMII also plans to collaborate with the Penang Water Supply Corporation and enlist the support of the local people through public forums, press conferences, petitions and demonstrations against the logging proposal. Information on these campaigns is conveyed through *You Tube*, a blog (saveulumuda.wordpress.com), Facebook, brochures, pamphlets, streamers, logo design campaigns and t-shirts.

2.4 Sahabat Alam Malaysia (SAM)

SAM is another NGO which actively educates the public in evironmental awareness and helps the community in aspects of development and social issues. It works closely with communities such as supporting the indigenous people against deforestations and the seizing of ancestral lands, helping the farmers whose crops have been destroyed by pollutions and pests, helping estate

workers whose health and safety are affected by the use of toxic chemicals, helping fishermen whose livelihood is threatened by depleting marine resources and the encroachment of trawlers, etc. According to Santharamohana of the Natural Resources Environment, SAM [together with CAP] is seen as the most provocative NGO in the issue of environmental harmony.xxviii He remarked:

> They always organise public dialogues, obtain signatures from local community, make opposition memorandums and organise workshops for farmers and coastal fishermen. They also provide legal support to the society and file cases in court if the cases could not be solved through negotiations.xxix

2.4 (a) Major Issues

Malaysia has some of the largest mangrove forests in Asia, however; the country has experienced a high destruction of coastal ecosystems over the past decade, due to, mainly from the felling of mangrove trees for charcoal, timber production and coastal development. SAM stated that three decades ago, Malaysia had 30 percent of the total land area grown with mangroves; today, however, it has only two percent or about 650,000 hectares.xxx In the island of Penang, the mangrove forest area has declined to only 900 hectares and only half of them are considered as forest reserves–the destruction since 1966 amounting to 130 hectares of mangroves per year.xxxi

1. Mangrove forests make up one of the most unique ecosystems, where they thrive in the transition zone between the ocean and land, the habitat where no other trees can survive. The tangled roots of the trees provide shelter for an incredible variety of terrestrial and many aquatic organisms such as estuarine fishes, crabs and various invertebrates. The mud of the mangrove swamps is actually a very important part of the food chain. The water in the swamp is very clean because the roots of the trees absorb the organic material from the water. Later, the fine particles in the water combine with the mangrove tree barks and leaves to form sediment. This allows algae to grow in the sediment. The algae are food for water fleas, snails, frogs and mudskippers. Decomposed leaves are food for crabs and shrimps. However, all these complex ecosystems are being destroyed by various activities of man. These include the excavation and refurbishment of mangrove forest areas for the coastal development of hotels, resorts, human habitations and industrialisation such as the construction of the airport in Bayan Lepas and the Batu Maung industrial estates in Penang.xxxii

The exploitation of mangroves endangers not only the mangrove forests, but also the interacting environmental resources and local societies which rely upon their existence. Mangrove forests are vital in the coastline zone as they act as a buffer against frequent hurricanes, tidal thrusts, tsunamis and cyclones. Coastal villages, which have lost mangroves, are easily battered by strong waves and storms which can cause critical coastal erosions and threaten villagers' lives. According to a research done on the Indonesia-Malaysia coastline, the aftermath of the 2004 tsunami which devastated the country's northern coastal areas, indicated that mangrove forests played a crucial role in saving human lives and property.xxxiii

M. Nizam Mahsharxxxiv of SAM pointed out that the 2004 tsunami proved that areas with mangrove forests are less affected as they are able to absorb the impact from the big waves, thus causing only minor damages.xxxv Interviews with fishermen conducted by a group of researchers at Kampung Kubang Badak, Langkawi in the state of Kedah, revealed that the fishermen were able to take shelter in the mangroves during the tsunami and the fish landing facility was only slightly damaged.

These fishermen were quick to point out that the Kubang Badak mangrove forest had buffered them against strong waves and quick-rising waters. The fishing village in Kuala Teriang, northeastern Langkawi, was not as fortunate. The village was flooded, property damaged and one life lost. This may be due to the clearing of the mangroves in the area, exacerbated by the construction of a manmade wave barrier near the Langkawi airport. The wave barrier and the

existing land mass created a narrow channel that could have increased the velocity of the waves and caused significant damage to the area.xxxvi

Another cause of mangrove deforestation is the building of ponds for the shrimp aquaculture industry. The Mangrove Action Project, an international network of over 450 NGOs and over 250 scientists and academics from 60 nations, notes that the shrimp aquaculture industry is the greatest threat to the world's remaining mangroves and it is estimated that 38 percent of recent mangrove losses may be attributable to shrimp farm development.xxxvii

The loss of mangrove trees has dwindled the incomes of mud crab catchers of Sitiawan, Perak, after the mangroves along Sungai Dinding were cut to make way for prawn breeding projects.xxxviii In Kuala Muda, Kedah, the livelihoods of 3,000 fishermen from 15 villages are now seriously threatened – within a few years, after the mangrove forest area was approved by the State Government for aquaculture in 1991, coastal fishermen reported that their incomes had dropped sharply from RM30 to RM5 per day.xxxix In Penang, there are hundreds of hectares of shrimp ponds such as in Kuala Sungai Pinang and Pulau Betong at Balik Pulau while there are shrimp ponds in Sungai Chenaan and Batu Kawan at Seberang Perai Selatan. Batu Kawan fishermen reported that areas where mangrove forests have been cut down have registered a gradual decline in fish catch over the years; this has led to the extinction of several commercial fish species in some places. The Penang Inshore Fishermen Welfare Association (PIFWA) concluded that 34 species of fish have become extinct and another 50 or more are becoming rare in Penang. xl

2.4 (b) SAM's Contributions to the Issues

There are many initiatives aimed at conservation and restoring the mangroves in the Malaysian northern region. One of the important avenues to protect mangroves is through awareness campaigns. On 26 July 2008, SAM, together with the Consumer Association of Penang (CAP) and the Penang Inshore Fishermen's Welfare Association (PIFWA), organised a mangrove awareness campaign to university students, school children and the local community in Taman Nilam, Sungai Petani, in the state of Kedah.xli In the campaign, SAM also put up streamers at the village containing information regarding the importance of mangrove forests to the ecological system and human life, statistics on the mangrove trees in Malaysia and the impact of mangrove destruction. A Public Service Anouncement (PSA) video about the negligence of the environment was also shown.xlii One major activity of the campaingn was replanting mangrove trees in which the participants sowed mangrove saplings in polybags.

SAM conducted a similar project on 26 December 2006 in Kedah, Penang and Perak to mark the second anniversary of the tsunami tragedy which hit these states two years before. Nine hundred mangrove trees were planted along the coast at the Merbok mangrove forest and 1,000 more at Kuala Tunjang, Kerpan, Kedah. In Penang, mangrove trees were planted in Kuala Kurau and Kuala Haji Ibrahim.xliii SAM recorded that they had succeded in planting more than 100,000 mangrove saplings in Penang.xliv In a 2008 press conference, SAM has urged all state governments to gazette the remaining mangrove areas as forest reserves for perpetuity and halt development projects and aquaculture in such areas.xlv

3. CONCLUSION

The research explores environmental issues related to river and coastal developments which have been among the major concerns of the ENGOs in Malaysia. An overview of the activities and roles played by the ENGOs in dealing with issues in the northern region may suggest that the ENGOs do not act on their own but in a public sphere and they can form the missing link between government and the people. Although ENGOs do not offer comprehensive solutions to some of the existing problems, they may increase public awareness about the state of their environment and advocate improvements in policies related to the issues.

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DETERIORATION OF COASTAL WATER AQUIFERS: CAUSES AND IMPACTS

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Abstract: Securing water for sustainable development is a major challenge facing the global community. Provision of safe adequate water to meet demands of domestic sector remains a focal objective to be attained for developing countries. Developing countries at present face fundamental environmental pressures induced by high population growth, rapid urbanization, and deficient water sector services reflecting on improper management of water resources. As such, communities seeking to cope with these challenges resort to an overexploitation of groundwater aquifer. The objective of the study is to assess the quality profile of potentially used coastal ground water aquifer and its impacts on public health for three main coastal cities in Lebanon (Beirut, Tripoli and Saida). Water samples were collected during the dry season, from drilled private wells of the indicated coastal cities that exhibit high population density. Water samples were assed for their physiochemical profile that encloses indicators of seawater intrusion (Conductivity, total dissolved solids, temperature, pH, dissolved oxygen, Ca and Mg content, alkalinity, nitrate, and sulphate). The collected data were analyzed using the software "AQUACHEM" interfaced to "PHREEQC" geochemical models, and the SigmaStat statistical Package. Preliminary results expose major progressive increase in water salinity mainly induced by seawater intrusion into the coastal aquifers. These results emphasize the importance of instating integrated water resource management to ensure sustainability and viability of this main water resource.

Key words: Coastal aquifers, IWRM, Lebanon, seawater intrusion, water modelling.

1. INTRODUCTION

Securing water for sustainable development is a major challenge facing the global community. At present, developing countries face environmental pressures induced by high population growth, rapid urbanization, and deficient water sector services reflecting on improper management of water resources (UN 2003; WHO 2006). As such, communities seeking to cope with these challenges resort to an overexploitation of groundwater aquifers. The overuse of groundwater in coastal areas due to high population density results in seawater intrusion into the coastal aquifer. Sea water intrusion is often associated with over pumping in coastal regions, resulting in overdraft conditions and creating an inland gradient of seawater (El Moujabber et al. 2006). The intrusion of saltwater into coastal aquifers is a wide spread phenomena, especially in Mediterranean regions where semi arid conditions lead to excessive pumping and high extraction rate and low recharge (Petalas et al. 2009).

Lebanon is relatively blessed with adequate resources in the region. However, recent attempts to estimate water demands and availability in Lebanon have revealed significant deficit. Domestic water need in Lebanon is estimated at 850,000 m^3/day and only 450,000 m^3/day is available. The daily domestic water consumption in Lebanon was estimated at 165 l/c/day in the mid-1990s. This figure is expected, due to population growth, to reach 260 l/c/day in 2015 (El-Fadel el al. 2003; Basma 2004; Korfali and Jurdi 2008). Beyond the impact of population growth, water demand has been rising in response to industrial development, increased reliance on irrigated agriculture, massive urbanization and rising living standards (Jurdi et al. 2003). The problem of water shortage is prominent in the populated coastal cities, and aggravated in the city Beirut, the capital of

Lebanon. Furthermore, coastal cities in Lebanon face increased water demand. Securing water for the populated cities in Lebanon and mainly for the city of Beirut has lead water authorities to adopt rationing of water supply, 10 hr every other day (El-Fadel et al. 2003). This water rationing by the Lebanese Water Authorities, as a remedial action, became a firmly established practice during the past four decades. Consumers were then motivated to resort to private wells for supplementary water. Exploitation of groundwater through private wells in Lebanon is still increasing up to the present time. The rush of uncontrolled drilling of private wells is more prevalent in coastal areas and especially in western part of Beirut city (Lababidi et al. 1987; Acra and Ayoub 2001; Korfali and Jurdi 2008). The excessive exploitation of ground water over the years leads to the infiltration of seawater and the deterioration. The extent of infiltration of sea water in private wells of Ras-Beirut sector of Beirut city and deterioration of the fresh water aquifer has been previously reported (Acra and Ayoub 2001; Korfali and Jurdi 2007, 2008). Thus the objective of this study is to assess the quality profile and extent of seawater intrusion in coastal groundwater aquifers of three main coastal urban populated cities in Lebanon (Beirut, Trippoli and Saida). These three cities exhibit poor management of water resources, high demand for water and variable over pumping of ground water aquifers.

2. MATERIALS AND METHODS

2.1 Study Area

The study included three coastal urban cities (Beirut, Trippoli and Saida) of variable characteristics with respect to population densities and water bearing succession. Beirut city is the capital of Lebanon. Beirut city and its suburbs (Greater Beirut) are overpopulated; it is the residence of 2.188 million citizens (1/3 of Lebanese citizens) and the population density is about 20,167 persons/km^2 and projected to be 33,000 persons/km^2 in the year 2015.Wheras, Trippoli city, 80 km north of Beirut; and Saida city, 42 km south of Beirut are smaller cities and less populated than Beirut. The total number of residence in Trippoli and Saida are respectively 250,000 and 167,000. The population density of Trippoli is 12,000 persons/km^2 and that of Saida is 5,000 persons/km^2 (MEO 2001; ERM 1995). Beirut city is noted for its excessive uncontrolled drilling pf private wells. The rush of uncontrolled drilling of private wells in Beirut city has been practiced since the years 1970. In the year 1972, Pathan (1977) noted that for Ras-Beirut sector (small coastal sector of Beirut city) the number of wells was 50 wells/km^2 and in a recent study (Korfali and Jurdi 2007) it has reached 375 wells/km^2. However, Literature does not cite the estimate number of private wells in either Tripploi city or Saida city. From our study, citizens in these cities mainly utilize municipality water for their domestic usage. But, recently due to population growth and water demand, citizens resorted to private well drilling.

The geological formations of Beirut city belong to the middle Cretaceous and the Quaternary period. The Cretaceous rocks are mainly of Sannine limestone formation (C4) that constitutes the most important karstic system of Lebanon, the best water bearing tower in Lebanon and classification as excellent aquifer. Whereas, the Quaternary deposits (mainly alluvial deposits) that consists of a sequence of gravel, sand, silt and clay can give minor water quantities. Whereas, the geological formation of Trippoli and Saida are nearly similar with respect to mainly belonging to the Quaternary period and consisting of the quaternary alluvial deposits; and to some extent to the Tertiary Miocene succession (m), the marine succession that is classified as aquifer. But, for Saida city patches of middle Cretaceous period exist constituting of C4 limestone type that is classified as excellent aquifer.

2.2 Sampling

Well water samples were collected during the dry season. The sampling sites were chosen after preliminary survey based on available well and allowance to take samples from well. The number of samples taken from Beirut city was 120. Whereas, due to the less occurrence of private wells in Trippli and Saida cities, only 30 well samples were collected from Trippoli and 21 samples from Saida. The method of sampling and collection are in accordance with Standard Methods for the Examination of Water and Wastewater (APHA 1998).

2.3 Analysis

Parameters sensitive to environmental changes were measured on site. Temperature, electrical conductivity (ECw) and total dissolved solids were measured using the HACH Model 44600 Conductivity/TDS Meter. The pH using the HACH pocket pH Model. Whereas, laboratory analysis was performed for the other needed parameters: titration procedure was used for alkalinity (0.02N H_2SO_4), Cl^-(0.014N mercuric nitrate), Ca, Mg and total hardness (0.01M EDTA); spectrophotometric SO_4^{2-}(turbidimetry) using the HACH Model ('HACH" Odyssey, DR, 2500); and flame photometer technique ("Thermo" FISHER, PFP7) for the analysis of Na and K.

2.4 Software Used IN Analysis Area

The statistical analysis of the measured physio-chemical parameters was performed using SigmaStat software. AquaChem interfaced to PHREEQC software was used to calculate the CH (Carbonate hardness) and NCH (Non carbonate hardness) in water samples and the ratio of water parameters related to seawater, and water type prediction.

3. RESULTS AND DISCUSSION

3.1 Water Quality

The mean values for the various measured well water parameters relating to water quality and seawater intrusion of the three studied coastal well cities are presented in Table 1. Well water samples were collected in the dry season. In the dry season, the recharge of ground water is nil. This leads to limited dilution of water parameters at the time when water use is at its highest peak.

Table 1. Mean values measured parameters of studied well water

	Beirut			**Trippoli**			**Saida**		
	Min	**Max**	**Mean**	**Min**	**Max**	**Mean**	**Min**	**Max**	**Mean**
Temperature(^{o}C)	22	26	24	21	23	22	21	23	22
pH	7.1	8.6	7.7	6.3	8.8	7.5	6.7	8.8	7.8
Cond (µS/cm)	900	5931	2415	495	3770	1505	459	4220	1671
Alkal(mg/CaCO3/l)	170	425	292	170	390	303	190	490	298
TH (mg/CaCO3/l)	430	1015	625	210	860	484	240	860	444
CH (mg/CaCO3/l)	160	460	305	115	400	310	185	480	287
NCH (mg/CaCO3/l)	270	555	320	10	490	194	20	490	145
Ca (mg/l)	48	180	110	16	176	100	40	172	105
Mg (mg/l)	10	127	76	12	117	51	4.8	117	48
Na (mg/l)	20	1015	495	53	497	264	29	550	279
Cl (mg/l)	200	2826	1059	200	960	574	150	1150	682
SO4 (mg/l)	15	155	78	15	140	55	11	160	50

The three studied coastal areas revealed high mean values for the mineral content of well water; and the highest for Beirut wells. The conductivity, as well as the concentrations of Cl, Na, Ca are

above drinking water standards recommended by USEPA (2003) or WHO (2006). The mean value of conductivity levels in Beirut wells was 2415 µS/cm (≈ 2 times higher than the recommended upper limit set by USEPA, 1250 µS/cm). Whereas, the conductivity values of Trippoli and Saida were respectively 1505 µS/cm and 1671µS/cm (≈1.2 times higher than USEPA recommended values). The chloride mean concentration (1059 mg/l) of Beirut wells is four folds those recommended upper level of USEPA (250 mg/l). Whereas, the mean Cl concentration of coastal wells of Trippoli was 574 mg/l and Saida was 682 mg/l (≈ 2.2 to 2.7 higher than set recommended values). This high chloride content for the three coastal cities does not imply a health hazard, but affects the taste of water (WHO 2006). The concentration of Ca in the three studied areas was slightly higher or approaching the recommended value (100 mg/l); the mean Ca concentration for Beirut, Trippoli and Saida were respectively 110 mg/l, 100 mg/l and 105 mg/l. On the other hand, the set recommended value for Mg by USEPA is 30 mg/l. The mean Mg concentration of Beirut wells was 76 mg/l (≈ 2.5 times higher than recommended value); whereas, those of Trippoli and Saida were respectively 51 mg/l and 48 mg/l (≈ 1.6 times higher than recommended values). Though WHO (2006) does not indicate the health hazards resulting from a considerable excess in the ion concentration of Cl, Mg, and Ca, still these ions affect the household infrastructure and impact of corrosion of domestic pipes (WHO 2006). The mean values of pH (7.5-7.8) in the three studied areas are typical of water arising from carbonate bedrock (Stumm and Morgan 1996). However the order of abundance (concentration in mmol/l) of major ions in the studied water wells (cations: Na > Ca > Mg; anions Cl > HCO3 >SO4) do not reflect the major ions arising from carbonate bedrock (cations: Ca > Mg > Na; anions HCO3 > Cl >SO4). This observation was also illustrated in the predicted water type of the wells by the AqauChem software. The AquaChem software predicted for Beirut wells, water type to be 60 % as Na-Cl, 25 % as Na-Ca-Cl-HCO3, and 15% as Na-Ca-Mg-Cl-HCO3. Ninety percent of samples had Na as highest concentration major cation and 95 % had Cl as highest concentration major anion. Trippoli and Siada showed similar water type about 65 % as Na-Ca-Cl-HCO3, 25 % as Na-Ca-Mg-Cl-HCO3, and 5% as NaCl. The overall water type of each studied coastal areas and major ion distribution are presented in stiff diagram (Figure 1). The water type of Beirut wells using mean measured values represented a water type similar to NaCl, while those of Trippoli and Saida represented a water type of Na-Ca-Cl-HCO3. The predicted well water types of the three coastal areas differ from those arising from the weathering and dissolution of karstic bed rock aquifer (calcite and /or dolomite) that evolve Ca-HCO3 and Ca-Mg-HCO3 water type. The revealed occurrence of Na and Cl in the water type (Figure 1) suggests that this factor is primarily due to seawater intrusion, a contaminant factor that prevails the natural environmental setting of the wells.

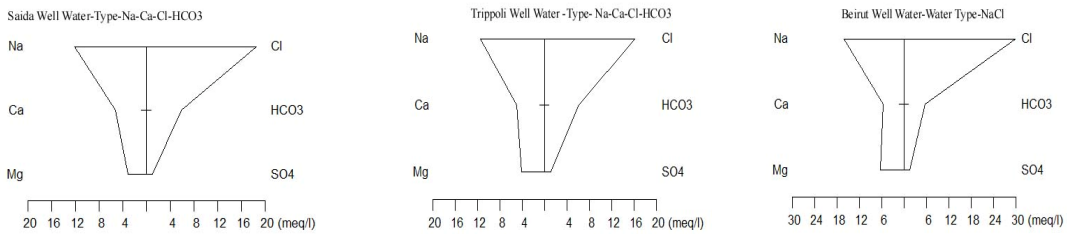


Figure1. Comparison of water type and major ions of the three studied areas.

2.2 Sea Water Intrusion-Indicators

Seawater in general has a uniform chemistry with the prevalence of Cl^- and Na^+ possessing a molar ratio of 0.86 (mass ratio= 0.55). Sea water solutes are specifically characterized by excess Cl^- over alkali ions (Na^+ and K^+) and Mg^{2+} greatly in excess of Ca^{2+}. In contrast continental fresh ground waters are characterized by variable chemical composition, the predominant anions are

HCO_3^-, SO_4^{2-} and Cl^- , and the primary main cations are Ca^{2+} and Mg^{2+} and to lesser extent the Na^+ and K^+. In most cases Ca^{2+} predominates over Mg^{2+}. Whereas, the chemical composition of groundwater in many coastal aquifers subjected to seawater intrusion deviates from simple conservative seawater-fresh water mixing (El Moujabber et al. 2006). Several criteria are suggested to be used as indicators for seawater intrusion into coastal aquifers. Revelle (1941) pointed out that the increase in total dissolved salts (electrical conductivity) is not sufficient proof of occurrence of seawater intrusion. Seawater intrusion involves mixing between saline and fresh water. Owing to its considerable salt content, a small fraction of sea water would dominate the chemical composition of groundwater mixture. The most obvious indication of seawater intrusion is an increase in Cl^- concentrations. Another important indicator related to the predominance of Cl^- in seawater is the ratio of chloride to carbonate and bicarbonate ions which is known as Simpson ratio (El Moujabber et al. 2006), bicarbonate and carbonate ions are present only in very small amount in sea water. Others (Acra and Ayoub 2001) have used beside Chloride increase, the increase in values of conductivity, total hardness (TH), magnesium hardness (MgH) and non carbonate hardness (NCH) as indicators for seawater intrusion, since they are higher in seawater when compared to those of groundwater. Moreover seawater intrusion is characterized by the ratio of Na/Cl. The Na/Cl ratios of coastal aquifers subjected to seawater intrusion are usually lower than the marine values, since when salt water enters into fresh aquifers, an exchange of cations occurs and Na^+ is taken by the exchanger (Petalas et al. 2009). Finally, the ratio of Na/Ca and Mg/Ca may be used as indicator owing to the fact that Na and Mg are higher in seawater than fresh ground water.

The seawater intrusion of the studied areas are assessed using the above indicated seawater intrusion indicators. Table 2 provides a comparison of the mean values of well water parameters from this study, the seawater samples and average wells in Lebanon adapted from Acra et al. 1883. Table 3 presents the ratios of the parameter that indicate seawater intrusion. Note that calcium hardness is CaH, magnesium hardness is MgH, carbonate hardness is CH, and noncarbonate hardness is NCH, all of these have the unit as mgCaCO3/l.

Table 2.Comparison of major well water parameters to seawater and ground water

	Cond. mS/cm	**CaH***	**MgH***	**CH***	**NCH***	**Cl** mg/l	**Na** mg/l
Beirut well water (n= 120)	2,415	306	306	305	320	1,059	495
Trippoli well water (n=30)	1,505	279	205	310	194	574	264
Saida well water (n=21)	1,617	270	190	287	145	682	279
Wells-Lebanon (n=225)	1,028	280	71	271	80	162	24
Sea water-Lebanon (n=15)	55,850	1,245	5,595	127	6.711	2,2815	12,320

Table 3.Mean values for ratios used as seawater intrusion indicators

	Cl/Cond.	**NCH/CH**	**Cl/carbonate**	**Na/Cl**	**Na/Ca**	**Mg/Ca**
Beirut well water (n= 120)	0.32	1.09	2.64	0.48	3.7	0.63
Trippoli well water (n=30)	0.32	0.82	1.88	0.44	2.9	0.46
Saida well water (n=21)	0.35	0.5	1.6	0.41	2.6	0.32
Wells-Lebanon (n=225)	0.16	0.3	0.6	0.13	0.21	0.15
Sea water-Lebanon (n=15)	0.41	53	147	0.55	24.73	2.69

The intrusion of seawater into wells of the studied coastal cities is obvious in Table 2. The conductivity, magnesium hardness, noncarbonate hardness, chloride and sodium are higher than the average values of Lebanon's wells (recall, these parameters are higher in seawater than groundwater). The highest aforementioned parameter values are for Beirut wells. This can be attributed to the fact that there is more number of drilled coastal wells/km^2 in Beirut city and the well extraction rate is higher. In addition, the coastal aquifer of Beirut belong to Cenomanian, Sannine formation (C4) karstified hard limestone susceptible to seawater intrusion (El Moujabber et al. 2006). While the number of wells in Trippoli city and Saida city are less and the coastal aquifer is mainly Quaternary alluvial deposits that enhance horizontal seawater intrusion. What are more explicit for the seawater intrusion of the studied wells are seawater ratios (Table 3). Contamination

by seawater may be classified by Simpson classification ratio of Cl^-/ (HCO_3^- + CO_3^{2-}). This classification included five classes: 0.5 for good water quality, 1.3 for slightly contaminated water, 2.8 moderately contaminated, 6.6 injuriously contaminated, and 15.5 highly contaminated water (Todd 1959). Consequently, Trippoli and Saida wells ratio of Cl^-/HCO_3^- + CO_3^{2-} are respectively 1.88 and 1.6 are slightly contaminated. Very few wells revealed values of 2.8 (moderately contaminated). Whereas, Beirut wells according to classification (ratio of Cl^-/HCO_3^- + CO_3^{2-} is 2.64) are moderately contaminated. Nevertheless, some wells had a ratio of 8.8 and are above injuriously contaminated wells. In regard to the ratio of Na/Cl, the ratio of studied wells is much higher than average Lebanon's groundwater and is approaching seawater (Table 3). The highest ratio is for Beirut wells (0.48). The lower ratio for the other two coastal cities (Trippoli and Saida) could also be attributed to these coastal aquifer formation, the alluvial deposits consisting of clay (exchanger) and Na^+ is taken by clay particles (Petalas, 2009). Though Mg/Ca is much below the seawater ratio, yet being higher than Lebanon's groundwater (Table 3) indicates the seawater intrusion into well waters. The deduction of seawater intrusion through ratio of Na/Cl is more explicit than Mg/Ca ratio due to Mg reactivity and solubility. Magnesium forms insoluble carbonates that allow it to exist at a lower apparent concentration than sodium compounds that occur in soluble form (Drever 1997). The seawater intrusion is further proven through the AquaChem software model and Stiff diagram (Figure 1) that predicted occurrence of Na and Cl as prime major ion water type. The intrusion of sea water into Beirut aquifer is so aggravated that the predicated water type of Beirut coastal wells to be of Na-Cl type.

4. MANEGMENT PLAN

Based on the results of the study, the deteriorative condition of the coastal aquifers is evident and is highly aggravated in the coastal aquifer of Beirut city. The results emphasize the importance of instating integrated water resource management plan to ensure sustainability and viability of coastal aquifers that are of main Lebanon's water resources. The absence of management plan is obvious in studied area due to the over pumping resulting in seawater intrusion. The management plan is to secure adequate sufficient water to meet domestic needs and halt the progressive deterioration of coastal aquifers. The proposed management strategy addresses three issues: legislative, water quantity and water conservation.

4.1 Regulation and Legislation

The Lebanese government since 1967 legislated a law (86/87, December) that prohibits ground water exploitation for private usage in Greater Beirut. The law was never implemented. Proper tools and instrumentations for the execution of the legislation by the respective authorities should be tackled and be of high priority in the management and protection plans of water resources. The regulations of the use of private wells and legislations should address the whole coastal zones of Lebanon. Recently Lebanese authorities set many legislations related to environmental protection, but no mention to the protection and preservation of ground water aquifers (MEO 2001). In addition, the licensing of private well drilling, as a whole in Lebanon, should be approved based on adequacy and quality of water, number of wells in the area and abstraction rate. As well as, water authorities set strict enforcement rules on the permissible abstraction rate and adopt water metering systems and pricing for the private well water utilization (Korfali and Jurdi 2008).

4.2 Water Quantity

Instating an integrated water resources management (IWRM) plan would cause an increase in water quantity and availability, and decrease the exploitation of coastal aquifer and quality

deterioration. Integrated water resource management is a systematic process for the sustainable development, allocation and monitoring of water resources use in the context of social, economic and environmental objectives. It involves integrating the work of a number of water sub-sectors, such as hydropower, water supply and sanitization, irrigation and drainage. One of the management's instruments of IWRM is the measure designed to mange water supply and demand (supply/demand water assessment). In Lebanon, there is no set national IWRM plan. The available document that could be considered as an IWRM is "Work Plan of the Ministry of Energy and Water years 2000-2009". The work plan presents plans, strategies, and policies relevant to potable water, irrigation and waste water (UNDP 2004). The document satisfies several aspects according to the guidelines for IWRM development. However, the document focused on domestic water supply instead of integrated water resource management. The plan is not yet all satisfied, it runs up till now in most as on going activities (part of which was accomplished is the rehabilitation of networks of water and wastewater). Therefore, the problem of supply and demand should be addressed more effectively by authorities with rapid interventions; such as, adopting a more rapid process for implementing the need of 16-20 dams through out the country (Amery 2002), and instating the rain harvesting water plan. In addition is the adoption of artificial recharging of ground water. Recharging of ground water was practiced by the Ministry of Water and Electric Resources from 1968-1975. Recharging proved to be quite effective especially in controlling seawater intrusion in aquifer of coastal /Beirut area. The operations were discontinued because of civil war. A new feasibility study for artificial recharge of coastal aquifers should be adopted. Moreover, a plan to protect water resources in Lebanon is of utmost need to increase water availability and it would reduce the projected deficient in water supply. Water resources in Lebanon are subjected to contamination resulting from haphazard management of municipal solid wastes and wastewater (Korfali and Jurdi 2008). Therefore, policies and plans for integrated management of solid wastes should be an integral part within national water management plans and among the identified priorities.

4.3 Water Conservation

Water conservation must address the community and can be achieved whether in household water use, household metering system, and in revisiting the water price.

The conservation of water at the household level can be achieved by establishment of appropriate mechanism to change peoples attitudes and reorient their behaviour with respect to water use such as: a) public awareness to reduce on water demand at household through media, lectures at schools and universities, b) involving the citizen in IWRM plan, c) water conservation to be part of school activities, and d) use of water saving appliance.

Water conservation could also be attained by the use of water metering system. Municipality water in Lebanon is not metered. Most cities do not have water meters and a country wide levy is applied. The levy is a flat rate tariff that is independent of the level of water use and provides no incentive for conservation. Therefore, the adoption of water metering system as a substitute for the existing system would decrease the wastage of water within household, since the consumer pays according to his consumption. Furthermore, the pricing policy in Lebanon does not take into account water amount usage, nor it covers operation and maintenance costs. Thus, water pricing should be adjusted to be based on usage amounts and maintenance. Implantation of new water pricing policies should be used as an incentive for improving water use efficiency.

5. CONCLUSION

The results of the study reflect the deteriorative water quality profile of the coastal aquifers of Saida and Tripploi cities and on the aggravated and high deteriorative profile of Beirut city aquifer resulting from seawater intrusion. Consequently, these results emphasize the necessity for

development of an integrated water resources management plan to ensure sustainability and viability of coastal groundwater. As well as, the study emphasize the importance for water resource assessment to sustain the provision of safe adequate water to meet demand of domestic sector.

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SIMULATION OF A KARSTIC GROUNDWATER SYSTEM USING ARTIFICIAL NEURAL NETWORKS (ANNs) - THE EXAMPLE OF EDWARDS AQUIFER

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Abstract: In the recent past, Artificial Neural Networks (ANNs) have found application in many hydrological problems. In groundwater management ANNs are usually used to predict the hydraulic head at a well location. The reason behind using ANNs is that numerical groundwater modeling may be very difficult to implement in karstic regions as the exact geometry of the flow conduits is rarely available and even when it is available in detail the aquifer is still very difficult to be simulated. A black box approach is used by ANNs to simulate the hydraulic head, taking as input hydrological parameters such as rainfall and temperature, as well as hydrogeological parameters such as pumping rates from nearby wells. Measured data from Edward's aquifer in Texas, USA are used in this work to train and evaluate an ANN. The Edwards Aquifer is a unique groundwater system and one of the most prolific artesian aquifers in the world. The present work focuses on simulation of hydraulic head change at an observation well in the area. The adopted ANN is a classic fully connected multilayer perceptron, with two hidden layers. All input parameters are directly or indirectly connected to the aquatic equilibrium and the ANN is treated as a sophisticated analogue to empirical models of the past. A correlation analysis of the measured data is used to determine the time lag between the current day and the day used for input of the measured rainfall levels. After the calibration process the testing data were used in order to check the ability of the ANN to interpolate or extrapolate in other regions, not used in the training procedure. The results show that there is a need for exact knowledge of pumping from each well in karstic aquifers as it is difficult to simulate the sudden drops and rises, which in this case can be more than 6 ft (approx. 2m). That aside, the ANN is still a useful tool to simulate karstic aquifers that are difficult to be simulated by numerical groundwater models.

Key words: artificial neural networks, hydraulic head simulation, karstic aquifer

1. INTRODUCTION

The objective of this study is to examine whether the simulation of a karstic aquifer hydraulic head can be performed by the sole use of a model based on the neural network theory. The karstic aquifers are quite difficult to simulate with numerical methods due to the fact that there is an extremely high degree of spatial diversity of hydraulic characteristics, such as hydraulic conductivity, specific storage etc. that prevent the researcher of constructing a trustworthy model analogue to the physical model.

With a completely different approach than that of numerical models, where the aquifer is the simulation object, the use of a neural network model is focused on simulating the hydrologic cycle. This approach has the advantage of applicability in any type of aquifer. The only requirement is a variety of data (meteorological, groundwater levels) in long periods of time that are not always available.

1.1 Karstic Aquifers

The occurrence of karstic phenomena is always connected with the existence of certain types of bedrock, which have a high solubility in water and leave hardly any residual so that the gaps are enlarged with the dilution procedure and permit the water flow through. To those bedrock, which are known as karstic (Boegli, 1980) belong carbonate rocks such as limestone and dolomite. Karstic aquifers respond differently than porous because water can travel through the fractures of the rock with high velocities. In most cases the governing equations are these of flow in open channel rather than through porous medium. Given the complexity of the fracture network which makes it practically impossible to describe exactly and simulate an aquifer in large scale, scientists try to find ways to bypass this issue using conceptual models that simplify the problem such as equivalent porous medium (Scanlon et al., 2003). Another method that has been applied lately is the use of an artificial neural network to simulate water table fluctuation at observation well locations.

1.2 Neural Networks in Hydrology

In the recent past, artificial neural networks (ANN) have found application in many hydrological problems. In groundwater management ANNs are usually used to predict the hydraulic head at a well location (Coppola et al., 2005; Daliakopoulos et al., 2005; Lallahem et al., 2005; Nayak et al., 2006). The reason behind using ANNs is that numerical groundwater modeling can be very difficult to implement in karstic regions as the exact geometry of the flow conduits is rarely available and even when it is available in detail the aquifer is still very difficult to be simulated.

The neural networks were initially formed as an oversimplified parallel to ‘biological’ models. The fact is that in both cases there are processing units (neurons), which communicate by sending signals to each other over a large number of weighted connections. In ANN each unit performs the relatively simple job of receiving inputs from neighbors or external sources and use of this to compute an output signal which is propagated to other units.

A black box approach is used by ANNs to simulate the hydraulic head, taking as input hydrological parameters such as rainfall and temperature, as well as hydrogeological parameters such as pumping rates from nearby wells. Available field data are used to train the network and evaluate the training process. The fact with neural network implementation in karstic aquifers is that the model does not need to know all the complex parameters that affect the output as it has an excellent ability to describe relations between parameters.

The reason why explicit knowledge of the complex hydrogeological parameters is not necessary is simple. The factors of the aquatic equilibrium can be divided into two categories: those that change over time such as rainfall, temperature etc, and the constant ones such as hydraulic conductivity. If during training of an ANN all time-changing parameters are given, the constants will be incorporated in the weights that will be computed throughout the training procedure.

2 AREA OF STUDY

Available measured data from Edward’s aquifer in Texas, USA are used in this work to train and evaluate the proposed ANN (Nikolos *et al.*, 2008). The San Antonio Segment of the Balcones Fault Zone Edwards Aquifer in south central Texas is one of the largest karst aquifer systems in the United States. The aquifer covers an area approximately 180 miles long and 5 to 40 miles wide and is the primary water source for much of this area, including the City of San Antonio, America's 7th largest city. The city has a semi-arid climate, and this makes water from the Edwards aquifer very important. Besides providing drinking water for cities in the region, the Edwards Aquifer is the principal source of water for agriculture and industry and provides springflow required for endangered species habitat, as well as recreational purposes and downstream uses.

Approximately 1,250 square miles of Edwards Limestone is exposed at the ground surface and composes the recharge zone of the aquifer. Streams flow south from the drainage area (the Texas Hill Country) and lose all or most of their baseflow as they cross the recharge zone. In addition, part of the rain that falls directly on the recharge zone also enters the aquifer. Groundwater moves through the aquifer and ultimately discharges from a number of locations. The residence time of water in the aquifer ranges from a few hours or days to many years, depending on depth of circulation, location, and other aquifer parameters.

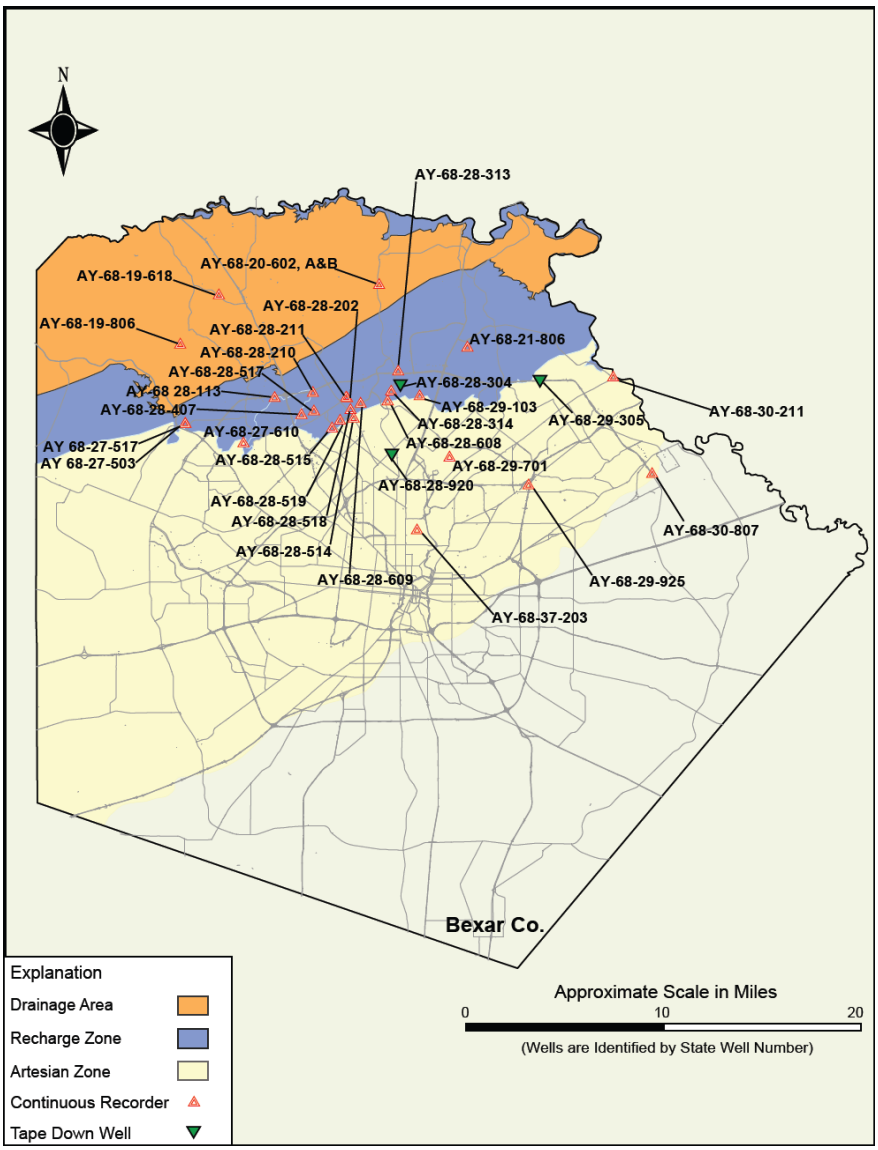


Figure 1. Map of Bexar County showing location of monitoring well J-17 (AY-68-37-203) (Schindel et al., 2007).

The Edwards Aquifer is a karst aquifer, characterized by the presence of sinkholes, sinking streams, caves, large springs, and a well-integrated subsurface drainage system. It is one of the most productive groundwater systems in the United States, characterized by extremely high capacity water wells and high spring discharges. The aquifer exhibits extremely high (cavernous) porosity and permeability, characteristic of many karst aquifers. In contrast, aquifers that occur in sand and gravel or in other rock types, such as sandstone, have a much lower permeability. Because the Edwards Aquifer is known for having areas of high permeability, it allows the transmission of large volumes of water, consequently enabling groundwater levels to respond quickly to rainfall (recharge) events (Schindel et al., 2007).

3. METHODOLOGY

The equation that is actually simulated by a neural network in most hydrological applications is the water budget. The present work focuses on simulating the hydraulic head change at an observation well, rather than the hydraulic head itself. The adopted ANN is a classic fully connected multilayer perceptron, trained in a supervised manner with the error back-propagation algorithm. Two hidden layers are used, thus the local features of the fitting function are extracted in the first hidden layer and the global features are extracted in the second hidden layer. The activation function is the commonly used logistic function, while the synaptic weights are determined in the training procedure through successive weight adaptations.

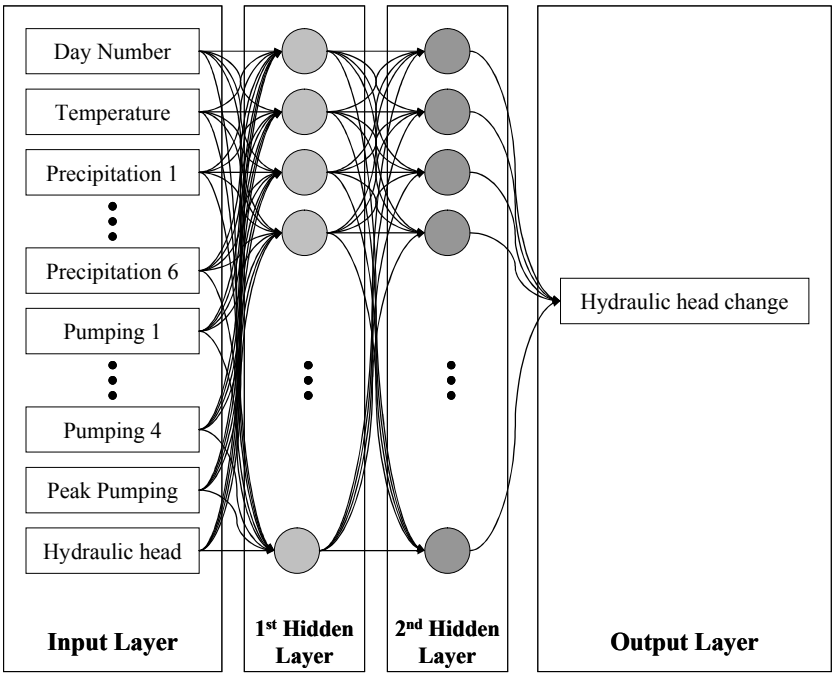


Figure 2. Architecture of a fully connected artificial neural network with fourteen input nodes, forty nodes in each of the two hidden layers and a single output node.

All input parameters are directly or indirectly connected to the aquatic equilibrium and the ANN is treated as a sophisticated analogue to empirical models of the past. More specifically, among the input parameters the precipitation and the pumping are parts of the water budget while day number and temperature are connected to evapotranspiration. The hydraulic head is associated with the inflow and outflow of the aquifer. A question arose when rainfall and pumping were concerned of whether the impact to hydraulic head change was instant or it delayed by a time lag. A major concern was to quantify the time lag of input parameters in order to retain a physical meaning. A correlation analysis of the measured data was used to determine the input parameters.

The output parameter (the hydraulic head change) is also highly connected to another aquatic equilibrium factor i.e. the change in the aquifer's storage. In similar works in the past, the output parameters have always been hydraulic heads. However the use of hydraulic head change as an output parameter provides the ability of a more physical representation of the system. Additionally, it enhances the accuracy of the prediction as the change to the hydraulic head has a much smaller value and the corresponding errors are expected to be of a smaller order of magnitude. This is supported by numerical experiments.

After the correlation analysis the final neural network consisted of 14 input nodes, namely: day number, temperature, rainfall of 6 days, cumulative pumping for 4 days, peak pumping and hydraulic head of the previous day. Each hidden layer consisted of 40 nodes and the output layer has the hydraulic head change at the observation well as the single output parameter.

Using back-propagation the neural network modified the connecting weights in order to minimize the training error. During the training phase 80% of the available data were used to train the model, and the rest of the data were kept for further testing. Moreover, 10% of the training data are used for evaluation purposes, and the evaluation error is computed in order to avoid overtraining. The training and evaluation errors of the neural network are calculated as the summed squared error between desired and calculated output for the training and evaluation data sets respectively:

$$E = \sum_{p} \frac{1}{2} \sum_{o=1}^{N_o} \left(d_o^p - y_o^p \right)^2 , \quad (1)$$

where p refers to the input pattern (known input that yield known output), N_o is the number of output nodes, d_o^p the desired output of node o for input pattern p, and y_o^p is the calculated output (Kroese and van der Smagt, 1996). The testing of the network, using the remaining 20% of the available data not used for training, provides useful information about the ability of the network to generalize its results and demonstrates whether the network can expand knowledge of the system behaviour to unknown areas (interpolate and extrapolate). During the testing, the model is provided with the input data set and produces the corresponding set of output data. These simulated values were subsequently compared to the observed ones and the relevant error was computed. In order to evaluate the calibration process and quantify the magnitude of the error, the bias, mean absolute error and root mean square error were computed and conclusions were deduced.

4. RESULTS

4.1 Correlation analysis

The correlation coefficients of rainfall or pumping with hydraulic head change for different values of time lag shown in Table 1 suggest that there is high correlation for more than one day between rainfall or pumping and hydraulic head change. This has a physical meaning as the effect of a rainfall incident or pumping might be apparent in the water table fluctuation for more than one day. By choosing a threshold equal to 0.05 for the correlation coefficient, 6 days of rainfall and 4 days of hydraulic head are qualified as proper input parameters. In case that the correlation coefficient was improperly selected and the chosen days are more than the appropriate ones the network should be able to detect this during training and eliminate the effect of the unnecessary parameters by giving small values to the corresponding weights.

Table 1. Correlation coefficients of rainfall and pumping with hydraulic head change for different values of time lag

Time lag	Rainfall – Hydraulic head change	Pumping – Hydraulic head change
0	0.3225	-0.2870
1	0.4059	-0.2515
2	0.2451	-0.1278
3	0.1577	-0.0725
4	0.1000	-0.0409
5	0.0793	-0.0176
6	0.0489	-0.0131
7	0.0390	-0.0172
8	0.0268	-0.0071
9	0.0184	0.0123
10	0.0168	0.0171

4.2 Calibration and testing

The neural network was trained for 1000 epochs and needed less than two hours on a computer with a 3.00 GHz Pentium D processor. The calibration procedure produced the weights as well as the training and evaluation error of 1.8E-04 and 2.0E-04 respectively. The values do not suggest overtraining and show that a fair training has been achieved.

After training and evaluation of the neural network the produced weights were used to obtain the simulated values of the hydraulic head change for the whole data set (Figure 3). A detail of Figure 3 is shown in Figure 4. The general trend of the observed values is followed by the simulated ones. However, the neural network has failed to describe some extreme points, which can be attributed to pumping near the observation well. It was a known issue of the initial data set that the pumping value was cumulative for all wells in the region rather than separate for each well. This is expected to import a certain error to the simulation.

Table 2. Frequency and probability of error to be in a certain range

Bin range	Frequency [d]	Probability
-2	1	0.01%
-1	48	0.68%
0	3008	42.31%
1	3730	52.47%
2	255	3.59%
3	44	0.62%
4	11	0.15%
5	7	0.10%
6	5	0.07%
Total	**7109**	**100%**

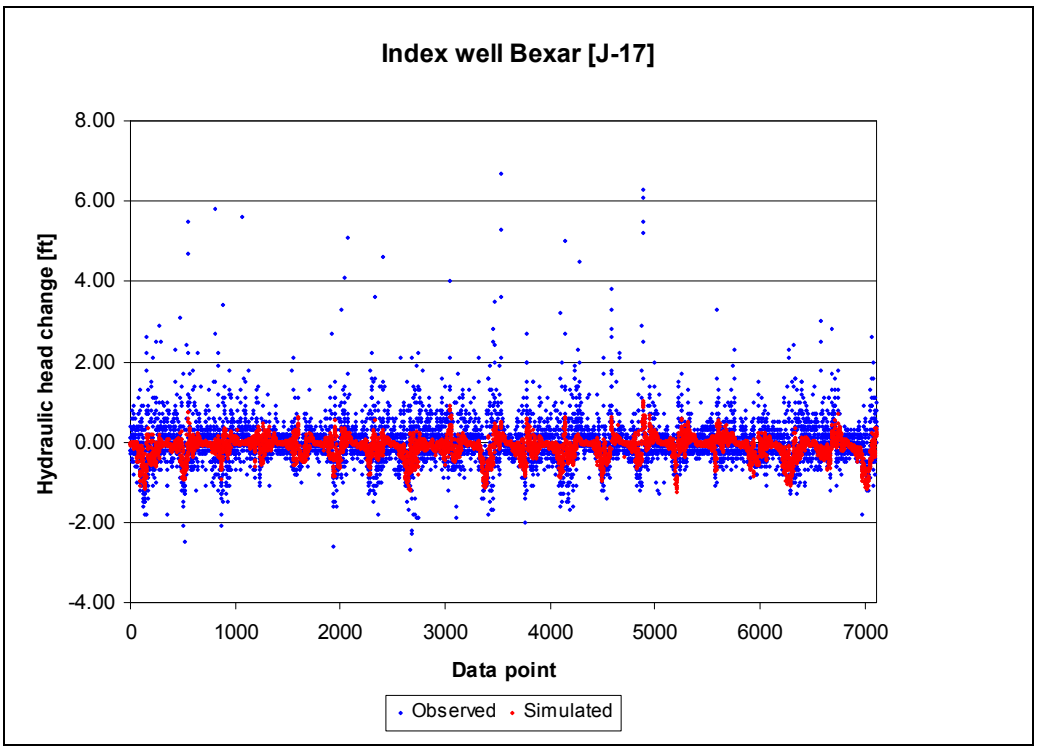


Figure 3. Observed and simulated values of hydraulic head change at monitoring well J-17 (AY-68-37-203).

Because of the large amount of data and the difficulty to get a clear picture of the calibration's value just from Figures 3 and 4, a statistical processing of the yielded error was considered necessary in order to check the reliability of the simulation. The error of the simulation was divided in zones of 1 ft and the frequency of days with error belonging to the corresponding zones was

computed and then divided by the total number of days. The corresponding results, (Table 2) demonstrate that the error for about 95% of the days spans between 0 and 1 ft (0 and 30 cm). This error can be considered as acceptable, given the high fluctuation of the aquifer's hydraulic head.

From the error yielded and the subsequent calculations of its statistical characteristics, the neural network showed an overall bias (average error) of 0.12 ft (3.7 cm), a mean absolute error of 0.33 ft (10 cm) and a root mean square error of 0.53 ft (16 cm). The bias suggests that the simulated values tend to underestimate the observed values and it can be explained by the extreme values which are not simulated by the ANN model. The error is not negligible when it comes to water resources management but in groundwater modeling terms it is not considered unreasonable. The extreme values that are not simulated by the ANN model are attributed to the fact that the pumping activities take place very close to observation locations. Considering the large amount of available data, the extreme values (in Figures 3 and 4) are a small portion of the available data, which is supported by the results of the statistical analysis.

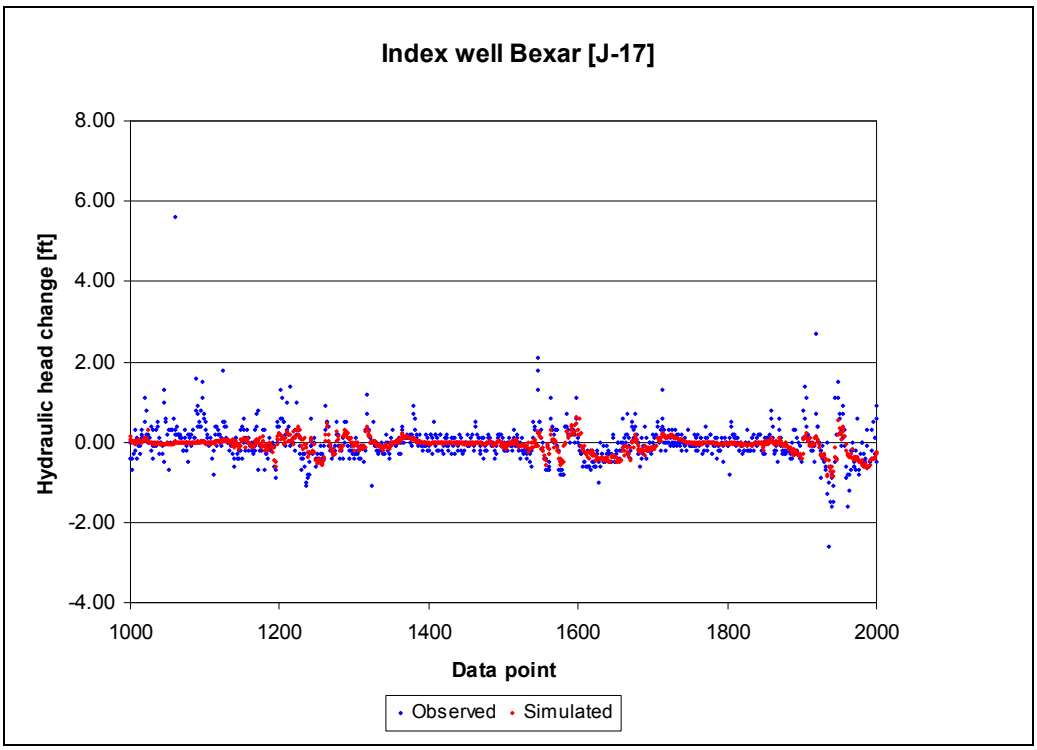


Figure 4. Detail of observed and simulated values of hydraulic head change at monitoring well J-17 (AY-68-37-203).

5. CONCLUSIONS

Artificial neural networks are considered as a promising tool for karstic groundwater simulation when detailed time series of hydrological and meteorological data are available. Unlike numerical groundwater models, which require thorough knowledge of the geological characteristics of the rocks and the geometrical characteristics of fractures in karstic aquifers, neural networks use a black box approach that considers only changing parts of the aquatic equilibrium and incorporates the stable parts in the networks weights through a training process. However, a large number of diverse data is needed in order to achieve a successful training and any lack of a parameter that has an effect in the water budget introduces an error in the model, which cannot be eliminated otherwise.

When trying to decide the input parameters of the neural network a correlation coefficient analysis of the parameters can provide hints of which parameters are important and should be included. In some cases when the effect of a certain parameter is obvious for more than one day, the

input nodes may include multiple days of a certain parameter. If the training is successful it should nullify the weights of any unnecessary input nodes.

In many cases the most difficult parameter to find is pumping from each one of the corresponding wells in the region. The exact knowledge of this parameter however is critical in order to train the network to effectively describe resulting abrupt changes of hydraulic head, very significant in karstic aquifers. The cumulative pumping of all the wells in a region can be used to describe the trend but not the sudden fluctuations of the water table. This lack of data is thought to be the main reason that introduced the error in data points with high values of hydraulic head change.

The resulting error between observed and simulated hydraulic head change is in 95% of the days between 0 and 1 ft (0 and 30 cm). This is not negligible when water management is concerned, but it can be considered acceptable for groundwater simulation.

When an artificial neural network is going to be used for water resources management data from each well in the region are needed as well as rainfall and temperature data from as many stations in the region are available, while when surface runoff occurs it should also be an input variable. All data must be available with the same frequency (daily, weekly etc.).

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A BURIED SALT BODY AS A SOURCE FOR GROUNDWATER SALINATION, KINNAROT BASIN, JORDAN RIFT VALLEY, ISRAEL

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Abstract: The Kinnarot Basin located south of the Sea of Galilee bordering Jordan to the east and Israel to the west. The basin comprises a link in the chain of pull-apart basins that characterize the long dislocation line known as the Dead Sea-Jordan Rift Valley. The basin floor is about -200 m below the msl and the shoulders attain an altitude of a few hundreds meters above. Both margins are built of Cretaceous to Neogene sediments interbedded with mostly Pliocene volcanic flows. The sedimentary fill of the basin, down to the Cretaceous rocks attains a thickness of 5-6 Km whereas 4.2 Km have been drilled by a deep oil well Zemah 1. The drill-hole penetrated outstanding thick succession of salt and igneous rocks. The fault system associated with the rift margins brings fresh water aquifers on both sides to juxtapose salt layers in the rift itself. There is a severe concern to loose huge quantities of water as a result of salination. An early fresh water excavation on the shoulders might prevent the deterioration.

Key words: Jordan Rift Valley, Kinnarot Basin, Salination, Salt body, Sea of Galilee

1. GEOGRAPHICAL, GEOLOGICAL AND HYDROLOGICAL BACKGROUND

The Kinnarot Basin located south of the Sea of Galilee, along the Dead Sea-Jordan Rift Valley (JRV) also referred to as "The Dead Sea Transform" (Fig. 1) comprises a link in the chain of pull-apart basins that characterize the structure of the conspicuous JRV. At the southeastern margins of the valley, the Ajlun Mountains in Jordan are reaching a 1300 m height (msl) while at the northeastern margin; the Golan Heights reach about 600 m high above the valley floor. To the west, the basaltic plateau of the eastern Galilee Mountains is rising up to 300 m above the Valley floor (Fig. 1).

The basin is a deep depression bounded by two NS-trending major fault systems with a dominant normal component of tilted blocks (Schulman 1959; Picard 1965). The eastern margin of the Basin consists of Cretaceous and Eocene carbonates and Plio-Pleistocene volcanic rocks; the western margin consists mostly of Neogene sedimentary and volcanic rocks (figure 1). The depression is filled with post-rift sediments consisting mostly of Neogene to Holocene marls, clays and conglomerates as well as bedded salt, salt bodies and igneous rocks. The source of the salts as well as of the other marine layers deposited around and within the basin is related to marine transgressions, sedimentation and evaporation.

Horowitz (Horowitz 2001) discusses the regional evolution of the JRV and the reader is referred to the voluminous publication list there.

In the center of the Kinnarot basin, Zemah-1 Borehole penetrated to a total depth of 4249-m of Miocene to Upper Pleistocene sequence consisting of rather unique thick succession of evaporates and igneous rocks. Studies of the cuttings, cores and electric logs extracted from Zemah-1 Borehole describe the complex environment of the deep basin in general and the borehole vicinity in detail (Marcus and Slager 1985; Shaliv 1991). The generalized stratigraphy is shown at Table 1. The stratigraphy of the Plio-Pleistocene sequence, topping the cover basalt formation, was mapped by seismic interpretation (Zurieli 2002). Deeper seismic interpretation of the current study has shown

that the salt penetrated by Zemah-1 Borehole is a representative of a large salt body extending throughout the Kinnarot Basin.

The most important aquifer in the study area is the limestone and dolomite Cenomanian-Turonian aquifer (Ajlun Group in Jordan and Judea Group in Israel). The Eocene limestone and the Neogene conglomerate are rather minor aquifers in the study area.

In the east side, in Jordan, an investigation of the Cretaceous aquifer layers in the Ajlun dome shows the inclination towards the Kinnarot valley (El-Naser 1991, Fig. 2). El Nasr (ibid) calculated that about 40 million cubic meter of fresh water per year is flowing out of the Ajlun aquifer system towards the JRV. Furthermore, according to his findings the flow regime into the JRV occurs at a depth greater then 500 m below surface.

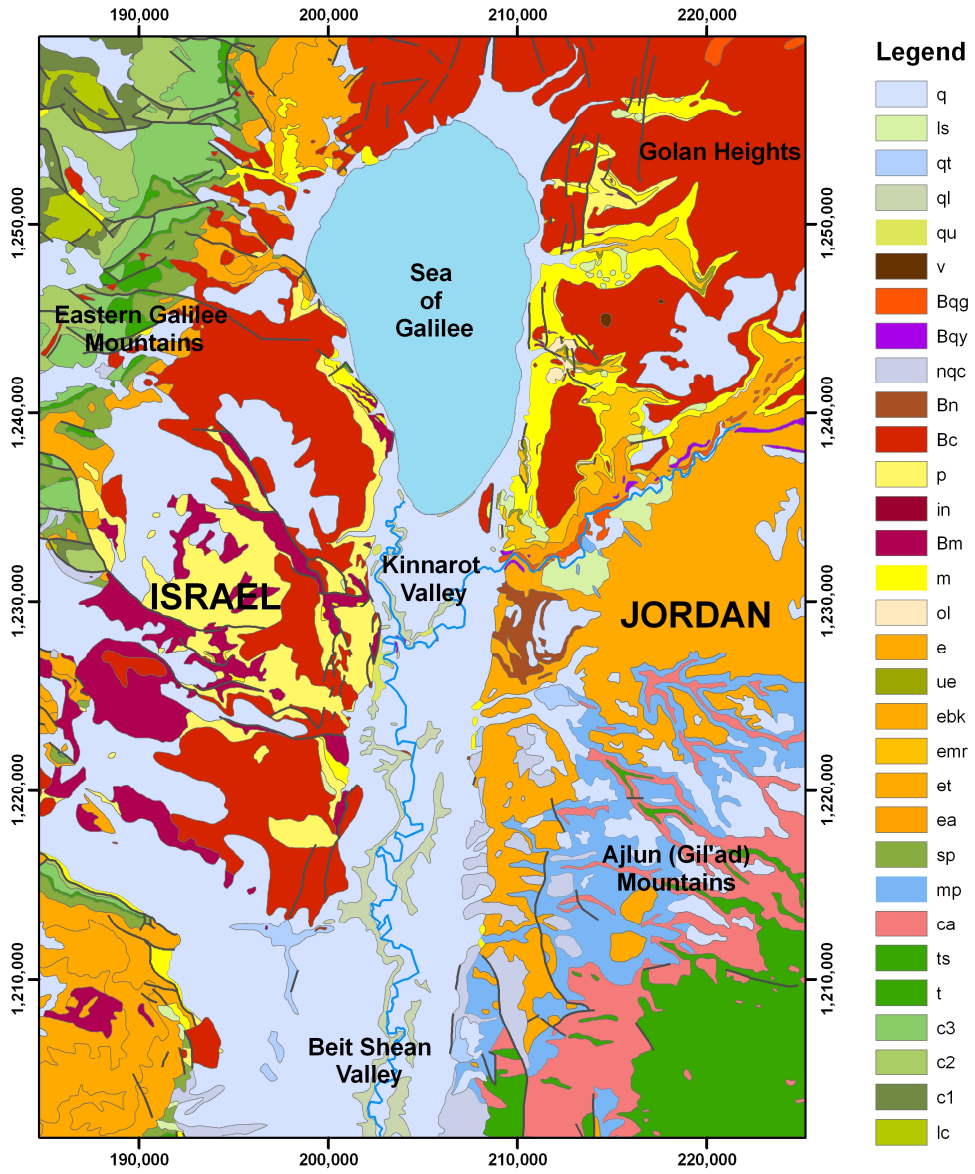


Figure 1: Location map of the study area. Geology after Sneh, et al. 1998. (q, ls, qt, ql, qu) various Quaternary Fms.; (βqy) Yarmouk Basalt, Quaternary; (nqc) undivided conglomerate units, Neogene-Quaternary; (βn) undivided Neogene volcanics; (βc) Cover and Intermediate basalts, Neogene-Quaternary); (p) Bira and Gesher Fms., Pliocene; (in) intrusive and pyroclastic rocks, Miocene: ; (βm) Lower Basalt and Late Miocene Volcanics, Miocene; (m) Hazeva and Dana Fms., Miocene; (e, eav, ebk, et, ea) various Eocene Fms.; (cts) undivided Cenomanian-Turonian-Senonian (Jordan); (sp, mp, mz, ma, ca, sc) Mount Scopus Group, Senonian-Paleocene; (ts) undivided Turonian-Santnian (Jordan); (t) Turonian Fms. (Israel); (c3, c2, cl) Cenomanian Fms.;

In the west side, in Israel, the deep-seated geology of Eastern Galilee Mountains is not yet fully understood. Nevertheless, the known regional geology implies that the hydrological relationship is similar to that of the eastern side. The Cretaceous and Eocene rocks are in hydraulic connection with the rift sediments; the potential juxtaposition to the salt endangers their quality.

The outstanding clastic nature of the basin-fill (Marcus and Slager 1985) as well as the local fault system increases the probability of the hydraulic connection between Cenomanian-Turonian fresh water aquifer and the salt body, thus endangering their quality.

Generally, the salination mechanism suggested for the study area implies fresh groundwater flowing from the margins dissolving partially the buried salt body. It is assumed that the brines produced by subsurface dissolution of the salt body become a key factor in salination process of the adjacent aquifers. A possible connection between brine forming by solid salt dissolution and the salination of the Sea of Galilee was suggested by Flexer, et al. 2000.

Table1: A simplified stratigrpic column of ZEMAH-1 borehole

Lithology	Thickness (meters)	Formations	Age
Marl, clay and conglomerates	486	Lisan, Naharaim, Ubadiya and Erk El Ahmar Formations	Plio-Pleistocene
Basalt	698	Cover Basalt Formation	Pliocene
Mainly salt with some lacustrine sediments and magmatic intrusions	1411	Bira and Gesher Formations with Intrusive rocks	
Mostly magmatic intrusions with some lacustrine sediments and salt beds	1547		
Conglomerates and marl	107	Hordos Formation	Miocene

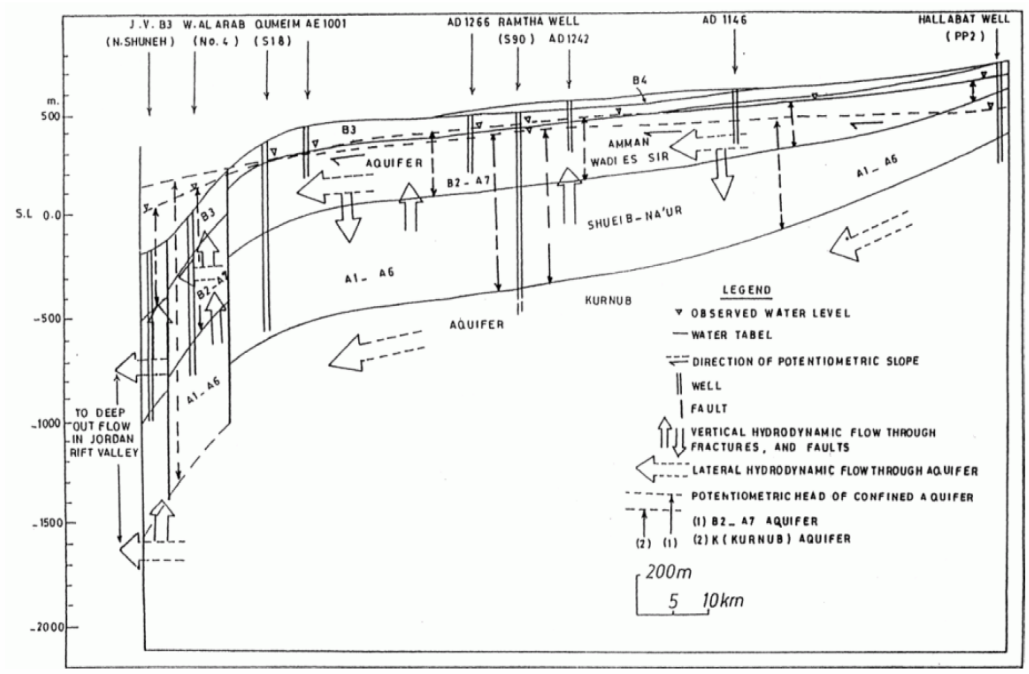


Figure 2: Hydrodynamic system of the northern part of Jordan (El-Naser 1991)

2. THE SALT BODY

Table 1 presents the stratigraphy and the outstanding thickness of the evaporitic and magmatic sequence penetrated in Zemah-1 borehole. The structure, the dimensions and the geographic extension of the salt body have been estimated and evaluated by 7 deep seismic lines (Fig. 3).

Table 1 demonstrates that the thick evaporitic and magmatic sequence penetrated in the well is divided into two members. The upper member consists mainly of salt, other thin sediments and a tongue of intrusive rocks which is according to the seismic interpretation, related to an individual sill (Fig. 4a). The lower member is mostly intrusive but holds within it some remains of the original Bira Formation represented by thin appearances of marls and evaporates.

According to the seismic interpretation Bira Formation sediments overlies unconformably a relief of early to middle Miocene rocks. The extant of the salt body is spreading through the entire basin, its thickness is rather constant and it has an uplifted structure at the middle and eastern side of the basin. The change in thickness of the salt body is attributed mainly to this relief and the vertical uplift is attributed to younger intrusive rocks penetrated as deep sills.

The salt body is covered by 500 meters thick sequence of the cover basalt formation. In one of the seismic lines (DS-3029 Fig. 4b) it seems that the salt body, under the pressure of intrusive rocks, has ruptured the thick basaltic layer and it is exposed to the young, Pleistocene rift sediments.

A time to depth conversion has located the intra-rift formations in their vertical position. A delineation of this interpretation in a geological profile (Fig. 5) shows the contact areas between inner- and outer-rift formations.

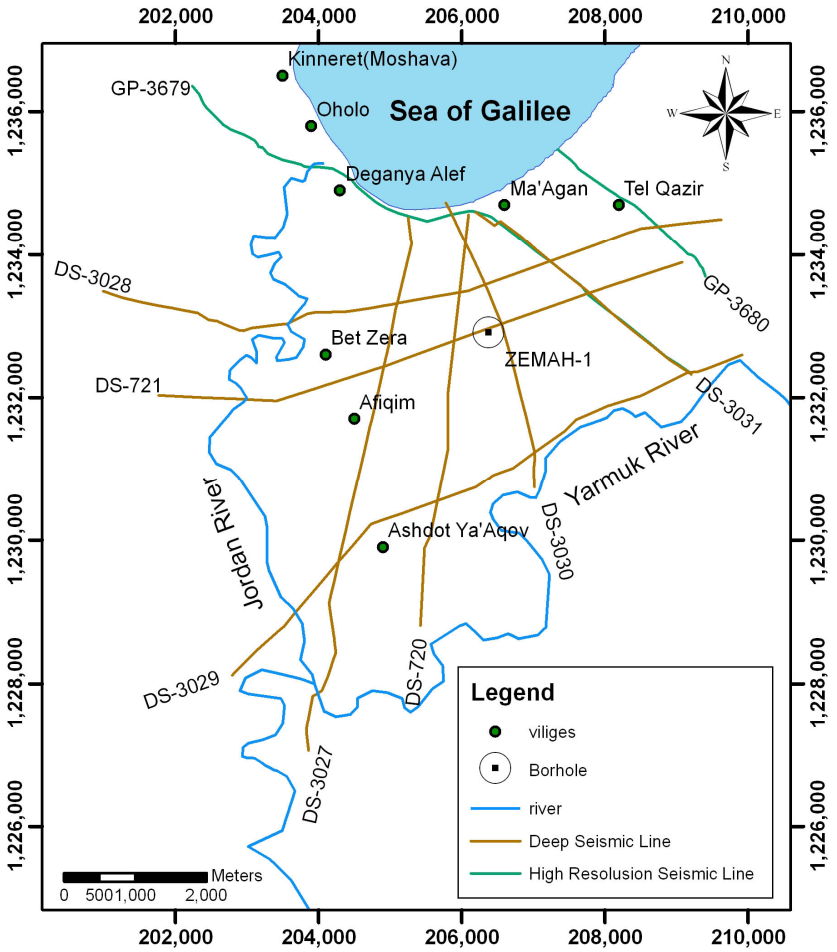


Figure 3: Seismic lines location map at the Kinnarot Basin.

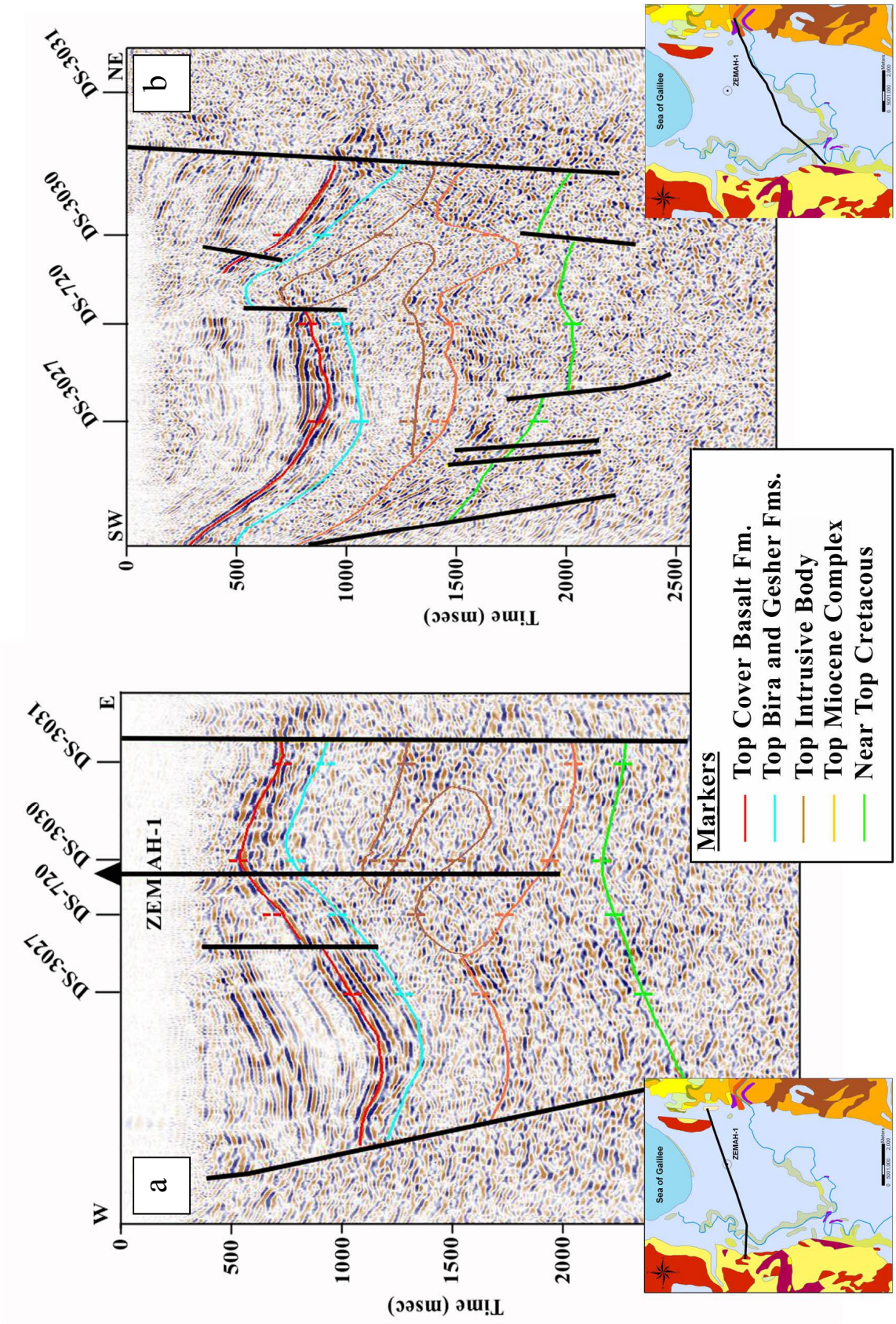


Figure 4: seismic profiles DS-721 (a) and DS-3029 (b)

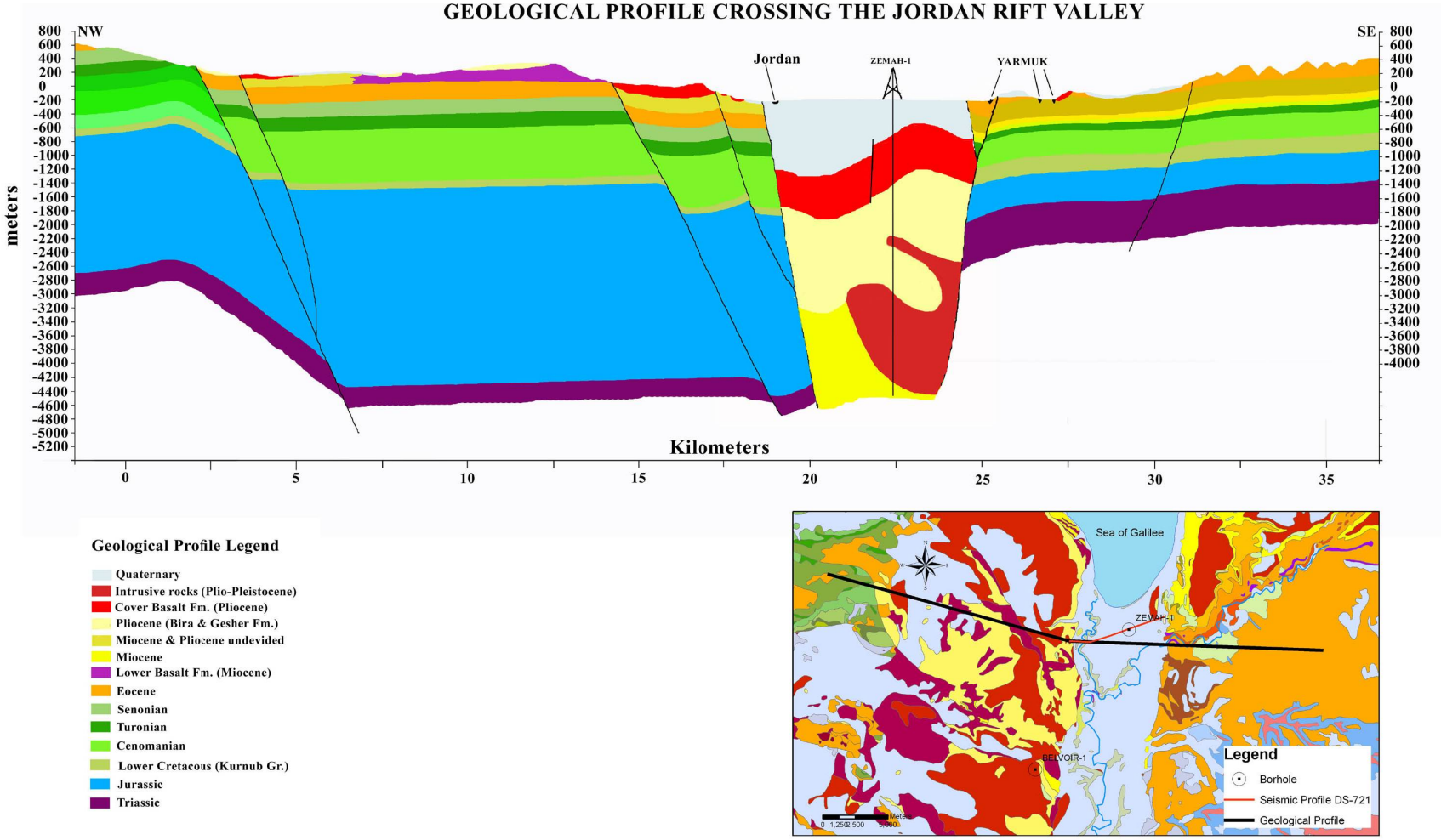


Figure 5: Geologigal Profile crossing the Jordan Rift Valley at the Kinnarot Basin. The geology of the Kinnarot basin is a projection of seismic line DS-721 after interpretation and manual time to depth conversion

3. HYDROGEOLOGICAL RELATIONSHIPS BETWEEN THE BASIN FILL AND SURROUNDING AQUIFERS

It is visible that the main aquifers from both the eastern and western sides of the rift, should physically encounter solid salt (Fig. 5). As was mentioned earlier, the hydrogeological model of Ajlun aquifer system (El-Naser 1991) has shown that large amount of fresh water is flowing to the rift from the aquifers of the Ajlun area in Jordan (Fig. 2), the calculations shows that about 40 MCM is leaving the Ajlun aquifer system and flowing to the rift every year in the area of the Kinnarot and Bet Shean basins. The structure of the Ajlun aquifer system is dipping north-westward and therefore it is estimated that large part of the mentioned amount is reaching the Kinnarot basin. A lack of similar information at the western side of the rift makes it impossible to determine amounts regarding those aquifers; nevertheless, the geological setting is clear. The salination mechanism suggested for the Kinnarot basin is of fresh groundwater flowing in the subsurface from the highlands in the east (Jordan) and perhaps also from the west (Israel). It is suggested that dissolution of the salts is by the fresh groundwater flowing into the rift and reaching the buried salt body. The brine formed in the rift is subjected to a stress regime that forces flow as described in the Zemah-1 Completion Report (Marcus, et al. 1984). The hydrogeological relationship between the basin and its margins indicates a flow potential from the margins to the basin and therefore the flow in the basin can be only in south or north direction. Flexer et al. (ibid) suggested that the brine flows on the major fault planes towards the Sea of Galilee.

4. CONCLUSIONS

The detailed combined geophysical and geological mapping of the study area indicates that the huge amount of evaporates buried in the deep basin-fill contribute considerably to the salination processes of the adjacent aquifers. Moreover, it is quite obvious that there are heavy losses of fresh water flowing to the valley and deteriorates. The water authorities in Jordan and Israel should consider fresh water mining on the valley margins before it reaches the salt body and salination occurs.

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THE PLACE OF THE INFORMAL WATER MARKET IN THE URBAN LANDSCAPE: SMALL-SCALE WATER PROVIDERS IN KISUMU, KENYA

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Abstract: Small-scale water providers (SSWPs) are reluctantly accepted as a viable alternative for delivery service to the citizenry. As a result, SSWPs are often viewed as "temporary" measure just taking advantage of failed public services so as to fleece their powerless customers. A survey was carried out in Kisumu to help understand the potential role played by these SSWPs in filling the gap left by the sole public utility. Factors that have seen the success and constraints faced by the SSWPs were examined. Content analysis was reinforced by personal observations from the field survey. Incidentally, SSWPs in Kisumu were most prevalent in the informal settlements and so the survey concentrated here. SSWPs dominate the informal water market. 42% of the residents, mainly residing in the informal settlements access water through alternative self supply. SSWPs are prevalent in areas with low coverage levels, in the affluent areas where the sole official water provider, KIWASCO, water supply is not sufficient and unplanned informal settlements that are difficult to access. Several factors affected the source of water including usage of the water, urgency, time of day, affordability and quality. 84% of the SSWPs encountered were single-purpose, single entrepreneur entities, with just 16% being community driven and/or NGO lead enterprises. Because of this, their establishment came into being as a result of a variety of reasons including, meeting consumer demand, responding to a crisis, as part of a larger business venture or for profits. Even so, less than 40% of the SSWPs reported having any meaningful communication with KIWASCO. In the midst of all this, cartels have sprung up and are killing the spirit of competition. But on the reverse, "book" enforcement of rules and standards by KIWASCO is not working effectively as it should. This paper re-emphasises the importance SSWPs play in water provision. It retaliates the position that they should be part of the solution to the challenges of water provision. They deserve recognition and support. They are adaptable to local conditions, convenient, have flexible payment system, offer employment and are income generating enterprises to those with meagre earnings. The study identified that the individualistic approach adapted by SSWPs is limiting their potential. It proposes franchising as a way forward.

Key words: Franchise; Kiosk; Kisumu; Small-scale water providers; Vendor.

1. INTRODUCTION

The provision of urban water services is evoking great emotional feelings among world governments amid many challenges reflected in the severe problems in its management to citizen in many parts, especially the developing countries. The evolution of the market economy has led to increase of uncontrolled urbanization as poor masses move into urban centres. Kisumu City Council (KCC) stopped building housing long time ago for its residents. Thus they no longer help residents find a better home within its area of jurisdiction, leaving immigrants to take matters in their own hands. The result is a spontaneous growth of fringe informal settlements which become crowded, swarming with life and remain largely un-served by public utilities posing a major challenge to the local municipal government, especially with respect water supply. Access to clean drinking water for many of the residents of these informal settlements, still remains a pipe dream.

The Water & Sanitation Program (WSP) estimated that 1 in 4 of Kenya's urban residents depend on Small-scale Water Providers (SSWPs) (Colligan & Vezina, 2000). Many of them form intermediary channels –unofficial 'pipes'- between the utility and the household. The growth of this market is a result of the unfortunate situations, majorly man-made. In Kisumu, water problems can

be traced to the segregation colonial planning policy. Through the enclave policy, water services were provided to the affluent only. This policy, unfortunately, has not been corrected and is perpetuated today through non-recognition of the informal settlements, which house 60% of the inhabitants of Kisumu (UN-HABITAT/UNEP, 2005). In addition, the deficiency in water services in Kisumu has been exacerbated by a number of other factors. Of significant is the increase in population without the responding increase in the provision of the services and high incidences of poverty. Rural-urban migration coupled with a high birth rate, estimated at 2.8% annually, have strained service delivery for the council, given that the council already suffers from strained resources, both human and financial (UN-HABITAT, 2004). Without any recourse, therefore, residents in some neighbourhoods have resulted to use of poor quality water that brings with it a lot of suffering. For instance, the incidence of the cholera outbreak reported by the Kisumu East Ministry of Health (MoH) office on 12th July 2007, in Manyatta settlement (Kenya Red Cross Society, 2007).

The burden of water provision weighs heavily upon the residents and it can be clearly read on the faces of the inhabitants, especially those living in the informal settlements. The council has failed to take advantages of the merits of urban areas (e.g. economies of scale and agglomeration (the city has a density of 828 people per sq. km (UN-HABITAT, 2004)), particularly for piped systems, while their potential disadvantages (e.g. the greater risk of infectious disease outbreaks when the water and sanitation systems fail) pose great risks.

As a result, SSWPs have increased in Kisumu. They hawk their wares (water) using many different modes including bicycles commonly referred to as boda boda and hand-operated pull carts. These hawkers have no legal status, pay no taxes or local government rates, but are nevertheless “accepted” as they supplement the water deficit within the city (SANA, 2000).

This paper focuses on these SSWPs that continue to contribute to the extension of water supply services at their own initiative and with resources mobilized largely from private sources. SSWPs often bear regulatory, economic and political risk, and in most urban settings where other factors come into play like the thorny issue of land tenure may also present commercial risk.

2. METHODOLOGY

A survey was carried out where both qualitative and quantitative methods of data collection was used. 256 questionnaires were administered to both users and providers. The study was able to quantify various issues that constitute challenges for the successful provision of water by SSWPs. A user-focussed methodology was adopted with the use of multiple methods such as semi- and unstructured interviews, and participant observation. The qualitative research methodology focused on the perceptions and experiences, obtained through elucidation of respondents’ perspectives, process and situational details. In addition, qualitative examination of texts, and various language-based techniques like conversation and discourse analysis was carried out with the aim of getting a generalisation for the study. The pluralistic approach of combining both qualitative and quantitative methods enhanced triangulation and verifies positions taken by the informants.

3. THE INFORMAL WATER MARKET

The main source of piped water in Kisumu (about 92%) is Lake Victoria, the world's second largest freshwater resource. The water distribution in the city is estimated at 112km of distribution mains. Pipes laid before 1970 account for 81% of the total length (JICA, 1998). In terms of water production, Kisumu is estimated to produce 18,200m3/d against a projected demand of 50,000m3/d (UN-HABITAT, 2004). The other, about 8% of the water comes from Kajulu treatment plant on river Kibos. Currently, according to the UN-HABITAT (2004) 40% of the population of Kisumu have access to some of this piped water supplied by Kisumu Water and Sewerage Company

(KIWASCO), the sole water utility in the city (since 2003) – keeping in mind illegal connections, unmetered water losses, leakages etc.

The huge deficit of about 60% is amply filled by the SSWPs. The informal water market thriving in the informal settlements forms a major component of the fabric of the livelihood of the residents of these settlements. These SSWPs are aggressive and have responded effectively to the user demands in Kisumu. Residents have been thrown back to their own resources to finance and deliver water services if they want to escape from the poverty trap that so much engrosses them. Two reasons emerged from the field survey that has continued to propel the existence of SSWPs: (i) the SSWPs use appropriate technologies following their understanding of the local environments. This ensures that they are able to supply water with minimum expenditure and (ii) as long as poverty is still in our midst, there will always be gaps in service provision, hence the need for SSWPs. The study further revealed that SSWPs were prevalent in areas with (i) low coverage levels, (where the gap between served and under-served or un-served customers exists; (ii) in the affluent areas where KIWASCO water supply is not sufficient; (iii) unplanned informal settlements that are difficult to access, especially during the rains. These settlements included the peri-urban settlements of Kisumu namely Nyalenda, Nyamasaria, Obunga, Bandari, and Manyatta. Generally, there are two categories of water users in the informal settlements.

(i) Owner/landlords: This is a static group whose data can easily be tracked and documented.

(ii) Tenants: A very versatile group, with no permanent residence. They keep in moving, making tracking of the numbers difficult.

These two categories form a web of networks of how they receive their water.The survey was able to identify several conduits of water supply (figure 1). For those at the edge of the network system they are most likely to get their water from water kiosk run by KIWASCO or sub-delegated to private operators (92%). As one moves away from the networks, vendors and shallow wells take over. Further on, water is drawn from nearby streams, drains and shallows wells. These sources are used by 42% of the residents of Kisumu, mainly those residing in the informal settlements. Unfortunately, 82% of these sources are unprotected and are a source of illness common in the area.

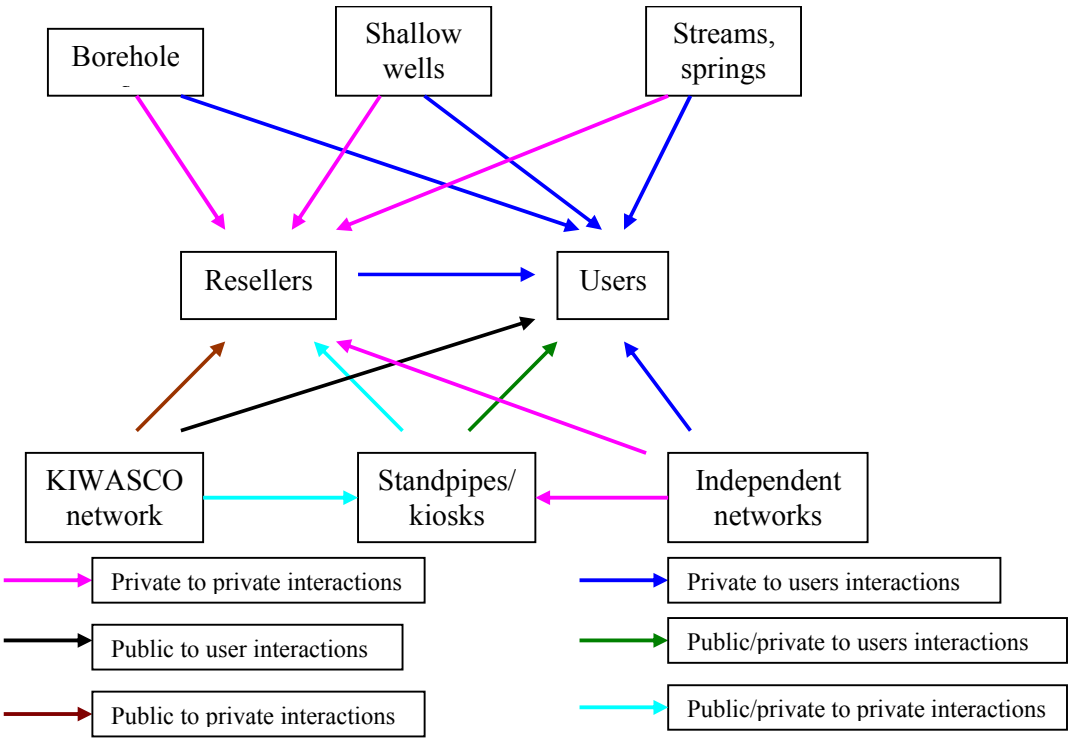


Figure 1: Chain of water supply in the informal settlements.

Several factors affect the source of water:

(i) Usage of the water: The survey revealed that 87% of the informal settlements dwellers have at least two sources of water for different purposes. Water for drinking, which is considered to be most important, is fetched from sources they have "faith" in while water for bathing, washing etc is fetched from sources with "least faith" (plate 1).



Plate 1: Residents washing clothes in a drain. Source: Adapted from "Cities Without Slums", UN-HABITAT, 2005.

(ii) Urgency and time of day: Long queues at the water points especially those run by KIWASCO and the danger in the night, make many dwellers (73%) result to water being delivered to them as an alternative.

(iii) Affordability: The existence of the business people has pushed prices very high especially during the dry seasons. Another danger slowly creeping in is the cartel-business mentality by a few operators. Users are left with no choices. KIWASCO is toothless with no "water" for these people and no regulations to govern the operators. The users are at the mercy of the operators. However, the existence of some NGOs, FBOs and community initiatives offers some relief on prices.

(iv) Quality: The frequent outbreaks of water-related diseases in the informal settlements is a testimony of the poor water quality being supplied. Water scarcity during the drier months of June – September raise water prices fifteen times from Ksh. 2 (US$ 0.03) to as high as Ksh. 30 (US$ 0.43) per 20-litre jerry can. This precipitates many residents in the informal settlements, who work in the informal employments, to resort to using unsafe alternatives. Interestingly, 86% of the residents rate (through visual inspection, taste and smell) the water they use as being acceptable. This is in sharp contrast with the frequent cholera incidences reported in the area.

The study further revealed SSWPs are ripe and very active in Kisumu, even though they don't have an official status. 92% are non-water professionals or specialists. They are unregulated and untaxed, and hence are able to offer affordable payment systems (cumulative daily or per item) to their customers. 72% of the consumers regard them as being reliable and convenient. They are adaptable to harsh environments, changing occasionally to keep pace with competition from rivals. Among the SSWPs are:

(i) Vendors/reseller: Vending of water has become a lucrative business in Kisumu's water stressed informal settlements. Water is transported in 20-litre jerry cans carried on pull carts (plate 3), transported on bicycles or carried on the head. Given the level of high un-employment in the informal settlements, many able-bodied young men have self employed themselves in the business. The distanced covered determine the cost of the water. The study revealed that the vendors command the highest price by volume. These types of water delivery modes posses the most quality risks as the water exchanges many hands and is out of the water pipe for long periods of time and possible distance before reaching the intended consumers. In essence it offers the least quality standards but this is what is offered to over 60% of the residents. However, vendors provided the most tailor-made service as they have multiple choices of water sources and can deliver to any place

requested by the customer. This group is preferred by people who have little time to look for water or have other tasks to attend to.

(ii) Shallow wells owners: Individual initiatives - mainly by landlords, community groups, Faith-Based Organisations (FBOs), and Non-Governmental Organisations (NGOs) - have resulted in hand-dug shallow wells springing up in some areas. Kisumu area has a high water table and is prone to frequent flooding during the rainy season. Due to lack of regulation, monitoring and supervision, wells are sometimes dug dangerously close to the house (plate 2) near pit latrines raising the suspicion of the water quality supplied to the consumers (water from these sources is never taken for laboratory tests). In the event of heavy rains, run-off gets into some wells that are not well constructed and covered, adding to contamination.



Plate 2: Manually operated shallow well near a house.

(iii) Water kiosks owners: The unconnected households near the networks get their water supply from public or sub-delegated private run standpipes otherwise referred to as water kiosks. Due to the hours of operation of KIWASCO run kiosks, 7am to 6pm, 91% of the residents complain of long queues, heckling and quarrelling at watering points, and/or poor service by the council.

The informal private sector in Kisumu is unregulated, virtually by definition, but they are thriving. The low levels of capital investment required to dig shallow wells – as the water table is relatively high, many are manually operated, and the business acumen of the entrepreneurs enticed by profits - has made water provision such a lucrative job that cartels have sprung up to “protect their business”.

84% of the SSWPs are private. No rules of engagement between the provider and the user exist. Every engagement is done in a user-demand market only. A few NGOs, FBOs and community initiatives exist though in some places especially in institutions like schools churches, etc. According to Mukami and Schwartz (2005), SSWPs have traditionally played three basic roles: "gap filler" in countries with high coverage levels but low service quality (measured by the number of days or hours within which services are available); "pioneer"--developing and operating systems in areas where there is no public service but there is customer demand; and "sub- concessionaire"--buying water in bulk from the utility and selling it on to customers. In Kisumu, SSWPs are increasingly assuming the additional function of "manager" of small service schemes that are providing water to small groups of houses.

Despite this obvious functions being played by the SSWPs, little support is coming from the local authorities. The council views them as just a temporary or stopgap measures. Since water is considered a public good, of importance to public health, the council considers it important to continue to provide the service in order to avert a catastrophy related to an outbreak of water-related diseases. Thus, they fear to be associated with these providers who they consider illegal.



Plate 3: Manual pull-cart water reseller filling Jerry-cans from a water kiosk.

3. IMPORTANCE OF THE SSWP

SSWPs play a significant role in serving the residents of Kisumu. They are significant service providers, for approximately 60% of the population, mainly living in the informal settlements and may be the only viable operators for the foreseeable future. They have to be recognised for the important role in service provision, compensating for - or supplementing - the limited financial and human resources of the KIWASCO.

84% of the SSWPs encountered were single-purpose, single entrepreneur entities with just 16% being community driven and/or NGO-lead enterprises. Their establishment came into being as a result of a variety of reasons including: (i) meeting consumer demand (e.g. individuals asked to provide access to water supply for a fee by their neighbours or communities); (ii) responding to a crisis (e.g. neighbourhood associations formed to develop an alternative to failing public systems); (iii) as part of a larger business venture such as estate/housing developer or landlord or (iv) for profits on seeing an business opportunity. The SSWPs have given an entrepreneurial approach to their service delivery and have developed in the process specialized user - provider relationships with their customers.

The urban poor have received insufficient attention from KIWASCO and so the SSWPs operate in an environment where the judge is also the prosecutor. The survey revealed a situation where no regulation, monitoring or supervision exists. The public health concerns have been left to the private sector. The local authorities are reactive only in situations of disease outbreaks. Given their business outlook, the capacity of the private sector to deal with disease outbreaks is obviously questionable.

Recognizing and supporting SSWPs may be a practical way for KCC to reach acceptable service coverage objectives more quickly. SSWPs form an important decentralized option for providing water services for the poor masses in the informal settlements. With increased recognition and promotion of Public-Private Partnerships (PPP), SSWPs maybe the only way out in the efforts of KCC to provide water for its residents.

The concept that water provision is a public monopoly due to its public-health benefits at best has made the important work being done by SSWPs not to be recognized and poorly documented in Kisumu. A such, murky statistics exist on the number of operators, tariffs have a wide range and scanty regulatory measures have been attempted at all. The shame of the local government failure to provide for all its citizens here has made the authorities leave everything to fate.

Even if the SSWPs charge a higher price, there are important advantages for poor residents, even from a financial point of view. The main advantage is in being able to buy in small quantities that will vary from day to day depending on household cash flow. SSWPs may also choose to sell water on credit and run a tab for their customers. Additionally, buying water in this fashion carries no risk of the customer being charged for water that leaked due to a poor connection.

SSWPs provide water services to a large proportion of informal settlement urban households, and particularly those living in areas some distance from the conventional water distribution networks. Without them, many of the un-served would be even worse off. For the sub-delegated water standpipes, operations have extended working hours, clearly indicating the flexibility and understanding of the private sector of the conditions of the residents. This is an indication that it maybe more effective to work with private sector if services are to be improved and the achievement of the MDGs is to be attained.

4. CONSTRAINTS FACED BY SSWP

The SSWPs in Kisumu are continuing to provide services to the residents despite harsh conditions of operation and their unrecognized status. Less than 40% of the SSWPs reported having any meaningful communication with KIWASCO. Many are struggling in their efforts to perform the noble task of supplying water to the residents. However, the study also revealed that not all informal water vendor systems deserve support. In some cases, the profits to be made from reselling scarce water have led key providers to create non-competitive environments (cartels), and the water supplies are in effect restricted in order to drive up prices. On one hand, the oversight provided by KIWASCO is not very effective. These cartels in essence are killing the spirit of competition. But on the reverse, "book" enforcement of rules and standards by KIWASCO is not working effectively as it should. This has left KIWASCO in a "catch 22" situation. To un-weave from this situation, mechanism of working with informal settlement groups and with vendors to determine how best to make the market function more effectively in the interests of users maybe the way out. Simply trying to close down the SSWPs on the grounds that they do not meet some official standard is in danger of restricting water supplies and driving prices up even further.

Additionally, lack of affordable financing is a constraint for 78% SSWPs, who fund investments mainly through their own earnings and savings, loans from friends and family, and money borrowed from formal and informal lenders. These informal providers generally operate without a subsidy and under difficult circumstances yet they continue to provide some service to the poor irrespective of the prices. Interestingly, they continue to show the willingness to expand their services. 17% pointed out that they could improve the palatability of their water if the had money.

Corruption was also sighted as a constraint. It seems KIWASCO and KCC health officials perpetuate the idea that "poverty is powerlessness" through corrupt dealings, seeking kick-backs in order to allow private investors to operate. This practice is a discouragement to the good progress otherwise being made by the private investors. Unfortunately for the local residents, they have no recourse when they suspect that SSWPs are engaging in monopolistic behaviour, selling contaminated water, or engaging in otherwise dubious practices.

5. CONCLUSIONS

The study has emphasised the important role the SSWPs are playing in water supply to the informal settlements in Kisumu. However, their presence is no excuse for KIWASCO and KCC to neglecting the task of finding less costly alternatives for universal access to piped water by all residents. The urban poor in Kisumu have found means to have more influence on service delivery to the estates. They have increased their own capacity through creation of jobs that increase their incomes and better organisation through community groupings. However, little has been done towards directly influencing public authority's responsiveness towards service delivery. The appropriate choices are more likely to emerge where KCC is responsive to the concerns of informal settlements residents, and the residents themselves are able to articulate and negotiate for their interests – taking us back to the issue of increasing the power of the urban poor to demand better water. But, some of the SSWPs do not want order because water is a hefty business in these

informal settlements. This is the greatest challenge KIWASCO and KCC has to grope with in the meantime.

Individualistic approach adapted by the investors has limited their potential. Many fear coming together for lack of trust and attitudinal problem. However, franchising may be the future. A few franchisors, who have already establishing a good reputation, through trust, commitment, confidence and credibility with their customers and the community, for provision of good water services, could be the key to expansion plans to meet the MDGs. New entrants could be made to affiliate to the well established small-scale providers (franchisors) and make the new entrants follow the standards for good service delivery. This way the licensing fees and the bureaucracy could be avoided. One advantage of this is that competition is enhanced, standard services are provided, new entrants in the water business are encouraged to provide water, and local monitoring of quality of water provided is increased. This arrangement has to be done with a lot of caution. Clear rules have to set by the franchisors. The daily commissions to be paid on daily basis and the annual fees can then be ploughed back into the expansion of the water services. As this will be a private venture, care has to be taken for those who may want to withdraw as it can leave the customers suffering again.

However, the window of opportunities for improvement is wide open. SSWPs have merits which should be harnessed positively and an engagement arrangement established with the KCC to ensure water services reaching the informal settlements are enough and of the acceptable quality.

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APPLICATION OF SUSTAINABLE ENGINEERING FOR URBAN AREAS: EXAMINING EFFECTIVENESS OF WATER CONSUMPTION IN URBAN REGENERATION

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Abstract: As urbanization is being a global issue, the conventional engineering methods for infrastructures have shown their limits in the view of sustainability. To overcome these limits, studies as sustainable engineering is introduced to achieve sustainability in urban areas. Among the methodologies applied for sustainable engineering, this study considers that material flow analysis (MFA) will be a useful method. Participating in the national urban regeneration project in Korea, this study purposes to examine the effectiveness of water consumption for the green infrastructure system which is proposed by the urban regeneration project in urban areas. By comparing scenarios, about 80% of more water consumption is expected for urban regeneration and by applying the green infrastructure system about 3~4% of water consumption is expected. Also by examining the water effectiveness considering water consumption and the benefits that are provided, it is concluded that urban regeneration applying the green infrastructure system will largely contribute in achieving urban sustainability.

Key words: Material Flow Analysis, Urban Regeneration, Urban Sustainability, Water Infrastructures

1. INTRODUCTION

The urbanization rate of the world is increasing, and thus urban sustainability has been a worldwide issue. According to UN Department of Economic and Social Affairs Population Division (2008), it is shown that the growth rate of urban population in developing countries is annually 3%. It is also expected that more than half of the world population will live in urban areas in 2010, and almost 70% of the world population will live there in 2050 (Figure.1).

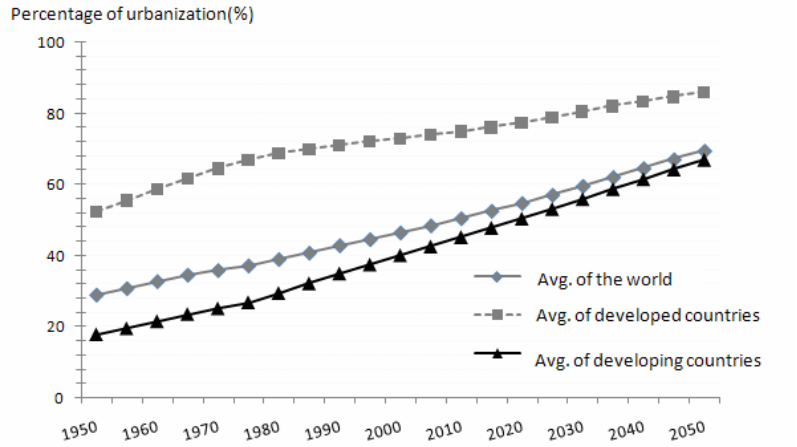


Figure 1. The urban population rate of the world, developed and developing countries

Rapid urbanization recently gives birth to social, economic and environmental problems, as follows (Suh 2007); e.g., slumism, heavy traffic, environmental pollution, and heat island problems. These problems hinder our urban area to achieve sustainability. Hence, we should overcome the problems with theoretical and technological studies. Among them, engineering works, i.e. design of

urban infrastructures, have been criticized by recent studies (Lee 2008; Park 2008; Wright 1996), as follows;

1. The conventional design of urban infrastructures does not fully consider the next generation for sustainability: According to WCED (1987), sustainable development is defined as, "development that meets the needs of the present without compromising the ability of future generations to meet their own needs". From this, it can be said that consideration of the next generation is need to achieve sustainability. Until now, infrastructures have been designed to meet urgent demands, to provide high performance in short terms, and also to minimize initial construction costs. Since urban infrastructures generally have a long life span, the cost for urban infrastructures should be considered in long terms. In long terms, studies show that the life cycle cost of infrastructures imposes a heavy burden to the next generation. Additionally, the infrastructures cannot provide full performance to the next generation as the surrounding conditions changes (Gleick 2003).

2. The conventional design of urban infrastructures cannot consider the essential principles for sustainability: As Park (2008) mentioned, three important principles are required to achieve sustainability; environmental conservation, allocate efficiency and social equity. To consider these principles in design of urban infrastructures, the related design objectives should be considered in the design process and the design of infrastructures should be optimized based on these design objectives. Though until know, the conventional design of infrastructures only focused on how to provide performance more cost-effectively for the high demands (Lee et al. 2008).

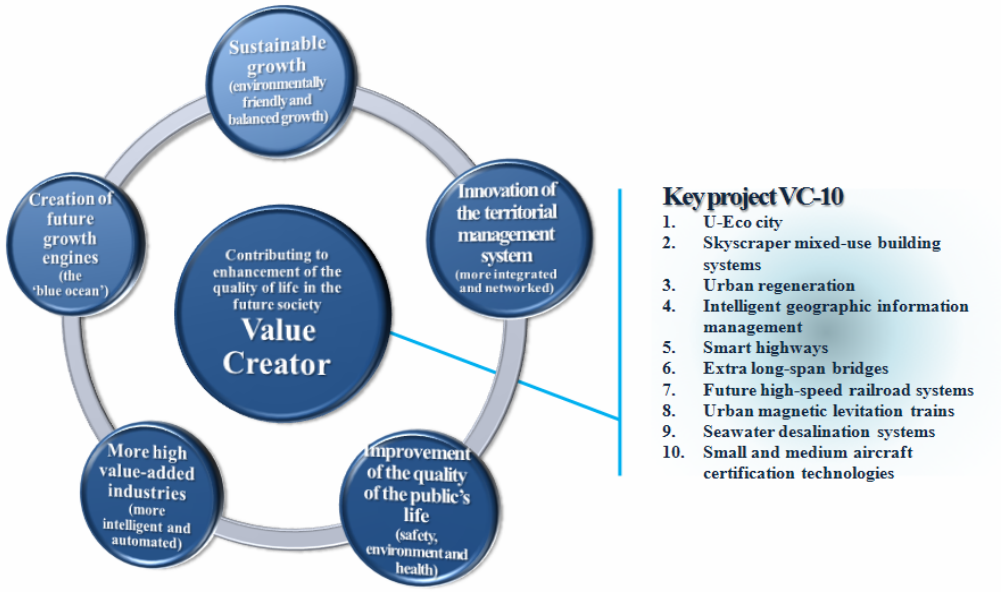
3. The conventional design of urban infrastructures cannot consider the complex values included in sustainability: The urban infrastructures consist of complex elements not only functional elements but also various elements as; social, economic, environmental and regulative elements which are related with urban activities. Lee (2008) says that to consider these complex elements and their complex relationship in design of urban infrastructures, they should be considered with comprehensive or systematic approach. Though until know, conventional design of urban infrastructures prefers analytical approaches and limits its boundary to systems which are easy to quantify. The weaknesses of conventional design of urban infrastructures are also observed in urban regeneration. The regeneration for sectors of Manchester and Amsterdam are the representative examples of urban regeneration based on the modern city theory. These cases succeeded to provide infrastructures for dwelling, water supply and electricity etc. efficiently to satisfy the rapid demands after World war Ⅱ. Though, social, environmental and economic problems in these areas after regeneration show that these cases have failed to achieve sustainability; e.g., Amsterdam had problems as 25% of the buildings being empty and for Manchester, problems as crime, poverty, unemployment, drugs occurred. As a result, both Manchester and Amsterdam are in new urban regeneration projects in reasons of these problems. Korea has also shown urban regeneration policies since the 1950's. Though, as a result of regeneration, the urban areas have been rather more complicated and social competition is derived. This is caused by focusing on developing dwelling sites and quantitative expansion during regeneration. These cases show that the urban regeneration projects failed to achieve urban sustainability and this is a worldwide challenge. In short, the conventional design of urban infrastructures has failed to achieve urban sustainability. This failure is thought to be related with the limits of engineering which has a role to design urban infrastructures to fit with the development aim of urban areas.

2. THE OBJECTIVE AND SCOPE

Recently in Korea, the Ministry of Land, Transport and Maritime affairs (MLTM) organized a project to regenerate the urban areas for urban sustainability in theoretical and practical methods.

Table 1. Technology innovation roadmap VC-10

● In 2006, the Ministry of land, transportation and maritime affairs selected 10 promising technologies, as"value creators (VC-10)", and organized a center to promote construction and transportation technology R&D projects.



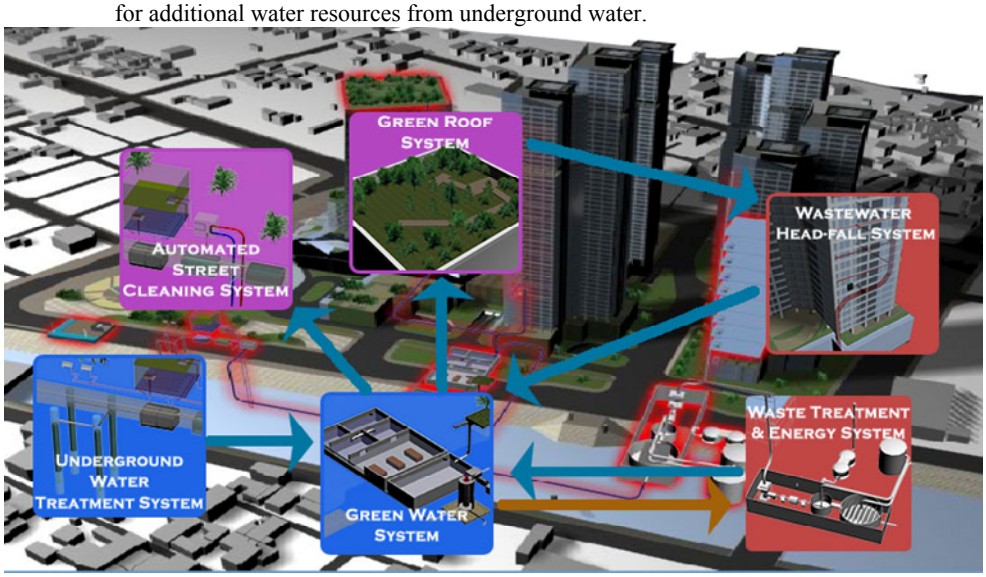
● Performance goals for the VC-10

- Achieve the level of construction and technology that may be ranked within the seven most advanced countries.
- Reduce the total social overhead capital spending on construction by 5 to10%.
- Reduce the distribution costs, traffic congestion costs, and traffic accident costs by 10%.
- Reduce the national disaster costs by 10%.
- Develop five global top construction and transportation technologies.
- Gain 10% of the international market by 2015.

As shown in Table 1, an innovative technical roadmap VC-10 has been developed at Dec, 2006 (KICTEP 2008). To support one of the value creators, urban regeneration, the Korea Urban Renaissance Center (KURC) was founded with a vision to "improve the existing urban environment and develop sustainable settlements for improving the life quality and the city's competitiveness". Involving in the urban regeneration project, this study focuses on the water consumption in urban areas which is an important content for urban sustainability. This study also examines the effectiveness of the so-called "green infrastructure system" that is proposed by the urban regeneration project (Table 2 shows the specific technologies that are proposed by the urban regeneration project which is so-called as the "green infrastructure system"). Especially, the physical boundary of this study is determined as the target area (9blocks) which is considered by the urban regeneration project (the basic data for this study is shown in Table 1). Though the urban regeneration project is in progress and the results of this project are not certain, this study is worth thinking about the design of infrastructures. It is because arguing about engineering for design of infrastructures from the view of urban sustainability excluding politics, safety-first policy and traditional practice is an essential process to achieve urban sustainability.

Table 2. Components of the green infrastructure system

○ **Summary of the green infrastructure system**
- Green water system: Integrated water treatment system treating household wastewater and rainfall for water reuse.
- Waste treatment & energy system: Energy generating system using household solid waste from wastewater.
- Green roof system: Green roof for reducing CO2 in urban areas, providing benefit for inhabitants and insuring additional water resources for buildings.
- Automated street cleaning system: Street cleaning system providing aesthetic view for residents and convenience by automation using reused water.
- Underground water treatment system: Underground water intake system connected with monitoring system for additional water resources from underground water.



3. METHODOLOGY: SUSTAINABLE ENGINEERING AND MATERIAL FLOW ANALYSIS

Recently, there have been studies to achieve urban sustainability in engineering areas called as 'sustainable engineering'. Choguill (1996) has shown a conceptual "10 steps" to improve the conventional design of infrastructure and achieve urban sustainability. Wright (1996) argued that there should be a fundamental change in philosophy of the individual engineers, regulatory policy and bureaucrats for urban sustainability. Walsh et al. (2006) reviewed the level of current resource use in urban areas in more developed and less developed countries and the level that would seem adequate for the urban growth in the future. Among studies, Park (2008) defined sustainable engineering as "offering a well-balanced design solution considering the three principles; environmental conservation, allocate efficiency and social equity for sustainability in design of infrastructures".

In this study, material flow analysis (MFA), is considered to be useful. The reason is, 1) other methodologies are relatively difficult to analyze the substantial water infrastructure system in a comprehensive scope because it becomes too complicated by the complex relationship embedded in the water infrastructure system. Though, material flow analysis simplifies the system to stocks and flows which makes it easier to manage the target material synthetically. 2) This methodology has a limit for analyzing in long terms because it analyzes the system in a static view rather than a dynamic view. However, because this study focuses on the water effectiveness in urban areas before and after urban regeneration, this study only requires 'snapshots' of the system before and after applying the proposed system. Thus, in this case urban metabolism (Kennedy et al. 2007), i.e., material flow analysis focusing on target materials in urban areas is considered to be a suitable method. Similarly, material flow analysis has been applied by such studies (Newcombe 1978; Center of Industrial Ecology of Yale university 2000). Newcombe (1978) applied material flow analysis to analyze the urban metabolism of Hong Kong and the Center of Industrial Ecology of Yale University (2000) analyzed the flow and stock of copper and zinc in national, continental and

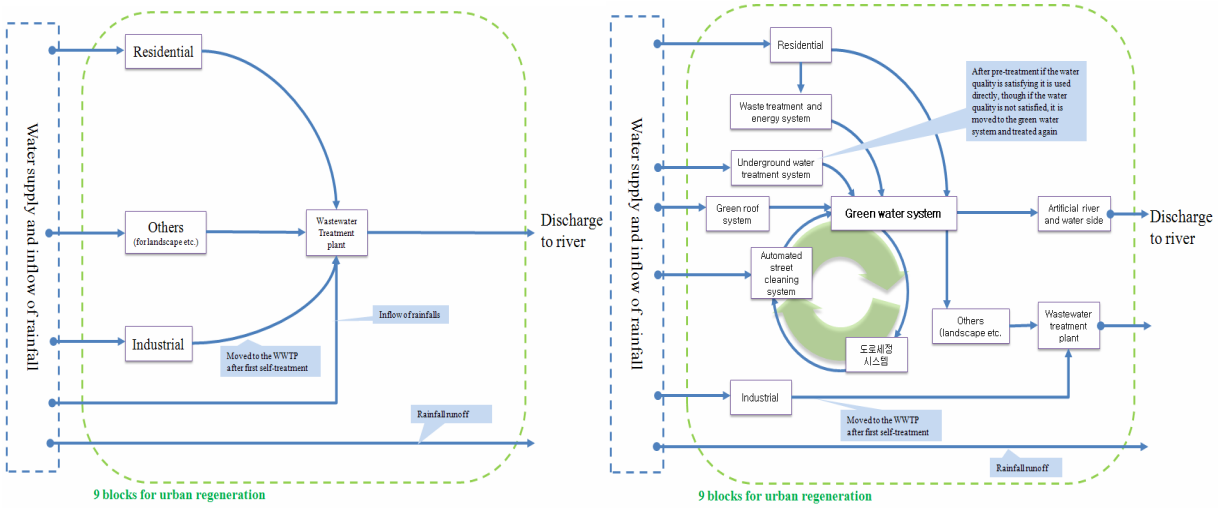
moreover global boundaries. Recently, this methodology is applied in areas such as environmental management, resource and waste management, industrial ecology to examine the efficiency of material usage (Lee et al 2006). From the characteristics which are shown, material flow analysis is considered to be a suitable methodology to this study as trying to examine the water effectiveness of the so-called "green infrastructure system" proposed by the urban regeneration project.

4. MODELING

This study analyzes the water flow in urban areas before and after applying the so-called "green infrastructure system" proposed by the urban regeneration project. The flow model established by this is shown at Fig. 3. The assumptions for the model are shown:

1. Characteristics of target area: this study assumed the target area as 9blocks which is same as the urban regeneration project (Park et al. 2008). This target area has social • economic • geometric • environment characteristics of the Daejeon city.
2. Estimation of demands: the demand of each processes are calculated by basic units and are assumed that the processes using household and industrial water have no changes before and after urban regeneration. Additionally, urban regeneration systems such as the automated street cleaning system and the artificial stream which provide benefits to the residents create additional demands. These demands are calculated based on the target amount proposed by the urban regeneration project.
3. Coequality of population: the population is expected to grow from 683 people to 4,500 people in the urban regeneration project. This meets the general expectation that as high benefits are provided, more population incoming is expected. In this study, the population is equaled to the population after regeneration (4,500 people) for comparison of cases with different population.
4. Inflow of precipitation: the rainfall runoff rate is assumed as 65% based on the data of Daejeon city and lose by inflow to groundwater, evaporation and transpiration is assumed as 5% based on standards of river design (MLTM 2000). That is, it is assumed that 30% of rainfall is flowed into the infrastructure system. (Meanwhile, because the rainfall data is assumed additional studies as scenario analysis are required to solve the uncertainty of data.)
5. Existing water reuse: in the present, there is a small amount of water reuse in the Daejeon city. Though, this amount is not considered in the model because the daily amount of reuse water is too small compared to the total daily water usage of Daejeon city. The daily amount of reuse water is 423m^3 which is very small compared to the total daily water usage of Daejeon, 479,118m^3.

Models (a) and (b) of Fig. 2 show the water flow of before and after applying the green infrastructure system. Comparing the models, when the "green infrastructure system" is not applied (Fig 2(a).), the water flow is composed of a one-way flow from inflow to discharge with no circulation neither water reuse nor additional water resource development. On the contrary, when the "green infrastructure system" is applied (Fig 2(b).), circulation of water, water reuse and additional water resource development from rainfall and groundwater is observed.



(a). Before regeneration *(b) After regeneration*

Figure 2. Models of water flows in urban areas before and after urban regeneration

5. RESULTS AND DISCUSSION

Three scenarios are established to see the water consumption of the so-called "green infrastructure system" included in the urban regeneration project which is described in table 2. Scenario 0 is the default scenario which is the case before urban regeneration for 9 blocks. Scenario 1 and 2 are the cases after urban regeneration, where scenario 1 is the scenario not applying the "green infrastructure system" and scenario 2 is applying the "green infrastructure system". As a result, water consumption required for each scenario is 2,429m^3 daily for scenario 0, and for 4,357m^3 and 2,512m^3 each for scenario 1 and scenario 2. From the results, it is shown that by urban regeneration, the additional water consumption is 80% compared to before regeneration (scenario 0 and scenario 1). Though, by applying the "green infrastructure system" with urban regeneration, the additional water consumption is only 3~4% (scenario 0 and scenario 2).

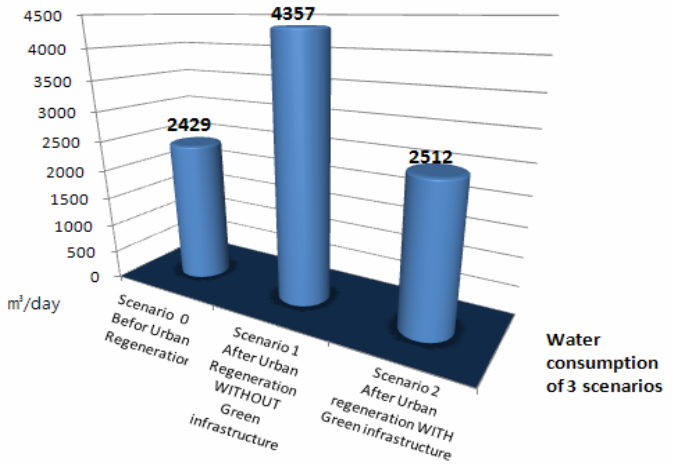


Figure 3. Analysis of water consumptio based on scenarios

Because more water is required by urban regeneration, it might be thought that the urban regeneration project is not reasonable in the view of urban sustainability. Though, this opinion is based on only considering the water consumption and not considering the benefits that are provided by the urban regeneration project. Even though there are limits to analyze the social, economic and environmental benefits quantitatively, the benefits provided by the urban regeneration project should be considered to examine the water consumption of urban areas in the view of urban sustainability. Various methods are available consider these benefits, among these methods this

study examined the effectiveness of water consumption by the magnitude of the benefit to unit water consumption as Eq. (1).

$$E_i = \frac{B_i}{Q_i} \quad (1)$$

E_i = effectiveness of water consumption of scenario *i*

B_i = total amount of benefits expected in scenario *i*

Q_i = expected water consumption of scenario *i*

As comparing scenario 0 and scenario 2, the water consumption for each scenario is Q_0 = 2,429m^3 and Q_2 = 2,512m^3 each. It is shown that the water consumption is similar with little difference (Q_0-Q_2=83m^3). For benefits, the urban regeneration project provides; convenience to the residents, eco-friendly place development and additional energy generation. By these, many social, economic and environmental benefits are created. So, it can be qualitatively said that the comparison of water consumption effectiveness for the two scenarios can be shown like Eq. (3). I.e., the effectiveness of water consumption for scenario 2 is larger than scenario 0.

$$Q_0 \cong Q_2, B_0 \ll B_2 \quad (2)$$

$$\text{So, } \frac{B_0}{Q_0} \ll \frac{B_2}{Q_2} \ (\ E_0 \ll E_2\ ) \quad (3)$$

For the other case, scenario 1 and scenario 2, they provide same benefits by urban regeneration ($Q_0 = Q_2$). Though, concerning water consumption for scenario 1 and scenario 2, there is a large difference between the two scenarios (Q_1 = 4,357 m^3, Q_2 = 2,512m^3, Q_1-Q_2=1845m^3). The effectiveness of water consumption between the two scenarios can be shown as Eq. (5). I.e., the effectiveness of water consumption for scenario 2 is larger than scenario 1.

$$Q_1 = 1.73\ Q_2, B_1 \cong B_2 \quad (4)$$

$$\text{So, } 1.73\frac{B_2}{Q_2} = \frac{B_2}{Q_2}\ (E_1 < E_2) \quad (5)$$

In conclusion, a large improvement is expected in the effectiveness of water consumption by applying the "green infrastructure system" compared with before urban regeneration. It also has been shown that applying the "green infrastructure system" is about 1.73 times more effective compared with situations after urban regeneration.

6. Conclusion and future studies

To summarize, this study has such meanings; first, compared with the conventional design of urban infrastructures, this study tried to consider the social, economic and environmental values in engineering works, i.e., design of infrastructures. Hence, this is considered to help in design and construction of infrastructures to contribute to urban sustainability not only considering its function or cost efficiency. Moreover, this study has meanings as being a practical study ongoing with a national project not being just a theoretical basic research. In other words, this study is expected to contribute to argue engineering about sustainability excluding politics, safety-first policy and traditional practice in large concerned projects. Nonetheless, this study has limits and subjects that need future studies; first, this study focuses on the effectiveness of water consumption for the "green infrastructure system" which is one of the performances among the various performances expected from the "green infrastructure system" proposed by the urban regeneration project. So, a feasibility study is required about the other performances and for this an integrated material flow analysis of energy, waste, atmosphere and water quality etc. should be established. Likewise, in this study the benefits are not determined quantitatively. There is a study in progress to quantify the qualitative benefits from another team included in the urban regeneration project. The results from this team will give an opportunity to determine the benefits of the "green infrastructure system" more accurately.

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WATER PRIVATIZATION IN GREECE: TURNING WATERS INTO PROFITS?

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Abstract: This paper offers a contemporary view on water privatization in Greece, focusing on country's majors urban centres, Athens and Thessaloniki. At first, it attempts a brief presentation and categorization of the changes related to water supply companies, worldwide. The basic arguments that support the expanding private sector involvement in the water supply industry are related to a new era of socio-economic development, especially among developed countries. For this era, water privatization's promotion is connected to a series of goals, such as financing new investments; implementing sufficient technical projects; promoting economic efficiency and higher profits; improving service quality and environmental conservation. All these goals are considered to be poorly-functioning in the frame of state-owned companies guided by 'supply-led' and 'equity of access' managerial goals. The paper highlights the expansion of water supply 'mercantilization' as well as the variety of privatization forms recently promoted. This analysis indicated that the implementation of concession contracts, in which assets are kept public while the rest network is managed according to 'market environmentalism' rules and axioms, is the most frequent form, at least among the advanced countries.

In the empirical part, the study presents relevant efforts in Greece during the past two decades. A primary assessment of the two major privatization projects, related to the water supply companies of the greater urban areas of Athens and Thessaloniki, is presented. The observed incremental profitability for the companies under study, reflects a successful case of transformation according to the rules of the market-environmentalist paradigm, though the state is a major regulator of the whole privatization process. This transformation is based on *geographical expansion*, coupled with *embedment/further promotion of commodification and commercialization* as well as *investment (utilized or forthcoming) in new activities*. Based on the Greek case, the paper claims that water privatization initiatives and the expansion of a demand-led model of water supply management should be both conceptualized in terms of a wider socio-political transformation towards the post-fordist era. In this frame, water supply systems transformation in an urban or peri-urban area is the outcome of the complex interplay of a series of global-to-local factors. The national economic, political and institutional settings are significant for this process, especially in the Greek case. .

Key words: Athens, Greece, post-fordist era, Thessaloniki, water privatization.

1. INTRODUCTION

In 1999, 5% of the world's population was served to some extent by semi or fully privatized water firms. By the end of 2007, this rate reached 11%, corresponding to a total number of 750 million people served (Pinsent Mansons, 2008). Although, more than 80% of water supply utilities are still publicly owned and operated, these upcoming trends demand a thorough analysis of water privatization issues. Apparently, this task requires the formation of a conceptual framework, integrating socio-economic, environmental and technical parameters able to shed light on water systems transformation in certain geographical areas.

This paper offers a contemporary view on water privatization in Greece, focusing on country's majors urban centres, Athens and Thessaloniki. At first, it attempts a brief presentation and categorization of the changes related to water supply companies, worldwide. The basic arguments that support the expanding private sector involvement in the water supply industry are related to a new era of socio-economic development, especially among developed countries. For this era, water privatization's promotion is connected to a series of goals and priorities, such as financing new investments; implementing sufficient technical projects; promoting economic efficiency and higher profits; improving service quality and environmental conservation. All goals outlined above are

considered to be poorly-functioning in the frame of state-owned companies, especially when these companies are guided by 'supply-led' and 'equity of access' managerial goals. The paper highlights the variety of privatization forms recently promoted, before presenting a primary assessment of the two major privatization projects, related to the water supply companies of the greater urban areas of Athens and Thessaloniki. Finally, the paper offers some tentative conclusions.

2. PRIVATIZATION, COMMERCIALIZATION, COMMODIFICATION AND THE POST-FORDIST ERA.

A conceptualization of recent transformations in the advanced and less developed countries, is apparently needed in order to interpretate changes related to the water sector. In fact, changing patterns of management and ownership in the above sector is not a new, 'post-fordist', phenomenon (Amin, 1994). Since the industrial revolution era, many countries have experienced a series of transformations in their water supply companies. Indeed, in parallel to the conditions of fully-state or fully-private water companies, many other cases of mixed schemes were put forward in urban or agricultural places around the globe (Baker, 2003b; Hassan, 1998).

However, water supply management and operation was carried out by state or publicly-owned companies for most of the 20th century. The promotion of state-led patterns of ownership, management and operation in the water companies was an efficient solution to a series of problems that came to front after the strong industrialization waves and the associated urbanization tendencies. One of the above emergent problems seeking answers, was the need to expand secure health conditions to the whole population in the urban agglomerations and the peri-urban areas. Providing basic social, welfare and health services for the working classes was a fundamental choice as well as a need for the fordist mode of regulation, positively related to the productive and accumulation strategies of this period. This is particularly true for the majority of developed (fordist) countries, at least until the mid-70s, and the 'state-hydraulic' model that these countries followed. The latter named after the 'great' hydraulic water supply infrastructures of the fordist era which were constructed according to the rule of supplying steadily increasing quantities of water for every firm, household and place needs them (Amin, 1994; Gandy, 1997; Swyngedouw, 2005) (*Table 1*).

Things were quite different for the developing world and its cities. Poor public investments and insufficient welfare policies were among the basic factors for the formation of relevant societies, thus leading to an important private sector involvement in water supply long before the 70s. Inequality in water access has been another major expression of the socially stratified character of the 'undeveloped' societies, as long as delivering water to the poor by jerrycan or tanker was historically combined with public supply systems for the middle/ upper population strata (Bakker, 2003b; Budds and McGranahan, 2003).

Trends towards increasing private sector participation have been triggered out since the late 70s and can be positively related to the 1973 economic, financial and political crisis. The latter crisis signals the passage to the already mentioned post-fordist era of development, at least among the advanced countries, which in turn has been followed by extensive reformations in the modes of regulation so far employed by G-7, OECD or EU members and other national authorities (Amin, 1994). Although explaining the why, where and who of this crisis is a controversial task, many discussants agree that the so-called 'western societies' are witnessing a neoliberal turn followed by more or less aggressive free-market policies and extensive flexibility (in productive, institutional, employment and geographical terms) (Bakker, 2003a; EUWI, 2004; Swyngedouw, 1997).

On a political level, this move is reflected upon many official reports and directives of international/ supranational organizations, such as the World Bank, the World Trade Organisation, and the International Monetary Fund. The neoliberal turn in the water sector of advanced countries mainly took the form of allowing and promoting private capital's investments in the former state-owned companies. This change reflected a wider move towards water industry's and water markets' 'opening up', in various geographical scales; the latter expressed in many ways, such as the

foundation of water banks or the expansion of bottled water sales (OECD, 2003; Water Remunicipalisation Tracker, 2008; World Bank, 2006). As far as developing countries are concerned, the major expression of the passage to the post-fordist era was signaled by further promoting private sector participation in existing companies. In other cases, private investments were directed to such countries through international initiatives seeking to resolve the striking problems of millions of people, lacking access to clean water for their everyday needs (Bakker, 2003b).

Nowadays, many different forms of private involvement in water services can be easily found around the world, with 'concession' contracts being by far the most popular ones; from the fully private provision (known as ‘divestiture’ and presupposing private asset ownership), to the mentioned concession contracts (where assets are kept public) and various forms of BOT (Build-Operate-Transfer) or Service/ Management/Lease contracts. Alongside these forms, ‘joint ventures’, ‘passive private’ or ‘passive public’ investments do exist, with different roles and responsibilities associated to the private sector in each case (OECD, 2003; Pinsent Mansons, 2008). Baker (2003:329) highlights that companies achieve economies of scale when dealing with concession contracts in urban areas. Thus, rural areas, underpopulated cities or less than half a million urban agglomerations, typically do not attract private companies' interest.

The basic concept that lies behind the promotion of privatization schemes in the water sector is that environmental goods degradation and inefficient use can be successfully and simultaneously faced, if these goods are allocated as fully-priced (including environmental external costs) economic goods through typical market-driven procedures. A variety of terms, such as 'market enviromentalism', ‘ecological modernization’ or ‘green capitalism’, are employed as best capturing the above notion. Introducing market mechanisms to environmental management, it is believed, is also the best way to achieve sustainable development goals and thus, retaining if not increasing a proper stock of resources for next generations wealth and development (Harvey, 1996). ‘Market-environmentalists’ can be described as these theorists and policy-makers that optimistically seek for the appropriate mixture of policies, technologies and practices allowing and enhancing new rounds of profitable and in parallel, environmentally-sensitive capital investment (Armstrong, 2006; Page and Kaika, 2003).

Table 1. *Privatization, commercialization, commodification and the passage to the post-fordist era for water supply companies among developed countries*

	Fordist era, state hydraulic model	**Post-fordist era, market-environmentalist model**
	Privatization	
Infrastructure owner	State	Private sector & state
Infrastructure manager	State	Private sector & state
	Commercialization	
Economic regulation	Command-and-control	Market-based instruments
Resource management	Growth-oriented	Scarcity-responsive
Supply-system management	Supply-led	Demand-led
Primary socio-economic goals	Equity of access (growth, quantity)	Efficiency (sustainability, quality)
Provision ethos	Service	Business
Primary decision makers	Public officials, experts, citizens	Company managers, experts, public officials, customers
Costumer participation & identity	Collective, citizen	Individualistic, customer
	Commodification	
Method of charging	Metered or proxy bills	Metered
Raw water pricing	Subsidized or free	Priced at full (incl. environmental) cost
Water supply pricing	Social equity (ability to pay)	Economic equity (benefit principle)
Allocation mechanism	Public policy	Competition
Water identity	Resource	Environment

Source: Based on Baker, 2005

However, privatization is an umbrella-term often used to describe quite different choices and procedures, thus leading to a theoretical confusion. Baker (2005) proposes a useful distinguishment between three interrelated, although distinct, processes that are connected to 'market-environmentalist' policy and practices, so far; ‘privatization’, ‘commercialization’ and

'commodification'. Precisely, privatization refers to the organizational type of change associated with the passage of ownership and/or management to the private sector. Commercialization refers to the institutional type of change related to the adoption of efficient management practices and profit-maximization goals. Finally, commodification is related to the process which turns a natural resource, such as water, to a commodity sold at prices regulated according to market mechanisms. The basic structural dimensions of these three processes and their changing character in relation to the move towards the post-fordist era, are codified in Table 1.

An analytical discussion of the above three processes goes beyond the scope of this paper. It is important to highlight that: i) all three processes are significant for the study of recent changes in water companies, especially among the economically developed countries as will be shown from the Greek case, and ii) they should be carefully distinguished from each other in any case. For example a water company can be private but not, at least fully, commercialized (e.g. not fully-pricing the water sold to some customers); or a public company can be a commercialized one which runs along the lines of full-cost pricing and profit-seeking practices. Still, commercialization is not unilaterally connected to commodification, even from a neoliberal point of view. In other words, turning water to a priced economic good is a necessary presupposition, that should be followed by commercialization and (if necessary) by privatization in order to achieve an ideal, efficiently functioning water company (Bakker, 2005; Harvey, 1996).

3. WATER PRIVATIZATION IN GREECE: RECENT TRENDS

EYDAP is the major water supplier in Greece, providing water services to the capital city of Athens as well as to many other municipalities and regions located mainly in the Attica region. The latter is the most heavily populated region in Greece, concentrating more than 4 million people and resembling a lot to other Mediterranean areas, at least as far as hydro-meteorological conditions are concerned. Indicatively, the mean annual temperature in the city, as measured from the 'Helliniko' station at the coastal zone near the centre of Athens, varies between 10.3 oC in January and 28 oC in July. The relative figures for average monthly rainfall are 5.3 mm for July and 64.1 mm for December. The 'Attica water region', one among the fourteen Greek water regions, is mainly constituted of the Greater Athens urban area, the Rest Attica area and some small portions of the Prefectures of Korinthos, Voiotia and Kyklades islands. The whole area of the water region exceeds 3200 km^2 (2.4% of country's total) with a total water consumption of 405 million m^3, marginally overcoming the area's water availability of 400 million m^3 per year (0.6% of country's total).

The 'Central Macedonia water region', encompasses the Greater urban area of Thessaloniki, the whole of Thessaloniki, Chalkidiki and Kilkis as well as small portions of Pella and Imathia prefectures. The Greater Thessaloniki urban area, located therein, is the second largest population and industrial agglomeration in Greece (almost 1 million inhabitants), located at the Southeastern edge of the Balkan peninsula. The area of the water region is 10389 km2 (7.9% of country's total) with a total potential of water resources at the amount of 7600 million m3 per year (11.1% of country's total) consumed to a maximum rate of 10% by all activities in the region. The mean annual temperature in the city of Thessaloniki, according to data published by the meteorological station in 'Micra', near the city's airport, varies between 5.2oC in January and 26.6^{o}C in July. Relative figures for average monthly rainfall are 20.4 mm for July and 54.9 mm for December (Tsagarakis et al, 2003).

As prevalent from the above data, water stress and scarcity is not by any means an accute problem, at least in hydro-environmental terms and for one of the two areas under study (Thessaloniki). The efforts for water supply privatization date back to the end of the '90s and they were both decisions of the social-democratic government. In fact, a first attempt was unfolded during 1991-1993 when the conservatives, in power at that time, facing a draught in Athens, added water pricing and rational management at the top of the political agenda. This was the period that significant measures towards Athenian water commodification were taken and institutionalized. For Kaika (2003), this draught was a constructed dilemma used against local population, accompanied

by a politically-driven 'shock' campaign that minimized protest against the increased water tarrifs that followed. In other words and the paternalistic greek social and political structures taken as granted, the draught was the appropriate vehicle for engaging in the market environmentalist paradigm, without proceeding first to a necessary social dialogue (Kallis and Coccossis, 2003). The official turning point between the state-hydraulic and the market-environmentalist paradigms for both Athens and Thessaloniki, however, can be found somewhere between 1998-1999 when two enactments were voted by the Greek Parliament (*Table 2*).

The basic water privatization scheme that was chosen was a mediate form between the fully private and the fully public, a relatively 'mild' political choice able to keep social tension around the issue at a low level. Both companies were divided into two legal entities; First, the 'EYDAP-assets' and 'EYATH-assets' companies that were kept public. The assets companies own and (in cases) operate the dams, the basic external infrastructure and the bulk water. Second, the homonymous 'S.A.' companies which own, operate and manage the plants and the distribution network, as in general terms recorded in Table 2. 'EYDAP S.A.' and 'EYATH S.A.' resemble a lot to 'joint-venture' firms, the Greek state being the major share holder (61% and 74% respectively by the end of 2008) in both of them. The rest shares were made available on the Athenian stock-exchange market. Although the general privatization form followed is of a concession type, the 'privatized' companies are under a strong state control, as obvious.

The basic arguments in favour of water privatization in the Greek case were connected with the market-environmentalist paradigm already adopted by other EU and OECD members at that time. In parallel, these arguments were formatted and diversified according to the economic, environmental, cultural and institutional priorities and tensions of the Greek socio-economic formation. The basic of these arguments for EYDAP so much as EYATH can be summarized as follows: i) they were poorly-functioning state-owned companies, guided by outdated 'supply-led' and 'equity of access' managerial goals, ii) they should search for capital expenditure and new investments (e.g. expansion or maintenance of the infrastructure) in an era of deficit-restrictive, inflation-controlling economic policy for the public sector, iii) they should implement sufficient technical projects in order to 'absorb' EU support funding, iv) they should improve efficiency in terms of service quality and environmental conservation, minimize water losses etc. Finally, v) they should promote economic efficiency and higher profits. The latter argument was prevalent though in a tacit manner, in the relevant discussions (Dore et al, 2004; Kaika, 2003; Kallis and Coccossis, 2003; Swyngedouw, 2005).

The water supply firms under study present important similarities, at least when their recent strategic plans and goals as well as their basic economic indicators are examined. As far as the former interrelated goals are concerned, companies declare their strategic choice to i) expand their water, wastewater and sanitation networks throughout the urban agglomerations they serve by encompassing new municipalities and communities, even beyond the Greater Athens and Thessaloniki administrative borders. Every enlargement of the service area since their foundation is firstly evaluated by a 'cost-benefit' analysis; ii) couple entrepreneurial development (c.f. attracting investments, profit-maximizing etc) with integrated services and stable consumption levels (c.f. good water quality, water-loss and unmetered consumption minimization, 'preventive' water pricing etc); iii) invest in new firms and/or diversify their own activities (c.f. invest in neighboring water companies of the Balkans for EYATH, set up new firms in new activities such as telecommunications and bottled water, taking advantage of company's infrastructure, see *Figure 1* for an indicative example) (EYDAP, 2008; EYATH, 2008).

Although a thorough reference in relevant data is out of paper's aim, an indicative example is that EYATH has recently epxanded its networks in four municipalities with more than a 100.000 inhabitants. In other words, the basic strategic axes for the companies under study are *geographical expansion*, coupled with *embedment/further promotion of commodification and commercialization* as well as *investment in new activities*. All three are basic dimensions of a transformation according the rules of the market-environmentalist paradigm.

A presentation of the basic economic and performance indicators for the two water suppliers

between 2004-2007 is offered in *Table 2*. This four-year period is of great importance for the study of entrepreneurial performance, as it stands for a mature stage of development after a first five-year period of establishing the new companies. As prevalent, all numbers and rates related to profits are much better than positive. Indicatively, both companies' profits exceed 75 million Euros (before taxes, at an aggregated level), while they increase their turnover more than 18%; increasing their profits by more than 73% and their profits coming from water sales by more than 14.5%. Net profits per employee change are at an extremely high record, at least for Athens, partly reflecting that both companies are operating with 13% less employees than four years before.

Table 2. *Basic economic, technical and performance indicators concerning the two major water-supply companies in Greece (2007)*

	EYDAP S.A. (Athens)	**EYATH S.A. (Thessaloniki)**
Foundation year	1999, Law 2744/1999 of the Greek Parliament	1998, Law 2651/1998, merger between OYTH and OATH
Background	1980, Law 1068/1980 merger between EEY and OAP.	1997, foundation of OYTH S.A.
Turnover (million €)	388,4	70,1
Turnover change*	+18,8%	+26,9%
Profits before taxes (million €)	58,5	17,4
Profits change	+73,6%	+103,9%
Employees	3680	509
Employees change	-13,2%	-12,8%
Net profits per employee change	+895%	+37,6% (for profits before taxes)
Water sales change (m^3)	+11,2%	+2,1%
Profit changes related to water sales (€)	+15,1%	+14,5%
Water customers	4.000.000	1.050.000
Water connections (millions)	1,940	0,47
Total network length (km)	8.078	1.800
External network length (km)	>310	>250
Water treatment units	4	1 + 3 chlorination units
Water supply reservoirs	56	12
Drillings	>100	>30
Average daily consumption (million m^3)	1,1	0,26
Average 4-month household consumption (m^3)	34	32,7
Change in average 4-month household consumption (m^3)		-4,7%
Average price for household customers (€/ m^3)	0,78	0,57
Change in average price for household customers (€/ m^3), lowest consumption level	+3% (0-5 m^3)	+30% (0-10 m^3)
Change in average price for household customers (€/ m^3), rest consumption levels	5.8%-6,5% (more than 5 m^3)	6,9%- 23,3% (more than 11 m^3)
Water losses percent (and change)	17,4% (-5,2%)	26,2% (-4,2%)

Source: EYDAP, 2008; EYATH, 2008, *All changes percent refer to the period between 2004 and 2007
Basic 'assets'-infrastructures and year of first operation: EYDAP: Marathonas dam (1929), Ylikis pumping station in Voiotia (1956), Mornos river reservoir & dam (1981), Evinos river tunnel & dam (1994). EYATH: Pumping stations in Kaloxori, Gallikos, Axios and Aravissos (1954-1978), Drillings in Axios and Chalkidona (1987) Aliakmonas dam & pumping station (2006)

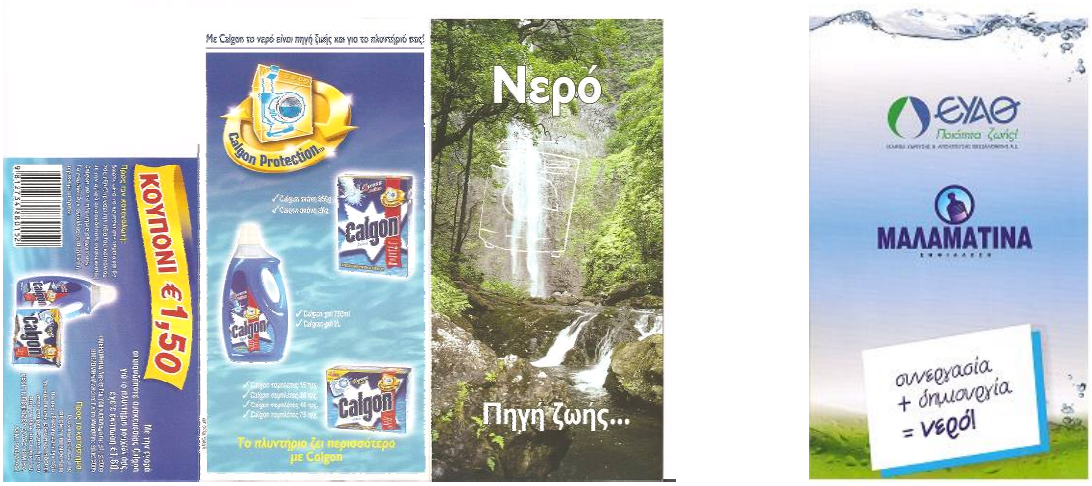


Figure 1. Two campaigns (in Greek) related to EYATH's profit-maximizing activities (the left concerns a discount coupon for washing machine products, posted to all water consumers during 2008, the right company's alliance with a Greek beverage firm for potable water production)

As obvious, companies profitability is coming from an integrated productivity-enhancing strategy, mainly based on, inter alia, increasing customers and expanding networks/infrastructures, rising water, sanitasion and wastewater tarrifs and finally, reducing the number of employees. For example, EYATH's profitability should be connected to significant increases in water prices for household customers during the recent period (EYDAP, 2008; EYATH, 2008).

As far as technical and performance indicators are concerned, both companies manage and operate extended networks, many plants and treatment units as well as other type of infrastucture, serving almost half of the Greek population. As two important indicators (water losses and average 4-month household consumption change), picturing companies environmental profile underline, the progress in such terms is less important, although profits and water losses change are not in any case comparable figures. Indeed, water losses are reduced at approximatelly 5% while consumption at 4,7% (for Thessaloniki), revealing firstly a need for further investments in internal network maintenance while outlining once again the relatively inflexible character of the water household needs, irrespective of the increments in charging (EYDAP, 2008; EYATH, 2008).

4. SOME TENTATIVE CONCLUSIONS

Based on the Greek case, the paper claimed that water privatization initiatives and the expansion of a demand-led model of water supply management are both better conceptualized in terms of a wider socio-political transformation towards the post-fordist era. In this frame, water supply systems transformation in an urban or peri-urban area were theorized as the outcome of the complex interplay of a series of global-to-local factors. As showed from the Greek case, for this restructuring process significant importance should be paid to the national economic, political and institutional setting.

Previous studies showed that, the implementation of concession contracts, were assets are kept public while the rest network is managed according to 'market environmentalism' rules and axioms, is the most frequent form, at least among the advanced countries. This study presented that relevant efforts in Greece during the past two decades, utilized a divergent privatization procedure by implementing two 'joint-venture' firms. In both these firms, the Greek state is the major share holder. As highlighted, the incremental profitability of these companies, reflects a successful case of transformation according the rules of the market-environmentalist paradigm, though the state is a major regulator and the guiding-partner of the whole privatization effort. This successful restructuring is mainly unfolding across the following axes; *geographical expansion*, coupled with *embedment/further promotion of commodification and commercialization* and finally, *investment (utilized or forthcoming) in new activities*. Of significant importance, for the Greek companies so far, are the first two axes of change. Much research has to be done in order to evaluate the prospects

and dangers of these changes for the water industry and the water consumers worldwide. What this paper tried to show is that water commodification and water firms' commercialization are a successful path to turn waters into profits, even if the state remains the major 'player'.

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CAPACITY BUILDING IN WASTEATER TREATMENT AND REUSE AS A STRATEGIC COMPONENT FOR SUSTAINABLE WATER MANAGEMENT IN THE LOWER JORDAN RIFT VALLEY

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Abstract: Public awareness programs are a key factor in order to provide direct positive impact on end users of research projects. Thus, in frame of the SMART Project (Sustainable Water Management in the Lower Jordan Rift Valley) training/dissemination activities focused on capacity building programs for primary schools in the research area were conducted. The paper aims to assess the results of the pilot training denominated Teaching Unit Water Fun, conducted in November 2008 in Ramallah, Palestine in a primary school for a group of 21 students between 10 and 11 years old. Also it is summarised, the principal results for the training for trainers organized for a group of 22 participants, teachers from Palestinian primary schools and Palestinian water administration authorities are discussed. Furthermore, the paper looks at the fundamental challenges to enhance the program at a national scale incorporating it into the official curriculum and evaluating its local impact in more effective and equitable use and allocation of water resources

Key words: Capacity building, Guidelines training modules, SMART-Project, Water resource management

INTRODUCTION

Capacity building (CB) oriented to sustainable management of available water resources can play a significant role in the alleviation of water constrains in the Lower Jordan Rift Valley. Thus, in the framework of the project Sustainable Water Management in the Lower Jordan Rift Valley (SMART), a training program, based on demonstration, technology transfer and capacity building in the field of Decentralized Wastewater Systems Solutions (DWWSS) was designed, developed and test as a pilot teaching unit for primary schools in Ramallah, Palestine.
As a dissemination strategy, the training program gives the possibility to pupils to conduct experiments in water/wastewater analysis and to build wastewater filters and constructed wetlands models. The training enhances the reflection process about wastewater components and gives arguments to the participants to understand the possibilities of wastewater treatment and its reuse purposes in decentralized systems. Additionally by the experiments, students understand wastewater as a resource; especially in regions with water shortage. Moreover, building a constructed wetland model, students recognise constructed wetlands treatment systems as a modern and ecological treatment technology that fit in well with the environment and can provide treated wastewater for irrigation.

The central methodological objective of the teaching Unit "Water Fun" is to achieve Human Resource Development (HRD), which represents one of the cross-section elements to accomplish economic and social development (see figure 2). The teaching unit is oriented to introduce concepts of water value and wastewater reuse by the hands-on contact with wastewater treatment eco-technologies and waste/water analysis experiments.

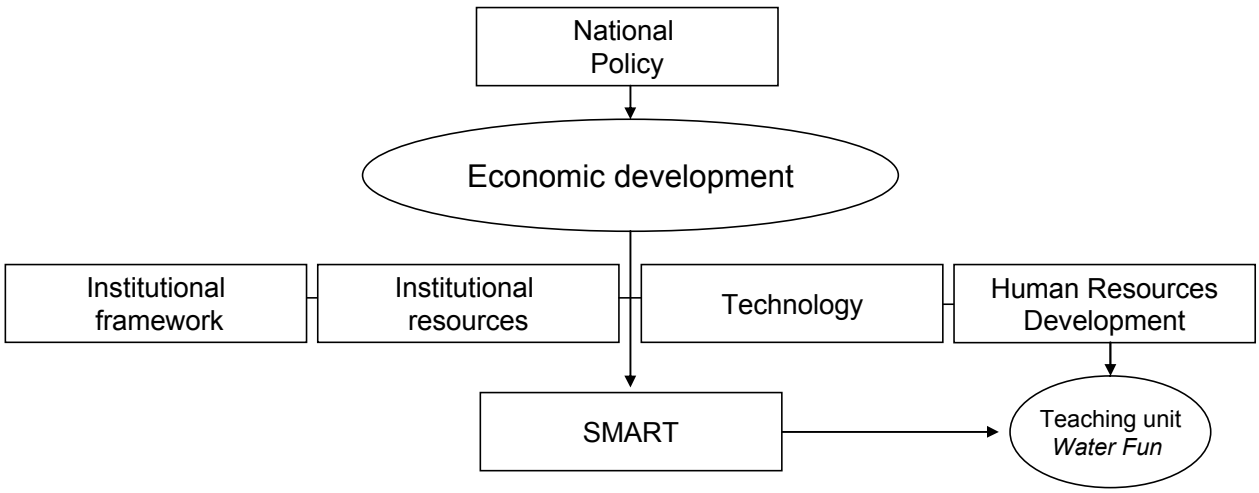


Figure 1 Capacity building framework. Adapted from (Al-Jayyousi 2001)

RESULTS

The pilot training activity denominated Teaching Unit "Water Fun" was conducted in November 2008 in Ramallah, Palestine in a primary school for a group of 21 students between 10 and 11 years old. The CB activity initiates with a training for trainers organized for a group of 22 participants. Teachers from different Palestinian primary schools and Palestinian water administration authorities took part in the activity (see figure 2). The selection of the participants was coordinated with the Palestinian Water Authority (PWA) and the Palestinian Ministry of Education (PME). The training for trainer activity was attended by teachers from nine different Schools of Ramallah as well representatives from the PWA and PME.

Selected Area

Nine different zones from the central west part of Ramallah were covered by the CB program. The teaching unit was conducted considering the schools located in the selected areas presented in Table 1. From each of the selected area a school was represented by one teacher, who was taking part in the training for trainers.

Code	Ramallah selected areas
1	Dir Bzea'
2	Betoneyia
3	Biet lieqiah
4	Al-janiah
5	Raskarkar
6	Bani Zaid
7	Kharbath Musbah
8	Kafer Neama
9	Bait Serah

Table 1 Schools covered by the training program

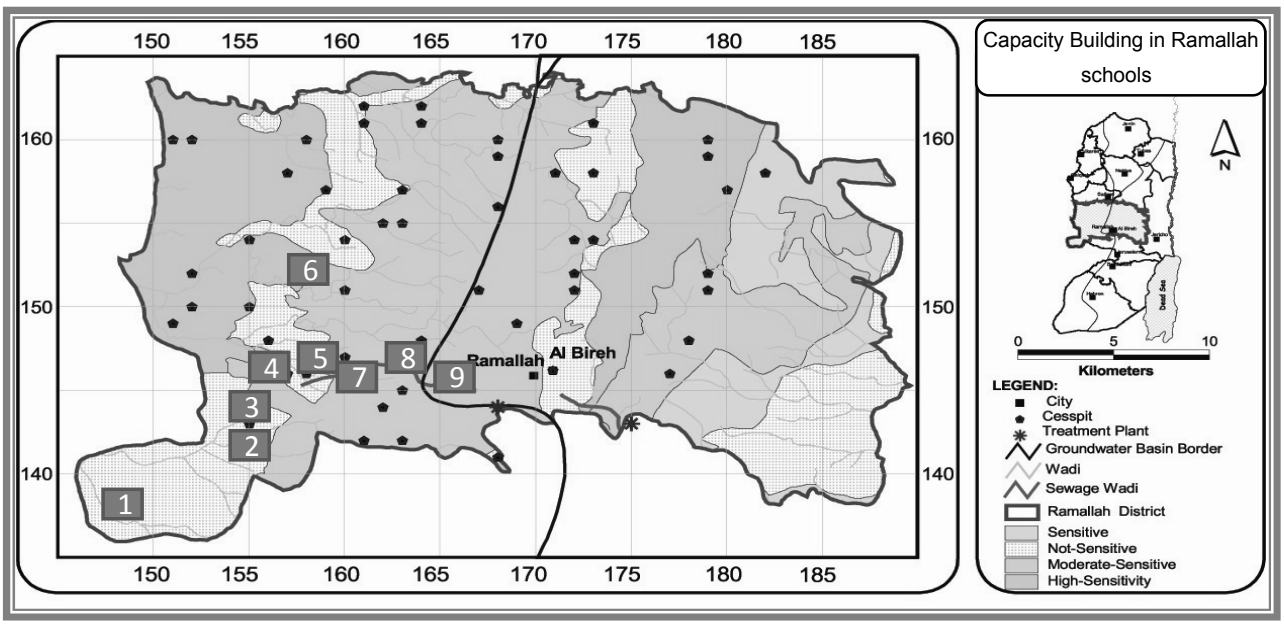


Figure 2 . Zones covered by the capacity building program in Ramallah City. ***Source: Adapted from (Mimi and Assi 2009)***

Elaboration of didactic materials

The teaching material based on the primary school teaching unit "Constructed Wetland" (Van Afferden et al. 2008) that was developed for European primary schools within the European Community funded project "Play with Water". This teaching unit was translated and modified according to suggestions from our local Palestinian partners. One of the important modifications was related to the teaching Unit's name. Originally the teaching unit was presented in Ramallah under the name "Play with Water". After the pilot activities it was realized that modifications in the name were necessary. Considering the water constrains in the Lower Jordan Rift Valley, children should avoid to "playing" with water. For this reason the name "Water Fun" will be adopted for future activities.



Figure 3 Teaching unit *Water Fun* applied in Ramallah, Palestine at 19.11.2008. Elaborated didactic material used during the teaching unit

The development and implementation of the teaching unit "Water Fun" enhances the capacity building dimension in IWRM. As a key element the program explores human resources development as a strategy to achieve water development management in the Lower Jordan River Basin. The main importance of CB is directly associated with the increasing need and value of water. Particularly in West Bank in the Palestinian territories where only less than 6% of the population is connected to treatment plants (Mahmoud et al. 2003), the implementation of such CB programs in schools have a direct impact on the water value attitudes and by this CB will facilitate the future implementation of a sustainable water recourses management.

The teaching unit sensitizes pupils but teachers as well for the basic and essential concepts related to water value and scarcity. Most of the water analysis experiments promote reflections and behaviour changes concerning water consumption and enhance a vision of considering wastewater as a resource and not as a waste.



Figure 4. Teaching unit *Water Fun* applied in Ramallah, Palestine at 19.11.2008. Picture A, children from Al-Shurook school build a constructed wetland model. Picture B, teachers from different Ramallah schools attend the CB activity in the Palestinian Water Authority PWA.

The teaching unit "Water Fun" as a capacity building strategy represents an adequate tool to empower local communities and organizations in the implementation of water policy and management initiatives trough education. The direct impact of the CB strategy on the Integrated Water Resources Management in the research area is associated with a common lack of basic knowledge of water resources management in the population. Thus, the development of water resources knowledge at the primary school level is a sustainable strategy to establish effective water management programs on the long-run.

The fundamental challenges to enhance the program at a national scale in the Palestinian territories, Israel and Jordan is a central point to be explore in the dissemination strategy of capacity building in SMART project. Incorporating the teaching unit "Water Fun" into the official curriculum of primary schools of these countries can have a local impact in water management and water valuation behaves. As a consequence a more effective and equitable use and allocation of water resources will be promote.

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RESEARCH ON BIOLOGICAL GRANULAR ACTIVATED CARBON (BGAC) IN SECONDARY TREATMENT WASTEWATER REUSE

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Abstract: In this study, the performances of Biological Granular Activated Carbon (BGAC) bio-adsorption and Granular Activated Carbon (GAC) adsorption in terms of secondary treatment wastewater reuse were investigated with real sewage effluent (RSE) of Maturation Pond M13, belong to the Binh Hung Hoa aerated lagoon wastewater treatment plant, Vietnam. The main aims of this study are to verify and compare the efficiency of secondary treatment wastewater reuse by GAC (adsorption) and acclimatized BGAC (bio-adsorption). The results indicated that the performance of bio-adsorption was significantly better than that of adsorption in all cases, showing the practical use of BGAC in filtration process. The most significance was observed at a real SE with a hydraulic loading v = $1m^3/m^2.h$, having 100% of $N-NH_4^+$ removal and 89% of $N-NO_2^-$ removal by using BGAC, respectively. In addition, although the compositions and properties of SE always change but the BGAC filtration system operates still stable and don't be shock and the COD removal efficiency is maintained stable over 30%.

Key words: Biological Granular Activated Carbon (BGAC), Granular Activated Carbon (GAC), bio-adsorption, adsorption, wastewater, reuse.

1. INTRODUCTION

The real sewage effluent (RSE) of Maturation Pond M13, belong to the Binh Hung Hoa aerated lagoon wastewater treatment plant, Vietnam. A potential alternative water resource which may replace existing water resources for non-potable use is renovated wastewater. Advanced treatment of treating wastewater will yield a better quality effluent which can be used as an alternative water resource and therefore reduces the demand on fresh water supplies. The daily volumes of domestic and industrial wastewater discharged to the canals in Hochiminh City were about 710,000 m^3 and 35,000 m^3 are the potential water resource for reuse (HCMC 2001). The domestic wastewater typically contains pathogens, suspended solids, nutrients (nitrogen and phosphorus) and other organic pollutants. For decreasing the environmental and health hazards, these pollutants need to be removed to permissible limits for safe disposal of wastewater (Rani and Dahiya). Consequently, removal of the organic contaminants and pathogens from wastewater is very important for its reuse in different activities.

Activated Carbon (AC) adsorption has been widely applied in removing organic matters from wastewater as it has a strong affinity for attaching organic substances even at low concentration. Having larger surface area for adsorption, Granular Activated Carbon (GAC) is one of the best adsorbents for removing various organic contaminants (Nishijima, Akama et al.). The GAC adsorption systems are therefore considered to apply for producing the effluent of high quality from sewage treatment plant which can be reused for various purposes. However, even though it has high adsorption capacity, GAC can only maintain its adsorption for a short time after its available adsorption site becomes exhausted with adsorbed organic pollutants (Aktas and Çeçen 2007). It is well known that GAC is also a good support media for microbial growth. Thus, Biological Granular Activated Carbon (BGAC) with attached biomass can effectively remove organic contaminants by bio-adsorption (both by adsorption and biodegradation) (Nishijima, Akama et al. 1997; Carvalho,

Duque et al. 2007). The concept of bio-adsorption is that adsorption is more dominant before BGAC is in full adsorption capacity and the biodegradation play major role after that (Hoang 2005; Carvalho, Duque et al. 2007). The BGAC bio-adsorption is usually applied in the bioreactors either fixed bed or fluidized bed configurations (Aktas and Çeçen 2007). The previous studies showed that BGAC bio-adsorption systems were very efficient (Zhao, Hickey et al. 1999; Maloney, Adrian et al. 2002; Loh and Ranganath 2005; Persson, Långmark et al. 2005; Ho, Chung et al. 2008) due to their simultaneous adsorption of non-biodegradable matter and oxidation of biodegradable contaminants in a single reactor.

2. MATERIALS AND METHODS

2.1 Study area

The wastewater was investigated with RSE of Maturation Pond M13, belong to the Binh Hung Hoa aerated lagoon wastewater treatment plant, Vietnam. The treatment plant was built in the framework "*Tan Hoa - Lo Gom canal sanitation and urban upgrading project*" – The cooperation between governments of Belgium and Vietnam (Committee 2006).

Table 1. The compositions and properties of RSE

Item	pH	Temp (^{0}C)	COD	SS	N-NH_4^+	DO
Influent range (mg/L)	6.8 – 7.5	26 – 29	40 – 60	30 – 45	410 - 580	2 – 6



Fig. 1. Map of the location of the experimental area at Binh Hung Hoa plant, Hochiminh City, Vietnam, including a model research

2.2 Experimental setup

The model was setup in the sentry box at Binh Hung Hoa treatment plan. The average temp was about 28 – 32 ^{0}C.

Table 2. Quality specifications of GAC (P.V.C Environment & Construction Consultants Co.)

Media	Size range (mm)	Density (kg/m^3)	Specific surface area (m^2/g)
GAC	2.36 – 4.76	410 - 580	-
Sand	1.5 – 2.0	1400	950

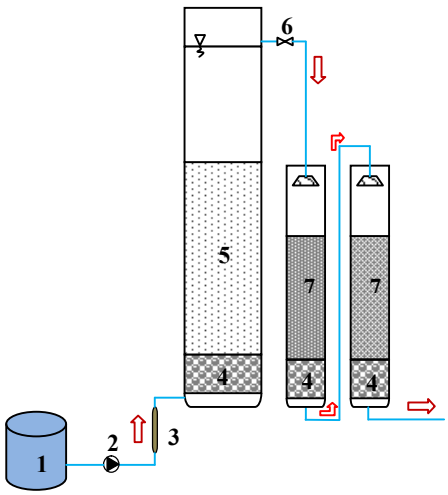


Fig. 2. Schematic diagram of the Model 1.

(1) Feed tank; (2) feed pump; (3) liquid flow meter; (4) support layer; (5) sand layer; (6) sampling port; (7) GAC layer.

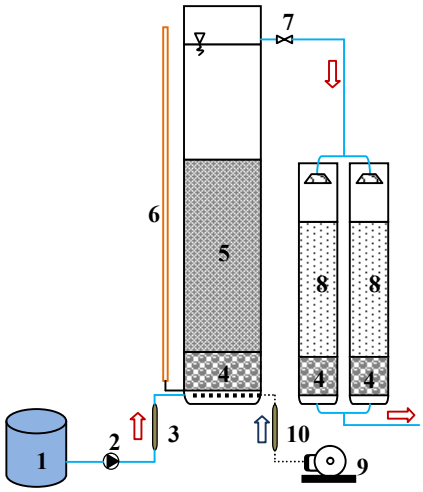


Fig. 3. Schematic diagram of the Model 2.

(1) Feed tank; (2) feed pump; (3) liquid flow meter; (4) support layer; (5) GAC layer; (6) water level indicator; (7) sampling port; (8) sand layer; (9) air compressor; (10) airflow meter.



Fig. 4. Granular Activated Carbon (GAC)



Fig. 5. Sand

2.3 Start-up and operational conditions

Model 1: The RSE of Maturation Pond was taken out directly to BSF. After that, it was introduced to two GACF. The wastewater was supplied continual top down. Model 1 was operated in 3 $m^3/m^2.h$ and 3.2 $m^3/m^2.h$ hydraulic loading, respectively.

Model 2: The RSE of Maturation Pond was taken out directly to BGACF. After that, it was introduced to two BSF. The wastewater was supplied continual top down. In this case, we used air blower in BGACF to enrich the dissolved oxygen (DO). The distribution system of air was connected with five pumices stone which were setup in bottom BGACF. Model 2 was operated in 1 $m^3/m^2.h$, 1.3 $m^3/m^2.h$, 2 $m^3/m^2.h$ and 3 $m^3/m^2.h$ hydraulic loading, respectively.

2.4 Analytical methods

Samples were collected every day, refrigerated at 4^0C before analysis, except for pH measurement which was done after sampling. Samples were analysed for COD; pH; Turbidity; Colour, Suspended Solid (SS); $N\text{-}NH_4^+$, $N\text{-}NO_3^-$, $N\text{-}NO_2^-$ were defined base on standard method (Amman 1998). Other parameters, such as pH and DO were measured with pH 211 Micro pH Meter and DO-5509 oxy-meter, respectively.

3. RESULTS AND DISCUSSION

3.1 Model 1

On first 6 days, the system was operated with hydraulic loading of 3.2 m^3/m^2.h. The removal efficiencies of some parameters such as: turbidity, colour, SS fluctuated from 30% to 60% when passing through BSF. However, at the end of filter cycle, removal efficiencies had decrease although the RSE concentration was not high.

The removal efficiencies fluctuated from 15% to 25% when passing through GACF. Removal efficiencies were defined by adsorption of GAC (Tawfik, Klapwijk et al. 2004).

After 6 days and later, the system was operated with hydraulic loading of 3 m^3/m^2.h. The removal efficiencies of some parameters such as: turbidity, colour, SS moved up and down from 25% to 40% when passing through BSF. Removal efficiencies maintained stable from start to finish cycle and lower than hydraulic loading of 3.2 m^3/m^2.h. When passing through GACF, removal efficiencies of turbidity, colour, SS maintained stable about 10%.

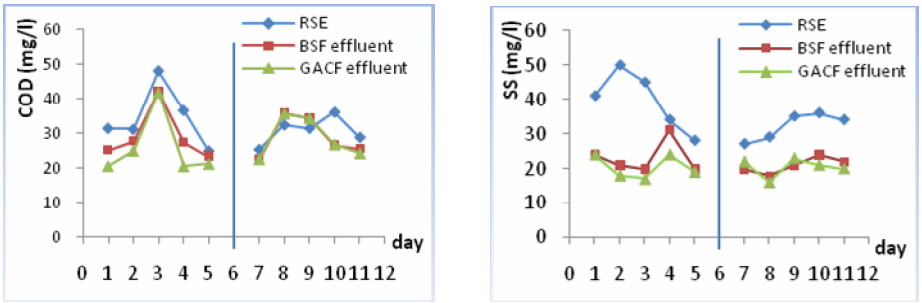


Fig. 6. Relation between COD – time and SS – time.

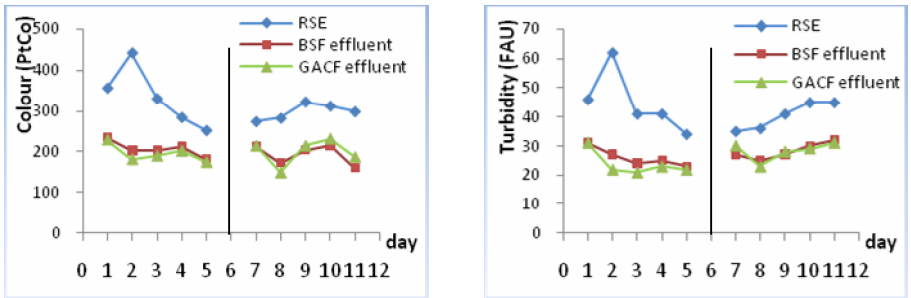


Fig. 7. Relation between Colour – Time and Turbidity – Time.

3.2 Model 2

On first 6 days, the system was operated with hydraulic loading of 1 m^3/m^2.h. The bio-films started to be formed and the removal efficiencies were low from 8% to 13%. However, there was a slight increase in removal efficiencies after that. Removal efficiencies were remained constant

about 40%. With hydraulic loading of 1.3 $m^3/m^2.h$, 2 $m^3/m^2.h$, 3 $m^3/m^2.h$, COD removal efficiency was kept stable about 30% when passing through BGACF and maintained constant from 20% to 25% when passing through BSF, respectively.

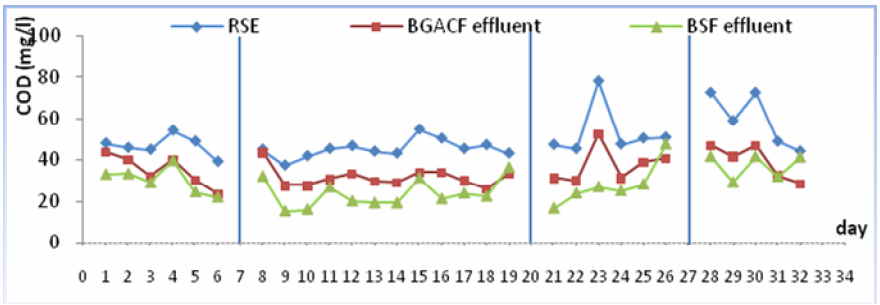


Fig. 8. Time course of the amount COD related to fourth hydraulic loading.

With hydraulic loading of 1 $m^3/m^2.h$, 1.3 $m^3/m^2.h$, the removal $N–NH_4^+$ efficiency was high and levelling off. All of the $N–NH_4^+$ was transferred into $N–NO_3^-$

$$NH_3 + O_2 \xrightarrow{enzyme} HNO_2 + O_2 \xrightarrow{enzyme} HNO_3$$

With hydraulic loading of 1 $m^3/m^2.h$, 1.3 $m^3/m^2.h$, the $N–NH_4^+$ concentration in RSE was about 16.5mg/l and after passing through BGACF, the removal efficiency was maintained stable over 50%. The transfer $N–NH_4^+$ efficiency was from 2% to 7%, respectively.

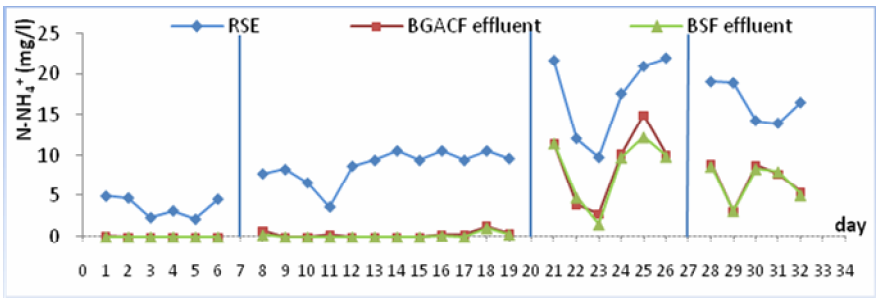


Fig. 9. Time course of the amount $N–NH_4^+$ related to 4 hydraulic loading.

In 2 hydraulic loading first, $N\text{-}NO_2^-$ concentration in RSE was from 2.5 to 3mg/l. When passing through BGACF, transfer $N\text{-}NO_2^-$ efficiency was increased slightly and level off from 95% to 98%. With BGACF, the process of transfer $N\text{-}NO_2^-$ efficiency was fluctuated in 2 hydraulic loading later. Although the process later of transfer $N\text{-}NO_2^-$ efficiency was not be high but it was remained stable when passing through BSF.

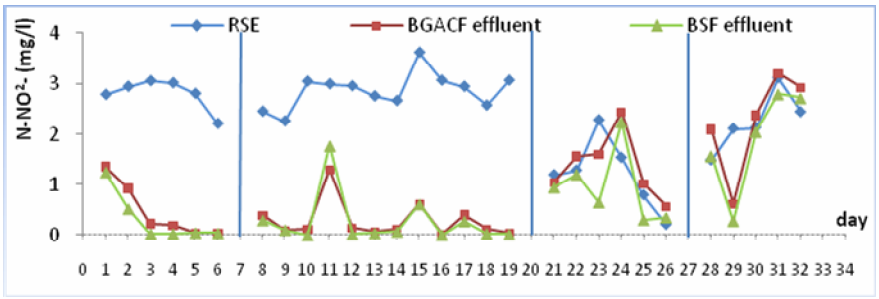


Fig. 10. Time course of the amount $N\text{-}NO_2^-$ related to 4 hydraulic loading.

The graph shows that the nitrification was remained stable in 2 hydraulic loading first. In last 2 hydraulic loading, the nitrification was still gotten well but its efficiency was not high and steady

when comparing to other ones above. The $N\text{-}NO_3^-$ transfer was about 12.6mg/l when passing through BGACF. With BSF, the value was average 3.51mg/l.

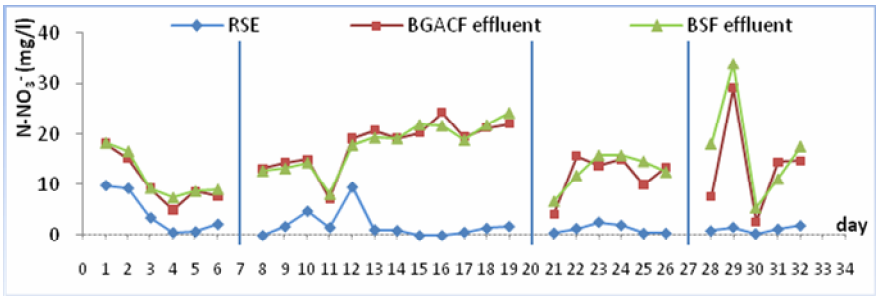


Fig. 11. Time course of the amount $N\text{-}NO_3^-$ related to 4 hydraulic loading.

The removal efficiencies of some parameters such as: turbidity, colour, SS were maintained stable from 45% to 55% when passing through BGACF. With BSF, removal efficiencies were fluctuated from 5% to 20%. The effluent was higher than Vietnam discharged standard TCVN 5502 – 2003.

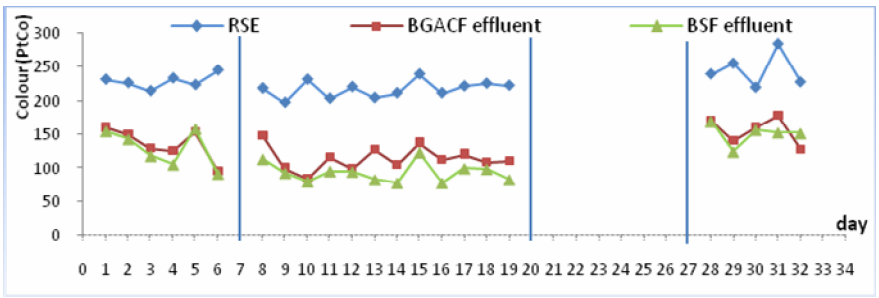


Fig. 12. Time course of the amount colour related to 3 hydraulic loading.

The biomass attached onto the GACs reached the steady phase with an amount of approximately 12.15 g/L



Fig.13. Biomass was observed under microscope (x40) and on the GACs surface.

The disinfection in hydraulic loading of 2 $m^3/m^2.h$ is very high. It's more than 97%. BGAC with attached biomass can effectively disinfect coliforms on the surface by both adsorption and biodegradation (Marshall 1992; Nishijima, Akama et al. 1997; Ueda and Horan 2000; Tawfik, Klapwijk et al. 2004; Carvalho, Duque et al. 2007).

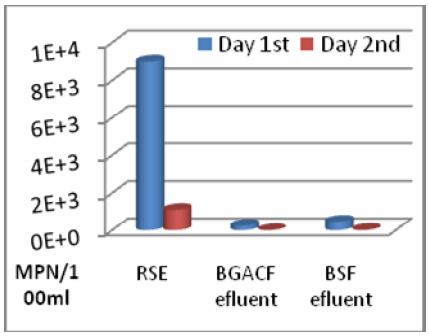


Fig. 14. The disinfection in day 1st and 2nd of hydraulic loading of 2 $m^3/m^2.h$.

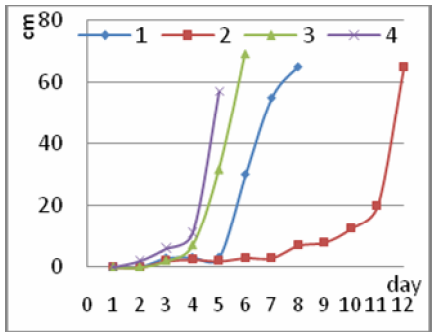


Fig. 15. Removal loading rates at different hydraulic loadings

The hydraulic loading at 1.3 $m^3/m^2.h$ had the highest number of days, 12 days.

4. CONCLUSIONS

In this study, the BGACF was setup to treat RSE for reuse. The higher removal efficiencies from BGACF than GACF were because of the simultaneous adsorption and biodegradation processes (Xing, Ngo et al. 2008). The removal efficiencies when using GAC were low, it were from 3.5% to 12.9%. The removal COD efficiency when using BGAC was over 30%. Removal efficiencies were remained stable about 40% with colour, turbidity and SS. The nitrification was still got well and the removal $N\text{-}NH_4^+$ efficiency from 48.6% to 100%.

The optimal hydraulic loading for BGACF in Binh Hung Hoa plant is 1.3 $m^3/m^2.h$.

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A COMPARATIVE ANALYSIS OF THE APPLICATION OF ECONOMIC INSTRUMENTS IN WASTEWATER TREATMENT BETWEEN GERMANY AND INDIA

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Abstract: Water pollution is one of the most pressing environmental problem. It reduces the quantity of useable water and increases the cost of supplying water. Economic incentives, and especially wastewater tariffs, are vital in achieving least-cost wastewater management. If economically efficient wastewater tariffs are incorporated and if the structure of the project is efficient by design, and little or no additional economic analysis is required.

The study presented in this paper where an attempt is made in calculating an effluent tax in controlling wastewater pollution. In this regard a comparison has been made in respect to India and Germany. Pollution tax in the perspective of wastewater disposal can be denoted as the price the polluter has to pay for using the wastewater disposal services from the environment. As there is no wastewater disposal market so the price cannot be determined in the market and the supply and demand schedules fail to get observed Murty et al(2000)in their study showed the case of industrial wastewater in India with excess COD concentration and the tax liabilities calculated using the average tax rate is of Rs 0.32(or.016 euro) per 100 gm. In case of Germany, the effluent charge comprises about 5% of total costs. For our study we consider the case of Leipzig, a city in the state of Saxony. Following the calculation of Murty et al(2000)the tax on annual wastewater quality per 100 grams in rupees is $\frac{5.5}{55} \times 100 \times 1000^{11}$. When estimated over the total population**,** the tax per head accounts to be very high for Germany as compared to the case study of Murty et al in India.

In Germany the water charge itself covers the water pollution charge. Both the more than proportional increase of the municipal wastewater charge and the high prospective demand for investment in sewage works has brought about the search for new organizational forms and different methods concerning water supply and wastewater disposal. But the wastewater policy instruments have been successful in reducing the wastewater pollution and enhancing technological progress in the country. Although the experience in applying economic instruments remains limited, particularly in India, there is evidence that effluent and user charges have the potential for effective application by helping to pay for environmental improvement. They should be accompanied by investment in wastewater treatment facilities and, locally, by appropriate regulatory instruments as well as programs to persuade water users to change their polluting behaviour.

Key words: Wastewater, effluent tax, Germany, India.

1. INTRODUCTION

Water pollution is one of the most pressing environmental problem. It reduces the quantity of useable water and increases the cost of supplying water. We here focus on municipal wastewater which mostly occurs in urban areas from domestic and industrial sources and is a major cause of water pollution. Wastewater pollution is a major problem but little investment has been made in the LDCs in the past on sewage treatment facilities. According to the World Bank, "The greatest challenge in the water and sanitation sector over the next two decades will be the implementation of low cost sewage treatment that will at the same time permit selective reuse of treated effluents

for agricultural and industrial purposes "(Volkman ,2003) . Figure 1 shows the interrelationship between wastewater generation, water pollution and the possible remedies.

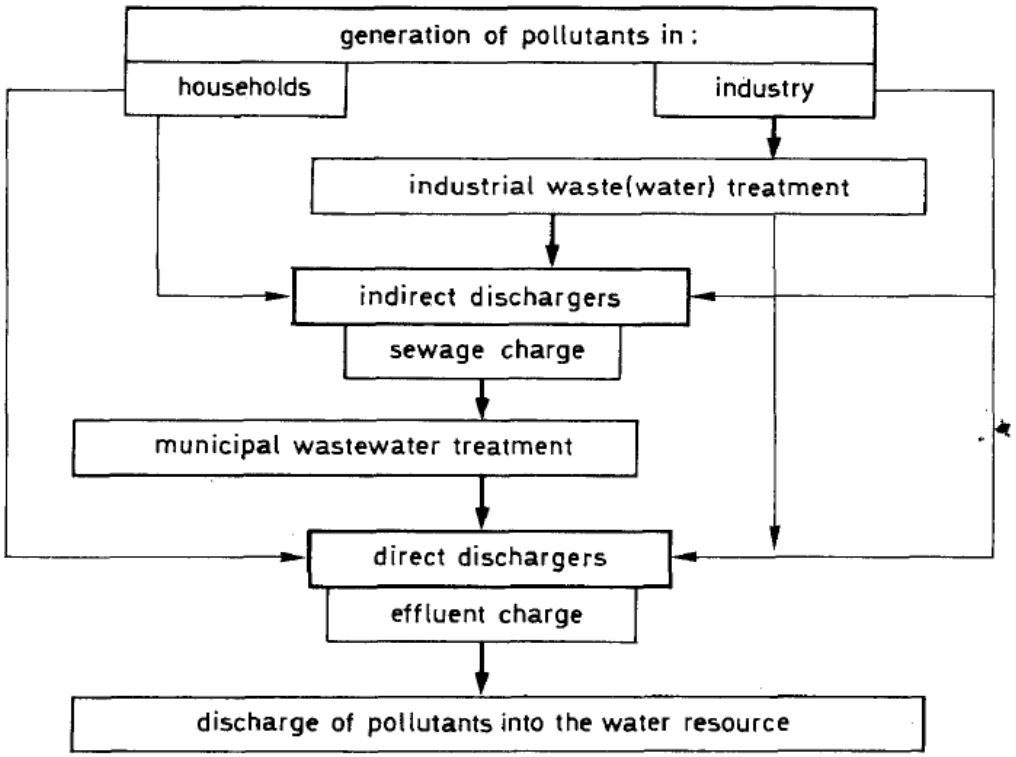


Figure 1: Waste, wastewater treatment, and water pollution (Source: Pethig and Fiedler(1989))

UNEP(United Nations Environmental programme)/WHO(World Health Organization)/HABITAT(United Nations Human settlement Programme)/WSSCC(Water Supply and Sanitation Collaborative Council), Strategic Action Plan on municipal wastewater management listed certain prerequisites for successful municipal wastewater management some of which focused on selecting appropriate technologies for efficient and cost–effective use of water resources and consider eco-technology alternatives, involving all stakeholders from the beginning and ensure transparency in management and decision–making processes, financial stability and sustainability by linking the municipal wastewater sector to other economic sectors, introducing innovative financial mechanisms involving private sector involvement and private- public partnership, and considering social equity and solidarity to reach cost recovery .

Once the initial capital investment has been made, the institutional arrangements are in place to maintain and continue to provide the services for which the investment is designed for. Though authorities invest in wastewater treatment but less attention is paid to sustain the facility. Economic incentives, and especially wastewater tariffs, are vital in achieving least-cost wastewater management. If economically efficient wastewater tariffs are incorporated into the project, then the project is efficient by design, and little or no additional economic analysis is required. In such a case, economic analysis is more productively focused on the overall management framework, rather than on an individual project(Dole., 2002).

Municipal wastewater treatment plants(MWWTP)in most of developed countries typically levy a "user charge" on all firms that discharge their wastewater into the public sewerage. The purpose of this charge is either to pay for the plant's purification service or simply to cover the plant's operating cost. In what follows we assume that the "user charge" is based on the waste content of industrial sewage. Effluent charges for water pollution has been successfully used in many countries

like China, France, Germany, Netherlands, etc. In France the system is designed primarily to raise revenues which are used to maintain or improve water quality(Bernstein., 1993). On the other hand Swedish municipalities usually charges a combination of fixed and variable charge which turn out to be effective due to the growing number of households and small industries attached to the sewer system and extended water treatment facilities. According to Pethig and Fiedler(1989)wastewater effluent charge on purification plants has virtually no theoretical support. Clearly, the allocative impact of emission charges is well understood and well established in theoretical environmental economics. But to the best of our knowledge it has not yet been rigorously investigated what the efficiency implications of water quality management are when two different types of emission charges are levied.

Merrett(2000)focus on the problem in assessing the impact of effluent charges where many are introduced to fund government environmental initiatives rather than to induce changes in behaviour on the part of the polluters. An additional source of complication is that many industrial effluent flows contain a variety of pollutants. A charge directed at one pollutant may induce process changes that lead to reductions in one or more pollutants.

The effluent tax rate was calculated by Murty et al(2000)by substituting the actual average concentration of the effluent in the wastewater standard(i.e 250 milligrams per litre).The effluent can be taken in terms of the concentration of COD(Chemical Oxygen Demand),BOD(Biological Oxygen Demand) or SS(Suspended Solids).

To the others' knowledge, the study presented in this paper is a study where an attempt in calculating a tax in controlling wastewater pollution where a comparison has been made in respect to India and Germany. In the context of India firstly it is advantageous to recognize that polluters and regulation inspectors are both eventually interested in maximizing own incomes subject to risks of detection, penalties, etc. That being so, an enforcement regime must ensure that the expected income maximizing strategy for the polluter, adjusted for risk, penalty, etc is to comply with the standard.The next section describes the theoretical framework of the effluent tax in respect to wastewater. Section 3 describes the background of the study. The results are reported in Section 4, and Section 5 concludes the paper.

2. THEORETICAL FRAMEWORK OF THE STUDY

Economic instruments are ideally situated for reconciling environmental concerns with development needs and integrating environmental and economic policy by virtue of their market correction quality, efficiency, flexibility in accommodating heterogeneity and adjusting to changing circumstances. Though there is a vast literature on the use of economic instruments for pollution control(Baumol., 1972; Baumol and Oates., 1998; Crandell.,1983; Pearce et al.,1989),but the idea was first proposed by Pigou. According to him, since social cost of pollution is in excess of the private cost to the polluter,the government should intervene with a tax ,making pollution more costly to the polluter and so the polluter will produce less pollution. This tax came to be known as the Pigouvian tax(Kolstad,2003). Coase(1960)advocates the role of state in defining and enforcing property rights for environmental resources and in mitigating transaction costs but rules out government intervention in the form of specifying standards or levying a tax to correct the externality. It is difficult to define property rights for natural resources like air, water in lakes, rivers and oceans, and scenic spots(Sankar., 1998).

Pollution tax in the perspective of wastewater disposal can be denoted as the price the polluter has to pay for using the wastewater disposal services from the environment. As there is no wastewater disposal market so the price cannot be determined in the market and the supply and demand schedules fail to get observed. However under the property right regime, environmental regulation by the government or the public can make the polluter liable to pay the price for the disposal of wastewater. The polluter here pays the price which incorporates the cost for environmental regulation. So we can consider the MCA(Marginal cost of abatement) as the demand price for the disposal of wastewater.

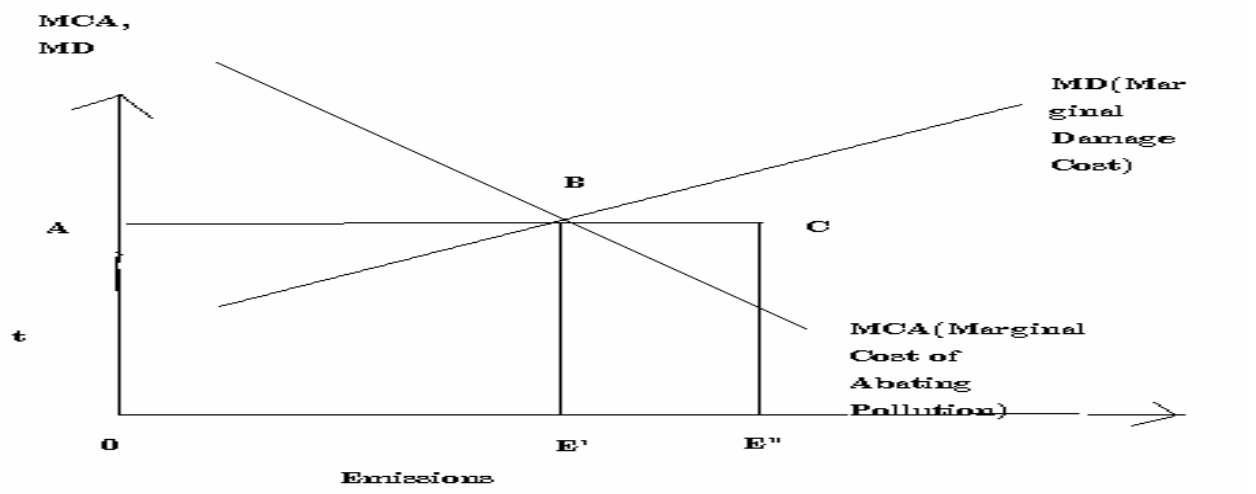


Figure 2: Wastewater pollution and its abatement

There involves an opportunity cost or damages by the public by allowing pollution. The supply price of wastewater disposal is the price charged to the polluter by the government or public for every unit of waste disposal. Figure 2 describes the supply curve of wastewater disposal as the rising MD(Marginal Damage) or supply price with respect to the pollution loads. E' and E" stand for pollution load with and without tax instrument. We impose a pollution tax, t. When the polluter is incorporating a pollution abatement technology then maximization of welfare requires that the pollution should be reduced upto a level where MCA=MD. Thus when a tax per unit of pollution, t is imposed on the polluter, he has an incentive to reduce it to an optimal level E'. So either he have to pay a tax of an amount E'BCE" or he have to reduce it to a level E'.

3. BACKGROUND OF THE STUDY

Germany having a population of 82.44 million is the most populous member state of the European Union. The organization and implementation of the water supply and wastewater disposal belong to the traditional duties of the municipalities, in accordance with state water laws. According to the Federal and Local law it is mandatory for the broad range of economic sectors to specifically pre-treat the wastewater before it may be discharged into a municipal system. This minimizes the difficult contaminations from sewage sludge. The federal regulations are further substantiated at the level of the 16 German federal states. Based on the different circumstances a state water law, state effluent charge act, etc. has been empowered. The requirements and standards are formulated mostly at the lowest level, where framework requirements of the respective higher level are observed.

In 1998 Germany has 6117 MWWTPs with a capacity of 138.5 million total inhabitants. The effluent quality has improved in the new federal states which signifies a good performance of the MWWTPs. The municipalities compete for industrial investments to foster their own economic development and to increase the local tax base(Holländer,2006). According to information from the Federal Statistical Office, the total volume of wastewater in 1998 was approx. 9.6 billion m^3, including about 4.9 billion m^3 of domestic and commercial wastewater. From 1991 to 1996, approx. € 22 billion were invested in the overall area of MWWTPs in Germany. The annual expense of municipalities and wastewater associations for public wastewater disposal amounts to over € 6 billion. Table 1 states the wastewater discharge in Germany.

*Table 1: Wastewater discharge into water bodies**

Total (mn/m^3)	Treated waste water (mn/m^3)	Untreated waste water (mn/m^3)
40 591.6	11 410.3	29 181.3

*Incl. foreign water and precipitation water from the sewage system, excl. unused water
Source: The German Water Sector(2003)

As far as the economic instruments are concerned, several countries apply effluent charges in order to finance necessary measures for wastewater collection and purification, and to provide financial incentives for reducing discharges of effluent. The German Effluent Charge Law authorizes States to levy charges on direct discharges of specified effluents into public waters. The pollutants considered for the purposes of effluent charges are settleable solids, COD, cadmium, mercury and toxicity to fish. In setting the charge base, the law established the right to discharge and includes all of the physical, chemical and biological data and monitoring procedures pertaining to wastewater quality. For each organization, the State also specifies a total discharge based on historical volumes of wastewater allowable per year. Since the effluent charge is combined with a permit procedure, the maximum effluent level is also specified. The actual effluent discharged by the organization must be of a quality equal to, or higher than, the minimum requirements laid out in the regulation. The taxable base is specified in terms of concentration per cubic meter of discharge volume or per ton of product produced.

The Effluent Charges Act or *Abwasserabgabengesetz, AbwAG*)(1994) makes the provision that a charge must be paid for the point source discharge of treated wastewater into a water body. Moreover, the payment of effluent charges in no way exempts one from the responsibility of treating wastewater. The charge is calculated according to the level of effluent discharged. The Wastewater Charges Acts defines different units of noxiousness for a number pollution indicators. Each unit of noxiousness incurs a charge of approx 35 euro. The accumulated sum of units multiplied by the unit charge results in the total wastewater charge to be paid by industrial and municipal water users. On the other hand the Wastewater Ordinance(*Abwasserverordnung*, AbwV) sets technical standards, such as legally binding pollutant limits, which are for various kinds of wastewater. Table 2 states the minimum requirements concerning the discharge of municipal wastewater according to Wastewater Ordinance(AbwV).

In India, about 25% of the population of 644 class I cities and class II towns inhabits in 87 coastal cities and towns. So far as the case of India is concerned, it is estimated that 22,900 mld of domestic wastewater is generated from urban centres against 13000mld of industrial wastewater. The treatment capacity available for domestic wastewater is only for 6000mld against 8000 mld of industrial wastewater. Government of India is assisting the local bodies to establish STP under the GAP(Ganga Action Plan) and subsequently under the National River Action Plan. CPCB has been playing a key role in abatement and control of pollution in the country by generating relevant data, providing scientific information, rendering technical inputs for formation of national policies and programs, training and development of manpower, through activities for promoting awareness at different levels of the Government and Public at large. The availability of treated effluent is regulated under the provision of Water Act, 1974 by their concerned State Pollution Control Boards. CPCB along with the State Boards take collective action with regard to highly polluting categories of industries and identified industries discharging effluents directly into rivers/lakes and other water bodies without requisite treatment.

Table 2: Minimum requirements concerning the discharge of municipal wastewater according to the Wastewater Ordinance.

Size classes of WWTP's	Chemical oxygen demand (COD)	Biological oxygen demand in 5 days (BOD5)	Ammonium nitrogen	Total nitrogen as total of ammonium-,nitrite-,and nitrate nitrogen	Phosphorus total(P total)
Population units	mg/l*	mg/l*	mg/l*	mg/l*	mg/l*
less than 1,000	150	40	-	-	-
between 1,000 and 5,000	110	25	-	-	-
between 5,000 and 10,000	90	20	10	-	-
between 10,000and 100,000	90	20	10	18	2
larger than 100,000	75	15	10	18	1

*1 population unit = 60 g BOD5/d in untreated wastewater *Qualified sample or 2 hr mixed sample*

The Water Act also gives powers to the State Boards gives power of entry and inspection into the premises of the polluters' premises. Consent of the Board is required to 'establish or take any steps to establish any industry, operation or process or any treatment and disposal system or any extension, which is likely to discharge sewage or trade effluent. As it was found that the Boards were overburdened and underfunded, the Water Cess(Prevention and Control of Pollution)Act, 1977 was enacted. Even after revisions in 1992, the rates of water cess varied between Rs1.50 to Rs 5 for kilolitre for various uses. These rates are too low compared with the opportunity costs of water. The Pollution boards mostly restrict themselves to CAC(Command and Control) mechanism. So it is the duty of the state agencies to regulate the volume of the pollution load discharged by the existing industries. Thus before staring any industry there is a need to grant permission from the state control boards. In India there is an existence of CAC mechanism with a set of laws designed in such a way which fails to perform any proactive role. Thus citizens cannot claim environmental protection as a right and the state is not bound to protect the environment. Poor monitoring by the pollution boards fail to sustain the environmental standards. Moreover existence of a high degree of political interference make the situation even more worse.

4. CALCULATING THE EFFLUENT TAX:

As followed from various studies in regard of an effluent tax, Mehta et al(1993)shows that the pollution control authority could set its charges at between Rs 1.35(or 0.02 euro) to Rs 1.45(or 0.02 euro) per 100 grams BOD reduction at 1991-92 prices. Under the tax-subsidy scheme this can be the rate per 100 grams of extra BOD beyond Minimum National standards(MINAS). The effluent tax rate according to Murty et al(2000)calculated by substituting the actual(average)concentration of the effluent (for eg COD)in the wastewater by the standard for India which is 250 mg/l. In their study conducted in 1999 for the firms in the sample with excess COD concentration in their wastewater the tax liabilities calculated using the average tax rate is of Rs 0.32(or.005 euro)per 100 gm. The tax liability is based on excess concentration times the volume of wastewater and hence a

firm with only 84 mgl excess COD and 6.39 million kilolitres of wastewater volume per annum end up with a tax liability of 1.6 million rupees.

In case of Germany, the effluent charge is about approx. € 5 per inhabitant per year. The economic deterrent function is greater in individual cases, however, since fines need to be calculated into the total if a plant exceeds the so-called monitoring standards. If we consider the case of an effluent treatment plant who caters over 100,000 population the minimum standard is 75 mgl. Calculating the effluent tax rate for a particular effluent, say COD we have to note that wastewater quantities cannot be metered, nor is individual pollutant load measurable.

In this context we consider the case of Leipzig, Germany. Leipzig is the largest city of the state Saxony. From 2001-03, Saxony together with the federation as well as of the European union got financial grant for the building and development of WWTP and drains systems at a value of approx. 409 millions €. The total investment amounts to 670 millions €. According to KWL(Kommunale Wasserwerke Leipzig), Rosental STP covering a population of more than 100,000 has a COD limit as measured by the government authorities is 25mg/l.The treatment plant currently serves 530,000 people. The total cost incurred by the municipality is 5.5 billion euro so as to meet the EU standard.

The volume of wastewater is 25000mg/kl.Further since the costs is in terms of million of rupees following Murty et al 2000, it is desirable to have the tax per unit of effluent in terms of rupees the marginal cost is multiplied by a factor of 100,000.Finally,since the tax on annual wastewater quality is now in terms of rupees per milligram and the excess is likely to be in kg it is desirable to have a tax in terms of rupees per 100 grams which is $5.5/\ 55 \times 100 \times 1000$[11]

According Stadt Leipzig(June 2007),the total population of the city accounts to 507,360.So the tax per head accounts to be very high if the cost is to be recovered. So far as the case of India is concerned the population being much higher, the tax per head always remains of a much lower amount.Lanz (2004)when comparing German water prices with international figures, the water saving effect has to be taken into account. Total household costs for an individual or a family seem the most sensible standard for such comparisons while merely looking at the volumetric charge per cubic metre is not a realistic measure of water price. It should be made clear that political decisions beyond the control of the water operators also increase water rates. The value of mere price comparisons in the water sector must be regarded as limited, as long as differences in service quality(reliability, quality, repair times, water losses etc.)aren't simultaneously assessed. So the wastewater tax rate is much higher in case of Germany and thus has a positive incentive to abate pollution. If the tax rates are at least as high as the cost of effluent treatment there is a beneficial effect .Moreover there is also a secondary incentive in the form of lower tax rates in the next round for tax revisions.

5. POLICY IMPLICATIONS AND CONCLUSIONS:

Despite growing evidence that environmental degradation is an important socio-economic problem, governments in developing countries like India have been trying to stop it. Even where there is strong political will, governments may not be able to act effectively because of institutional deficiencies**.** Finding the right mix of policy instruments can help to ensure effective water pollution control. In developing countries, cost-effectiveness and administrative capacity are the two most important criteria for selection. Water pricing policies must take into consideration over-use and water pollution degradation. Although the experience in applying economic instruments remains limited, particularly in India, there is evidence that effluent and user charges have the potential for effective application by helping to pay for environmental improvement. They should be

accompanied by investment in wastewater treatment facilities and locally, by appropriate regulatory instruments as well as programs to persuade water users to change their polluting behaviour. In case of Germany as we can see that the water charge itself covers the water pollution charge. Both the more than proportional increase of the municipal wastewater charge and the high prospective demand for investment in sewage works has brought about the search for new organizational forms and different methods concerning water supply and wastewater disposal. But the wastewater policy instruments have been successful in reducing the wastewater pollution and enhancing technological progress in the country. Local governments do not necessarily have the right incentive to engage in procurement and contracting out efficiently . It appears that the higher the level of control that local governments have over the revenues the higher their incentive to organize local services efficiently. When local governments have soft budget constraint they have difficulty imposing a hard budget constraint on their service providers . As a result these service providers do not necessarily have cost minimization objectives – instead they seek to maximize employment, revenue or rent sharing.

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MBR TECHNOLOGY FOR WASTEWATER RECLAMATION IN RURAL AREAS

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Abstract: Membrane bioreactors (MBR) with submerged membrane modules have set the standard for the next generation of biological wastewater treatment plants as they offer two main advantages; a significantly improved effluent quality and a substantially smaller footprint. Besides the huge application in industrial wastewater treatment there are many municipal applications where MBR technology is being implemented. An example is the Bega Valley Sewerage Program (BWSP) in Australia which includes state-of-the-art MBR plants. Bega Valley is located on the southeastern coastline of New South Wales (about 450 km south of Sydney). It includes the towns of Cobargo, Wolumla, Kalaru and Candelo. The capacity of the valley's wastewater treatment systems was being stretched by urban growth and by seasonal population increases during the holiday period. Some unsewered villages in the valley were at risk for environmental and public health issues caused by discharge from septic tanks. In order to maintain Australia's Environment Protection Authority compliance and to enhance environmental outcomes, the Bega Valley Shire Council developed the Bega Valley Sewerage Program, which includes the installation of new pressurized sewage collection systems coupled with membrane bioreactor (MBR) treatment plants in the towns of Cobargo, Wolumla, Kalaru and Candelo. Each plant has a footprint of approximately 20 x 15 meters. The Cobargo MBR plant was commissioned in July 2006. The remaining three plants followed in 2007. The entire program is designed to keep capital and operating costs at a minimum and to produce a very high quality effluent for reuse. Reclaimed water from the MBR plants will be used in an irrigation scheme on public facilities such as the Cobargo Showground, Wolumla Recreation Reserve, Candelo Showground and the Sapphire Coast Turf Club, replacing potable water as the primary source. In future, reclaimed water may also be used for toilet flushing and to provide a vehicle wash down facility. This paper describes the design features of the four MBR plants as well as commissioning and operating results; furthermore, it explains how this design can be implemented in other rural areas around the world.

Key words: MBR, membranes, wastewater

1. INTRODUCTION

In an MBR, ultrafiltration membrane modules are submerged in the activated sludge to combine the biological step and the solid-liquid separation into a single process. Since the membrane acts as a barrier, this improves the effluent quality. Also, the membrane barrier eliminates the secondary clarifier and allows the activated sludge to be more concentrated. This reduces the volume requirement for biological tanks, thus saving space and construction costs. Overall, the MBR process reduces footprint significantly compared to the combination of wastewater treatment followed by sand filtration or ultrafiltration. The footprint savings due to the wastewater treatment plant alone can be as much as 50%, and there are additional footprint savings since the additional tertiary filtration steps are eliminated.

1.1 Example of MBR Technology

There aremany different configurations of MBR technology. One example that optimizes both membrane and module design is the PURON™ submerged hollow fiber UF module from the authors' company. The patented module is designed to avoid the clogging and sludging that is an

issue with some MBR module designs offered today. The module features hollow fiber membranes with a pore size of approximately 0.05 micron.

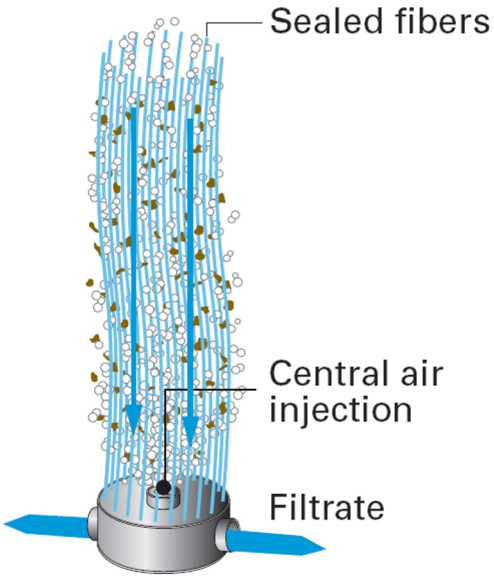


Figure 1. Membrane bundle

The lower ends of the membrane fibers are fixed in a header while the upper ends are individually sealed and are free to move laterally as shown in Figure 1. All solids and particulates remain on the outside of the fibers while permeate is sucked out of the inside of the fibers by means of a vacuum. The fibers are arranged in bundles and are submerged vertically into the activated sludge. To maintain the filtration rate of the membrane modules, air scouring is carried out at regular intervals. An air nozzle is integrated into the center of the bundles to apply the air for scouring purposes. The central arrangement of the air nozzles inside the membrane bundles reduces the energy consumption, since the air is injected at the place where the risk of sludging is highest. The module design ensures that even hairs and fibrous compounds will be removed reliably from the system, so that a coarse pre-screen can be used, thus improving capital and operating costs. A special feature of these membranes is their enormous mechanical strength. This mechanical strength is provided by a braid inside the membrane material. The individual fiber bundles are connected in rows. Several of the rows are mounted into a frame made of stainless steel to form a module. The free moving fibers combined with central aeration ensure stable filtration during plant operation, long membrane life, and low operating costs by reducing the need for energy, cleaning and maintenance.

2. CASE STUDY: BEAG VALLEY (AUSTRALIA)

Bega Valley Shire is located about 450 miles south of Sydney and is the largest local government in coastal NSW with a population of over 30,000. The villages of Cobargo, Wolumla, Kalaru and Candelo are located in the Bega Valley, and are described below. These villages were unsewered and were experiencing challenges related to septic tank effluent discharge.

Cobargo is a rural residential settlement in the north of the BegaValley Shire. The landscape is dominated by foothills and valleys on the coastal hinterland. Surrounding agricultural land is dominated by dairy and beef grazing. The village has a permanent population of around 400 people and also has a small holiday population during peak tourist periods. The village has a busy commercial area located on both sides of the highway. The previously used septic systems consisted

of absorption trenches, transpiration beds and aerated systems that were considered to be ineffective for the number of households serviced in the area and the soil type.

Wolumla is a small rural village to the south of Bega. Wolumla is located on rolling hills on the coastal hinterland and has been predominantly cleared for beef, dairy and sheep grazing. The township has a permanent population of around 380 people and also has a small holiday population during peak tourist periods. The township is predominantly low density urban housing with a small commercial area located on the main street including a hotel and general store. Wolumla relied on septic systems like Cobargo.

Candelo is a small rural residential settlement located on the coastal hinterland approximately 20 km southwest of Bega. The village has a permanent population of about 350 people. A small commercial area is located on the main street. Dairy and beef properties are located along the Candelo Creek floodplain and extend into the rolling hills surrounding the village. Candelo also relied on septic systems.

Kalaru is a small rural residential settlement located in the Bega Valley Shire on the far south coast of New South Wales. The township has a permanent population of about 300 people with a considerable increase in population during peak holiday periods. The township consists of mainly low density housing and contains a large caravan park, which caters for both holiday accommodation and permanent residences. The surrounding landscape consists of fairly flat topography on a floodplain and a number of swamps and wetlands. Surrounding agriculture is dominated by dairy and beef grazing with smaller market gardens and horse studs on rural residential properties closer to the town center. Kalaru also relied on septic systems that are considered to be ineffective due to the number of households serviced and soil type.

2.1 The Bega Valley Sewerage Programm

The Bega Valley Sewerage Program is an alliance formed between Tenix Alliance and the Bega Valley Shire Council to upgrade five existing sewage treatment plants and install Greenfield sewerage systems for five villages the region (Stone, 2005). The program includes updating the existing plants to Biological Nutrient Removal (BNR) facilities and installing new pressurized sewage collection systems coupled with membrane bioreactor (MBR) treatment plants in the towns of Cobargo, Wolumla, Kalaru and Candelo. The program also includes the operations and maintenance of the new and existing sewage treatment plants for ten years, with an option for Council to extend the agreement for a further five years. All collection systems will remain the responsibility of Council.

The entire program is designed to keep capital and operating costs at a minimum and to produce a very high quality effluent for reuse. The reclaimed water from the upgraded existing plants will be used for golf courses adjacent to the plants, and at a nearby dairy farm. Reclaimed water from the MBR plants will be used in an irrigation scheme on public facilities such as the Cobargo Showground, Wolumla Recreation Reserve, Candelo Showground and the Sapphire Coast Turf Club, replacing potable water as the primary source. In future, reclaimed water may also be used for toilet flushing and to provide a vehicle wash down facility.

2.1 The MBR plants

The four MBR systems are identical, resulting in significant cost savings for the Bega Valley Shire Council and the community. Each system is designed for 800 people equivalents or an annual average flow of 180 m^3/day. The maximum daily flow is 300m^3/day and the peak hourly flow is 360 m^3/day. Each plant includes pre-screening with 3 mm perforated plate screens followed by a biological treatment system with nitrification and denitrification. The UF membranes are submerged in the mixed liquor and used to separate the treated wastewater from the suspended solids. Figure 2 shows a three dimensional graphic of the plant.

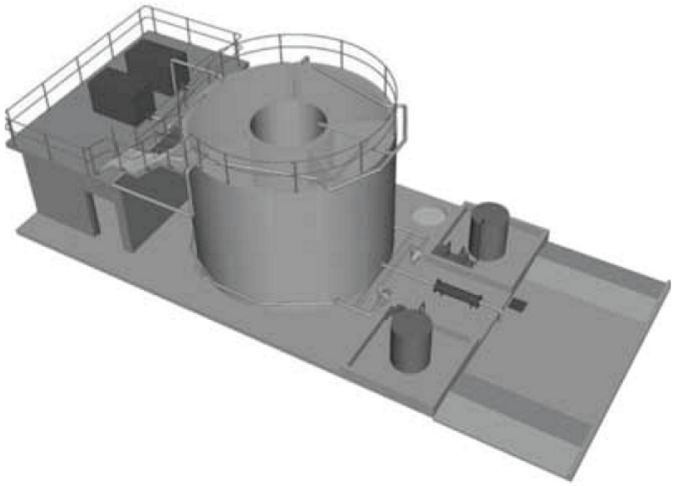


Figure 2. MBR treatment plant

Each plant has a footprint of approximately 20 x 15 meters. Each system includes 2 modules of 235 square metre. The membrane modules are type PSH 500 C2-8 and were supplied by Koch Membrane Systems. A photograph of one of the modules that was installed at Bega Valley is shown in Figure 3.



Figure 3. The MBR module

The Cobargo MBR plant was first to be commissioned in July 2006. The three other plants of Wolumla, Kalaru and Candelo were commissioned in 2007. The plant at Cobargo is shown in Figure 4.



Figure 4. The MBR plant at Cobargo

3. APPLICATION OF MBR IN RURAL AREAS

There are a several reasons why MBR technology can be an appropriate choice for rural areas. MBR systems provide excellent water quality without the risk of upsets that can occur with a secondary clarifier. So, an MBR system can operate with minimal supervision but still guarantee high water quality appropriate for reuse. Many MBR systems in remote areas are designed with a SCADA system allowing remote access to the data and then with an operator visiting the plant on a daily or weekly basis to perform checks, do routine maintenance and top up chemical tanks. MBR systems are modular, and so can be designed to easily accommodate additional future demand. MBR systems take up approximately half the footprint of a more conventional wastewater treatment plant, so are less visible to the community. If desired, an MBR system can be installed so that it completely blends into the landscape. For example:

- In a farming community the MBR could be installed inside a building that looked like a barn.
- In a park, the MBR building could be partially underground with landscaping over the building so that the building is not obvious other than at access points.
- In a small town, the MBR could be installed inside a building that looked like a typical house in the town.

Many rural areas derive income from tourism and so MBR technology helps by providing a system that is less intrusive to the environment, and by providing high quality water. Some rural communities have selected MBR technology to provide high quality water for discharge in order to maintain the quality of their streams, rivers and lakes. MBR effluent can also be recycled to provide additional water to the community to ensure sufficient water resources to cope with the additional water requirements of the tourist season. MBR effluent is reused today for irrigation of golf courses and parks and for toilet flushing in hotels. The Village of Cloudcroft, NM is currently constructing an MBR system that will be part of an indirect potable reuse scheme to provide additional potable water to the village, which has 750 permanent residents and a transient population that can exceed 2,000 people at the weekends and during the summer tourist season (Howe, 2007).

4. CONCLUSION

The MBR systems supplied as part of the Bega Valley Sewerage Program are helping several small communities to reduce the risk of environmental and public health issues related to discharge from septic systems. Additionally, the MBR effluent is high enough quality to be recycled, thus providing additional water resources to the communities. The first MBR system at Cobargo has been running for almost three years. MBR systems are appropriate for many rural areas because they require minimal supervision, can be designed to blend into the landscape, and provide high quality effluent for recycling.

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WASTEWATER RECYCLING WITH MBR AND RO

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Abstract: Increasing competition for limited water resources and continual pressure to reduce costs are huge challenges for the water treatment industry. The same factors are relevant for both municipal and industrial markets and for water and wastewater treatment. While water reuse is the obvious way of addressing the resource issue it places increased focus on producing water of the appropriate quality for reuse using an economically viable process. As a result the membrane industry is working hard to develop products and processes that achieve the required technical performance at an acceptable price for all sectors. Reverse osmosis (RO) has been used widely as part of a wastewater recycling scheme. The RO will remove most dissolved solids so the water can be recycled into the process or for secondary uses. However, there are some major challenges for RO systems operating on biologically treated wastewater. One challenge is plugging of the brine spacer due to high levels of suspended solids in the feed. Many RO systems today use ultrafiltration (UF) as pretreatment to remove suspended solids. The UF system does an excellent job of providing water with low suspended solids as measured by SDI (silt density index) to feed an RO. However, the UF system requires additional space and does not effectively reduce the amount of dissolved solids such as organics that are fed to an RO system. The UF system can also be susceptible to plant upsets from a conventional wastewater treatment plant. Instead of separating the biological treatment process and the ultrafiltration step, the latest wastewater treatment plants combine both processes in a membrane bioreactor (MBR). The membranes are integrated right into the aeration tank of the biological process. Advantages of this process are that a clarifier is no longer needed, and the effluent quality is considerably better, which improves RO performance. Additionally, the MBR process reduces footprint significantly compared to the combination of wastewater treatment followed by membrane filtration. The membrane bioreactor (MBR) process is being deployed around the world to an ever-increasing degree in a variety of applications. The scale of these applications is also growing. There are a number of local considerations to take into account such as climatic considerations, availability of land and potential for reuse, and MBR presents a viable solution to all of these difficulties. In addition, MBR produces an effluent suitable as a high quality feed water source for reverse osmosis treatment making the combination of the two techniques an attractive proposition. This presentation will describe the benefits of using MBR as pretreatment for RO systems. Data from MBR systems will be used to show how the pretreatment needs of RO are met.

Key words: MBR, membranes, RO

1. INTRODUCTION

Currently, biologically-treated municipal and industrial wastewater is typically discharged to the environment in most parts of the world. It is, however, a potential water resource from which high quality water for reuse can be produced. For example, at the City of Scottsdale, Arizona, Water Campus a 14 MGD (million gallons per day) water reuse plant has been in operation since 1999, and other even larger plants have recently been built around the world. Reuse applications include both non-potable applications such as irrigation and production of industrial process water, and indirect potable reuse including aquifer recharge. For many reuse applications, effluent from conventional wastewater treatment plants is frequently further treated by reverse osmosis (RO) to reduce the total dissolved solids in the effluent for water reuse applications.

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2. CHALLENGES OF RO SYSTEMS FOR WATER REUSE

There are some major challenges for RO systems operating on biologically treated wastewater. One challenge is plugging of the brine spacer due to high levels of suspended solids in the feed. RO membranes used for most water reuse applications today are typically spiral-wound elements, where membrane leaf units are rolled around a central permeate tube. Feed water flows into a brine spacer, typically made of vexar-type netting with a thickness of between 28 and 31 thousandths of an inch. If there is a high level of suspended solids in the feed water, the brine spacer can become plugged.

A second challenge is organic fouling, since many biologically treated wastewaters contain high levels of organics. The organics in the feed water are rejected by the RO membrane and are progressively concentrated as the water flows across the membranes. Towards the outlet of the RO system, the concentration of organics can lead to excessive fouling of the membrane.

A third challenge is biofouling. Biofouling requires both the presence of microorganisms and a food source. The organics in wastewater make an excellent food source for microorganisms. In addition, some treated wastewaters have high levels of bacteria present and so biogrowth occurs quickly in the RO system. Even when the RO feed water is disinfected, an RO system is not sterile and so biogrowth can still occur, albeit at a slower pace. In the past, RO systems treating wastewater used cellulose-acetate (CA) membranes, since these membranes can withstand some free chlorine. Feeding water with a chlorine residual minimized biogrowth. However, since CA membranes have limited pH stability and operate at high pressures, the trend today is to use thin film composite (TFC™) membranes. These membranes are not chlorine-tolerant and so strategies other than chlorine addition to avoid biofouling must be used when treating wastewater. These strategies may include use of an alternate disinfectant such as chloramine, or the minimization of organics and microorganisms in the feed by providing more extensive pretreatment.

Another issue with RO systems operating on some wastewaters is calcium phosphate scaling. Since municipal effluent can be relatively high in phosphate, RO systems operating at high water recoveries sometimes suffer from calcium phosphate scaling. This can be handled by operating at lower water recovery, utilizing acid or antiscalant to minimize scaling, or modifying the operating conditions of the wastewater treatment plant (WWTP) to reduce the amount of phosphate in the RO feed.

Issues such as plugging, fouling and scaling lead to operation of the RO system at higher feed pressures with increased power consumption, increased chemical costs for cleaning and shorter membrane life. Hence, to minimize overall water reuse system lifecycle costs, the pretreatment steps before the RO should be chosen carefully.

3. RO PRETREATMENT

There are many different pretreatment schemes and operational strategies. The optimal pretreatment scheme varies depending on many factors including cost of power, cost of chemicals, cost of labor, cost of land, wastewater source and the existing wastewater treatment system (if any).

3.1 Conventional Pretreatment

A typical “conventional pretreatment” scheme for effluent prior to RO might be primary treatment, biological treatment and solids-liquid separation such as secondary clarification. The crucial part of the process is the solids-liquid separation. With a conventional sedimentation process, there is often insufficient removal of bacteria and suspended solids. Tertiary treatment, such as sand filtration, may be used to improve the solids-liquid separation to provide higher quality

water to feed an RO system. A coagulant such as ferric chloride may be used in combination with the sand filtration to enhance solids and organics removal. Typical secondary effluent is approximately 20 ppm TSS (total suspended solids) and 20 ppm BOD (biological oxygen demand) and is relatively high in organics. Sand filtration will reduce these levels to a range where RO systems can be operated. However, upsets in the secondary clarifier can lead to effluent with higher levels of TSS and BOD causing problems for the RO, such as plugging of the brine spacer with suspended solids and fouling due to organics. Power consumption for RO systems with this type of pretreatment tends to be high, and membrane life is often quite short.

More extensive pretreatment schemes that have been more successful in protecting the RO membranes have included lime-softening to reduce solids and organics. However, this is an expensive choice in terms of operating costs and is not totally successful in preventing fouling of TFC RO membranes.

3.2 Ultrafiltration as RO Pretreatment

Many water reuse systems today use ultrafiltration (UF) as pretreatment to RO to remove suspended solids. Most of these systems use hollow fiber membranes. UF membranes do an excellent job of providing water with low suspended solids to feed an RO. The typical feedwater to UF membranes in a water reuse scheme is either secondary or tertiary effluent. Since the UF system is an extra treatment step, it requires additional footprint, and the operating costs incurred are in addition to the operating costs for the conventional WWTP. Also, UF does not effectively reduce the amount of dissolved solids such as organics or phosphates that are fed to an RO system. Hence, UF will not prevent organic fouling or calcium phosphate scaling in the RO system. The UF system can also be susceptible to upsets from a conventional WWTP. The UF system protects the RO system from the upsets, but these upsets can increase the operating costs for the UF system.

3.3 Membrane Bioreactor as RO Pretreatment

A further development over ultrafiltration is a membrane bioreactor (MBR). Instead of separating the biological treatment process and the ultrafiltration step, the latest wastewater treatment plants combine both processes in a membrane bioreactor (MBR). In an MBR, the ultrafiltration membrane modules are submerged in the activated sludge to combine the biological step and the solid-liquid separation into a single process. Since the membrane acts as a barrier, this improves the effluent quality. Also, the membrane barrier eliminates the secondary clarifier and allows the activated sludge to be more concentrated. This reduces the volume requirement for biological tanks, thus saving space and money. Overall, the MBR process reduces footprint significantly compared to the combination of wastewater treatment followed by sand filtration or ultrafiltration. The footprint savings due to the wastewater treatment plant alone can be as much as 50%, and there are additional footprint savings since the additional tertiary filtration steps are eliminated.

4. CASE STUDY: INDUSTRIAL WASTEWATER RECYCLING (AUSTRALIA)

An industrial company in Australia uses a large amount of water, and so wanted to reduce the volume of fresh water needed by recycling the wastewater. A biological wastewater treatment process was required to treat the wastewater to reduce the COD (Chemical Oxygen Demand) level in the wastewater and to provide a barrier to suspended solids. A reverse osmosis (RO) system was needed to further reduce organics and color and to reduce the Total Dissolved Solids to less than 400 mg/l. MBR technology was selected for the wastewater treatment process since it was able to provide the best quality water to feed the RO, and minimize overall operating costs. After evaluating bids from several suppliers, a general contractor was selected to provide the turnkey wastewater recycling system. The system use MBR modules from the authors' company, as well as an RO System from the authors' company. This system is the first commercial system in the world to use large-diameter RO elements for water reuse, although similar modules have been operated in a demonstration plant at the City of Scottsdale, AZ. One of the reasons the MBR modules were chosen is the successful operation of these modules treating industrial effluent in Europe.

The wastewater recycling plant consists of four stages; mechanical pretreatment, the biological process, the membrane filtration system, and the reverse osmosis system. The system was commissioned in March/April 2006. After coarse impurities from the industrial process have been removed via prescreens, the mechanical pretreatment stage, the wastewater is fed into the bioreactor. After sufficient retention time in the biological tanks, the treated water is fed into the membrane tank where it is separated from the activated sludge by the membrane modules, which are submerged in the tank. The concentrated activated sludge is recycled to the aeration tank.

The MBR effluent is the feed to the RO system. Large diameter 18-inch by 60-inch RO elements, with seven times the membrane area of 8-inch by 4-inch RO elements, were selected to minimize the number of components and O-ring for ease of operation and greater reliability.

The MBR system is designed to treat 400,000 gallons per day of wastewater. The RO system then recovers 75% of the MBR effluent as product water for reuse. during the summer tourist season (Howe, 2007).

5. COSTS OF ALTERNATE PRETREATMENT TECHNOLOGIES

Costs of the three alternate pretreatment technologies discussed above will vary depending on issues specific to a site. Hence, rather than attempting to estimate costs, the authors have opted to compare relative costs for the three processes.

5.1 Footprint

Tertiary filtration steps with sand filtration or UF require conventional wastewater treatment prior to the tertiary filtration step. Sand filtration typically requires more footprint than a UF system. Since the MBR system reduces the volume of the bioreactor tanks and eliminates the secondary clarifier, the footprint for an MBR process is much smaller than either of the other options.

5.2 Civil Works

Since MBR requires least tankage, the cost of civil works is lowest for MBR. For a large WWTP, sand filtration will probably be gravity sand filtration, so the cost of civil works is highest for this option.

5.3 Equipment Costs

The conventional WWTP plus sand filtration option uses the least equipment, so is lowest equipment cost. Both MBR and UF require membrane filtration systems. Equipment cost is of a similar order of magnitude for both systems – there is probably more variability depending on system configuration from supplier to supplier than between UF and MBR specifically. UF systems typically operate at slightly higher fluxes than MBR systems (although this depends on the quality of secondary effluent feeding the UF system), thus reducing the cost of the membrane component of the equipment for the UF system. Since the water quality feeding the RO system is worst with sand filtration as pretreatment, the RO system following this would typically be designed at a lower flux rate, thus increasing the number of membranes, vessels, skids and equipment cost for the RO system. The difference in design parameters selected for an RO following UF versus RO following MBR is less apparent, although as more MBR systems are installed as RO pretreatment, the RO designs may be further optimized to take advantage of the improved water quality after MBR.

5.4 Power Consumption

The major uses of energy in the treatment scheme are the energy related to the biological treatment process, the energy for the solids-liquid separation process and the energy for the RO process. Since the bioreactor in an MBR system operates at a high concentration of mixed liquor, the alpha factor for oxygen adsorption into the activated sludge is lower. Hence the power consumption for the MBR process for biological treatment is higher than for a conventional WWTP. The solids-liquid separation process for a secondary clarifier followed by sand filtration is lowest because this is mainly a gravity-driven process. The power consumption for the MBR and UF processes are similar, with variation depending on the operating trans-membrane pressure of a specific membrane configuration, the operating mode recommended by the supplier, and the requirements for air to scour the membranes. Power consumption for the RO process is expected to be highest for the conventional plant because the RO membranes are expected to suffer significant irreversible fouling, increasing the operating pressure of the process. There is insufficient data today to determine whether RO after MBR would operate at lower pressure due to less organic fouling in comparison with RO after UF.

5.5 Chemical Consumption

The conventional WWTP plus sand filtration uses the least chemicals for pretreatment, but the most chemicals to clean the RO system. Both the MBR process and UF process use some chemicals, but the chemical consumption of the MBR process is typically less than for the UF process. Both will also use chemicals to clean the RO system.

5.6 Membrane Replacement

The conventional process does not use membranes for pretreatment so there is no membrane replacement cost to factor in to the analysis. There will be a requirement to replace filter media

over time, but this is typically a fairly low cost. The cost of membranes for the UF and MBR processes are a similar order of magnitude. Since the conventional system uses more RO elements due to operating at lower fluxes, and is expected to experience more fouling and more frequent membrane replacement, the costs of RO membrane replacement for the conventional system will be higher than for the RO systems using UF or MBR pretreatment.

5.7 Susceptibility to Upsets

One cost factor that needs to be considered, although it can be very difficult to estimate a value at the design phase of a project, is the potential risk related to the susceptibility to upsets. While all three treatment schemes discussed are successfully used today for RO pretreatment, the risk of an upset varies. The costs of upsets can include downtime to clean the membranes, additional capital equipment to allow for additional cleanings, additional labor to monitor the process, additional power and chemical consumption due to operation with fouled membranes, and additional membrane replacement.

6. OPTIMIZING THE TREATMENT TRAIN

The optimal treatment train for a specific application requires consideration of the factors above. A primary consideration is frequently footprint. If space is limited, MBR may be the only choice of pretreatment that will fit in the available space. A secondary consideration is whether all the water from a wastewater treatment plant will be recycled, or whether just a sidestream will be recycled. If most of the wastewater will be discharged and the discharge requirements are not particularly stringent, then a conventional WWTP may be the best option for the whole plant, with a sidestream treated to tertiary standards with either sand filtration or UF. Another consideration is the final water quality requirement. For example, if water with very low ammonia levels after the RO is needed, MBR may be selected to minimize ammonia in the RO feed since ammonia is not well-rejected by RO at pH less than 9. Other considerations include costs of land, civil works, equipment, power, chemicals and labor, and the length of time considered when evaluating the options. High land and civil costs tend to favor MBR technology. For a large municipal plant looking at a 20 year net present value analysis, membrane filtration pretreatment is generally favored over conventional pretreatment due to the savings in RO membrane replacement and power. For an industrial company looking at a short payback, the preference for conventional or MBR will depend on the relative cost of civil works and land versus the equipment cost.

4. CONCLUSION

The use of RO for water reuse is growing rapidly. MBR technology can be used as pretreatment to RO for industrial and municipal effluents to provide high quality feed water to the RO, to minimize footprint and the cost of civil works, and to reduce the susceptibility of the treatment train to upsets. The optimal treatment train for a specific application depends on a number of factors, but we expect that, in the future, MBR technology will be considered for an increasing number of reuse applications.

A FRAMEWORK FOR WASTEWATER REUSE IN JORDAN: FROM PRESENT STATUS TO FUTURE POTENTIAL, APPLYING THE WASTEWATER REUSE INDEX (WRI) AS INDICATED

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Abstract: Pressing water scarcity in Jordan increasingly increased the demands to the use of marginal water for agriculture, which the treated wastewater will be most prominent among them. Water management studies reveal that no single source could fully solve the nation's water shortage and many integrated actions are needed to ensure water availability, suitability and sustainability. Among these options the developments of new water resources such as treated wastewater and brackish water have the potential to augment water supplies, thereby narrowing the gap between available freshwater and total demand. Agriculture is one of the primary essential activities in Jordan in general and in the Jordan Valley in particular. Treated waste water is could be a valuable source for irrigation in the agricultural sector, as an alternative for fresh water resources that are urgently needed by the rapidly growing urban populations. Currently there is an increasing percentage of irrigated areas using treated wastewater. With a fast growing population and expansion of the irrigated areas to meet the food demand the pressure on water resources in Jordan remains of imminent importance. Hence, the urgent call for an analysis of current and potential role of treated wastewater seems justified. Under the umbrella of the project on the Sustainable Management of Available Water Resources with Innovative Technologies (SMART) funded by the German Federal Ministry for Education and Research in Germany an investigation has been carried out in the Jordan Valley to estimate the current wastewater reuse quantities and the potential to increase its utility for agricultural production. In general, the reuse as percentage of total treatment is applied for national and international comparisons. Yet, this index is of limited use for policy decisions as it can not reflect potentialities of waste water use. Therefore, this study introduces a wastewater reuse index (WRI) that reflects the actual proportion of wastewater reused from the total generated wastewater. We found that the WRI in Jordan steadily increased from 30 per cent in 2004 to 38 in 2007. Efficient use of treated waste water requires the application of new technologies in Jordan like dwellings connected to the sewer system, decentralization of treatment plants to rural and urban settlements and prevention of high evaporation rates from stabilization ponds

Key words: Wastewater – Reuse – Water Reuse Index - Agriculture- Irrigation

1. INTRODUCTION

Jordan represents a typically water constrained economy that is daily confronted with decisions on its water use. With a fast growing population and an expanding agricultural sector, the demand for alternatives of fresh water resources remains imminent. The An most important strategy for the Jordanian government is to meet the water demand for agricultural sector produce is to rely more on treated wastewater.

The basic principle to use collected wastewater is their treatments that adjust the quality to serve the following end-users: irrigation, artificial recharge, potable water supply, toilet flushing, and industrial water supply. Reuse of wastewater has been practiced in many areas worldwide for thousands of years and is motivated by two strong economic incentives to reuse reclaimed wastewater (Abu-Madi 2004): to decrease the water scarcity in the region, and/or avoid the cost of the deterioration of water resources and the environment caused by untreated or partly treated wastewater.

Reducing the agricultural demand for fresh water in the region is not easy, but non-conventional water sources can assist in reducing the overall water quantities/percentage utilized by the agricultural sector. Wastewater is therefore an important additional source as it can be treated and reused not only by the agricultural sector for crop irrigation but also for landscape irrigation, groundwater recharge, and even some recreational purposes.(Aydın and Gür 2002; Monte 2007; Mekala, Davidson et al. 2008)

2. REUSE FOR AGRICULTURAL IRRIGATION

In both developed and developing countries, treated wastewater is used for agricultural irrigation both directly and indirectly (Westcot 1997; Hussain, Raschid et al. 2002; Carr, Blumenthal et al. 2004). In most cases reclaimed wastewater is used for unrestricted irrigation and can be used for all crops, even those consumed raw or uncooked. Most of the treated wastewater in Jordan is blended with freshwater from the King Talal Reservoir (KTR) and used downstream in the Jordan Valley for unrestricted irrigation (Shatanawi and Fayyad December, 1996).



Figure 1: AS-SAMRA WASTE STABILIZATION POND, (Picture taken by the author on Nov,2007)

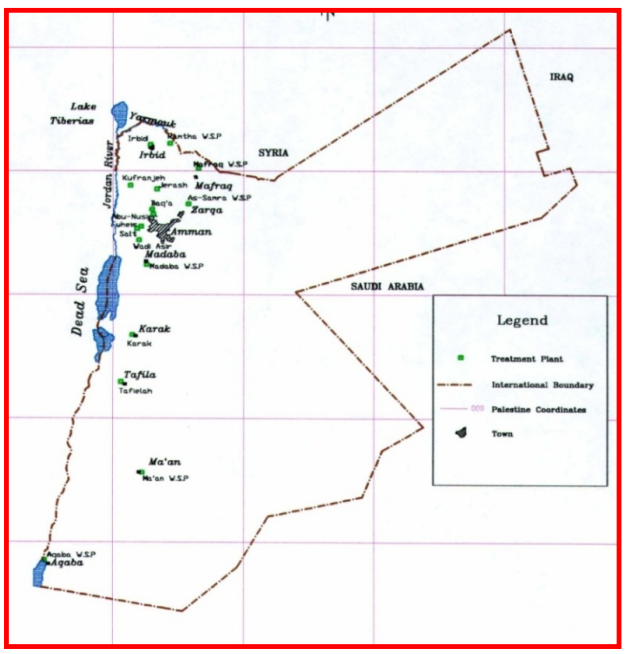


Figure 2: the location of WWTP in Jordan

Although the agricultural sector contributes a relatively small amount to Jordan's GDP (Central Bank of Jordan 2008), the government recognizes its economic and social importance, including a central role in food security and trade balance improvement.(AL-JALOUDY 2007). Moreover, with the high food prices and globally increasing demand for fodder and biofuel, the agricultural sector will only gain in importance in the coming years (Keyzer, M.D. Merbis et al. 2005).

Wastewater treatment and improvement are required as had been emphasized by (Oron, Campos et al. 1999) to minimize the health and environmental risks and to elevate the utilization of wastewater as a solution to water shortage problems. Two major drawbacks can be identified when no central facilities exist: lack of a collection system to accumulate all disposed wastes, and insufficient well-operated sewage treatment facilities.

3. MEASURING WASTEWATER REUSE

Water scarcity has made wastewater reuse more prominent in technical and policy literature as well as in national and international professional meetings. Several indicators are being used to quantify achievements and progress in wastewater reuse (Scott, Faruqui et al. 2004; Gabriel 2005).

In this study we argue that an appropriate indicator should take into account all wastewater production, both collected and uncollected, so as to provide a sufficient measure of potential. This is, for example, shown in Figure 3, where an index is used that divides wastewater reuse by wastewater treatment. There are two values for Jordan, the highest value, 90.1%, is calculated using the reported volume of treated wastewater. The lower value, 39.7%, is the ratio of wastewater reuse to the estimated generation of wastewater (assumed to be 80% of water withdrawals; Nayif Saider, MWI personal communication). As can be seen in the figure, using treatment in the denominator provides a misleadingly high estimate of the current reuse rate.

Given the potentially large gap between actual and apparent reuse, as shown in Figure 3, we argue that it is important to base measures of wastewater reuse on complete wastewater generation including on-site and low-cost means of reuse, in order to properly capture potential (FAO. and WHO. 2003). Currently available measures of reuse are based on collected urban wastewater and typically omit wastewater that does not pass through conventional collection and treatment. This limits the ability to estimate potential, and makes international comparison difficult. Therefore we propose to use the wastewater reuse index (WRI) that is defined as:

$$WRI = \frac{R}{G} \times 100, \quad 0 \leq WRI \leq 100$$

Where, R is total wastewater reused and G is total wastewater generation.

The WRI offers standard criteria enabling water resource managers and policy makers to put a figure on the gap between achievements at different junctures, and recognizes water saving efforts such as low water consumption and reducing losses.

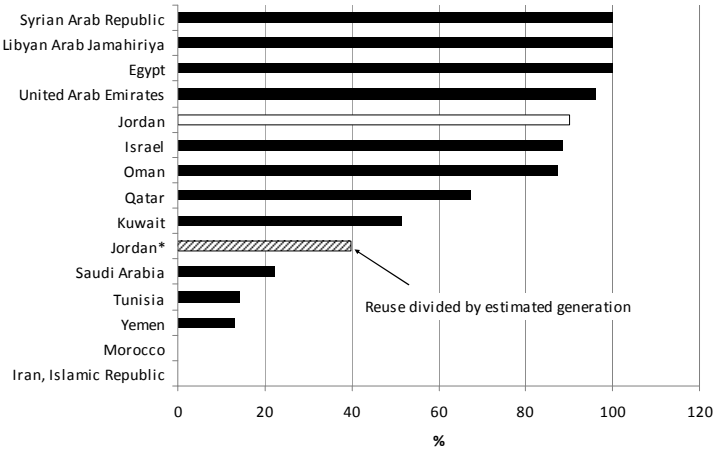


Figure 3: Wastewater reuse as percentage of treatment in the MENA region, 2003-2007 (Data from AQUASTAT, accessed 16 December 2008)

In the following, all quantities are listed in MCM/year. The relevant variables are as follows:
G = total wastewater generation (urban, rural, commercial, and industrial)
C = amount of wastewater collected (by sewage and on-site systems)
T = amount of wastewater treated (as effluent from WWTPs and appropriate on-site systems)
R = amount of wastewater reused
WRI = Wastewater Reuse Index (%)
x = collection as percentage of total production,
y = treatment as percentage of total collection,
z = reuse as percentage of total treatment.
The resulting estimates for the WRI are shown in Table 1.

Table 1: WRI data

Symbol	Values	2004	2005	2006	2007	2006 For JV
G	Total wastewater generation (MCM)	220.62	225.6	229.04	240.7	200.4
C	Amount of wastewater collected(MCM)	101.79	107.364	110.91	113.8	103.5
T	Amount of wastewater treated(MCM)	74.2	78.99	86.79	77.87	79.49
R	Amount of wastewater reused(MCM)	67	72	79.778	90.97	72.69
X	Collection as percentage of total production (%)	46.14%	47.59%	48.42%	47.29%	64%
Y	Treatment as percentage of total collection (%)	72.90%	73.57%	78.25%	68.41%	77%
Z	Reuse as percentage of total treatment (%)	90.30%	91.15%	91.92%	116.8%	91%
	Water Reuse Index (%)	30.40%	31.92%	34.83%	37.79%	45%

The WRI for all of Jordan in 2006 was 34.8 % while it was 45% at the Jordan Valley research area. It is clear that the WRI is quite low in Jordan, even though it is increasing slightly in subsequent years, as shown in Table 1. We observe that important efficiency gains can be obtained in the production of reused waste water as currently only 50 per cent of the total wastewater generated is being collected, of which 25 per cent is lost in the process. In general the following measures are recommended to increase the efficiency of the process:

(i) To increase the WRI more dwellings would need to be connected to the sewer system. Currently approximately 61% of dwellings in Jordan (MWI 2007) are connected to the sewer network system, while the rest of dwellings are disposed of in cesspools.

(ii) Decentralized WWT could help to increase reuse since many rural areas and some cities have no WWTP due to hilly terrain and lack of investment and there is some unaccounted loss from the network.

(iii) Finally, there is high evaporation from the stabilization pond and lagoons at the WWTP.

International comparison

Because of a paucity of international data, it is difficult to carry out a true international comparison for the indicator we are proposing. As is clear from the method used here, if sensible estimates of wastewater generation can be constructed, then it is possible to improve on the estimates of wastewater generation and use those for a preliminary comparison. The discussion in this section will use the measures that have been adopted in the resources cited.

In the Middle East there is a significant increase in water reuse to meet an ultimate objective of reusing 50 to 70 percent at least of the total wastewater volume (EPA 2004). In Israel during the drought year of 1990-91, agricultural allocations were severely cut and the proportion of wastewater reuse (which constituted a safe supply) rose to over 24 percent of total allocations (Shelef and Azov 1996). In normal years, Israel reuses more than 65% of its total domestic sewage production (Friedler 2001). Some nations evaluate reuse through the comparison of water reuse potential with total water use. In the United States, municipal water reuse accounted for 1.5% of total freshwater withdrawals in 2000. In Tunisia, recycled water accounted for 4.3% of available water resources in 1996. In Israel, it accounted for 15% of available water resources in the year 2000. The volume of treated wastewater compared to irrigation water resources is 7% in Tunisia, 8% in Jordan, 24% in Israel, and 32% in Kuwait. Approximately 10% of the treated effluent is being reused in Kuwait, 20-30% in Tunisia, 85% in Jordan, and 92% in Israel (G. Kamizoulis, A. Bahri et al. 1999).

Wastewater and Reuse in Jordan

In Jordan, the agricultural sector consumes around 64% of available water per year with one-third of this amount consumed in the Jordan Valley and about 50% reclaimed water (TWW). All in

all, agriculture consumes less than 20% of the total amount of freshwater available in the Jordan Valley (*Table 2*). Of the 22 WWTPs in Jordan only three receive TWW (Annex, *Table 4: the total effluent from WWTP and the actual amount of WWT reused in 2006* (Tab. 2) from septic tanks and not through the wastewater network. In 2006 the total effluent was 87 MCM, of which 91.9% was reused by agriculture after mixing it with fresh water during its inflow in the wadis (blended water). Jordan wants to increase the amount of TWW by improving the sewer network since TWW is vital to the water balance, then reallocate the fresh water used in agriculture to domestic use (data Ministry of Water and Irrigation 2006/ 2007).

The effluent from the 22 operating WWTP in Jordan is used primarily for agriculture purposes in the immediate vicinity, while surplus TWW flows along wadis where it either evaporates or is captured in water bodies like dams and ponds. It is known that farmers alongside the wadis are illegally pumping the effluent to irrigate their crops thwarting the intended destination and intended reuse of that water. However, the volumes of these illegal flows are unknown.

Table 2: Water supply for different sector and the actual demand, Ministry of Water and Irrigation data (2006/ 2007)

Demand Requirements	Ground Water	Surface water	Treated Wastewater	Total
	MCM			
Domestic	214.0007	79.75	0	293.751
Rular area	0.745	7		7.745
Industry & Remote Areas	44.894	3.527	0	48.421
Agriculture	244.81	176.366	90.97	512.146
Agriculture (High land)		77.46		77.46
Total Supply Demand	504.4497	344.103	90.97	939.523
Actual Demand				1512
Deficit				572.477

In the year 2006, the amount of water supplied was about 925 million cubic meters (MCM) while the actual demand was 1512 MCM, the municipal uses represented around 32 %, irrigation around 63 %, and industrial uses around 5% of the total consumption. According to MWI assumption "the wastewater (WW) generated is assumed to be 80% of the total volume" that mean (WWG = 230 MCM/year) with only approximately 111 MCM reaching the WWTP. Several reasons are cited for this loss the most important being that only approximately 61% of the total households are connected to the sewer system. This means that approximately 39 % of Jordanian households are not connected to the sewer network system. In other words, there is a considerable amount of the influent lost without recycling or reuse. Most of the non-connected households depend on cesspools, which can lead to groundwater contamination.

Irrigated area in the JV:

The Jordanian Highland consumes around 300 MCM of fresh ground water per year for irrigation, whereas total consumption per year in the Jordan Valley is 220 MCM of which only approximately 100 MCM is fresh water. That means that the Jordan Valley uses approximately 42 % of the total available fresh water for irrigation purposes while 58% of the total fresh water is consumed in the Highland. The Jordan Valley consumes almost 90 MCM freshwater and 90 MCM treated wastewater. The Middle Jordan Valley consumes nothing of the available fresh water totally depending on blended water.

The objective of this chapter is to quantify the gap between achievements in wastewater reuse and real consumptions that could provide a better insight into the problem of reuse efficiency, through using the wastewater reuse index (WRI) to give a clear picture of the quantities of influents and effluents, as well as the potential reuse of effluents presently.

Generally there are two types of WWTPs in Jordan; one is the centralized WWTP recognised as a governmental institution, while the other is the decentralized WWTP such as those installed at airports, universities and private companies. There are 22 governmental (87 MCM in 2006) and 23 private WWTPs (less than 3 MCM in 2006) (Figure 4).

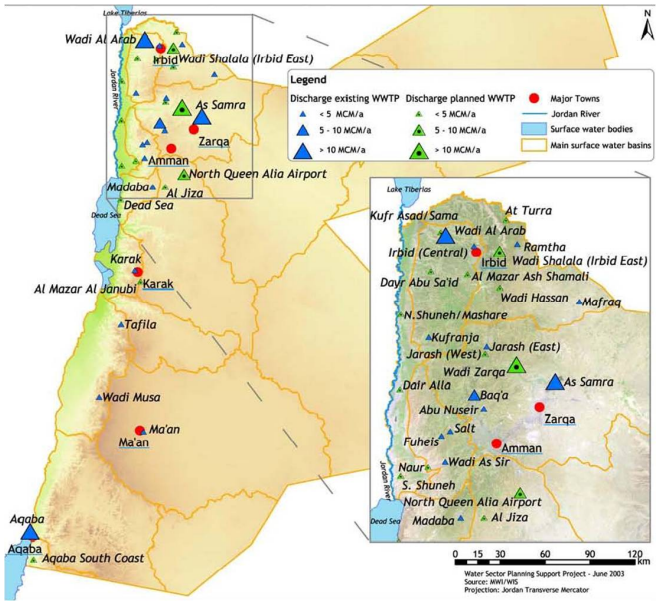


Figure 4: Centralized and decentralized WWTP in Jordan.

Governmental WWTPs receive sewage water from the public sewage network system that fall under the Water Authority of Jordan (WAJ) jurisdiction. Private WWTPs handle wastewater drained from local premises with no connection to the public network and are not part of the WAJ mandate.

At this paper only the centralized governmental WWTPs was considered due to the fact that it has a significant mandate to treat and reuse effluent compared to the effluents from the decentralized (private) WWTPs.

Sewage System in Jordan

There are 39% of households using private cesspools for discharging sewage water, which indicates a huge deviation in the share of dwellings connected to the public sewage network system among the governorates. The highest percent of connection (80 %) is in Amman governorate whereas the Karak governorates have the lowest percent (13 %). The Amman Governorate, which receives the biggest share of municipal potable water (more than 40 %), has almost 78 % of its dwellings connected to the public sewage network system (Annex, Table 5).

Influents and effluents of WWTP's

The total municipal water distribution for domestic use according to the data of MWI was approximately 286.3 MCM in 2006; as shown in (Annex, Table 7), where approximately 110.9 MCM was received as influents at the WWTPs. Meanwhile, the MWI assumes that 80 % (or 229 MCM) of domestic water will be generated as wastewater (229.043 MCM), this assumption by the ministry was made during mid of eighties. This means 48.42% of the generated wastewater from domestic uses does not reach WWPs due to the following reasons:

(i) Approximately 61% of dwellings (Annex, Table 6) in Jordan are connected to the sewer network system, while the remaining use cesspools;
(ii) Some municipal water is lost to illegal water abstraction;
(iii) Technical loses due to leakage in the water supply networks estimated around 25-40%, according to WAJ; and,
(iv) Percolation of cesspools to groundwater (40% of the dwellings drain their wastewater into cesspools).

It is estimated that the total amounts of wastewater subject to deep percolation to groundwater are significant, due to the above reasons. In the long term this could cause serious groundwater contamination.

As'samra WWTP receives a 73.8% of the total amount of influents and is the largest WWTP in Jordan and even of the Middle East; Al Zarqa and Amman are its largest suppliers (Table 7). The effluent of this WWTP is also the main supplier of reclaimed water for the King Talal Reservoir (KTR) that is used for the agricultural sector in the JV.

Effluents Outlet

The net effluents (Table 8) refer to the actual effluent passing through the WWTPs and equal the gross effluent of each WWTP minus the amount of water consumed by agriculture at the premises and vicinities of the WWTPs (licensed consumption).

There is a significant amount of effluents that come from Assamra, Baq'a, Wadi Arab and Irbid as can be seen from (Table 8), but only effluents coming from Assamra and Baq'a are used in irrigation. This means that approximately 6 MCM per year is not utilized; the effluent from North treatment plant like Irbid has poor quality where it is rejected to the Jordan River.

There are three dams (King Talal Reservoir, Shu'aeb, and Kafrain) that receive effluents from some WWTP. Since these effluents run through wadies and are mixed with fresh surface water it becomes blended water. All amounts of water stored in these dams are designated for agricultural use in the Jordan Valley. The total effluent water draining into these dams is around 58 MCM annually, of which 55 MCM is received by KTR alone. King Talal Reservoir (KTR) is considered a vital water source for agriculture sustainability in the middle Jordan Valley, since it is the principal recipient of effluents (53 MCM /year) mainly from As'samra, Baq'a, Jerash and Abu-Nusier WWTP's. In addition, many springs and stormwater runoff accumulate into KTR.

Farmers at the middle Jordan Valley depend entirely on the KTR as a source of irrigation water, and they do not receive any surface water from King Abdulla Canal (KAC). Furthermore, they have to share this limited resource with new development areas (DAs) recently connected to the KTR system (DA 19, 20, 21) (**Error! Reference source not found.**).

In addition, farmers alongside Wadi Al-Zarqa' use TWW for uncontrolled cultivation. No data and relevant information about the cultivated areas along wadi Zarqa, crop pattern, and the actual consumption of water amounts is presently available.

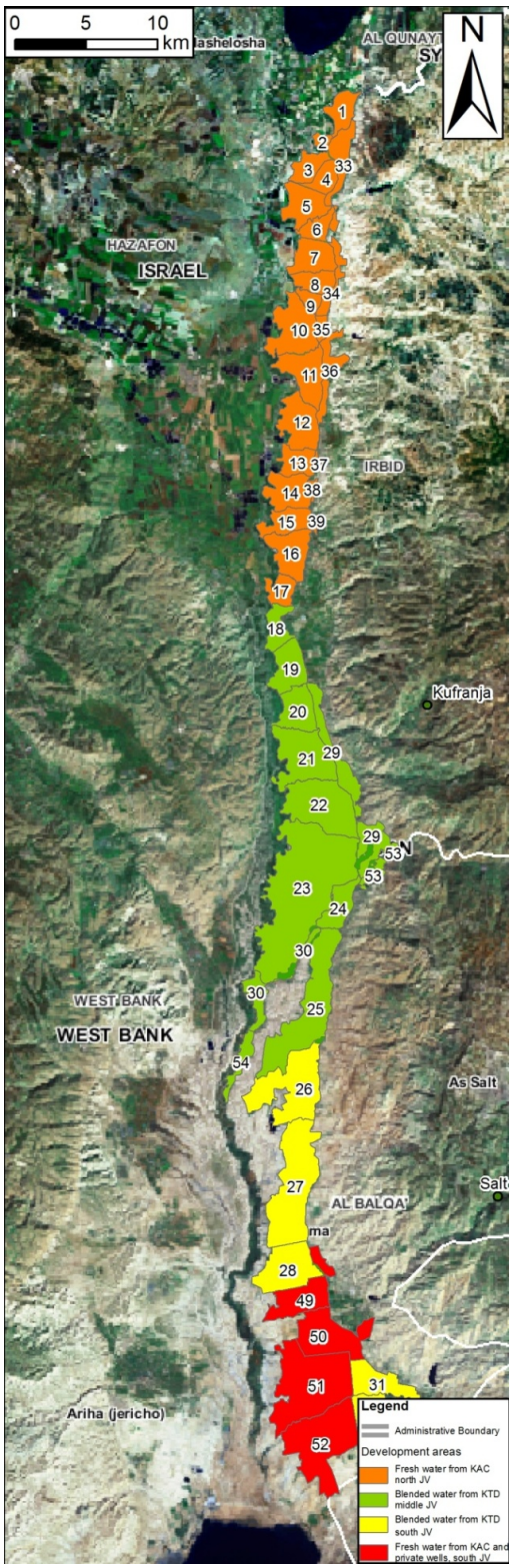


Figure 5: Development's area in JV

Table 3: Effluents of WWT Plants flowing into dams

Dam	WWT Plant feed dam	Effluent of WWT Plant (MCM/ **Year**)
KTR	As'samra	49.65
	Baq'a	3.59
Total		53.24
Shu'aeb	Salt	1.42
	Fuhais	0.61
Total		2.03
Kafrain	Wadi Al-Seer	1.05
Total effluents (MCM/ Year)		56.32

Wastewater Reuse

The collected wastewater must be treated to adjust its quality to the following end-users: irrigation, artificial recharge, potable water supply, toilet flushing, and industrial water supply. Reuse of wastewater has been practiced in many areas worldwide for thousands of years.

Reuse for agricultural irrigation

In most cases it is used for unrestricted irrigation. Reclaimed wastewater can be used for all crops even those consumed raw or uncooked. The reuse of WW in agriculture has been practiced worldwide in developed and development countries such as in Australia, Federal Republic of Germany, India, Mexico, Tunisia, China, Guatemala, India and United States of America (Buechler, Mekala et al. 2006).

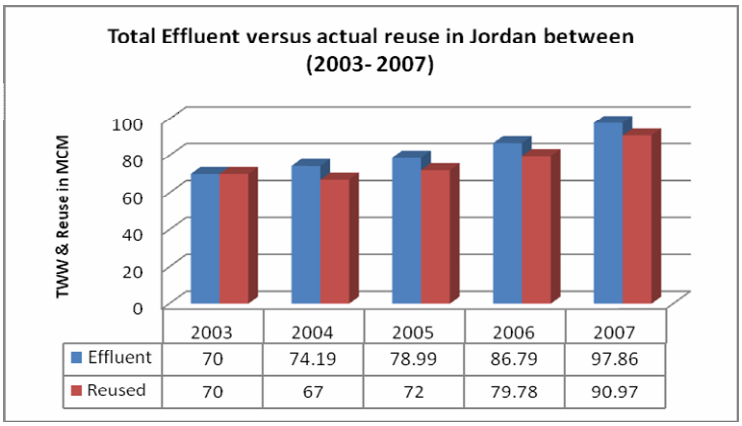


Figure 6: the Total Effluent versus actual reuse in Jordan between (2003- 2007) in MCM/year

Rural and suburban areas without large-scale wastewater collection and treatment systems commonly depend on septic systems. Wastewater is collected in a tank, and then distributed to the surrounding soil through perforated pipes. Septic systems work effectively only in very low density development. In higher-density developments, septic systems can severely impair groundwater quality.

Compared to conventional systems (the centralized WWT), an alternative collection systems such as adapting new technologies with less expensive and require less excavation. Reduced excavation means that less polluting sediment is disturbed into streams. This alternative sewer system tends to resist leakage better than conventional gravity collection systems.

This can lead to increase the amount of treated wastewater in Jordan through applying new technologies such as decentralized wastewater treatment system (On-site and/or cluster system used to collect, treat, and disperse or reclaim wastewater from a small community or service area.) or by

using composting toilet systems (a technology that uses a biological process to degrade human waste into a humus-like end product, sometimes called biological toilets, dry toilets and waterless toilets) contain and control the composting of excrement, toilet paper, carbon additive, and, optionally, food wastes. Unlike a septic system a composting toilet system relies on unsaturated conditions (material cannot be fully immersed in water), where aerobic bacteria and fungi break down wastes, just as they do in a yard waste composter. Sized and operated properly, a composting toilet breaks down waste to 10 to 30 percent of its original volume. The resulting end-product is a stable soil-like material called "humus", which legally in other countries such as United States must be either buried or removed by a licensed seepage hauler. In other countries, humus is used as a soil conditioner on edible crops. The primary objective of the composting toilet system is to contain, immobilize or destroy organisms that cause human disease (pathogens), thereby reducing the risk of human infection to acceptable levels without contaminating the immediate or distant environment and harming its inhabitants. A secondary objective is to transform the nutrients in human excrement into fully oxidized, stable plant-available forms that can be used as a soil conditioner for plants and trees. So that means it will be directly used at the area surrounding for house garden and agriculture.

Conclusion and Recommendation

In this paper we present a wastewater reuse index, defined as the total volume of wastewater reused divided by the total wastewater generation. We demonstrate with data from Jordan that using treated wastewater as a proxy for wastewater generation results in a misleadingly high value for the reuse index. Instead, we estimate wastewater generation as a proportion of water withdrawals, as described before, assuming that the ratio of wastewater generation to water withdrawals is 80% for Jordan.

We argue that the wastewater reuse index is a useful measure for estimating the potential for wastewater reuse in Jordan and that it can be used for policy guidance. Concerning its application in Jordan, the WRI indicates that there is considerable scope for expanding wastewater reuse, which prompted a more detailed look at the constraints on wastewater treatment and reuse in different areas in the study area within the Jordan Valley. The appropriate approach to increasing wastewater treatment depends on local conditions. In some cases the appropriate response is to increase the connection of dwelling to sewer system. In others, particularly in hilly or rural areas, a better option is to adopt technology such as composting toilet or decentralized wastewater treatment plant.

The decentralized approach to wastewater collection and treatment offers a new means of addressing wastewater management. Common to all of these options is on-site wastewater treatment by means of low-cost treatment systems, combined with direct use the treatment products (water, compost, and biogas). This approach can sustainably meet wastewater management requirements.

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Annexes:

Table 4: the total effluent from WWTP and the actual amount of WWT reused in 2006

WWTP's	Effluent MCM	Actual reuse	WWTP's	Effluent MCM	Actual reuse
AS-SAMRA	58.775	58.775	TAFILA	0.333	0.125
AQABA	4.921	4.921	WADI AL SEER	0.892	0.892
RAMTHA W.S.P	1.23	1.23	FUHIS	0.577	0.577
MAFRAQ W.S.P	0.636	0.636	WADI ARAB	3.516	0
MADABA W.S.P	1.493	1.493	WADI HASSAN	0.388	0.388
MA'AN W.S.P	0.862	0.862	WADI MOUSA	0.631	0.631
IRBID	2.235	0	TALL – MANTAH	0.091	0
JERASH	1.179	1.179	AKADEER	1.152	1.152
KUFRANJA	1.058	1.058	AL- LAJJOUN	0.232	0
ABU-NUSIER	0.808	0.08	TOTAL M.C.M (per year)	86.787	79.778

Table 5: Municipal water consumption for each governorate in Jordan (MWI , 2006)

Governorate	2004	2005	2006	Consumption 2006
	m^3/year			%
Amman	118,536,066	119,869,739	121,953,318	42.6
El Zarqa	37,687,744	38,447,913	40,324,912	14.08
IRBID	32,754,703	34,376,280	34,195,729	11.94
MAFRAQ	16,903,277	17,482,806	17,604,297	6.15
El Balqa	20,177,343	21,274,250	21,168,767	7.39
KARAK	11,030,435	11,023,232	11,466,121	4
TAFILA	3,070,173	3,496,374	3,705,131	1.29
MA'AN	7,068,872	7,107,804	7,452,019	2.6
JERASH	4,362,633	4,081,985	4,135,507	1.44
AL- LAJJOUN	3,101,994	3,649,708	3,643,033	1.27
MADABA	6,057,704	6,172,765	6,369,242	2.22
AQABA	15,020,565	15,012,503	14,285,763	4.99
Total	275,771,509	281,995,359	286,303,839	

Source: Ministry of water and Irrigation(MWI) , Water Authority of Jordan (WAJ)

Table 6:. Total Subscribers to water and sanitation system in Jordan, 2006.

WAJ Directorate	Total Subscribers to water	Total Subscribers to sanitation	Served % Per Directorate	Served % Per Governorate
Amman	409222	328230	80%	80%
Irbid	78840	41581	53%	
Al Kourah	11475	0	0%	
Al Ramth	11466	4917	43%	
Bani Kinanah	10726	2	0%	
Bani Obiead	15644	5093	33%	
North Ghor	10768	0	0%	37%
Al Zraqa	83483	57675	69%	
Al Risyafa	33398	25580	77%	71%
Maádaba	15352	7336	48%	
Theiban	4388	2	0%	37%
Al Salt	21662	11765	54%	
Ain Albasha	16671	14399	86%	
Al Fuhis	5215	4290	82%	
South Shouna	6082	0	0%	
Maadi	6207	0	0%	55%
Al Karak	16238	4340	27%	
Ghor Al safi	3856	0	0%	
Al Qaser	4978	0	0%	
South Mazar	9622	45	0%	13%
Al Tafila	11990	2359	20%	20%
Maán	8939	1900	21%	
Wadi Mousa	6330	2059	33%	
Al Shoubak	2078	0	0%	23%
Al Mafraq	25368	4915	19%	
North Badia	7712	0	0%	15%
Ajloun	15202	4739	31%	31%
Jarash	20882	7252	35%	35%
Al Aqaba	23275	16904	73%	73%
Total	897069	545383	61%	61%

Source: MWI /WAJ, 2008

Table 7: Influents and effluents of WWT Plants, 2006

WWTP	Influent		Effluent	
	MCM/Year	%	MCM/Year	%
AS-SAMRA W.S.P	81.84	73.79	58.78	67.72
AQABA MECH	2.46	2.22	2.64	3.04
AQABA W.S.P	2.27	2.05	2.28	2.63
RAMTHA W.S.P	1.28	1.15	1.23	1.42
MAFRAQ W.S.P	0.68	0.61	0.64	0.73
MADABA W.S.P	1.67	1.51	1.49	1.72
MA'AN W.S.P	0.97	0.87	0.86	0.99
IRBID	2.32	2.09	2.23	2.58
JERASH	1.21	1.09	1.18	1.36
KUFRANJA	1.24	1.11	1.06	1.22
ABU-NUSIER	0.84	0.76	0.81	0.93
SALT	1.58	1.42	1.42	1.64
BAQA'	4.01	3.61	3.81	4.39
KARAK	0.59	0.53	0.55	0.63
TAFILA	0.37	0.33	0.33	0.38
WADI AL SEER	0.99	0.89	0.89	1.03
FUHIS	0.61	0.55	0.58	0.67
WADI ARAB	3.64	3.28	3.52	4.05
WADI HASSAN	0.4	0.36	0.39	0.45
WADI MOUSA	0.61	0.55	0.63	0.73
TALL – MANTAH	0.1	0.09	0.09	0.1
AKADER	1.05	0.95	1.15	1.33
AL- LAJJOUN	0.18	0.17	0.23	0.27
TOTAL M.C.M (per year)	110.91		86.79	

Source: MWI, 2008

Table 8: Net effluent exiting WT Plants.

WWTP	Effluent	Water consumption before the outlet	Net effluent*
	(MCM/ Year)		
As'samra	69.65	20	49.65
Aqaba	4.2	4.2	0
Ramtha	1.18	1.18	0
Mafraq	0.6	0.6	0
Madaba	1.57	1.57	0
Ma'an	0.87	0.22	0.65
Irbid	2.25	0	2.25
Jerash	1.22	0	1.22
Kufranja	1.22	0.63	0.59
Abu-Nusier	0.83	0	0.83
Salt	1.47	0.05	1.42
Baq'a	4.08	0.49	3.59
Karak	0.55	0.64	0
Tafila	0.37	0.12	0.25
Wadi Al-Seer	1.12	0.07	1.05
Fuhais	0.61	0	0.61
Wadi Arab	3.7	0	3.7
Wadi Hassan	0.27	0.27	0
Wadi Musa	0.71	0.71	0
Tall Al-Mantah	0.1	0	0.1
Al-Akader	1.16	1.16	0
Al-Lajjoun	0.17	0	0.17
Total (MCM/ Year)	97.9	31.91	66.08

* Net effluent is the effluent minus water amounts consumed in premises and vicinities of WT Plants

SYSTEMS SOLUTIONS FOR DECENTRALIZED WASTEWATER MANAGEMENT IN JORDAN: IDENTIFICATION AND DESCRIPTION OF POTENTIAL REGIONS AND SITES

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Key words: SMART-Project, decentralized wastewater management, Jordan, rural sector, suburban sector, tourism sector, treatment potential.

RESULTS

This study was conduced within the framework of the SMART-project on "Sustainable Management of Available Water Resources with Innovative Technologies" in the Lower Jordan Rift Valley. Its key objective is to select regions that offer the highest potential for the future implementation of system solutions for decentralized wastewater treatment and reuse. Site identification was accomplished by applying a "filtration process" for the rural, suburban and tourism sector. The filtration included the following criteria and parameters:

- General potential and future demand for decentralized wastewater treatment and reuse solutions
- Connection degree to existing sewer systems and future development plans
- Population density and groundwater vulnerability in the investigation area
- Recommendations of local stakeholders, key professionals and high-level officials

The total population in the investigation area at the end of 2007 was calculated at 4.7 million. This presents approximately 82% of the total Jordanian population, although the surface of the investigation area just represents 6% of the total country. The rural population in the investigation area was calculated by adding the population of individual villages with less than 5000 inhabitants. The 411 communities identified had a population of 0.55 million in 2007 and count for 12% of the population in the investigation area. The mean population per village was calculated at 1111 inhabitants. The urban population in the investigation area was calculated at 4.1 million in 2007. The 105 urban communities count for 88% of the population in the investigation area.

In 2007, 21 treatment plants operated in Jordan, 13 of which are located in the investigation area. These plants have been mainly designed to treat the wastewater of cities and to serve suburban areas in the surrounding of these cities. The connection degree of the population in the investigation area to sewer systems was calculated at 69%. Assuming a connection degree of 5% in the rural sector, a connection degree to the existing central treatment plants for the urban sector in the investigation area was calculated at 78%.

The annual wastewater production available for decentralized wastewater treatment in the rural sector of the investigation area was estimated with nearly 16 million m3 at the end of 2006. The

corresponding overall potential of treatment capacity was estimated with 0.52 Million pe. The future need of wastewater treatment and reuse facilities of the rural sector was calculated with 0.48 Million m3 and an overall potential of new treatment capacity of nearly 15,000 pe per year. The reuse for treated wastewater in the Jordanian rural sector is well established. The main portion of reclaimed water is directly sold to farmers; a minor part is released to wadis and reused indirectly for irrigation. Therefore the local reuse of treated water shouldn't be a limiting factor in the rural sector where generally enough space is available for the cultivation of vegetables and crops in small units even close to the point of wastewater generation and treatment.

The overall potential for implementing decentralized wastewater treatment systems in the urban (suburban) sector was estimated at nearly 29 million m3 wastewater in 2007 with a treatment capacity of 0.9 million pe. The future need of wastewater treatment and reuse facilities of the urban sector was estimated at an annual wastewater production available for decentralized wastewater treatment of 3.6 Million m3 and a new annual treatment capacity of 0.11 Million pe. Considering seasonal fluctuations in water availability and need, the reuse of treated wastewater in the (sub-)urban areas might be restricted by space requirements. For this reason, the potential of artificial recharge as a space-saving alternative for the reuse of treated wastewater for irrigation or landscaping should be considered.

For the tourism sector, the calculation results in a total annual wastewater generation of approximately 3.2 million m3, which is equivalent to a total treatment capacity of approximately 26,000 pe. The future annual increase in wastewater production was estimated with approximately 0.11 million m3 per year. This is an equivalent of new annual treatment capacities of only 900 pe.

The available data were included in a GIS-database and three regions have been defined: Region 1 with existing central WWT, Region 2 with already planned central WWT and Region 3 with the highest potential for decentralized WWT. The regions and selected sites from the rural and suburban sector are described in more detail considering topography, population, location, housing, social structure, climate and farming and current wastewater situation.

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QUALITATIVE CHARACTERISTICS OF THREE WASTE STABILIZATION PONDS IN NORTH EAST GREECE

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Abstract: This paper examines three waste stabilization pond located in Vamvakofito, Skopos and Charopos, in the Prefecture of Serres. The parameters, data and assumptions produced during the systems' design are presented along with the validity of these assumptions and the modifications taking place during the construction and operation of the systems. Design assumptions are compared to real time data, while the systems' efficiency is evaluated. The systems were monitored for a year, and their results were checked. Following a yearlong sample collection, on a biweekly basis, at the systems' inflow and outflow and after a series of chemical, biological and microbiological analyses, the systems' efficiency for the removal of E.coli , T. coliforms, SS, BOD5, COD, N-total, P-total, inorganic elements and heavy metals is estimated. Temperature and weather conditions were also recorded. Therefore, seasonal variations are highlighted. The effluents suitability for irrigation purposes is examined, locating the systems' weaknesses. The assessment of the ponds' operation and their behaviour not only provides information for these specific systems but also offers guidelines for the design and construction of similar systems at the region in the future by taking into account the local conditions.

Key words: urban wastewater, waste stabilization pond, efficiency, heavy metals.

1. INTRODUCTION

Waste stabilization ponds are widely internationally used, for the treatment of both urban and industrial wastewater. However, in Greece their use is limited, representing just 8% of all urban wastewater treatment plants in the country, with 90% of these systems situated at regions of the North Greece, and 76% of which are located in the Prefecture of Serres (Gratziou et al, 2006a),

This paper presents the qualitative characteristics of three waste stabilization ponds located in Vamvakofito, Skopos and Charopo, in the Prefecture of Serres. The quality of influent and effluent wastewater, as well as the systems' efficiency to the removal of BOD_5, SS, N, P, S, Mg, Na, Zn, B, Br,Ca,Cl, Co, Fe, K, Li, Mo, U, Al, As, Ba, Cd, Cr, Cu, Mn, Ni, Pb, Sb, Sc, Sn, Sr, V, W, Y and Zr, are investigated, with sample measurements and analysis. Seasonal variations are detected and the effluents' suitability for reuse is examined. The performance of each system is different, depending on its construction and maintenance.

2. DESIGN REFERENCES AND CURRENT SITUATION

All the aforementioned systems receive only domestic wastewater and it is not predicted, for the future, their connection to some industry. They were all designed based on the same assumptions: daily flow rate 120 L/d, influent organic load BOD_5 45 g/p/d, influent suspended solids SS 60 gr/p/d, influent coliforms $5*10^6$/100 ml, detention time in the 1st pond 15-30 days. For the maturation ponds design, the detention time was chosen as 8 days, with the effluent's required characteristics as follows: BOD_5 30 mg/L and coliforms 5,000 /100 ml. The study propose the construction of a facultative pond with a depth of 2.40 m and 3 maturation ponds with a depth of

1.5 m and rocfilter before the final effluent for algae's filtration (Barboutidis J., 1980). The rockfilter consist in gravel of 15 mm and 75mm diameter and 1.5 m depth. During the construction, some modifications occurred, mainly to the number and the dimensions of the ponds (Table 1), resulting to a smaller retention time than it was predicted. It was not place geomembrane at the ponds' bottom and slopes, but just a compressed layer of clay. The wastewaters are collected through the existing combine (multi-flow) sewer systems and through the main pipes they are conducted to the ponds. The outflow takes place almost superficially. Generally, it was ascertained insufficient: construction, operation of the inflow's works, maintenance of the ponds and the observance of the safety rules.

Table 1. Modifications between Construction - Design in the studied system

n	*Vamvakofito*		*Skopos*		*Charopo*	
	Design	*truction*	*Design*	*truction*	*Design*	*truction*
s	M.M.M.RF	'.M.M. RF	F.M.M.M	F.M. RF	².M.M.M RF	M.M.M RF
m²	7,200	4,934	4,224	2,112	7,415	6,132
n	2.40-1.50	2.40-1.50	2.40-1.50	2.40-1.50	2.40-1.50	1.40-1.50
e sewerage	very 5 years	never	very 5 years	never	every 5 years	never

The minimal wastewater's temperature, that was recorded at the ponds during the measurements' period, was 5 ^{o}C, with a mean wastewater's temperature during winter 10 ^{o}C, while the design were made for a minimal wastewater's temperature (winter) 12 ^{o}C.

3. MATERIALS AND METHODS

According to the European Community Directive (EU Directive) for the qualitative control of the urban wastewater's treatment, for capability until and 50,000 e.p., they are required 12 samples annually (CEE Directive, 1991). The measurements that were implemented for the ponds' qualitative control were more. They were taken 32 samplings (instant samples) to the points at the inflow of the 1st pond and the outflow of the last pond, during the period from October 2006 until and September 2007. In order to take into account the potential seasonal variations during the months October, January, April and July, the sampling was taken every week. During the other months the collection of the samples were taken fortnightly. In order to take into account the major part of the weekly variations to the quality of the influent - effluent and to avoid the influence of the local parameters, such as the increase of the visitors during weekends, the samples were collected every time at a different day, according to the guides of Pearson, H.W. et als, 1987. The samples were received approximately the same morning period for every pond, while the temperature and other seasonal conditions were recorded (rainfall, overcast, sunlight). Samples were collected after strong rainfalls in order to appreciate the potential influences to the influents' and effluents' quality.

The parameters and the chemical elements that were measured are: suspended solids (SS), BOD_5, COD, pH, N-total, N-NH_4, P-total, faecal and total coliforms, and 72 more chemical elements like Ag, Al, As, Au, B, Ba, Be, Bi, Br, Ca, Cd, Ce, Cl, Co, Cr, Cs, Cu, Dy, Er, Eu, Fe, Ga, Gd, Ge, Hf, Hg, Ho, In, Ir, K, La, Li, Lu, Mg, Mn, Mo, Na, Nb, Nd, Ni, Os, P, Pb, Pd, Pr, Pt, Rb, Re, Rh, Ru, S, Sb, Sc, Se, Si, Sm, Sn, Sr, Ta, Tb, Te, Th, Ti, Tl, Tm, U, V, W, Y, Yb, Zn and Zr for the control and the estimation of the effluents' suitability for reuse.

The samples were placed into bottles of polyethylene of 1000 ml, and were transferred immediately to the laboratories, were a part of the analyses took place. For the analysis of the heavy metals and the other microelements, a quantity of 200 ml of every sample was filtered through membrane with opening of pores 0,45 μm and at the end it was sharpened with nitric acid so as to have a final concentration of 1% HNO_3. Afterwards, the samples were placed into bottles of polyethylene of 100 ml and conserved, until to be analyzed, into fridge at a temperature from 0 ^{o}C to 4 ^{o}C.

The chemical analyses of the elements that were previously mentioned were made only for the months that there was a weekly sampling and were implemented to the laboratories ACME Analytical Laboratories Ltd, at Canada with ICP/ES Inductively Coupled Plasma Emission Spectroscopy.

4. RESULTS AND DISCUSSION

The elements Ag, Au, Be, Bi, Cd, Dy, Er, Eu, Ga, Gd, Ge, Hf, Hg, Ho, In, Ir, La, Lu, Nb, Nd, Os, Pd, Pr, Pt, Re, Rh, Ru, Sm, Ta, Tb, Te, Th, Ti, Tl, Tm and Yb, didn't exceed the traceability limits and therefore they weren't valued. Table 2 presents the monthly mean concentrations of heavy metals, at the inflow and the outflow of the examined systems.

Table 2. Heavy Metals, Monthly Mean concentration in the studied system (N=4)

Element	*Oct-06*			*Jan-07*			*Apr-07*			*Jul-07*		
(μg /L)	*V*	*Sk*	*Ch*	*V*	*Sk*	*Ch*	*V*	*Sk*	*Ch*	*V*	*Sk*	*Ch*
						Inflow						
Al	43.00	22.00	48.00	34.00	20.00	26.00	141.0	334.0	105.0	66.00	331.0	369.0
As	2.30	1.40	1.90	1.80	1.60	1.50	2.10	2.20	2.00	2.10	4.60	2.30
Ba	42.93	57.01	17.84	59.84	41.68	17.63	63.90	47.41	14.70	65.84	20.43	50.90
Cr	1.90	6.60	2.00	1.50	2.80	1.60	2.50	3.00	1.10	1.40	1.30	3.20
Cu	1.70	1.50	3.60	3.70	2.20	1.70	1.90	13.80	7.00	2.40	9.50	15.80
Mn	2.61	18.61	14.63	21.44	14.18	19.64	55.76	35.55	33.82	60.30	38.01	38.07
Ni	0.40	<0.20	1.70	1.30	0.40	0.60	1.30	0.80	1.10	1.50	1.80	1.70
Pb	4.70	6.30	13.90	29.80	11.40	13.80	0.20	2.40	1.10	0.50	1.20	2.30
Sb	0.18	0.10	0.18	0.28	0.14	0.23	0.22	0.26	0.43	0.23	0.48	0.28
Sc	2.00	3.00	2.00	3.00	4.00	3.00	3.00	4.00	3.00	4.00	3.00	4.00
Sn	0.09	0.09	0.17	0.13	0.10	0.13	0.12	0.12	0.06	0.15	0.16	0.18
Sr	475.8	428.5	292.3	481.2	347.2	238.5	513.9	387.9	262.4	535.1	272.9	378.5
V	1.50	5.40	1.20	0.60	5.00	0.60	1.00	6.40	1.80	1.00	2.00	6.80
W	0.02	0.04	0.02	<0.02	0.14	0.07	<0.02	0.07	0.02	0.02	<0.02	0.06
Y	0.02	0.03	0.04	0.02	0.01	0.02	0.01	0.07	<0.01	0.01	0.01	0.08
Zn	49.10	25.20	113.9	42.40	28.40	29.6	20.30	142.7	56.60	29.60	154.6	82.00
Zr	0.05	0.04	0.06	0.21	0.05	0.05	0.07	0.13	0.07	0.06	0.05	0.11
						Outflow						
Al	24.00	41.00	23.00	32.00	27.00	325.0	81.00	23.00	9.00	167.0	132.0	313.0
As	3.50	1.70	2.00	2.10	1.80	2.2.	2.10	2.00	4.00	2.40	2.00	3.70
Ba	42.93	55.99	18.20	52.10	40.90	29.36	13.12	36.04	55.00	65.66	40.18	21.73
Cr	2.70	6.90	3.80	1.50	3.10	2.60	1.90	2.40	2.30	1.50	2.60	2.10
Cu	4.80	5.60	1.60	3.20	2.70	17.00	5.40	1.90	2.90	7.90	3.20	7.50
Mn	4.70	32.33	12.75	7.03	18.91	35.17	55.76	15.14	30.41	34.19	27.31	18.09
Ni	1.40	2.10	0.50	1.40	0.80	1.60	1.40	0.90	1.00	3.10	1.60	2.50
Pb	16.50	32.30	7.10	22.20	18.70	18.20	2.00	0.20	0.20	0.80	0.60	1.70
Sb	0.28	0.17	0.15	0.31	0.20	0.72	0.44	0.21	0.21	0.26	0.25	0.49
Sc	3.00	3.00	2.00	3.00	4.00	3.00	3.00	3.00	4.00	4.00	4.00	3.00
Sn	0.09	0.20	0.10	0.11	0.12	0.16	0.06	0.14	0.08	0.44	0.19	0.19
Sr	471.2	426.3	344.5	485.0	344.1	268.7	267.3	340.0	523.9	541.9	335.6	289.2
V	1.90	4.30	1.00	0.02	4.90	1.10	1.90	4.70	1.00	1.20	5.20	2.10
W	<0.02	0.04	0.02	0.03	0.23	0.06	<0.02	0.05	0.02	0.02	0.06	0.02
Y	0.04	0.04	0.02	0.02	0.02	0.08	0.01	0.02	0.02	0.02	0.01	0.01
Zn	112.6	209.0	48.00	38.90	45.40	231.0	176.1	32.30	23.5	59.90	52.40	222.0
Zr	0.04	0.05	0.03	0.06	0.05	0.26	0.09	0.05	0.05	0.05	0.04	0.08

V= Vamvakofito's stabilization ponds system, Sk= Skopos' system, Ch = Charop's system

Table 3 presents the monthly mean concentrations of the rest chemical elements which were analyzed with ICP/ES Inductively Plasma Emission Spectroscopy and gave traceable results at the inflow and the outflow.

In Table 4 and Table 5 the mean monthly Sodium adsorption ratio (SAR) and Electricity Conductibility (EC) rates, in the studied system, are respectively presented.

Table 3.Trace elements, Monthly Mean concentrations in the studied system (N=4)

Element	*Oct-06*			*Jan-07*			*Apr-07*			*Jul-07*		
(μg /L)	*V*	*Sk*	*Ch*	*V*	*Sk*	*Ch*	*V*	*Sk*	*Ch*	*V*	*Sk*	*Ch*
						Inflow						
B	264	123	206	234	117	171	273	85	200	291	100	233
Br	118	175	39	102	155	112	115	158	16	129	184	23
Ca	80634	103259	72745	86872	66395	64888	90827	75295	72141	87347	71607	66965
Ce	0.03	0.02	0.04	0.03	0.01	0.01	0.01	0.07	0.01	0.01	0.09	0.03
Cl	39000	42000	19000	58000	48000	26000	52000	39000	22000	54000	39000	23000
Co	0.20	0.05	0.09	0.30	0.11	0.10	0.28	0.15	0.18	0.32	0.20	0.23
Cs	0.02	0.02	0.10	0.03	0.03	0.08	0.03	0.02	0.07	0.03	0.02	0.06
Fe	10.00	16.53	38.00	47.00	16	42.00	24.00	67.00	41.00	18.00	45.00	25.00
K	12293	3409	7296	14237	3611	7776	13190	3921	7464	13977	4554	8850
Li	15.60	4.50	15.10	16.60	6.70	13.50	17.20	6.80	14.60	17.70	6.90	15.70
Mg	20486	22228	7666	21452	15885	6854	21920	15020	7503	22730	15624	7817
Mo	1.90	1.30	2.20	1.10	1.10	0.70	2.60	1.10	1.00	2.80	1.20	1.10
Na	77198	63187	47068	88130	63051	42356	88398	61829	47490	86410	61635	45946
Rb	8.42	2.82	7.27	9.15	3.18	7.13	9.56	3.33	7.38	10.01	3.53	7.89
Se	0.60	1.20	<0.05	0.50	1.00	<0.50	0.70	1.20	<0.50	0.60	1.10	<0.50
Si	12326	14344	10675	10929	11659	8570	13069	13678	10304	14497	15367	12177
U	14.38	5.86	4.31	15.47	3.99	3.77	15.90	1.39	1.15	16.77	1.40	1.21
						Outflow						
B	435	170	228	255	109	215	194	112	277	303	125	222
Br	138	174	36	97	153	24	27	150	112	131	178	32
Ca	74429	104911	77015	82155	65636	75679	72253	67126	89655	85067	63637	67996
Ce	0.05	0.05	0.02	0.03	0.03	0.10	0.01	0.02	0.01	0.04	0.03	0.04
Cl	57000	43000	21000	59000	50000	27000	22000	40000	54000	57000	43000	24000
Co	0.24	0.20	0.09	0.31	0.16	0.46	0.13	0.19	0.24	0.32	0.24	0.19
Cs	0.02	0.02	0.16	0.02	0.03	0.09	0.05	0.02	0.02	0.01	0.02	0.04
Fe	11	55	12.00	53	37	67.00	27	28	22.00	32	35	46.00
K	20757	4784	7628	15445	4413	8635	7464	5155	7439	15899	5641	8622
Li	18.90	5.10	16.70	17.60	6.80	14.30	13.30	6.20	17.40	17.10	6.60	15.30
Mg	23701	22779	7845	21960	15739	7263	7442	14427	22559	23130	15067	7718
Mo	1.90	5.20	0.90	1.30	1.00	4.00	0.90	1.60	1.10	1.10	1.70	1.00
Na	77500	74562	47833	85373	67994	45582	46588	65647	89125	88906	65317	45842
Rb	11.54	4.18	8.27	8.24	4.17	7.78	7.16	4.34	9.36	9.88	4.58	7.94
Se	0.60	1.20	<0.05	<0.50	0.90	<0.50	<0.50	1.00	<0.50	0.50	1.00	<0.50
Si	15012	13993	10834	10851	11933	9095	9678	12401	12967	14468	14152	11606
U	12.45	6.40	4.56	15.94	4.05	1.58	1.18	3.82	1.16	15.76	4.06	1.28

As it results from the analysis of the data and the results of the chemical analyses, the concentrations of the heavy metals and the rest microelements which were counted were, during the whole year, much lower than the limits set by the Greek and the international specifications for the wastewater, even for the sensitive receivers as well as for the irrigation water, (Prescot, M.B. 1992), (Antonopoulos, B. and G.Tsakiris, 2006), (Agelakis, A.N, et als, 2004).

The majority of the elements present a relevant stability at their concentrations during the year. The elements Aluminum (Al), Copper (Cu), Manganese (Mn), Lead (Pb) and Zinc (Zn) constitute an exception. Al, Cu and Mn presented a significant increase of their concentrations to all of the

three systems during the spring and summer months April and July (inflow samples). Certainly, as it was mentioned before, the outflow's concentrations remained under the limits. Al can present toxicity when its values at the irrigation waters exceed the 5mg/L, while Cu and Mn shouldn't exceed the 0.20 μg/L. The systems' efficiency for those elements is not stable at all. However, temperature's increase seemed to have a positive effect to their efficiency, catching up 93%, 86% and 57% respectively. Zn presented increased values at the inflow's samples during the spring and summer months April and July at the Skopos' system, while Charopo's system had an increased inflow concentration on October. The effluent concentrations were particularly increased during January and July at the Charopo's ponds, during October the Skopos' ponds and during April and October at the Vamvakofito's ponds. However, the concentrations remained even in this case much lower than the limits, which for irrigation are prescribed to 2mg/L. The systems' efficiency for the Zn's removal didn't show stability, neither any connection to the temperature. Pb presented a significant reduction of its concentration (inflow samples) with the temperature's increase.

The elements which presented the higher concentrations were Ca Mg, K, Na, S, P, Si and Cl, reflecting their content at the natural waters (Ca, Mg, K, Si), as well as their high contents (Na, Cl, P, S) at the domestic wastewater due to the use of detergents and liquids of atomic and domestic cleanness, etc.

Table 4. Mean Sodium adsorption ratio (SAR) in the studied system (N=4)

System	*Oct-06*		*Jan-07*		*Apr-07*		*Jul-07*	
	Inflows	*Outflows*	*Inflows*	*Outflows*	*Inflows*	*Outflows*	*Inflows*	*Outflows*
Vamvakofito	1.98	2.84	2.20	2.15	2.15	1.39	2.13	2.20
Skopos	1.47	0.91	1.80	1.95	1.70	1.91	1.72	1.90
Charopo	1.40	1.38	1.33	1.33	1.42	1.38	1.42	1.40

The SAR values range the ponds' water in class I, where the danger of sodium is small, and consequently, from this point of view, they are suitable for irrigation. Similarly the rate of EC which varied between 950 and 1002 (Table 5) and pH, which varied from 7.15 to 8.11 (Table 7).

Table 5. Electricity Conductibility (EC) μs/cm, in the studied system (N=4)

System	*Oct-06*		*Jan-07*		*Apr-07*		*Jul-07*	
	Inflows	*Outflows*	*Inflows*	*Outflows*	*Inflows*	*Outflows*	*Inflows*	*Outflows*
Vamvakofito	1554	961	1105	953	1465	960	1550	989
Skopos	1249	997	1540	1002	1100	967	1552	992
Charopo	1340	1000	1345	973	1454	955	1449	950

Table 6 presents the mean monthly concentrations of the pollutants SS, BOD_5, COD, N-total, N-NH_4,, N-NO_3, P-total, FC and TC, at the inflow and the outflow of the systems under study.

Table 6: *SS, BOD_5, COD, N-total, N-NH_4,, N-NO_3, , P-total, FC and TC, Mean Monthly concentrations, (N=4)*

Element (mg/L)	*Oct-06*			*Jan-07*			*Apr-07*			*Jul-07*		
	V	*Sk*	*Ch*	*V*	*Sk*	*Ch*	*V*	*Sk*	*Ch*	*V*	*Sk*	*Ch*
						Inflow						
SS	48.7	21.5	49.2	45.3	26.8	24.2	78.4	55.1	221.0	84.8	71.9	71.9
BOD_5	320.0	128.0	102.0	316.7	158.2	131.3	362.5	188.0	360.0	237.5	190.6	191.6
COD	471.5	132.0	108.0	471.7	169.2	139.0	537.0	222.5	385.0	470.5	218.8	228.8
N-total	28.5	21.5	40.6	25.3	49.0	23.2	26.0	12.6	106.2	25.0	24.20	24.2
N-NH_4	16.2	17.2	32.2	17.2	40.7	18.8	19.7	10.5	86.8	18.0	19.3	19.3
P-total	6.5	2.8	3.4	7.7	5.9	5.4	7.1	12.80	12.8	7.3	5.6	5.6
FC 10^3 (cfu/100 mL)	105.7	36.2	24.8	143.7	13.9	4.6	88.7	20.6	14.0	102.5	35.9	10.9

$TC10^3$ (cfu/100 mL)	183.8	71.2	48.9	263.0	63.9	28.5	160.5	66.7	24.2	160.5	74.6	24.6

	Oct-06			*Jan-07*			*Apr-07*			*Jul-07*		
	V	*Sk*	*Ch*	*V*	*Sk*	*Ch*	*V*	*Sk*	*Ch*	*V*	*Sk*	*Ch*
						Outflow						
SS	27.9	3.8	36.9	28.2	18.1	18.5	18.4	33.4	56.7	25.5	25.7	25.7
BOD_5	80.0	54.9	51.0	88.3	65.0	71.2	86.0	58.0	85.0	86.5	59.0	59.0
COD	83.0	59.0	56.6	91.0	97.5	77.5	87.0	85.0	89.0	88.5	113.4	113.4
N-total	6.1	2.6	10.2	11.4	6.9	14.2	9.4	4.6	2.1	9.3	4.8	3.8
$N\text{-}NH_4$	0.9	0.2	7.9	3.5	2.3	11.3	1.3	1.3	1.8	2.7	1.8	1.7
$N\text{-}NO_3$	1.4	1.8	0.8	1.4	3.4	2.1	1.5	2.4	0.6	1.5	2.0	1.0
P-total	2.8	2.2	2.6	4.6	3.7	4.0	3.7	7.5	3.2	3.3	2.6	2.6
FC 10^3 (cfu/100 mL)	3.0	1.83	5.7	3.4	1.5	1.4	2.1	3.2	3.2	2.6	4.7	2.5
$TC10^3$ (cfu/100 mL)	62.1	31.8	10.3	66.2	20.3	9.4	23.4	25.3	11.2	65.0	22.2	5.2

The ration COD/BOD at the inflow was approximately 1.48 at Vamvakofito and 1.10 at the other systems.

Table 7: Environmental Conditions in the studied system (N=4)

Environmental Conditions	*Region*	*2006*			*2007*								
		Oct	*Nov*	*Dec*	*Jan*	*Feb*	*Mar*	*Apr*	*May*	*June*	*July*	*Aug*	*Sept*
Air Temperature °C	*V*	12 °	9 °	7 °	0 °	8 °	7 °	14 °	19 °	20 °	22 °	25 °	16 °
	Sk	12 °	10 °	3 °	-4 °	4 °	5 °	10 °	12 °	16 °	20 °	19 °	14 °
	Ch	10 °	10 °	3 °	-2 °	4 °	5 °	6 °	12 °	16 °	20 °	19 °	14 °
Water Temperature °C	*V*	16 °	16 °	14 °	9 °	12 °	14°	17 °	22 °	22 °	24 °	24 °	22 °
	Sk	16 °	16 °	13 °	5 °	12 °	13 °	17 °	18 °	22 °	24 °	22 °	20 °
	Ch	15 °	14 °	11 °	7 °	10 °	12 °	12 °	18 °	22 °	24 °	22 °	20 °
*Weather * Conditions*	*V*	S/C	S/R	C	S/R	S/C	S/C	S/C	S	S	S	S	S
	Sk	S	R	C	S/R	S	S	S/R	C/S	S	S	S	S
	Ch	S	S	C	S/C	S	S	S/R	C/S	S	S	S	S
pH inflows	*V*	7.42	7.45	7.20	7.13	7.05	7.32	7.40	7.31	7.44	7.38	7.57	7.28
	Sk	7.71	7.43	7.51	7.34	7.16	7.51	7.62	7.06	7.85	7.56	8.1	7.55
	Ch	7.31	7.12	6.96	7.08	7.33	7.10	8.78	7.06	7.85	7.56	8.0	7.54
pH outflows	*V*	7.72	7.86	7.61	7.38	7.28	7.65	7.42	7.89	7.40	7.91	7.78	7.34
	Sk	7.99	7.78	7.65	7.66	7.32	7.38	7.49	7.54	7.92	7.68	7.89	7.75
	Ch	7.20	7.15	7.15	7.25	7.47	7.20	8.11	7.54	7.93	7.69	7.89	7.75

** S= Sunlight, C = cloudiness, R = Rain*

It should be noted that sampling was instant, thus it is possible that some measurements don't represent the actual picture of the installation. Therefore, in order to have a more accurate estimation of the system's efficiency, they should be taken into account the means values of the whole studying period, as presented in Table 8.

After strong rainfalls it was observed a significant increase of the load of SS and COD at the inflow of the ponds, while the nitrogen's load seemed to be lower up to 50%.

The systems' efficiency to the removal of the pollutants *SS, BOD_5, COD, N-T, $N\text{-}NH_4$, $N\text{-}NO_3$, P-T, FC* and *TC* presented significant variations (Table 8). The temperature's increase had a positive influence on phosphorus' removal to all the systems. The relatively small efficiency (47%-57%) and the significant variation at the solids' removal is possibly due to the fact that the sludge hadn't been removed throughout the operating period of the systems, despite the fact that, according to the study, the sludge should be removed every 5 years. The efficiency of similar systems at Arad-Israel and at Kokkinohoma-Kavala for solids' removal was 94% and 91% respectively (Gratziou M. et al, 2006b). However, the results of the SS's concentrations at the effluents are

considered satisfactory. SS's concentration at the outflow didn't exceed the 56.7 mg/lt, with a mean value not higher than 34.45 mg/lt (Charopo). The mean concentration for all the ponds is 25.56 mg/lt. Y. Recault, reports that the equivalent mean concentration for the Waste Stabilisation Ponds at Bretagne was 50 mg/lt (Y. Recault et al 2004), while the results from 178 similar systems studied at France were 60 mg/lt (Y. Recault et al, 1995). The SS's concentration at 80% of the effluent's samples was lower than the limit for irrigation which is 35 mg/L (Angelakis A., 2004). Systems' efficiency at BOD_5's removal (Table 8), is not satisfactory. This fact is mainly due to the insufficient implementation of the study. The ponds that were constructed were fewer and their total area was smaller than the predicted according to the study. That is to say, there was a reduction of the required area approximately 30%. Considering the fact that the design was made for a minimum temperature higher than the actual (12 ^{o}C αντί 9 ^{o}C, 5 ^{o}C, 7 ^{o}C), then the ponds' area is much smaller than it was really required. The efficiency of a similar system which operates at Kokkinohoma (Kavala) is 99.5% with an effluent's concentration 25 mg/lt, and another system's efficiency at Arad-Israel is 98% (Gratziou M. et al, 2006b). The efficiency at COD's is considered satisfactory. The maximum value of the unfiltered COD at the outflow was 123 mg/lt, with a permissible limit 125 mg/lt. The mean value for the three systems was 87.47 mg/lt. The mean equivalent efficiency of 178 similar systems at France was 99 mg/lt (Y. Recault et al, 1995). Nitrogen's and phosphorus removal (65-79% and 38-50% respectively) is noticeably satisfactory. N-total's maximum concentration at the ponds' effluents was 14.2 mg/lt, while the limit for most cultivations is 30 mg/L. Bibliographically it is reported P-total's removal 30-45%, while at the Mediterranean's region the annual phosphorus' removal can achieve 50%, and the summer months even 63 (Picot et al., 2004). At the studied systems the mean removal during the summer months was 54.5%, whereas the maximum value achieved 75%. Systems' efficiency at FC's and TC's removal is not considered to be satisfactory. This is due to the incorrect design and even more to the ponds' insufficient construction and to the inexistence of maintenance. The emissions were much higher than the limits for restricted irrigation.

Table 8:SS, BOD_5,, COD, N-T, $N\text{-}NH_4$,, $N\text{-}NO_3$, P-T, FC and TC, Mean annual concentrations and removals (N=32)

System	*Vamvakofito*				*Skopos*				*Charopo*			
mg/L	*Mean*	*STD*	*Max*	*Removal*	*Mean*	*STD*	*Max*	*Removal*	*Mean*	*STD*	*Max*	*Removal*
SS	25.00	4.56	28.30	57% (38-93)	20.25	12.62	33.50	55% (32-82)	34.45	16.65	56.70	47% (24-74)
BOD_5	85.20	3.60	88.50	73% (70-77)	59.23	4.23	65.10	55% (39-64)	66.55	14.84	85.30	59% (46-74)
COD	87.37	3.35	123.00	82% (81-84)	88.72	22.97	114.10	52% (42-62)	86.33	28.49	113.4	54% (44-77)
N-total	9.05	2.19	11.47	65% (55-79)	4.73	1.76	7.00	79% (63-88)	7.58	5.63	14.2	74% (39-98)
$N\text{-}NH_4$	2.10	1.21	3.52	88% (80-94)	1.40	0.90	2.33	93% (88-99)	5.68	4.74	11.30	76% (40-98)
$N\text{-}NO_3$	1.45	0.06	1.52		2.40	0.71	3.41		1.12	0.67	2.12	
P-total	3.60	0.76	4.62	50% (40-57)	4.00	2.42	7.50	38% (21-54)	3.10	0.66	3.22	45% (24-75)
FC 10^3 cfu/100mL	2.78	0.56	3.40	97% (97-98)	3.13	1.60	4.71	87% (88-99)	3.20	1.82	5.72	75% (70-77)
TC10^3 cfu/100mL	54.18	20.59	66.23	71% (60-85)	24.90	5.04	31.85	64% (55-70)	9.03	2.65	11.20	70% (54-79)

Systems' efficiency for the reduction of BOD, FC and TC can be improved with the combination of the following activities: (i) expansion of the unit and total repair (or design) of the entrance works, (ii) construction at the ponds' exits of overflow devices with stems of floats, (iii) replacement of the inflows and the outflows, so as to be at diagonal opposite corners of every pond, (iv) recirculation of a part of the flow (Parissopoulos G.et als, 2003), (v) construction of a deep anaero-

bic pond, before the ponds, with a retention time 3 days, which could sensibly improve the efficiency (Parissopoulos G.et als, 2003).

5. CONCLUSIONS

The total estimation of the operation of the waste stabilization systems is a time-consuming and expensive procedure and it requires specialized staff which will be able to explain the information received. However, it is the only way to improve the ponds' design according to the local conditions. Nevertheless, considering the benefits that result from the effluents reuse, it is finally an economically efficient procedure. The results presented at this paper, came up after annual observation. Certainly, more time and succession of the measurements are required in order to have more reliable results. However, some first estimation can be made concerning the pond's efficiency and its prospects of improvement, so as to be able to reuse the effluents.

Waste Stabilization ponds' efficiency is not stable throughout the year, and is slightly improved during the summer period. As it comes up from the results of the chemical analysis and the analysis of the facts, heavy metals and the rest microelements that were measured, they present a relevant stability at their concentrations throughout the year. An exception constitute the elements Al, Cu, Mn, Zn which presented a significant increase of their concentrations during the spring and summer months, while Pb concentration presented a considerable reduction. All the elements had concentrations, throughout the year, much lower than the limits prescribed by the Greek and international wastewater's specifications, for the sensitive receivers as well as for the irrigation water. The values of SAR, pH, EC and the concentrations of N-total, $N\text{-}NH_4$, P-total, SS and COD constitute the ponds' water suitable for irrigation. However, the concentrations of BOD_5, FC, TC show that the waters don't fulfill the prescriptions for irrigation.

Systems' efficiency for the reduction of BOD, FC and TC can be improved with a combination of activities, such as like: redesign of the systems with the proper wastewaters' temperature and expansion of the units, total repair and replacement of the entrance and exit works, recirculation of part of the flow and design of a deep anaerobic pond before the other ponds.

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BIODEGRADABLE ORGANIC CARBON CHANGING DURING WATER TREATMENT

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Abstract: A biodegradable character of the organic carbon is an important feature affecting water grade in the distributive systems of water supply. The paper studies content of the biodegradable organic carbon in a few water types (river, artesian, tap and model ones) and its variation in the water treatment process by means of different methods. It is shown that neither the Dnieper River water after its coagulation, settling, filtration nor tap water do not meet the requirements as to its biological stability. There is not any biodegradable carbon in water only after its after treatment using the carbonic filter and disinfection with ultraviolet rays.

Key words: Biodegradable organic carbon, Drinking water treatment, Natural organic matters.

1. INTRODUCTION

An availability of the natural organic matter (NOM) in the natural water applied to get drinking water determines to a great degree the water treatment technology and, finally, the grade of drinking water. In so doing, it is of vital importance to find out the content of the NOM or the total organic carbon (TOC) dissolved which is available biologically. A biodegradable character of the TOC is an important factor influencing the water grade in the water-supply distributing systems (Egeberg, 2002; Raczyk-Stanislawiak, 2004; Goncharuk, 2006). An availability of the NOM in the biodegradable form in the drinking water can lead to the unpleasant both smell and taste, be a formation reason of the toxic disinfection products (generally, trihalogenomethane and halogenacetic acids), give rise to a biofouling of the water supply systems. The NOM in the surface waters consist in general from the humus kind matter: hydrophilic acids, carbonic ones, aminoacids, carbohydrates and hydrocarbons. In the natural waters with a high color of water degree to which the Dnieper River basin waters belong to, the content of humus matter is as high as 50–90% of the NOM. An availability of the biodegradable compounds in water is due to, on the one hand, a specific formation character of the natural water in the water basin and, on the other hand, a formation of the biologically accessible organic carbon during the water treatment processes, particularly at its treatment with the oxidants.

The biologically degradable organic carbon (BDOC) is a part of the organic carbon, which is mineralized by the heterotrophic microorganisms. The magnitude of the BDOC is measured through a determination of the difference between the initial concentration of the total organic carbon and the final one after its incubation with the mixed population of microorganisms (Huck, 1990).

A correlation TOC/BDOC might be applied as an indicator of the relative biological stability of substances capable to biodegrade (Van der Kooij, 1982). It has also been shown that the BDOC

magnitude cannot be used as a biological fouling index, because the correlation between the BDOC and a number of the heterotrophic bacteria has not been found. This is likely because the BDOC determination limit is equal to 0.1–0.2 mg/l, and both the carbon capable to mineralize and that utilized by the cell biomass enter into the BDOC index. Thus, the biologically available carbon in the purified water is a substrate for accumulation of the biodestruction products and a growth of the microorganisms in the distributing systems which is stipulated by presence in water of assimilable organic carbon (Van der Kooij, 1982). It is why a part determination of the BDOC in the total content of the TOC is currently under an intent look in the potable water treatment technology. As paper (Servais at al., 1991; Andrews and Huck, 1996; LeChevallier at al., 1996; Volk and LeChevallier, 2000; Escobar and Randall, 2001a, b) bear witness, the BDOC magnitudes in the water under treatment equal to correspondingly 0.15–0.2 mg/l at 20°C and 0.3 mg/l at 30°C may correspond to a biologically stable water state.

A role of the NOM in both the treatment technology of potable water and its final grade is the problem of an extremely topical importance for the surface sources of the centralized potable water supply of the Ukraine. A state analysis of the surface waters of the Dnieper River shows that the organic matter content as concerns its chemical oxygen demand (COD) for the main water artery of the Ukraine varies from (mgO/l) 54.0–65.0 (worst-case values) and 40.5–43.2 (mean values) in the Kanev water reservoir to 36.0–40.0 (worst-case values) and 25.0–30.2 (mean values) in the Kachovsk water reservoir. This corresponds to a content variation of the dissolved TOC from 24 to ~10 mg/l surpassing much this index characteristic for a number of river basins of the West and Central Europe. It is known that it is possible to remove 50–70% of the organic matter in the coagulation process. Therefore, the residual content of the TOC in the Dnieper basin water might be 3–7 mg/l, what is far higher as compared to the BDOC level for the biologically stable water.

It is known that a part of the BDOC in the TOC increases while running the water ozonation processes (Nishijima at al., 2003, 2004). This is a known technique in the water treatment technology at combining the ozonation with the following filtration of water through the biologically active carbon (BAC). Such a technique substantially increases the BAC running resource and decreases the TOC loading on the BAC.

The goal of the paper given consisted in determining the BDOC content in a few kinds of water (river, artesian, tap and model ones) and its variation in water treatment process with different methods. Taking into account that the BDOC content in the surface supply sources depends on the natural and environmental conditions of water composition formation, this information has to be of an extreme importance at a development of the water treatment technology and an improvement of the final grade of potable water.

2. MATERIALS AND METHODS

The Dniper River water taken off in the Kyiv region, tap one from a tap, tap one from a tap after its filtration through activated carbon and disinfection, the solutions of fulvic acid (FA) after different modes of their treatment with ozonation have been taken as the subject matters. A choice of the FA solutions is because the raised FA content as compared with humic acids is characteristic for the Dniper water (Linnik at al., 1995). So, a characteristic feature of the Kyiv reservoir water is a high content of the humic matter with a carbon to nitrogen ratio of 9:18. Under the laky conditions of the water storage reservoir, there occur a gradual deposition of the humic acids and an enrichment of water with the more hydrophilic FA with a moderate molecular weight and the organic solutes with a high content of nitrogen. In the Kyiv reservoir water, concentrations of the

humic acids and FA are ca. 0.4–1.1 mg/l and 70–140 mg/l, respectively. The FA content in the Dniper River water after its purification according to the classical technological scheme was ca. 35–60 mg/l.

2.1. FA preparation.

The FA under study have been reached according to the Forsith's method (Alexandrova, 1980) from a high-moor peat were used as a sorbate. 400 g of air-dry peat were pored with 2 l of 0.1 M NaOH solution. The mixture was stirred up for 24 hours. Dark supernatant fluid that contained humic and fulvic acids was acidified to pH 2.0 by adding concentrated sulfuric acid for separating the fraction of humic acids. Filtration was used to separate the solution of fulvic acids. Next, 4 M NaOH solution was added to the FA-containing extract to achieve pH 7. The resultant solution was used as a preparation of peat fulvic acids.

2.2 FA solution ozonation and filtration.

The ozonation of the FA solutions has been performed as follows: 5 l of the FA solution have been treated with the ozone/airy mixture. The ozone concentration in that mixture was 5 mg/l. The delivery rate of that mixture was 2.5 l/min. The solutions were ozonized for 2, 4 and 6 min, and the corresponding ozone dosages were 6, 12 and 18 mg/l.

The filter aimed at tap water treatment was loaded with the KAU activated carbon. It is being produced by means of treatment of the shredded stones with a concentrated alkali. After flushing with water, it follows a treatment with the hot hydrochloric acid, then it is flushed again, carbonized and activated with a vapor. The major adsorptive characteristics of that carbon were as follows: the total specific surface of 1036.4 m^2/g, the limiting adsorptive pore volume of 0.51 cm^3/g, the microporous space volume of 0.16 cm^3/g.

The total organic carbon has been determined using the catalytic combustion method (the Shimadzu TOC-V CSN device) at a temperature of 800°C.

2.3 BDOC dtermination.

The BDOC part in the TOC has been determined using technique represented in (Joret at al., 1998; Raczyk-Stanislawiak at al, 2004; Trulleyova and Rulik, 2004). It is pointed out in these papers that, at the BDOC determining, it is the best thing to fix the microorganisms at the inorganic carriers (quartz sand) and to use the microorganisms from the potable water treatment stations and the distributing systems.

To determine the BDOC, we applied the biologically active sand taken from the fast filters of the Dnieper waterworks station. Just before investigation, that sand has been rinsed out with distilled water up to reaching the constant TOC content in the rinsing water. The TOC content parameters in the rinsing water are represented in Table 1.

Table 1. Variation in the TOC in the rinsing waters after the sand rinse

Amount of rinsing	TOC content in rinsing water, mg/l
10	8,0
20	8,0
30	0,6
40	0,4
50	0,4

Note. The volume of each rinse was 500 ml.

After rinse, 100 g of the moist biologically active sand has been placed into 5 incubatory flasks, and then 300 ml of the water under study have been added there. The trial run has been performed in the temperature-controlled room at a temperature of 20±2°C at a continual aeration of the water sample with an intensity of 4 l/h. Air from the aerator passed through the air purifier filled with the distilled water. The incubatory period was 5 days. Every day, the water sample has been taken from the incubatory flasks and has been filtered through the glass filter (0.7 μm). Next, the TOC content has been found out in each sample.

The relationship offered by Van-der-Kooij, 1982 has been used to compute the BDOC:

$$BDOC = TOC_{in} - TOC_{min} - TOC_r, \quad (1)$$

where TOC_{in} is the TOC content in the flask before the incubatory period (mg/l), TOC_{min} is the minimum TOC content over the incubatory period (mg/l), and TOC_r is the TOC content in the rinsing water after final rinse of sand (mg/l).

RESULTS AND DISCUSSION

In the first series of the trial runs, the BDOC was determined in the initial Dniper River water, the same water after its coagulation, settling and filtration, the tap water taken just from a tap, the tap water after its finishing purification through activated carbon and then disinfected with ultraviolet and artesian water taken from a well room. Table 2 shows the results of these trial runs.

Table 2. The BDOC content in the initial natural water and after its treatment

Type of water	TOC during incubation. mg/l						BDOC	
	initial	1 day	2 day	3 day	4 day	5 day	mg/l	%
Dniper water	12.7	11.2	11.0	11.6	11.5	12.1	1.4	11.0
Dniper water after coagulation	7.5	6.0	6.6	6.0	6.2	6.6	1.5	20.0
Tap water	7.7	7.4	7.1	8.3	7.8	8.2	0.6	7.7
Purified tap water	1.7	2.0	2.0	2.6	2.9	3.4	0	0
Artesian water	2.1	2.7	2.8	3.2	4.3	3.8	0	0

As may be seen from the data of Table 2, the BDOC content in the initial Dniper River water is comparatively small (11% regarding the TOC). It increases up to 20% as a result of water treatment with coagulation. This associates with the fact that the organic substances pertaining to the BDOC are as a rule more hydrophilic and consequently are worse removed at coagulation as compared with the resistant organic matter. Some increase of the TOC in the process of its daily determination may associate with dying off and abruption of the microorganisms from a carrier, and this brings about a certain growth in the TOC concentration at the end of an incubatory period.

As it is seen from the data of Table 2, neither the Dniper River water after its coagulation/settling/filtration, nor the tap water do not meet the requirements of the biological stability, and the BDOC content in water after its coagulation surpasses its content limits recommended in (Servais at al., 1991; Andrews and Huck, 1996; LeChevallier at al., 1996; Volk and LeChevallier, 2000; Escobar and Randall, 2001a, b) almost into an order of its scale. There is not any BDOC in water after only its afterpurification using the charcoal filter and disinfection with an ultraviolet. There was not also observed any BDOC in the artesian water taken from the pump-room.

In the next series of trial runs, a variation in the BDOC content with respect to the TOC in the model solutions of FA was under study. The experiments have been performed in the acidulous (pH 2) and neutral (pH 6) media. The results reached are represented in Tables 3 and 4.

As it is seen from the data of Tables 3 and 4, a trend to a decrease in the BDOC part is generally observed with increasing the contact time of solutions with the ozone/airy mixture at ozonation of the FA solutions with a high BDOC content in both the acidulous and neutral media. This effect is

more notable in the acidulous medium. This is likely due to a high oxidative capability of ozone in relation to easy oxidizable organic matters.

Table 3. Variation in the BDOC content in the ozonized solutions of FA at different contents of the initial TOC (pH 2)

Ozonation	TOC during incubation, mg/l						BDOC	
time, min	initial	1 day	2 day	3 day	4 day	5 day	mg/l	%
Initial TOC content – 22.4 mg/l								
0	22.4	19.5	22.0	15.0	12.2	14.3	10.2	45.5
2	21.9	21.3	19.5	18.6	10.3	11.9	9.6	43.8
4	22.9	23.9	19.5	15.9	17.6	18.7	7.0	30.6
6	22.1	19.1	15.7	12.3	12.0	13.1	9.7	43.9
Initial TOC content – 29.7 mg/l								
0	29.7	29.6	27.0	16.0	16.0	11.7	18.0	60.6
2	26.7	27.0	24.9	23.4	17.4	16.6	10.1	37.8
4	28.5	35.4	26.6	29.8	20.2	19.9	8.6	30.2
6	29.4	33.4	27.9	32.4	31.5	30.3	1.5	5.1
Initial TOC content – mg/l								
0	36.8	35.4	43.6	28.4	29.4	20.1	16.7	45.4
2	37.8	30.1	34.7	32.5	32.2	20.7	17.1	45.2
4	28.9	35.7	27.4	28.0	23.6	23.3	5.6	19.4
6	22.9	37.3	35.1	22.9	22.6	24.5	6.6	22.6

Table 4. Variation in the BDOC content in the ozonized solutions of FA at different contents of the initial TOC (pH 6)

Ozonation	TOC during incubation, mg/l						BDOC	
time, min	initial	1 day	2 day	3 day	4 day	5 day	mg/l	%
Initial TOC content – 14.1 mg/l								
0	14.1	9.2	8.3	10.2	5.7	9.2	7.9	56.4
2	12.7	8.3	6.3	7.9	5.1	5.6	7.2	56.6
4	13.7	6.5	6.3	8.2	7.5	7.2	7.0	51.0
6	13.6	6.4	6.9	8.3	7.1	6.6	6.8	49.8
Initial TOC content – 33.2 mg/l								
0	33.2	23.5	20.3	18.2	16.7	8.8	24.1	72.4
2	32.5	22.2	17.6	16.9	15.4	16.3	16.8	51.5
4	35.1	22.8	20.4	17.5	8.9	15.9	25.85	73.6
6	34.2	22.1	20.6	16.7	9.0	16.5	24.7	72.4
Initial TOC content – 35.2 mg/l								
0	35.2	17.2	16.3	17.7	18.7	19.3	18.5	52.6
2	31.5	17.2	17.3	17.8	16	17.1	15.1	47.9
4	32.1	17	18.3	18.4	17.2	19.6	14.7	45.8
6	33.7	18.8	18.1	17.4	17.6	17.8	15.9	47.2

It was of interest to find out in which way ozone treatment affects the BDOC part at its pre-removal by means of modeling the self-purification process. To that end, the solution containing FA has been subjected to aeration for 14 days with adding the biologically active sand. The initial solution of these acids before aeration (as it was determined in a separate trial run) contained 24.1 mg/l of the BDOC. It was succeed to reduce the BDOC content up to 3.5 mg/l, i.e. almost 7 times. The aeration process has been controlled through rating a variation in the COD in its course. After reaching the constant CODs, the FA solutions have been subjected to ozonation. Table 5 shows these experimental results.

Table 5. *Variation in the BDOC content in the aerated solution while the ozonation process of a solution (pH 6)*

Ozonation	TOC during incubation, mg/l						BDOC	
time, min	initial	1 day	2 day	3 day	4 day	5 day	mg/l	%
0	33.1	41.0	52.6	49.0	29.6	31.6	3.5	10.6
2	31.8	38.6	55.0	23.1	25.4	32.9	8.8	27.5
6	30.5	31.0	33.8	30.2	31.9	15.9	14.7	48.0
20	31.0	28.9	31.6	16.3	20.3	23.9	14.7	47.4

Ozonation of a solution for 6 and 20 min leads to an increase in the BDOC content up to 14.7 mg/l, and this can essentially raise an effectiveness of the following extraction of the TOC and BDOC from water at its filtration through the BAC. It follows also from the data of Table 5 that ozonation of the FA solutions more than 6 min is pointless, because it does not bring about an increase in the BDOC part.

4. CONCLUSIONS

Thus, it is very likely that, in the water treatment processes, it is expedient to combine the pre-aeration of water with the major technological processes to reduce the BDOC content before coagulation, and that will have a positive effect on the coagulation effectiveness and will reduce any capability of a deposit to decay. In combination with filtration through an active carbon after the coagulation process, ozonation will provide a means to reach the water stable reasonably biologically and to avert formation of the disinfection toxic products after chlorination.

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NATURAL ORGANIC MATTERS REMOVING BY ACTIVATED CARBON IN PRESENCE OF HYDROGEN PEROXIDE

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Abstract: It is studied an effect of the surface chemistry variation of the active carbon on the parameters of the equilibrium adsorption of the fulvic acids from their aqueous solutions on the carbon in the presence of hydrogen peroxide. A capability of the multicomponent approach to describe the adsorption equilibrium through use at least two hypothetical components in the Freundlich's equation has been considered. The concepts as to "conditional component" have been offered to apply for the quantitative assessment of an adsorbability from the multicomponent mixture of the organic substances. An effectiveness of adsorption on the active carbon is shown to increase substantially at introducing the hydrogen peroxide in the solution of fulvic acids, and this due to the catalytic oxidation effect of the organic matter. This effect is the most effective at the higher pH values, where the carbon particles are in the isoelectric state.

Key words: Adsorption, fulvic acids, hydrogen peroxide, oxidized activated carbon

1. INTRODUCTION

An effective employment of the active carbons (AC) in the purification and afterpurification of the natural waters and wastewaters is determined in the first place with the capability to prolong the service term of the adsorptive filters, their overload and the following remote regeneration. Time of service of the AC are known can be prolongated due to simultaneous passing of the bio-regeneration and adsorption on the AC processes because of a development of the biological activity in the AC layer (Seredynska-Sobecka at al., 2006; Urfer and Huck, 2000; Servais at al., 1994). Depending on implementing the process conditions, bioregeneration of the AC can provide 65–95% recovery of their adsorptive properties (Klymenko at al., 2003; Klymenko at al., 2002; Klymenko at al., 2006). That process runs in a most effective way in the case when the water containing the biologically available organic compounds goes in the filter with the AC (Nishijima at al., 2003). A transformation of the bioresistant organic substances into the biodegradable form is most commonly reached by a means of the prior oxidation with ozone or other oxidants (Yavich at al., 2004; Guzzella at al., 2002). Except for a biological oxidation of the organic matter adsorbed, a combination of the adsorption processes and the heterogeneous catalysis is appears promising from an prolongation viewpoint of the AC service life in the systems with carbon and oxidant.

Carbon is known capable to catalyze some oxidation reaction due to its capability to take up electrons because of an availability of the unpaired electrons at the edges of crystallites (Guoxian at al., 2005). This leads to a decomposition of contaminations in the AC suspension (Georgi and Kopinke, 2005; Lucking at al., 1998).

Hydrogen peroxide is one of the promising oxidant in such an angle. As it has been pointed out in a number of papers (Chou and Huang, 1999; Francisco, 1998), an oxidation in the AC/H_2O_2 system may be considered like the Fenton oxidative process in the presence of the heterogeneous catalyst. The pH of a medium essentially affects an effectiveness of the heterophase oxidation in that system (Georgi and Kopinke, 2005) and its optimum is ca. 3.

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The decomposition processes of hydrogen peroxide and the catalytic oxidation of the organic substratum by means of acting the OH· radicals on a part of the organic compound run concurrently in that system. That part is in the free porous and inter-porous spaces of the AC, whereas the adsorbed part is actually inactive (Georgi and Kopinke, 2005).

An effect of adsorption on an effectiveness of the catalytic oxidation of the organic matter with hydrogen peroxide in the presence of the AC is not enough depicted in literature. It is pointed out that the highest catalytic activity of the AC in decomposition of H_2O_2 leads finally to a lower catalytic effectiveness in the organic matter oxidation (Huang at al., 2003). A modification of the AC surface with the functional groups is shown to affect not only the H_2O_2 decomposition but also the 4-chlorophenol oxidation. The acidic groups reduce a catalytic activity of the AC with respect to the H_2O_2 decomposition, stimulating so an increase in the catalytic activity of the AC with respect to the 4-chlorophenol (Huang at al., 2003). A simplified mechanism of the catalytic action of the AC with respect to the organic substratum in the presence of H_2O_2 is offered in (Miller and Valentine, 1995). Its gist boils down to the fact that a decrease in the catalytic activity of the AC/H_2O_2 system at an oxidative carbon treatment reduces the surface activity as concerns an absorption of the peroxide decomposition products. This promotes thus an increase in probability of the effective collisions between the radicals and organic molecules.

A catalytic activity of the H_2O_2 decomposition of the AC increases with increasing the pH of isoelectric point (Huang at al., 2003). This is because the basic groups play a key role in the H_2O_2 decomposition on the AC surface (Olivera at al., 2004). An increase in the concentration of these groups on the surface results in an increase of the pH corresponding to a zero charge point (Menender at al., 1996).

Thus, despite a somewhat limited character of the literature data, their analysis testifies that a combination of adsorption and heterophase oxidation is promising as concerns water purification goals. To substantiate an effectiveness of a joint application of these processes, the more thorough fundamental and applied researches are demanded.

The goal of this paper is to investigate an effect of varying the AC surface chemistry on the parameters of the equilibrium adsorption of the fulvic acids (FA) and their catalytic oxidation at the presence of hydrogen peroxide.

In the paper (Smith, 1994), there has been considered an opportunity to apply the multicomponent approach to describe the adsorptive equilibrium according to the Freundlich's model. In line with this paper, at considering adsorption from the multicomponent system, to which the FA solution belong to, the Freundlich's equation in the logarithmic coordinates has either two or three principal sections in the form of the straight lines. The first section represents itself a non-adsorbing part of the natural organic matter (NOM), which is expressed with a vertical line. The second one is a weakly-adsorbing part of the NOM, where the strongly-adsorbing component and a proportional fraction part of the more weakly-adsorbing component are removed at the mean dosages of an adsorbent. The third section is a strongly-adsorbing part of the NOM and represents itself a sharply inclined area implying a competition for the AC surface accessible among the components with a high adsorptive capability (Smith, 1994; Smith and Weber, 1990). It has been shown that an availability of the compounds differing in their adsorbing capability in the multicomponent mixture has an effect on the isotherm form in each area. Further, each area is described with distinctive n_i and $K_{F,i}$ values, that is an exponent magnitude for the strongly-adsorbing component will be higher than that for the weakly-adsorbing one.

We have offered to apply a "conditional component" conception for the quantitative rating of adsorbability from the multicomponent mixture of the organic substances. Further, at rating the adsorptive equilibrium constant and variation of the free adsorption energy, we have suggested to represent its concentration in the form of COD or total organic carbon (TOC). The gist of this method is represented in paper (Koganovsky at al., 1990).

The characterization of multicomponent adsorption equilibrium for natural organic matters using a "conventional component" was successfully effected by applying the ideal adsorption solution

theory (IAST) (Smith and Weber, 1990; Sontheimer at al., 1988; Harrington at al., 1989). That approach involves the solution of the equation:

$$C_{o,i} - q_i M = \frac{q_i}{\sum_{i=1}^{N} q_i} \left[\frac{\sum_{i=1}^{N} q_i \gamma_i}{\gamma_i K_{F,i}} \right]^n \qquad (1)$$

Where γ_i and $K_{F,i}$ are equilibrium characteristics of an individual hypothetical component *i*, *C* and *q* are the concentration of the *i*-th component in liquid and solid phases respectively, *M* is sorbent dose.

In (Koganovsky at al., 1990) it is proposed to employ for the quantitative assessment of the adsorbability of a multicomponent mixture of organic matters from aqueous solutions adsorption equilibrium constant, using a "conventional component" to express the concentration of a mixture of organic matters.

The equation of isotherm of adsorption of organic compound from aqueous solution may be represented as:

$$K_i = \frac{a_i \varphi_i}{\left[V_a - a_i \left(V_i - V_{H_2O} \right) \right] C_i} \qquad (2)$$

Where a_i is the specific value of equilibrium adsorption of the *i*-th component at the concentration C_i, expressed in terms of molal concentration, V_a is the volume of the adsorption space of the sorbent; K_i is a constant of selective-adsorption isotherm; φ_i is the activity coefficient of the *i*-th component; V_i and V_{H2O} are the molar volumes of adsorbed components. The essence of "conventional component" method is, in accordance with (Koganovsky at al., 1990), that Eq.2 rearranged so that graphical determination of adsorption equilibrium constant can be performed without knowing the molar volumes of adsorbed matters. To this end, Eq.2 is rearranged in the form:

$$\frac{a_i \varphi_i}{\left[V_a - a_i \left(V_i - V_{H_2O} \right) \right]} = K_i C_i \qquad (3)$$

From where:

$$\frac{V_a}{a_i \varphi_i} = (V_i - V_{H_2O}) + \frac{1}{K_i C_i} \qquad (4)$$

In the range of low solution concentration, φ_i is only slightly different from unity. Therefore, the graphical representation of V_a/a_i versus $1/C_i$ allows one to determine K_i as the cotangent of the slope angle of straight line to the ordinate axis.

Thus, the both approaches (the multicomponent one and the "conditional component" method) provide means to assess in which way variation of the AC surface chemistry from aqueous solutions affects the equilibrium adsorption parameters of the FA in the presence of hydrogen peroxide.

2. MATERIALS AND METHODS

2.1. Preparation of FA.

The FA under study have been reached according to the Forsith's method (Alexandrova, 1980) from a high-moor peat were used as a sorbate. 400 g of air-dry peat were pored with 2 l of 0.1 M NaOH solution. The mixture was stirred up for 24 hours. Dark supernatant fluid that contained humic and fulvic acids was acidified to pH 2.0 by adding concentrated sulfuric acid for separating the fraction of humic acids. Filtration was used to separate the solution of fulvic acids. Next, 4 M NaOH solution was added to the FA-containing extract to achieve pH 7. The resultant solution was used as a preparation of peat fulvic acids.

2.2. Activated carbon and it's oxidation.

The KAU label carbon has been applied in the experiments. It is obtained by way of treating of the shredded bones with the concentrated alkali. After water rinsing, there follows its treatment with a hot hydrochloric acid, rinsing again, carbonization and activation with a steam. The carbon has been oxidized according to techniques described in Refs (Menender at al., 1996; Klymenko at al., in press). The KAU oxidation has been performed with nitric acid and hydrogen peroxide. The AC sample oxidized with hydrogen peroxide is termed next as KAU-O, and ones oxidized with nitric acid are termed as KAU-N3 and KAU-N9, where 3 and 9 numbers specify a continuation of the carbon treatment with an oxidant (in hours).

2.3. Computation.

The AC applied in the paper, have been characterized according to the parameters of their porous structure and surface chemistry. The former has been determined from the nitrogen adsorption/desorption isotherms at 77 K measured using the Quantachrome Autosorb Automated Gas Sorption System (Quantachrome Corporation). These isotherms made use of to compute the surface areas, S, the total volume of pores, V_a, and the volume of micropores in a sorbent, $V_{\mu p}$. All above-mentioned parameters have been computed according to the functional density theory (FDT), the t-method, and the BET and Dubinin-Radushkevich's models.

An alternative assessment of the structural-adsorptive characteristics of the AC samples has been performed using the adsorption measuring method from an aqueous solution of the standard adsorbate (p-chloraniline) (Koganovsky at al., 1990).

At an adsorption from the FA aqueous solutions, the adsorptive characteristics of the AC samples have been assessed according to the Freundlich's model using approaches quoted in Refs. (Smith, 1994; Smith and Weber, 1990) and the "conditional component" method (Koganovsky at al., 1990).

The cationic and anionic static exchange capacities (SEC) as well as an amount of the surfase groups of the AC have been determined by means of the Boehm's titration method without determining any carbonyl groups (Tarkovskaya, 1981). The ζ-potential of the AC particles has been found out via the microelectrophoresis method in the horizontal cell of a dead-end type at the lower steady-state level.

Table 1 shows the characteristics of the AC samples applied in the paper given.

The pH of the FA under study was ca. 3, because it is an optimum value to run an effective heterophase oxidation in the AC/H_2O_2 system. With the AC samples, the adsorption isotherms have been measured through varying the FA concentration in a solution at a constant preweighted amount of the AC (0.1 g) in the presence of H_2O_2 (5 and 10 mg/dm^3) and in its absence.

The total organic carbon has been determined through the catalytic burning method at 800°C using the Shimadzu TOC-V CSN device.

Table 1. Characteristics of AC samples.

Characteristics of samples	KAU	KAU-O	KAU-N3	KAU-N9
S_{BET} for nitrogen, m^2/g	1036.4	639.5	847.6	1054.0
V_a for nitrogen, cm^3/g	0.49	0.26	0.36	0.42
V_{mi} for nitrogen, cm^3/g	0.17	0.08	0.24	0.24
Cationic SEC , meq/g_{AC}	0.15	0.85	0.85	1.0
Anionic SEC , meq/g_{AC}	0.46	0.44	0.49	0.40
Quantity of surface group, meq/g_{AC}				
carboxylic	0.10	0.55	0.50	0.40
lactonic	0	0.10	0.05	0
phenolic	0.05	0.20	0.30	0.30
pH_{IEP}	4.8	2.6	3.0	3.5

3. RESULTS AND DISCUSSION

Figures 1a–1e represent the FA adsorption isotherms from their aqueous solutions on the non-oxidized AC sample and the oxidized one in the presence of 5 and 10 mg/dm^3 of H_2O_2. In accordance with (Klymenko at al., in press; Smith, 1994; Smith and Weber, 1990) we have split these isotherms into two areas: the weakly-adsorbing "conditional component" and the strongly-adsorbing one of the FA. There have been assessed the K_F and n constants of the Freundlich's equation for the FA adsorption isotherm on the whole, and the n_1 and n_2 magnitudes for the two conditional components chosen. Further, the adsorptive equilibrium constants and correspondingly the variations in the Gibbs free adsorption energy ($-\Delta G°_a$) have been determined for the each area according to the "conditional component" method. Table 2 illustrates the results of these computations.

As it is seen of Table 2, the AC oxidation (with both hydrogen peroxide and nitric acid) reduces the adsorptive capacity in the FA on the whole, what is due to a heterogeneity of the FA structure and an increase in the content of the carboxylic and phenolic groups on the AC surface. With the well adsorptive compounds, an increase in the energetic inhomogeneity of a surface enlarges a driving force of adsorption (Klymenko at al., in press).

Table 2. FA adsorbable characteristics from their aqueous solutions in AC/H_2O_2 systems

System with FA	K_F	n	n_1	n_2	$(-\Delta G_{a1}{}^0)$, kJ/mol	$(-\Delta G_{a2}{}^0)$,
KAU	1.41	1.03	0.21	1.13	11.42	18.97
KAU + 5 mg/ H_2O_2 l	2.51	1.38	0.31	1.53	16.60	19.41
KAU + 10 mg/H_2O_2 l	3.47	1.38	0.57	1.80	19.76	19.67
KAU -O	0.21	0.74	0.26	0.10	8.08	18.76
KAU -O+ 5 mg/ H_2O_2 l	2.19	1.43	0.88	1.46	19.94	19.97
KAU -O+10 mg/ H_2O_2 l	1.38	1.16	0.89	1.49	19.77	19.75
KAU -N3	0.50	0.81	0.18	1.06	11.38	18.69
KAU-N3+ 5 mg/ H_2O_2 l	2.40	1.28	0.23	1.75	15.14	20.30
KAU-N3+ 10 mg/ H_2O_2 l	1.58	1.09	0.25	1.43	12.54	19.61
KAU -N9	0.81	0.75	0.21	0.85	12.54	18.69
KAU-N9+ 5 mg/ H_2O_2 l	2.19	1.22	0.19	1.06	15.85	20.13
KAU-N9+10 mg/ H_2O_2 l	2.19	1.23	0.22	1.18	16.15	19.88

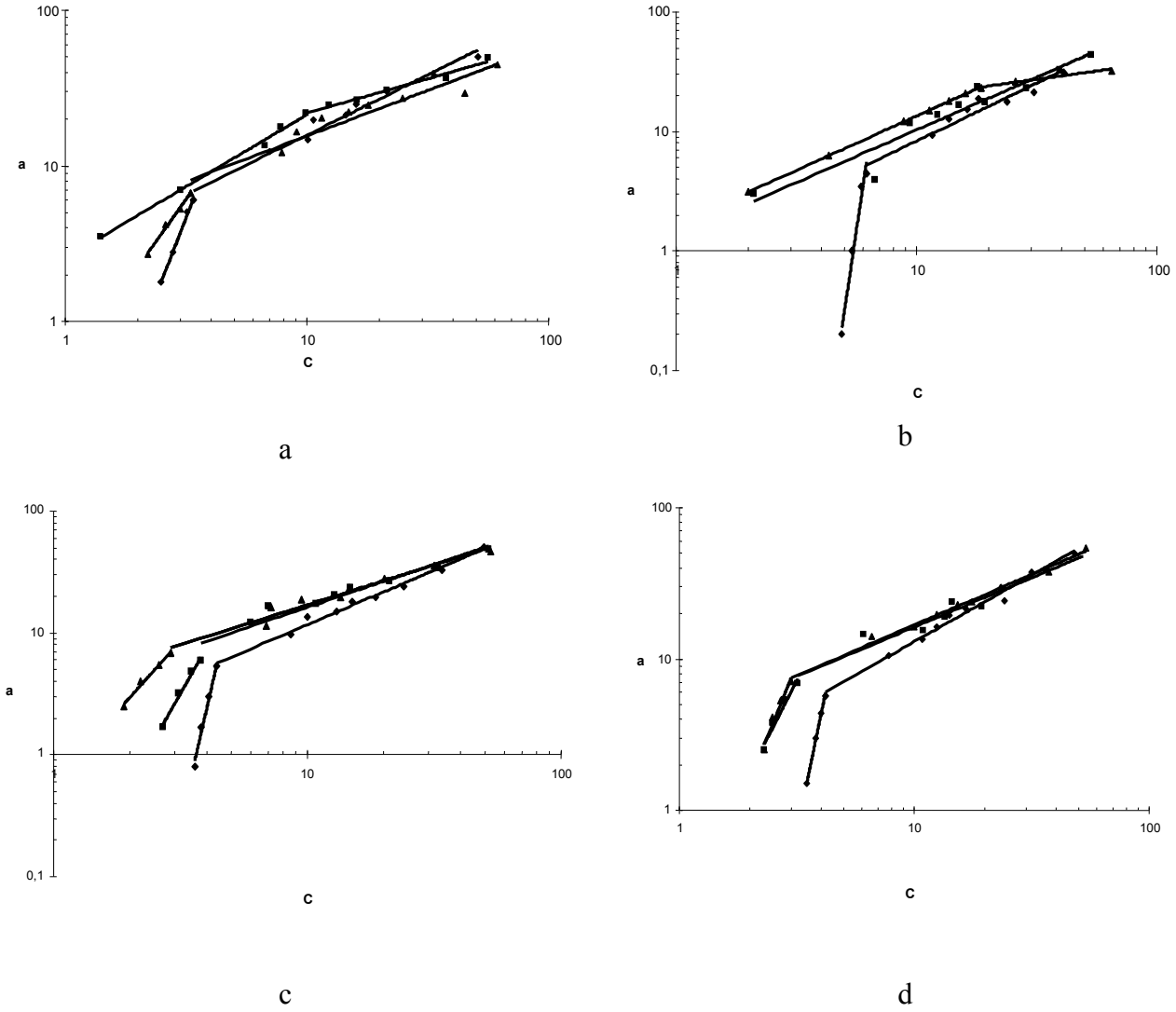


Figure 1. The FA adsorption isotherms on KAU (a), KAU -O (b), KAU -N3 (c) and KAU-N9 (d) in the coordinates of the Freundlich equation: ♦ – without adding H_2O_2; ▲ – in present of 5 mg/ H_2O_2 l; ■ – in present of 10 mg/H_2O_2 l.

However, an introduction into the system with the unoxidized AC of hydrogen peroxide in its amount 5 or 10 mg/l results in an essential increase in the average constant: both K_F and n. This testifies most likely to an effect of the catalytic oxidation of the FA, which takes place simultaneously with an adsorption. The same effect comes to pass also in the systems with oxidized carbon in the presence of H_2O_2. Despite the AC oxidation reduces its adsorptive characteristics, in the presence of H_2O_2, those are higher than on both the unoxidized and oxidized AC samples without any peroxide. An increase in the hydrogen peroxide concentration from 5 to 10 mg/l results in a growth of the catalytic oxidation effect in the case of the unoxidized AC sample only. As concerns the oxidized samples, that effect slightly reduces.

The magnitudes of $-\Delta G_{a1}^0$ increase substantially for the unoxidized KAU sample and the KAU-O in the presence of hydrogen peroxide with increasing the constant in the Freundlich's equation: 5.18–8.34 and 11.86–11.69 kJ/mol with the H_2O_2 dosage of 5 and 10 mg/l, respectively. The magnitude of $-\Delta G_{a2}^0$ for strongly adsorbing AC samples stays on actually invariable. At the AC samples oxidized with nitric acid, an introduction of hydrogen peroxide increases $-\Delta G_{a1}^0$ as follows: for the KAU-N3 and KAU-N9 samples, on 4.02–1.16 and 3.31–3.36 kJ/mol with the H_2O_2

dosage of 5 and 10 mg/l, respectively. Analogously the KAU-O, the magnitude of $-\Delta G_{a1}^{0}$ alters insignificantly.

Such a diversified influence of an introduction of hydrogen peroxide into the system with the AC is due to a simultaneous development of action of a few factors. On the one hand, an alteration in the surface chemistry and an increase in its heterogeneity due to the AC oxidation reduce the adsorptive capacity of the AC as to the FA. This reduction is leveled off however with introducing the hydrogen peroxide into the system as a result of the catalytic effect development. The results reached are at some variance with the data reported in Refs. (Huang at al., 2003; Miller and Valentine, 1995), where it is pointed out that a decrease in the catalytic activity of the H_2O_2 decomposition by way of the AC oxidative treatment increases an effect of the adsorbate catalytic oxidation (4-chlorophenol). Furthermore, it is shown that a catalytic activity of the hydrogen peroxide decomposition on the AC increases with increasing in turn pH of the isoelectric point of the AC particles. In our case, an increase in content of the acidulous groups on the AC surface even though has to decrease the catalytic activity of the H_2O_2 decomposition, it also reduces at the same time a catalytic activity regarding the FA (Table 2). An effectiveness in the catalytic oxidation of the FA increases with increasing in turn pH of the isoelectric point of the AC particles. This effect is particularly clear-cut at a dosage of hydrogen peroxide of 10 mg/l when a reduction in adsorbability of the FA is observed (Table 3).

Table 3. Influence value of pH of isoelectric point of AC on changing of FA adsorbable on AC in present 10 mg/l H_2O_2.

Sample	pH_{IEP}	Constants of Freundlich's equation	
		K_F	n
KAU	4.8	3.47	1.38
KAU-N9	3.5	2.19	1.23
KAU-N3	3.0	1.58	1.09
KAU-O	2.6	1.38	1.16

However, an analysis of the constants in the Freundlich's equation is of more substantial interest to predict an employment effectiveness of the AC systems with hydrogen peroxide at a conditional breaking up the adsorption isotherm in the logarithmic coordinates into two areas (Figure 1), as it is described above. As may be seen of the data depicted in Table 2, the n_1 and n_2 constants characterizing heterogeneity of the energetic centers on the surface and coupled with a driving force of adsorption (i.e. the adsorption energy) increase with introducing H_2O_2 into the system. For that reason, a magnitude of $-\Delta G_{a1}^{0}$ for the weakly-adsorbing "conditional component" increases, too. Most likely, this is not due to an increase in the adsorption interaction force (what follows from comparing the $-\Delta G_{a1}^{0}$ magnitudes for the oxidized and unoxidized AC samples in the absence of H_2O_2, but due to a lower equilibrium concentration of the FA because of the catalytic oxidation at the same initial concentrations of the FA and adsorbent dosages). From a viewpoint of the subsequent deep purification of water from the FA using the biofiltration method through the biologically AC, an analysis of a variation in the concentration range characterizing an area of the weakly-adsorbing "conditional component" is of greatest interest. As follows of the data of Table 4, a concentration range of the weakly-adsorbing "conditional component" essentially expands in the case of the KAU-O carbon.

Table 4. Change in concentration limits of slightly adsorbable "conventional component" in dependence of processes condition.

Sample	Concentration limits up to, mgC/l		
	without H_2O_2	5 mg/l H_2O_2	10 mg/l H_2O_2
KAU	3,5	3,5	10,3
KAU-O	6,1	20,0	отс.
KAU-N3	4,4	3,8	3,0
KAU-N9	4,2	3,2	3,1

It may be seen also from Table 4 that, along with improving the FA recovery effect from their aqueous solution, a concentration range for the weakly-adsorbing "conditional component" has increased ca. 3.3 times as a result of the concurrently running processes of adsorption and catalytic oxidation on the KAU-O AC in the presence of hydrogen peroxide. This factor might have an essential positive influence on the effectiveness of the subsequent filtration of the FA solutions through the biologically AC. As we showed before (Klymenko at al., 2002; Klymenko at al., 2006), a reduction in the adsorption energy of the organic substratum observed in so doing increases a rate of the AC spontaneous bioregeneration and elongates its resource.

4. CONCLUSIONS

Thus, an analysis of the experimental data considered reveals that the adsorption effectiveness of the FA on the AC essentially increases at introducing hydrogen peroxide in their solution, and this is due to a catalytic oxidation effect of the organic matter. This effect is most markedly exhibited at the AC with a higher pH of the isoelectric state of the AC particles. A technique of the AC oxidation (with either hydrogen peroxide or nitric acid) has a slight effect only at a contribution alteration of the catalytic oxidation into the overall effectiveness of the adsorption-catalytic recovery of the FA from their aqueous solutions using the AC. An application of the AC oxidized with hydrogen peroxide in the AC/H_2O_2 system essentially expands a range of the residual equilibrium concentration, which pertains to the weakly-adsorbing part of the FA in their aqueous solution.

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PERFORMANCE ASSESSMENT OF SECONDARY SETTLING TANKS USING CFD MODELING

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Abstract: Amongst the principal separation processes used to treat effluents in the water industry is the sedimentation of solid particulates from the carrier fluid known as mixed liquor by the force of gravity. New York City DEP is in the process of upgrading its Upper East River water pollution control plants (WPCP's) to incorporate biological nitrogen removal (BNR). The rectangular final settling tanks (FSTs) are a central link in the treatment process and often times a limiting factor in terms of the solids handling capacity especially when high throughput requirements need to be met. The objective of this study was to develop a 3D Computational Fluid Dynamics (CFD) model based on the exact geometry of the existing Gould II type FSTs in Battery "E" at the Wards Island WPCP and calibrate and validate it with in-situ data collected at the site. The performance of the model was also evaluated by comparison with the results from one of the rectangular sedimentation tanks reported by Larsen in 1977 in Sweden, where detailed liquid flow and SS concentrations were measured experimentally and showed very good agreement. The goal of this study is to illustrate how the different modeling approaches applied on the physical phenomena that take place in the tank affect the outcome of a CFD model and its predictions. It is common for internal baffles to be added at the inlet and within the tank to handle high flow requirements such as in wet weather storms. This model has been used as a tool to assess the internal behavior of such baffles and assess the clarifier's performance based on different inlet baffle configurations.

Key words: Biological Nitrogen Removal, CFD Modeling, Gould Type II Final Clarifiers.

1. INTRODUCTION

The New York City Department of Environmental Protection (NYCDEP) is performing a comprehensive update of its water pollution control plants (WPCPs), especially those that discharge into the Upper East River (UER), for nitrogen removal to alleviate the problem of hypoxia encountered in the receiving waters of the Long Island Sound (LIS). The biological nitrogen removal (BNR) update will necessitate a higher mixed liquor suspended solids (MLSS) loading in the aeration tank and hence the final settling tanks (FSTs) have been the focus of study to ascertain their capabilities to handle the higher solids and peak flow loadings and still produce an effluent SS that meets the discharge permit limits. One of the step feed aeration tank trains, Battery "E", located at the Wards Island WPCP was upgraded to the BNR mode with an average dry weather flow of 12 m3/s. Recognizing the critical nature of FST's in upgrading plant performance, a three dimensional (3D) computer model was developed to assess proposed enhancement alternatives. The model was specifically developed for the Gould II type rectangular tanks for which a detailed description is given by (Ramalingam et. al. 2007). (Griborio 2004) and (McCorquodale et. al. 2005) presented the development and application of a CFD model for performance prediction of circular clarifiers under a different geometric configuration while (McCorquodale et.al. 2006) demonstrated how such a model can be used to improve the performance of rectangular clarifiers based on a 2D CFD model. Recently (Barnard et. al. 2007) reported on the circular final clarifier performance of a 65 year old

design which rivals that of modern designs using CFD to identify the reasons of their excellent performance. It is well known that inlet structures (energy dissipating inlet baffles) are used to reduce fluid momentum, converting it into kinetic energy in the form of turbulence and eventually through enhanced flocculation increase the settling characteristics and overall performance of final clarifiers. While such structures are indeed important, very limited modeling information is available on Gould II Type rectangular clarifiers.

This paper addresses some of the major steps taken in developing such a 3D CFD model. The first part will focus on a) the flocculation sub-model that was incorporated within the CCNY 3D CFD model (since flocculation was identified as one of the vital parameters that influences the capture efficiency of SS in the FSTs). The impact of improved flocculation is typically observed by lower DSS values along the path of flow, (McCorquodale 2006). In-situ DSS measurements performed in the FSTs of Battery "E" at the Wards Island WPCP indicated that there is a substantial reduction of DSS as flow passes through the confined region between the inlet port and the influent baffle. Thus the modeling of the flocculation potential within this region was selected as a crucial factor for predicting the concentration of SS in the FST effluent. b) The settling characteristics delineation of the suspended solids will then be addressed followed by c) tracer studies for hydraulic assessment of the tanks.

The second part of this paper will report on the model's predictions based on the proposed inlet design (Xanthos et. al 2008) for the 4H7V configuration shown in figure 6 combined with the addition of a perforated in-tank baffle to achieve better clarified effluent under average and peak flow and loading rates in comparison to the original inlet geometry of 1H1V. Model Development

1.1 Flocculation

The primary function of flocculation in the FST is to improve the settling velocities of SS by increasing the fraction of larger size flocs, reduce the concentration of DSS and thus ensure a better quality effluent. A parameter used to gauge the effectiveness of flocculation is the flocculated suspended solids (FSS) which are defined as the SS that remain in the supernatant after 30 minutes of flocculation time at a velocity gradient "G" of 30 s^{-1} followed by 30 minutes of settling time. The FSS is assumed to be the minimum amount of SS that can be achieved in the effluent of the FSTs from well flocculated mixed liquor. The mechanism of flocculation has been studied extensively and the current understanding of this aggregation phenomena has been discussed in (Maximova and Dahl 2006). Laboratory jar tests have shown that the flocculation rate is related to the hydrodynamic shear, where this shear stress G can be affected by the configuration (geometry) of the FST. It is known that the addition of baffles improves the SS capture performance by modifying the hydrodynamics within the tank and consequently improving flocculation. The hydrodynamic shear induced flocculation is referred to as orthokinetic coagulation. Parker (Parker, Kaufman et al. 1970; Parker, Kaufman et al. 1971) developed a differential equation describing the kinetics of flocculation in turbulent mixing:

$$\frac{dX_l}{dt} = K_B \cdot X \cdot G^2 - K_A \cdot X \cdot X_l \cdot G \qquad (1)$$

This equation incorporates the potential floc formation due to turbulent mixing and floc breakup due to the stresses applied on the floc, where X_l is the concentration of primary (large) particles, X is the concentration of flocculated particles and G is the mean velocity gradient. K_A and K_B are experimentally determined coefficients.

The velocity gradient G, the key component of this widely used equation (1) has not yet been defined rigorously. The most frequently cited study was by Camp and Stein (Camp and Stein 1943),

$$\Phi = \mu G^2 = \mu \left[\left(\frac{\partial u}{\partial y} + \frac{\partial v}{\partial x} \right)^2 + \left(\frac{\partial u}{\partial z} + \frac{\partial w}{\partial x} \right)^2 + \left(\frac{\partial w}{\partial y} + \frac{\partial v}{\partial z} \right)^2 \right] \qquad (2)$$

where the root-mean-square velocity gradient was introduced as an approximation to the fluid shear velocity G. Φ is the dissipation function or viscous energy dissipation rate per unit volume, and μ is the dynamic viscosity. Several researchers (Timothy and Mark 1997; Graber 1998; Francisco and Ismael 2005) have proposed that the traditional definition of G should be modified to:

$$\mu G^2 = \mu\left[2\left(\frac{\partial u}{\partial x}\right)^2 + 2\left(\frac{\partial v}{\partial y}\right)^2 + 2\left(\frac{\partial w}{\partial z}\right)^2 + \left(\frac{\partial u}{\partial y} + \frac{\partial v}{\partial x}\right)^2 + \left(\frac{\partial u}{\partial z} + \frac{\partial w}{\partial x}\right)^2 + \left(\frac{\partial w}{\partial y} + \frac{\partial v}{\partial z}\right)^2\right] \quad (3)$$

since such a modification would account for the particle collisions due to the relative motion caused both by shear strain and normal strain. Assuming locally Gaussian and isotropic turbulence, the G can be also calculated based on the local turbulence kinetic energy dissipation rate ε (Mei and Hu 1999), represented as:

$$G = \sqrt{\frac{\varepsilon}{\nu}} \quad (4)$$

Where ν is the kinematic viscosity. This method of calculating G includes all the shear and normal components of the flow and can be easily implemented with the k-ε turbulence model. The flocculation coefficients K_A and K_B in equation (1) are usually determined using jar test experiments. The G value in such a test is estimated by the power input, P and the viscosityν following equation (5);

$$G = \sqrt{\frac{P}{\nu V}} \quad (5)$$

where V is the total volume of the jar. This averaged G value for a jar test is provided by the manufacturer in the form of a chart that plots G versus the rpm of the stirrer used. Experiments (Ducoste 1997) have shown that while a constant power input has been applied to a stirrer in a jar, turbulence intensity and local turbulent energy dissipation rate varies and is dependent on the impeller type and jar size. Additionally, the calculation of G by the traditional jar test method assumes total transfer of power and hence by dividing the entire volume it gives an average value for the reactor. However, flocculation is a local phenomenon and average values are not appropriate when modeling FSTs. Thus, applying flocculation coefficients determined by the jar test to the full-scale FST is questionable (Graber 1998). In fact, Equation 5 is derived from the case of a one-dimensional, laminar flow, where the G value is homogeneous. In FSTs, the total dissipation of turbulent flow is caused by the large scale mean velocity gradient and the turbulent dissipation at a much smaller scale, the kolmogoroff microscale. Hence, G is not a conservative variable and cannot be simply averaged without accounting for the distribution mentioned above. To obtain the appropriate values of G which would provide the appropriate flocculation coefficients, the jar test experiment was simulated using the CFD program in hand. Using equation (4) the local values of G were computed and with a nonlinear data regression on multiple jar test flocculation data, the new flocculation coefficients K_A and K_B, where obtained as given in table 1.

1.1. Settling Characteristics

The settling characteristics of the suspended solids (SS) in the mixed liquor (MLSS) are site specific and reflective of the type of wastewater and the biology being supported in that plant. Settling velocities were measured for the discrete and zone/compression settling concentration ranges to be used as input parameters in the CCNY CFD model. The threshold concentration for discrete settling of the SS appeared in the range of 400 to 600 mg/L. Hence all discrete particle settling experiments were conducted below this threshold range with some dilutions being as low as 160 mg/L. The data from experiments conducted in a series of settling columns developed at CCNY revealed that 75% of the SS had a velocity greater than 6 m/hr., 10% less than 1.5 m/hr., and the remaining 15% had a settling velocity between 1.5 and 6 m/hr. The threshold SS concentration that resulted in zone or hindered settling was in the range of 1000 to 1200mg/L while the threshold SS

concentration for compression settling was reported in the range of 4000 to 6000 mg/L. The data collected in this study did not support the need for mathematically delineating between zone settling and compression settling and therefore a single Vesilind type (Vesilind 1968) of equation was used to define both zones. Based on data collected over a period of seasonal changes in temperature and operating SVIs, the maximum settling velocity, V_0 and decay rate coefficient given in table 1 were recommended for use in the CCNY CFD model.

1.2. Tracer Studies for Tank hydraulic performance

The objective of the tracer studies was to document the gross movement of flow through the FSTs. The approach used was a modification of the "Multipoint Dispersion Test Procedure" recommended by R.M. Crosby. The procedure was modified by including several sampling locations along the path of flow and sampling at multiple depths. Figure 1 shows typical dye profiles measured just downstream of the cross collector, 42.4 m. from the influent gate at the different depths sampled. It is evident that a significant velocity gradient exists across the depth of flow caused by the density gradient of the mixed liquor entering the tank. Figure 1.b shows the dye curve comparison of the CFD model versus the field test, one of the methods used to calibrate and validate the CFD model in this study. Based on the analysis of data from the combined bench-scale and full-scale experiments, the settling and flocculation characteristics of the MLSS in the FSTs as shown in Table 1, were incorporated in the 3-D model developed.

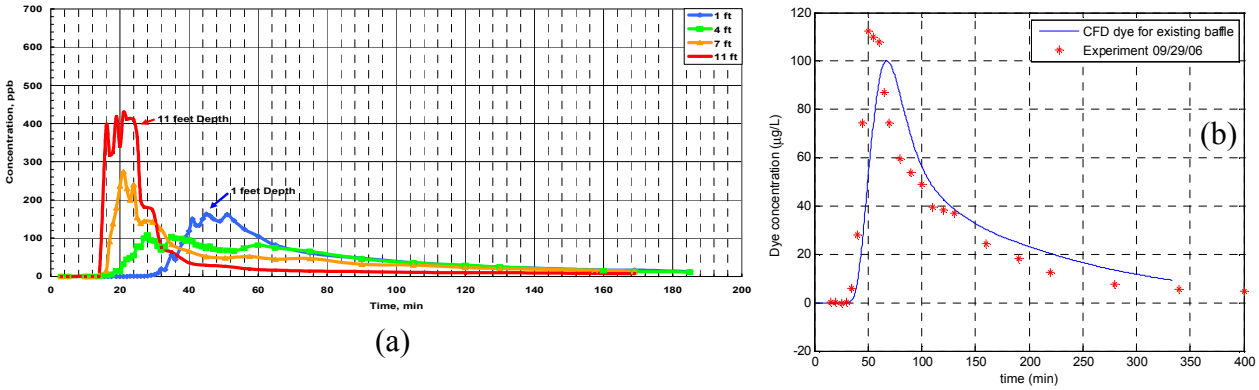


Figure 1: a) Sampling Location S5, pump Sampler after cross collector, Wards Island Tank 3838, April 21, 2006, b) CFD [illegible]

2. HYDRODYNAMIC MODELING

2.1. Preface

The flow field was obtained using a commercial Computational Fluid Dynamics (CFD) Model, Fluent 6.3.26™ as the base platform, where the exact geometry of the FSTs at Battery "E" was modeled with an element tally as high as 1.5 million depending on the modeled case. Computer aided design software was used to translate all the construction drawings of the tanks; Figure 2 shows a section of that digitized model up to the mid point of the tank where the RAS withdrawal is located in two different perspectives. Figure 3 illustrates the modeled FST inlet configuration with the energy dissipating target baffle and the slotted inlet baffle of the original design. The actual image of the later is depicted in Figure 4(a). Figure 4(b) illustrates the CFD simulated inlet configuration where the red lines represent the lines along which quantitative flow parameters were obtained for comparison and performance purposes. The k-ε turbulence model was used for the simulations where the empirical constants were given the standard values suggested by (Rodi 1993) while standard wall functions were used. Modeling the rectangular tanks posed many geometrical challenges due to the particular features that they contain. Figure 5(a) illustrates a perspective effluent view of the actual tank with all its features, while Figure 5(b) illustrates the discretized

domain that was used for the CFD modeling. The level of detail of the model is clearly demonstrated in Figure 5 where all the concrete support columns for the weirs and the scrapper mechanism as well as scrappers are shown. The tank introduces the flow at the inlet using a perforated baffle shown in Figure 4(a) where the bottom plank was removed due to clogging problems. A scrapper system at the base of the tank assists in the transport of the bulk of the solids towards the center withdrawal location.

Table 1. CFD Model Input Parameters obtained at Wards Island

Description	Variable	Value	Units
Large Floc Settling Velocity	V_1	9.19	m/hr
Medium Floc Settling Velocity	V_2	4.8	m/hr
Small Floc Settling Velocity	V_3	0.77	m/hr
Large Floc Fraction	F_1	0.75	NA
Medium Floc Fraction	F_2	0.15	NA
Small Floc Fraction	F_3	0.1	NA
Non-Settleable Solids Threshold	X FSS	4	mg/L
Discrete Threshold	X_d	600	mg/L
Hindered Threshold	X_h	120	mg/L
Initial Settling Velocity for Zone/Compression Settling	V_0	12.2	mg/L
Decay Rate for Zone/Compression Settling	K_1	0.43	L/g
Floc - aggregation coefficient	K_A	8.50 E-04	L/g
Floc - breakup coefficient	K_B	7.80 E-08	s
Tank flow rate	F_{IN}	91,607	m^3/day
Return Activated Sludge flow rate	F_{RAS}	51,346	m^3/day

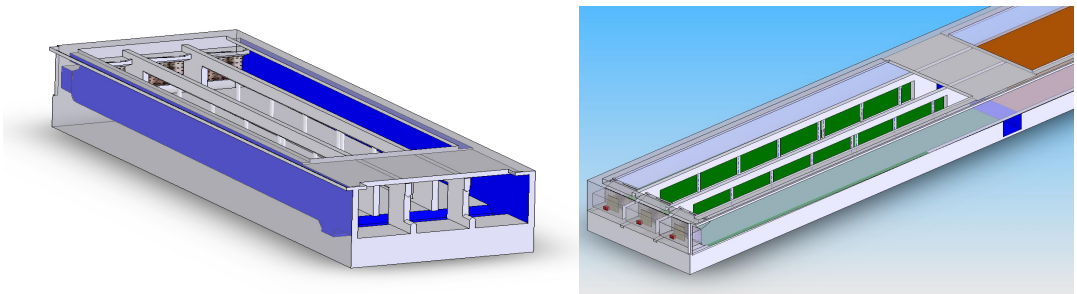


Figure 2: 3D CAD section model of the Final Settling Tank at Wards Island WPCP

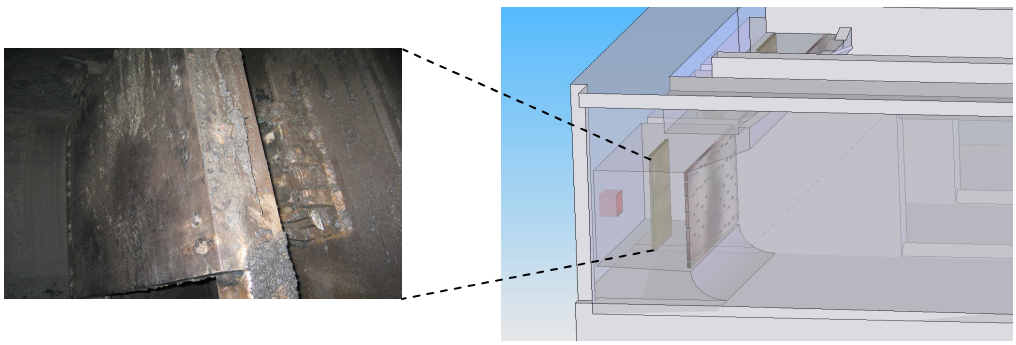


Figure 3. Side view of the influent side of one of the final tanks. The influent sluice gate is shown in red followed by the energy dissipating baffle and the slotted influent baffle. Chain and flights are not shown. Image on the left illustrates the real energy dissipating baffle in the tanks.

2.2. Inlet Baffle Sensitivity using the Hydrodynamic Model

Initially the 3-D CFD model developed by NYCDEP and The City College of New York was used without considering the influence of the solids in the waste water i.e. fluid was treated as pure

water, called the HM model hereafter. The computational time required for each permutation of inlet design in the solids coupled model (SCM) incorporating the suspended solids (SS) settling characteristics, flocculation effects and rheology were extensive. Hence, the HM model was used as a screening tool to identify the best inlet baffle options which would then be modeled using the SCM. Initial optimization runs using the HM model highly supported a new inlet baffle configuration, the one abbreviated as 4H7V as shown in figure 6 where 4H stands for 4 horizontal 30cm planks removed and 7V stands for 7 vertical 30cm planks removed, i.e. leaving 30cm openings into the tank. Figure 6(b) illustrates the actual implemented inlet baffles as proposed by the CFD modeling endeavor.

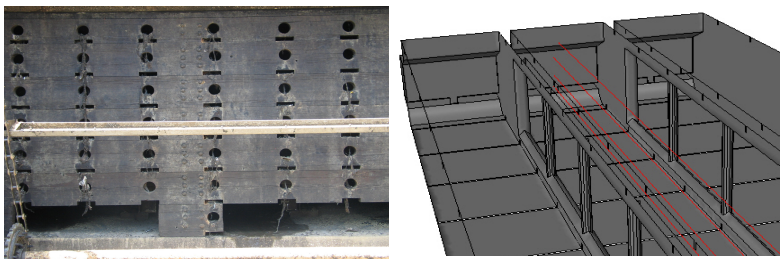


Figure 4: a) Actual inlet perforated baffle with bottom slot removed; b) 1H1V inlet baffle configuration used in the CFD model., 1H1V is an abbreviation for 1 Horizontal plank removed from the bottom of the baffle and 1V for one vertical plank present in the center of the baffle.

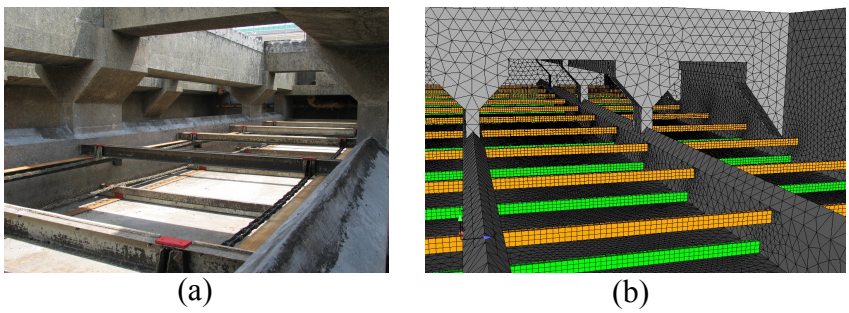


Figure 5: a) Actual image of effluent side of the FST at Battery "E", b) Perspective view of effluent of CFD model of the FST at Battery "E".

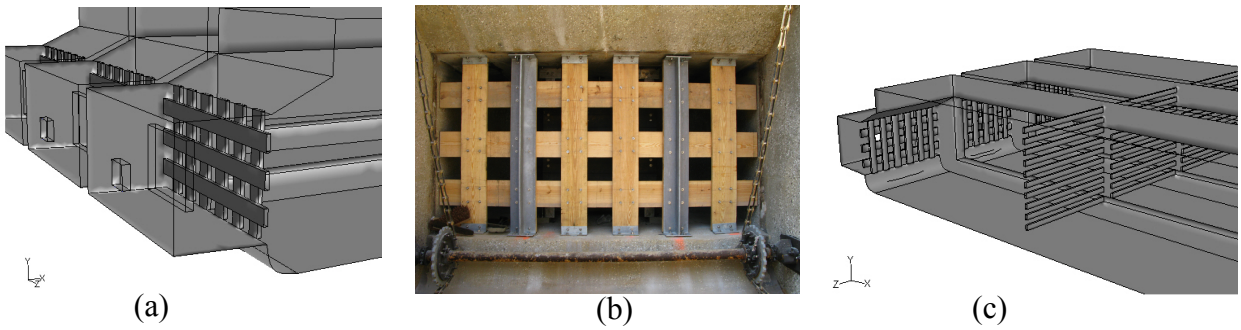


Figure 6: a) 4H7V inlet baffle configuration, b) Actual 4H7V tank upgrade, c) 4H7V with in tank baffle at 10m

Using the *HM model* a comparison based on the X-velocity contours and vectors is shown in Figures 7.a and 7.b where the 1H1V case is compared with the 4H7V. Figure 7.a illustrates the X-velocity contours and vectors for the 1H1V configuration with a distinct recirculation bubble at the inlet a pattern which is expected whilst the flow is introduced in the tank through a 30cm horizontal slot at the bottom of the tank. This recirculation pattern is completely averted in the 4H7V case

where the flow assumes an almost uniform flow downstream into the tank. This qualitative observation can be further investigated if the x-velocity is plotted as a function of tank height along the tank. Figure 8 shows the x-velocity distribution at various depths of the tank where the effect of the change in the inlet baffle configuration from the 1H1V case to the 4H7V case is clear. The HM model predicts an almost plug flow condition with a much slower overall velocity through out the depth of the tank.

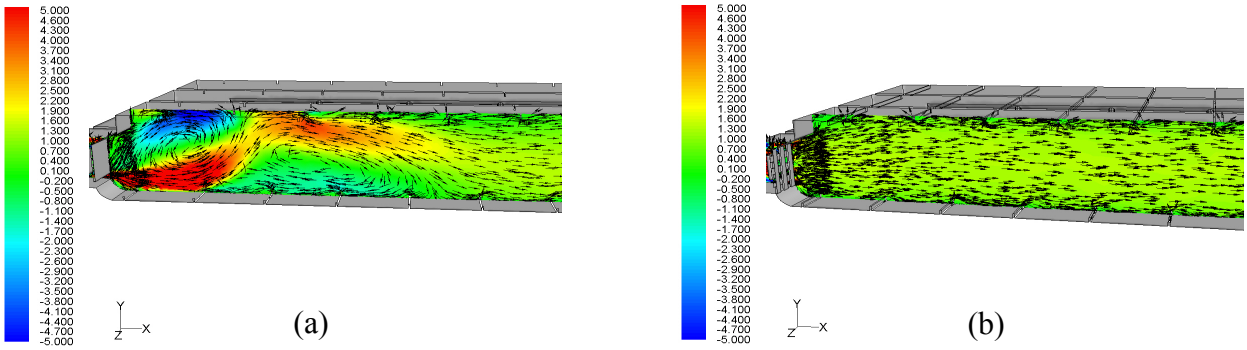


Figure 7: a) 1H1V, X-velocity contours and vectors. b) 4H7V, X-velocity contours and vectors; (cm/sec). HM Model

2.3. In Tank baffle addition using the SCM on the 4H7V case

Different influent baffle arrangements can have a significant influence in the hydrodynamics of the flow in the first half of the clarifier. Proper structural implementation and placement of devices such as baffles can improve significantly the hydraulics and eliminate issues such as short circuiting and reversed flow. Inlets should be designed to dissipate the influent flow energy and allow for the even distribution of flow in all directions in the tank and should promote flocculation by the correct harnessing of the energy, eliminate scouring of solids off the blanket and thus have minimal disturbance on the blanket and reduce short circuiting and overall density current effects. Energy dissipating inlets as well as in-tank structures such as baffle plates if correctly positioned can promote re-flocculation and provide uniform flow distribution. Using the 4H7V inlet baffle configuration as discussed in section 3.2 an in-tank baffle was added at a distance of 10m (see Figure 6.c) to investigate its effect on the flow characteristics and SS settling efficiency using the SCM. Rather than concentrating on the flow field and its pattern, the large particle solids fraction are plotted in figure 9 at three different elevations (0.5m, 1m and 2m from the bottom) with and without the baffle addition. It is clear that the baffle does reduce the large solids fraction substantially, a reduction which is also observed for both medium and small particles.

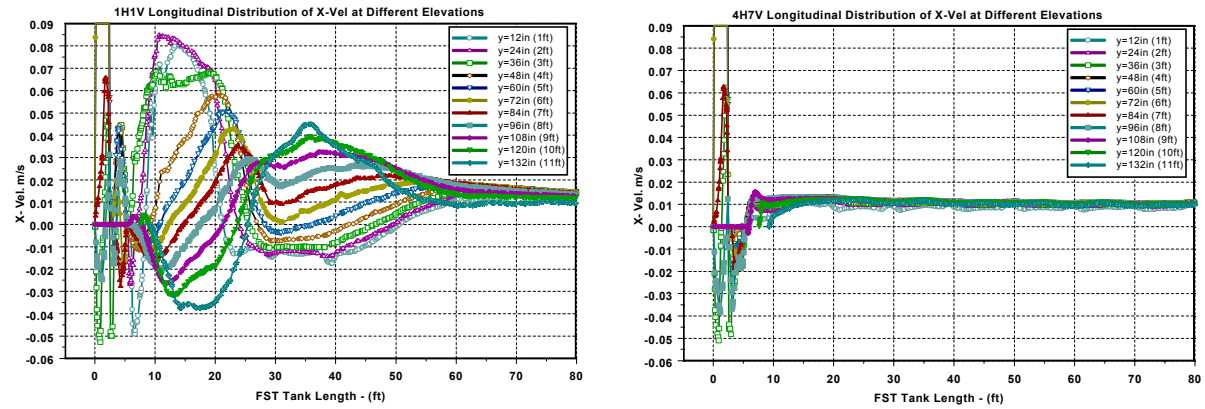


Figure 8. Model predictions for X-velocity as a function of tank depth. a) 1H1V case, b) 4H7V case as shown in figure 6. Units (m/s)

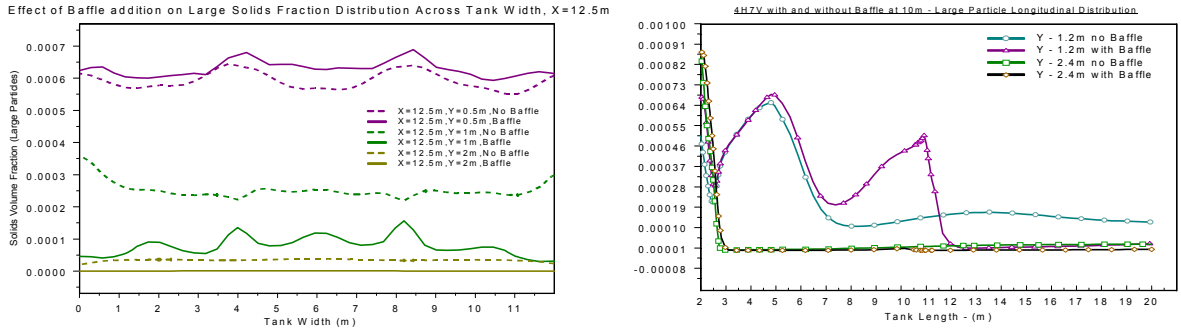


Figure 9. Effect of baffle addition on large fraction distribution relative to depth across the width of the tank at a location past the baffle. 1) Across tank's width, b) Along tank's length.

3. CONCLUSIONS

A 3D CFD program was set up to evaluate the effects of different inlet and in-tank baffle configurations on the hydrodynamic behavior of Gould II type FSTs. A flocculation model was incorporated that resolved the parameter of flocculation at the local grid size rather than being used as an average term. All settling characteristics were locally determined to be used as input parameters in the model. A 4H7V inlet slotted baffle and an in-tank baffle at 10 meters were selected as the optimum configuration to optimize process performance and minimize effluent SS. The simulations demonstrate that the 4H7V case ensures plug flow pattern immediately after the baffle which promotes high quality effluent from the FSTs. The in-tank baffle also showed reduction in recirculation and avoidance of re-entrainment of already clarified liquid into the inlet zone improving the overall fluid velocity and providing zones of further flocculation. This enhances the overall settling efficiency of the SS as seen in figure 9. The study heralds the notion that that a well calibrated CFD model can be used to optimize clarifier design and improve their overall performance.

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WASTEWATER MANAGEMENT IN NICOSIA PROVINCE, CYPRUS, WITH THE USE OF MULTI-CRITERIA METHODS

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Abstract: The paper presents a wastewater management proposal for four agricultural municipalities in the Nicosia's province in Cyprus, Lithrododa, Mathiatis, Analioda and Sia. A number of both centralized and decentralized management solutions are examined. The existing situation is recorded and data are gathered to get an accurate picture of the study area. Several proposals of alternative scenarios for the selection of the optimal layout of the treatment units and their entailing costs are presented. The choice of the "optimal treatment method" and the "optimal scenario" is achieved by the combined use of the AHP and TOPSIS multi-criteria methods with the application of financial, environmental and social criteria.

Key words: wastewater, urban, management, decentralized treatment systems, multi-criteria analysis.

1. INTRODUCTION

This paper examines a proposed urban wastewater management scheme in four rural localities in the Nicosia District. This includes the village of Lythrodontas, Mathiatis, Analiontas and Sia, all in a distance of 6 to 10 km between them. The option of a decentralized management solution for the study area examined given a number of important relevant considerations: (i) landscape and relief of surrounding area, (ii) distance separating the settlements, (iii) possibility to reuse the treated wastewater locally, (iv) disadvantages of pipeline installation over a long stretch of land, (v) environmental concerns – controlling and regulating discharges. The option of a centralized management solution is also examined with the aim of reducing construction and maintenance costs. The choice of the "optimal treatment method" and the "optimal scenario" is achieved by the combined use of the AHP and TOPSIS multi-criteria methods with the application of financial, environmental and social criteria.

2. URBAN WASTEWATER MANAGEMENT POLICY IN CYPRUS

2.1 Sewage Management

Today, the Cyprus government policy with regard to water conservation focuses on the maximum exploitation of other non-conventional sources of water, such as recycled water which is supplied in good quality. In its effort to promote and implement a wastewater management plan is line with the EU directive 91/271/EU Cyprus has set up the Sewage and Recycling Agency under the authority and supervision of the Water Development Board. The agency undertakes the responsibility to set up and see that the National Plan for the implementation of the directive materializes. To this effect, the agency is charged with task of planning, designing promoting the construction of sewage systems and submitting progress reports to the European Committee. The above course of action is taken in coordination with the Urban Sewage Councils (sewage management Councils alo-

nes with each township, municipality or community involved contribute through their locally appointed councils to the implementation of the Directive). The implementation programme comprises 36 rural communities/municipalities, among which is Lythrodontas. In cases where the surrounding relief allows it, the water Development Board proposes joint management of two or more communities and this for environment and other economic reasons as the present paper examines. Figure 1 is a map illustrating the Cyprus implementation programme of the Urban Wastewater Treatment Directive 91/271/EEC.

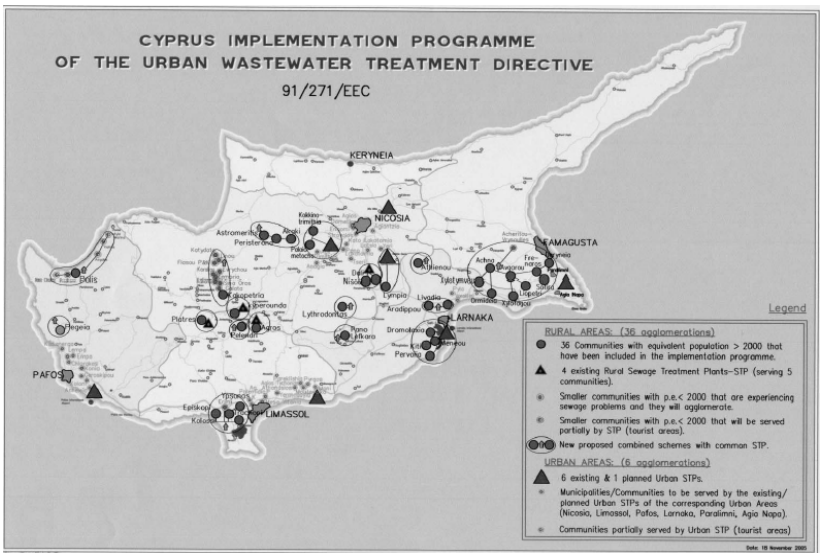


Figure1. Map of Cyprus implementation programme of the Urban Wastewater Treatment Directive 91/271/EEC.

2.2 Level of use recycled wastewater.

In Cyprus the treated urban sewage is used for irrigation and water table enrichment (Pafos plant). Crops irrigated using this method both for human and livestock consumption includes wheat, barley, com and clover. Treated wastewater is also used for the irrigation of green areas, golf courses, football fields etc. When, there is low demand during the winter season the treated wastewater is discharged into the sea as is the case in Larnaka and Limassol. The bar chart in Figure 2 shows the percentage distribution of recycled water (Information gained from Water Development Board, republic of Cyprus)

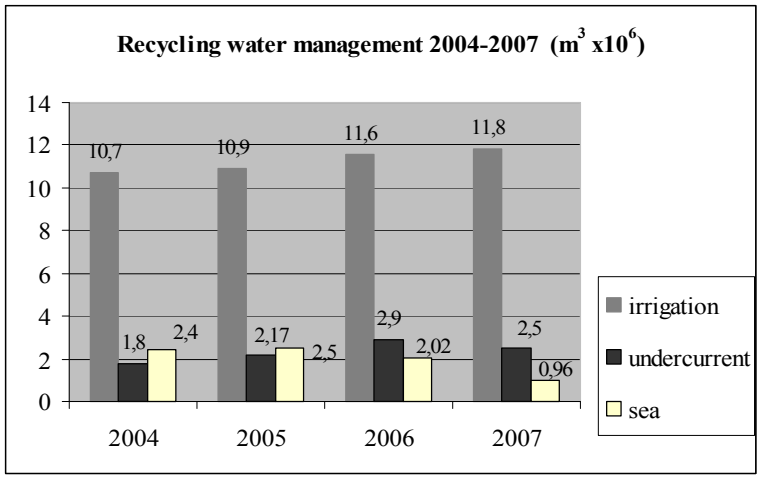


Figure2. Distribution of recycled water in Cyprus, years 2004-2007

Table 1, below, gives projected quantities of recycled water up to 2025, given Cyprus future objective to reclaim water from secondary sources, thus fulfilling the harmonization criteria as

stated in the European Directive. In Table 2 we have the proposed charges for treated water as set by Water Development Board.

Table 1. Projected quantities of recycled water in Cyprus

Treatment Stations	2012		2015		2025	
	Quantity	Increase	Quantity	Increase	Quantity	Increase
Cities	$46x10^6$	(58%)*	$51\ x10^6$	(10%)**	69,000,000	(26%)***
Rural communities	$13\ x10^6$	(95%)*	$14\ x10^6$	(7%)**	16,000,000	(12.5%)***
Total	$59\ x10^6$	(43%)*	$65\ x10^6$	(9%)**	85,000,000	(23.5%)***

** Increase in terms of 2007 ** in terms of 2012, *** in terms of 2015*

2.3 State contribution to urban wastewater management schemes.

Within the general framework of economic support for rural areas the state contribution to the above scheme is as follows:

For communities with a population of over 2001 the state contributes 80% towards capital expenditures. This cover the construction costs for: a) the sewage collection networks, b) the network of pipes and pump stations for the transport of sewage to the treatment plant, c) the storage tank of the treated water and d) the main pipe from the storage tank to the irrigation network. For communities with a population between 501-2001 the state contributes 90%, communities with a population between 201-500 the percentage get in 95% and for communities with up to 200 people the state contribute 100%. The remaining part of the capital expenditures along with the running costs is contributed by the community through loans repayable by the users in the form of sewage charges.

Table 2. Charges for treated water

Usage	Suggested charge euro cent/ m^3	
	Recycled water	Water from natural sources
Arable area, for agricultural production	5.12	16.23
Persons, for agricultural production	6.83	18.79
Play area	15.37	34.17
Hotel's park irrigation	15.37	34.17
Golf courses irrigation	20.50	34.17
Pumping underground water, enriched by recycled water	7.69	
Overbuy of above mentioned supplies	markup 50%	
Communal park and green areas for rural communities, where the WWTP is within their limits and the amount do not transcend 10% of the approved one	gratuitous	

3. FEATURES OF AREA UNDER STUDY

3.1 Geographic features

The communities under study are semimountainons, located toward the center of Cyprus in the south-eastern part of the Nicosia district, 25 kilometres from the capital (Nicosia). Their distance, from one another, ranges from 4 to 6 km, with a total population of 6900 people. The settlement of

Lythrodontas is located 280m above sea level at the foot of the Kionia Mountain, which is an extension of the Troodos mountain range, the biggest on the island. West of the settlement is the Machaira Forest and north-west is the Lythrodonta Forest which has been designated as a National Forest Park since 2003. The river Amylos flows through the settlement. The settlement of Mathiatis is located 4 km northeast of Lythrodonta with the highest elevation (360m) of all the rest. The river Giallias flows round the settlement. The relief surrounding the settlement of Sia is made up of gentle slopes with the exception of the south-east side adjacent to Mathiatis the Mountains terrain of Xylia. Three rivers flow across Sia: Vrisis, Kilias and Giavris. This settlement is 8.5km from Lithrodonta and 270m above sea level. The settlement of Analionta is located 6 km from Lithrodonta and Mathiati and 10.5 km from Sia. Its elevation is 230 m with two rivers flowing through.

3.2 Existing state of infrastructure and sources of polution

The communities are integrated into state programmes aiming to resolve the sewage issue. At present, works are being carried out with regard to the sewage network while the biological treatment is under study in Lythrodonta using Aerated Filter System. The main source of pollution in the area stems from the accumulated sewage from the fast growing settlements. All the surrounding communities have experienced a rapid population growth. The biggest population growth (by 80% between of 2001-2007) is to be found in Analionta due to an urban decentralization in the Nicosia district. The community of Lythrodonta has grown by 52% during the same period. Within the confines of Mathiati there are two disused mines from where gold and pyrites were extracted. A quarry from where inert materials are extracted is to be found in Sia. It should be noted that due to prolonged drought of the recent years the once fertile land in the area remains unexploited.

4. PLANNING PROCEDURE FOR A COMPLETE SEWAGE MANAGEMENT SYSTEM

4.1 Methodology

The procedure that was applied in drafting the scheme is as follows: 1) The existing situation in the given area is recorded (table 3). For each settlement the following are recorded – (i) the sources of pollution originating from activities in the primary, secondary and tertiary sector, and an estimate of the volume of liquid waste discharges, (ii) the existing state of the sewage system and its management in each settlement, (iii) the existing natural recipients of wastewater, (iv) land use and land surface area, (v) activations and irrigations demands, (vi) the geological base. 2) The necessity for infrastructure implementation in accordance with the EU Directive 91/271, the state legislation and the particular local environmental concerns is investigated. 3) The possible recipients and the specifications are defined. 4) Alternative scenarios and their costs are formulated. 5) Initially the optimum location for new treatment plants is selected based on the following criteria: (i) character of the area – tourism, agriculture, archaeological sites, land under building restrictions, (ii) land use: cultivations, vineyards, salt lakes, pastures, follow land, marshland, forests, (iii) landscape and topography: defining the area in terms of elevation, assessing settlements in terms of elevation differences and slope, (iv) Type of soil, permeability, electrical conductivity, depth of water table, problems with water salinity (v) availability of the required surface, (vi) distances of the settlements, (vii) existence of a natural reservoir, (viii) total cost (construction, operation, maintenance of network and treatment plant, pipes, fuel, pumps, (ix) proximity to recipient of treated materials. 6) Next stage is the selection of a suitable system for swage treatment based on the following criteria: (a) Financial: Costs for construction, operation- maintenance, fuel, land, (b) Efficiency of system (%) for BOD COD, totally suspended solids (TSS), total phosphorus (TP) and total nitrogen (TN), (c) Social: aesthetic effect of the system with regard to the natural features in the specific area,

annoyance caused by the system affecting people lives through unpleasant smells, toxic air pollutants, noise pollution, insects. On the positive side is social approved and consensus.7) Ultimate, the selection of the most feasible scenario is made given the economic operational and social criteria described above.

Table 3. Existing situation of communities

Community	Lithrodontas	Mathiatis	Sia	Analiontas
Number of settlements	1	1	1	1
Area (10^3 m^2)	42,500	28,000	22,000	37,500
Equivalent Population	4,000	700	600	540
Landscape and topography	Semi-mountainous			
Type of soil	Clay, Hornfels			
Main cultivations	Olive trees, potatoes, wheat, barley, alfalfa	Olive trees, potatoes, wheat, barley, alfalfa	Olive trees, potatoes, wheat, barley	Olive trees, wheat, barley vegetables
Heath (10^3 m^2)	-	-	-	-
Cultivated land	80%	90%	95%	100%
Forests	20%	10%	5%	-
sewage network	Yes		pluvial	
Waste Water Treatment Plant	4000 E.P.		cesspools	
Reservoir	Water tank		Land	

4.2 Formulating alternative scenarios

Immediate compliance with national and European regulative demands is required only for Lythrodontas. For the other communities the study is carried out for environmental reasons, mainly due to the necessity for water conservation, indispensable for the Cyprus Island. Another reason is the agricultural exploitation of the treated sewage over a large cultivated area. As shown in (table 3) these communities may well constitute possible recipients of treated sewage given their extensive arable land.

Based on the criteria 1 to 3 which refer to paragraph 4.1 and the apparently feasible or not joint management option and given that settlement linkup with the system is attainable, the distances of settlement and recipients and the required degree of treatment, the following scenarios were formulated: (1) Scenario A– Centralized solution: Construction of two units. Joint management of three settlements. The first Wastewater Treatment Plant (WWTP) based in Lythrodonta with provision for expansion will serve also the settlements of Mathiati and Analionta. The second WWTP will serve just Sia, since a pipe link between Lythrodonta and Sia is not feasible due to the obstacles of the Xylia mountain top and the Delikipos Forest. If Sia links with Mathiati and then with the Lythrodonta WWTP, again pumps are needed given the difference in elevation between Mathiati and Sia pumping stations. For the Sia Unit the following treatment methods were studied: (a) SBR, (b) MBR, (c) constructed wetlands, most suitable for small scale units (Gratziou et al, 2006). (2) Scenario B- Decentralized solution: Construction of four units. Each community manages its sewage independently, that is through a WWTP constructed in each settlement. For the same reasons, in this case too, the above-mentioned treatment systems were examined. (3)Scenario C– Decentralized Solution: Construction of three units. Joint Management of Lithrodonta and Mathiati sewage, with provision for unit expansion. In Analionta and Sia there will be separate sewage management.

4.3 Alternatives evaluation methodology

The usual procedure performed by the authorities for the selection of the appropriate process for the treatment of the wastewaters revolves invariably around the cost factor which may not always lead to the optimal solution. A wrong solution will inevitably result in non-viable and inefficient investments that are likely to have a negative impact on the environment (Anagnostopoulos et al 2008). In order to meet decision Makers (DMs) requirements a four level hierarchy is constructed

(Fig. 3). The first level is occupied by the main goal of the analysis, while the last by the evaluation alternatives. Respectively the second and the third level are formed by analysis objectives and subobjectives.

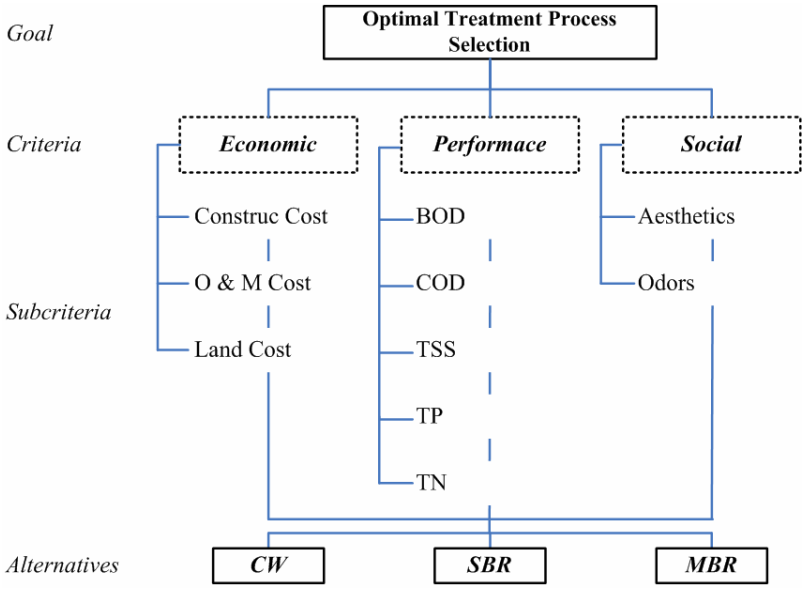


Figure 3. Decision Problem Hierarchy.

In the present paper the combined use of Analytic Hierarchy Process (AHP) and the Technique for Order Preference by Similarity to ideal Solution (TOPSIS) is performed in order to evaluate treatment process alternatives according to DMs preferences. Developed by T.L. Saaty (1977), AHP is a multicriteria method for dealing with complex decision-making problems in which many competing alternatives (projects, actions, scenarios) exist. The alternatives are ranked, depending on how they contribute in achieving an overall goal, forming matrices of pair wise comparisons. AHP is based on a hierarchical structuring of the elements that are involved in a decision problem. The hierarchy incorporates the knowledge, the experience and the intuition of the decision-maker for the specific problem (Forman & Selly 2002, Banai-Kashani, R., 1989).

$$\left(A - \mathrm{I}_{\max} I\right) \times w = 0 \tag{1}$$

$$c_i = \frac{S_i^-}{S_i^+ + S_i^-},\ 0 \le c_i \le 1 \tag{2}$$

$$S_i^+ = \left(\sum_{j=1}^{m} w_j^p \times \left(x_{ij}^s - x_j^{s+} \right)^2 \right)^{1/2} \tag{3}$$

$$S_i^- = \left(\sum_{j=1}^{m} w_j^p \times \left(x_{ij}^s - x_j^{s-} \right)^2 \right)^{1/2} \tag{4}$$

Relative priorities of the hierarchy's objectives and sub-objectives are obtained comparing every element in a branch against of its peers with respect to the parent node. Technically this is achieved forming pairwise comparison matrices A_{ij} for every one of the hierarchy's branches. Every element a_{ij} of the matrix represents the importance of the row element i over the element in the column j. AHP provides DMs with the nine level fundamental scale of preferences in order to establish the measurement of the dominance relation. Respectively the inverse relation a_{ji} equals to $1/a_{ij}$. Relative priorities w are then established as the right principal eigenvector $\mathrm{I}_{\max}$ (Eq.1) of the

pairwise comparison matrix A_{ij} (Saaty 1995). Alternatives rankings are obtained using the estimation of the relative closeness index c_i (Eq.2) according to TOPSIS methodological framework for synthesizing DMs preferences (Yoon & Hwang 1995). The measure of relative closeness, for every on of the alternatives, is estimated via normalization of the proximity and the remoteness from the ideal and anti-ideal solution respectively according to the equations 3 and 4. In the present compact Sequencing Batch Reactor (SBR), Membrane BioReactor (MBR) and Constructed Wetlands (CW) are considered as alternative treatment options

5. RESULTS

The proposed multicriteria analysis performance and the corresponding sensitivity analysis implementation lead to the selection of the MBR technology as the most appropriate, when the construction of a treatment plant is considered in each one of the Mathiatis, Sia and Analiontas communities. The above is conduced mainly due to the increased land requirements cost in Cyprus (20,000€ per 1000 sqr meters as appraised by the land registry) and to the stringent discharge quotas set by the state when reuse options for the effluents are taken under consideration. It is noticed that CW option is in any case ranked second. The final scenarios, therefore, are modified as follows: (i) Scenario A: Extension of the WWTP in Lythrodontas to supply an additional daily 250 m3, an WWTP in Sia using the MBR system. (ii) Scenario B: WWTP Lythrodonta such as it is and the development of MBR systems for the other three settlements (iii) Scenario C: Extension of the WWTP Lythrodonta for an additional daily supply of 140 m3 with MBR systems for Sia and Analionta. Table 4 indicates the total costs for the initial expenditure of the project as well as those for operation-maintenance of the three scenarios. It is obvious that Scenario A is the most desirable.

Table 4.Total costs of Scenarios

Total Cost	ScenarioA	ScenarioB	Scenario C
Initial Cost	580,200 €	676,760 €	600,000 €
Operation and maintenance	48,300 €	72,400 €	52,200 €

6. CONCLUSIONS

As already mentioned the communities of Mathiatis, Sia and Analiontas are not included in the categories the European directive 91/271/EU requires the installation of urban wastewater treatment plants. However, the Republic of Cyprus, in an effort to alleviate the problem of water shortage, is willing to achieve the most feasible management of its rare water recourses for communities with a population between 501 – 2000 by contributing 90% of the initial costs for wastewater management. The proposed solution can offer environmental as well as financial benefits to the communities since the generated sewage will no longer be deposited into the ground while the recycled water will be used for sale to various recipients. The residents of the communities will be charged the extra operational cost. However, this will be reduced through the revenues resulting from the sale of recycled water. It is worth noting that the treated wastewater is more beneficial to cultivations and to farmers in that large quantities of fertilizers will be avoided with greater crop yields. Moreover, the farmer will be asked to pay 11.96 cents per m3 less when using recycled water for irrigation instead of fresh water.

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DEMAND FOR AND INTEREST IN DECENTRALISED WASTEWATER TREATMENT AND RE-USE: AN EMPIRICAL ANALYSIS OF STAKEHOLDER VIEWS

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Abstract: Decentralised wastewater treatment technology and local re-use of treated wastewater is one of the most promising ways to tackle water scarcity problems in water scarce countries, such as Jordan, in the future. However, in the past, it has proved very difficult to implement appropriate technologies for wastewater re-use due to institutional barriers and public opposition. In this paper, the focus is directed away from technical aspects of wastewater treatment and re-use (WWT&R) towards investigating the possibilities and impediments of implementing decentralised solutions from a social perspective. The overall aim is to assess the perceived demand for and views towards WWT&R among different stakeholders. Investigating the social side of WWT implementation is considered to be important in order to engage relevant stakeholders in the decision-making process, and take their concerns and views into account in the decision-making process. A case study was carried out in the Jordanian part of the Lower Jordan River Basin taking different stakeholder groups that would be involved in the decision-making process into account (e.g. government officials responsible for wastewater at different levels of administration, representatives of municipalities, and end-users). This enabled us to investigate views and perceptions regarding decentralised treatment plants and re-use from different perspectives. As such it helped to find out in what respects there is convergence and in what aspects there is divergence of views. Data was collected by means of qualitative face-to-face interviews with government officials and municipality councils, and focus groups with end-users in the villages. Our findings confirm that there is urgent demand for treated wastewater in Jordan. Overall, stakeholders were more positive than negative towards decentralised WWT&R. It was perceived that WWT&R would solve many of the problems Jordan currently faces, such as pollution of freshwater resources, social and health problems resulting from overflowing cesspits and low income among farmers. Interviews and focus groups revealed two main obstacles towards the introduction of decentralised WWT&R: 1) financing of the plant, and 2) a lacking sense of responsibility among municipality councils and residents.

Key words: wastewater treatment and re-use, Jordan, public participation, acceptance, concerns, SMART.

1. INTRODUCTION AND AIMS

Decentralised wastewater treatment technology and local re-use of treated wastewater is one of the most promising ways to tackle water scarcity problems in water scarce countries, such as Jordan, in the future. However, in the past, it has proved very difficult to implement appropriate technologies for wastewater re-use due to institutional barriers and public opposition (Dolničar and Saunder, 2006, Hurlimann and McKay, 2005).

In this paper, the focus is directed away from technical aspects of wastewater treatment and re-use (WWT&R) towards investigating the possibilities and impediments of implementing decentralised solutions from a social perspective in the Jordanian part of the Lower Jordan River Basin. The aim is to assess the perceived demand for and views towards WWT&R among different stakeholders. While a range of technologies for the wastewater treatment and re-use are readily available to suit different climatic, geological and topographical conditions, investigating the non-technical social side of their implementation is considered to be important in order to 1) engage

relevant stakeholders in the decision-making process, 2) take concerns into account and 3) tailor respective solutions in a way that suits end-users' needs.

In the past decade, the notion to use treated wastewater to supplement conventional water sources in the water-scarce regions of the world has experienced a sudden increase in recognition, as the finite nature of natural water resources has become increasingly apparent. Experts from the US and Australia suggest a range of options for which treated wastewater can be used. These re-use options are considered to generate a range of potential benefits for the communities, the environment and freshwater resources.

Despite these advantages, the general public has a number of concerns with respect to treated wastewater. These are mainly related to the perceived low quality of treated wastewater and potential health problems (Marks, et al., 2002, Bino, M.J., 2006 and Dolničar and Saunder, 2006). Even though the elicitation of the public's perceptions is essential for the successful implementation of WWTP, the limited literature on public participation has merely focussed on lacking acceptance, one of the main impediments for implementing wastewater re-use technology (e.g. Haruvy,1996, AQUAREC, 2006). However, this does not yet answer the question whether decentralised treatment and reuse would be implementable.

In order to get a deep insight into the perceived demand for and concerns with decentralised WWT&R a case study was carried out in the Jordanian part of the Lower Jordan River Basin taking different stakeholder groups into account (for study area see Figure 1). It was felt that, apart from end-users, all other stakeholders that would be involved in the decision-making process, such as government officials responsible for wastewater at different levels of administration and representatives of municipalities should be included in the study. Approaching all relevant stakeholder groups allows an investigation of views and perceptions regarding decentralised treatment plants and re-use from different perspectives. As such it helps to find out in what respects there is convergence and in what aspects there is divergence of views. Such an analysis can also be considered as an important prerequisite for a successful implementation of such solutions. Specific aims of this research task are to:

- assess the interest in setting up decentralised wastewater treatment plants (WWTP).
- assess the perceived demand for and interest in re-using treated wastewater from such plants understand the concerns related to decentralised WWT&R.
- test for end-users' willingness-to-pay for treatment and re-use services.
- compare interests/concerns between stakeholder groups.

2. SAMPLING AND DATA COLLECTION

Considerable effort was spent to identify stakeholders in relevant institutions and citizens to gather information on demand for and acceptability of WWT&R. The aim was to include administrators concerned with wastewater at the central government and governorate level, representatives of municipalities or village councils in rural areas that are not yet connected to wastewater systems as well as households and potential end-users of treated wastewater in these communities. Initially, a range of experts at the Ministry of Water and Irrigation (MWI) and the Water Authority of Jordan (WAJ) who are concerned with wastewater treatment were selected in collaboration with project partners from the Jordanian consulting company EcoConsult. Further relevant government experts at the governorate level were recommended by MWI and WAJ representatives. Since available resources constrained representative sampling of village councils and users, data collection was focused on three villages that had previously been selected within the SMART project as potential test sites for decentralised WWT&R, the villages Ira, Yarqa and Al-Ramah in the Lower Jordan River Basin. These villages were considered to require decentralised WWT&R as it would be expensive to connect these villages to central wastewater systems due to

due to their remoteness and mountainous topography (van Afferden and Daoud, 2008). End-users were selected together with the respective village councils and included an equal distribution of gender and age groups. End-users were residents who would get connected to the sewerage system and/or would re-use treated wastewater. Figure 2 illustrates the relevant stakeholder levels concerned with wastewater treatment and re-use in Jordan. The identification of the various stakeholder groups ensured stakeholder representativeness rather than socio-economic representativeness (e.g. representativeness in terms of age, income, etc.) throughout the Jordanian part of the Lower Jordan River Basin, as this was considered to better illuminate the range of views and perceptions regarding WWT&R.

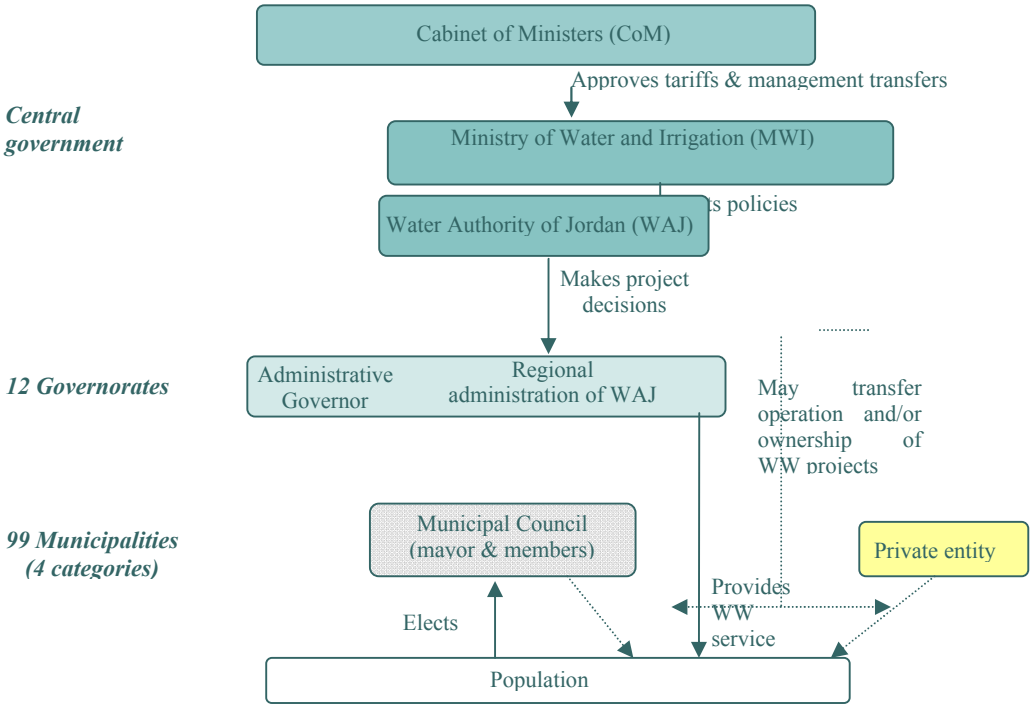


Figure 1. Responsibilities and stakeholders in the context of wastewater treatment. (Source: own compilation)

Qualitative interviews were considered to be the most appropriate method to obtain information from experts at the governmental and governorate level as well as of village councils, as they are particularly suited to describe individual perceptions. The data collection took place in form of semi-structured face-to-face interviews. In order to investigate end-users' views and perceptions regarding decentralised WWT&R it was decided to use focus groups. This data collection mode was considered useful because it enabled us to provide participants with relevant information on decentralised treatment plants and re-use, and encouraged participants to discuss issues introduced by the moderator and to raise issues that are of relevance to them. In each group several questions were discussed among 8-15 participants. The number of interviews and focus groups, involved institutions and dates of implementation are summarised in Table 1.

Table 1. Number and dates of interviews and focus groups

Stakeholder group	**Institution**	**No of interviews**	**Language**	**Date**
Central Government	MWI & WAJ	3 face-to-face interviews	English	January 2008
Governorate	Irbid & Balqa	4 face-to-face interviews	English	January 2008
Village councils	Ira, Yarqa & Al-Ramah	3 face-to-face interviews	Arabic	April 2008
End-users (residents)	Ira, Yarqa & Al-Ramah	3 focus groups	Arabic	April 2008

3. SAMPLING AND DATA COLLECTION

This section presents the findings from the interviews and focus groups with respect to (1) perceived advantages of decentralised WWT&R, (2) concerns regarding decentralised WWT&R, and (3) end-users' willingness to contribute to the costs of the services associated to decentralised WWT&R.

3.1 Advantages of decentralised WWT&R in rural Jordan

Views towards WWT&R in rural areas were positive in all stakeholder groups, and there was agreement that decentralised wastewater treatment would protect freshwater sources. Stakeholders at the Government/Governorate level looked at the big picture, e.g. cost-effectiveness, whereas stakeholders from municipalities voiced local benefits, such as preventing odour and social problems between neighbours (see Table 5). There was a strong interest among village councils and end-users to change the unsatisfactory situation related to overflowing cesspits. Note, that the interview and focus group discussions sometimes shifted from the focus on decentralised wastewater treatment plants to advantages of wastewater treatment in general. This section includes arguments for both decentralised and general wastewater treatment.

Table 2. Perceived advantages of decentralised WWT&R

Government/Governorate	**Municipality**	**End-users**
- Protection of groundwater - Replacement of conventional fertilizer - Increase area of cultivated land - Increase yield/ farmers' income - Reduce subsidies paid to farmers - Cost-effectiveness - Suited to mountain areas - Job opportunities	- Reduce groundwater depletion - Stop leakage and overflow from cesspits - Prevent social problems - Reduce flies and health problems	- Prevent contamination of rainwater wells - Reduce social problems - More convenient - Reduce health problems - Cheaper - Reduce odour

Government and Governorate representatives seemed to be well informed about the advantages of decentralised WWT&R when comparing their statements to advantages stated by the literature. However, a range of advantages revolved around expected improvements to the rural economy (e.g. employment for residents, increasing treatment in rural Jordan. It also shows that it is important to take the views of local stakeholders and end-users into account.

3.2 Concerns regarding decentralised WWT&R

Overall, perceived advantages seemed to outweigh the perceived concerns. While a number of minor concerns were raised, specifically on the municipal and end-user level (e.g. odour and flies), bigger issues, such as risk of leakage and environmental pollution were mainly raised by the interviews held on the Government level and to a far lesser extent by end-users. Interviews and focus groups revealed that lacking affordability was perceived to be a key problem in all stakeholder groups. Both village councils and end-users were concerned that they might have to take responsibility for operating and monitoring the plant (see Table 6 for an overview of concerns).

Table 3. Perceived concerns regarding WWT&R

Government/Governorate	Municipality	End-users
- Monitoring - More staff - Leakage - Affordability	- Odour - Flies - Responsibility - Affordability	- Odour - Risk of penetration - Responsibility - Affordability

While the stakeholders voiced some concerns, there was no strict opposition in any of the interview and focus groups, and the range of advantages related to treated wastewater indentified in Section 3.4 shows that stakeholders are generally in favour of decentralised treatment plants. This is an interesting finding, as one would have expected stronger disagreement among end-users. One explanation for this positive result can be found in Hartley (2006) who identified a range of conditions that enhance public acceptance. Table 7 lists these conditions and indicates for each condition whether it is met according to end-users' perceptions. The table reveals that the majority of conditions were met. Furthermore it shows that stakeholders are very interested in obtaining all the benefits associated to decentralised WWT&R, but at a low cost.

Table 4. Conditions that increase public acceptance (Source: based on Hartley, 2006, p. 116)

Type of condition	**Condition met as perceived by end-users**
▪ Degree of human contact is minimal	*
▪ Protection of public health is clear	✔
▪ Protection of environment is a clear benefit of re-use	✔
▪ Water conservation is a clear benefit of re-use	✔
▪ Cost of treatment and sewerage systems is reasonable	No
▪ Perception of wastewater as a source for recycled water is minimal	*
▪ Awareness of water supply problems in the community is high	✔
▪ Role of reclaimed water in the overall water supply scheme is clear	✔
▪ Perception of the quality of reclaimed water is high	✔
▪ Confidence in local management of treatment plant is high	No
* issue not raised in the focus groups	

3.3 Financial support for WWT&R from end-users

The focus groups were also used to test whether end-users have a sense of responsibility when it comes to financing the services provided by the wastewater treatment plant. Hence, participants were prompted at the end of the discussions to state whether they would be willing to pay for 1) using treated wastewater, e.g. for irrigation, and 2) for the connection to the sewerage system.

Reactions to the question whether participants would pay a small amount for treated wastewater varied among focus groups: In Ira, none of the participants would pay anything, not even the price they currently pay for treated wastewater in irrigation (10 fils/m³), because they felt that the service should be paid for by the Government. In Yarqa, some participants stated that they would be willing to pay up to 0.5 JD/m³ in order to enhance irrigation of olive trees. In Al Ramah, the majority of participants were positive towards paying for treated wastewater: Some participants stated that they would pay 10 fils/m³ and one participant was willing to pay 100 JD/month in order irrigate his crops. However, there was also opposition towards paying for treated wastewater: "*No, I will not even pay 10 fils to get polluted water while I pay 400 JDs to get water from Husban's stream*". The payment question did not relate the payment to the amount of treated wastewater that would be used.

Willingness to pay for sewerage connection was strongly anchored to the amount of money people currently pay for tanker disposal. Since some households do rarely pump their cesspits, willingness to pay was very low or non-existent: "*I do not pump my cesspit, or maybe I do every 4 or 5 year, I am not willing to pay per month*". In total six out of X participants stated that they would be willing to pay up to 10 JD per year.

This is an interesting finding, as it shows that the actual beneficiaries of the wastewater treatment plant show very few signs of personal responsibility with respect to financing the sewerage and treatment service, despite the fact that end-users were aware of the range of benefits associated to WWT&R. Hence, considerable effort needs to be spent on enhancing the sense of responsibility and stressing that payments by users are a prerequisite for the implementation of a decentralised treatment plant. A sophisticated Willingness-to-Pay study with end-users would help to educate people and better assess ability and willingness to pay. On this basis it would then be possible to design a realistic and informed tariff structure.

4. CONCLUSIONS AND RECOMMENDATIONS

This study confirms that there is urgent demand for treated wastewater in Jordan. Among all stakeholder levels, main uses for treated wastewater were seen in the agricultural sector, in particular for irrigating fodder, fruit and vegetables, and as a substitute for conventional fertilizer. Stakeholders also appreciated the idea to introduce decentralised WWT&R to rural areas. It was perceived that decentralised WWT&R would solve many of the problems Jordan currently faces, such as pollution of freshwater resources, social and health problems resulting from overflowing cesspits and low income among farmers. However, financing issues, risk of leakage, monitoring, odour and responsibility issues were major concerns.

Contrary to similar research that found strong initial resistance towards re-use of treated wastewater in the general public due to concerns regarding water quality health risks (e.g. Russel and Lux, 2006), in this study, all stakeholders were clearly more positive than negative towards decentralised WWT&R. This demonstrates the utmost urgency for additional water sources as perceived by all stakeholders, and might explain why "low quality" of treated wastewater for the irrigation of crops was not raised as an issue of concern. This finding is in line with Po, et al.'s (2003) notion that awareness of water shortage problems and the need to conserve water leads to higher acceptance to re-use wastewater.

Interestingly, concerns related to re-use of wastewater in the US and Australian literature do not seem to apply to Jordan (no concerns were raised about low quality of treated wastewater). However, we found that that the main obstacles towards the introduction of decentralized WWT&R are financing problems and a lacking sense of responsibility among municipality councils and residents.

Even though all stakeholder levels appeared to be interested in WWT&R, attention needs to be paid to the concerns raised by end-users. Some of the concerns, such as odour and flies associated with WWTPs, may be based on misconceptions. However concerns related to financing and the responsibility for operation are serious matters that complicate the decision-making and implementation process. In order to ensure successful implementation of decentralised WWT&R it is important to address all issues of concern and to identify solutions. The interviews and focus groups conducted in this study should be regarded as an initial step in a public participation process. Further engagement with the public is necessary alongside the entire decision-making process (AQUAREC, 2006 and US EPA, 2004).

The study also revealed that stakeholder representativeness was important. While the existing literature on acceptance literature mainly focuses on the general public's (end-users') views vis-à-vis reuse only, the study shows that it is useful to take the views and perceptions of different stakeholders into account as there is not necessarily full convergence. According to our findings, all stakeholders were interested in WWT&R and all of them also had concerns. While there were a lot of common advantages and concerns among all stakeholder groups, there were also some

significant divergences. Information on these divergences is important for a successful decision-making process.

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FIELD STUDY OF STORMWATER TREATMENT SYSTEMS AND CATCH BASIN INSERTS FOR THE REMOVAL OF POLLUTANTS FROM URBAN RUNNOFF

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Abstract: Non-point source (NPS) pollution, unlike point source pollution from domestic and industrial and wastewater treatment plants, comes from many diffuse sources. It is caused by the movement of rainfall or snowmelt that picks up and carries pollutants to receiving waters. NPS pollution represents the main cause of the contamination of many rivers, streams and other water bodies in the United States. Best management practices (BMPs), e.g., oil and grit separators, grassed swales, vegetated filter strips, retention ponds, catch basin inserts, are intended to reduce NPS pollutants including those from highway runoff. A catch basin insert (CBIs) is a device that reduces stormwater pollution from the runoff, without requiring any land use. It is attached to the entrance of or mounted inside of a catch basin. Our studies focused on two major BMPs—CBIs and inlet treatment systems. Six commercially-available CBIs and two water quality inlet treatment systems were tested at field sites of New York State Department of Transportation. The CBI study focused on the installation characteristics, durability and maintenance of the catch basin inserts, as well as whether the inserts can be conveniently, safely, and economically installed and maintained. Six commercially–available devices were installed along highways in Westchester, New York and were monitored. The CBIs were the Ultra-Urban Filter, Passive Skimmer, Stream Guard, FloGard Filter, Hydro-Kleen Filtration System, Silt Sack. The pollutant removal efficiencies at two existing water quality inlet treatment systems were also measured.

Key words: Stormwater treatment systems, catch basin insert, pollutants, urban runoff.

1. INTRODUCTION

Non-point source (NPS) pollution, unlike point source pollution from domestic and industrial and wastewater treatment plants, comes from many diffuse sources. It is caused by the movement of rainfall or snowmelt that picks up and carries pollutants to receiving waters. NPS pollution represents the main cause of the contamination of many rivers, streams and other water bodies in the United States.

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Our studies focused on two major BMPs—CBIs and inlet treatment systems. Six commercially-available CBIs and two water quality inlet treatment systems were tested at field sites of New York State Department of Transportation. The CBI study focused on the installation characteristics, durability and maintenance of the catch basin inserts, as well as whether the inserts can be conveniently, safely, and economically installed and maintained. Six commercially–available devices were installed along highways in Westchester, New York and were monitored. The CBIs

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were the Ultra-Urban Filter, Passive Skimmer, Stream Guard , FloGard Filter , Hydro-Kleen Filtration System, Silt Sack.

The pollutant removal efficiencies at two existing water quality inlet treatment systems were also measured. These systems were installed at Hauppauge and Bay Shore in Long Island, New York, respectively. Two auto samplers were installed at both locations to collect the influent and effluent samples during rainfall events for four seasons. The collected samples were analyzed to evaluate the pollutant removal performances of the systems. The parameters studied were conductivity, pH, total Kjeldahl nitrogen (TKN), total phosphorus (TP), total petroleum hydrocarbon (TPH), 5-day biochemical oxygen demand (BOD5), total suspended solids (TSS) and fecal coliform bacteria (FCB). The stormwater treatment systems were the Vortechs TM System and the V2B1TM System. The two devices were already installed in the field on RTE. 27A – Bay Shore, NY (VortechsTM) and RTE. 347 – Suffolk County, NY (V2B1TM). In order to meet the objectives of study, samples were collected at the same time from the inlet and the outlet chambers of each unit during the storm events and analyzed in the laboratory for the above mentioned parameters. The units were monitored for one year, over the four seasons, and the samples were collected for the "First Flush" and for the remaining part of each event.

1.1 Methodology

We studied two different types of technologies for the stormwater runoff problem. In one part of the project, two stormwater treatment systems were analytically-tested at the field scale for removal of certain parameters and relative cost effectiveness. The two full-scale stormwater treatment units were instrumented with sample collection devices and a sampling protocol adopted to collect runoff samples during storm events. Samples were collected from a location immediately before, and immediately following the treatment units simultaneously.

The second part of the project consists of qualitatively monitoring catch basin inserts for maintenance, durability, ease of installation and cost effectiveness, based upon monthly field inspection with photographical reports. For both parts of the project, samples and data collected in the field were stored, analyzed and evaluated at our laboratory. The installation details are reported, with descriptions of each site and the strategies adopted for the field monitoring.

1.2 Stormwater Treatment Systems

Vortechs$_{TM}$ and V2B1$_{TM}$ units were both installed at locations in Long Island, NY, by the NYS Department of Transportation and instrumented for this field study. The Vortechs$_{TM}$ system is located on the east bound RTE. 27A in Bay Shore at the entrance of a parking lot for a shopping center, while the V2B1$_{TM}$ is located on the west bound RTE. 347 in Suffolk County, just west of RTE 111. A sampling station was installed on each location of the stormwater units, and since both stormwater treatment systems have a very similar design and operation procedure, the same sampling systems was used to collect runoff at both locations. Two custom-built wooden storage boxes were used to house all the instruments utilized in the sampling systems of the study.

The stormwater sampling protocol consisted of two parts: "PART A" designed to collect the samples from the "first flush" of the storm event, where six samples were collected in the first thirty minutes of the event; "PART B" programmed with a longer time table to capture samples from the entire event. In this second part of the stormwater program, eighteen samples were collected in six hours, so that the entire program covered a total of six and half hours of the rain event. The sampling was triggered when was started when both the rain intensity would reach the 0.05 in/hr and the water level in the treatment device chamber reached the liquid level detector connected with the autosampler. Note that the sampling did not include flow measurement, and for that reason an accurate calculation of loading (i.e., a quantitative comparison) can not be made using the results

of the analytical results. Instead, a qualitative comparison is made by comparing the concentration of a particular parameter measured at the inlet and outlet at the same time.

Based on the weather forecast, prior to a storm event, the samplers were stationed at the site, calibrated according to manufacturer specifications and programmed to collect samples. Personnel would visit the monitoring station during the event to ensure that the collector started and at the end of the event to retrieve the collection info from the autosampler using a laptop with installed the Flowlink® software and bring the collected samples to the Polytechnic University laboratory (Brooklyn, NY) for analysis.

2. CATCH BASIN INSERT FIELD STUDY

This part of the project consisted of installing Catch Basin Inserts (CBIs) in the field and monitoring through four seasons, over 12 months. Every month the site was inspected, examining each CBI, removing the grating, and checking the condition of each CBI. To track the CBI performance, photographs and careful observations were taken, under different weather conditions and length of service. During each inspection the following information were recorded:

- Condition of area around the inlet, e.g., accumulation of debris, oil, dirt, etc.
- Structural integrity of the CBI, i.e., needs any repair or replacement.
- Condition of the sorbent material, if applicable.
- Extent of sediment trapped into the filter.

If damage to the CBI was noted, it was replaced with a new one and taken to the laboratory for further inspection. Also, if the sorbent material was saturated with oil, the device was replaced. If the accumulated sediment was more than the 50% of the filter's capacity, the filter was removed, cleaned, and the sediment was collected. Any sediment collected after a clean out of the filter or after a replacement was brought to the laboratory for gravimetric analysis.

Catch Basin Inserts (CBIs) were installed at locations in Westchester, NY, following the manufacturers' installation guidelines, as described below:

Ultra-Urban™ **–** The filter was installed in the Goldens Bridge Metro North station parking lot, just west of I-684. It is a drop-in device that does not require any special labor for the installation, but does require two people to lift and install. The CBI is supported by a stainless steel frame, furnished by the manufacturer and custom made because of the dimension of the catch basin. Two filters were placed next to each other to cover the inlet area of the catch basin.

FloGard Plus™ **–** As mentioned above, the filter was also installed in the Goldens Bridge Metro North station parking lot, just west of I-684, into a different catch basin. The device was frame mounted and because of the special dimensions of the catch basin, two filters were installed next to each other, in order to cover the catch basin inlet area. This is also a drop-in device and, once the grate was removed, no special labor was required for the installation. Two people are needed to lift the CBI.

HydroKleen™ **–** This filter was also installed in the Goldens Bridge Metro North station parking lot, just west of I-684, into 3rd catch basin. After the grate was lifted, the device was installed following the manufacturer guidelines, placing the sedimentation chamber next to the curb. The sedimentation chamber allows sediment to settle out of the runoff before passing through the filter media chamber. Three layers of media where used to fill the filter chamber: two layers of oil sorbent material at the top and one layer of activated carbon at the bottom. Because of the catch basin dimensions, two devices were necessary to cover the catch area.

Stream Guard™ Catch Basin Insert **–** This filter was located in the parking lot of the I-684 north bound rest area – between exit 8th and 9th. It is a fabric bag that can fit almost any inlet shape or

size because of its fabric adapter "skirt". The installation of this unit did not require any custom made frame. Once the grate was removed, the CBI skirt was laid over the opening and the grating replaced. To facilitate the removal of the filter, two metal sticks where used to hang and hold it while the grate was removed. Two people are needed to lift the grating.

Stream Guard™ Passive Skimmer – This device was also located in the I-684 north bound rest area – between exit 8th and 9th, but in the truck parking lot. The device is made of a mesh fabric and has the shape of a pillow stuffed with sorbent beads. No special attention was needed for the installation of this filter, as it is designed to float in the sump of any ordinary catch basin. The only recommendation was to anchor the pillow to the catch basin vault using the attached rope and ring to avoid loss of the filter through the discharge pipe.

Silt Sack® –The filter was installed in the rest area of I-684 South – between exit 5th and 4th, in a catch basin at the end of the parking lot, as shown in Figure 2. The device is a polypropylene sack that fits into the catch basin and no special frame is required. It was necessary to remove the grate, mount the filter inside and hold the lifting loops outside of the catch basin frame, and replace the grate to hold the filter. The installation process required three people, to hold the filter in place while replacing the grate. Special attention was required during the maintenance inspections, such as when lifting the filter, because it was very heavy when full of sediment.

3. STORMWATER TREATMENT SYSTEM RESULTS

Vortechs™ – Table 1 summarized the 6 storms studied at the Vortechs™ site, reporting details regarding the storms and the sampling collections.

Table 1 – Summary of runoff events sampled during the study of the Vortechs™

Event Details						Sampling Details	
Starting date	Starting time	Ending Time	Duration [min]	Depth [in]	Mean [in/hr]	Starting Time	Ending Time
03/31/2004	1:58AM	2:01PM	724	0.49	0.04	6:40AM	1:06PM
05/24/2004	7:44AM	8:28AM	45	0.18	0.24	7:50AM	2:16PM(1)
09/18/2004	8:11AM	12:44PM	274	0.40	0.09	10:02AM	4:28PM(1)
12/06/2004	1:16PM	7:55PM	400	0.33	0.05	1:53PM	8:19PM(1)
02/10/2004	12:35AM	12:34PM	720	0.14	0.01	7:48AM	2:14PM(1)
04/23/2005	9:35AM	5:30PM	476	1.06	0.13	10:13AM	4:39PM

(1) The sampling collection ended after the event.

As shown in Table 1 every sampling cycle has duration of 386 minutes, which is the sum of "PART A" (6 samples every 5 minutes for a total of 25 minutes) and "PART B" (18 samples every 20 minutes for a total of 360 minutes) of the sampling protocol, plus 1 minutes that elapse between both parts. The rain intensity shown in Table 1 is the result of the total rain during the event divided by the total duration of the event.

Table 2 shows the analyses that were performed on each samples collection. The event of the 04/23/2005 was sampled to perform only the BOD_5 test, because of problems that occurred during analysis of samples from the event of the 12/06/2004. No measurement of stormwater flow through the system was made, and thus a true loading can not be calculated for each parameter in order to make a quantitative comparison. However, a comparison of the inlet and outlet concentrations is made and under the assumption that the flow is the same.

The analytical results of samples collected on each storm event at the Vortechs™ System site are summarized in Table 3. The "Field Test Summary" table reports, for each storm event, mean concentration, median concentration, standard deviation, max and minimum concentration of the results of the analysis on the samples from the inlet and the outlet chamber of the device, for each

parameter studied in this project. The last column of the table shows the removal efficiency for each parameter.

Table 2 – Analysis performed on samples collected for the Vortechs$_{TM}$ System.

Starting Date	Parameter Analyzed							
	TKN	TP	BOD_5	FCB	TSS	TPH	pH	Cond.
03/31/2004	X	X		X	X		X	X
05/24/2004	X	X	X		X	X	X	X
09/18/2004	X	X	X	X	X	X	X	X
12/06/2004	X	X		X	X	X	X	X
02/10/2004	X	X	X	X	X	X	X	X
04/23/2005			X					

Table 3 – Field testing summary for the Vortechs^{TM} System

Parameter		N. of Samples	Mean Concentr.	Median Concentr.	Standard Deviation	Max Concentr.	Min Concentr.	Removal Efficiency
TSS [mg/L]	I	120	65.19	42.00	58.40	213.33	0.00	**50.36%**
	O	120	32.36	17.14	39.62	293.33	2.86	
VSS [mg/L]	I	120	29.73	20.00	24.80	93.33	0.00	**50.53%**
	O	120	14.71	9.29	16.33	106.67	0.00	
TPH [ppm]	I	96	4.16	1.77	4.79	18.47	0.00	**65.05%**
	O	96	1.46	1.13	1.32	6.71	0.00	
TKN [mg/L]	I	45	1.93	2.00	1.03	4.09	0.23	**34.95%**
	O	45	1.25	0.93	0.77	2.99	0.41	
TP [mg/L]	I	45	0.90	0.93	0.46	2.10	0.12	**27.67%**
	O	45	0.65	0.61	0.36	1.69	0.12	
BOD_5 [mg/L-O_2]	I	32	12.24	11.85	5.95	22.44	2.31	**44.17%**
	O	32	6.84	7.09	3.78	14.76	0.74	
FCB [MPN/100mL]	I	32	50431	13000	85963	350000	1700	**27.93%**
	O	32	36345	4250	60340	220000	230	

The analytical results of samples collected on each storm event at the Vortechs^{TM} System site are summarized in Table 3. The "Field Test Summary" table reports, for each storm event, mean concentration, median concentration, standard deviation, max and minimum concentration of the results of the analysis on the samples from the inlet and the outlet chamber of the device, for each parameter studied in this project. The last column of the table shows the removal efficiency for each parameter.

V2B1^{TM} – Table 4 summarized the events studied at the V2B1^{TM} site. Details regarding the storms and the sampling collections are reported. Table 5 summarizes the analyses performed on the samples collected for each storm event and available in this study for the evaluation of the performance of the V2B1^{TM} system. No measurement of stormwater flow through the system was made, and thus a true loading can not be calculated for each parameter in order to make a quantitative comparison. However, a comparison of the inlet and outlet concentrations is made and under the assumption that the flow is the same.

Table 4 – Summary of runoff events sampled during the study of the V2B1$_{TM}$ System

Event Details						Sampling Details	
Starting date	Starting time	Ending Time	Duration [min]	Depth [in]	Mean [in/hr]	Starting Time	Ending Time
03/31/2004	1:24AM	12:53PM	690	1.31	0.11	3:30AM	9:56AM
05/16/2004	12:17AM	1:39AM	83	0.15	0.11	12:33AM	7:00AM(1)
09/08/2004	12:02AM	3:50PM	949	3.37	0.21	4:44AM	11:10AM
11/12/2004	9:13AM	8:25AM(*)	1393	1.15	0.05	8:06PM	2:33AM(*)
03/28/2005	11:28PM(**)	7:02PM	1167	1.56	0.08	9:44AM	4:10PM

(*) Next Day; (**) Previous Day;
(1) The sampling collection ended after the event.

Table 5 – Analysis available per each samples collection for the V2B1$_{TM}$ System.

Starting date	Parameter Analyzed							
	TKN	TP	BOD_5	FCB	TSS	TPH	pH	Cond.
03/31/2004	X		X	X			X	X
05/16/2004	X	X		X	X	X	X	X
09/08/2004	X	X	X	X	X	X	X	X
11/12/2004	X	X	X	X	X	X	X	X
03/28/2005	X	X	X		X	X	X	X

Table 6 is a summary of the overall concentrations and removal efficiencies of each parameter studied in all storm events for the V2B1TM system.

Table 6 – Field testing summary for the V2B1^{TM} System

Parameter		N. of Samples	Mean Concentr.	Median Concentr.	Standard Deviation	Max Concentr.	Min Concentr.	Removal Efficiency
TSS [mg/L]	I	96	48.62	20.00	71.40	355.00	0.00	**-20.69%**
	O	96	58.68	20.00	87.71	465.00	0.00	
VSS [mg/L]	I	96	19.70	10.71	21.54	105.00	0.00	**-8.17%**
	O	96	21.31	12.00	24.76	130.00	0.00	
TPH [ppm]	I	96	2.51	1.62	2.38	12.69	0.55	**23.98%**
	O	96	1.91	1.03	2.16	10.56	0.22	
TKN [mg/L]	I	45	1.16	0.69	0.92	3.51	0.17	**22.47%**
	O	45	0.90	0.73	0.71	2.99	0.22	
TP [mg/L]	I	36	0.32	0.24	0.21	1.05	0.10	**-6.57%**
	O	36	0.34	0.34	0.19	0.82	0.05	
BOD_5 [mg/L-O_2]	I	32	7.03	6.88	3.04	14.46	2.67	**2.95%**
	O	32	6.82	5.42	4.03	18.60	3.05	
FCB [MPN/100mL]	I	32	37138	9000	60457	300000	220	**21.44%**
	O	32	29175	6500	62207	280000	50	

4. CATCH BASIN INSERT RESULTS

CBIs were monitored in the field over one year and the following Table 7 shows the details of the field inspection for each device: installation date, replacements, days of operation, cleanup and unistallation date. CBIs were inspected every month and, when required, they were cleaned and the

collected sediment were brought to the laboratory for separation and weighing. In the laboratory, after drying, samples were separated into four main categories.

Table 7 – Field study summary of the Catch Basin Inserts

C.B.I. Name	Date	Notes	Monitoring Time [Days]	Replacements
Silt Sack	7/31/2003	Installed		0
	12/18/2003	Cleaned	140	
	8/9/2004	Removed	235	
	Total Monitored Days =		**375**	
FloGard	8/19/2003	Installed		1
	12/18/2003	Cleaned	121	
	4/13/2004	Replaced	117	
	8/9/2004	Removed	118	
	Total Monitored Days =		**356**	
Ultra Urban	5/3/2003	Installed		0
	6/10/2003	Cleaned	38	
	7/15/2003	Cleaned	35	
	12/18/2003	Cleaned	156	
	8/9/2004	Removed	235	
	Total Monitored Days =		**464**	
Hydro Kleen	2/20/2004	Installed		1
	7/1/2004	Cleaned	132	
	10/22/2004	Replaced(1)	113	
	5/6/2005	Removed	309(2)	
	Total Monitored Days =		**441**	
Stream Guard	5/3/2003	Installed		3
	6/10/2003	Replaced	38	
	9/18/2003	Removed	100	
	10/10/2003	Installed		
	11/12/2003	Replaced	33	
	3/10/2004	Replaced	119	
	7/1/2004	Removed	113	
	Total Monitored Days =		**403**	
Passive Skimmer	6/10/2003	Installed		2
	12/18/2003	Replaced	191	
	1/22/2004	Removed	35	
	2/4/2004	Installed		
	7/1/2004	Removed	147	
	Total Monitored Days =		**373**	

(1) Only the media were replaced but the first chamber (sedimentation) was not cleaned because there were not enough sediment to be collected.
(2) The counting of these days starts from the inspection of 07/01/2004 since no sediment were collected on 10/22/2004.

4.1 Catch Basin Insert Study

Table 8 summarizes the performance of the CBIs during the entire period, reporting on number of filter replacements, days of service (lifetime), the amount of sediment captured, and the average amount of sediments captured per day of operation. All six filters were studied for more than one year, covering 4 seasons. Five of the six devices were observed to remove sediment and oil from stormwater, although certain units were more effective than others. Passive Skimmer is not designed to remove sediment, therefore only hydrocarbons were removed from stormwater.

Table 8 – Summary of the CBIs field study

CBI	No. of Replacements (during monitoring time)	Monitoring Time [day]	Sediment Captured [kg]	Sediment Captured / Day [kg/day]
Ultra Urban®	0	464	50.70	0.11
Flo Gard Plus™	1	356	39.74	0.11
Hydro-Kleen™	1	441	43.34	0.10
Siltsack®	0	375	93.72	0.25
Stream Guard™	3	403	11.26	0.03
Passive Skimmer™	2	373	---	---

As shown in Table 8, the Ultra Urban® filter, Flo Gard Plus™ and Hydro-Kleen™ removed nearly the same amount of sediment per day. It is likely that they had very similar influent loading of sediment, also, since they were installed near each other at the end of the same parking lot. The Hydro-Kleen™ may have removed a slightly lower amount because it was located after the Flo Gard Plus™ filter, along the same curb.

The Siltsack® Filter trapped the largest amount of sediment and the highest rate of sediment captured per day, most likely due to the filter's location. The filter was installed in a catch basin located at the end of the curb in a parking lot that borders a picnic area. During the storm events, sediments were eroded from the picnic area, travelled to the sidewalk and curb nearby the catch basin where the Siltsack® filter was installed (see Figure 1).



Figure 1 – Siltsack® - Sediment were eroded from the nearby picnic area.

The Stream Guard™ filter collected the least amount of sediment, apparently due to the slope of the parking lot in the installed area. In fact, the other filters were located in a parking lot with a slope that facilitated sediment transport to the catch basins. The Stream Guard™ filter location had a very small slope and resulting in a slower runoff velocity and, consequently, a lower concentration of sediments transported in the runoff to the catch basin.

Although all six devices were found to remove oil from the runoff, only five were designed to absorb it. Five filters were manufactured with hydrocarbon–absorbing polymers that are reported to remove oil from the stormwater runoff and trap it without leaching. Through the field inspections and monitoring the color of the sorbent material that become darker after every storm event, we noticed that all five filters removed oil. However no analytical data are available to evaluate the oil removal efficiency of each device, since no analytical tests were performed for this part of the study. The Siltsack® filter is the only filter that was not furnished with oil sorbent material, however, we observed that, after few months of operation, sediments accumulated on the bottom of the filter and trapped some oil from the runoff. While not measured for the field study, the capability to hold a large quantity of sediments will likely result in a significant oil removal efficiency by the Siltsack® filter.

Table 7 shows also how often each filter was replaced during the study period. The Ultra Urban® filter and Siltsack® were the only two filters that were never replaced. At the end of the monitoring period, the Ultra Urban® filter still appeared capable of sediment and oil removal, no damage were noted to the filter, and the sorbent material was not completely oil–saturated. Therefore, the Ultra Urban® filter was capable to operate longer than 464 days without any replacement. The Siltsack® filter, on the other hand, had a tear on the side of the sack (Figure 2) after 375 days of operation (at the end of the monitoring time), and would need to be replaced if longer operation was required.



Figure 2 – Siltsack® - Hole on the geotextile fabric.

The Flo Gard Plus™ filter was replaced once during the monitoring period because of a loss of sorbent material due to a hole in the mesh (see Figures 3 & 4). In fact, we note that the mesh holding the sorbent material was weak and easily torn. At the end of the monitoring study, 118 days since the last replacement, the sorbent material was not completely saturated and appeared capable of continued use.

The Hydro-Kleen™ filtration system was also replaced once during the monitoring period, after verifying the sorbent material was saturated with oil, 132 days after installation. Only the pillows that hold the sorbent material and the activated carbon were replaced. The filter was removed 309 days after the last replacement and it was again found completely oil–saturated. It is possible that the sorbent reached saturation prior to removal.



Figure 3 – FloGard Plus™ - Loose of absorbent material.

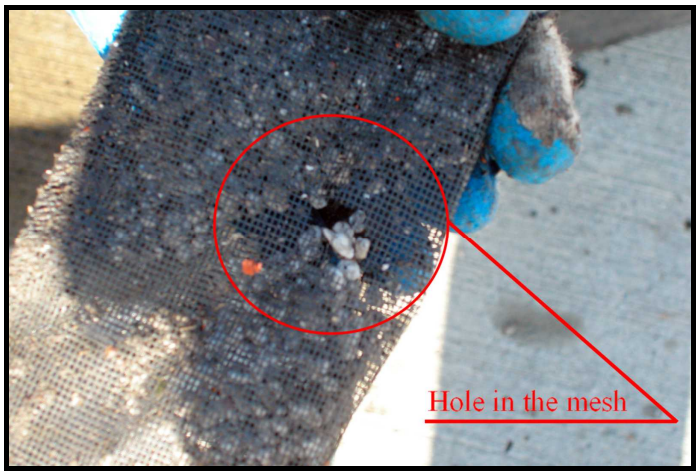


Figure 4 – FloGard Plus™ – Hole in the mesh that holds the absorbent material.

The Stream Guard™ filter was replaced three times during the study period. The filter was replaced because of damage to the geotextile fabric all three times, and not because the sorbent was oil–saturated. Figure 5 shows the holes in the filter fabric. It appeared that these holes were caused from lit cigarettes that people would throw into the catch basin and, during dry weather, these cigarettes would have the time to make a hole through the fabric. After noticing these holes, we found that a lit cigarette would require about three seconds to make a hole through the fabric (see Figure 6).



Figure 5 – Stream Guard™ filter, holes in the fabric caused by lit cigarettes

The Passive Skimmer™, like the Stream Guard™ and the Flo Gard™ filter, was replaced because of damages to the filter during normal operation and not because sorbent material became oil saturated.



Figure 6 – Stream Guard™ – Test performed with a lit cigarette on the fabric.



Figure 7 – Passive Skimmer, slit in the mesh with loss of absorbent material.

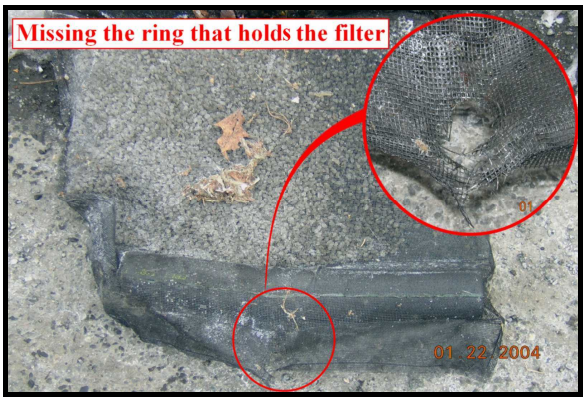


Figure 8 – Passive Skimmer, missing the ring that allows the tying of the filter.

As shown in the Figure 7, the mesh holding the absorbent material is very weak and easily torn. Furthermore, the device was replaced early in the study because the entire filter was lost. The rope that secured the floating pillow disconnected (see Figure 8) and the pillow was carried away during the first storm event. The second Passive Skimmer was replaced because a hole in the mesh of the pillow was observed. After the last replacement, the third filter remained in operation for 147 days, at which time it was nearly oil-saturated.

CBI Operation and Maintenance: During this study, a monthly basic maintenance was performed, in order to assess the maintenance requirements of these devices for a long–term use in a parking lot environment. We notice that the maintenance interval of the devices and the difficulty vary seasonally, being more difficult during the fall and the winter seasons. It shows a large quantity of leaves collected at the entrance of the catch basin, affecting the operation of the catch basin. When a large portion of these leaves falls into the filter installed inside, they affect the performance of the

CBI. In our case, in fact, the large quantity of leaves obstructs the lid of the catch basin, causing the by–passing of the runoff and the leaves inside the filter would clog the system. Therefore, during the fall season, if in the drainage area trees are present, we suggest a CBI inspection once every two weeks, and an increase of the street sweeping activity in the drainage area.

During the winter season, we encountered problems with the inspection and eventual cleanout or replacement of the device, related to the snow accumulated on top of the lid and inside of the CBI. The large amount of snow complicated the inspection of the CBI that required shoveling of the snow that was covering the catch basin. The snow inside the CBI, instead, would increase the weight of the system complicating the lifting during cleanout or replacement of the filter.

Ultra Urban®: The Ultra Urban® filter is a high–strength, corrugated plastic device with the shape of a box. It was easily mounted inside the catch basin, supported by a custom–size frame, and the installation of two filters was necessary to fit the dimension of the catch basin.

Due to the fact that there were two filters installed on the same frame, lifting the filter from inside the catch basin was not easy due to its weight, especially during the snow season Installation and replacement of a large number of these devices at the same time is difficult due to the rigid construction, large shape and volume that require a large truck for the transportation. Another problem related to its shape and size, as shown in the Figure 9 is that the filter obstructed the draining pipe that connect the previous catch basin, holding back a lot of leaf litter and floatables.



Figure 9 – The Ultra Urban® obstructed the draining pipe.

During operation, the CBI performed well, removing sediments and oil for the entire evaluation period, remaining unsaturated at the end of it and requiring very little maintenance. The device exhibited high capability of operation with low and high flow rate and the presence of sorbent material on three sides of the box appears to facilitate the absorption of hydrocarbons from the runoff. The mesh that holds the sorbent material appeared to remain very strong and durable during the observation period.

Flo Gard Plus™: The Flo Gard Plus™ is also a frame–mounted filter and like the UUF filter above, it has to be custom–sized for each site. Also, in this case two filters were needed to fit the catch basin. The filter is easy to install and the filter pouches are replaceable. However, the snap hooks that hold the pouches are not easy to be unplugged. Like for the Ultra Urban® filter, also this device needs to be lifted during the cleanout or replacement. The only difference is that each filter in this case can be removed separately, making the lifting easier. In the event of snow accumulation within the filter, lifting of the device is more complicated and replacement of the pouches is impossible unless the snow is first removed from the filter. Also, the depth of the filter obstructed the piping that connects the other catch basins, impeding litter coming from previous catch basins to flow with the stormwater.



Figure 10 – FloGard Plus™ is deep and obstructs connecting draining piping

The filter appeared to collect oil and sediment for the entire study period, but on one occasion was releasing sorbent material due to a tear in the mesh and replacement of the pouch was required. We noted that this mesh holding the sorbent material is too weak and can be easily torn by the litter. If it is replaced with a stronger mesh material, it can be expected that very little maintenance required throughout one year.

Hydro-Kleen™: The Hydro-Kleen™ is a multimedia filtration system (hydrophobic cellulose and activated carbon are used as filtration media) with a pre–settling sedimentation chamber and overflow by–pass protection. The device was easily installed after its dimensions were customized from the manufacturer. Two filters were mounted on the same frame, next to each other, in order to cover the area of the catch basin inlet.

The replacement of the filtration media of the Hydro-Kleen™ system proved to be very easy, since no lifting of the structure is required and the filtration pillows are simply layered in the chamber, next to pre–settling chamber, where no sediment are present. The cleanout of the sedimentation chamber was not easy to perform due to standing water that was always retained within the chamber. Also, during dry weather, it was complicated to scoop out the sediment from the chamber and the use of a vacuum cleaner is suggested. The design of the system does not allow high flow rates to be treated, as high flow will tend to by–pass the filter since the level of the overflow by–pass outlets is very close to the level of the standing water in the first sedimentation chamber. Also, because of the shape and the size of the device, it caused a partial obstruction of the drainage pipe.

The sedimentation chamber did not appear to be capable enough under the parking lot conditions, where the device was installed during this study. Particularly during the fall season, the device was easily clogged with a large quantity of leaves, thereby increasing the likelihood for the runoff to by–pass.

During the period of evaluation, the device was observed to remove sediments and oils from the runoff; this was easily detected by visual observation of the device and the change in color of the sorbent material. However, since no runoff samples were collected and analyzed, it was impossible to evaluate the removal of other pollutants (organics, metals, etc.) through the activated carbon filter from this field study.

Siltsack®: The Siltsack® filter is a permeable geotextile bag held within a catch basin between the grating and the rim of the catch basin. This configuration has two main advantages: the first is to allow the filter to fit in a variety of catch basins without the need to be customized. The second advantage is the small volume that it occupies during transportation since it can be folded. Both advantages make the device very practical for the operator that would have to install or replace

large quantities of filters. Another relevant aspect of this device is that it does not contain any sorbent material and this makes its disposal is easier, since no special procedures are required.

The installation and replacement processes were difficult on our initial attempt, but become relatively easy with the experience. During installation the edges of the device need to be held outside of the catch basin, while at least two operators lift the lid and lower it straight (downward) on the rim of the catch basin. A main problem that we notice during the cleanout of the filter was the weight of the sediments inside the filter, which would complicate lifting of the device.

The device, during the entire period of evaluation, performed very well, removing primarily sediments and required very little maintenance.

StreamGuard™ Cacth Basin Insert: The StreamGuard™ filter is a polypropylene geotextile basket that, like the Siltsack® filter, is held within the catch basin between the lid and the rim of the catch basin.

Similarly, this device presents two advantages in that it fits into a variety of catch basins without requiring customizing, and it can also be folded, making the device easy to transport. Installation and replacement of the filter are similar to the Siltsack®, but they differ in that this device can not accumulate large quantity of sediment and that makes it easier to lift from the catch basin when performing routine maintenance cleanout or replacement. On the other hand, due to its lower capacity, the device will require more frequent maintenance.

During operation, we noted that the device collected sediments and oils, and appeared to also remove floatable pollutants from the stormwater runoff. The main problem that we observed on this device is that the fabric material is very weak and easy to break. Many times we found holes in the fabric caused by lit cigarettes, especially during dry weather conditions. The small diameter of the basket leads to clogging of the filter, particularly during the fall season when a large quantity of leaves accumulated at the bottom of the filter and resulted in standing water so that subsequent runoff by–passed through the overflow holes.

StreamGuard™ Passive Skimmer: The Passive Skimmer is a floatable device in the shape of a pillow, filled with hydrocarbon–absorbing polymers. Its simple configuration made this device very easy to install, maintain, and replace, since no specific expertise or procedures were required. The only problem that we noticed during the evaluation period was related to the mesh that holds the sorbent material—it is very weak and easy to break. We noted one occasion, after rubbing on the bottom of the catch basin, a tear in the mesh, causing loss of sorbent material. On another occasion, we found the ring that is used to tie off the filter disconnected because the mesh had torn, again, resulting in a loss of the all filter material.

During operation, we believe the device removed oil because the sorbent material darkened, but because of its floating configuration we are uncertain about the ability of the skimmer to absorb all the floating oil within the catch basin.

2. CONCLUSIONS

Two Stormwater Treatment Systems and six Catch Basin Inserts were instrumented and monitored in the field for one year to evaluate their effectiveness at contaminant removal. The two Stormwater Treatment Systems, the Vortechs™ and the V2B1™, were both installed at locations in Long Island, NY, by the NYS Department of Transportation and instrumented for this field study. They were evaluated by comparison of the inflow and the outflow concentrations, in order to determine their removal efficiency for six parameters: TSS, TPH, TKN, TP, BOD5 and FCB. Samples were collected at each site during five storm events and lab analyses were performed at the Polytechnic University laboratory, in Brooklyn, NY, following the Standard Method and EPA Procedures. Also, the costs for installation and maintenance were studied. In addition, six CBIs were installed in the Westchester area and were monitored for one year to evaluate their maintenance requirements, durability and cost effectiveness.

No measurement of stormwater flow through the system was made, and thus a true loading can not be calculated for each parameter in order to make a quantitative comparison. However, a comparison of the inlet and outlet concentrations is made and under the assumption that the flow is the same. The evaluation of the Vortechs™ showed that the system removes on average 50% of the TSS, which is lower than the manufacturer's estimated annual removal efficiency (80%). In addition, the study showed that the system is capable of removing on average 65% of TPH, 35% of the TKN, 30% of TP and FCB and 45% of the BOD_5. Finally, the system presented positive removal efficiency for all the parameters that were analyzed in this study.

The V2B1™ system evaluation, on the other hand, showed a very low removal efficiency on average, and occasionally resulted in negative values for some parameters. The average TSS removal efficiency was -20% (the manufacturer's estimated annual removal efficiency is 80%), and was only found to remove TSS effectively during one storm event. With respect to the other parameters, the V2B1™ removed on average 24% of TPH, 22% of TKN, -6% of TP, 3% of BOD_5 and 21% of FCB.

In terms of costs, the two devices are very similar, both having an average unit cost of $16,000 an installation cost about the 30% of the unit cost, and a maintenance cost less than $2,000 per year. Considering that both units have comparable costs, the most effective stormwater treatment systems was the Vortechs™ system, in view of the fact that it was more efficient in pollutant removal.

All six CBIs studied during for a one year period were capable of collecting sediment and removing oil from stormwater runoff. Our field observations are that CBI filter selection should consider the site conditions and the purpose of the installation.

In terms of durability the most effective device was the Ultra Urban® filter, which has a high capacity for holding sediment and after one year of operation was not oil–saturated, nor did the structure show signs of damage or wear and tear. All the other devices, except for the Siltsack®, were replaced during the monitoring time: the FloGard™ Plus and the Hydro-Kleen™ were replaced once, the Passive Skimmer twice and the Stream Guard™ three times.

In terms of annual cost for filtration media, the lowest cost was the Siltsack® at $65 per year, while the cost of the Ultra Urban® was $733, the FloGard™ Plus was $228, the Passive Skimmer was $240, and the Stream Guard™ was $260. However, the Siltsack® is not furnished with an oil–absorbing media and the installation of this device is suggested in sites where only the collection of sediment is needed. Note that we observed some capacity for this CBI to remove oil from runoff as a result of the accumulated sediment, not by design.

In terms of Operation & Maintenance all the devices were relatively easy to operate and maintain and they should all, with the exception of the Passive Skimmer, have comparable annual maintenance cost (approximately $640 per year) relative to the labor of a two person crew equipped with a van. The annual maintenance cost of the Stream Guard™ Passive Skimmer was estimated about $480 per year. A significant difference will be noted in terms of storage for the devices, as the Hydro-Kleen™, Ultra Urban® and FloGard Plus™ require significantly more space, while the Siltsack®, the StreamGuard™, and the StreamGuard™ passive skimmer can be stored with much less space requirement.

REMOVAL OF RADIONUCLIDES BY *BASILLUS POLYMYXA* IMV 8910 FROM AQUEOUS SOLUTION

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Abstract: This study focuses on the removal of such radionuclides, as uranium (VI), strontium and cesium by *Bacillus polymyxa* IMV 8910 from aqueous solutions. It is shown that U(VI) and Sr are effectively sorbed by bacteria, but cesium was not concentrated by one. Uptake capacity of biomass is characterized by high values obtained even with low metal concentration in solution. Both processes of U (VI) and Sr sorption by *B. polymyxa* IMV 8910 can be described well by the Langmuir isotherm with a_{max} = 1000 μmol/g, k = 11.11, R = 0.992 and a_{max} = 435 μmol/g, k = 3.3, R = 0.999, respectively. The optimal range of pH for U (VI) sorption is 4.0-6.0, for Sr^{2+} – 4.0-8.0. Bivalent alkaline-earth metal cations act as competitors for U (VI) and Sr biosorption. Desorption efficiency increases in the order: dH_2O <<< $NaHCO_3$ < EDTA < HCl ≤ $MgCl_2$ ≤ $CaCl_2$ for strontium, and in the order: dH_2O < $CaCl_2$ <<< NaOH < H_2SO_4 < HCl < EDTA < HNO_3 ≤ $NaHCO_3$ < Na_2CO_3 for uranium. Biosorbent can be used for treatment of low charge waste streams, which containing U (VI) and Sr ions.

Key words: *Bacillus polymyxa*, biosorption, desorption, strontium, uranium.

1. INTRODUCTION

Although mining and refining industries produce aqueous waste streams containing low metal concentrations, the heavy metal and radionuclide contaminations of environment are the most dangerous types of pollution. The treatment of low charge effluents, conciliating economic and technical constraints is impossible by traditional physical-chemical processes. For example non-selective precipitation generates metallic sludges containing various metals difficult to valorize, ion-exchange techniques require large volumes of resins for treatment of high delivery of low metallic concentration flows and generate a high concentration of salt residue (Kratochvil and Volesky 1998). It has been suggested that biomass could be used to decontaminate these wastes and to concentrate metals.

The capability of some microorganisms to concentrate metallic elements has been known for some time. It is known two main mechanisms of biological removal of heavy metals and radionuclides for today (Davis et al. 2003): biosorption and biaccumulation. Biosorption is a term that describes the removal of heavy metals by the passive binding to non-living biomass from aqueous solution. This implies that removal mechanism is not metabolically controlled. In contrast, the term bioaccumulation describes an active process because removal of metals requires the metabolic activity of a living organism.

Biological methods are the most ecologically appropriate techniques. The major advantages of microbial treatment are: self-reproducibility, adaptability, recyclisation of bioproducts, specificity, and good cost/benefit ratio (Gazso 2001).

It is well known that both organic and inorganic reagents like different types of mineral acids, fatty acids, polysaccharides, proteins and chelating agents are generated by microorganisms. These metabolic products can effectively binding different metal ions. *Bacillus polymyxa* (new nomenclature - *Paenibacillus polymyxa*) is a Gram-positive, neutrophilic, heterotrophic, and

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widespread in soils bacterium, which is characterized by intensive exopolysaccharide secretion (Prado Acosta et al. 2005).

Motivated by this, the purposes of the present research are to test *B. polymyxa* IMV 8910 strain for its ability to adsorb uranium (VI), strontium and cesium from aqueous solutions and to investigate the performances of this bacterium to remove these radionuclides (adsorption capacity, pH effect etc).

2. MATERIALS AND METHODS

2.1 Bacterium and growth conditions

The strain of *B. polymyxa* IMV 8910 used in this study was obtained from Institute of Microbiology and Virology, National Academy of Sciences of Ukraine. The bacteria were subcultured in the laboratory using a meat-peptone broth with addition of glucose (30 g/l) at 30°C with agitation. Cultures were harvested at 24 h by centrifugation at 10 000×*g* for 15 min. Cells were triple washed by distilled water (dH_2O) and resuspended in a minimal volume of dH_2O at a concentration of approximately 0.6 g/l.

2.2 Metal uptake experiments

Biosorption experiments were carried out in triplicate series. Uranium is provided as uranyl sulfate ($UO_2SO_4 \cdot 3H_2O$), strontium and cesium – as chlorides ($SrCl_2 \cdot 6H_2O$, CsCl). The pH was adjusted with 1 N HCl and 1 N NaOH before the addition of cells suspension aliquot. The experiments were performed at room temperature. As sorption equilibrium was reached, biomass was removed by centrifugation at 10 000×*g* for 15 min. The residual concentration of uranium in solution was determined by arsenazo III method. Dissolved Sr was measured on atomic absorption spectrophotometer (N_2O/C_2H_2 reducing flame). Sr was read at wavelength of 460.7 nm. Cs was measured on atomic absorption spectrophotometer (propane/air flame, the emission mode) and was read at wavelength of 852.1 nm.

Sorption capacity is calculated by

$$a = (C_o - C_e)V/m \quad (1)$$

where a is the sorption capacity (μmol/g of biomass), C_o the initial metal concentration, C_e is the residual metal concentration in solution (μmol/l), V the volume of solution (l) and m the sorbent mass (g).

The two adsorption isotherm models (Langmuir and Freundlich) for single metal systems were used to describe uranium sorption by microbial biomass. Isotherms were obtained for concentration range 10 to 1200 μmol/l radionuclides. A general form of the Langmuir model equation is

$$a = a_{max}(kC_e/1+kC_e) \quad (2)$$

where a_{max} is the maximal sorption capacity (μmol/g of biomass) and k – constant related to energy of adsorption.

A general form of the Freundlich model equation is

$$a = kC_e^{1/n} \quad (3)$$

where k and n are empirically determined constants, with k being related to the maximum binding capacity, and n – related to the affinity or binding strength.

Competition effect of mono- and bivalent metal ions on uranium and strontium sorption in concentration range 0.1 to 50 mmol/l Me (K^+, Na^+, Ca^{2+}, Mg^{2+}) was studied.

For study of metabolic activity effect on U (VI) and Sr sorption such inhibitors as DCCD (*N,N'*-dicyclohexylcarbodiimide), DNP (2,4-dinitrophenol) and NaN_3 were used. The effect of DCCD was studied for concentrations range 1-100 μmol/l; DNP – 10-500 μmol/l; NaN_3 – 0.1-10 mmol/l. Glucose was used as energy source at the concentration 30g/l. Water bath was used to provide required temperature (20, 40 and 60°C) during sorption experiments. Thermally inactivation of biomass was carried out by triply heating of cell suspension in water at 80°C during 15 min.

The blocking of carboxylic groups of cell wall surface was carry out by etherification reaction in an acid medium (Beveridge and Murray 1980). The microbial biomass was suspended in an ethanol solution, acidification by 0.1 M HCl. The reaction was carried out for 24 h. Microbial cells were triple washed by dH_2O and removed by centrifugation for study their sorption capacity.

2.3 Metal desorption from microbial biomass

Microbial biomass after biosorption was thoroughly rinsed in dH_2O, and cells were separated by centrifugation. Biomass was extracted by salt solutions, chelating agents, acids and alkalies, respectively. Extractions were carried out by mixing on a shaker during 1 h. Biomass was removed by centrifugation, and the residual concentration of radionuclides in solutions was measured.

Desorption degree is calculated by

$$D = 100\% \cdot C/q \quad (4)$$

where D is desorption degree (%), C – residual concentration of metal in solution of displacing agent (μmol/l), q – amount of metal, which are sorbed by biomass (μmol/l).

3. RESULTS AND DISCUSSION

3.1 Equilibrium studies

Based on the investigations we found that *B. polymyxa* IMV 8910 intensively adsorb uranyl ion and strontium, whereas cesium was not sorbed by bacteria. Representative sorption isotherms for UO_2^{2+} and Sr^{2+} at pH 6, plotted in Figure 1, illustrate the evolution of metal concentration in biomass versus residual metallic concentration in solution.

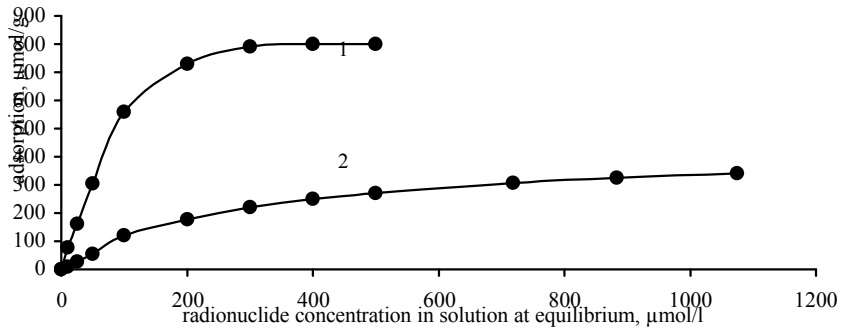


Figure1. Uranium (1) and strontium (2) isotherms at pH 6.

The experimental data showed that both processes of uranium and strontium sorption by *B. polymyxa* IMV 8910 can be described by Langmuir and Freundlich isotherms (Table 1). The Langmuir model seems to be better model for describing U (VI) and Sr sorption by microbial

biomass. It is indicates that the adsorption of uranium occurs in the monolayer on the specific binding sites.

Table 1. Application of Langmuir and Freundlich isotherms to the results for U (VI) and Sr sorption by B. polymyxa IMV 8910

Radionuclides	Langmuir model				Freundlich model	
	a_{max}, µmol/g	k, μmol^{-1}	R	lgβ	1/n	R
Sr^{2+}	435	3.3	0.999	2.56	0.517	0.973
UO_2^{2+}	1000	11.1	0.992	3.19	0.592	0.915

3.2 Uranium and strontium uptake in dependence on the pH

The pH value plays an important role in biosorption phenomenon, mainly by its influence on the cell wall chemistry and metal chemistry in solution (Guibal et al 1992). On the one hand, the cell wall contains amines, amides, carboxylic and other surface functional groups which are protonated or deprotonated, depending on the pH value of the aqueous medium. On the other hand, uranyl chemistry is complicated by a hydrolysis phenomenon: when pH increases from acidic value to a neutral and alkalic value, various hydrolysed species exist: UO_2^{2+}, $(UO_2)_2(OH)_2^{2+}$, $UO_2(OH)^+$, $(UO_2)_3(OH)_5^+$, $(UO_2)_4(OH)_7^+$, $UO_2(OH)_2$, $(UO_2)_2(OH)_3CO_3$, $UO_2(CO_3)_2^{2-}$ and $UO_2(CO_3)_3^{4-}$, the repartition of which is conditioned by the pH value.

The strong pH dependence of U (VI) biosorption by *B. polymyxa* IMV 8910 is depicted in Figure 2. It is shown, an optimal range of pH for uranium sorption is 4.0-6.0, when reactive sites on the cell surface are negatively charged, and major forms of uranium hydrolysed species are UO_2^{2+} and $(UO_2)_3(OH)_5^+$ (Moulin and Moulin 2001).

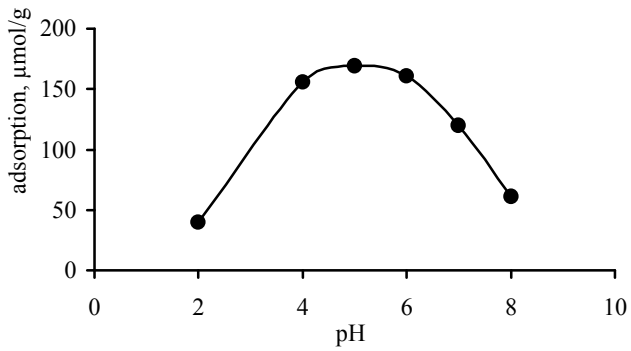


Figure 2. Sorption of uranium by B. polymyxa IMV 8910 in dependence on the pH.

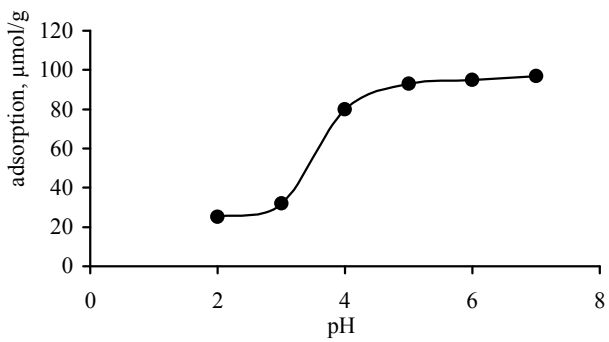


Figure 3. Sorption of strontium by B. polymyxa IMV 8910 in dependence on the pH.

Figure 3 illustrates the effect of the pH on strontium biosorption: sorption increases from low values at pH 2 to nearly 100% uptake at pH 6. Low ion sorption for alkaline-earth metal and alkali metal ions at low pH values and high in mild acidic solutions are known for all cation exchangers. *B. polymyxa* IMV 8910 intensively produce acidic exopolysaccharides (Madden et al. 1986), which are negatively charged, so that positively charged metal ions can be sorbed onto their surfaces.

3.3 Competitive effect of mono- and bivalent metal ions in U(VI) and Sr biosorption

In natural and wastewater radionuclides may compete with different cations in process of sorption. It is shown, both processes of UO_2^{2+} and Sr^{2+} uptake are not inhibited by monovalent cations: Na^+ and K^+ (data are not presented). But bivalent cations Ca^{2+} and Mg^{2+} act as efficient competitors for strontium biosorption: about 100% of Sr is displaced for Me^{2+} concentration 0.01 mol/l (Figure 4). U (VI) sorption is not so depressed by bivalent cations as Sr one. Uranium uptake decreased only on 23 % at Ca^{2+} concentration in solution 0.05 mol/l. Decreasing of uranium sorption in the presence of Ca^{2+} ions might be due to their competition for the metal-binding sites (Strandberg et al. 1981) and forming complete mixing complexes between U (VI) and Ca, e.g. $Ca_2UO_2(CO_3)_3^0$ (pH 6,0-8,5) and $CaUO_2(CO_3)_3^{2-}$ (pH 8,5-9,0) (Fox et al. 2006).

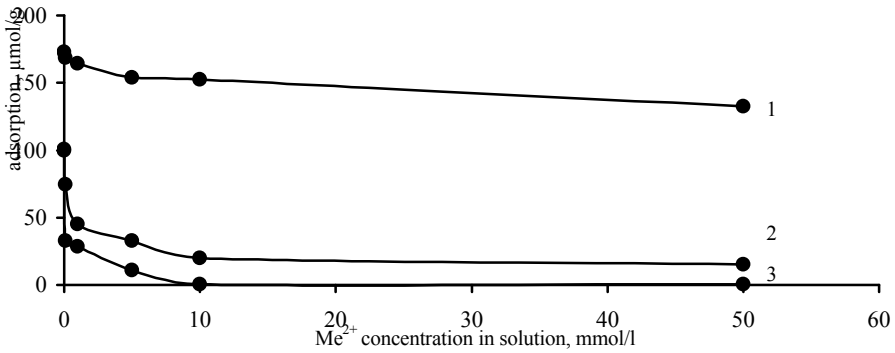


Figure 4. Competitive effect of Me^{2+} ions on radionuclides biosorption: 1- effect of Ca^{2+} on U (VI) sorption; 2 and 3 – effect of Mg^{2+} and Ca^{2+} on Sr sorption, respectively.

3.4 Interaction of B. polymyxa IMV 8910 cells with uranium (VI) and strontium

3.4.1 Mechanism of U (VI) and Sr sorption by B. polymyxa IMV 8910

Mechanisms by which microorganisms can remove radionuclides from aqueous solutions have been investigated by several authors. Merroun et al. (2005) concluded, based on TEM (transmission electron microscopy) analysis, that U sorbed by the cells of *B. sphaericus* JG-A12 is located at the cell surface. Stranberg et al. (1981) have shown that uranium uptake by *S. cerevisiae* did not require cellular metabolism; however, the rapid intacellular uptake of U (VI) by *P. aeruginosa* suggested the possible involvement of metabolically mediated active transport. Other microorganisms such as *Stenotrophomonas maltophilia* JG-2, *Acidithiobacillus ferrooxidans*, *Pseudomonas sp.* EPS-5028 can accumulate U intracellularly via passive transport mechanisms, resulting in the formation of dense uranium deposits inside the cells (Merroun et al. 2004, Marques et al. 1991). Kazy et al. (2009) suggested that the high intracellular uranium accumulation by a *Pseudomonas sp.* strain could be due to the result of a combined ion-exchange–complexation–microprecipitation

mechanism, which is a new report for uranium biosorption by metabolically inactivated bacterial cells.

In regard to strontium sorption, Faison et al. (1990) have found that Sr binding by *Micrococcus luteus* occurred in the absence of nutrients and did not require metabolic activity. It is shown that Sr-binding activity is localized on the cell envelope. Biosorption of Sr^{2+} has been demonstrated by yeast and filamentous fungal biomass also (de Rome and Gadd 1991). Laboratory and brewing strains of *S. cerevisiae* can bind strontium by different mechanisms: as external Sr^{2+} adsorption and as intracellular Sr^{2+} accumulation (Avery and Tobin 1992).

For investigation of mechanisms of uranium (VI) and strontium removal by *B. polymyxa* IMV 8910 was studied U and Sr uptake in dependence on temperature and metabolic activity. It is shown, that the interaction of uranyl and strontium ions with bacterial cells was not influenced by temperature within the range 20 to 60^{o}C effectively. Also processes of U and Sr removal were not dependent on the presence in solution such metabolic inhibitors as DCCD, DNP and NaN_3 and such energy source as glucose. Microbial biomass inactivated by thermal pretreatment in water for 15 min at 80^{o}C showed the same sorption kinetics and maximal U (VI) and Sr uptake binding capacity as bacterial cells in native form.

So, uranium (VI) and strontium removal by *B. polymyxa* IMV 8910 occurs on the biosorption mechanism and is the binding of radionuclide ions by functional groups of the cell wall and the mucilage.

3.4.2 The blocking of carboxylic groups cell wall surface

Extracellular association of metal ions with bacterial cell surfaces is primarily due to physical and chemical interactions involving adsorption, ion exchange, and complexation and does not directly depend on metabolism. Bacterial cell walls, exopolymers, proteins, and lipids contain functional groups such as carboxylate, hydroxyl and phosphate, which are capable to form complexes with uranium (Fein et al. 1997, Kazy et al. 2009). Faison et al. (1990) concluded that a rapid, metabolism-independent uptake of dissolved strontium could be described as an "ion-exchange" process involving carboxyl and hydrocarboxyl sites on external cell walls. Previous studies indicated, based on the results of spectral analysis, that active groups on the cell surface of *B. polymyxa* IMV 8910, which may take part in U (VI) binding, are carboxylic, hydroxylic, phosphate, amine and amide groups (Spasenova et al. 2006).

Although the cell wall and the mucilage have mostly weak sites of complexation (-COO^- and -SO_3^-), but in great number, that provides their essential complexing roles (Davis et al. 2003, Buffle 1988). The important role of carboxylic groups in the adsorption process has been clearly demonstrated in many researches (Davis et al. 2003, Sar and D'Souza 2001, Gonzales-Munoz et al. 1997). Experimental data shows that blocking of COO^- groups on the cell surface of *B. polymyxa* IMV 8910 by etherification is reducing of sorption capacity of bacterium biomass on about 20% and more than 50% for uranium and strontium accordingly (Table 2).

Table 2. Sorption capacity of B. polymyxa IMV 8910 intact cells and after blocking of COO- groups on the cell surface

Microbial biomass	**Sorption capacity a_s, µmol/g**	
	UO_2^{2+}	**Sr^{2+}**
Intact cells	805	414
After blocking of COO^- groups	637	198

3.5 Uranium and strontium desorption from B. polymyxa IMV 8910 biomass

So, it is shown, that *B. polymyxa* IMV 8910 biomass can effectively remove such radionuclides as uranium (VI) and strontium from aqueous solution. However, the technical application potential

of this process would be substantially enhanced if the potential for recovering the U (VI) and Sr that have been adsorbed by the biomass would develop. The realization of such a potential would make the use of biomass similar to that of activated carbon or other synthetic adsorbents.

Recovery of the adsorbed radionuclides could be achieved by the use of an appropriate elution solution capable of effectively stripping the adsorbed uranium and strontium from the waste biomass and bringing it back into solution. A number of desorption agents have been tested for their ability to release bound UO_2^{2+} and Sr^{2+} ions from *B. polymyxa* IMV 8910 biomass (Table 3). Water washing was ineffective in removing uranium and strontium from cells. Strontium can be efficiently displaced by extraction of microbial biomass with $CaCl_2$, $MgCl_2 \cdot 6H_2O$, HCl, H_2SO_4, and less efficiently with EDTA, $NaHCO_3$, Na_2CO_3, NaOH. The high degree of strontium desorption by $CaCl_2$ and $MgCl_2 \cdot 6H_2O$ can be explain by the strong competitive effect of the chemically similar cations Ca^{2+} and Mg^{2+} in Sr uptake. These results confirm that Sr binding to microbial cells is an equilibrium ion-exchange process and that Sr may be either displaces by certain other cations for which the cells have a high affinity or stripped by anions that form stable complexes with metals.

Table 3. U (VI) and Sr desorption from B. polymyxa IMV 8910 biomass

Desorption agent		**Desorption degree, %**	
		UO_2^{2+}	**Sr^{2+}**
dH_2O		6.8	2.1
NaOH	0.1M	63.1	12.2
Na_2CO_3	0.05M	85.5	13.5
$NaHCO_3$	0.1M	75.2	15.6
H_2SO_4	0.05M	64.0	18.2
HCl	0.1M	66.8	21.7
HNO_3	0.1 M	78.6	20.3
EDTA	0.05 M	69.7	16.8
$CaCl_2$	0.05 M	13.6	21.0
$MgCl_2 \cdot 6H_2O$	0.05 M	7.0	21.7

Desorption efficiency for uranium increases in the order: $dH_2O < CaCl_2 <<< NaOH < H_2SO_4 < HCl < EDTA < HNO_3 \leq NaHCO_3 < Na_2CO_3$. The high level of bound uranium removing by extraction of microbial cells with Na_2CO_3 is caused by forming stable soluble anion complexes with U (V) - $[UO_2(CO_3)_3]^{4-}$ (Gonzales-Munoz et al. 1997, Wazne et al. 2003). The uranium desorption degree by sodium bicarbonate is sufficiently great – 75.2%. But this elution solution is considerably milder than for the carbonate solution and consequently less damaging to the biomass, as well as easier to work with from a process application point of view. The mechanism of action of bicarbonate during elution is the same as that of carbonate (Tsezos 1984).

4. CONCLUSIONS

The use of microbial cells of *B. polymyxa* IMV 8910 as biosorbent for uranium (VI) and strontium offers a potential alternative to existing methods for decontamination or recovery or both of radionuclides from low charge waste streams. The ability of this organism to bind U (VI) and Sr in the absence of cofactors and its high affinity for these radionuclides are encouraging in this context.

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OCCURRENCE AND BIODEGRADABILITY OF EMERGING POLLUTANTS IN THE JORDAN VALLEY

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Abstract: In particular in arid regions the reuse of waste water is an important issue. Elimination of persistent emerging pollutants represents a key factor in integrated water resources management in arid regions, and identifying suitable treatment processes to eliminate such compounds becomes inevitably necessary. As part of the SMART Jordan Valley project, new integrated approaches for water management, aquifer recharge, and waste water reuse are developed. It is the objective of this study (i) to assess the occurrence of emerging pollutants in the Jordan valley and (ii) to review and examine the biodegradability of selected key compounds. Among the most frequently detected compounds during a sampling campaign in 2007 were pharmaceutical residues such as carbamazepine, diclofenac, or naproxen, and X-ray contrast agents such as diatrizoic acid and iopromide, all typically found in Europe and the US as well. To gain further insight into elimination processes, biodegradation studies were conducted with batch tests and flow-through soil columns under unsaturated, aerobic conditions. Results demonstrated biodegradation for pharmaceutical residues such as ibuprofen, diclofenac, bezafibrate, or naproxen. The degradation rate was faster in waste water treatment plant effluent as compared to raw waste water, most probably due to competing substrate consumption in raw waste water. The antiepileptic carbamazepine was not degraded in the batch tests and showed only moderate removal during soil passage, probably due to sorption. The results of this study and previously published data emphasize the need for further studies under more defined conditions to elucidate the specific conditions under which biodegradation of emerging pollutants is possible.

Key words: Biodegradation, emerging pollutants, waste water reuse.

1. INTRODUCTION

The release of emerging pollutants such as pharmaceutical compounds and their metabolites into the aquatic environment has become an issue of increasing concern over recent years. In influents and effluents of European and US waste water treatment plants (WWTPs), more than 80 pharmaceuticals and some of their metabolites have been detected, and in minor concentrations some of these compounds also have been found in surface water and groundwater. Concentrations in the range of ng/L up to µg/L were reported in surface waters, e.g. diclofenac 0.01 – 5.5 µg/L, ibuprofen 0.005 – 7.1 µg/L, gemfibrozil 0.06 – 4.8 µg/L, carbamazepine 0.01 – 6.3 µg/L, or naproxen 0.01 – 5.2 µg/L (e.g. Sacher et al. 2001, Debska et al. 2004). In areas of extreme water scarcity such as the Lower Jordan River Valley waste water reuse and artifical recharge of groundwater are important measures to meet the increasing water demand in the region. Waste water reuse can lead to the accumulation of persistent organic pollutants (POPs), e.g. pharmaceuticals, endocrine disruptors (e.g. Bisphenol A) or organophosphorous compounds (e.g. tris(2-chloroethyl) phosphate), used as placticizers (Wintgens et al. 2008, Reemtsma et al. 2008, Zoller et al. 2008). In our study, a sampling campaign was conducted to gain information on the occurrence and distribution of emerging pollutants in water samples of the Jordan valley. Waste water, surface water, and groundwater samples have been taken regularly. Selected results of the analytical screening program are presented in this paper. Biodegradation of specific compounds was studied on laboratory scale in batch experiments. Elimination of pharmaceutical residues and x-ray

contrast agents additionally was examined in flow-through soil columns under unsaturated, aerobic conditions.

2. MATERIALS AND METHODS

2.1 Sampling Campaign and Analysis

Within the period spring 2007 to spring 2008, samples of wastewater, surface water and groundwater have been collected from different sites at the Lower Jordan River Valley and the West Bank. After sampling the pH was adjusted to approx. 2 by adding hydrochloric acid to stabilize the samples for the transport to Germany. Pharmaceutical residues including x-ray contrast agents were analysed by HPLC-MS-MS after solid phase extraction (SPE) (Sacher et al. 2001).

2.2 Batch Experiments

Using filtered raw waste water and treated waste water, two active and one control batch tests were set up. A mixture of emerging pollutants was added, containing 2-5 µg/L of each substance (pharmaceuticals, endocrine disrupters and organophosphorous compounds). The active batches were inoculated with activated sludge from the WWTP Eggenstein-Leopoldshafen and incubated at room temperature under aerobic conditions through constant aeration. The control batches were set up in the same way, but autoclaved and cooled at < 4°C. Microbiological analysis of bacteria by the most-probable-number (MPN) technique at the end of the experiment confirmed that no viable organisms were present in the control batches.

2.3 Flow-through Soil Column Experiments

Unsaturated column experiments were conducted to study biodegradation under vadose zone conditions (figure 1). Four columns of stainless steel, filled with natural vadose zone soil, were irrigated with treated waste water from the WWTP Eggenstein-Leopoldshafen. Every two weeks the reservoir bottles containing raw waste water or effluent of the WWTP, respectively, were renewed. The reservoir bottles were stored in a refrigerator at 2°C. Hydraulic retention time in the columns was 6-7 days. By comparing leachate concentrations of the 2 biologically inhibited (cooled down to 2°C) and the 2 bioactive columns (20°C), biodegradation of the trace organics during soil passage was assessed. Temperature and water content in the columns were controlled by temperature sensors and tensiometers (fig. 1). 5µg/L of each substance were spiked into the reservoir bottles.

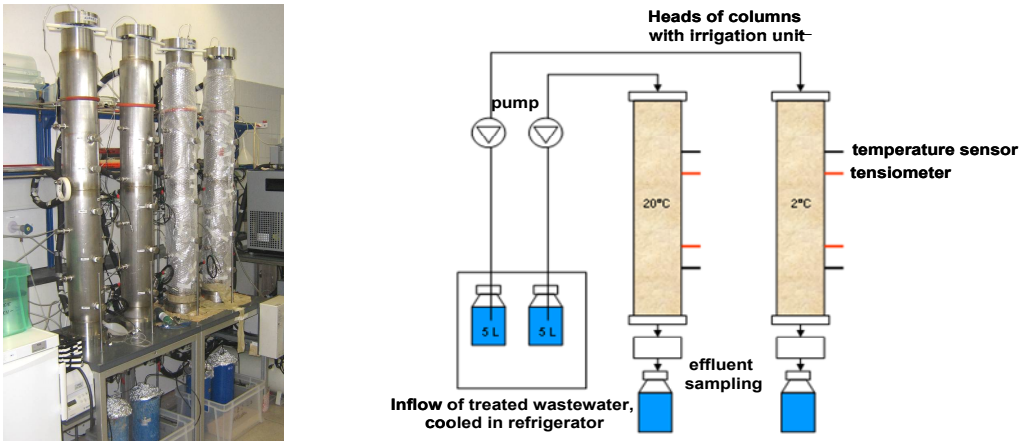


Figure 1. Laboratory setup and scheme of the soil column experiment.

3. RESULTS AND DISCUSSION

3.1 Screening of emerging pollutants

In total 29 different compounds out of a list of more than 50 target analytes were detected. The most frequently detected substances among the pharmaceuticals are presented in table 1. Median concentrations represent data from Jordan only. Similar pharmaceuticals have been detected in waste water and surface water samples taken at the Westbank in Palestine.

Table 1. Median concentrations of selected pharmaceuticals in the Lower Jordan Valley compared with literature data from Europe and the USA.

		Median concentration			Literature*
Group	Substance	GW	SW	WW	
		[ng/L]	[ng/L]	[ng/L]	[ng/L]
		LOD 20 ng/L	LOD 20 ng/L	LOD 50 ng/L	
Lipid regulators	Gemfibrozil	70	160	1850	(SW) 840 – 4760, (b)
	Bezafibrate	< LOD	37	305	(SW) 130, (c)
Antiepileptic	Carbamazepine	107	305	2100	(GW) max. 900, (a) (SW) 870 – 6300, (b)
Antiphlogistics	Naproxen	< LOD	82	159	(SW) max. 32, (d)
	Ibuprofen	< LOD	63,5	250	(SW) 1000, (e) (WW) 230 – 6400, (f)
	Diclofenac	< LOD	34	315	(SW) 680 – 5450, (b)
X-ray contrast agents	Iopromide	1600	1300	860	(SW) 8100, (g)
	Diatrizoic acid	120	110	110	(GW) 1100, (a)
	Total number of samples	14	19	5	

* Literature data represent mean values, if not marked otherwise (max. = maximum values); a: Sacher et al. 2001, b: Andreozzi et al. 2003, c: Möder et al. 2007, d: Benotti et al. 2009, e: Kolpin et al. 2002, f: Ternes 1998, g: Ternes & Hirsch 2000

Comparison with literature data shows that the detected values for pharmaceuticals in Jordan are in the same range as in Europe and the USA. As can be deduced from the data in table 1,

carbamazepine and the x-ray contrast agents diatrizoic acid and iopromide are among the more persistent compounds that also were found in groundwater. This finding is in accordance with previously published studies on the persistence during waste water treatment and in the environment (Sacher et al. 2001, Clara et al. 2004, Joss et al. 2006). On the other hand, higher removal rates during waste water treatment have been reported for ibuprofen, naproxen, and bezafibrate (Zwiener et al. 2000, Quintana et al. 2005), which were not detected in groundwater samples.

3.2 Elimination of emerging pollutants in batch and column studies

Comparison of the removal of pharmaceuticals during a 12 week batch experiment with influent and effluent from a waste water treatment plant shows better removal efficiency in the effluent batch assay. Fig. 2 depicts 5 of the substances investigated. Carbamazepine and diatrizoic acid (table 2) were not removed in any of the assays. However, bezafibrate, naproxen, diclofenac, and ibuprofen proved to be biodegradable (figure 2). All data reflect the net removal after correction for losses in the control batches. Slower biodegradation of the emerging pollutants in raw waste water might be due to competition with easily degradable organic matter present in the samples. Thus DOC decreased by more than 50 % from averaged 46 mg/L to 20 mg/L in the two batches set up with raw waste water, while DOC values in the batches with treated waste water varied between 7 and 8 mg/L. Nitrification was observed in both the raw and treated waste water batch assays.

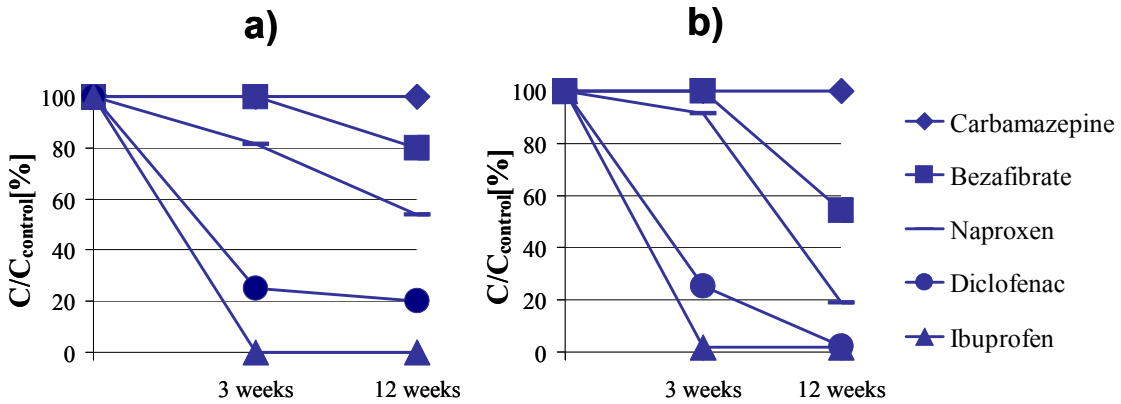


Figure 2. Biodegradation of 5 pharmaceuticals during the batch experiments with (a) filtered raw waste water and (b) effluent from a waste water treatment plant.

Elimination of emerging pollutants also has been investigated during soil passage in columns operated at 2°C and 20°C (figure 3). In both columns, ibuprofen was removed almost completely. Diclofenac, bezafibrate, and naproxen show a median elimination of 66%, 100%, and 50% in the 20° C column, respectively, whereas lower elimination was observed in the 2°C column (figure 3). Carbamazepine and diatrizoic acid show moderate to no removal, respectively, and behave similar at 2 and 20°C.

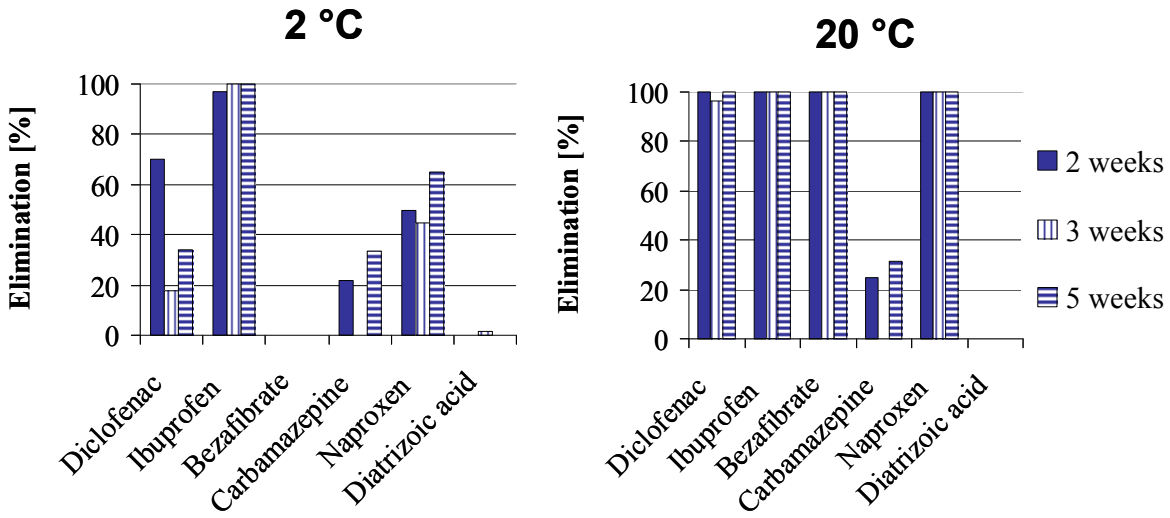


Figure 3. Elimination of 6 pharmaceuticals in flow-through soil columns at 2°C and 20°C

The results demonstrate good reproducibility over time (figure 3). Complete elimination of diclofenac, bezafibrate, and naproxen was observed at 20oC whereas significantly less pronounced removal occurred at 2oC. The temperature controlled difference in elimination rates clearly indicates biological degradation of these compounds at room temperature. The elimination of ibuprofen, carbamazepine, and diatrizoic acid was similar in both columns. However, ibuprofen was completely eliminated even at 2oC thus indicating an abiotic removal mechanism such as adsorption. Table 2 gives a short overview about previously published data. Biological degradation of diclofenac, bezafibrate, and naproxen was observed also in previous studies though contradictory data exist for diclofenac. Also for diatrizoic acid which neither showed removal in the batch assays nor during soil passage in our study, contradictory results regarding its biodegradability have been reported. For carbamazepine and ibuprofen, no indication for a temperature – elimination relation was observed in our study. Carbamazepine is known to be highly persistent during waste water treatment. Ibuprofen has been reported to be easily biodegradable, and biodegradation might also be possible at low temperature. However, Scheytt et al. (2007) reported also significant retardation and less biotransformation than expected from literature for ibuprofen during unsaturated column experiments.

Table 2. Elimination of selected pharmaceuticals in this study compared to literature data

Substance	Elimination in this study *		Elimination in literature	
	Batch (effluent)	Soil passage (20°)	observed	not observed
Bezafibrate	+	+ + +	a, d	/
Carbamazepine	-	+	/	b, g
Naproxen	+ +	+ + +	d, h	/
Ibuprofen	+ + +	+ + +	a, c, i	/
Diclofenac	+ +	+ + +	e	a, d
Diatrizoic acid	-	-	f, j	a

* - no removal, + removal < 50%, ++ removal 50 – 90%, +++ removal > 90%; a: Joss et al. 2006, b: Clara et al. 2004, c: Zwiener et al. 2000, d: Quintana et al. 2005, e: González et al. 2006, f: Drewes et al. 2001, g: Scheytt et al. 2006, h: Kimura et al. 2007, i: Smook et al. 2008, j: Haiß & Kümmerer 2006

Obviously, removal rates in WWTPs and in the environment vary significantly. For instance, in case of diclofenac in some WWTPs elimination rates of 50 - 90 % appeared, whereas almost no removal was observed in other WWTPs (Joss et al. 2006). There is a lack of understanding with respect to the biodegradability of specific pollutant mixtures, the effect of concentration ratios, and the relevance of co-metabolic processes. On the other hand, persistent pollutants such as carbamazepine might be suitable compounds to be used as anthropogenic tracers (Clara et al. 2004, Wolf et al. 2009).

4. CONCLUSION

Emerging organic pollutants such as diclofenac, carbamazepine and diatrizoic acid, which are commonly detected in European and US waters, have been assessed in waste water, surface water, and groundwater in the Lower Jordan Valley. The compounds and concentrations detected were in the same order of magnitude as reported for Europe and the USA. However, intensive water reuse in arid regions as well as water recycling strategies under moderate climate conditions enhance the risk of accumulation of persistent emerging pollutants. Therefore elimination processes should be improved. In particular with respect to biodegradability, contradictory results were obtained. More studies into biodegradation efficiency as function of waste water composition (metabolic/co-metabolic processes) and redox conditions (aerobic/anaerobic processes) are encouraged in order to develop sustainable treatment schemes for waste water reuse and groundwater recharge.

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BIOLOGICAL INDICATORS OF ORGANIC POLLUTION OF NWORIE RIVER, IN IMO STATE NIGERIA

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Abstract: Water samples from Nworie River in Imo State were assessed for bacteriological and physiochemical qualities using standard laid down procedure. Some of the results obtained indicated that upstream samples had higher microbial loads than the other sample which were 3.5X105 cfu/ml, 6.2X107 cfu/ml, and 3.0X103 cfu/ml respectively on MacConkay agar, nutrient agar and Sabauraud Dextrose Agar. Micro-organisms identified included *Escherichia coli*, *Staphylococcus aureus*, *Streptococcus faecalis*, *Aspergillus species*, *Mucor sp.*, and *Talaromyces sp.* The physiochemical properties of the river gave the value of 5.6, 17.3ms/cm and 4.5mg/l for pH, conductivity and salinity respectively. These qualities could justify the level of human activity on the river and the same time shows that the river has a large contamination index which is unsuitable for portable uses. The aim of this research is to identify and isolate micro-organisms associated with Nworie Rivers as to assess its level of contamination.

Key words: Bacteriological, Physiochemical, Conductivity.

1. INTRODUCTION/ LITERATURE REVIEW

The drinking water of most communities and municipalities come from surface sources- rivers, streams and lakes [1]. Such natural water supplies particularly streams and rivers are likely to be polluted with domestic, agricultural and industrial wastes [1]. Pathogens are a type of pollution that proves very harmful. They can cause illnesses that range from typhoid and dysentery to minor respiratory and diseases. Pathogens include such organisms as bacteria, viruses and protozoan. These pollutants enter water way through untreated sewage, storm drains, septic tanks, runoff from farms and particularly boats that dump sewage. Though, microscopic, these pollutants have tremendous effects evidence by their ability to cause sickness [1]. Water is fundamental to all life and makes up more than 80 percent of human body. There is need therefore for adequate water supply for a community. For this reason, humans have spent large sums of money and much energy in other to provide water to meet their needs [6]. Water can be perfectly clear, odourless and tasteless and yet be unsafe for drinking. Surface water is the basic natural sources of water to man for his varied and intense agricultural and industrial activities [22]. Runoffs held in reservoirs provide water supplies for urban centers diverted from large rivers and it provides irrigation for highly productive low lands [20]. The salinity of this water is small but during the period of flood, the hardness. Alkalinity and the total solid of river water decrease considerably [4]. Contrary to this, suspended solids and organic impurities contaminated the water heavily due to wash-off from the top mantle of soil. After the flood, mineralization of water increases gradually and reaches a maximum in dry season when the fraction of the "Base flow" of river becomes predominant [1]. Throughout history, the pollution of water has been a factor in determine human welfare. But water polluted by natural sources has caused great hardship to people if used for drinking or for irrigation [1] [13]. Water being a very good solvent could dissolve a variety of substances, which may include lead, calcium, iron, mercury, chromium, nickel, magnesium, silica, cadmium, arsenic, and gases such as ammonia and carbon dioxide. If the concentration of these substances or gases is above the

minimum level recommended by World Health Organization (WHO), then, that water could be said to be unfit for drinking or for human consumption [1] [30].

2. MATERIALS AND METHODS

The site under study was where the Federal Medical Center Owerri discharges its waste water into the river. Also a central site was chosen upstream of the discharge point along Amakohia, where the inhabitants uses the river as a source of portable water. All glass wares were sterilized using dry heat in an electric hot oven at 1600C for 2 hours [22]. The media and diluents used were sterilized at 1210C for 15 minutes. Wire loop and inoculating needle were sterilized by flaming in a naked gas flame until red-hot. Glass rods were further sterilized by dipping in alcohol and flaming in a Bunsen flame during streaking. The diluents used are normal saline, which was prepared by dissolving 8.5g NaCl (BDH) in 1 liter of water. Ten- fold serial dilution of the samples was carried out using physiological normal saline as diluents. 0.1 ml of each dilution was inoculated in triplicates on Nutrient Agar, MacConkey, Sabauraud Dextrose Agar and incubated at 370C for 24 hours. Number of colonies that appeared after incubation was subculture on a fresh Nutrient Agar medium for purity and then transferred onto slants, stored at 40C in a refrigerator for further use. Bacterial isolates were identified using macroscopic and microscopic examination of the cultures followed by biochemical tests. Gram staining method described by Cruickshank et al; [15] was adopted. Thin smear of the isolates were made on a clean grease free glass slide. Heat fixed by passing Bunsen burner flame for 2-3 times then flooded with malachite green and steamed for minute by heating over a beaker of boiling water. Rinsed in distilled water and flooded with saframin for 20 seconds and later washed in tap water, blotted dry and then examined under oil immersion objectives lens. Motility test as described by Ogbulie et al; [24] was used to differentiate motile and non-motile bacteria. Parameters determined include Biological Oxygen Demand (BOD), Chemical Oxygen Demand (COD), pH, and Temperature.

3. RESULTS

The results of the total count on the three agar media (Nutrient agar, MacConkey and SDA agar) use for enumeration and isolation were represented in figure 1. Some of the results obtained indicated that upstream samples had higher microbial loads than the other sample which were 3.5X105 cfu/ml, 6.2X107 cfu/ml, and 3.0X103 cfu/ml respectively on MacConkay agar, nutrient agar and Sabauraud Dextrose Agar. A total of 6 bacterial isolates were obtained from water samples collected from the three different sampling stations along Nworie River. Micro-organisms identified included *Escherichia coli*, *Staphylococcus aureus*, *Streptococcus faecalis*, *Aspergillus species*, *Mucor sp.*, and *Talaromyces sp*. The physiochemical properties of the river gave the value of 5.6, 17.3ms/cm, 4.5mg/l and 22.80C for pH, conductivity, salinity and temperature respectively. These qualities could justify the level of human activity on the river and the same time shows that the river has a large contamination index which is unsuitable for portable uses. The aim of this research is to identify and isolate micro-organisms associated with Nworie Rivers as to assess its level of contamination.

Table 1. Microbial load of the water samples.

S/N	SAMPLING STATIONS	TOTAL HETEROTROPHIC BACTERIAL COUNT (cfu/ml).
1	Station A: (FMC).	$6.2x10^7$
2	Station B: (Amakohia Road).	$3.5x10^5$
3	Station C: (Behind Emmanuel College).	$3.0x10^3$

Table 2. Physical and chemical parameters.

PARAMETERS	VALUES
Salinity (mg/l)	4.5
Conductivity (ms/cm)	17.3
pH	5.6
Temperature (^{0}c)	22.8

4. DISCUSSION/CONCLUSION

4.1 Discussion

Highest bacteria count was observed at station A which receives the highest refuse disposal and gets more polluted as the Federal Medical Center and other industries discharge their effluent. Followed by Station B which although, is relatively high receives sawdust and other wood particles as its effluent from timber shade. Samples taken from station C show low bacterial presence. This would be as a result of the absence of human activities in these sites and the inevitable influence of human activities in these sites and the inevitable influence of water dilution as reported by other workers [6]. The high bacterial counts obtained in this study could be influenced by the existing environmental conditions. This is in view of the fact that these samples were collected during dry season months of November through January which have been reported by Barrd and Powland [7] to influence the isolation of bacterial groups in the tropics. This is in view of the fact that Nigeria like other countries in the tropics with pronounce dry season may have concentrated water and other factors. Similar concentrations of water have been reported by Okpokwasili and Ogbulie [25] to account for increased bacteriological and physio-chemical quality of river. The fundamental facts of this study is that the river seems to be biologically polluted during dry season months of November through January which may unravel the high risk of water borne diseases during this period. Regrettably, the period of November and January, during which the organism were isolated coincides with the period of least water availability for personal hygiene.

4.1 Conclusion

Clearly, the problems associated with river pollution are the capabilities to disrupt life on our planet to a greater extent. Congress has passed laws in order to combat river pollution thus acknowledging the fact that river pollution is indeed a serious issue. But the government alone cannot solve the entire problem. It is ultimately up to us to be informed, responsible and involved when it comes to the problems we face with our river. We must become familiar with our local river "Nworie", its resources and learn ways of disposing harmful household wastes so that they don't end up in sewage treatment plants that can't handle them or landfills not designed to receive hazardous materials. If these measures are not taken and river pollution continues, life on earth will suffer severely. Global environmental collapse is not inevitable. The World environmentalist must work together to ensure that new industrialized economies do not add to the world's environmental problems. Politicians must think of sustainable development rather than economic development. Conservation strategies have to become more widely accepted and people must learn that energy use can be dramatically diminished without sacrificing comfort. In short, the result gotten from Nworie River had not much difference from the ones that were given by past scientists on that same river during the same season. This shows that enough measures have not been taken to see to the improvement of Nworie River.

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THE RISING CONCERN THAT IS RELATED TO OCCURRENCE OF XENOBIOTICS IN TERTIARY TREATED URBAN WASTEWATER INTENTED FOR REUSE APPLICATIONS

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Abstract: For much of the last forty years, research on the effects of chemical pollution of the environment that is related to urban wastewaters' discharge and reuse has focused almost exclusively on conventional ´priority´ pollutants. However, during the last several years there has been a growing level of concern related to the hypothesis that various chemicals may exhibit endocrine disrupting effects. This is due to increased incidences of endocrine-related diseases in humans, including declining male fertility, and more significantly, to adverse physiological effects observed in wildlife where cause and effect relationships are more evident. In fact, the evidence from these incidences suggests that the changes in the reproductive health of humans, including breast and testicular cancer, birth defects, etc. could be linked to exposure to EDCs (endocrine disrupting compounds). However, no definite cause and effect data have yet been established. EDCs or potential EDCs include compounds such as alkylphenols, alkylphenol polyethoxylates, phthalates, bisphenol-A, polybrominated flame retardants, dioxins, furans, herbicides, pesticides and steroid hormones. As our knowledge of endocrine disrupters increases, so does the list of chemicals that exhibit these properties. In addition to this, thousands of tons of pharmacologically active substances are used annually, but surprisingly little is known about the ultimate fate of most drugs after their intended use. As the use of cosmetics and antibiotics grows, increasing quantities of their active organic ingredients are released into the environment through wastewater systems. In Cyprus, as well as in many other countries facing prolonged droughts and implementing wastewater reuse schemes for irrigation and groundwater discharge, the existence of xenobiotic compounds in the tertiary treated wastewaters is a new concern. This study describes the application of a recently developed multi-residue method for the determination of 29 multi-class pharmaceuticals using off line solid phase extraction followed by liquid chromatography-triple quadropole mass spectrometry (LC-MS-MS). Target compounds included analgesics and anti-inflammatories, lipid regulators and cholesterol lowering statin drugs, psychiatric drugs, an antiulcer agent, histamine H1 and H2 receptor antagonists, antibiotics and ß-blockers. The method was applied for the analysis of pharmaceutical residues at three sewage treatment plants in Cyprus serving major coastal Mediterranean cities. The samples were taken from inlet, after the secondary treatment stage and outlet. The presence of 19 pharmaceuticals was confirmed. For some of the compounds high concentrations were obtained for the final effluents (e.g. ofloxacin: 4.82 µg/L, diclofenac: 5.51 µg/L, carbamazepine: 27.27 µg/L, metoprolol: 9.59 µg/L). Concerning the elimination potential what was derived from the study is that the biological (secondary step) and physico-chemical treatments (tertiary step) do not have the same impact on the removal percentage since the majority of the compounds are removed during secondary treatment.

Key words: Cyprus, multi-residue method, pharmaceuticals, reuse, wastewater, xenobiotics.

1. INTRODUCTION

Cyprus has turned its attention to the utilization of treated urban wastewater either for irrigation purposes or groundwater replenishment due to the prolonged drought periods it faces. Currently, there is an increasingly growing momentum towards the reuse of wastewater while at the same time the concern with respect to the existence of xenobiotic compounds including pharmaceutical residues in the treated wastewater effluents follows also an increasing trend. After application, some pharmaceuticals are largely metabolized before they are excreted, while others are only moderated or poorly metabolized. Others like for example x-ray contrast media are excreted completely intact. After excretion, the pharmaceutical compounds either in the parent or metabolized form enter the municipal sewage treatment plants. If the compounds are not eliminated in the treatment plants, they may enter the aquatic environment reaching drinking water resources or accumulate on soil

during irrigation. In reality their behavior, fate and sinks constitute nowadays a quite intense research field since the analytical methods that are needed for their quantification are far from been standardized and, the potential effects that may have on the environment are not determined. It is generally agreed that acute effects of pharmaceuticals to aquatic organisms are unlikely (Vieno et al. 2007). But at the same time, there are not sufficient data on the subtle and chronic changes in the environment caused by the continuous discharge of mixtures of pharmaceutical residues in a variety of organisms or crops that are irrigated with treated wastewater and consumed by humans.

The existing conventional treatment processes applied are not designed to remove various classes of organic xenobiotic compounds including pharmaceutical active ingredients. In reality, pharmaceuticals' removal depends on their ability to biodegrade and adsorb on suspended matter. Many of these compounds are hydrophilic and their sorption to sludge is limited (Joss et al. 2006). Other, like fluoroquinolones, although being very hydrophilic are mainly eliminated from the aqueous phase through sorption to sludge. This can be possibly attributed to the electrostatic interactions with the cell membranes of the microorganisms (Golet et al. 2003; Lindberg et al. 2006). There are many factors that can affect the removal, besides the compound itself, during the treatment. These factors include the hydraulic retention time (Tauxe-Würsch et al. 2005), the solids retention time (Kreuzinger et al. 2004, Clara et al. 2005), the dilution of raw sewage with rain water (Joss et al. 2006), the temperature (Castiglioni et al. 2006) and of course the type of treatment (Chelliapan et al. 2006, Gros et al. 2006, Al-Rifai et al. 2007; Gomez et al. 2007; Radjenovic et al. 2007; Vieno et al. 2007; Reif et al. 2008).

The main objective of this study was to investigate the occurrence and removal efficiency of 29 pharmaceutical compounds at the three largest urban wastewater treatment plants in Cyprus which apply activated sludge treatment. The pharmaceuticals investigated are: analgesics and anti-inflammatories (ketoprofen, naproxen, ibuprofen, indomethacine, diclofenac, mefenamic acid, acetaminophen, propyphenazone), lipid regulators and cholesterol lowering statin drugs (clofibric acid, gemfibrozil, bezafibrate, pravastatin, mevastatin), psychiatric drugs (carbamazepine, fluoxetine, paroxetine), antiulcer agent (lansoprazole), histamine H_1 and H_2 receptor antagonists (loratadine, famotidine, ranitidine), antibiotics (erythromycin, azythromycin, sulfamethoxazole, trimethoprim, ofloxacin) and ß-blockers (atenolol, sotalol, metoprolol, propranolol). For the study of the compounds a simultaneous determination of the 29 multi-class pharmaceuticals using off line solid phase extraction (SPE) followed by liquid chromatography-triple quadrupole mass spectrometry (LC-MS-MS) was applied.

2. MATERIALS AND METHODS

2.1 Description of sampling sites (UWTP)

Wastewater samples used in this study were collected from three different Urban Wastewater Treatment Plants (UWTP I, II and III) located in the south of Cyprus. This study was designed to cover the efficiencies of different treatment trains by sampling at various stages in the treatment plant. Treatment consists of pretreatment, preliminary treatment, primary sedimentation, secondary (biological) treatment and final tertiary treatment. Pretreated wastewater undergoes a physical process of settling in a primary clarifier. Secondary treatment occurs in pre-nitrification (anaerobic) and nitrification (aerobic) tanks, and two secondary clarifiers (UWTP III) or uses either conventional activated sludge plants including aeration tanks (UWTP II) or oxidation ditches (UWTP I). All of the plants apply a tertiary treatment to decrease the microbial activity of the effluent (disinfection). Table 1 shows the conventional parameters of the three plants in respect to their quality characteristics.

2.2 Sampling

Samples were collected from the inlet, after the secondary treatment step and finally from the outlet. 24-h composite samples were collected in January 2008 using a time-proportional automatic sampler (ISCO 6712). The time between sampling and analysis was less than seven days.

Table 1: Characteristics of wastewater from inlet, after secondary treatment and outlet for the period 06/2007 – 01/2008

UWTP	I			II			III		
Parameter	A	B	C	A	B	C	A	B	C
pH	6.9-8.2	7.3-7.5	7.4-8.4	6.7-8.4	7.4-8.1	7.8-8.7	7.2-8.5	7.5-8.1	7.1-8.4
Temperature (ºC)	12.7-20.4	12.8-18.9	12-19-8	11.1-15.4	11-13.6	10.8-19.3	10.3-13	11.9-12.6	12.4-21.5
Conductivity (µS/cm)	1654-3400	1483-3150	1586-2930	1589-1982	1612-1658	1207-1783	1143-1792	1226-1265	1040-1378
BOD_5 (mg/L)	177-279		1-5.2	135-375	1-2.5	0-4.6	378-599		1-7.2
COD (mg/L)	863-1203	131-226	19-33	689-913	82-117	15-22	887-1236	109-234	21-40
TSS (mg/L)	386-534	3.1-5.6	1-1.5	379-568	4.6-9.7	0.9-1.2	357-675	3-5.2	0.8-1.2
Ammonia (mg/L)	32-99	3.7-28	1.6-5.2	14-46.2	1.45-7.3	0.8-3.3	46-85	3.9-11.7	1-1.6

A: inlet, B: after secondary treatment, C: outlet

2.3 Chemicals

All pharmaceutical standards used were of high purity grade (>90%). Ibuprofen, naproxen, ketoprofen, diclofenac and gemfibrozil were kindly supplied by Jescuder (Rubí, Spain). Indomethacine, acetaminophen, mefenamic acid, clofibric acid, bezafibrate, mevastatin, azythromycin dihydrate, erythromycin hydrate, carbamazepine, fluoxetine hydrochloride, lansoprazole, loratadine, famotidine, ranitidine hydrochloride, sulfamethoxazole, trimethoprim, ofloxacin, atenolol, metoprolol, propranolol hydrochloride and sotalol hydrochloride were purchased from Sigma-Aldrich (Steinheim, Germany). Propyphenazone, pravastatin and paroxetine hydrochloride were from LGC Promochem (London, UK). Isotopically labelled compounds, used as internal standards, were ^{13}C-phenacetin obtained from Sigma-Aldrich, mecoprop-d_3 from Dr. Ehrenstorfer (Augsburg, Germany), ibuprofen-d_3, atenolol-d_7 and carbamazepine-d_{10} from CDN Isotopes (Quebec, Canada).

Individual stock standard solutions were prepared on a weight basis in methanol and stored at -20ºC. A mixture of all pharmaceutical standards was prepared by appropriate dilution of individual stock solutions. Further dilutions of this mixture were prepared in methanol-water (25:75, v/v) before each analytical run and were used as working standard solutions. Stock solutions of ofloxacin, pravastatin and sulfamethoxazole were renewed monthly before their use due to their limited stability. Stock solutions of internal standards were also prepared in methanol and were stored at -20ºC. A mixture of these standards, used for internal standard calibration, was also prepared by diluting the individual stock solutions in methanol.

The cartridges used for solid phase extraction were Oasis HLB (60mg, 3mL and 200mg, 6mL) from Waters Corporation (Milford, MA, USA). 1µm glass fiber and 0.45µm nylon membrane filters were purchased from Whatman (UK).

HPLC-grade methanol, acetonitrile and water (LiChrosolv) were supplied by Merck (Darmstadt, Germany). Hydrochloric acid 37%, NH_4Ac and HAc were from Merck (Darmstadt, Germany). Nitrogen for drying 99.995% of purity was from Air Liquide (Spain).

2.4 Identification and quantification of compounds

The method of analysis is presented elsewhere (Gros et al. 2006) in detail and is shortly illustrated below. Target pharmaceuticals were extracted by off line SPE using Oasis HLB

cartridges (60 mg, 3 mL) at sample pH. Elution was performed with 2 x 4 mL of methanol. The extract was evaporated under a gentle nitrogen stream and reconstituted with 1mL of methanol-water (25:75, v/v). Finally, 10 μL of a 10 ng/μL standard mixture of the internal standards mecoprop-d_3, ibuprofen-d_3, for the analysis in negative ion (NI) mode and ^{13}C-Phenacetin, atenolol-d_7 and carbamazepine-$d_{10,}$ for the analysis in positive ion (PI) mode, were added in the extract for internal standard calibration and to compensate possible matrix effects. LC analysis was performed using a Waters 2690 HPLC system (Milford, MA, USA) coupled to a Micromass Quattro triple quadrupole mass spectrometer, equipped with a Z-spray ESI interface (Manchester, UK). Chromatographic separation was achieved with a Purospher Star RP-18 endcapped column (125x2.0 mm, particle size 5μm) and a C_{18} guard column, both supplied by Merck (Darmstadt, Germany). For the analysis in NI mode, eluent A was methanol and eluent B was water at a flow rate of 0.2 mL/min. The elution gradient started with 20% of eluent A, increasing to 80% in 20 minutes, raising to 90% in a 4 minute gradient and then, back to initial conditions within 3 minutes. The analysis in PI mode was performed using as eluent A a mixture of acetonitrile-methanol (2:1) and as eluent B a buffer consisting in NH_4Ac 5mM / HAc at pH=4.7, also at a flow rate of 0.2 mL/min. The elution gradient started with 15% of eluent A, keeping isocratic conditions for three minutes. Then, eluent A increased to 95% in 22 minutes and was held for 7 minutes. The sample injection volume was set at 20 μL.

Mass spectrometry was performed using a Waters Quattro Ultima triple quadrupole mass spectrometer, equipped with a Z-spray ESI interface (Micromass, Manchester, UK). The mass spectrometer was operated in the multiple reaction monitoring mode (MRM) selecting two transitions for each compound (for detailed explanation see Gros et al. 2006). The first one was for quantitation purposes whereas the second one was used for confirmation. The recoveries obtained were generally higher than 60%. The overall variability of the method was below 15%, for all compounds with method detection limits (MDL) for wastewater matrix varying between 3 and 160 ng/L.

3. RESULTS AND DISCUSSION

3.1 Presence of pharmaceutical compounds in the raw, secondary and tertiary treated effluent

The data obtained from all samples analyzed are presented in Table 2. From the 29 compounds studied 19 were actually determined and quantified. The concentrations of indomethacine, mefenamic acid, pravastatin, mevastatin, fluoxetine, paroxetine, lansoprazole and loratadine were always found to be below the detection limit of the method (MDL) in all samples. The concentrations of gemfibrozil and clofibric acid were below the method detection limit in all samples besides those of UWTP III where no concentrations were detected. Ketoprofen was detected only in the influent of UWTPs I (0.34 μg/L) and III (1.75 μg/L) and in the secondary treated effluent of UWTP III (0.27 μg/L). Santos et al. (2007) reported a concentration range for ketoprofen of 0.2 in the effluent to 2.5 in the influent μg/L. Gros et al. (2006) reported a concentration range for the same compound of 0.13 μg/L in the effluent to 0.97 μg/L in the effluent. Therefore, the concentrations determined in Cyprus fall within an expected range of concentration.

Naproxen was found only in the influent (0.21 μg/L) and the secondary treated effluent (0.03 μg/L) of UWTP III. Conversely, Santos et al. (2007) pointed out a concentration range for this compound of 1.1-27.4 μg/L in the influents and of 0.22 to 4.28 μg/L in the effluent samples. Gros et al. (2006) reported a concentration range of bld-0.16 μg/L in the effluents and of blq-0.19 μg/L in the influents. Hence, the concentration of this compound was found in low levels in Cyprus in comparison to the other studies. Propyphenazone was found only in the secondary and finally treated effluent at UWTP I (0.03-0.04 μg/L correspondingly). Gros et al. (2006) reported a concentration level that was below the detection limit while Koutsouba et al. (2003) report a concentration range of 0.01-0.02 μg/L. Clofibric acid and gemfibrozil were not found to exist in UWTP II while in all other samples were found to be below MDL. Famotidine, ranitidine and trimethoprim were found in the influents and in the samples taken after the secondary treatment in

all treatment plants. Their concentrations though, were below the MDL in the final effluents at all treatment plants. Besides Gros et al. (2006), no other studies included the examination of the existence of these compounds in urban wastewaters. In that study, the mean concentration of ranitidine in the influents was 0.188 μg/L and in the effluents 0.135 μg/L while famotidine was always found below the limit of detection of the method.

Table 2: Detected levels of the target analytes in wastewater samples (in μg/L)

UWTP	I			II			III		
Compounds	A	B	C	A	B	C	A	B	C
Ketoprofen	0.34	bld[1]	bld	bld	bld	bld	1.75	0.27	bld
Naproxen	bld	bld	bld	bld	bld	bld	0.21	0.03	bld
Ibuprofen	1.43	0.52	bld	1.31	0.28	0.28	2.20	4.34	3.46
Indomethacine	bld	bld	bld	bld	bld	bld	bld	bld	bld
Diclofenac	0.61	2.11	0.68	2.43	15.41	5.51	0.73	2.99	0.12
Mefenamic acid	bld	bld	bld	bld	bld	bld	bld	bld	bld
Acetaminophen	309.29	0.11	0.07	77.56	0.11	0.07	405.37	0.05	0.10
Propyphenazone	bld	0.04	0.03	bld	bld	bld	bld	bld	bld
Clofibric acid	bld	bld	bld	0.00	0.00	0.00	bld	bld	bld
Gemfibrozil	bld	0.00	bld	0.00	0.00	0.00	bld	bld	bld
Bezafibrate	0.51	0.14	0.05	0.73	0.29	0.22	0.99	0.14	0.11
Pravastatin	bld	bld	bld	bld	bld	bld	bld	bld	bld
Mevastatin	bld	bld	bld	bld	bld	bld	bld	bld	bld
Carbamazepine	0.76	0.84	0.57	14.45	24.54	27.27	2.61	1.49	1.38
Fluoxetine	bld	bld	bld	bld	bld	bld	bld	bld	bld
Paroxetine	bld	bld	bld	bld	bld	bld	bld	bld	bld
Lansoprazole	bld	bld	bld	bld	bld	bld	bld	bld	bld
Loratadine	bld	bld	bld	bld	bld	bld	bld	bld	bld
Famotidine	1.00	0.59	bld	2.78	5.06	bld	1.30	0.38	bld
Ranitidine	0.07	0.08	bld	0.14	0.47	bld	0.43	0.31	bld
Erythromycin	0.38	0.20	0.03	0.28	0.25	0.40	0.70	0.42	bld
Azythromycin	1.15	1.60	0.18	0.66	0.30	0.20	1.68	0.53	0.03
Sulfamethoxazole	1.07	0.19	0.01	1.51	0.78	0.46	5.41	0.64	0.03
Trimethoprim	0.05	bld	bld	0.14	0.09	blq[2]	0.35	0.06	bld
Ofloxacin	22.62	3.02	1.29	34.74	5.93	4.82	59.38	3.33	1.90
Atenolol	3.29	0.12	0.13	3.15	0.89	0.73	5.81	0.92	0.94
Sotalol	2.81	0.10	0.11	2.70	0.76	0.62	4.97	0.79	0.81
Metoprolol	1.30	0.98	0.57	2.09	1.23	9.59	1.49	1.31	0.69
Propranolol	0.27	0.49	blq	0.41	0.59	0.28	0.23	0.44	blq

[1]bld: below limit of detection
[2]blq: below limit of quantitation
A: inlet, B: after secondary treatment, C: outlet

In all influent samples, all the β-blockers, all the antibiotics, famotidine, ranitidine, carbamazepine, bezafibrate, acetaminophen, diclofenac, and ibuprofen were detected above their LOQ. The highest concentrations recorded were for acetaminophen; 405.37 μg/L at UWTP III, 309.29 μg/L at UWTP I and 77.56 μg/L at UWTP II. Acetaminophen was found in the raw wastewater at significantly higher concentrations than any other target compound. A similar result was obtained by Gómez et al., (2007). In their study, acetaminophen was also found to be in much higher concentrations than any other compound but the concentration found (up to 246 μg/L) was lower than the one found in the present study in Cyprus. In both studies however, despite this very high input, the elimination efficiency reached during the treatment was almost 100%. This can be attributed to the fact that the compound probably undergoes a rapid biodegradation since the

complete removal was achieved already during the secondary treatment (Table 2). The concentrations of ibuprofen fluctuated between 0.28 and 4.34 μg/L, which is in accordance with other concentrations reported by other studies for this compound (Tauxe Würsch et al. 2005; Buser et al. 1999; Stumpf et al. 1999). However in the study of Santos et al. (2007) higher concentrations were observed i.e. 12.1-373.1 μg/L in the influent samples and 0.8-4.82 μg/L in the effluent samples. Moreover, in the study of Gómez et al. (2007) the concentrations of ibuprofen reached 168 μg/L. Ofloxacin showed the second highest concentration; 59.38 μg/L at UWTP III, 34.74 μg/L at UWTP II and 22.62 μg/L at UWTP I. Vieno et al. (2007) report a mean concentration of 100 ng/L in the influent and 14 ng/L in the effluent while in the study undertaken by Gros et al. (2006) the concentrations of ofloxacin were bld in al samples examined.

The pharmaceutical residues were not entirely eliminated by the applied treatment processes. The β-blockers, antibiotics (except trimethoprim), and carbamazepine, diclofenac, bezafibrate and ibuprofen were found in all of the effluent samples of the three UWTPs. The highest concentration value in the effluents was found for carbamazepine (27.27 μg/L at UWTP II).

The concentrations of pharmaceuticals in the urban wastewaters depend on several parameters. For example, the consumption pattern of the drugs varies from country to country and also within country. Further, the flow rate of the raw and treated sewages varies and so does the efficiency of the treatment process to eliminate pharmaceuticals. The concentrations can vary greatly among UWTPs and even within an UWTP at different time periods as can be seen in the study of Tixier et al. (2003). Variation in the effluent loads is expected since the load is affected by the efficiency of the treatment process to eliminate pharmaceuticals. The difference however that is observed for a number of compounds like for example sulfamethoxazole, carbamazepine, acetaminophen, diclofenac and ketoprofen in the influent of the three UWTPs is most probably attributed to variations in the consumption patterns among regions. The efficiency of the treatment process to eliminate pharmaceuticals is discussed below.

4. CONCLUSIONS

The analytical method described in the present work, based on SPE following by LC-ESI-tandem MS, proved to be a reliable and rapid evaluation tool for the pharmaceutical residues examined.

While the data are still the results of one short-term study, the presence of many pharmaceutical compounds is confirmed. In addition to this, in comparison with the results obtained by other studies in other countries some of the compounds quantified in this investigation, high concentrations were obtained for the final effluents (e.g. ofloxacin at all UWTPs, diclofenac at UWTP II, carbamazepine mainly at UWTP II and also at UWTP III, metoprolol at UWTP II). It is also apparent that the biological (secondary step) and physico-chemical treatments (tertiary step) do not have the same impact on the removal since the majority of the compounds are removed during secondary treatment.

Since the treated effluents are used for irrigation purposes, negative environmental impacts may occur over long periods where concentration to these compounds to higher levels occurs through partitioning to soil and possibly plants, persistence and bioconcentration. In addition, a lot of work is still to be done in the direction of chronic toxicity, estrogenicity, genotoxicity and a number of other biological effects. Moreover, a lot of effort must be directed towards the evaluation of the effects of pharmaceutical degradation products and their potential concentrations in the treated effluents and also finally the existence of such compounds in crops irrigated by treated effluents. One has to bear in mind that since the toxicity of one pharmaceutical could be enhanced by the presence of the compounds with similar activity the overall risk could be important. Therefore further work is needed to determine tailored guidelines for different water reuse applications.

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TOXIC METALS, CADMIUM AND LEAD IN SOIL AND PLANTS IN THE FLOODED AREA OF THE TISA RIVER

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Abstract: As the result of current and historical metal mining, floodplains in many parts of the world have become contaminated by metal-rich waste in concentrations that may pose a hazard to human livelihoods and sustainable development. Environmental and human health impacts commonly arise because of the prolonged residence time of heavy metals in river sediments and alluvial soils and their bioaccumulatory nature in plants and animals.
Therefore this paper points out the problem of heavy metals accumulation along the alluvial banks of the river Tisa that are used in agricultural production. The contamination of soil and plant material is of anthropogenic origin. There are facts that indicate the accident in Romania in 2000 contributed to the accumulation of heavy metals in the area around the river and their migration into plants.
Measurements were conducted on the soil by the river in the vicinity of the town of Senta, and on selected fruit and vegetable sorts from that area. Considering the high concentrations of lead and cadmium and their eco-toxicity, these two metals have been specially reviewed. The residual concentration of metals available to the plant was measured in the soil at distances of 530, 580, 630 and 680 cm from the river and at 30 - 60 cm depth. As plant materials, in which the presence of various heavy metals was detected in higher concentrations, were used carrot, broccoli, summer and autumn cabbage, and berries. A highly raised content of almost all elements was found in locations flooded by the river in comparison to the soil sample taken 680 cm from the bank, which presented the reference level. The concentrations of lead and cadmium were multiply increased in some plants. In addition to this, a higher level of other toxic elements such as Cu, Zn and Se was noticeable.

Key words: heavy metals, accumulation, alluvial soil, plants, the Tisa River

1. INTRODUCTION

Natural waters are frequently contaminated by trace metals as a result of human activities. Metals differ from other toxic substances in that they are neither created nor destroyed by humans. Humans have used heavy metals in many different areas for thousands of years. This use influences their potential for health effects in at least two major ways: first, by environmental transport, that is, by human or anthropogenic contributions to air, water, soil, and food, and second by altering the speciation or biochemical form of the element. Since surface soil represents a main sink, it is a suitable material to define the levels and spatial distribution of pollutants, associated with wet and dry depositions. Heavy elements in atmospheric emissions mainly accumulate in surface soils and the pollution usually decreases with soil depth (Dumontet et al. 1990). Furthermore, the retention of heavy elements in surface soils may favour their transfer to the biosphere via uptake by wild plants and crops. This involves a serious risk of contamination of the food chain.

Owing to their toxicity persistence and tendency to accumulate in water and sediment, heavy metals and metalloids, when occurring in higher concentrations, become severe poisons for all living organisms (Has-Schön et al. 2006).

There are facts that indicate the accident in Romania in 2000 contributed to the accumulation of heavy metals in the alluvial area around the river Tisa and their migration into plants. Therefore, the aim of this study was to determine the concentration of heavy metals in the soil and different kinds of fruits and vegetables for human consumption. In plenty of elements detected in plants which have been raised on flooded area of Tisa, Pb and Cd stand out due to their eco-toxicity.

2. TOXIC METALS IN ENVIRONMENT

2.1 Toxic metals in soil and plants

Research throughout many years has showed that the concentrations of heavy metals, especially in urban and industrial centers are enormously high. Furthermore, high concentrations of heavy metals constitute a significant potential threat to human health because they are associated to many adverse effects on health (Jarup 2003). Although several adverse health effects of heavy metals have been known for a long time, the exposure to these elements continues. Moreover, it is even increasing in some parts of world, in particular in the less developed countries, though emissions have declined in most developed countries over the last 100 years.

Primary sources of high concentrations of heavy metals in soil are mostly of anthropogenic kind. An increase in contaminant emission may have a substantial implication on local agriculture, as heavy metal and other chemical material may enter and even accumulate in agricultural soils through agricultural activities and atmospheric deposition, which could enhance the risk of chemical contamination of the food for human use as well as the food chains in agricultural ecosystem (Tripathi et al. 1997).

Each soil type has a diverse heavy metal accumulating capacity. The capacity could be significantly influenced by many different factors such as pH, total capacity, cation adsorption, presence of clay, organic substances, Fe, Al, Mn -oxides etc (Blagojević 2003).

Heavy metals are classified by their harmfulness to essential and toxic ones. Iron, copper, zinc, molybdenum, manganese, cobalt, selenium, iodine, magnesium and nickel, could be considered as essential metals. These elements become toxic as they exceed the harmless level in the tissue. All plants are capable of uptaking and accumulating essential elements, which are necessary for their normal physiology. Moreover, some plants have ability to uptake elements that have no physiological functions. Those unusable elements present the second group of heavy metals which are very toxic: lead, mercury, arsenic, cadmium, chromium and cobalt.

Plants can be also classified by their way of treating heavy metals to: metal excluders, indicators and accumulators. Metal excluders have two ways to prevent adverse effects of heavy metals. The first one is to block heavy metal's approach to metabolically active compartments such as cytosol, mitochondria and chloroplast (Memon at al. 1980, 1981). This way heavy metal is bound to root cells' walls or vacuole. This mechanism implicates metal binding to root cells' walls or to vacuole. The second defense mechanism is a toxic shock prevention and compensation by mycorrhizas, ectomycorrhizas, heat shock proteins, amino and organic acids, peptides, phytochelatins and metallothioneins (Rauser 1999).

The indicators are kind of plants which accumulate certain concentrations of heavy metals in their overground parts. Level of heavy metals in their tissues generally reflects metal levels in the soil on which these plants grow. The accumulators and the hyper accumulators have various ways of inner metal distribution. These plants may localize selected metals mostly in their roots and stems, or they may accumulate and store other metals in their nontoxic form for later distribution and use (Memon at al. 2001).

In the tissue of distinct accumulators elevated heavy metal concentrations may far exceed those present in the nearby nonaccumulating plants just like in the soil on which they grow. It has been proposed that a plant containing more than 0.1% of Ni, Co, Cu, Cr or Pb or 1% of Zn in its leaves on a dry weight basis is called a hyperaccumulator, irrespective of the metal concentration in the soil (Baker 1990). Accumulators are often used for phytoremediation purposes. It is of great interest that plant species, which have no exclusion mechanism, absorb and translocate large concentrations of metals in their roots and accumulate them in their growing parts, especially in their leaves, without showing any toxicity symptoms (Brooks 1983).

A direct and an indirect influence of heavy metals in soil to the plant growth and its physionomy depends on metal form rather than its concentration level. The metal form also has decisive role in metal transfer from plants to other organisms (Van Assche 1990). Physiologically, toxic processes induce structural and functional cell membrane disruption by changing its permeability and disturbance of electrolyte concentration ratio in the plasma. These attained negative effects proceed

disorders in photosynthesis and in watery, mineral and breathing regime. These disorders lead further to reductions in production of the organic matters and to changes in chemical structure in the plant's tissue. In addition, heavy metals excess may stimulate the formation of free radicals and reactive oxygen species, perhaps resulting in oxidative stress (Dietz at al. 1999). Growing slowdown and stunting are the first morphological signs and visible symptoms of heavy metals' harmful effects on plants. Ultimately, these effects are demonstrated through the premature extinction of the oldest leaves, slowdown in growth of young, and plant's death as a result of extremely high concentrations of heavy metals.

2.2 The bioaccessibility and toxic effects of lead

Lead is one of the most ubiquitous and useful metals known to humans and it is detectable in practically all phases of the inert environment and in all biological systems. The main sources of lead pollution are mines, smelters and lead processing operations. Despite of restrictions in using lead as a fuel additive, exhaust gases are still a significant source of this metal (Castro-Gonzales 2008).

There are diverse physical and chemical factors qualifying lead's spreading into environment. In nature, lead exists in two inorganic forms like Pb (II) and rarely Pb (IV), and also in some organic compounds. It is usually found combined with two or more other elements to form lead compounds. Lead accumulates in first layers of soil like other metals.

Solubility and size of lead containing particles are the main determiners of accumulation and toxic influence of lead. The metal is absorbed into an organism and transported by the blood stream to the other tissues. Once absorbed, lead accumulates in high concentrations in bones, teeth, kidney, brain and spleen. Lead in blood has an estimate half-life of 35 days, in soft tissue 40 days and in bones 20-30 years (Goyer 2001).

2.3. The bioaccessibility and toxic effects of cadmium

Cadmium is an industrial and environmental pollutant usually combined with ionic forms of oxygen, chlorine or sulfur. Natural as well anthropogenic sources of cadmium, which include industrial emissions and the application of fertilizer and sewage sludge to farm land, increased cadmium environmental levels (Castro-Gonzales 2008). Almost the whole amount of released cadmium deposits to the ground accumulating with time. Its mobility and accesibility for plants increases with the increasing of acidity of the soil. Majority of plants absorbs and transports cadmium in Cd^{2+} form. After being uptaken by the root system, Cd rapidly reaches overground plant's organs. Green leafy vegetables like lettuce, spinach, cabbage are specially susceptible to elevated cadmium concentrations. Changes in plants and microorganisms caused by high cadmium concentrations lead to changes in soil structures and ultimately in the whole ecosystem. In the general human population, cadmium exposure occurs mainly through sources: the first source is contaminated water and food while the second is inhalation of cadmium particles during industrial or everyday activities. Cadmium affects adversely a number of organs and tissues such as: kidneys, heart, lungs, skeletal systems, brain and the central nervous system (Goyer 2001).

3. MATERIAL AND METHODS

The Tisa river presents a big international river. It is 966 km long and flows along middle Europe to its mouth to the Danube river in Serbia. The northern alluvial bank of the Tisa in Serbia is used in agricultural production. This regions is highly influenced by the river. One of the evidence of these influences is a heavy metal accumulation caused by flood waves, which have occurred

frequently in the past few years. Namely, in 2000 a Romanian mine accident induced 200 000 m^3 of wastewater which came all way to Serbia. These water waves caused flooding and soil contamination with heavy metals as the main components of waste from the mine. As a consequence of this accident, heavy metals have persisted and will persist in the flooded area and in the river bottom sediment for many years.

As the location for sampling, flooded area along the river Tisa was selected. Samples were taken by sond at 118th km in the close vicinity of town Senta from different depths (soil samples from 0 to 30cm 0 to 60cm), to assess the horizontal and vertical differences in pollution. Distances from the river flow were 530, 580 and 630 cm, along with reference distance of 680 cm.

Soil samples were dried at 105 ^oC to a constant mass, and afterwards determination of heavy metals in the soil available for plants was performed with Atomic Absorption Spectroscopy using graphic civete (GFAA-Varian Spectra AA800), with Lakanen Ervio's method (Msz. 214770-50.98.3.3 1998).

In order to relate heavy metals' concentrations in the analyzed soil to human's exposure to pollutants through contaminated food, different kind of plants were sampled at 118th and 110th river km in the same alluvial area. As plant materials were used carrot, broccoli, summer and autumn cabbage and berries. Content of heavy metals in chosen plants was analyzed with Atomic Absorption Spectroscopy according to Hungarian standard (Msz. 1483-3.98 1998) since all measurements were taken in a laboratory of Institute of agriculture and farming, as a part of Corvinus University in Budapest, Hungary. The survey was carried out in 2005 during the seasonal period.

4. RESULTS AND DISCUSSION

If we compare heavy metals' concentrations tested on the Tisa's territory, with concentrations in reference soil that is observed as concentration fund, it could be seen that alluvial area of the Tisa River is far richer with the most of elements. In that context concentration ratio was defined with coefficient K:

$$K = \frac{C_m}{C_n}$$

in wich:

C_m - heavy metal concentration at the experimental distance from the river bank
C_n - heavy metal concentration at the reference distance from the river bank.

Results of the elemental analysis are given in Figures 1 and 2 for two selected elements, cadmium and lead which were available to examined plants in the soil.

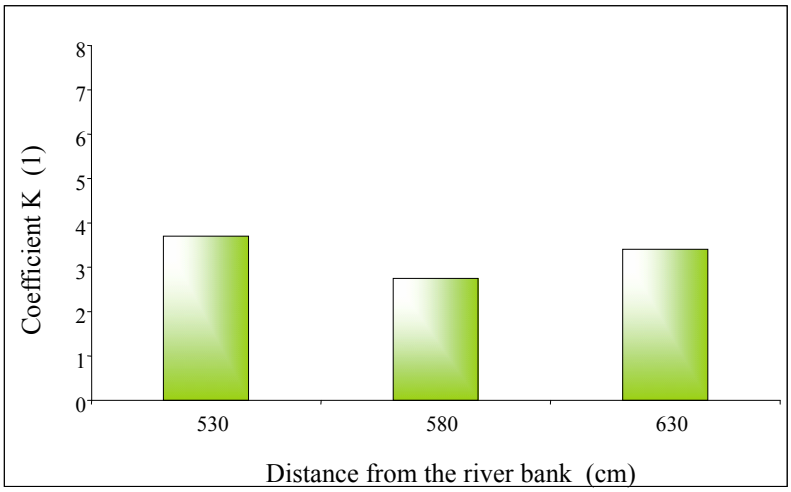


Figure 1. Value of coefficient K for cadmium in the Tisa river alluvial area

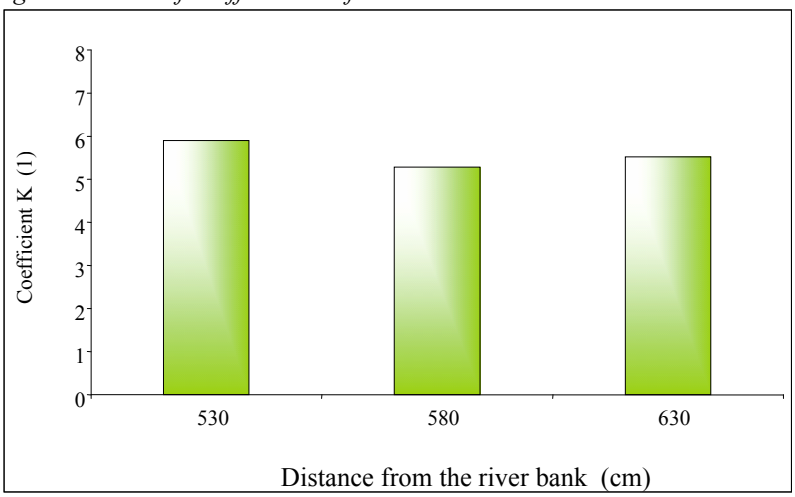


Figure 2. Value of coefficient K for lead in the Tisa river alluvial area

It is obvious that the concentrations of cadmium and lead in alluvial area of the Tisa are higher in the flooded soil rather than in the reference part. The highest concentrations were found in the soil samples at the area nearest to the river.

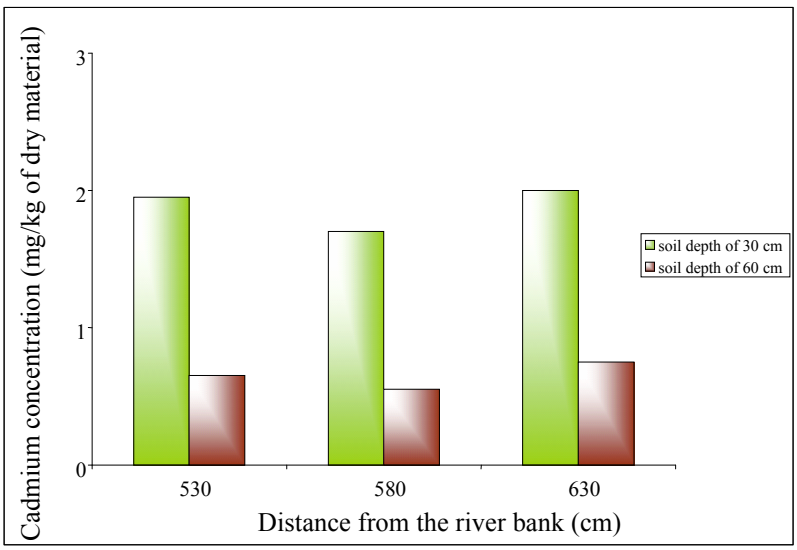


Figure 3. Cadmium concentration values in relation to soil depth

Furthermore, both elements showed variation in concentration with depth, Figures 3 and 4. Analysis of soil samples showed that the concentrations of Cd and Pb related to mining activities

significantly increases with depth. This acknowledges anthropogenic origin and slow translocation process of these two metals. In addition to this, Pb concentration in the Tisa's bank topsoil exceeded worldwide average for soil (17mg/kg, Koljonen 1992).

Although the metal concentrations in sediment samples were relatively high, the acute toxicity potential was low. The analyzed soil samples contained no substances in acutely toxic concentrations to the selected test species.

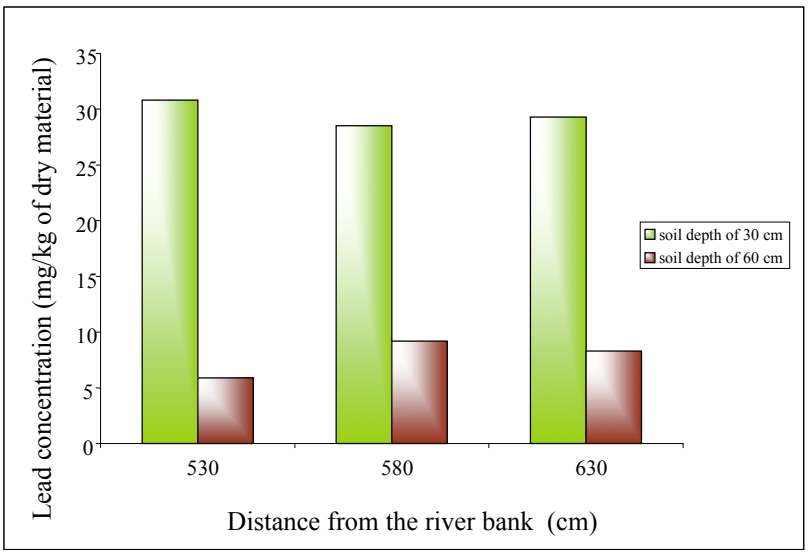


Figure 4. Lead concentration values in relation to soil depth

Next figures, figures 5 and 6, show measured concentration of cadmium and lead respectively in carrot and summer cabbage planted in the alluvial area by the Tisa river. Cadmium and lead concentration were compared with maximally allowed concentration MAC of these metals in the plants cultivated in the contaminated soil.

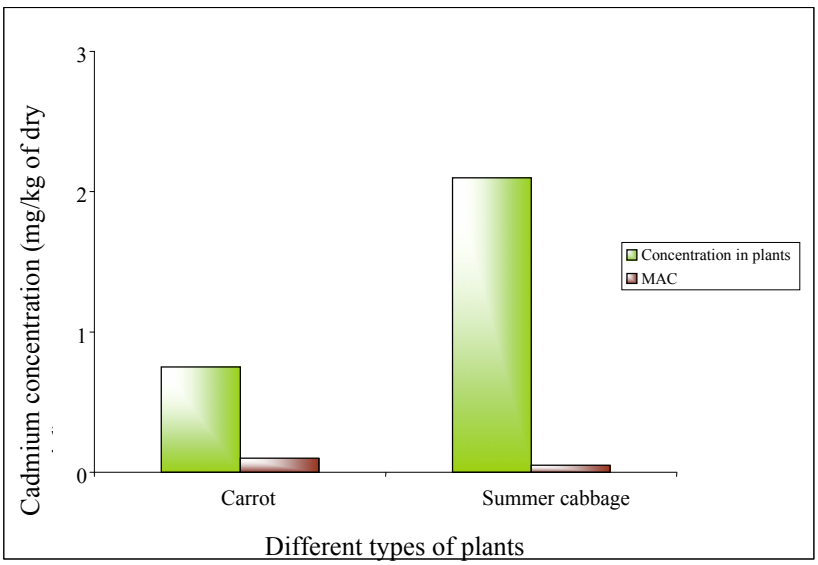


Figure 5. Cadmium concentration in plants in the Tisa river alluvial area
**MAC - maximally allowed concentration*

The highest metal concentrations were observed in the summer cabbage which speaks to higher metal accumulation in the mentioned plant and plant's susceptibility to these heavy metals. Comparing the measured values with the one that we could find in Hungarian standards (Msz. 214770-50.98.3.3, Msz. 1483-3.98) we concluded that the concentrations of Pb and Cd were far more higher in singular plants.

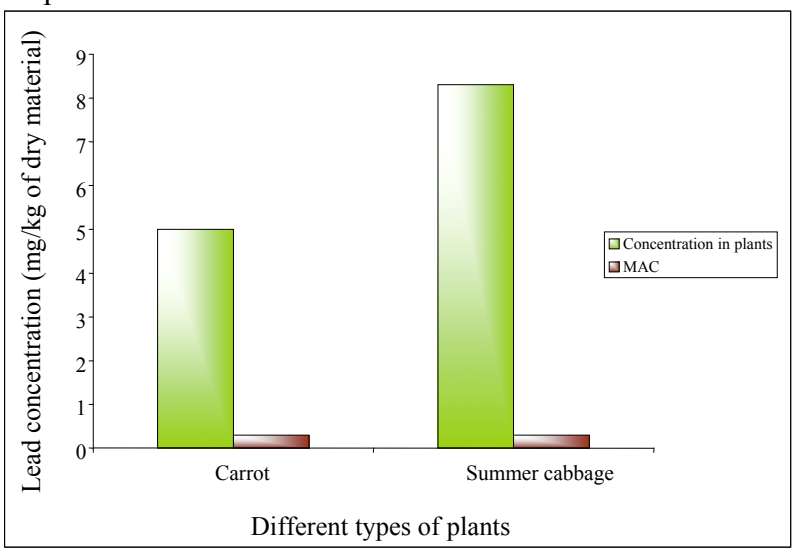


Figure 6. Lead concentration in plants in the Tisa river alluvial area

Research of accumulated elements in plants, cultivated in the same area, was conducted as well on cabbage, broccoli, carrot, red and black currant. The results of the elemental analysis are given in the Table 1.

Table 1. Analysis of other elements accumulated in examined plants
** LOD - Limit of detection*

Unit	**mg / kg dry material**					
Plants	**Carrot**	**Broccoli**	**Cabbage**		**Currant**	
Element			**summer**	**autumn**	**black**	**red**
Al	180,80	29,57	25,32	22,25	26,71	33,05
As	< LOD*	< LOD*	< LOD*	< LOD*	< LOD*	< LOD*
Ba	15,16	3,06	35,42	3,71	2,45	2,94
Cd	0,78	0,22	2,06	0,27	< LOD*	0,38
Co	< LOD*	< LOD*	2,63	< LOD*	< LOD*	0,13
Cr	1,88	< LOD*	9,09	< LOD*	< LOD*	< LOD*
Cu	11,65	3,93	17,40	4,46	10,34	9,31
Fe	349,9	61,49	52,00	66,08	50,79	52,47
Ga	< LOD*	< LOD*	4,43	< LOD*	< LOD*	2,76
Li	0,26	< LOD*	3,94	< LOD*	< LOD*	0,22
Mn	18,31	15,80	140,7	16,18	11,67	10,50
Mo	0,26	0,84	2,42	0,65	0,24	0,24
Ni	< LOD*	< LOD*	9,55	< LOD*	0,54	1,17
P	2237	3666	1875	3870	3300	2972
Pb	4,97	< LOD*	8,314	< LOD*	0,36	1,36
Se	< LOD*	< LOD*	< LOD*	< LOD*	1,18	0,72
Si	143,2	42,36	174,3	37,28	25,85	36,57
Sr	16,77	10,67	93,60	12,21	7,13	9,20
V	0,55	0,24	3,19	< LOD*	0,25	0,25
Zn	31,18	24,96	51,38	24,79	19,40	14,97

High concentrations of some heavy metals were noticed in some particular plants as given in the Table 1. The presence of Mo and Se was not proven in the soil, but still found in plants. Presence of Se could be considered as a consequence of using Se in the production of Fermin factory in Senta.

5. CONCLUSION

During the accidents in Romania in 2000, new wave of polluted water arrived in Tisa, bringing 20000 tones of mud and debris containing different heavy metals. Consequences of the accidents cannot be calculated yet as they will affect river ecosystem and its flooding area for many years. Once heavy metals reach the soil, they can persist there for thousands of years which implies a strong recommendation for monitoring, preventing and control of heavy metal spreading into environment. It is necessary to investigate potentially polluted soil, and in the case of proven contamination, it is important to prevent expansion of pollutants before they reach the food chain.

Analyzing the samples, it was concluded that the flooded area of Tisa (downstream from Senta) with agriculture process performed, was contaminated with heavy metals acceptable for plants. The concentration levels were higher in flooded than in area that was not flooded, Cd's 3 times and Pb's 5 times higher. It is obvious that these pollutants originated from the river. The soils in the Tisa river area have a high retention (buffer) capacity of heavy metals with contents of Pb and Cd decreasing with soil depth. High concentrations of toxic heavy elements in soil implied that pollutants were also absorbed by plants. In fact, the edible parts of plants consumed by animals and humans in this area contained variable and mostly unpredictable quantities of heavy elements (mainly Pb and Cd) in relation to local physical and chemical parameters and plant species. Recent soil research, within contaminations of cultivable and uncultivable soils were compared, directly dismissed agrochemical measures as a potential cause of detected heavy metal pollution.

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REMOVAL OF SOME METAL IONS FROM AQUEOUS SOLUTIONS USING A MAGNETIC ION EXCHANGE RESIN WITH SULPHONATE FUNCTIONALITY

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Abstract: Ion exchange onto low cost media such as synthetic resin ion exchangers offers an attractive and inexpensive option for the removal of metal ions from aqueous solutions. Magnetic resin ion exchangers have special interest due to easy separation of the loaded resin from waste stream using an external magnetic field. In this work, magnetic ion exchanger with sulphonate functionality was prepared and characterized using FT-IR. The investigated resin is characterized by easy of solid/liquid separation due to the imparted magnetic properties of resin. Good stability in both HNO_3 and NaOH where an uptake efficiency of 98.1 ±0.5% was recorded over three measurements. The uptake behavior of the resin towards different metal ions using batch experiments at different experimental conditions such as pH, contact time, initial metal ion concentration was studied. It is found that a selective separation of Cu(II) from Ag(I) and Hg(II) in the strong acidic media from their aqueous mixture is obtained. The resin showed a powerful uptake behavior towards the studied metal ions. The uptake follows the sequence Ag(I) > Hg(II) > Cu(II) with maximum uptake efficiencies of 4.1, 3,0 and 2.9 mmol/g, respectively. The resin obtained showed comparable uptakes to the commercial resin (Dowex Marathon C) but with faster kinetics. Kinetic characteristics of uptake process were evaluated where they were found to follow first order kinetics. Thermodynamic parameters (ΔH°, ΔS° and ΔG°) were evaluated. The positive values of ΔH° for all adsorption processes indicate the endothermic nature. The negative values of ΔG° indicate that the adsorption reaction is spontaneous. The observed positive value of ΔS° indicates a more randomness state due to the liberation of water molecules during the exchange process. The regeneration experiments showed efficiency > 95% using 0.4 M EDTA for the studied metal ions. Column experiments were performed to recover the metal ions at different flow rates to conclude the best conditions for using a column of the investigated resin in the industrial field.

Key words: Adsorption, ion exchange, magnetic resins, sulphonate.

1. INTRODUCTION

The increase in nuclear and industrial activities has intensified environmental pollution and deteriorates some ecosystems with the accumulation of pollutants such as heavy metals, synthetic compounds and toxic radioactive wastes (Yamaura et al. 2002). Mercury, for instance, causes damage of the central nervous system and chromosomes, destroying kidney and chest pain. The major sources of mercury pollution in the aquatic environment are industries such as chloralkali, paints, electrical industries, rubber processing, oil refining and fertilizers (Nrmasivayam and Benthilkumar 1997; Atia et al. 2003a). Copper ions due to their toxicity inhibit natural growth and development of plants and affect negatively self-purification of water and living organisms in the water environment. The potential source of copper ions is the industrial effluents including metal cleaning and plating baths beside fertilizer industry (Özer et al. 2004; Atia et al. 2003b). On the other hand, some metal ions possess economic values such as silver. Silver is used in silverware, metal alloys, and beverage processing (Atia et al. 2005). Many separation methods such as chemical reduction, membrane separation, biological treatment, ion exchange, precipitation and adsorption have been used to remove metal ions from wastewater (Lin and Juang 2002; Gode and Pehlivan 2003; Maria et al. 2003; Donia et al. 2008). Ion exchange onto low cost media such as synthetic resins offers an attractive and inexpensive option for the removal of dissolved metals (Donais et al. 1999). Cation-exchange resins generally contain bound sulfonic acid groups; less commonly, these groups are carboxylic, phosphonic, phosphinic, etc. (Lin et al. 2000; Pehlivan and Altun 2006). Magnetic resins have special interest due to the easy separation of the loaded resin from waste

stream using an external magnetic field (Atia et al. 2007; Lee et al. 2003; Zhang et al. 2001). The major goal of the present study is the preparation of a magnetic glycidyl methacrylate ion exchanger loaded with sulphonate functionality. The adsorption behavior of the resin obtained towards Hg(II), Cu(II) and Ag(I) was investigated and compared with a commercial resin (Dowex Marathon C). Kinetic and thermodynamic characteristics of adsorption process have also been highlighted.

2. EXPERIMENTAL

2.1 Chemicals

Glycidyl methacrylate (GMA), divinylbenzene (DVB-55%), benzoyl peroxide (Bz_2O_2) and sodium hydrogensulphite were obtained from Aldrich (USA). All other chemicals were of analytical grade and were used as received. Silver nitrate, cupper acetate and mercuric chloride were used as sources for their corresponding metal ions.

2.2 Preparation of Magnetic Resin Immobilized With Sulphonate Moities

The preparation process was carried out through subsequent treatment of magnetic GMA/DVB resin with sodium hydrogensulphite (Bondar et al. 2004). The parent GMA/DVB magnetic resin was prepared as early reported (Atia et al. 2007).

One gram of the resin (GMA/DVB) was reacted with 4 g of sodium hydrogensulphite dissolved in water/dimethylformamide solution at weigh ratio of 2:3:7 of $NaHSO_3$:H_2O:DMF. The suspension was heated at 70^{o}C for 72 h to give the resin R-SO_3Na. The success of modification process was obtained from FT-IR Nexeus-Nicolite-Model 640-MSA measurements using KBr pellets. The spectrum shown in Figure 1 displays number of peaks near 1125, 1040 and 682 cm^{-1}. These peaks are characteristics for sulphonate group. On the other hand, the characteristic peaks for epoxy group at 1261, 907 and 757 cm^{-1} disappeared from the spectrum. This behavior indicates the success of loading of magnetic GMA/DVB by sulphonate group to give R-SO_3Na.

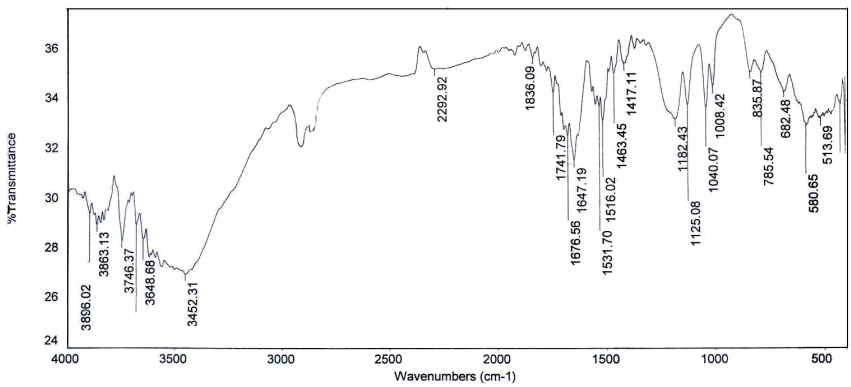


Figure 1. IR spectra of resin R-SO_3Na.

2.3 Adsorption Measurements

Stock solution (0.01 M) of the metal ion under study was prepared by dissolving an appropriate weight of pure metal salt in distilled water. A stock solution (0.01 M) of EDTA was prepared in distilled water and was standardized against a standard solution of $MgSO_4.7H_2O$ using Eriochome Black-T (EBT) as indicator.

Uptake measurements under controlled pH were carried out by shaking 0.1 g of resin with 100 mL of the metal ion solutions of concentration 5 mM using a Vibromatic-384 shaker at 300 rpm.

The desired pH was adjusted using HCl or HNO_3. The equilibration was fixed at 3 h and 25°C after which the residual concentrations of the metal ions were estimated by titration against 5 mM EDTA using murexide and PAR as indicators for Cu(II) and Hg(II) (Atia et al. 2003a,b). Ammonia/ammonium chloride buffer (pH = 10) and acetic/sodium acetate buffer (pH = 4) were used during titration process of Cu(II) and Hg(II). The residual concentration of Ag(I) was determined via titration against 5 mM EDTA using Volhard method (West 1969). The amount of metal ions adsorbed per unit mass of adsorbent (q_e) was calculated by the following equation:

$$q_e = (Ci - C_f)\, V / m \qquad (1)$$

Where q_e is the amount of the metal ion adsorbed in mmol/g, Ci and C_f are the initial and final concentrations of metal ion in solution in mmol/L, V is the solution volume in litres, and m is the mass of sorbent in g.

The uptake of each metal ion by the resin as a function of time was carried out by shaking 0.1 g of resin with 100 mL of the metal ion solution of concentration 5 mM, natural pH (Ag(I) 6.5; Hg(II) 5.8; Cu(II) 5.7) and 25°C. Five milliliters of the supernatant (free of the suspended solid) were taken at different time intervals where the concentration of metal ion in the supernatant was determined.

The adsorption isotherms for each metal ion were obtained at different temperatures and natural pHs (Ag(I) 6.5; Hg(II) 5.8; Cu(II) 5.7). A 0.1 g resin was conditioned with 100 mL metal ion solution with different concentrations for 3 h at 25, 35, 45 or 55°C. Five milliliters of the solution were taken at the end of the experiment where the residual concentration of metal ion was determined.

2.4 Column Studies

Column experiments were performed in a plastic column (length 3.6 cm, diameter 1 cm). A small piece of filter paper was placed at the bottom of the column, and then a known quantity of the resin under study was placed in the column. A metal ion solution with initial concentration of 5 mM was allowed to flow downward through the column at a specific flow rate. Samples were collected from the outlet of the column at different time intervals and analyzed for metal ion concentration. The experiment was terminated when the concentration of the metal ion at the outlet of the column equals the initial concentration.

2.5 Resin Regeneration

Regeneration experiments were performed by equilibrating 0.1 g resin with 100 ml (5×10^{-3} M) of metal ion for 3 h. The total uptake was estimated then the solution was decanted and the resin was washed thoroughly with distilled water. The resin loaded by metal ions was equilibrated for 3 h in 100 ml of 0.4 M EDTA. Afterwards, the resin was carefully washed with distilled water for reuse in the second run of uptake. The efficiency of regeneration was calculated by dividing the total uptake in the second run by that of the first run.

3. RESULTS AND DISCUSSIONS

The stability of the resin against acid and alkali media was examined by soaking 1 g of resin in 0.5 M HNO_3 or 0.1 M NaOH for 24 h. The stability was checked by measuring the total uptake of Hg(II) by the resin before and after acid or base treatment. The resin showed a good stability in both HNO_3 and NaOH where an uptake efficiency of 98.1 ±0.5% was recorded over three measurements.

3.1 Uptake of Metal Ions Using Batch Method

As reported in Table 1, The highest uptake for the three metal ions was reported at the natural pHs. For Cu(II), a small decrease in the uptake was noticed as the pH decreased. This may be attributed to the competitive adsorption between the protons and Cu(II) towards the sulphonate group. For Ag(I) and Hg(II), the uptake decreases dramatically till reaching zero at pH 1.3. This behavior may be attributed to the formation of soluble anionic species such as $AgCl_2^-$, $Ag(NO_3)_2^-$, $HgCl_3^-$ or $Hg(NO_3)_3^-$ which have no ability to interact with negatively charged $-SO_3^-$. This phenomenon may be used for the selective separation of Cu(II) from Ag(I) and Hg(II) in the strong acidic media.

Table 1. Effect of pH on the uptake of the metal ions by resin R-SO_3Na at 25°C

		Uptake (mmol/g)		
Acid	**pH**	**Cu(II)**	**Ag(I)**	**Hg(II)**
HCl	1.3	1.90	0.00	0.00
	1.6	1.98	Precipitation	0.30
	1.9	2.24		0.54
	2.1	2.63		0.90
	3.0	2.90		1.70
HNO_3	1.3	2.00	0.00	0.00
	1.6	2.10	0.40	0.43
	1.9	2.21	0.80	0.71
	2.1	2.73	1.30	1.04
	3.0	2.90	2.10	1.80
Not modified		2.90	3.70	3.00

Figure 2 a,b shows the change in the uptake of both R-SO_3Na and Dowex Marathon C towards the investigated metal ions as a function of time at natural pH (Ag(I) 6.5; Hg(II) 5.8; Cu(II) 5.7). The uptake values obtained follow the order Ag(I) > Hg(II) ≥ Cu(II). On the other hand, the uptake capacities of the investigated resin are comparable to that of Dowex Marathon C but with relatively faster rate.

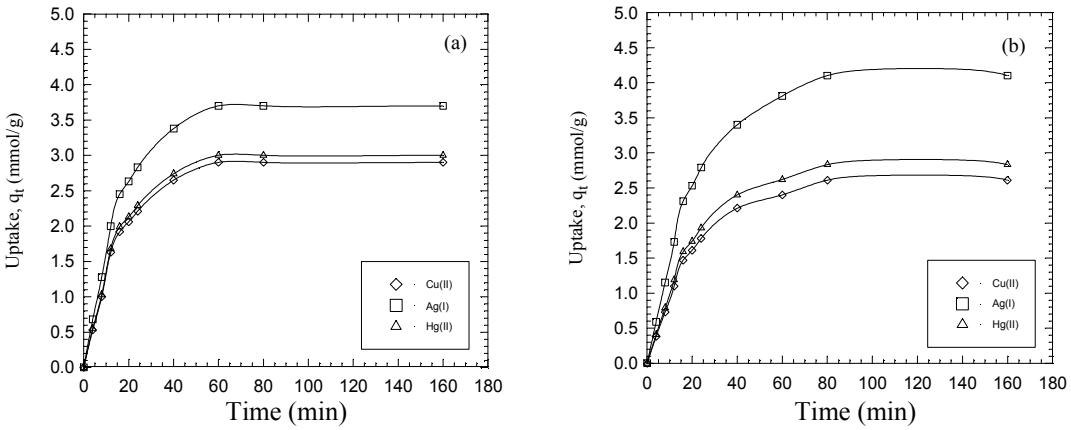


Figure 2 a,b. Uptake of Ag(I), Hg(II) and Cu(II) by resins R-SO_3Na (a) and Dowex Marathon C (b) as a function of time using 5 mM metal ion solutions at natural pH and 25°C.

The time/adsorption data obtained in Figure 2a were treated according to pseudo first order kinetics model

$$\log (q_e - q_t) = \log q_e - (\frac{K}{2.303})\, t \qquad (2)$$

where q_e is the uptake capacity of the metal ion at equilibrium, q_t is the uptake capacity of the metal ion at time t, K is the overall rate constant (min^{-1}). Figure 3 shows the plotting of log ($q_e - q_t$) versus (t) which gave straight lines with good fitting (R^2) to pseudo first order model.

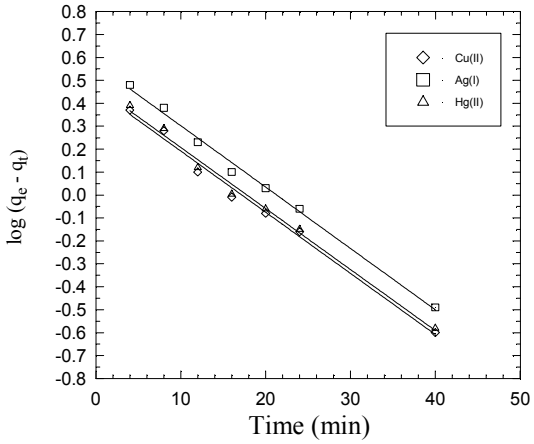


Figure 3. Pseudo first order plots for the adsorption of Ag(I), Hg(II) and Cu(II) on resin R-SO_3Na at natural pH.

From the values of slope (K/2.303) and intercept (log q_e) of these lines, the values of overall adsorption rate constant (K) and equilibrium uptake capacities (q_e) were calculated. The values of q_e were found to be comparable to those experimentally obtained. This indicates that the adsorption reaction proceeds according to pseudo first order kinetics. The effect of intraparticle diffusion on the rate of adsorption was verified from the treatment of adsorption/time data (Figure 2a) according to the following equation

$$q_t = x_{i+} k_p t^{0.5} \quad (3)$$

where q_t is the amount adsorbed (mmol/g) at a given time t (min), k_p (mmol/g $min^{0.5}$) is the intraparticle diffusion rate constant and x_i is the intercept of straight line which is proportional to the boundary layer thickness. The straight lines obtained in Figure 4 indicate that the adsorption is governed by intraparticle diffusion. The negative value of x_i indicates that the boundary layer (external film resistance) has no significant effect on the diffusion rate.

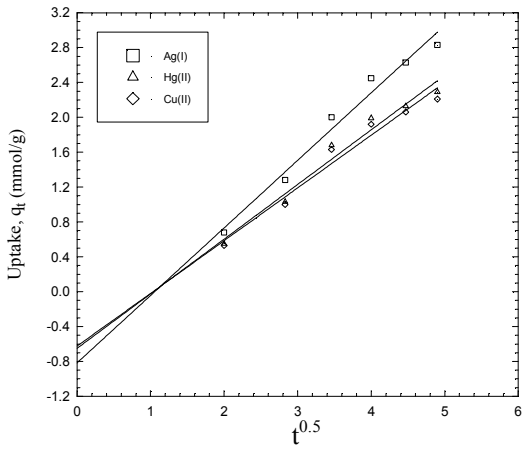


Figure 4. Uptake of Ag(I), Hg(II) and Cu(II) on R-SO_3Na as a function of square root of time.

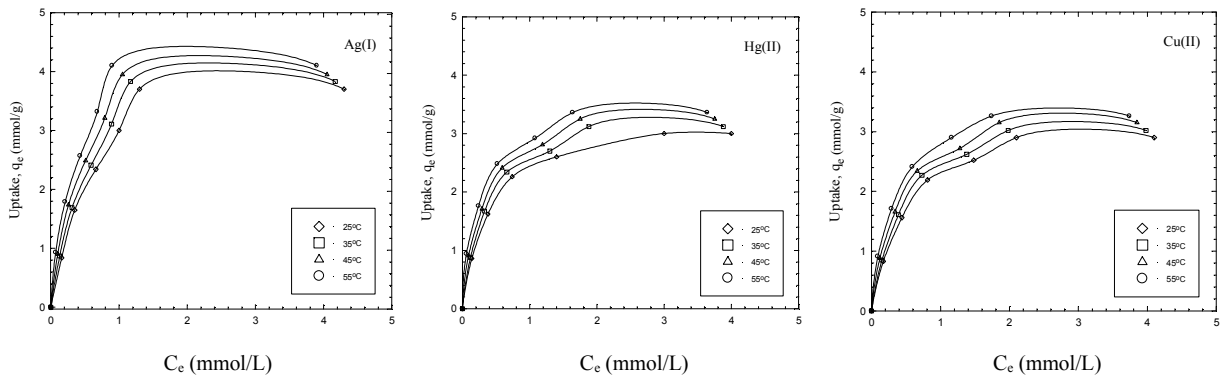


Figure 5. Adsorption isotherms of Ag(I), Hg(II) and Cu(II) on resin R-SO_3Na, at different temperatures.

Figure 5 shows the adsorption isotherms of Ag(I), Hg(II) and Cu(II) on resin at different temperatures and natural pH. It was found that, at all studied temperatures, the uptake follows the sequence Ag(I) > Hg(II) > Cu(II).

The adsorption data of the metal ions on the resin at different temperatures were analyzed according to Langmuir adsorption model (Atia et al. 2005)

$$\frac{C_e}{q_e} = \frac{C_e}{q_s} + \frac{1}{K_{ads} q_s} \quad (4)$$

where C_e is the equilibrium concentration of the metal ion (mM), q_e is the adsorbing capacity at equilibrium (mmol/g), q_s is the maximum capacity (mmol/g) and K_{ads} is the binding constant. Plotting of C_e/q_e vs C_e gave straight lines, indicating that the adsorption process proceeds according to Langmuir isotherm. The values of q_s and K_{ads} were obtained from the slope and intercept of the straight lines, respectively and are given in Table 2. The calculated values of q_s are comparable with those obtained experimentally. The higher q_s value for Ag(I) compared to Hg(II) and Cu(II) is explained by that Ag(I) cation is attached to only one sulphonate site, where Hg(II) and Cu(II) are attached to two sulphonate sites. At the same temperature, the binding constant (K_{ads} value) follows the order Hg(II) > Cu(II) > Ag(I). This trend indicates to the preferential adsorption of (i) ions of higher charge; and (ii) ions of less hydrated volume (Lin et al. 2000; Pehlivan and Altun 2006).

Table 2. Langmuir and thermodynamic parameters for the adsorption of the metal ions on resin R-SO_3Na

		Langmuir parameters				**Thermodynamic parameters**			
Metal ion	**Temp. (°C)**	K_{ads} **(L/mol)**	q_e **(mmol/g)**		R^2	ΔH^o **(kJ/mol)**	ΔS^o **(J/mol.K)**	ΔG^o **(kJ/mol)**	$T\Delta S^o$ **(kJ/mol)**
			Calc	**Exp.**					
Ag(I)	25	2179	4.17	3.70	0.988	16.82	120.16	-18.99	35.81
	35	2660	4.19	3.83	0.992			-20.19	37.01
	45	3185	4.32	3.95	0.995			-21.39	38.21
	55	4093	4.40	4.10	0.996			-22.59	39.41
Hg(II)	25	2689	3.33	3.00	0.997	15.87	118.60	-19.47	35.34
	35	3096	3.44	3.12	0.997			-20.66	36.53
	45	3824	3.52	3.25	0.998			-21.84	37.71
	55	4838	3.58	3.36	0.998			-23.03	38.90
Cu(II)	25	2336	3.27	2.90	0.996	13.96	110.96	-19.12	33.07
	35	2601	3.36	3.02	0.997			-20.22	34.18
	45	3073	3.47	3.15	0.998			-21.33	35.29
	55	3908	3.52	3.26	0.997			-22.43	36.39

Thermodynamic parameters of the uptake process was obtained from the plotting of binding constant (K_{ads}) against temperature according to van't Hoff equation

$$\log K_{ads} = \frac{-\Delta H^\circ}{2.303\,RT} + \frac{\Delta S^\circ}{2.303\,R} \quad (5)$$

The values of ΔH° and ΔS° were calculated from the slope and intercept of that straight line relationship. On the other hand, the free energy of adsorption was obtained from Gibbs free energy equation

$$\Delta G^\circ = \Delta H^\circ - T\Delta S^\circ \quad (6)$$

The values of ΔH°, ΔS° and ΔG° are given in Table 2. The positive values of ΔH° indicate the endothermic nature of the adsorption process. The negative values of ΔG° indicate that the adsorption reaction is spontaneous. The observed positive value of ΔS° indicates a more randomness state due to the liberation of water molecules during the exchange process. The observed increase in the negative values of ΔG^o with increasing temperature implies that the adsorption process becomes more spontaneous at higher temperature for all metal ions. The higher values of $|T\Delta S^\circ|$ relative to $|\Delta H^\circ|$ indicate that the adsorption process is entropically dependent.

3.2 Uptake of Metal Ions Using Column Method

The variation of relative concentration (C/C_o) of metal ion as a function of time at different flow rates and a fixed bed volume were investigated. The data obtained were reported in Table 3.

Table (3) Data of column studies for the uptake of different metal ions on R-SO_3Na at different flow rates.

Metal ion	Flow rate (mL/min)	Breakthrough time, t_b (min)	Saturation time, t_s (min)	q_s (mmol/g)*	Break-through capacity (mmol/g)	Removal efficiency (%)
Ag(I)	1	400	488	3.70	3.33	90.0
	2	198	245	3.69	3.29	89.2
	3	129	168	3.72	3.23	86.8
Hg(II)	1	340	404	3.10	2.83	91.3
	2	165	201	3.05	2.75	90.2
	3	107	134	3.00	2.67	89.0
Cu(II)	1	330	390	3.00	2.75	91.7
	2	158	197	2.95	2.63	89.2
	3	101	131	2.90	2.53	87.2

**Using the equation: $q_s = 0.5\ (v_b + v_t)\ C_o$ /g of resin; v_b and v_t are measured in L; C_o in mmol/L (Atia et al. 2007)*

At the same bed height, both the total adsorption capacity (q_s) and the breakthrough point are obtained within a short time at higher flow rate (Figure 6). For each metal ion, the removal efficiencies were not be significantly affected by changing the flow rate. So, using the flow rate of 3 mL/min for all experiments is the most preferable rate because it is time safer without losing any significant efficiency of removal.

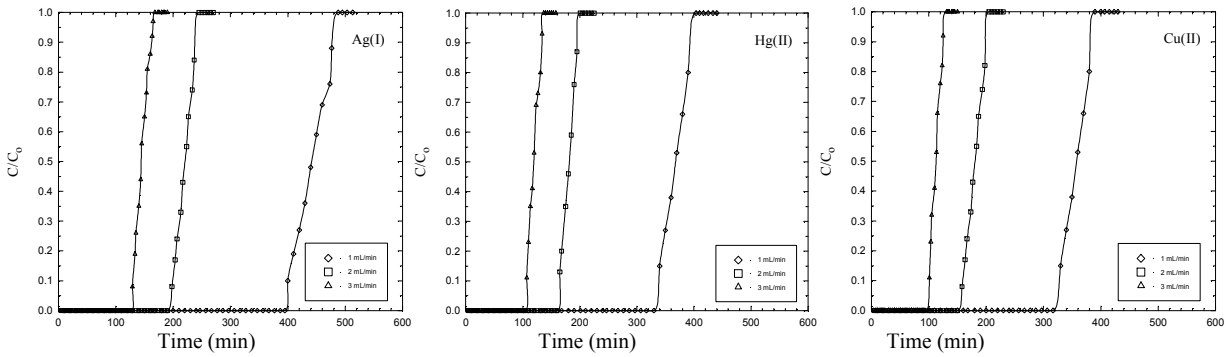


Figure 6. Effect of flow rate on the uptake of Ag(I), Hg(II) and Cu(II) by resin R-SO_3Na.

3.3 Resin Regeneration

The efficiency of regeneration was obtained using the following equation

$$\text{Efficiency of regeneration (\%)} = \frac{\textit{Total adsorption capacity in the second run}}{\textit{Total adsorption capacity in the first run}} \times 100\% \quad (7)$$

The regeneration of the loaded resin was carried out using 0.4 M EDTA. The efficiency of regeneration was found to be 95 ± 1 over five adsorption/ desorption cycles.

3.4 Conclusions

Resin R-SO_3Na showed a high affinity towards the studied metal ions. The investigated resin is characterized by: (i) high uptake values towards different metal ions; (ii) fast kinetics (iii) easy of solid/liquid separation due to the magnetic properties of resin; and (iv) higher durability in acidic and basic media and easy of regeneration with higher efficiency up to 95%. All of the above characteristics make this resin promising in the field of waste and drinking water treatment.

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PHYTOREMEDIATION AS A NATURAL PROCESS TO DECREASE ROAD SALT IMPACTS ON SOIL AND WATER RESOURCES

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Abstract: For increasing driving safety during the cold winters in Nordic countries like Sweden, regional road authorities apply considerable amount of deicing chemicals like salt, in the roads. The kind of salt which is generally used is sodium chloride (NRC, 1991). It is now very clear that chemical deicers have negative effects on the environment. Salt activities have adverse effects on soil, water supplies, vegetation and ecosystem. In this research the capability of Phytoremediation process as a new clean technology has been discussed. In this method plants are used for in situ soil cleanup (UNEP, 2007). Regarding road salt control, the suitable plant must tolerate salinity and frost. In this project Tall Wheatgrass has been introduced as a first nominee for this purpose. This plant is very resistant in saline environment and is able to tolerate winter frost (USDA, 2007). It is predicted that this plant can be used as a natural controller of road salt distribution. By establishing this plant in each side of the highways in width of at least 5 meters and making buffer zones we can control about 500 kg NaCl/Km/year. In very sensitive areas where the water resources are close to the highways or the geometrical conditions are in a way that quality of water resources are in danger this buffer zone could be wider for better protection. By Tall Wheatgrass buffer zones of 10 meters in each side of the highways we can control nearly one tone sodium chloride in each kilometer per year. 500 kg of sodium chloride is removed by harvesting and the same amount will be immobilized in the roots tissue. The results of this project can be validated by field experiments. For any special case, field experiments are recommended before applying this method.

Key words: Phytoremediation, road salt, sodium chloride, groundwater, salinity, Tall Wheatgrass.

1. INTRODUCTION

For increasing driving safety during the cold winters in Nordic countries like Sweden, regional road authorities have applied considerable amounts of deicing chemicals like salt, in the roads. Just in United States 10 million tone salts is applied to the roads every year (FHWA 1989 & TRB 1974). Salt will help the snow to be melted and avoids freezing on the roads. Salt decreases the freezing point of water to about – 10.0 ◦c. The kind of salt which is generally used is sodium chloride which is cheaper than other salts and has more effectiveness on the frost (NRC, 1991). Chemical concentration, time of deicing application, weather conditions, type of road surface, topography and traffic volume all affect the procedure of melting snow and ice. Despite of secure driving which chemical deicers provide, they have negative impacts on the environments. Salt activities have adverse effects on soil, water supplies, vegetation and environmental ecosystem in the vicinity of the roads. These impacts are not confined by just the road vicinity. Since salt, can infiltrate to the groundwater, water supplies can be affected by the road salts even kilometers far from roads. Soil material, hydraulic gradient of the soil layers, salt amount, precipitation value, road topography and infiltration rate are the main factors which influence salt movement in the soil and water. Researchers have tried to find acceptable substitution for regular deicers like sodium chloride. Usage of phytoremediation against road salt is going to be discussed further on. Phytoremediation is a new clean technology which uses plants for cleaning the environment (UNEP, 2007). This method can be used for different contaminations.

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2. AIM, OBJECTIVES, LIMITATIONS, METHODOLOGY

2.1 Aim

The overall aim of this research is to investigate possible ways of applying Phytoremediation processes on road salt contaminations and to searching for alternative plant(s) to be used for cleaning salt pollutions. How to desalinate the environment or at least how to decrease the amount of salt spreading by using the plant(s) is going to be described and discussed.

2.2 Objectives

This research moves toward achieving the following objectives:

- Investigating Phytoremediation processes applicable on salt contaminations
- Finding target plant(s)
- Quantifying the abilities of the plant(s) to decrease salt contamination

2.3 Limitations

The Phytoremediation efficiency depends on the plant species and the environmental conditions where the plant is growing. For achieving efficient function careful consideration must be allocated for choosing the plant. The first main requirement for choosing method is site investigations. The characteristics of the soil and water must be determined. Field tests and site investigations regarding the salt phytoremediation capability of the plant (s) is mandatory to achieve fine result.

2.4 Road salt activity overview

In southern and central Sweden, the highways received 10-14 ton NaCl/Km during the years 1971-1977(Löfgren, 2001). It means an average amount of 1.43-2 ton NaCl/Km/Year. The Swedish National Road Administration (SNRA) uses nearly 200000 to 300000 tones of sodium chloride in national roads every winter (Thunqvist, 2003).

2.5 Methodology

For this project, the first step was gathering information as much as possible about phytoremediation and its ability for site cleanup. Information for the main techniques of phytoremediation and their functions was gathered.

The second step was searching for suitable plant(s). The appropriate plant must have two characteristics of salinity resistivity and coldness tolerance. Combining these two criteria makes it very difficult to find suitable plant. For this purpose different databases and articles have been reviewed. The main reference for the characteristics of the plant for this project is USDA database (United States Department of Agriculture).

3. PHYTOREMEDIATION

Phytoremediation is a new clean technology that can be used in different cases for vast area of environmental problems. In this research it has been tried to find possible solution to avoid or

reduce or remove contamination of road salts in the soil and water with Phytoremediation techniques.

3.1 What is Phytoremediation?

Phytoremediation is the use of living green plants for in situ removing contaminants from polluted soil, water, environment and air (NASC, 2007). For each contamination, special plant that has been examined against the contamination is used to remove the pollution. It is an economic alternative to decrease leaching of contaminants to the soil and groundwater, avoid run off of polluted storm water and beginning an early step of clean up.

3.2 Phytoremediation against salt contaminations

To use Phytoremediation method against salt contaminations, special plants and trees which are salt tolerant must be selected. An important point about Phytoremediation process in salt impacted sites is high plant biomass production (Greenberg, 2007). Unfortunately in saline environment the plant production will be decreased due to lack of nutrition uptake caused by saline soil (Greenberg, 2007).

Two main requirements must be satisfied by the plant to be used as a Phytoremediation tool regarding deicing salt activities:

- High tolerance in saline soil with high absorption ratio
- High tolerance in cold winters

Except those, high productivity and easy development is very ideal regarding the Phytoremediation objectives.

4. TALL WHEATGRASS

Tall Wheatgrass is a plant which is very tolerant to salinity (USDA, 2007). Figure 1 shows the plant.



Figure 1. Tall Wheatgrass (agropyron trichophorum)
Source: Jose Hernandez @ USDA-NRCS PLANTS Database

4.1 Description

Tall Wheatgrass is a tall, coarse and late-maturing plant. It is native to saline meadows of southeastern Europe and Asia Minor. It is coarse, not drought tolerant and slow to establish. However, it has ability to thrive in sub irrigated saline soil where foxtail barley is usually the dominant grass (Smoliak et al., 2007.) The plant can not grow in dry, alkaline sites. The minimum amount of precipitation which is needed by Tall Wheatgrass is 16 inches annually (Smoliak et al.,

2007). It can survive cold winter with good spring recovery. One type of this plant can be grown in -43 °F (- 6.11°C) and can tolerate high saline environment (USDA, 2007).

4.2 Salinity Tolerance

Corps like beans will lose 20% of their yield with small amount of salinity (EC of 1.0 (dS/m)). In the other hand Tall Wheatgrass can tolerate salinity of 7.5 dS/m with yield loss of just 4.2% (Alberta Environment, P.78, 2001). In a research Tall wheatgrass species has been tested in saline environment (Suyama et al., 2006). Two species of Tall Wheatgrass were tested in this research, ***Thinopyrum ponticum (TWG1)*** and ***Agropyron elongatum (TWG2)***. They tested this plant in high salinity condition. They irrigated that with saline-sodic drainage water in a commercial farm in five points in Fresno County. Tall Wheatgrass growing in high salinity (EC_e= 17.6-19.1 dS/m) produced about 5.9 to 8.3 ton/ha of Dry Matter (DM). The local Tall Wheatgrass of Texas in normal conditions produced about 11.7-13.5 t/ha year with weekly freshwater irrigation and 100Kg/ha/year applied nitrogen (Suyama et al., 2006). The average product in this research is about 7 t/ha year (5.9-8.3). According to very harsh condition this amount of product is very high in comparison to the normal situations. In table 1, mineral contents of Tall Wheatgrass forages are indicated. Sodium concentration in TWG1 is about 8.9 g/Kg DM with salinity of 19.1 dS/m. This table shows that tissues of Tall Wheatgrass are reflection of salinity of irrigation water and the field. High salinity tolerance of Tall Wheatgrass is combined by its ability to absorb salinity from the field and irrigation water.

Table 1: Mineral contents of Tall Wheatgrass (TWG) forages

	EC_e (ds/m)	Na (g/Kg DM)	Se (g/Kg DM)	No3-N (g/Kg DM)	S (g/Kg DM)	Ca (g/Kg DM)	Mg (g/Kg DM)	K (g/Kg DM)
TWG1	19.1±0.9	8.9±1.6	6.12±0.3	131.6±23.8	3.6±0.3	2.2±0.3	1.3±0.1	13.8±0.5
TWG2	17.6±0.4	6.4±1.1	7.38±0.7	59.8±23.3	3.5±0.5	2.3±0.3	1.1±0.2	16.2±1.2
Salinity(EC_e is shown to determine salinity of the fields)								

(Source: Suyama et al., 2006)

In other research by C.M Grieve et al. (2004) an evaluation of some forage crops has been conducted regarding different ion concentration in saline-sodic environment growth. In the irrigation water Na+ was the dominant ion and then the following ions, SO4, Cl-, Mg2+ and Ca2+ respectively (Grieve et al., 2004). The forages included Tall wheatgrass (Agropyron elongatum) with common name of Jose has been tested. In two salinity stage the plant has been tested and ion concentration in the shoot was measured (See figure 2). Ion concentration in the plant shoot for different salinity stage of 15 and 20 dS/m is clear in the figure 2. Ions of Na+ and Cl- have high concentration in the shoot. Approximate Sodium concentration in the shoot is 600 mmol/Kg (figure 2).

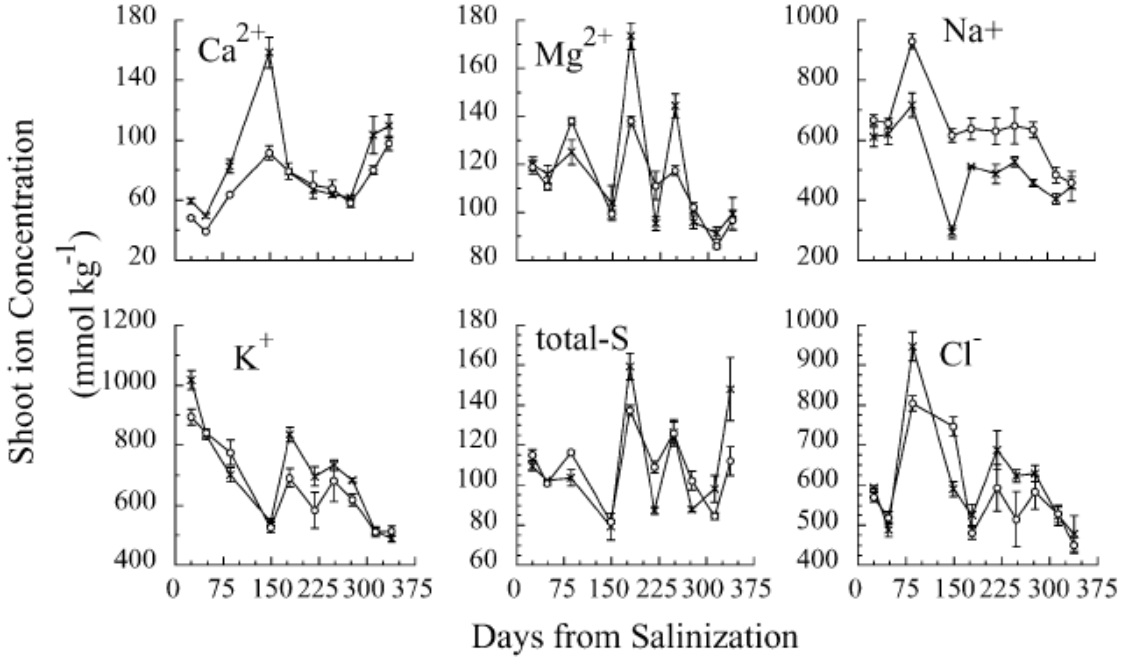


Figure 2: Shoot Ca2+, Mg2+, Na+, K+, total-S, and Cl_ determined in sequential harvests of tall wheatgrass cv. 'Jose' grown at two salinity levels [EC = 15 or 25 dSm_1]. Salinity applied, 25 Aug 2000; first harvest 20 Sep 2000. At each harvest, values are the means of three replications; error bars = S.E. (Source: Grieve et al., 2004)

4.3 Capacity of contamination removal

In order to measure the capacity of salt contamination removal of Tall Wheatgrass we have to determine the productivity and salt concentration in the tissues of Tall Wheatgrass. Tall Wheatgrass species in normal condition produce about 11.7 – 13.5 ton/ha year (Suyama et al., 2006). In very saline environment this production will be decreased to 5.9-8.3 ton/ha year (Suyama et al., 2006). Concentration of sodium and chloride in the shoot according to study of C.M. Grieve et al. (2004) varies during the year (See figure 2). The mean value of concentration for the sodium is about 600 mmol/Kg and nearly the same for the chloride. Therefore the average amount of NaCl in the Tall Wheatgrass shoot is about 600 mmol/Kg. It means 36 gr/kg DW, So in the very saline environment each kg of plant has about 36 gr/kg sodium chloride.

Tall Wheatgrass production in saline environment: 7 ton/ha
&
NaCl concentration in the shoot: 36 gram/kg
↓
Total NaCl removal: (36/1000)*7000=252 kg/ha

The average sodium chloride concentration in the Tall Wheatgrass shoot in one hectare is 252 kg/year. Therefore each hectare of Tall Wheatgrass can remove 252 kg sodium chloride per year. This amount of concentration is on the above ground of the plant. It is estimated that the same amount of concentration is in the below ground part of the plant in the roots. According to this, one hectare of Tall Wheatgrass can immobilize 252 kg of sodium chloride in the ground per year.

If along the highways in each side in width of 5 meters Tall Wheatgrass been grown then we have one hectare of the plants in the road sides and we can control about 500 kg sodium chloride per kilometer per year. 252 kg will be removed by harvesting the shoot and 252 kg will be

immobilized in the ground. This amount can be achieved by just 5 meters buffer zone in each side of the road. Salt concentration is more near the road and it can be said that the first 20 meters from the road has the most contaminant concentration. After this distance salt will have the opportunity to go under the soil surface. If the buffer zone can be wider to at least 10 meters or more, the efficiency of salt removal would be doubled. By 10 meters buffer zone in each side of the road, the approximate amount of 1 ton salt would be controlled per kilometer per year. In southern and central Sweden, the highways received 1.43-2 tons NaCl/Km per year (Löfgren, 2001). Therefore it can be seen that for this amount of salt activities Tall Wheatgrass Phytoextraction process is efficient.

4.4 Cost estimation

One of the advantages of Phytoremediation process is its low expenses in comparison with other methods like excavation. Since phytoremediation is a technology which uses plants, then by nature it is inexpensive application when compared to technologies that involve the use of large scale, energy consuming equipment.

According to report of Phytotech, Inc. cleanup costs, including treatment and disposal, can range from $20 - $80 per cubic yard of contaminated soil. This cost includes incineration of plants and ash disposal at a cost of $500 per cubic yard of material (Fiegl et al., 2006). If the plant can be used as forage (like Tall Wheatgrass) for livestock this cost will be omitted. The phytoremediation cost for one acre of sandy loam soil for 50 cm depth is estimated to range from $60,000 to $100,000 but the approximate cost of excavation and disposal of the contaminated soil at the landfill is about $400,000. Rate of uptake from the soil and the number of crops which are needed for soil cleanup affects the cost of phytoremediation. The entire process of remediation should be considered for cost determination from growing, maintaining, harvesting plants to disposing (Fiegl et al., 2006).

5. CONCLUSION

Phytoremediation technology is a clean and new system that could be applied for various cases in different conditions. This technology like others has some advantages and disadvantages:

- Early estimates of phytoremediation costs indicate that this technology has generally lower costs than other remediation technologies. Although actual cost of routine application of phytoremediation process is still unclear (Pivetz, 2001).
- Phytoremediation doesn’t have destructive impact on soil fertility and structure in comparison with chemical remediation such as soil washing and acid extraction. Beside vegetation help the structure of the soil without considering remediation impacts (Pivetz, 2001).
- The most important limitation of phytoremediation process is that this technology works up to the depth of plant roots. Remediation ability is limited to shallow depths. Trees have more remediation ability potential than plants and shrubs because of the root length. Effectiveness of phytoremediation procedure depends on the contamination depth and the zone of spreading.
- Since this is natural process technology and depends on growth rate of plants, the long time period is needed for cleanup and depending on the conditions may take 10 to 20 years. Heavy metal contaminated site would require 13 to 14 years to be remediated. Choosing fast growing plants and harvesting several times during the years instead of hyper accumulators will decrease this limitation (Pivetz, 2001).

6. SUGGESTIONS

Phytoremediation efficiency is very dependent on the hydro geometrical conditions of the site. For any site, special field test is recommended to examine ability of plants for that special condition. Too many parameters will influence the result of this method. For instance Ph of the soil, precipitation amount, temperature, salinity degree and mineral content of the soil, all affect the result.

This method can be applied to sites where the rate of infiltration is low. Plants need time to uptake pollutants and if the infiltration rate is too high then usage of this method is nearly useless. In this case pollutants will not stay in the roots zone to be absorbed by the plants root. Phytoremediation is a process which takes maybe some years to completely cleanup the environment. This process is not a solution for urgent cases. Phytoremediation can be applied as a long term protection barrier in sides of the highways or around water supplies. Other complementary methods should be used for achievement of better results. It is not absolute solution for salt contamination and it can be recommended as a complementary solution for environmental protection.

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WATER POLLUTION SOURCES IN WESTERN PART OF ROMANIA. CAUSES AND CONSEQUENCES

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Abstract: The lack of interest for a clean environment (especially clean waters) from the period before 1990 (and unfortunately even after 1990) had as consequence a low quality for surface and underground water bodies from western part of Romania. Large cities as Timi oara, Arad, Re i a are still representing main polluters even it were adopted friendly environment measures. Other important sources of pollution were represented by large pig farms which were in majority closed during the last years. Unfortunately we can still observe the adverse effects of these farms at surface and in underground. Generally, the agriculture was one of the main polluters in Romania's western part. The legislation adopted in the last years according to the European principles created the necessary framework for an improved situation regarding the water sources and bodies. The paper will present maps and records which are showing a progress, from low quality situations to an improved present. This paper will focus on the water pollution sources in western part of Romania, presenting the causes and the consequences of the pollution phenomenon, the main subject being Bega hydrographical basin. A chapter of this paper will discuss the legislative framework regarding water protection also referring to Romanian researches in these domains and from the mentioned area.

Keywords: Banat, Bega, causes, consequences, pollution

1. INTRODUCTION

The hydrographical space Banat is spreading from south of river Mures to the confluence of Danube and Cerna rivers, having a total surface of 18393,15 km^2, which represents 7,7% of Romania's territory.

The rivers which collect their waters from this territory have the specific characteristics of Romania's south-western part, but in the same time they can be characterized as individual fluvial systems with specific characteristics of each hydrographical basin. The human influence has a well defined role in water flowing, part of the hydrotechnical arrangements having an age of over 250 years.



Figure 1. Banat hydrographical space

Banat hydrographical space is composed by 6 hydrographical basins and includes also the hydrographical basins of Danube's tributaries rivers from Cerna and Nera basins. The hydrographical network has a length of 6245 km excepting the length of Danube section which borders Banat hydrographical space and has 145 km. The density of hydrographical network is 0,34 km/km^2, value very close to Romania's hydrographical network density. In hydrographical space Banat, the average flow calculated on 30 years has values between 1 l/s/km^2 and 40 l/s/km^2.

Located in the western part of Romania the hydrographic basin of the river Bega is part of the hydrographic space Banat, an area that presents hydro technical arrangements over 250 years old.

Tabel 1 Demographics and administrative characteristics of the hydrographic basin of the river Bega

Demographics and administrative characteristics of the territory present in hydrographic basin of the river Bega.					
Crt. no.	County	Surface (km^2)	% from the total surface on the h.b.	Population (habitants)	% from the total popTulation on the h.b.
1.	Hunedoara	9.99	0.222	0	0
2.	Caraş-Severin	0.06	0.001	0	0
3.	Timiş	4030.01	89.707	483587	97.68
4.	Arad	452.38	10.070	11465	2.32
	Total	4492.44	100	495052	100

The river Bega springs from the Poiana Rusca Mountains, has a length of 170 km on the territory of our country and empties into the Tisa River beyond the state border. His most significant affluent is the river Bega-Veche who is 107 km long and represents an old route of the Bega River.



Figure 2. Average annual flow in Bega river basin (according to Water Branch Banat)

2. WATER POLLUTION SOURCES IDENTIFIED IN THE HYDROGRAPHIC BASIN OF THE RIVER BEGA

2.1 Significant punctiform water pollution sources

According to the Water Frame Work 2000/60/EC the significant pressures are the one that blockades the reaching of the environment objectives for the water bodies.

The major water pollution sources presented in the hydrographic basin of the river Bega are:

- Domestically pollution sources:

- Jimapaterm Jimbolia has a capacity of the mechanical-biological cleaning plant of 42 l/s. According to the N.T.P.A. 001/2002 overtaking at the following indicators were recorded: suspensions (54,453 mg/l), CBO_5 (29,342 mg/l), CCOMn (26,439 mg/l), ammonium (30,634 mg/l), retrievable (25,865 mg/l), CCOCr (99,637 mg/l), fixed residue (1010,345 mg/l), phosphates (8,835 mg/l), Nt (34,796 mg/l), Pt (10,265 mg/l).
- R.A. Apă-Canal Aquatim Timişoara (Timişoara city water sewage station) collects the tail water from 350000 habitants and from economical agents. The overtaking according to the N.T.P.A. 001/2002 are recorded at the following indicators: suspensions (95,41 mg/l÷110,944 mg/l), CBO_5 (48,5 mg/l÷50,93 mg/l), ammonium(33,88 mg/l), Cu (0,118 mg/l), retrievable (27,07 mg/l÷28,678 mg/l), CCOCr (147,677 mg/l-167,77 mg/l), mineral oil (1,548 mg/l), Nt (36,073 mg/l÷36,953 mg/l) şi Pt (9,455 mg/l÷10,403 mg/l).
- The City Hall of Tomeşti - In the cleaning plant of the town Făget are evacuated tail waters from 1500 habitants and from the Glass Factory Stitom Tomeşti. Unfortunately the cleaning plant is in a improper state and in the year 2002 the promotion of a Sapard project was initiated.
- The City Hall of Făget - The mechanical-biological cleaning plant of the city, works only with a misfit mechanical step. Efforts are making to rehabilitate the canalization and the cleaning plant, with the hope to attain new founds for these purposes.

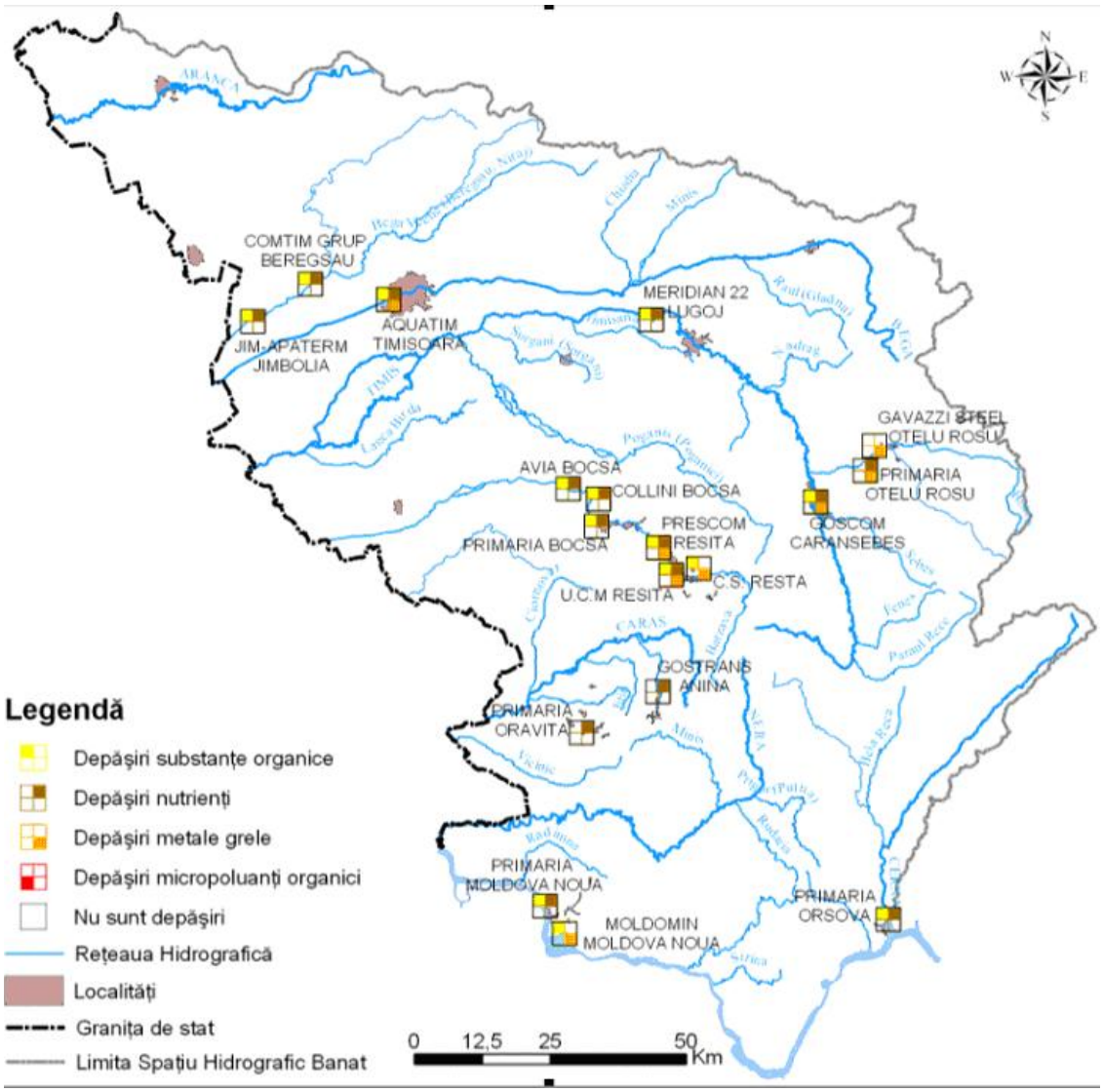


Figure 3. Significant sources of water pollution

- Industrial pollution sources:
    - S.C. Sugar Factory S.A. Timişoara - Her function capacity has diminished until the closure of the unit in the year 2000. The factory's tail waters are not discharged into the river Bega, because the absorbed volumes are compensated through evapotranspiration.
    - S.C. Solventul S.A Timişoara - The Chemical enterprise has totally interrupted her activity, and on the plant's platform are currently a few small units with storing spaces or other activities with no serous impact for the environment. The collected overflow discharged from the platform is only pluvial, qualitative and quantitative monitored by Direcția Apelor Banat.
    - S.C. Solventul Margina - In the period with maximum production the phenol waters, resulted from the pyroligneous acid fabrication, were discharged through dispersion on the infiltration fields especially installed. In present the arrangements against flooding on the Bega River and the bed of river recalibration follows a path that protects the enterprise area, ruling out in high water the possibilities that the heavy water flow washes out the infested area with phenol waters.
    - Glass Factory Stitom Tomeşti - The factory profile involves the glass manufacturing, polluting the Bega River until the year 1995 when the factory connected to the canalization of the town Tomeşti.
- Agrarian pollution sources
    - S.C. Comtim Group S.R.L. Beregsău - The society unrolls an activity in the stock raising field, and discharges into the Bega River after only a primarily cleaning due to the fact that the cleaning plant is out of use from 20 years. The medium discharge flow is 2,43 l/s. According to the N.T.P.A. 001/2002 overtaking at the following indicators were recorded: suspensions (557,93 mg/l), CBO_5 (743,4 mg/l), CCOMn (102,451 mg/l), NH_4^+ (318,922 m/l), phenols (0,002 mg/l), retrievebaels (99,749 mg/l), CCOCr (1670,776 mg/l), Nt (348,574 mg/l), Pt (126,011 mg/l).

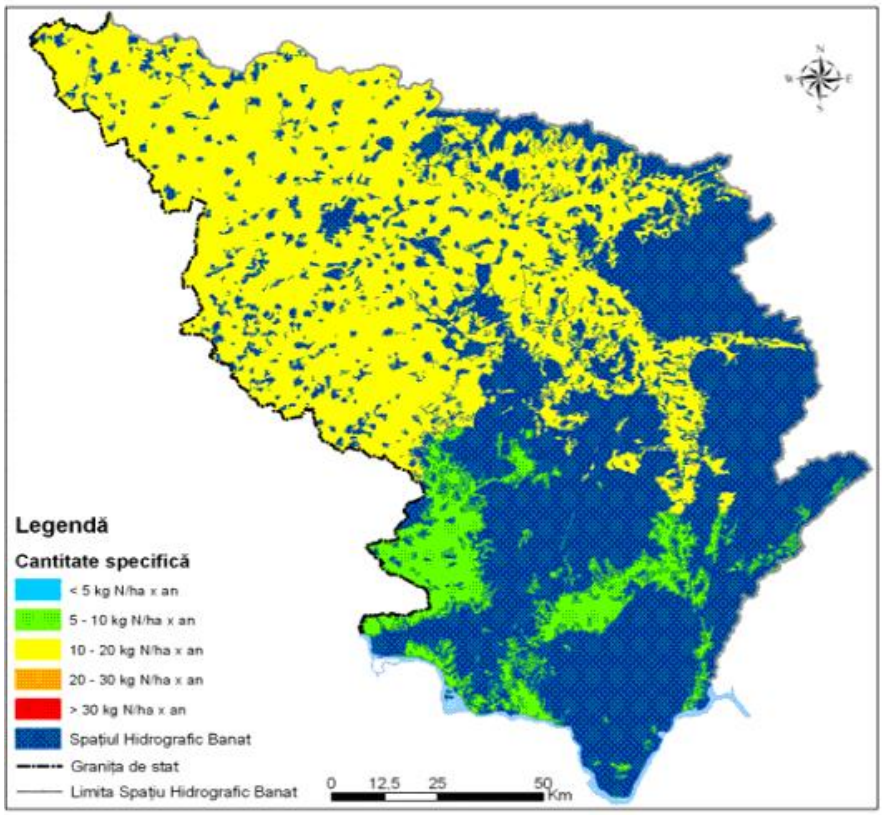


Figure 4. N specific quantity in Banat hydrographical space

2.2 Significant waters scattered pollution sources including manner of land utilization

The arable land occupies one third of the basin's surface influencing the use of fertilizers between 10-20 kg. N/ha·year and 2-5 kg P/ha·year, the use of pesticides that is lower than 0,5 kg /ha·year and the stock raising. The phenomenon is amplified by the fact that only 37,5 % from the population are connected to the centralized canalization and because of the intensive agriculture applied in the north-western part of hydrographic basin of the river Bega.

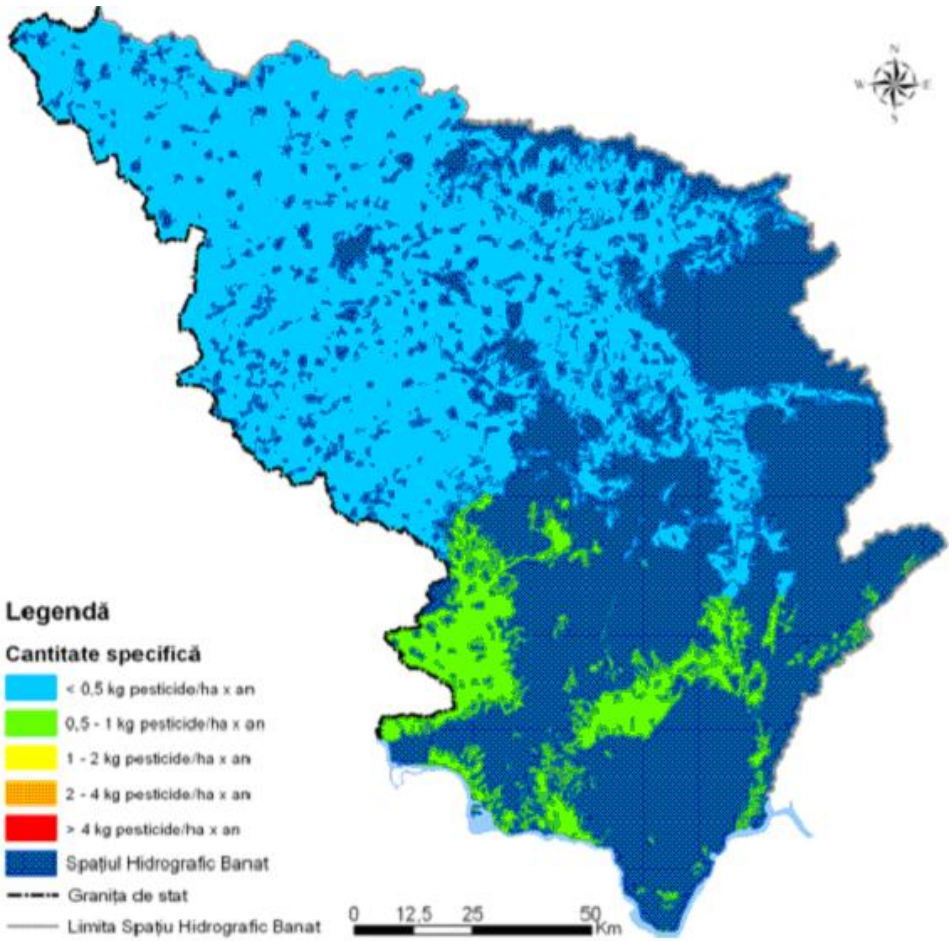


Figure 5. Pesticides in Banat hydrographical space

2.3 Significant hydro morphological pressures

The hydro morphological pressures must be identified to establish the surface water bodies highly modified. There are a range of activities developed or related on surface waters but only one of them exerts pressures over the quality of the surface waters. The main hydro morphological pressures in the hydrographic basin of the river Bega are due to:

- The barrier lake of Surduc – located in the superior basin of the river Bega was accomplished in the year 1976 through a dam across the valley of the Gladna river. The most significant roles of the channel storage are : the water supplementation of the downstream areas, mainly the Timişoara municipality, to assure the water supply of irrigation arrangements and aquaculture upstream of Timisoara, to die out the high flood waves on the Gladna river and other affluent, to assure an energy supply by the MHC of Surduc, to maintain an salubrious annual flow of minimum 2,1 m^3/s in winter and 2,6 m^3/s in summer and the existence of a quality vacation land;

- the Timis – Bega diversion involves a double connection between the river Timiş and Bega with an alimentation channel Coştei – Chizătău that supplements the small and medium flows of the Bega river and a discharge channel Topolvăț – Hitiaş for high floods situations;
- Extractions and returns of water: there are two significant extractions from AquaTim with an overflow of 2,0 m^3/s and the National Administration of the Land Reclamation with the Şag - Topolovăț System, and only one important return of water from AquaTim with a discharge flow of 2,2 m^3/s.
- The navigable channel Bega has in the present a recreational navigation on the Timişoara – Sânmihaiul Român section.

3. THE MONITORING OF SURFACE WATER QUALITY IN THE HYDROGRAPHIC BASIN OF THE RIVER BEGA

3.1 Global water quality registered in the order I surveillance section

On the river Bega the water quality was followed in four surveillance sections: Luncani, Balinț, upstream Timişoara and Otelec. The Luncani section is located in the superior area of the Bega river were there are no punctual pollution sources. The Balinț section is located downstream of a few small pollution sources but with a constant danger of phenol pollution in the period with high precipitation. The water quality is in the first category except the CBO5 (5,3 mg/l), phenols (0,005 mg/l) and iron (0,36 mg/l), with values enclosed in the second category of water quality, overstepping that are due to the local conditions. One of the most important sections is the section upstream Timişoara that establishes the quality of the water for drinking entrapped at the Waterworks 2 and 4 of Timişoara. The water quality maintains in the first categories excepting the phenols (0,06 mg/l). The last section Otelec is closed to the Serbian border with a water quality that is found in the second category for the special toxics: ammonium (3,55mg/l), phenols (0,008mg/l), phosphorus (0,48 mg/l).

The river Bega-Veche is the most polluted watercourse from Banat and has two surveillance sections. The first section is Pişchia located in the superior area of the watercourse with a global quality of water included in the second category with pollution sources derivate from the local area. In the second section Cenei that is found in the inferior part of river the water quality is included in the third category for the indicators of the special toxics group: ammonium (2,76 mg/l), phenols (0,259 mg/l), phosphorus(0,30 mg/l) due to the Butchery Beregsău that belongs to the S.C. Comtim Group S.R.L, and who discharges without a cleaning in the river Bega –Veche.

3.2 The critical areas regarding the improper water quality

The navigable channel of Bega (downhill Timişoara-border line) on a 34 km length suffers a severe water degradation comparative with the upstream section (of I category) being mainly affected by:

- discharges of waste water from Timişoara municipality with a overflow of 1798,35 l/s , water insufficient cleaned due to a low efficiency rate of the cleaning plant;
- an accentuated mineralization process of the organically substances from the slime stored on the layer bed of the Bega river who' s mineralization implies a high oxygen consumption; the slow water flow in the navigable channel doesn't promote the auto cleaning process and contributes to important bed materials apparition. The origin of those slimes is on the affluent that collects the flow from the hilly area depending on the hydrological regime. In the year 2002 DHV has conducted a feasibility study and noticed that the bed materials from Bega Channel are polluted with substances that are dangerous for the environment.

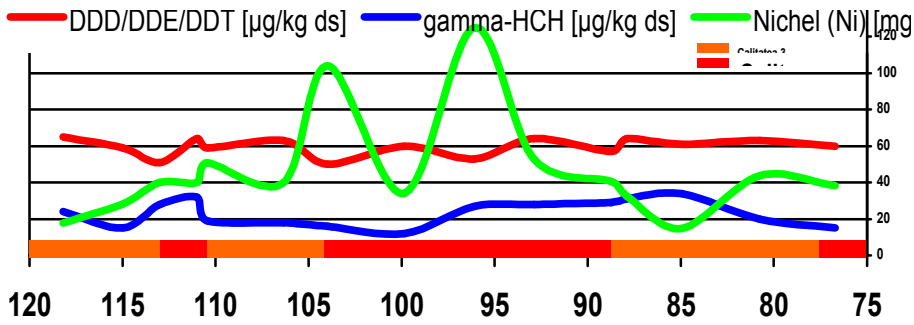


Figure 6. Variation of the bed material quality and pollutants in the Bega Channel (according to W. B. B.)

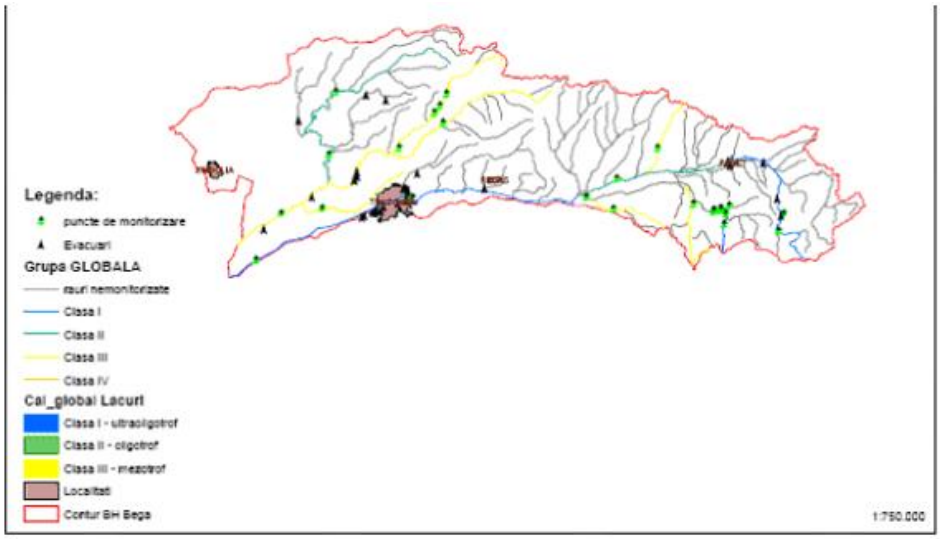


Figure 7. Global characterization of surface waters quality in the hydrographic basin of the Bega River (according to W.B.B.)

4. LEGISLATIVE FRAMEWORK

According to Romanian Water Law 107/1996 completed and modified with Law 310/2004 and in conformity with Order 913/2001, National Administration of Romanian Waters realizes the Director Schemes and the Schemes of Hydrographical Basins Arrangements.

In this sense, at the level of National Administration of Romanian Waters it was created the Direction of Management Plans and Hydrographical Basins Arrangement Plans, and in the frame of Banat Water Direction it was created the Bureau of Director Schemes, Arrangements and Basins Management. Also, it was designated a person in charge for these activities at the level of each water management system.

At the level of each hydrographical basin, according to Law 107/1996 – Water Law, and Government Decision 1212/29.11.2000, were realized Basin Committees. In Banat Hydrographical Basins works a Hydrographical Basin Committee which gather all the main decision factors from water domain and which includes in the team members of Environment Ministry, of National Administration of Romanian Waters, of local authorities, water users and public. This Committee has as purpose the appliance of sustainable management principles regarding the water resources.

The management plan of Danube hydrographical district represents the main instrument for an integrated and sustainable management of water resources. The States members of International Commission for the Protection of Danube River establish that this plan must include two parts: Part A – the General Plan which includes problems of basin importance with trans-border effects and Part B meaning the National Management Plan which in Romania's case will comprise 11 plans, one for each Water Direction and the Danube Management Plan, Danube Delta and sea waters.

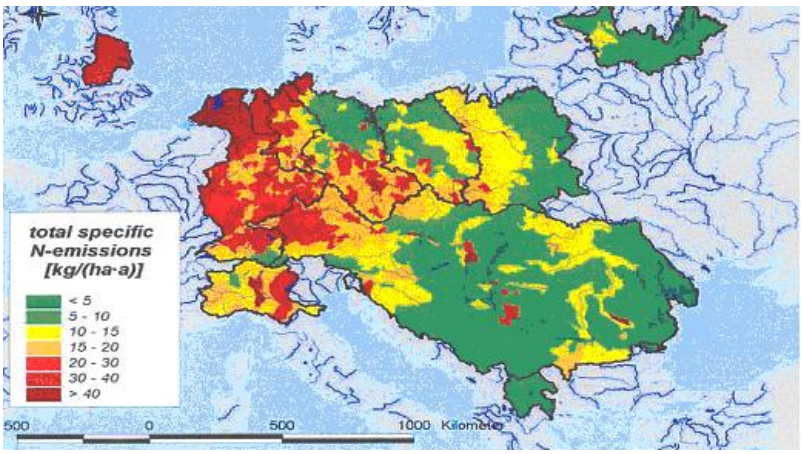


Figure 8. Total specific N emissions for trans-border European basins

5. CONCLUSIONS

In the years 1960-1989 the water quality has suffered severe damages in regard to the reference estate from the ’50, because of the economic development. After the year 1989 the water quality has improved due to the socio-economical activities and also because a new mechanism in the water domain including a principle called “the polluter pays”. Still there are numerous water bodies in the hydrographic basin of the river Bega that haven’t achieved a “god state” according to the European Community Water Frame Work because of the punctual, scattered and hydro morphological pressures. In the hydrographic basin of the river Bega was inventoried 12 punctual pollution sources from which 2 are represented by the towns with over 10000 habitants with cleaning plants that function improperly. Considering the relatively high number of water bodies possibly to be at risk it is necessary a special monitoring that could supervise the hole water body to estimate more accurately all the pressures including the scattered ones. Another significant aspect regards the existence of hydro technical works that are 270 years old, massive canalizations works that have transformed the plain affluent of the Bega River in components of a drainage system. Even so the territory of the Hydrographic Basin of the river Bega was affected by severe floods. The last case was registered in the spring of the year 2005 between 15 and 19 April when due to highly precipitation and to the water resulted from the snow melting the river cotes have grown causing the flooding of 70% of the county’s surface.

Considering the experience achieved during the last flooding it is safe to say that the hydro technical works have an adequate behavior in conditions of high water in spring and fall, but in case of historical floods due to the changing of layer morphology the existent works proved to be under measured for the water volumes leaked on the inferior course of the Bega river. Are required new analyzes and measures to assure an optimal water flow in high water.

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NEW APPROACH FOR DESIGN OF GROUNDWATER QUALITY MONITORING NETWORK USING ENTROPY THEORY

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Abstract: Assessment of groundwater quality monitoring networks requires methods to determine the potential efficiency and cost-effectiveness of the current monitoring programs. To this end, the concept of entropy has been considered as a promising method in previous studies since it quantitatively measures the information produced by a network. In this study, the measure of Transinformation in the discrete entropy theory and the Transinformation-Distance (T-D) curves are used to quantify the efficiency of a monitoring network. This paper introduces a new approach for decreasing the dispersion in results by using cluster analysis which utilizes fuzzy equivalence relations. The sampling (temporal) frequency determination method recommends the future frequency of sampling for each location based on some criteria such as direction, magnitude, correlation with neighboring stations, and uncertainty of the concentration trend derived from representative historical concentration data. For the final step, the optimized groundwater quality monitoring network in space and time dimensions are introduced. The proposed methodology is applied to groundwater resources in the Tehran-Karadj Aquifer, Tehran, Iran.

Key words: Clustering by Fuzzy Equivalence Relations; Discrete Entropy Method; Groundwater Quality Monitoring; Transinformation; Temporal Frequency.

1. INTRODUCTION

To determine physical, chemical, and biological characteristics of groundwater, a selection of sampling points and (temporal) sampling frequency should be incorporated to design a groundwater quality monitoring network as stated by Loaiciga et al. (1992). Despite an enormous amount of efforts and investments made on monitoring of groundwater quality, a wide range of shortcomings has been identified in current monitoring networks, and as a result, the outcome of the current data collection systems is very insufficient for providing needed information on groundwater quality (Ozkul et al. 2000). Another concern is to have a proper design for the groundwater quality networks, since there are a lot of difficulties in selecting the parameters, such as temporal and spatial sampling frequencies, monitoring variables, duration of sampling, and finally, the sampling objective (Harmancioglu et al. 1999). Considering the aforementioned issues, the design procedures for groundwater quality networks need more critical investigations. To do so, in the past few years, most of the developed countries have begun to redesign their monitoring programs to modify or revise their existing networks. Based on recent studies, the analysis of water quality monitoring data is considered as a statistical procedure; therefore, the issues such as assessment and design of water quality monitoring programs are addressed by using statistical methods. Statistical analyses should be used to select the spatial and temporal design features of a network. These statistical analyses should be based on time series analysis, decision theory, regression theory, and optimization methods. Also, spatial-analytical techniques, such as kriging and cokriging, should be utilized to investigate the spatial dependencies between the data and water quality sampling networks (Schilperoort and Groot 1983; Jager et al. 1990).

Unlike general methodologies, the recent design methodologies do not fulfill the expectation for the networks to produce data for which the application of statistical data analysis techniques is permitted. As an outcome, it is difficult to assess the produced information in many existing networks due to the network designs being inefficient from a lack of exact definition for the information contained in the data and the lack of explanation on how the data was measured. Another deficiency is due to imprecisely defining the data value or utility, which makes the network have weakness in the contained information and inefficiency in terms of the cost of getting the data. The last item that makes the network inefficient is the method's restriction on transferring the information in space and time (Ozkul et al. 2000).

The entropy concept of information theory is one of the best methods to assess the networks. This theory has been used for hydrometric and water quality networks. The entropy concept is able to provide exact definition of information in tangible terms, which is one of the best characteristics of this concept. Another feature of the entropy concept is to express the information in specific units such as Nats, Napiers, decibels or bits; these features together let the entropy concept have a complete quantitative measure.

Entropy measures were shown to be more advantageous, as addressed in the study by Yang and Burn (1994), which showed that on the basis of information transmission characteristics of each site, they reflect a directional association among sampling sites. The entropy method was used by Ozkul et al. (2000) to assess the monitoring networks for existing water quality programs. In addition, Mogheir et al. (2004) used discrete and analytical entropy-based approaches to characterize the spatial variability of groundwater quality, while Sarlak and Sorman (2006) and Karamouz et al. (2008) used the analytical entropy approach to evaluate existing monitoring stations in rivers.

Most of the referred studies were using analytical approaches which required incorporating probability distributions of the random variables; however, the alternative to the analytical approach is to adopt the discrete approach as addressed by Mogheir et al. (2004).

To evaluate the efficiency of a monitoring network in this study, the measure of Transinformation in the discrete entropy theory and Transinformation-Distance (T-D) curves were used. To cluster the study area into homogenous zones, a cluster analysis was used that applied fuzzy equivalence relations. This was done by considering major characteristics of each station, and subsequently generating different T-D curves for different zones.

As stated in the study by Faisal et al. (2004), a critical area of concern in monitoring network design for groundwater quality is how to determine the sampling frequency. Historical data and associated confidence limits were used by Sanders et al. (1993) as indicators to evaluate the sampling frequency.

Furthermore, Ling et al. (2003) concluded that most of the mentioned approaches are statistically sound and improve the existing monitoring plans. However, one should consider that these techniques are designed for large scale sites and are limited by several problems when applied to the sites with smaller scales. As an example, the number of sampling points in small scale sites needed to derive a dependable spatial correlation structure is insufficient.

An innovative methodology is used in this study to improve existing monitoring plans for groundwater at small scale sites. This methodology recommends the future sampling frequency for each sampling location based on factors such as magnitude, direction, correlation with neighboring stations and uncertainty of the concentration trend derived from representative historical concentration data.

2. METHODOLOGY

Shannon's entropy describes the amount of uncertainty in any probability distribution; therefore, one can use the entropy concept to measure the uncertainty and indirectly obtain information in probabilistic terms. The entropy concept Shannon defined is informational entropy (Harmancioglu et al. 1999).

Based on Shannon's definition, information is attained when there is uncertainty about an event; therefore, little information is conveyed when alternatives with a high probability occur. The occurrence probability of a certain alternative is the measure of uncertainty or the degree of likelihood of a sign, symbol or number, which was referred to as entropy by Shannon. The obtained information is measured as the amount of uncertainty or entropy, which by removing the uncertainty, the result becomes available as an information.

The joint or conditional probability needed to calculate the information measures can be obtained from a contingency table. An example of a two-dimensional contingency table is shown in Table 1. In the contingency table, the random variable x has a range of values consisting of v categories (class intervals), while the random variable y is assumed to have u categories (class intervals). The cell density or the joint frequency for (i,j) is denoted by f_{ij}, i = 1, 2, . . . , v; j = 1, 2, . . . , u; while the first subscript, i, refers to the row and the second subscript, j, refers to the column. Also, $f_{i.}$ and $f_{.i}$ are the marginal frequencies for the row and column values of the variables, respectively.

Table 1. Two-dimensional contingency table (frequency)

x	y					Total
	1	2	3	…	u	
1	f_{11}	f_{12}	f_{13}	...	f_{1u}	$f_{1.}$
2	f_{21}	f_{22}	f_{23}	...	f_{2u}	$f_{2.}$
3	f_{31}	f_{32}	f_{33}	...	f_{3u}	$f_{3.}$
.	.	.	.	.	.	.
.	.	.	.	.	.	.
.	.	.	.	.	.	.
v	f_{v1}	f_{v2}	f_{v3}	...	f_{vu}	$f_{v.}$
Total	$f_{.1}$	$f_{.2}$	$f_{.3}$	...	$f_{.u}$	f_x or f_y

2.1. Entropy Theory Coefficients

The coefficients or information measures of the entropy theory are information contents, marginal entropy, conditional entropy, joint entropy and mutual entropy, also known as Transinformation.

For a random variable like x, the marginal entropy, H(x), can be defined as the potential information of the variable. For two random variables, such as x and y, the conditional entropy, $H(x|y)$, is a measure of the information content of x that is not contained in the random variable y. The joint entropy , $H(x,y)$, is the total information content contained in both x and y variables. Transinformation, $T(x,y)$, or information between variables x and y, is interpreted as the reduction in uncertainty in variable x due to the knowledge of the random variable y. These information measures for discrete variables can be expressed as:

$$H(x) = -\sum_{i=1}^{n} P(x_i) \ln P(x_i) \tag{1}$$

$$H(x|y) = -\sum_{i=1}^{n}\sum_{j=1}^{m} P(x_i, y_j) \ln P(x_i|y_j) \tag{2}$$

$$H(x,y) = -\sum_{i=1}^{n}\sum_{j=1}^{m} P(x_i, y_j) \ln P(x_i, y_j) \tag{3}$$

$$T(x,y) = -\sum_{i=1}^{n}\sum_{j=1}^{m} P(x_i, y_j) \ln[\frac{P(x_i, y_j)}{P(x_i)P(y_j)}] \tag{4}$$

where x and y are two discrete variables with values x_i, i = 1, 2, . . . , n; y_j, j= 1, 2, . . .,m, defined in the same probability space, where each variable has a discrete probability of occurrence $P(x_i)$. $P(x_i, y_j)$ is the joint probability of x_i, y_j and $P(x_i|y_j)$ is the probability of x_i conditional on y_j.

Mogheir et al. (2004) suggested that the exponential decay curve could be the best representation of the discrete T-Model, which could be presented as:

$$T(D) = G.e^{(-kD)} + Q \tag{5}$$

where the exponential decay curve starts with $T_0 = G + Q$ at distance (D= 0); and the curve decays to reach a Q value ($T_{\min}$) at infinite geometric distance with a constant rate k (1/m). The units of G and Q are expressed in the same way as the unit for T "Nats", natural units, when the logarithmic base is e (Husain, 1989). Equation (5) can be rewritten as:

$$T(D) = (T_0 - T_{\min}).e^{(-kD)} + T_{\min} \tag{6}$$

Finally the below equation provides the optimal distance between wells, L_T:

$$L_T = \frac{Ln(T_0 - T_{\min}) - Ln(\mathrm{e})}{k} \tag{7}$$

where e is the required accuracy that corresponds to financial issues. A lower e is needed if higher financial support can be afforded. In this paper, the value of e is considered as 0.01 Nats which is considerably logical.

Motulsky (2008) recommended to fit the exponential decay curve to the T-Model using the least square fitting procedure with the GRAPHPAD PRISM 5 statistical software. To quantify the goodness of fit between the exponential decay curve and discrete models, one can use the coefficient of determination, R^2.

2.2. Fuzzy Cluster Analysis

The process of dividing data elements into classes or clusters is called data clustering. In this process the items in the same class are selected to be as similar as possible, and items in different classes are selected to be as dissimilar as possible. In this study, stations were categorized into groups based on similarities in significant properties such as marginal entropy, mean, variance and spatial location of potential stations. To produce a fuzzy similarity matrix, the max-min method was used as a fuzzy equivalence relation, using the following equation:

$$r_{ij} = \frac{\sum_{k=1}^{m} \min(x_{ik}, x_{jk})}{\sum_{k=1}^{m} \max(x_{ik}, x_{jk})} \quad i, j = 1,2,...,n \tag{8}$$

where m is the number of considered properties, n is the number of stations, x_{ik} is the value of the k-th property in station i, and r_{ij} is the fuzzy membership degree of the similarity matrix. Fuzzy equivalence relations provide fuzzy membership degrees that are more appropriate for categorizing stations into similar groups compared to other equivalence relations.

To improve the accuracy of the T-D curves, fuzzy cluster analysis was used to categorize the study area into homogenous zones by considering major characteristics of each station. Different T-D curves were then calculated for different zones. To have higher coefficient of determination (R^2) a combination of the mentioned properties is applied.

2.3. Temporal Frequency

One of the important variables in the design of a groundwater monitoring network is sampling frequency, as addressed in the study by Zhou (1995). If sampling occurs rarely, there is a high probability of losing important information, thereby affecting the accuracy of monitoring. Also, the obtained information could be redundant if sampling occurs too often, which is very inefficient in

terms of the cost, since sampling requires a large portion of the operating and monitoring cost of groundwater quality monitoring.

The sampling frequency determination at each well was designed to provide an accurate estimate of the trend in concentration change. In addition, one of the key factors considered in sampling frequency design was the magnitude of concentration in each period, where higher concentrations require higher temporal frequency. Another important factor is the direction of change. Similar to the magnitude of concentration, higher concentration slope trends require higher temporal frequency. To calculate the slope of historical data, one can use iteratively reweighted least square (IRLS) robust regression, a weighted least squares regression with regression coefficients obtained from the following equation:

$$b = (X'WX)^{-1} X'WY \tag{9}$$

where b is the coefficient matrix, X and Y are the predictor and response matrices, respectively. Weight values assigned for each observation in Y are contained in W, which is the weight matrix. The influence of outliers is reduced by assigning weights that vary inversely with the size of the residual in robust regression. Until the estimation process stabilizes, the weights are revised as each iteration yields new residuals. This approach was selected instead of the ordinary least squares method to overcome the difficulties associated with data with outliers.

Dispersion and inhomogeneity of data around the mean is a dominant property, which requires consideration in design of temporal frequency. For instance, if variability of data around the mean is too large, reliability of the mean decreases, and higher temporal frequency is needed. Another important issue to consider is the relationship between wells as a whole in addition to considering the characteristics of each well separately. The following equation calculates the correlation of one specific well with other wells:

$$C.\mathrm{I}(\mathrm{X_i}) = 1.0 - \left[\frac{\sum_{j=1}^{n} \left(\frac{\mathrm{Correlation}(\mathrm{X_i}, \mathrm{X_j})}{D_i} \right)}{\sum_{j=1}^{n} \left(\frac{1.0}{D_i} \right)} \right] \qquad \mathrm{i} \neq \mathrm{j} \;\&\; \mathrm{i, j} = 1,2,...,\mathrm{n} \tag{10}$$

where C.I(X_i) is the correlation index in station i, D_i is the distance between stations i and j, n is number of wells, and $Correlation(X_i, X_j)$ is the correlation between stations i and j.

By considering dominant properties of concentration trends and performing fuzzy cluster analysis, stations were categorized into four groups. As mentioned in previous sections, these classes or clusters contain stations that are as similar as possible, while stations in different classes are as dissimilar as possible. It should be emphasized that the mentioned approaches are in agreement with approaches suggested by the US EPA (1999).

3. SITE APPLICATION

The proposed model was used for the redesign of the groundwater quality monitoring system in the Tehran-Karadj Aquifer,Tehran, Iran. The aquifer covers more than 1,800 km^2, where approximately 865 million cubic meters of water per year is provided for domestic consumption to over 10 million people, which is at least 30 percent of Tehran's domestic water demand. The current wastewater disposal and water allocation schemes have caused soil and groundwater pollution as well as high groundwater table variations in many parts of the study area. Comparing the average groundwater table in April 1999 and April 2006 shows about 6 meters decrease in water table level in this aquifer that is mainly due to excessive usage and also lack of a comprehensive demand management.

Present groundwater quality monitoring data is available from 64 stations with semiannual temporal frequency from May 1998 to November 2007. The groundwater quality data obtained from the existing monitoring system shows some high values for TDS, nitrate and coliform bacteria

violating the standard levels. In this study, the existing groundwater quality monitoring systems were assessed and redesigned considering the water quality variable of electrical conductivity (EC).

4. RESULT AND DISCUSSION

Since the mentioned aquifer supplies the water demands of several domestic and agricultural zones in the Tehran, EC is considered as an appropriate water quality variable. The available EC data was used for developing the contingency tables and the T-D curves after doing fuzzy cluster analysis. Figure 1 shows a T-D curve without using fuzzy clustering. This figure shows a high dispersion in data points. Figure 2 shows three separate T-D curves from three different clusters, which have much less data dispersion compared to Figure 1.

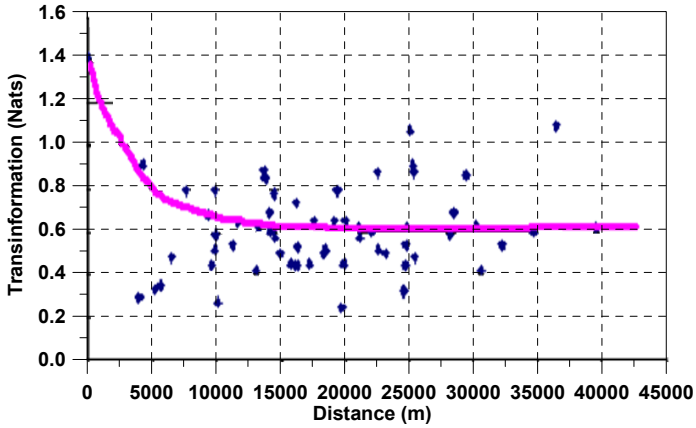
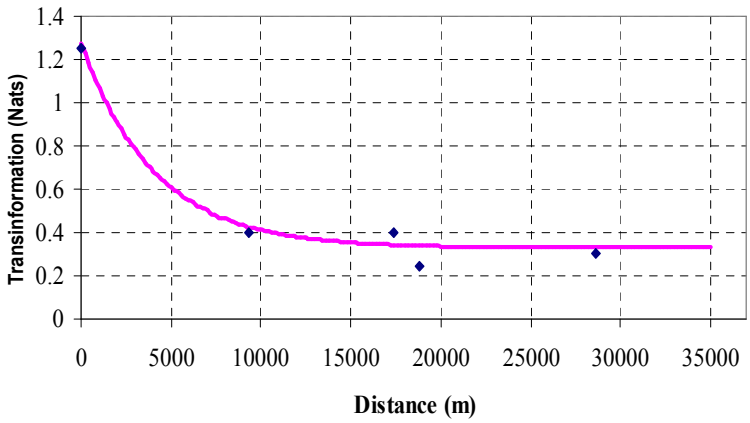
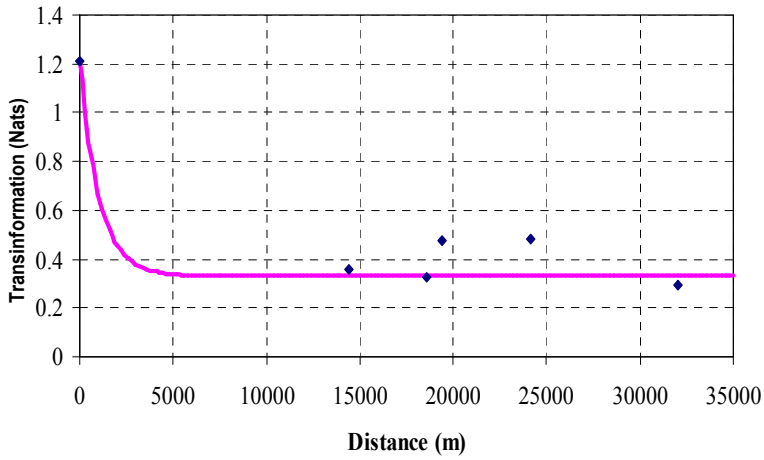


Figure 1. Variation of the Transinformation versus distance without fuzzy clustering

Figure 2. Variation of Transinformation versus distance in different zones with fuzzy clustering





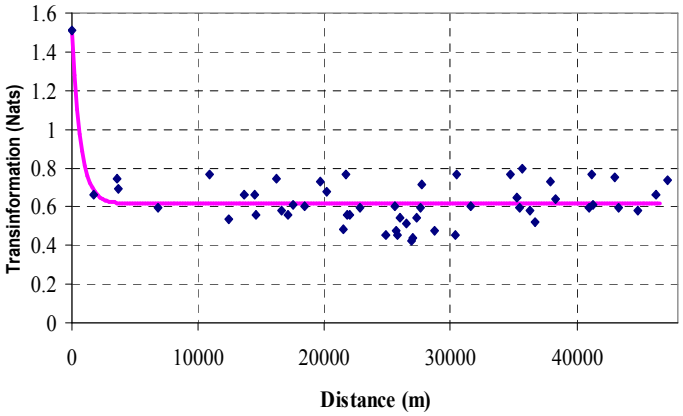


Table 2. T-D curve equations for each zone

State	Curve Fitted	R^2	Number of Stations	Optimal Distance (m)
Without clustering	$0.8211 * EXP(-0.0006239X) + 0.621$	0.25	64	4,187
Cluster 1	$(0.8811 * EXP(-0.0009639X)) + 0.3299$	0.95	5	4,648
Cluster 2	$(0.9416 * EXP(-0.0002442X)) + 0.3299$	0.98	5	6,525
Cluster 3	$(0.893 * EXP(-0.001512X)) + 0.617$	0.51	54	2,970

Table 2 presents the corresponding equation to each curve, coefficient of determination (R^2), number of stations in each cluster, and optimal distance that each well covers. Results show that the marginal entropy had the greatest effect on clustering. As shown in Table 2, R^2 increased significantly by performing cluster analysis. The final proposed monitoring network is shown in Figure 3. As shown in this figure, from the 64 current monitoring stations, 36 stations are selected and 28 stations are removed. Additionally, based on the available historical data, 19 stations were located that provide sufficient information.

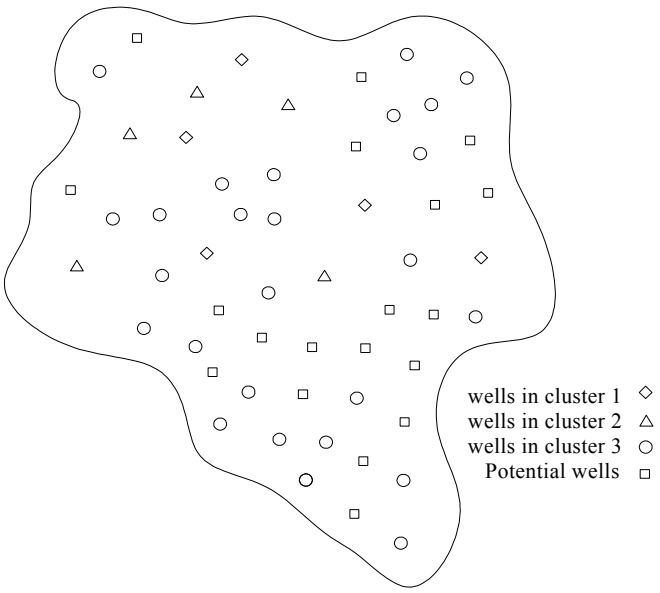


Figure 3. The location of the selected existing and potential monitoring wells

Considering the mean, variance, slope (using robust regression), and C.I in each station, similar stations were categorized into one cluster, and a temporal frequency was assigned to different wells. Having higher mean, variance, slope, or C.I in concentration trends result in a higher temporal frequency and vice versa.

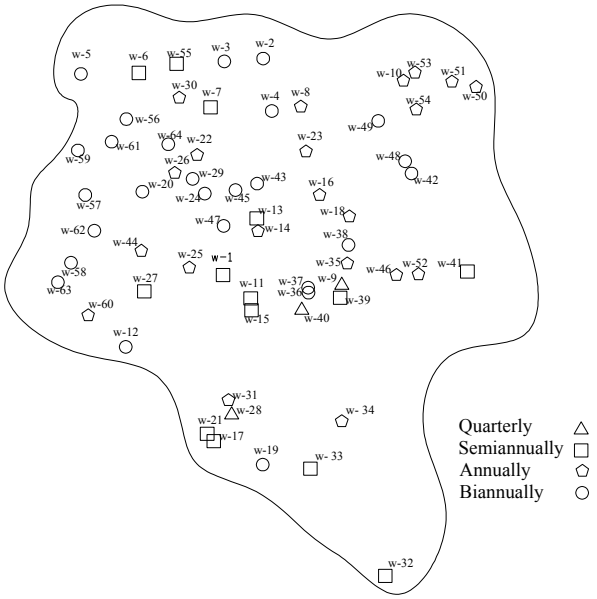


Figure 4. Temporal frequency of each well

5. CONCLUSION

A spatial-temporal methodology for improving existing groundwater quality monitoring networks were presented in this paper. Fuzzy clustering was used to divide the study area into three homogeneous zones. For the next step, T-D curves were generated to measure the mutual content of information in each cluster to optimize the monitoring cost by having a better selection in each zone. Optimal distance for each well was determined, and a well-sitting map was generated accordingly (Figure 3). This methodology was applied to a network site containing 64 monitoring wells, and the effectiveness and validity of the proposed methodology was verified.

ACKNOWLEDGMENT:

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HYDROCHEMICAL VARIABILITY OF SPRING WATER IN THE UPPER JORDAN RIFT VALLEY

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Abstract: The hydrochemical characteristics of twenty springs, distributed along the north – south cross section of the Upper Jordan Rift Valley were determined. The major ions of fifty water spring samples: Ca^{2+}, Mg^{2+}, Na^{+}, K^{+}, Cl^{-}, SO_4^{2-}, NO_3^{-} and HCO_3^{-} were analyzed in addition to heavy metals including: Fe, Zn, Li, Cr, Co, Ni, Cu, Cu, Mn, Al, Cd, Ba, Pb and Bi. An increase in the concentration trend of Na^{+}, SO_4^{2-}, NO_3^{-}, as well as of Fe, Co, Bi, Li was noticed towards the south caused by natural and manmade polluted sources. This put remarks on the surface – ground water interaction sources. The industrial wastewater flow discharge in Faria area increased the concentrations of Co, Bi, Li (from west to east) in the samples taken from the area threatening the quality of groundwater. The majority of the spring water samples fall into the normal earth alkaline water group with prevailing bicarbonate and sulfate or chloride. The southern springs are classified as normal earth alkaline water with prevailing bicarbonate, which are classified as high TDS and Na – Cl water type. The benefit for maximized hygienic water in quantitative and qualitative manners was checked through qualitative water changes.

Keywords: Spring water, Hydrochemical variability, Jordan Rift Valley, Faria

1. INTRODUCTION

Water springs were considered as the major source for drinking purposes in the rural Palestinian communities during the last decades. Springs played a major role in localizing the majority of the Palestinian villages according to their water needs. The stratigraphical outcropping in the eastern side of the Jordan Rift Valley is sedimentary rocks composed of limestone, dolomite and marly limestone rock, while the others are outcropping from evaporates of Lisan formation types. Determining the degree of water quality deterioration of these springs will help the residents of the targeted villages as well as the water planners of the Palestinian Water Authority to find appropriate methods for improving the quality of the polluted springs to be used for drinking purposes without harming the health of the people. This research will deal with the hydrochemistry and water quality assurance of twenty springs distributed north-south of the Jordan Rift Valley. Some of them lie within the west-east direction of the Faria basin. The main objective of this research is to determine the pollution level of these springs in reference to physical and chemical parameters as well as the trends of the hydrochemical variability of the north-south and east-west directions. The water quality of these springs was determined through measuring the concentration of the major and minor ion concentrations. Their suitability for irrigation purposes and the anthropogenic pollution of these springs were also evaluated.

2. STUDY AREA

The study area lies within the eastern flank of the West Bank Aquifers. Its groundwater generally flows towards the Jordan Valley. The available potential resource of this basin is

estimated to be 150 MCM/yr (Tahal, 1990). This basin supports over 79 large flowing springs of an average discharge greater than 0.1 liter/sec. These springs constitute around 90% of the total annual spring discharge in the West Bank. Sixteen of the springs are used for domestic and agricultural purposes. The study area consists of eight surface catchments including: Malih Shubash (4 springs), Malih (2 springs), Faria catchment (8 springs), Al-Ahmer (1 spring), Al Auja (1 spring), Nuweima (3 springs) and Qilt (1 spring), (Figure 1).

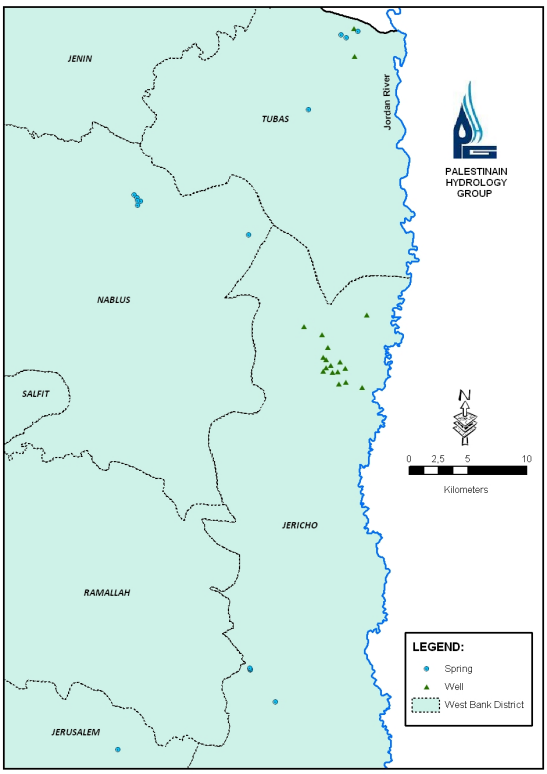


Figure 1: Location area with the sampled spring sites.

3. RESULTS & DISCUSSION

Fifty water spring samples were analyzed for the hydrochemical parameters of Ca^{2+}, Mg^{2+}, Na^{+}, K^{+}, Cl^{-}, SO_4^{2-}, NO_3^{-}, HCO_3^{-} as well as for heavy metals of Fe, Zn, Li, Cr, Co, Ni, Cu, Cu, Mn, Al, Cd, Ba, Pb, Bi in order to understand the hydrochemical characteristics of the water springs as well as to understand their variability trends in the north south and west east directions.

3.1 WATER TYPES

Three types of water springs were found: Na-Cl, Na-HCO_3 and Ca-HCO_3. Piper diagrams were plotted for all the water samples analyzed in terms of major ions using AquaChem 5.1 software. It is clear from the graph that most of the samples fall into the normal earth alkaline water group with prevailing bicarbonate and sulfate or chloride (Figure 2). Samples taken from Al Jesir and Shosah springs are classified under normal earth alkaline water with prevailing bicarbonate. The sample taken from Al Hilwah spring is earth alkaline water with increased portions of alkalis (prevailing sulfate and chloride), this water type is characterized by its high TDS and Cl^{-} which may be an indication of severe contamination with wastewater, the sample taken from Al Sultan spring was normal earth alkaline water with prevailing sulfate while the one taken from Mayta spring was alkaline water with prevailing bicarbonate. However, that from Tayoon spring was normal earth alkaline water with prevailing bicarbonate. This type of water originates through natural processes by the dissolution of carbon dioxide (CO_2) from the atmosphere and from the soil horizon which

causes the dissolution of the carbonate minerals, calcite $CaCO_3$ and dolomite $(Ca, Mg)(CO_3)_2$ of the aquifer.

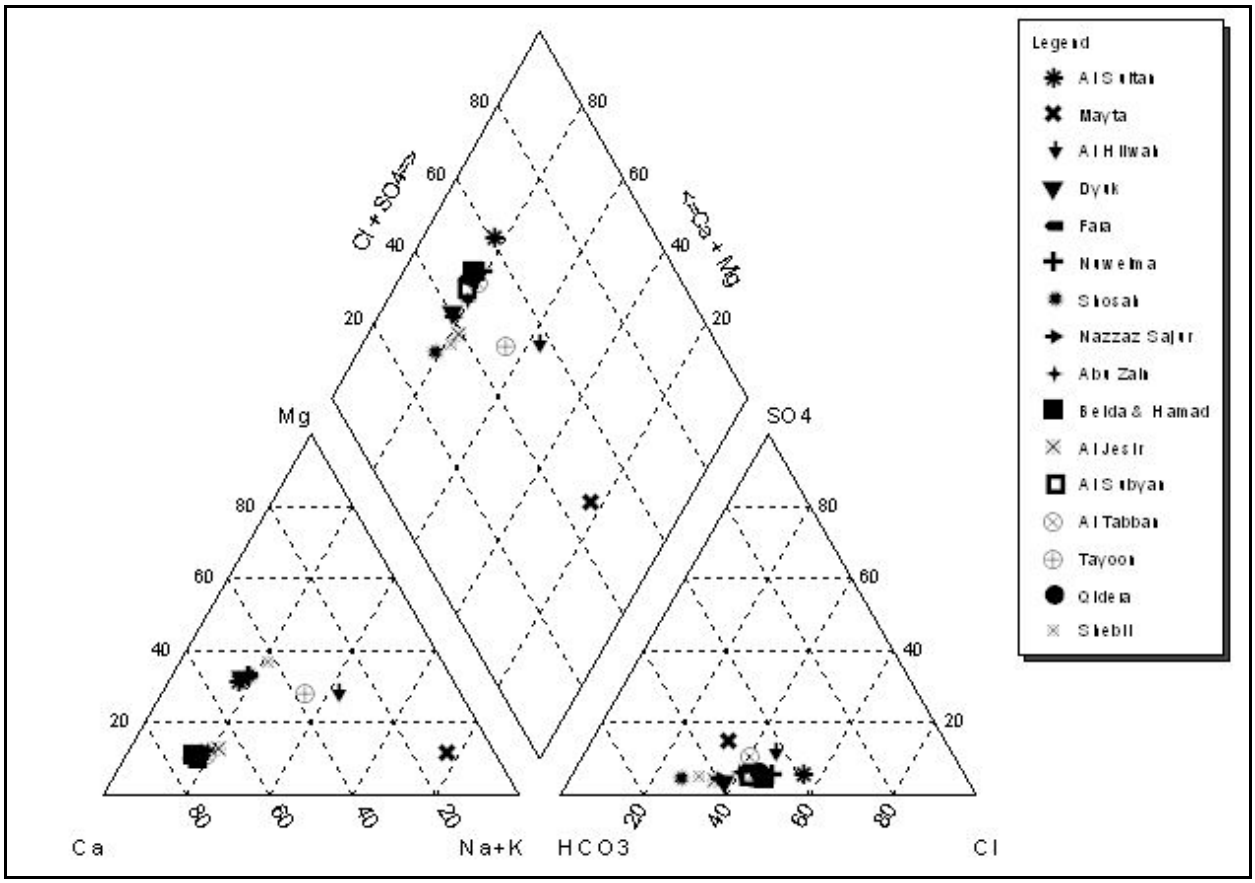


Figure 2: Piper Diagram for the springs

Durov diagram is used to display some possible geochemical processes that could affect the water genesis (Domenico and Schwartz, 1990) (Figure 3). Samples from Mayta spring corresponds to Field 6, Shosah spring lies in Field 2, those from Tayoon, Nuweima, Al Sultan, Dyuk, Al Hilwah and Shebli springs lie within the Field 5. The plotted water spring samples in Duruv Diagram agree with their relevant water types of the springs.

3.2 STATISTICAL ANALYSIS

The water of the spring samples shows the following ionic strengths for major ions: $Ca^{2+} > Na^{+} > Mg^{2+} > K^{+}$ and $Cl^{-} > HCO_3^{-} > SO_4^{2-} > NO_3^{-}$

The analyzed parameters were subjected to product linear correlation analysis (Mendenhall, et. al, 2003). Correlation coefficients were calculated for all possible pairs of variables. These numerically express the extent to which two variables are perfectly correlated, a value of 0 indicates that the variables have no correlation at all (i.e., they are completely independent) and a value of -1 indicates that they are perfectly negatively correlated (an increase in one corresponds to a decrease in the other).

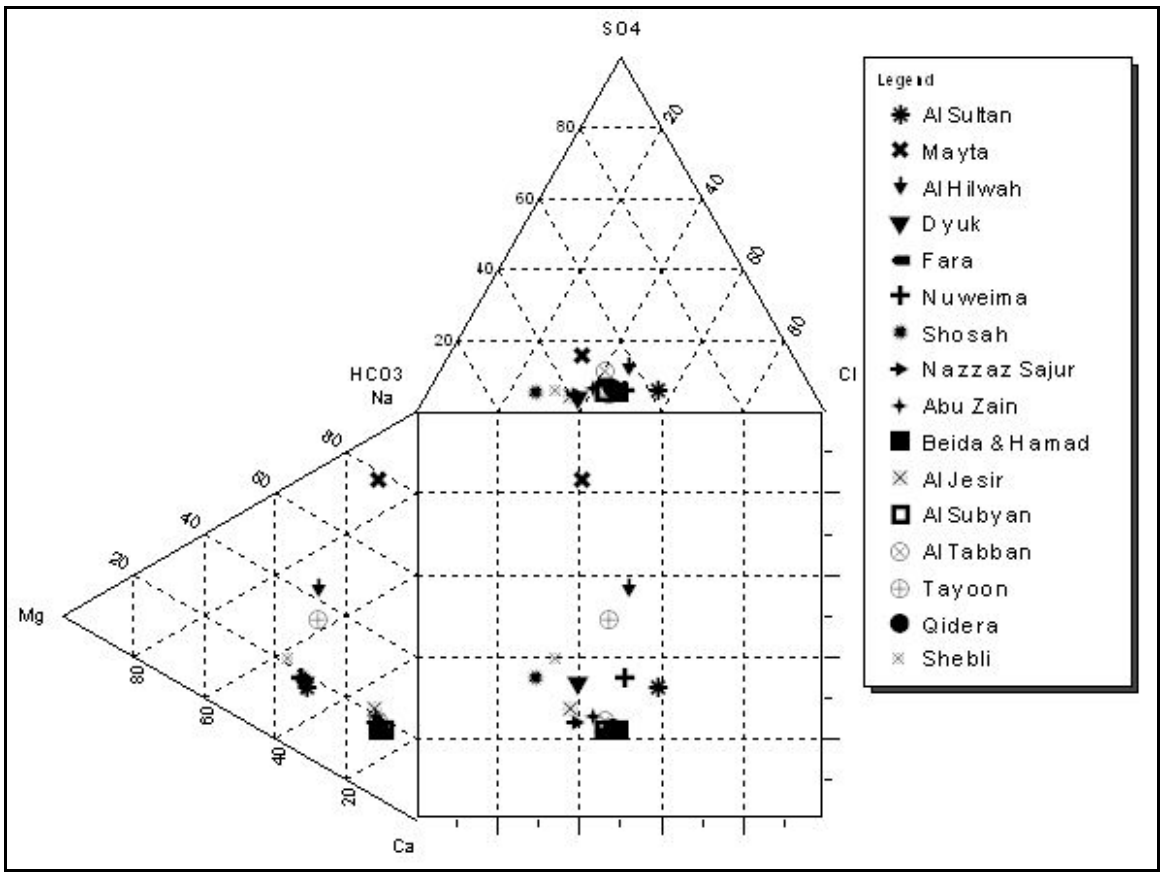


Figure 3: Durov diagram for the spring samples.

A good correlation in the spring water analyses were found between Fe and Al with all cations as well as SO_4 and Cl from the anions in addition to the Zn, Mn, Bi. Ca and Mg show good correlation to Bi, Al, Zn. Cr shows a negative correlation to Na and positive correlation to Co and Ni. Barium shows a positive correlation to Mn and negative correlation to Cr. In the well matrix correlation, the major cations show good correlation to Al and Ba. The following correlation pairs were found: (Fe to Mn and Pb), (Cr to Na and Mg), (Zn to Ni and Al), (Ni to Li), (SO_4 to Ba) and (Bi to Al). The quality of the recharge water and its interaction with soil and rocks during its percolation and its storage in the aquifers are key factors in the chemistry of groundwater (Todd, 1980). These interactions involve mainly dissolution and precipitation processes, which are controlled by the solubility products of the different involved mineral phases. One of the procedures to determine the equilibrium state of water with a mineral phase in the matrix of the aquifer is by calculating the saturation index. The results of the saturation index show that all the spring water samples were found to be undersaturated with respect to all the minerals. These relationships may be comparatively simple and uncomplicated as in the case of an aquifer that receives direct recharge by rainfall and from which water is discharged without contacting any other aquifer or other water.

3.3 WATER QUALITY EVALUATION

Accordingly most of the samples collected from the springs are considered to be of fresh water type with the exception of that taken from Al Maleh which is brackish. Calcium concentration in the majority of the springs are in the range that reflect the limestone origin, with the exception of the northern springs that reflects the Na–Cl type (Lloyd, J. (1999). From the results it is noticeable that high concentrations of Mg^{2+} usually accompany high concentrations of Ca^{2+} which may be an indication of dolomitization, especially for the northern springs. The low Mg^{2+} concentrations reflect the carbonate origin. All sampled springs are within the WHO range with the exception of

the Malih spring, where an increase reaching a maximum limit of 608 mg/L and chloride concentration of 830 mg/L and has the Na-Cl type. The Na concentrations of the sampled wells are higher than the WHO standard which reflects the Na-Cl type. The K concentrations are lower than its limit, with the exception of the northern springs and the sampled groundwater wells. The results show that the sulfate concentrations are under the range of the WHO limits. No health-based guideline value is proposed for chloride in drinking water. However, chloride concentrations in excess of about 250 mg/l can rise to detectable taste in water. The chloride concentrations of the sampled springs are within the WHO range, with the exception to Malih spring. The HCO_3 concentration results in the sampled springs reflect the Ca-HCO_3 water type. The nitrate results of the water samples are within the range of the WHO limits. The results of the trace elements show that all springs, with the exception of Malih springs have a concentration within the WHO standards with the exception of Pb and partially Al. Tables 1 and 2 summarizes the average hydrochemical parameters of the analyzed water spring samples highlighting those that exceed different water quality standards.

Table 1: Major ion concentrations compared to standards of the sampled springs

Spring	**Na**	**K**	**Mg**	**Ca**	**NO_3**	**Cl**	**HCO_3**	**SO_4**
	mg/l							
Malih	608	42	94	291	1.2	830	122.4	98.2
Mayta	290	2	22	39	7.6	65	178.5	39.1
Al Hilwah	120	5	42	74	8	72.6	117	24.5
Dyuk	30	3	30	79	5.2	66.2	178.5	7.1
Fara	25	1	8	101	2.7	35.6	75.2	8.1
Nuweima	29	3	28	69	5.7	55	92	8.1
Shosah	29	3	28	69	6.3	34.1	153	7.3
Qidera	17	2	6	66	3.3	32	61	5.4
Nazzaz Sajur	17	2	6	60	3.1	27.3	76.5	4.2
Al Sultan	30	3	30	83	9.8	68.3	82	8.7
Abu Zain	18	1	6	57	2.8	33.1	77	6.1
Shebli	26	2	24	46	3	33.3	117.1	7
Beida & Hamad	14	1	5	56	3.5	34.2	61.2	4.2
Al Jesir	16	2	5	45	3.4	31.2	92	4.1
Al Subyan	16	2	5	62	4.8	28.8	61	4
Al Tabban	16	2	5	53	3	26.8	56	9.2
Tayoon	44	2	19	42	3.8	33.8	67	4.1

: Exceeding Palestinian Standards,
: Exceeding WHO Standards
: Exceeding both Palestinian and WHO Standards

By comparing the results with the hardness classification, all the water samples range from being hard to very hard with the exception of that taken from Al Jesir which is moderately hard.

3.4 TRENDS

Geographical trends of the measured parameters were prepared for the springs in Al Fara which include: AQ/37A, AQ/O37B, AQ/038, AQ/039, AQ/035, AQ/041 and AQ/022 (Figure 4 and 5). An increasing concentration trend is shown from West to East for the spring parameters of Na, K, Cl, SO_4, CO, Bi while a decreasing trend is shown for the following parameters of Ca, HCO_3, LI, Ni, Pb, NO_3 and Al. Increasing concentration trends of the North – South profile are shown for Mg, Ca,

Na, K, HCO_3, SO_4, Pb and Bi; while a decreasing trend is shown for the parameters of NO_3, Cl, Ba and Al (Figure 4).

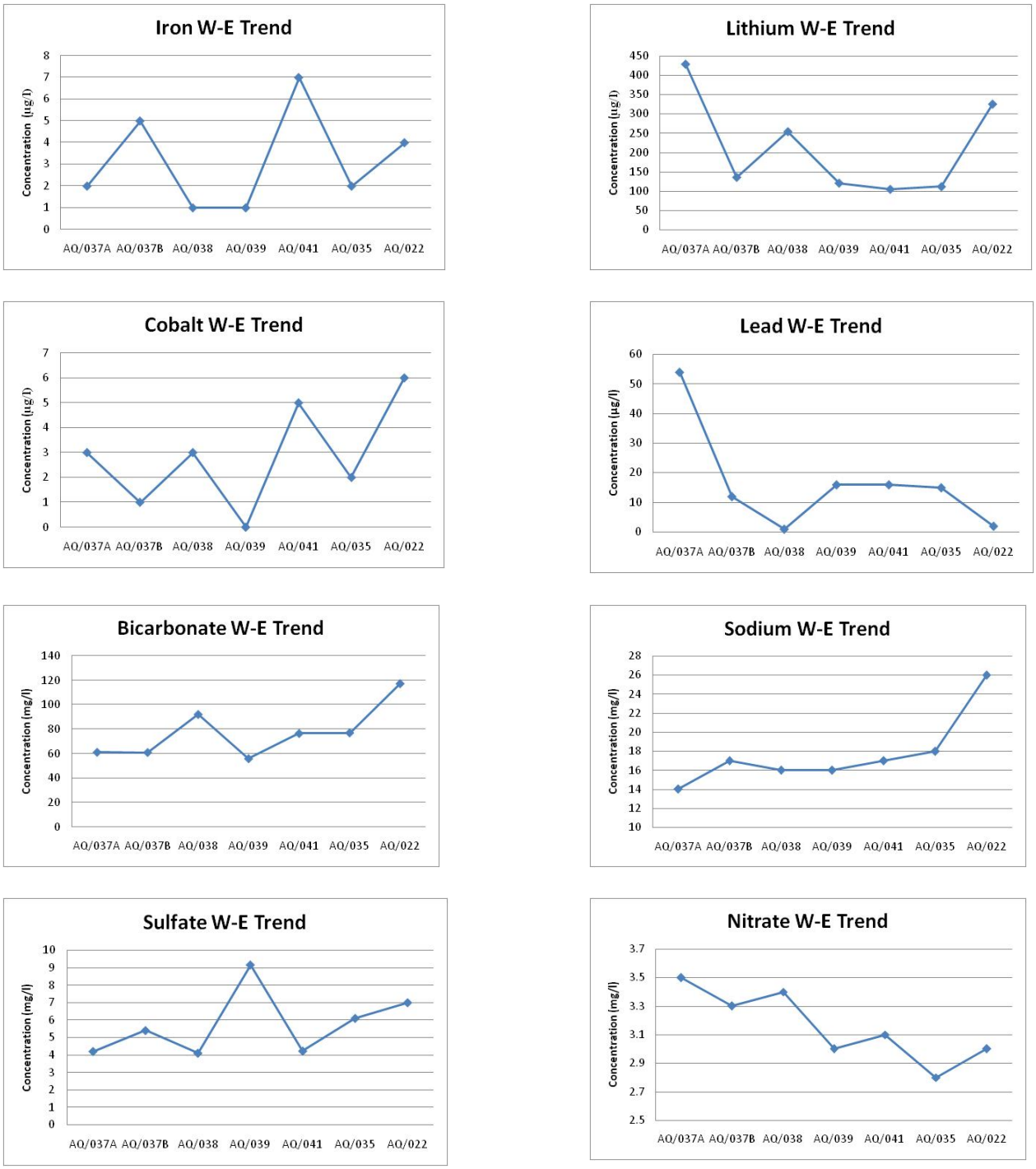


Figure 4: W-E trends of major ions and trace elements

Table 2: Trace element concentrations compared to standards of the sampled springs

Spring	Fe	Zn	Li	Cr	Co	Ni	Cu	Mn	Al	Cd	Ba	Pb	Bi
	µg/l												
Malih	1287	254	145	0	0	3	0	15	984	0	54	23	473
Mayta	663	49	67	0	0	11	0	19	578	0	65	18	98
Al Hilwah	36	113	76	0	2	3	0	2	77	0	66	13	146

Dyuk	16	172	64	1	0	1	0	0	57	0	39	12	153
Fara	32	160	186	3	1	12	0	2	78	0	165	25	75
Nuweima	18	83	365	1	2	3	0	2	54	0	43	28	113
Shosah	12	57	289	0	3	12	0	1	59	0	43	51	115
Qidera	5	48	136	0	1	4	0	2	62	0	127	12	7
Nazzaz Sajur	7	41	105	3	5	7	0	2	55	0	106	16	19
Al Sultan	19	26	1	0	4	3	0	0	44	0	32	12	98
Abu Zain	2	29	112	0	2	5	0	0	34	0	112	15	2
Shebli	4	32	325	0	6	2	0	0	20	0	45	2	98
Beida & Hamad	2	43	428	0	3	7	0	0	29	0	92	54	31
Al Jesir	1	23	254	2	3	1	0	0	33	0	93	1	45
Al Subyan	2	45	188	1	1	9	0	0	41	0	96	31	2
Al Tabban	1	32	121	0	0	5	3	0	23	0	98	16	22
Tayoon	5	23	347	0	5	3	0	1	22	0	31	17	86

: Exceeding both Palestinian and WHO Standards

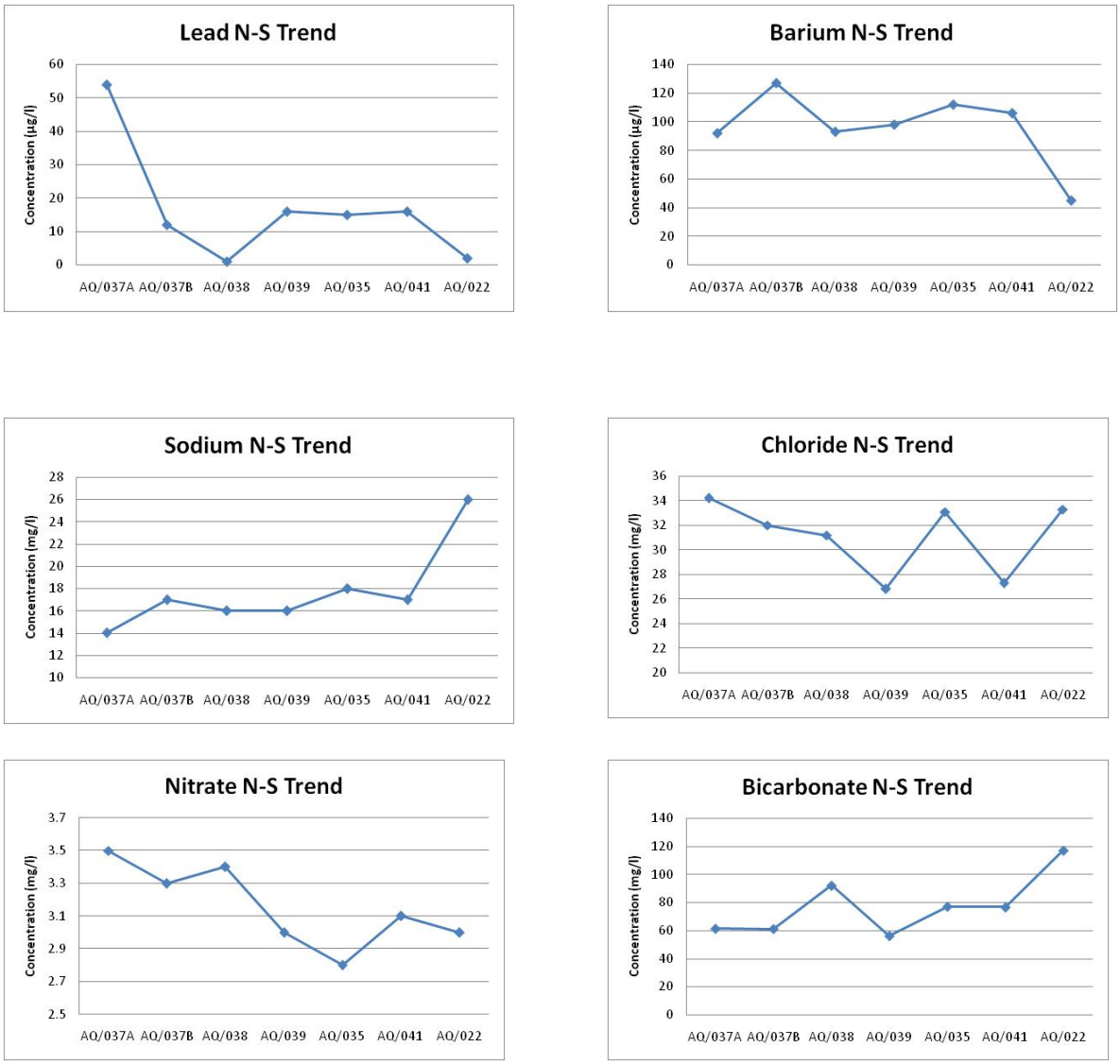


Figure 5: N-S trends of major ions and trace elements

4. CONCLUSION

The hydrochemical characteristics of twenty springs, distributed north – south in the Upper Jordan Rift Valley indicate that the increasing concentration trends of Na^{+}, SO_4^{2-}, NO_3^{-}, as well as of Fe, Co, Bi towards the south is due to natural and manmade polluted sources. The industrial wastewater flow discharge in Faria area increased the concentrations of Co, Bi, Li (from west to east) and threatens the quality of the groundwater. The majority of the spring water samples fall into the normal earth alkaline water group with prevailing bicarbonate and sulfate or chloride. The southern springs show high TDS and are of Na – Cl water type. The benefit for maximized hygienic water in quantitative and qualitative manners was checked through qualitative water changes.

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A SECURITY-ORIENTED MANUAL QUALITY SAMPLING METHODOLOGY FOR WATER SYSTEMS

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Abstract: The management and security of water resources will be a key challenge in the years ahead. This work investigates the problem of computing solutions for manual sampling of water quality in drinking water distribution networks. This work contributes to research be presenting a problem formulation based on the underlying substance propagation dynamics, coupled with the impact dynamics describing the "damage" caused by a contamination in the drinking water distribution network. The proposed solution methodology optimizes a risk-objective, in order to compute a manual sampling scheme comprised of sampling nodes and times, using evolutionary computation techniques. To illustrate the methodology, we present simulation results of a real water distribution network.

Keywords: water distribution networks, manual sampling, fault diagnosis, security.

1. INTRODUCTION

Water utilities are responsible for the delivery of drinking water to consumers, and for regularly monitoring water quality at various locations in the network, so as to assure the water quality is within certain prespecified bounds. Water quality checks are important since they verify the suitability of the water to be consumed. Water quality is characterized by the concentrations of various substances, organic or inorganic, which may correspond to either disinfectants or contaminants. In addition, quality monitoring is important for detecting contamination problems that may occur within the water distribution network.

According to the World Health Organization (WHO, 2006), a free chlorine residual concentration must exist in drinking water distribution systems, with minimum target concentration $0.2\ \frac{mg}{L}$ at point of delivery and $0.5\ \frac{mg}{L}$ for high-risk circumstances. It is usual practice to supply water with a few tenths of a milligram per litre of chlorine residual. Specialized sensors measuring the chlorine residual on-line can be used, however these are usually expensive equipment and have so far not been widely utilized, especially by small water utilities. When these chlorine concentration sensors are not available, the standard approach is for the water utility to perform manual water sampling for quality analysis. Water quality can be checked at a few nodes within a network, and for a few times during the day. Sampling locations may be selected by water utility personnel in an arbitrary fashion, which could be subject to human bias, or may be chosen using certain regulatory requirements, depending on consumer distributions and historical data. However, due to the drinking water distribution network's complexity, the decision of the time and the location for sampling may not be suitable from a fault diagnosis and security perspective.

Contamination faults in the drinking water distribution network can occur anywhere due to natural or accidental events, or even malicious attacks. The impact in terms of losses or damages caused due to such a contamination depends on the time and place the fault occurs. For example, a contamination occurring at an end pipe in the network cannot propagate to consumers other than those served by that pipe; on the other hand, a contamination near the water supply source may propagate to the majority of consumers served. Due to the time-varying nature of demands, water flows in

pipes fluctuate during each day, and that influences the rate of propagation of a substance in the water distribution system.

Fault tolerance and security in water distribution systems is an area of increasing research interest, and requires the synergy of various fields such as control systems, water engineering, optimization and risk management. Currently, various methodologies have been proposed for computing a set of locations, in order to install quality concentration sensors that can detect contaminations, while satisfying certain constraints and optimizing certain objectives. The primary assumption is that quality concentration sensors permanently installed at certain locations in the network, can detect contaminants above a certain concentration threshold. Using on-line sensors, however, for quality sampling, and in specific for fault diagnosis, is not widely accepted due to calibration problems, as well as due to the cost. In practice, manual sampling and chemical analysis at a laboratory is, for the time being, an effective and established approach for detecting quality faults.

The general location problem has been examined in various research disciplines, such as operations research, systems theory and control, combinatorial optimization and others; it describes the problem of selecting at which locations to place certain facilities (such as a public service) or sensors. One of the first mathematical formulations is the "Set Covering" combinatorial problem, in which the underlying system is expressed as a graph, where nodes represent locations and arcs represent functional or geographical relations between location nodes. The problem can be solved as an integer optimization program, computing the location nodes so that all the remaining nodes are served based on the prescribed optimization and constraints (Christofides and Paixão, 1993).

A similar problem is the "Maximal Covering", which, instead of full coverage, it searchers for solutions that maximize a certain serving distance or time (Church and ReVelle, 1974). Population weights are assigned to each node in the graph while distance weights are assigned to arcs. The optimization program maximizes the number of people served, constrained by a fixed number of facilities and a limiting serving distance. The problem of selecting locations for water sampling in drinking water distribution networks was first formulated as a problem of "Maximal Covering" (Lee and Deininger, 1992). The objective was to find the locations from where to measure water quality, so that the maximum of water demand is checked. Using the hydraulic data, the authors computed the "contribution" of water from one node to another. They proposed a scenario-based approach for handling flow variances during a day. An optimization algorithm was formulated for finding the best locations covering the maximum demand. Various methods such as heuristics and genetic algorithms were proposed for solving this optimization problem (Kumar et al., 1997; Al-Zahrani and Moeid, 2001). The computed solutions were evaluated using randomized input patterns. For the fault detection and security problem, other mathematical formulations have been developed for solving the sensor placement problem using different impact metrics, such as the time of detection, population affected, or amount of consumed water (Berry et al., 2006a, Hart et al., 2007). Based on this work, Boccelli and Hart (2007) have examined the quality monitoring problem by extending the formulation for computing sensor locations that maximize the coverage, constrained by a desired water age distribution. Within the water management community, significant research interest on security issues in drinking water distribution systems was instigated after the design competition entitled "Battle of the Water Sensor Network" (Ostfeld, 2008), where the objective was to place sensors in two real water distribution networks in order to minimize a number of objectives.

The problem of manual sampling was discussed by Berry et al. (2006b), where the authors examined the problem of scheduling manual sampling for contaminant detection. They proposed a mixed integer program for computing the sampling route; i.e. the location and time to take samples, while considering certain real conditions such as utility working hours, the time required for sampling and the travelling time between nodes. Solving this optimization problem, however, required significant computational power, something water utilities may lack.

In the present work we concentrate on the mathematical formulation of the manual sampling problem, when considering risk-objectives. Although this problem is related with the sensor placement problem for which there has been some research, there are significant differences, the most

important of which is that the solution is not a set of nodes, but a set of nodes coupled with time, which answers *when* and *where* to perform sampling, under certain constraints.

In Section 2, we present the problem formulation and in Section 3 the solution methodology; in Section 4 we demonstrate the methodology with a real example. Finally, in Section 5, we present some final conclusions and future work.

2. PROBLEM FORMULATION

In this section we define the dynamics related to the propagation of a contaminant within a drinking water distribution system. Moreover we present measurable metrics characterizing the "damage" caused due to water consumption.

Consider a drinking water distribution network, comprised of pipes, junctions and water storages. The topology of this network can be represented as a graph, with edges corresponding to pipes and m nodes correspond to junctions and water storages. For modelling purposes, each pipe is *a priori* segmented into a number of finite volume cells, by using a suitable algorithm; in total m_0 finite volume cells are considered. Let n be the number of all junctions, water storages and finite volume cells in all pipes, such that $n = m + m_0$. The set of all network indices, corresponding to each cell, is defined as $V = \{v_1, ..., v_n\}$; in addition we define and the set of all node indices V_0, comprised of $v_i \in V, i = 1, ..., m$.

Let $x_i(k)$ denote the contaminant concentration at discrete time k within finite volume cell v_i; the advection and reaction dynamics describing the propagation of the substance in the water distribution network, can be expressed in a state-space formulation

$$x(k+1) = A(k)x(k) + g(x(k)) + \varphi(k;\theta), \tag{1}$$

where the vector $x(k) = [x_1(k), ..., x_n(k)]^\top$ is the state of chlorine dynamics; $A(k)$ is an $n \times n$ matrix that characterizes the advection dynamics and $g(x(k))$ is a function that describes the reaction dynamics of the contaminant. Function $\varphi(k;\theta)$ describes the change in the contaminant concentration at the contamination source; the location of the contamination and the characteristics of the input signal depend on the fault parameter θ.

When a contamination fault occurs, the contaminant is propagated in the network, and eventually it may reach consumers who will consume the outflow water. The purpose of a security-oriented manual sampling should be to reduce the risk of "damage" caused by the contaminant consumption; therefore, it is necessary to consider metrics which measure the impact of a contamination fault on the system. Let $W \subset V_0$ be the set of N_d node indices corresponding to locations which outflow water to consumers, based on demand requests. For each "demand" node $w_i \in W, i = 1, ..., N_d$, an impact value $\xi_i(k)$ can be computed at each time step k. The impact of a fault depends on the volume and contaminant concentration of the contaminated water consumed.

In the present work, we define $\xi_i(k)$ to be the volume of contaminated water consumed at demand node $w_i \in W$, until time k. The state-space representation of the impact dynamic is given by

$$\xi_i(k+1) = \xi_i(k) + f_\Xi(x_{w_i}(k), d_{w_i}(k)), \tag{2}$$

where $d_{w_i}(k)$ is the outflow demand, measured in $\frac{m^3}{s}$, at demand node w_i. Function $f_\Xi(\cdot)$ is a non-negative function which computes the impact increase, i.e. the volume of polluted water consumed, at each time step, and is given by

$$f_\Xi(x_{w_i}(k), d_{w_i}(k)) = \begin{cases} d_{w_i}(k)\Delta t & \text{if } x_{w_i}(k) > \eta \\ 0 & \text{if } x_{w_i}(k) \leq \eta \end{cases} \tag{3}$$

where $\eta \geq 0$ is the concentration threshold above which the consumed volume is assumed to be polluted and Δt is the length of each time-step. Furthermore, we define $\psi(k)$ as the output vector, given by

$$\psi(k) = f_\Psi(\xi(k)) \tag{4}$$

where the function $f_\Psi(\cdot)$ characterizes the total impact, or "damage", which has caused due to a contamination fault. For the contaminated water consumption volume considered in the present work, the total impact is given by the aggregated volume of consumed contaminated water, i.e. $f_\Psi(\xi(k)) = \mathbf{1}^\top \xi(k)$, where $\mathbf{1}$ is a vector of ones of appropriate dimension.

3. METHODOLOGY

Given a network topology with certain hydraulic dynamics which are subject to uncertainties, let $S \subseteq V_0$ be the set of N_s candidate sampling nodes, i.e. locations where water sampling is feasible, such that $S = \{s_1, ..., s_{N_s}\}$. From a practical viewpoint, we consider that sampling can be performed at certain times within a day (e.g. due to working hours); we define T as the finite set of time steps at which sampling can be conducted within a period with discrete length T_H, i.e. $T = \{\tau_1, ..., \tau_{N_\tau}\}, \tau_i \leq T_H$. Hydraulic dynamics are considered approximately periodic, with a 24-hour period, due to the periodic nature of consumer demands. The set Q of all feasible sampling locations and times, is given by

$$Q = \{(i, j) \mid i \in S, j \in T\}, \tag{5}$$

such that $Q = \{q_1, ..., q_{N_q}\}$ and $N_q = N_s \cdot N_\tau$.

The manual sampling problem can be defined as finding the set $Y \subset Q$ of M sampling locations and times (where $|Y| = M < N_q$), such that one or more security-oriented risk objectives are optimized.

An approach to solve the sampling problem is to consider a number of "representative" quality faults that can occur within a water distribution system. Assuming that the hydraulics are periodic with period of one day, we can construct a finite set Θ of fault parameters, $\Theta = \{\theta_1, ..., \theta_{N_\theta}\}$, which correspond to N_θ fault functions $\varphi(k; \theta_i), i = 1, ..., N_\theta$. For simplicity, the elements in Θ can be obtained through grid sampling, for faults that have start time within one period. Let $t_s(\theta_i)$ be a function that computes the start time of the fault characterized by parameter θ_i. From a practical perspective, the detection time of a fault is bounded by a finite constant, because even if it is not detected during sampling, it will eventually be detected after some time instance t_{max} by other methods such as customer complaints or increased hospitalizations.

Let $x(k; \theta_i)$ and $\psi(k; \theta_i)$ correspond to the state $x(k)$ and output impact vector $\psi(k)$ when a fault with parameter θ_i occurs. We define the overall-impact matrix Ω of size $N_\theta \times N_q$, such that its (i, j) element, $\omega_{(i,j)}$, corresponds to the impact of fault with parameter θ_i, when manual sampling $q_j \in Q, q_j = (\mu_j, \lambda_j)$ is conducted, i.e. at the node μ_j and time λ_j. In specific, $\omega_{(i,j)}$ is given by

$$\omega_{(i,j)} = \begin{cases} \psi(\lambda_j; \theta_i) & \text{if } t_s(\theta_i) \leq \lambda_j \text{ and } x_{\mu_j}(\lambda_j; \theta_i) \geq \epsilon \\ \psi(\lambda_j + T_H; \theta_i) & \text{if } t_s(\theta_i) > \lambda_j \text{ and } x_{\mu_j}(\lambda_j + T_H; \theta_i) \geq \epsilon \\ \psi(t_s(\theta_i) + t_{max}; \theta_i) & \text{otherwise} \end{cases} \tag{6}$$

where ϵ is non-negative concentration threshold above which detection is possible and T_H is discrete length of one period considered.

In our problem formulation, we consider minimizing the average and maximum impact, with respect to the various contamination faults in Θ. Assume that the task is to select M sampling locations and times, out of N_q candidates, and let $L = \{1, ..., N_q\}$ be the set of sampling indices. We define $\mathbb{X}$ as the set of all M-combinations from the set L, of size $|\mathbb{X}| = \binom{N_q}{M}$, such that

$$\mathbb{X} = \{X \mid X \in L^M, X_i = X_j \Leftrightarrow i = j\}. \tag{7}$$

The general single-objective optimization problem is formulated as

$$Y = \underset{X \in \mathbb{X}}{\operatorname{argmin}} \{f(X; \Omega) \mid |X| = M\}, \tag{8}$$

where M is the number of samples taken within one period, and Ω is the overall-impact matrix. The function $f(X;\Omega)$ is the objective and its formulation depends on the risk-objectives considered; in the present work, the average and the maximum impact are considered.

The average impact metric is suitable for optimizing reliability, when a contamination event can occur at any node with equal probability. For the overall-impact matrix Ω whose (i,j) element is $\omega_{(i,j)}$, the average impact objective for solution X is given by

$$f(X;\Omega) = \frac{1}{N_\theta} \sum_{i=1}^{N_\theta} \min_{j \in X} \omega_{(i,j)}. \tag{9}$$

The maximum impact is used to reduce the effect of the most extreme contamination fault, in term of causing the most damage and is given by

$$f(X;\Omega) = \max_{i=1,\ldots,N_\theta} \min_{j \in X} \omega_{(i,j)}. \tag{10}$$

In summary, the methodology proposes the following steps:

1. Model the water distribution system using a mathematical formulation as expressed in (1)
2. Select the type of impact metric considered in (2)-(4)
3. Construct the set of representative fault parameters Θ and the set of feasible sampling locations and times Q
4. For each fault, simulate the model of the water distribution network and for each sampling location and time, compute the impact using (6), in order to construct the overall-impact matrix Ω
5. Choose the risk-objective (9),(1) to the considered in the optimization problem (8)
6. Solve the problem by using a suitable integer nonlinear optimization algorithm.

3. APPLICATION

In this section we illustrate the formulation and the solution methodology for the manual sampling problem in a real drinking water distribution network, operating under realistic conditions. The network is comprised of 321 pipes connected to m =198 junctions, 100 of which are used for water consumption. Disinfected water is supplied to the network from a single node. The structural characteristics are assumed to be known, i.e. pipe length, diameters and pipe roughness coefficients, node elevations and daily average consumption volume at each node. In addition, historical flow-data are provided, measured at the supply node, and are assumed to describe the normal operation over all nodes; the normalized flow for a 24-hour period is depicted in Fig. 1. We consider the set of nodes $V_0 = \{v_1, ..., v_{198}\}$ and a $\Delta t = 5$ minute time step for computing the hydraulic and quality dynamics; each 24-hour period is therefore comprised of $T_H = 288$ time steps.

There are $N_d = 100$ nodes in the network which outflow water to consumers; the set of demand nodes is $W = \{w_1, ..., w_{100}\}$, and $W \subset V_0$. We further assume that the concentration threshold η above which the consumed volume is assumed to be polluted, is $\eta = 0$. In this example, we consider that the set of candidate sampling locations is the same as the demand nodes, i.e. $S \equiv W$. Regarding the finite set T comprised of time steps at which sampling can be conducted, we assume that sampling can be performed every 30 minutes, thus there are maximum $N_\tau = 48$ time instances for manual sampling within one day. From these, the set Q of all feasible sampling locations and times has size $N_q = 4800$.

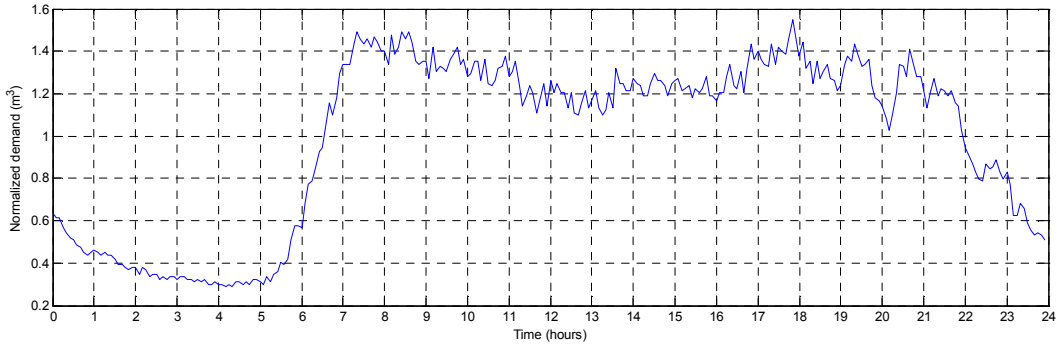


Figure 1. Normalized demand within one period

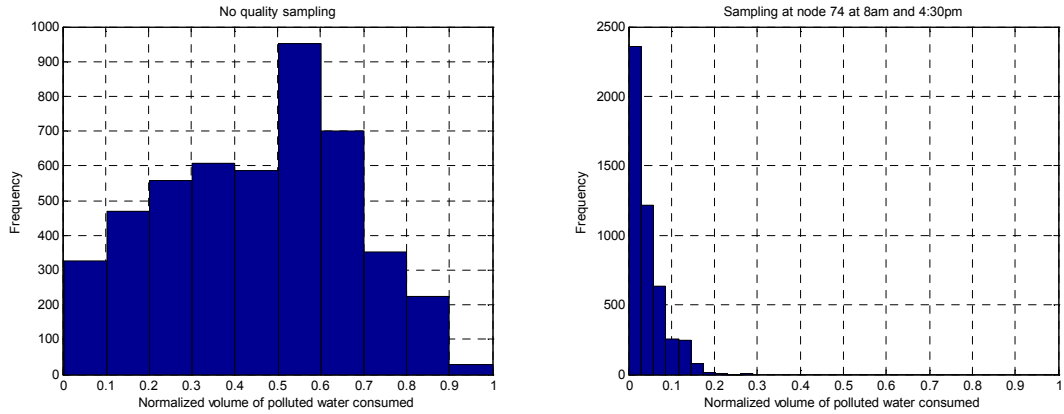


Figure 2. Histogram of the normalized impact for all the faults considered

For the construction of the fault parameter set Θ, we consider faults which start the contamination at a certain location in the network, and continue to contaminate the water at that location until detection. In specific, a fault with parameter $\theta \in \Theta$ is modeled as a step-function of $1\frac{mg}{L}$, which starts at a certain time within a certain period $1 \leq t_s(\theta) \leq 288$ and is terminated when the contamination is detected through sampling, or after a certain number of time-steps has passed, in specific $t_{max} = 288$. We construct the set Θ of fault parameters characterizing faults which occur at one demand node every half hour within one day; 48 faults are therefore considered for each node; Θ has size $N_\theta = 4800$. The first histogram in Fig. 2 depicts the normalized impact histogram of the faults considered on the system; it appears that a large number of faults cause extensive damage to the system if undetected.

For constructing the overall-impact matrix Ω, we assume that the time-varying matrix $A(k)$ is known for all times, that no reactions occur in the network $g(x(k)) = 0$ and that the outflow $d_w(k)$ for all demand nodes $w \in W$ are known. We repetitively simulate the operation of the system corresponding to duration of 48 hours, applying one fault θ_i at a time, for $i = 1, ..., 4800$. Within one simulation, we measure the impacts based on the various sampling schemes r_j, for $j = 1, ..., 4800$, using (6). Finally, the overall-impact matrix Ω of size 4800×4800 is constructed.

We examine the single-objective optimization problem, for computing a solution with $M = \{1, 2, 3\}$ sampling events within one period, i.e. a day, for minimizing the average (and maximum) volume of polluted water consumed. The optimization problem described by (8)-(10) is nonlinear, and for solving it we utilize an integer evolutionary optimization algorithm. Genetic algorithms have proven suitable for these types of optimization problems with constraints, in computing optimal or near-optimal solutions.

Table 1. Solutions for various sampling schemes and objectives

Samples	Objective	Sampling Period	Solution	% Max Impact
1	mean	all day	(74, 9am)	7,20%
2	mean	all day	(74, 6am), (74, 2:30pm)	4,00%
3	mean	all day	(74, 1am), (74, 10pm), (74, 4:30pm)	2,78%
2	mean	8am-5pm	(74, 8am), (74, 4:30am)	4,13%
3	mean	8am-5pm	(74, 8am), (77, 12am), (74, 5pm)	3,21%
1	max	all day	(58, 9:30am)	41,91%
2	max	all day	(58, 9am), (68, 6pm)	20,85%
3	max	all day	(76, 5am), (72, 11:30am), (78, 6pm)	15,78%

Table 2. Solutions for various sensor placements

Samples	Objective	Nodes	% Max Impact
1	mean	74	0,70%
2	mean	74, 151	0,45%
3	mean	74, 146, 147	0,37%
1	max	74	4,64%
2	max	78, 147	2,02%
3	max	56, 77, 147	1,90%

In each problem examined, the evolutionary algorithm iterated for 50 epochs, and for epoch a population of $10\,000$ individual solutions was build. The following scenarios were considered: a) optimizing the average or the maximum consumption of polluted water for various numbers of samples, $M = \{1, 2, 3\}$, samples and b) optimizing by considering limited sampling period, e.g. during work hours.

The results for various scenarios are summarized in Table 1. Each element in the solution corresponds to a node and a time when sampling should be performed. For instance, for the first row, if only one sampling is to be conducted within a day, by sampling at node "74" at 9am, the average consumption of polluted water consumed is minimized; furthermore, for this solution, the objective function, i.e. the average consumption of polluted water is 7.20% with respect to the maximum impact that can occur due to a contamination fault. When all-day sampling period is considered, some solutions may correspond to non-working hours; this however can be adjusted by imposing constraints to the optimization algorithm. As expected, the constrained solutions are equal or worse to the all-day sampling solutions. For the network examined, it appears that node "74" is quite important from a average impact objective.

The second histogram in Fig. 2 depicts the normalized impacts of all faults considered in this example, when the two-samplings per day solution within 8am-5pm is considered. The histogram demonstrates the significant decrease of the impact of all faults considered.

Regarding minimizing the maximum polluted water consumption, different results have been computed; for example, for two-samplings per day, examining node "58" at 9am and node "68" at 6pm can reduce the maximum impact by 20.85% of the maximum impact of the worst contamination fault. We should note that, as shown with previous studies by the authors, after a certain number of samples, increasing their number taken within a day (or the number of sensors installed) has a little effect in decreasing the average or maximum impact.

It is worth comparing some of the results of Table 1 with the results of a different problem, the sensor placement problem. Using a solution methodology that has been discussed in our previous work (Eliades and Polycarpou, 2008), we solved the quality sensor placement problem for $M = \{1, 2, 3\}$ sensors, using the same average and maximum impact objectives. By comparing the results of Table 1 with Table 2, we observe there are differences in the selection of nodes computed, as well as in the reduction with respect to the maximum impact; in fact we observe an approximately ten-fold reduction in the average and maximum impact. This will be further discussed in the conclusions.

4. CONCLUSIONS AND FUTURE WORK

Quality sampling is important in drinking water distribution network, and is receiving increasingly importance, along with security issues arising due to the fact that they are consider as part of the "critical infrastructure". In this paper we addressed certain issues regarding the selection of locations and times for manual sampling, in order to examine the quality of water, i.e. whether sufficient quantity of chlorine residual exists, or whether the concentration of a substance exceeds the pre-specified limits. The selection of location and time for manual sampling in this work is driven by security-oriented criteria, for minimizing the possible damage caused due to a contamination event, which may occur accidentally or even intentionally. The mathematical formulation describing the dynamics involved has been presented, and a solution methodology has been proposed for solving the problem. The problem is formulated into a single objective optimization problem, utilizing certain risk-objectives such as the average and maximum impact. A simulation example has been presented on a real drinking water distribution network. The optimization problem was solved using a single-objective evolutionary algorithm.

Although sensor placement in drinking water distribution networks offers increased security in comparison with manual sampling, as shown in the simulation examples, manual sampling is currently applied by most of the water utilities, and through the laboratory chemical analysis of the samples, a large number of faults can be detected. This work can be extended by considering time delays, dynamic manual sampling scheduling depending on the changes in the hydraulic dynamics and mixed sensor and manual sampling schemes.

ACKNOWLEDGEMENTS

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LINKAGE OF A FINITE ELEMENT FLOW MODEL WITH A SOIL MOISTURE MODEL: CHALLENGES UNDER SEMIARID CONDITIONS

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Abstract: The study areas of Wadis Qilt (Palestine) and Al Arab (Jordan) belong to the multilateral SMART-project in the Lower Jordan Valley. The strategy of SMART is to upscale knowledge, extracted from local catchment areas to the project scale. The aim of the sub-project is to evaluate natural resources on catchment scale by combining hydrochemical and hydraulical methods to develop a high precision model. Concerning the quantification of the system, two separated models will be linked: a numerical finite element flow-model for the groundwater passage and a new developed hydrological model JAMS.

Key words: SMART, integrated water resources management, coupling/linking models, numerical finite element flow-model, Lower Jordan Valley.

1. INTRODUCTION

The arid to semiarid Middle East is a region of extreme growth of population. Hence, the rare and over-exploitated water resources in that region have to be more protected against anthropogenic and geogenic pollution. One way to help solving that complex issue is to develop an intelligent and integrated strategy to manage all available water resources, which is the aim of the multilateral SMART-project in the Lower Jordan Valley.

To generate such an IWRM, all water resources (groundwater, surface runoff, waste water) of the valley and its shoulders have to be quanti- and qualitatively evaluated. The strategy of SMART is to upscale knowledge, extracted from local catchment areas to the project scale, which covers the area between Sea of Galilee, Jerusalem, Dead Sea and Amman. The study areas of the here presented sub-project are the Wadis Qilt (Palestine) and Al Arab (Jordan).

2. OBJECTIVES AND REALISATION

The aim of the sub-project is to evaluate natural resources on catchment scale by combining hydrochemical and hydraulical methods to develop a high precision model.

Concerning the quantification of the system, two separated models will be linked: a numerical finite element flow-model for the groundwater passage and a new developed hydro-logical model JAMS, which is excellently prepared for humid conditions. However, the empirical equations and input parameters have to be adjusted onto Wadi Al Arabs semiarid and Wadi Qilts arid conditions. In the semiarid and arid region it is important to know the main infiltration processes. A big contrast to the humid climatic zone is the thin or even not existing soil cover. The challenge is to understand the infiltration processes without a soil cover, which controls mainly the groundwater infiltration processes in the humid climatic zone. Consequently, questions have to be solved such as what role

does the thickness of the unsaturated hard rock zone and which of its conditions have an effect on groundwater recharge. The focus is not on the known parameter of percolation.

The advantage of combining two models like JAMS and Feflow is the calculation grids are identical. The power of JAMS is the highly accurate assessment of soil moisture balance and consequently of surface runoff and groundwater recharge.

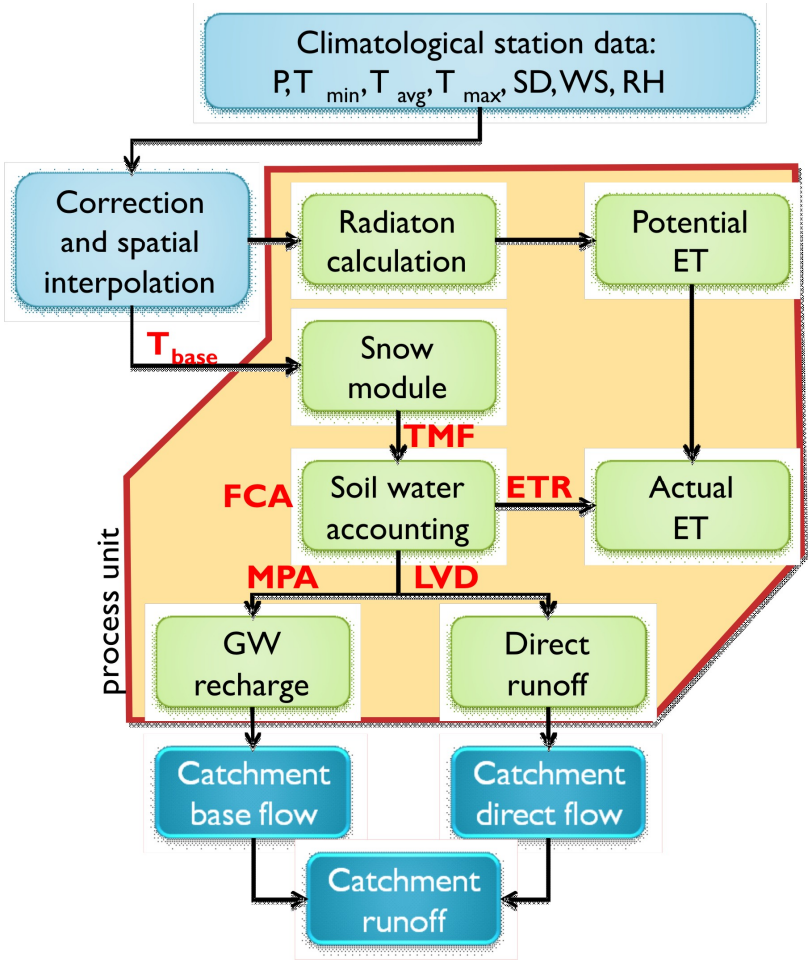


Figure 1: The internal program code of the soil moisture model JAMS (Krause 2009).

In Figure 1, the internal program code is visualised. The model uses local climate data and input parameters such as slope, aspect, land cover, soil, and altitude. The parameter soil can be further divided by specific physical conditions such as field capacity, thickness, hydraulic conductivity and possibility of capillary rise of water. For land cover, additional vegetable characteristics like stomata resistance, root depth, growth height and leaf area index define the hydrologically relevant type of cover.

The mentioned input data for JAMS are spatially organized in hydrological response units (HRU) which leads to a spatially discriminated output. The HRU mesh builds a raster of defined scale, and therefore, it is necessary to up- or downscale the input data, e.g. satellite scenes, aerial photographs and field data. Out of this data, JAMS calculates the evaporation, runoff and recharge. After the adaption of JAMS, the spatially and temporarily differentiated calculation of runoff and groundwater recharge is possible. Beside climatic gradients, the key issue is, to correctly evaluate the evapotranspiration in respect to the different classes of land use.

In the study area Wadi Al Arab, the groundwater recharge was calculated as area-indicated output parameter of JAMS. Figure 2 show the distributions of input-parameters of JAMS in the catchment area. For the land cover of Wadi al Arab, 13 different classes have been determined based on satellite data and aerial photographs. From the climatic database, values from September

2000 till December 2008 were used. The directly measured surface runoff for the years 2000-2005 (DAISY-Siebert et al 2009) was used to calibrate the runoff model.

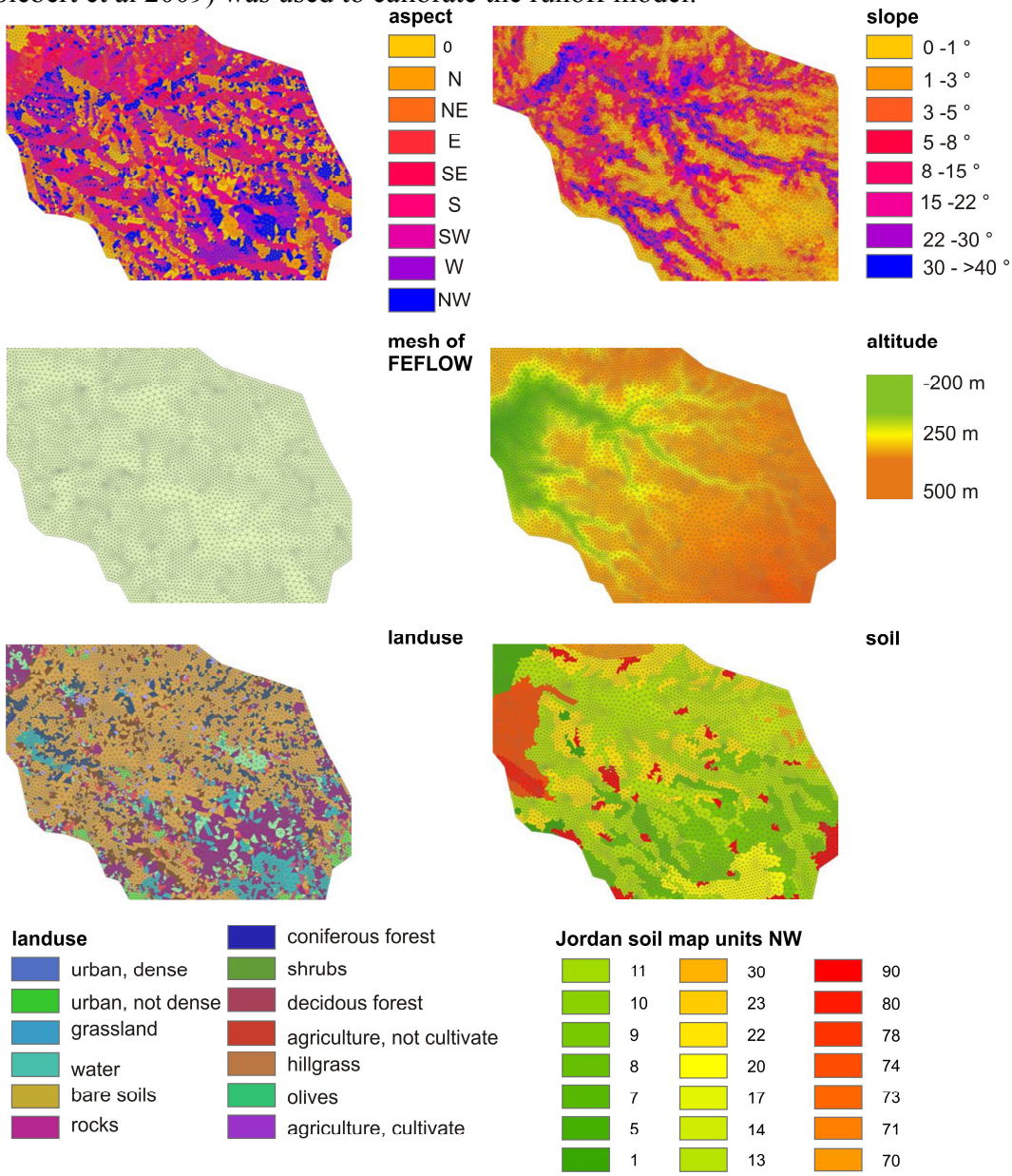


Figure 2: The different input parameters of the soil moisture model JAMS for Wadi al Arab.

Figure 3 show the correlation between the observed and simulated direct runoff. The measured runoff in Wadi Al Arab consists obviously solely of direct runoff without any base-flow components. First calculation results show that the annual groundwater recharge varies between 50 and 70 mm and the direct annual runoff of Wadi Al Arab is about 30 to 50 mm. To control the modelled dimension of groundwater recharge, the water level fluctuation method was used (Healy & Cook 2002). This method is based on monthly data of groundwater level that were available for the period of 1982 until 1998. The outcomes of water level fluctuation analyses and the modelled results are within the same range of 40 to 70 mm groundwater recharge per year in the catchment area of Wadi al Arab.

The modelled groundwater recharge is the fundamental source, to generate out of the steady state model (Figure 4) a transient numerical flow model. As mentioned above, the main advantage is, finite-element meshes of both JAMS and FeFlow are directly comparable. Hence, the spatially distributed results of JAMS provide spatial differentiated and more precise recharge values for the transient model.

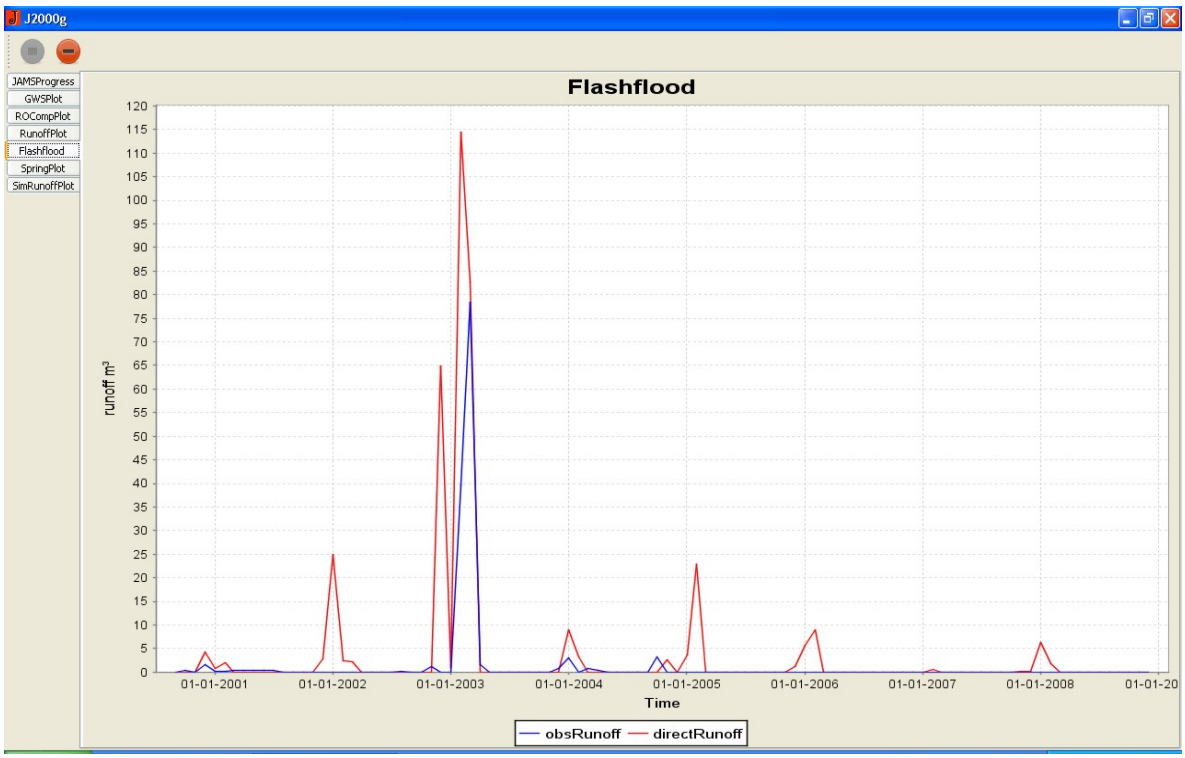


Figure 3: The equation between observed (blue) and simulated (red) direct runoff in JAMS.

One of the major challenges is the temporal discretization or time lag between the time when the recharge leaves the soil crust (JAMS model) and the moment when it enters the groundwater table (transient model). In fact, the aim is to evaluate the effects of retardation of the unsaturated or vadose zone in dependence to the hydraulic parameters of the underground and the entire groundwater reservoir. From a modeller's point of view, different models often use identical parameters that are differently termed. That's why; the individual definitions for values (recharge, base flow, exfiltration of JAMS, infiltration of FeFlow, etc.) of both models have to be linked by an interface. The link between both models is a transfer function, which can describe the thickness, the material and the structure of the vadose zone (Figure 5).

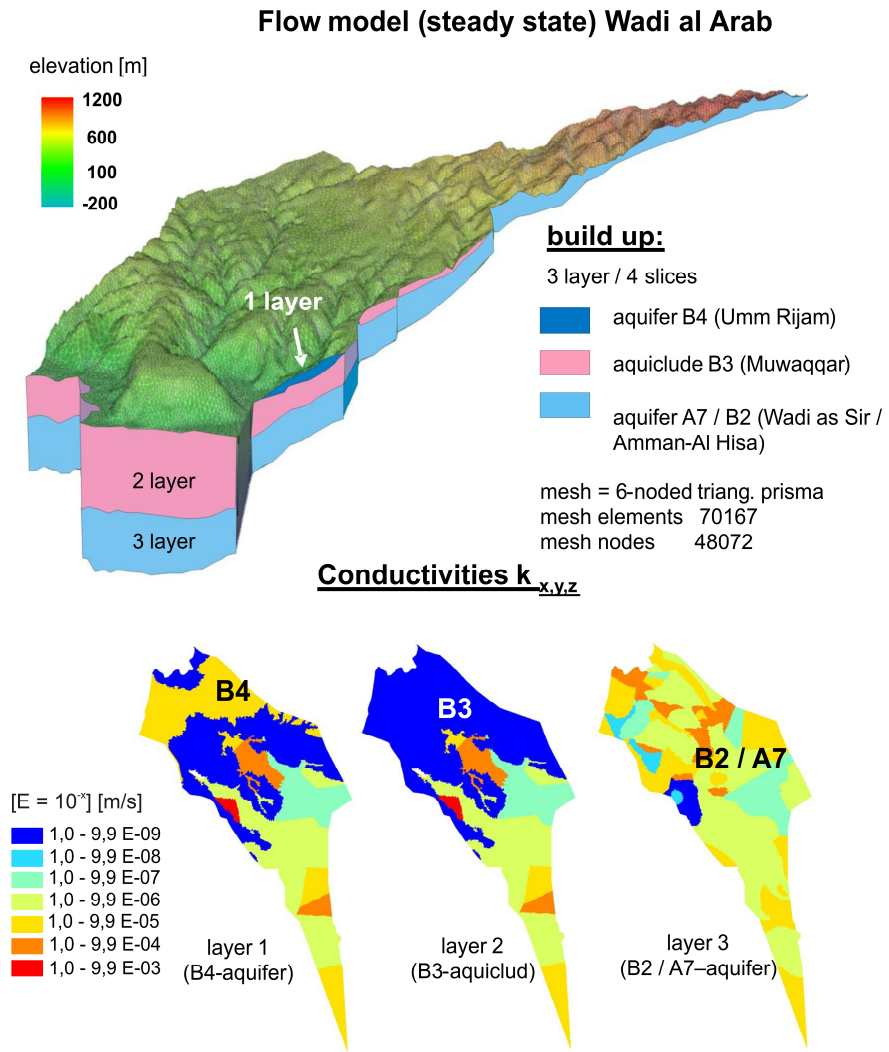


Figure 4: The buildup of the steady state flow model of Wadi al Arab.

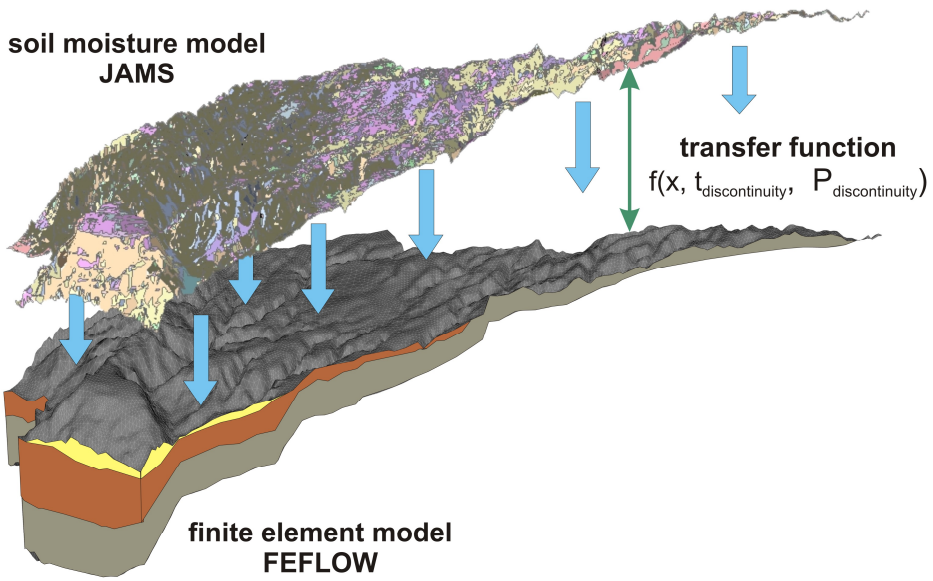


Figure 5: Linking of JAMS and FEFLOW by a transfer function.

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AIR BUBBLE GEOMETRY AT THIN WEIR FLOWS WITHIN INCLINED OPEN CHANNELS

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Abstract: In this experimental investigation the mean geometry of air bubbles which are generated because of the water flow downstream a thin weir – in an inclined (angle φ) open channel, is presented, analysed and discussed. The dimensionless air bubble's length, height and area are elaborated and compared both to the weir height and among them, for φ=0o-3o-6o-9o-11o and 14.5o and various Froude numbers. This bubble is created always over the channel bottom, its ceiling lies over the weir height and its length is larger than its height. If the area of the air bubble is known, then the total water area and weight between suitable upstream and downstream cross sections may be found.

Key words: Air Bubble. Weir Flow.

1. INTRODUCTION

1.1 General

Thin weirs are hydraulic structures placed in open channels mainly for water discharge measurements. Usually the channel is horizontal and the weir is vertical.

Apart from its main use the flow structure around a thin weir is very important for hydraulic engineers.

In this paper the rectangular channel (with B width) is mainly sloped with an inclination angle φ and a slope $J_o=\sin\varphi$, while the weir is perpendicular to the channel bottom. (Demetriou 2006), has presented a large number of experimental measurements not only for the discharge determination but even for some very important characteristics of such flows.

Fig. 1 schematically shows the weir flow configuration. The entire flow has an upstream head which is the difference between upstream depth d_1 and weir height w, $h= d_1-w$. Each depth d_1 is measured at a short distance $4\cdot h$, where – at cross section 1 – the flow is actually quasi uniform (gradually varied), not affected by the presence of the weir's body and consequent flow separation. On the crest of the weir the water depth is h' and $d_1'=h'+w$, while the upstream water area is A_1. Corresponding downstream area - along L – is A, while at cross section 2, the depth has its minimum, d_2, and at any x distance the water depth is d. All d depths form the downstream mean free surface water profile, while N_x is the total force exerted by the weir body on the flowing water along x.

In this experimental investigation an air bubble (or air pocket) was always present in touch to the downstream face of the weir, although no artificial aeration was used – but the flow was freely left to develop by itself in purpose, since the physical part of the flow was of great importance. If the air bubble has a mean area A', then the entire water area between cross sections 1 and 2 is $A+A_1-A'$.

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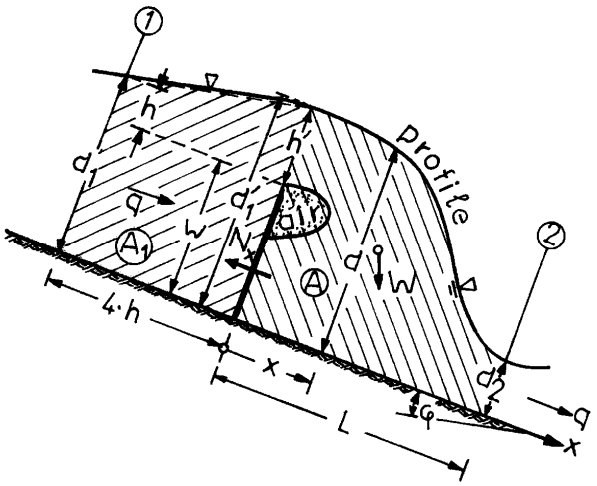


Figure 1. Flow characteristics.

The main objective of this paper is the experimental determination of the air bubble geometry, although some other quantities such as the d_2/d_1' ratio vs h/w and angle φ, are also examined. The main flow parameter is the Froude number at cross section 2,

$$Fr_2 = q/g^{1/2} \cdot d_2^{3/2} \qquad (1)$$

where q the discharge per unit width. Since at this cross section the flow is always supercritical Fr_2 numbers are always larger than unity.

1.2 The air bubble geometry

In order to determine the various geometrical elements of air pockets, all bubbles' outlines were recorded through the transparent side wall of the laboratory channel. Fig. 2 shows the general shape of these air pockets which were freely formed (no artificial aeration), and presented a rather typical configuration, since they were always in touch with the downstream face of the weir, they had a mean length b and a mean height α and were at an average distance t over the channel bottom – where $t>d_2>0$. The ceiling of any bubble was over the weir's height, i.e. $(\alpha+t)>w$, while its bottom was sloping towards the weir body.

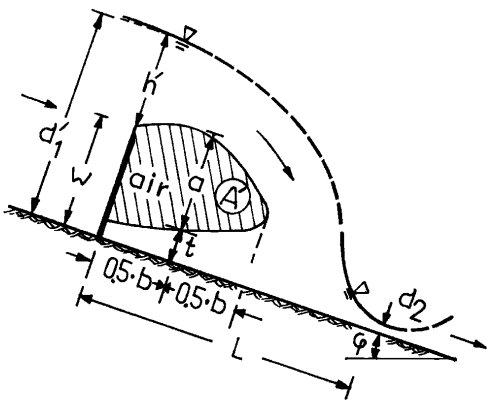


Figure 2. Air bubble geometry.

3. THE EXPERIMENTS

All experimental measurements were performed in the Hydraulics Laboratory of N.T.U.A. A small tilting perspex flume was used with inclination angles $\varphi=0^o$-3^o-6^o-9^o-11^o and 14.5^o and

w/B=0.5-0.75-1.0-1.25 ratios. The discharges were volumetrically measured and all depths were determined with a depth meter. A number of 24 runs were organised and a large number of depths were recorded. Fr_2 varied between 2.6 and 16.0, Re_2 ranged between 6,250 and 31,200, while w/h were in the field 0.8 and 10.

All air pocket outlines were recorded through the transparent side wall of the flume and all area A' were approximately measured after the end of experiments. Areas A_1 were also approximately measured, while areas A were calculated through the determination of the water free surface profiles' equations between x=0 and x=L. The total water area is $A_t=A_1+A-A'$, while corresponding water weight (per unit width) is $W = A_t \cdot \gamma$, where γ=specific water weight.

4. RESULTS. ANALYSIS AND DISCUSSION

Fig. 3 presents d_2/d_1' experimental ratios vs h/w and angle φ. This ratio appears as no angle φ dependent, and the relative experimental straight line has an equation of the form

$$d_2/d_1' = 0.29 \cdot (h/w) \qquad (2)$$

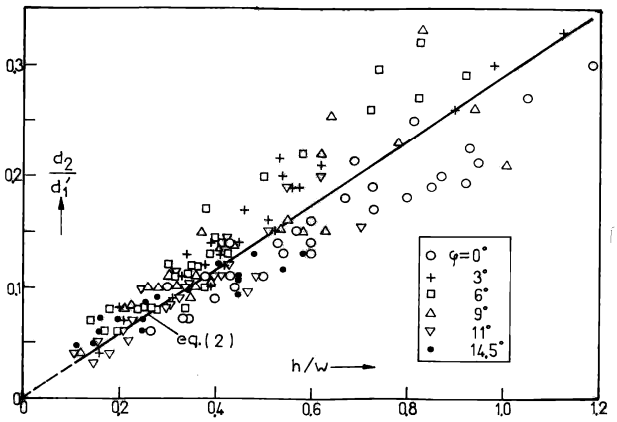


Figure 3. d_2/d_1' *vs h/w and angle φ.*

Fig. 4 presents all t/w ratios vs Fr_2 and angles φ, from which it is clear that these ratios are not systematic in relation to angles φ, since all experimental points are very close and mixed among them. t/w is decreasing when Fr_2 is increasing and the pertinent curve's equation is

$$t/w = 0.030 + 2.896 \cdot Fr_2^{-1} - 3.428 \cdot e^{-Fr_2} \qquad (3)$$

where for very large Fr_2 t/w ratios tend to the asymptotic value 0.030, while the characteristic value t/w=0.5 corresponds to $Fr_2 \cong 5.85$ -for any angle φ.

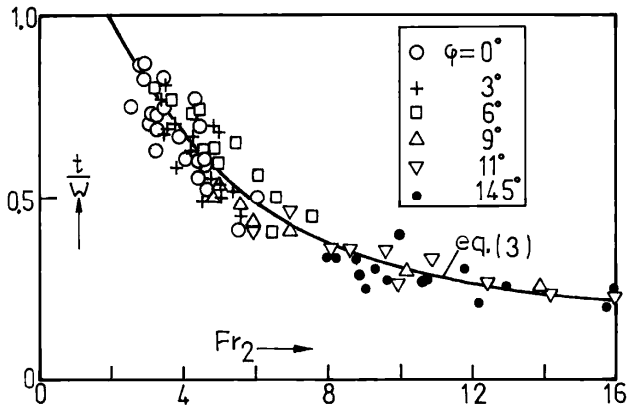


Figure 4. t/w vs Fr_2 *and angle φ.*

Fig. 5 shows the results of the experimental measurements for the average dimensionless height α/w of the air bubble. α/w are linearly increasing when Fr_2 are increasing and the curve α/w vs Fr_2 – independently of angle φ – has the empirical equation

$$\alpha/w = 0.10 \cdot (Fr_2 - 1) \tag{4}$$

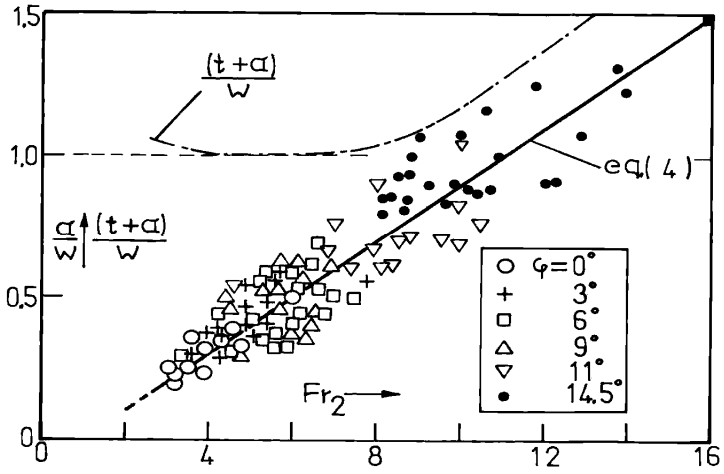


Figure 5. α/w *and* $(t+\alpha)/w$ *vs* Fr_2 *and angle* φ.

In the same diagram the total dimensionless distance from channel bottom $(t+\alpha)/w$ is also shown, where t/w is taken from the previous Fig. 4. The pertinent curve $(t+\alpha)/w$ vs Fr_2 and all angles φ has a double character since for any $(t+\alpha)/w$=const. there are two corresponding values of Fr_2 numbers. For $Fr_2 \cong 6$ the $(t+\alpha)/w$ curve has a value around unity, while the rest of this curve is over unity: Dividing eqs. (3) and (4) the t/α ratio can be taken, giving $(t/\alpha)>1$ for Fr_2 less than 6, and $(t/\alpha)<1$ for Fr_2 larger than this characteristic Fr_2 value.

Fig. 6 presents t/d_2 vs Fr_2 and angle φ, with an experimental equation (φ in degrees),

$$t/d_2 = (1.186 - 0.043 \cdot \varphi - 0.001 \cdot \varphi^2) \cdot (Fr_2 - 1), \tag{5}$$

from which $t/d_2>1$.

Fig. 7 shows the dimensionless length b/w of the air bubble: b/w is increasing when Fr_2 is increasing, and a pertiment empirical equation may be deduced,

$$b/w = 0.538 + 0.0009 \cdot Fr_2^3 \tag{6}$$

which gives $b/w \cong 1$ for Fr_2 around 8. Dividing eq. (6) to eq. (4) b/α may be received, with $b/\alpha>1$.

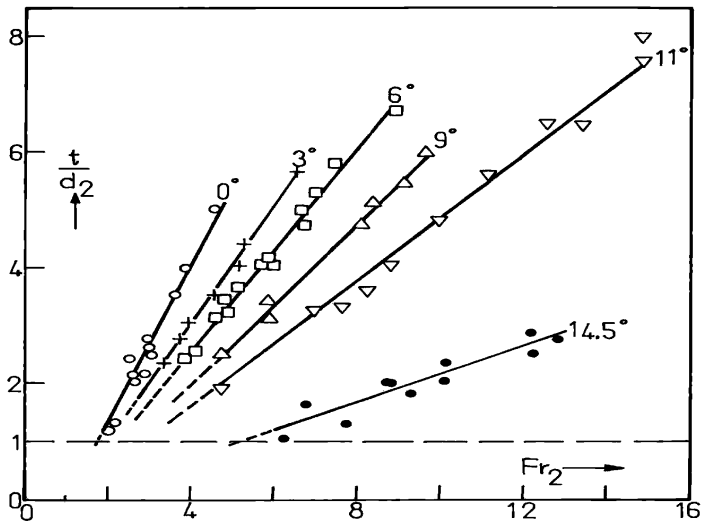


Figure 6. t/d_2 vs Fr_2 and angles φ.

In general, when Fr_2 is increasing t/w is decreasing, while α/w and b/w are increasing – independently of angle φ. This means that the air pocket is also increasing its area A' and volume $A' \cdot B$.

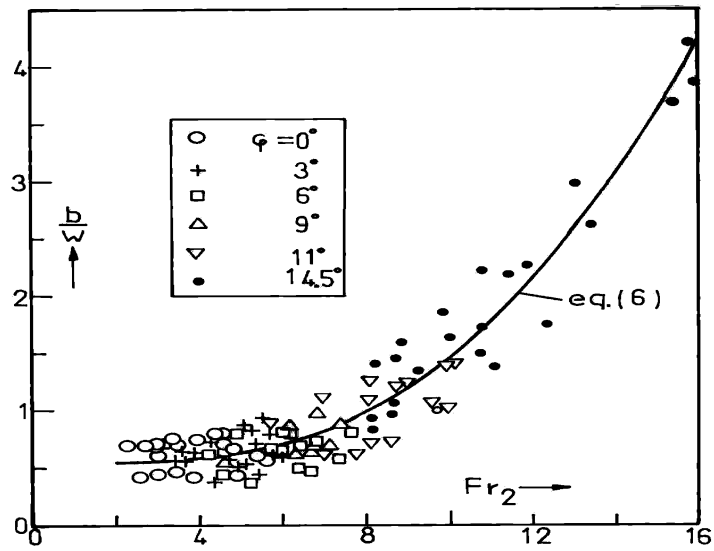


Figure 7. b/w vs Fr_2 and angles φ.

All areas A' of the bubbles were properly measured and compared to other important quantities. A suitable approximate equation, which was determined from the measurements may be given in dimensionless form,

$$A'/w^2 \cong 0.06 \cdot (Fr_2 - 1) \cdot (0.538 + 0.0009 \cdot Fr_2^3), \tag{7}$$

which for $Fr_2 \rightarrow 1$ (no supercritical flow at cross section 2) gives $A' \rightarrow 0$ (no air pochet).

In order to further determine the dimensionless area of the air pocket, $A'/L \cdot d_1'$ vs d_2/d_1' and angles φ were examined. Fig. 8 shows the experimental ratios $(100 \cdot A'/L \cdot d_1')$ vs $100 \cdot (d_2/d_1')$ for all angles φ – with $0^o \leq \varphi \leq 14.5^o$. It is clear that the experimental results are not systematic in relation to angle φ, and thus a single curve is traced through the experimental points, with a final empirical equation

$$A'/L \cdot d_1' = 0.003 + 0.001 \cdot (d_2/d_1')^{-2} + 408 \cdot e^{-100(d_2/d_1')} \tag{8}$$

All areas A' are relatively small, since they change from $(0.4\%)\cdot(L\cdot d_1')$ – for $d_2/d_1' = 0.08$ to $(1.4\%)\cdot(L\cdot d_1')$ – for $d_2/d_1' = 0.3$: In general $A'/L\cdot d_1'$ is increasing when d_2/d_1' is decreasing. Since $d_1' = w + h'$, when $w\to 0$ h' tends to a constant flow depth d along the channel, i.e. $A'/L\cdot d_1'$ tends to $A'/L\cdot d$. On the other hand when d_2/d_1' acquires very large values, $A'/L\cdot d_1'$ practically tends to zero (no air pocket), while eq. (8) may combine with eq. (2) to give another relationship between $A'/L\cdot d_1'$, h/w and all angles φ.

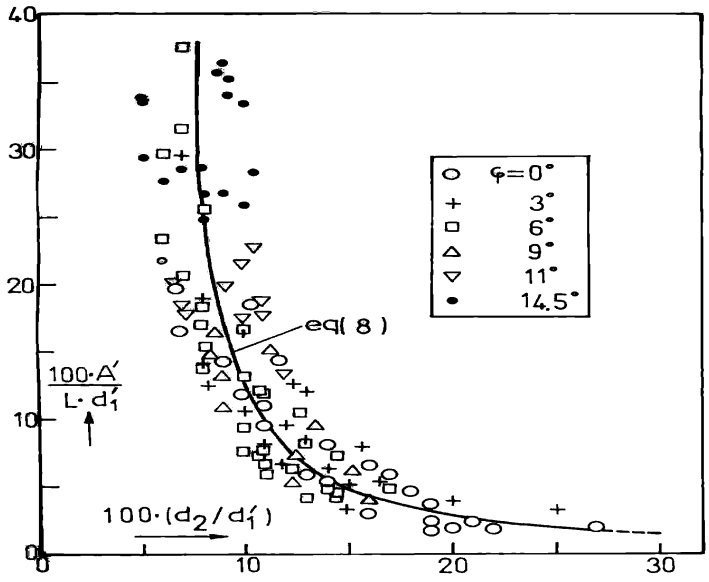


Figure 8. $100\cdot A'/L\cdot d_1'$ *vs* $100\cdot(d_2/d_1')$ *and angle φ.*

2. CONCLUSIONS

In this experimental study the air bubble's (or air pocket's) mean geometry at the water flow downstream a thin weir – within an inclined (angle φ) open channel, is investigated, analysed and discussed. Figs. 1 and 2 give the general flow characteristics, while Figs 4, 5, 6, 7 and 8, dimensionlessly present the experimental results of t/w, α/w, (t+α)/w, t/d_2, b/w and $A'/L\cdot d_1'$ respectively, vs Fr_2 and angles φ – in the range $0^o \le \varphi \le 14.5^o$. Eqs. (3), (4), (5), (6), (7) and (8) further describe the pertinent air bubble geometrical characteristics, where $t > d_2 > 0$, $(\alpha + t) > w$ and $b > \alpha$. The air bubble is generated because of the flow – without any exterior air discharge downstream the thin weir.

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EFFICIENCY OF COMBINING BIOREGENERATION OF ACTIVATED CARBONS WITH OTHER ENVIRONMENTALLY ACCEPTABLE METHODS OF REGENERATION

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Abstract: The quantitative estimation of the spontaneous bioregeneration degree of activated carbon (AC) with native biofilm is made under continuous using of the filters for drinking water purification. The possibilities of the using of other methods of adsorption capacity restoration of AC (by washing of water, of alkaline solution, of chlorophorm) are studied. It is shown that practically full restoration of AC qualities can be realized by the combination of thermal treatment under low temperature with the washing of alkaline solution. In case of choice of the AC type for long term water purification in biofiltration regime it is expediently to be oriented on the mezoporous sorbents mainly.

Key words: Biologically active carbon, Regeneration

1. INTRODUCTION

The efficiency of using activated carbons (AC) in the systems of drinking water treatment during its long-term filtration is determined, primarily, by the possibility of AC regeneration with the help of ecologically and economically acceptable methods. One of the most promising methods is bioregeneration, which involves the simultaneous processes of adsorption and biodestruction of the compounds adsorbed during the filtration of water through a bed of AC (Magic-Knezev and Van der Kooij, 2004).

However, gradual reduction of the AC bioregeneration efficiency with time implies the need of additional regeneration of the spent AC.

Reasons for incomplete bioregeneration of AC and reduction of the degree of natural bioregeneration probably are determined by the following factors. On the one hand there is a violation of the balance during the service and washing away of the biofilm by a flow of fluid. On the other hand, there is an irreversible filling of AC micropores and part of supermicropores with an adsorbed substrate and water molecules. These matters due to an increased adsorption potential in micropores are not actually removed from the microporous space under conditions of low temperature methods of AC treatment.

There are no data in the literature available on the quantitative assessment of a degree of spontaneous AC bioregeneration in processes of the long-term operation of carbon filters and factors affecting this process.

In accordance with earlier conceptions developed by the authors (Klimenko at al., 2003) under conditions of filtration through the AC with attached water microflora containing organic compounds of water, the service life of AC increases as compared with theoretical expectations. Assessment of the enhanced service life of AC according to (Klimenko at al., 2003) can be achieved by comparing equilibrium and dynamic adsorptions. The equilibrium specific adsorption is the maximum possible value of adsorption on AC in the absence of biodestruction of adsorbed compounds. Spontaneous bioregeneration of AC occurs under conditions of long-term dynamic

filtration of the solution of organic matter through AC with the native or immobilized biofilm. That is why the dynamic adsorption capacity may significantly exceed the equilibrium one. Equilibrium is established between the adsorption rate of organic matters $\delta C_{ads}/\delta\tau$ and the rate of their biodestruction during diffusion through the biofilm $\delta C_{bio}/\delta\tau$. Under conditions of the long-term filtration of tap water through AC the only source of carbon and other biogenic elements for microorganisms is organic and inorganic compounds dissolved in water. Their content is small and limited by the standards of drinking water quality. Under such conditions of the substrate shortage the biofilm formed on the AC surface contains a great amount of metabolites of the vital activity of bacteria, for example, exopolysaccharides and other extracellular polymer matters, "inert" or dead biomass that is always produced by microorganisms, and soluble products of the vital activity of microorganisms (Laspidou and Rittman, 2004; Laspidou and Rittman, 2002; Burleigh and Dawes 1967; Boylen and Ensign, 1970; Barker and Stuckey, 1999). Therefore, the degree of spontaneous bioregeneration of AC, besides other factors, is determined to a great extent by the properties of the biofilm on AC.

In accordance with conceptions developed in (Goncharuk, 2005) hydrodynamic conditions of water filtration directly affect the structure of biofilm. As was shown in paper (Viera, 1993), the density of biofilm from a pure culture increased with the rise of the shearing hydrodynamic force applied to the biofilm.

Kwok et al. (1998) believe that a very dense biofilm in the airlift bioreactor evolves due to the fact that the tearing off forces removed the low density biomass, which again grew on external layers of the carrier leaving only dense and strongly adhesive part of the biofilm in the internal layer. In paper (Melo and Pinheiro, 1984) the internal "solid" and external "loose" layers were identified in the biofilm grown in the hydrodynamic conditions similar to those in paper (Kwok et al., 1998). It was shown that the biofilm could be readily removed from external layers by the flow of liquid. In addition, it was found out that the fraction of "loose" layer in the biofilm gets smaller as the liquid flow rate gets higher. In connection with above-mentioned facts, the authors of paper (Viera, 1993) consider that the structure of biofilm on a carrier is determined by both physical factors (shearing force of the fluid flow) and biological ones. The latter are determined by the fact that bacteria strengthen the extracellular polysaccharide matrix in order to protect themselves from aggressive forces of the surrounding liquid.

In paper Rittman and Manem (1992) showed that low internal layers of the biofilm adjoining the carrier had a density (5-10) times as high as that of the external layers. They also measured the biofilm porosity, which reduced from 84-93% in the upper layers to 58-67% in the lower ones. They also measured the amount of live biomass. In the vicinity of the carrier the ratio of live biomass to the total amount of adhesive solid particles declined indicating the enrichment of the film with "dead" or "inert" biomass. The amount of live biomass constituted about 30% of the total biomass in internal layers of the biofilm, whereas its content in external layers was close to 100%.

The approaches considered of building a biofilm on the surface of carrier are also valid for activated carbon as was shown in paper (Seredynska-Sobecka at al., 2006). At the early stages of filtration and colonizing AC with microorganisms greater part of microbial production inside the filter is removed with water instead of building a fixed biofilm on AC. Later on we can probably observe "consolidation" of particles of the biofilm and its formation on the surface of carbon particles.

The specificity considered of the film structure formation, especially under conditions of a deficit of nutrient substrate, allows us to make an assumption as to reasons for exhaustion of capabilities of AC spontaneous bioregeneration and its actual termination. During intensive periodic flushing of AC the external layers of the biofilm containing "active" biomass are mostly washed away. A shortage of the substrate in the tap water subjected to filtration determines slow growth of the external layer and, consequently, gradual accumulation of denser internal layers with greater content of the inert biomass and exopolysaccharides.

Dynamic equilibrium between the components retained in the biofilm and washed out by the water flow is established during the long-term operation of an adsorption filter with its periodic

flushing by inverse flow of water. As a consequence, a certain degree of spontaneous bioregeneration of AC is ensured during the filter service owing to the prevention of excessive biomass forming on the AC surface and blocking of the adsorption filter. Additional flushing of the filter in the intensive mode by inverse flow of water makes it possible to partially wash out the inert biomass, certain extracellular polymer compounds and soluble products of vital activity of microorganisms clearing part of the AC surface for adsorption of dissolved organic matters and their subsequent biodestruction. This results in maintaining a specific degree of bioregeneration of AC and extending the service life of the AC filter with immobilized or native biofilm.

It is well known that the AC surface features hydrophobic nature; therefore it is reasonable to assume that mostly nonpolar compounds hardly soluble in water will be accumulated on the AC surface. In order to remove these matters, it would be apparently efficient to flush the spent sorbent with hydrophobic solvents, for example, chloroform. The matters having hydrophilic nature can be also accumulated on the AC. For the removal of the latter it is reasonable to try flushing with solutions of acids or alkalis.

During the process of recovering the AC adsorption capacity it is necessary to estimate the degree of liberation of its porous space. It is well known that if an adsorbent has pores with cross-section close to the Van der Waals diameter of adsorbate molecules, adsorption in such micropores is determined by the impact of the total field of all atoms of a solid body not only in the region of the internal surface of a pore, but also in the volume of the internal space of a pore. This is responsible for enhanced energy of adsorption of organic compound and hampers its reverse diffusion on the phase interface (Koganovskij at al., 1990). In addition, near the boundary of the solid body it is intrinsic for the second component – water to have a special stable state where the planes of its molecules mostly oriented in parallel direction with respect to the walls restricting the water film. The boundary films of water represent quasi-two-dimensional structures similar to those that are observed in liquid crystals (Antonchenko, 1986).

Even the temperature shift from –50°C to +75°C does not yield any significant changes in the orientation order of the water structure indicating a prevailing impact of the surface on the nearest layers of water as compared with disordering impact of temperature.

As was shown in paper (Alcaňiz-Monge at al., 2002) on the basis of investigations of the water state in AC micropores, the water in the microporous structure has the density typical for the structure in solid phase equal to about 0.93 g/cm^3. During adsorption of organic matters from a multicomponent aqueous solution in the region, which is far from saturation, the adsorption layer contains both molecules of the organic component and molecules of water. Micropores of AC can be accessible for adsorption of water molecules and inaccessible for large molecules of organic matterss referred primarily to natural organic compounds (Kunio Ebie at al, 2001).

Thus, it is conceivable that for complete liberation of the adsorption space during non-thermal regeneration of AC it is sufficient to create conditions for the removal of structured water from supermicro- and microporous space of AC. On the other hand, for complete recovery of the initial adsorption properties of AC it is necessary to free the carbon from the attached biofilm. It is well known that the attached microorganisms can be removed from the AC surface by exposure to ultrasound (Magic-Knezev and Van der Kooij, 2004). This technology is frequently used for detachment and quantitative assessment of biofilms and microorganisms associated into conglomerates (Mathieu at al., 1993; Van der Kooij at al., 1995). Ultrasound treatment can also inactivate microorganisms (Hua and Thompson, 2000). These circumstances contribute to better removal of inactivated microorganisms and recovery of the initial properties of AC.

Hence, the purpose of the present study was to establish the degree of recovery of adsorption properties of AC taken from adsorption filters that were used for the drinking water treatment during a long time as a result of the spontaneous bioregeneration and improvement of its efficiency by other methods. As an object of investigations we used samples of carbon from an adsorption filter utilized in tap water treatment. The filter was in service during 3.5 years.

2. MATERIALS AND METHODS

The analysis of averaged data on the quality of drinking water filtered through AC showed that the content of total organic carbon (TOC) in the water amounted to 6.4-8.0 mg/dm^3, permanganate oxidability (PO) was at the level of 4.4-5.0 mgO/dm^3, water color index was 14-20 degrees. After the adsorption filters the water quality complied with requirements of the normative documents in terms of the following indicators: TOC $\leq$ 3 mg/dm^3, PO $\leq$ 4 mgO/dm^3, and color index $\leq$ 15 degrees.

Initial fresh activated carbon KAU used in the adsorption filters had the following structural and sorption characteristics: effective specific surface area of mesopores S_{me} = 630 m^2/g, adsorption space volume V_a=0.39 cm^3/g; volume of micropores V_{mi}= 0.12 cm^3/g. The AC KAU was prepared from apricot stones as follows: (1) treatment of crushed apricot stones with a hot solution of strong alkali; (2) washing out with water to bring pH of solution down to 10; (3) treatment of crushed apricot stones with hot hydrochloric acid; (4) washing with water to bring pH of solution up to 4; (5) carbonization at 350-700°C; (6) activation with steam at 800-850°C.

Height of the filter bed was 100 cm, mass of carbon in the filter was 32 kg, water filtration rate was 3-4 m/h, and filter service time was 3.5 years.

2.1. Analytical procedures.

Control of the plant operation was conducted by analyzing water samples taken from layers located at different heights of the KAU bed for determining the content of fulvic acids (COD, TOC). COD of solutions was determined by the standardized procedure with potassium bichromate. Total organic carbon was determined by the method of catalytic burning at 800°C using a Shimadzu TOC-V CSN instrument. Thermogravimetric curves were obtained by using a Derivatograf Q-1000.

2.2. Determination of structural and sorption characteristics of AC.

Estimation of the structural and adsorption characteristics of AC after spontaneous bioregeneration and additional recovery of the capacity by other methods was conducted by measuring adsorption of p-chloroaniline on samples of carbons taken from the upper part of the adsorption filter at the height of 1 cm (sample N. 1) and from the lower part at the height of about 100 cm (sample N. 2).

This study dealt with estimation of the following structural and sorption characteristics:

- Surface area of mesopores (S_{me})
- Total adsorption volume of pores (V_a)
- Volume of micropores (V_{mi})

The method of measuring adsorption from aqueous solution of the standardized p-chloroaniline (Koganovskij at al., 1990) makes it possible to estimate with sufficient accuracy the specific surface area that coincides with the specific surface area determined by the adsorption of vapors of n-hexane and by the thermal desorption of argon. P-chloroaniline was used as a standardized adsorbate due to the following reasons: 1) it has a low solubility in water; 2) aromatic ring of p-chloroaniline is oriented at the phase interface in such way that its plane is parallel to the carbon surface (such orientation of a molecule corresponds to Van der Waals' area of the molecule projection of 5 nm^2).

Estimation of the structural and adsorption characteristics of AC was conducted by the t-method also with the use of p-chloroaniline. The competence of p-chloraniline application for the estimation of textural characteristics of the AC is based in (Koganovskij at al., 1990). The application of t-method is valid under the condition when the isotherm under test and the standard one are similar. The condition is provided at the identical chemical nature of surface of both the standard sorbent and that under test. The adsorption isotherm of standard solute (p-chloraniline) was used. This

isotherm was then compared with the adsorption isotherm of the same adsorbate on nonporous graphite black (standard sorbent). As expected, the mean statistical thickness of the adsorbate layer in this case is the same as in the case of AC under test at the equal relative equilibrium concentrations (C/C_s). in such a case, there is a unified curve $t=f(\tilde{N}/\tilde{N}_s)$, which should be calculated from the isotherm of standard solute (p-chloraniline), on graphite black for all the identical sorbents:

$$t = \frac{a_i V^*}{S_a}, \quad (1)$$

where a_i is the equilibrium specific adsorption (mol/g), V^* is the molar volume of the adsorbate (cm^3/mol), and S_a is the specific surface of graphite black (m^2/g). The a_iV^* vs. C/C_s isotherm was determined by use of the adsorbent with unknown specific surface. These values have then been plotted as a_iV^* vs. t:

If the adsorbent has micropores with heightened adsorptive potential as compared to other types of pores, the volume of adsorbate can be represented in the form

$$a_iV^* = V_{mi} + S_{me}t; \quad (2)$$

where a_iV^* is the total volume of adsorbed solute in the porous structure of AC, V_{mi} is the volume of solute which is adsorbed with heightened adsorptive energy on micropores, $S_{me}t$ is the volume of solute which its adsorbed with the same selectivity as that on graphite black, and S_{me} is the specific surface of AC mesopores.

As $t = a_bV^*=S_{a,b}$; we can write

$$a_iV^* = V_{mi} + S_{me}\frac{a_{i,b}V^*}{S_{a,b}} \quad (3)$$

where a_b is the specific adsorption on graphite black and $S_{a,b}$ is the known specific surface of graphite black. The form of Eq. (7) enables one to calculate the adsorbate amount, which is adsorbed as a single component into the micropores ($V_{mi} = a_{mi}V^*$). Micropores with a heightened adsorptive potential are filled up first at small equilibrium concentrations. Because $a_{mi}V^*$ is constant, and the growth of adsorptive volume of AC takes place owing to adsorption on a surface of broader pores, in these pores adsorption selectivity of adsorbed substance equals that of the nonporous graphite black. Thus, the isotherm reached according to Eq. (7) is transformed into a straight line, which intercepts the ordinate axis at the volume of adsorbate that is placed in the true micropores. The S_{me} value is easily determined from the slope angle of a straight line at the known surface of the standard sorbent sample ($S_{a,b}$).

Effective specific surface area S_{ef} and volume of pores of KAU carbon samples were estimated by the BET method using adsorption of p-chloroaniline from the aqueous solution and nitrogen on a vacuum-adsorption unit Quantochrome Autosorb 6.

2.3. Carbon treatment methods.

Boiling in water. 1 g of carbon was boiled during 30 minutes.

Heating at temperature +190°C. 1 g of dried carbon was heated at temperature 190°C during 2 hours.

Ultrasonic treatment. 1 g of AC was subjected to ultrasonic treatment during 1 or 3 minutes at frequency 40 kHz.

Treatment with the sodium hydroxide solution. 1 g of dried carbon was poured over successively with three portions of 4% NaOH solution having volumes 20, 20, and 10 cm^3 respectively and each portion was shaken on a shaking apparatus during 1 hour.

Treatment with chloroform. 1 g of dried carbon was poured over successively with three portions of chloroform having volumes 20, 20, and 10 cm^3 respectively and each portion was shaken on a shaking apparatus during 1 hour.

3. RESULTS AND DISCUSSION

To estimate the adsorption capacity of spent carbons in terms of natural organic matters, an adsorption isotherm of their main component in water (fulvic acids) was measured in TOC units (see Fig. 1).

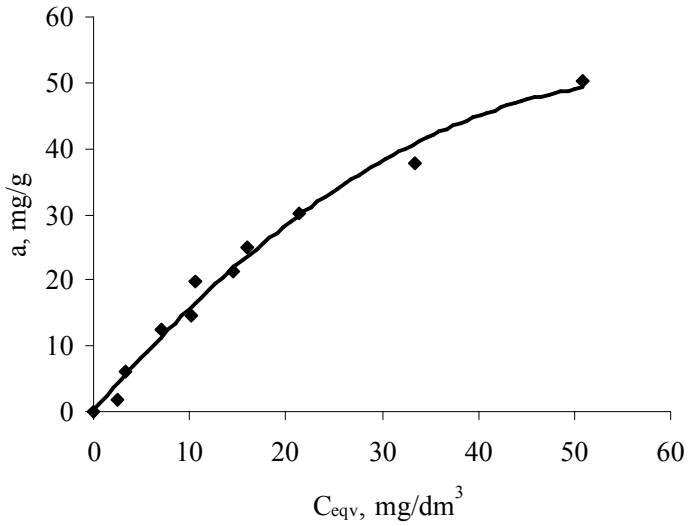


Fig.1 Isotherm of fulvic acids on AC KAU

The equilibrium value of adsorption of organic matters (by the example of fulvic acids) at concentration 2-3 mg/dm^3 in terms of total organic carbon amounted to 30 mg/g, which corresponds to ~ 54% of the maximum adsorption capacity equal to 55.5 mg TOC/g. Hence, it can be tentatively seen that approximately 400 m^2/g of 630 m^2/g of the effective specific surface area of mesopores (S_{me}) can be occupied due to equilibrium adsorption assuming that the entire surface area S_{me} is accessible for adsorption of organic compounds from the tap water. Thus, the adsorption space of carbon KAU accessible for fulvic acids is occupied by ~ 54%, while ~ 46% remain free. This value should be taken into account in estimating the degree of AC bioregeneration later on. In this case, one should keep in mind that during the filtration of solution equilibrium conditions are not achieved. At filtration rate of 3-4 m/h the time of AC contact with the solution amounts to approximately 20 minutes, while the settling time of adsorption equilibrium during adsorption of fulvic acids is achieved within 14 days. Approximate amount of adsorbed organic matters in terms of TOC after 200 m^3 of water passed through carbon KAU amounts to approximately 1400 g for the entire period of service. According to the adsorption isotherm the equilibrium quantity of organic matter that can be adsorbed amounts to 960 g. Comparison of these values indicates that biodestruction processes do proceed on AC, especially with due regard for the fact that equilibrium condition is not achieved during the filtration process.

To estimate the degree of natural bioregeneration of AC during the service, we estimated the structural and sorption characteristics of AC samples 1 and 2. The data obtained are presented in Table 1.

Table 1. Restoration of AC adsorption characteristic after different type of treatment.

Restoration type	Degree of AC adsorption characteristic restoration					
	S_{me}		V_a		V_{mi}	
	m^2/g	% from initial	cm^3/g	% from initial	cm^3/g	% from initial
Initial AC	630	-	0.39	-	0.12	-
AC after bioregeneration						
Sample 1	533	84.7	0.23	58.9	0.02	16.6
Sample 2	520	82.6	0.20	51.2	0	0
Boiling:						
Sample 1	630	100.0	0.26	66.7	0	0
Sample 2	170	27.0	0.07	17.9	0	0
Heating:						
Sample 1	603	95.9	0.33	84.6	0.01	8.3
Sample 2	357	56.8	0.22	56.4	0.01	8.3
Ultrasound:						
Sample 1	458	72.8	0.21	53.8	0.04	33.0
Sample 2	566	90.0	0.20	51.3	0.045	37.5
Washing by water						
Sample 1	375	59.6	0.16	41.0	0.02	16.7
Sample 2	-	-	0.25	64.1	0.01	8.3
Washing by alkali solution:						
Sample 1	377	60.0	0.07	17.9	0	0
Sample 2	609	96.8	0.25	64.1	0	0
Washing by chloroform:						
Sample 1	522	83.0	0.17	43.6	0	0
Sample 2	367	58.3	0.13	33.3	0	0

As can be seen from Table 1, the recovery degree of the effective specific surface area of mesopores for carbon KAU resulting from spontaneous bioregeneration amounted to 83-85%, while the degree of liberation V_a constituted 51-59%. Thus, the value of S_{me} after spontaneous bioregeneration of KAU amounted to 530 m^2/g, i.e., only 130 m^2/g remained occupied (against ~ 400 m^2/g for the case where bioregeneration was not present). Micropores only slightly recovered in the upper part of the adsorption column and were absolutely not involved in the process of natural bioregeneration in its lower part.

The reasons of irreversible filling of part of pores of the adsorption volume and micropores may include, on the one hand, blocking of the inlet to pores by microorganisms and products of their vital activity and, on the other hand, filling of micro- and part of supermicropores by the water molecules having the special structure and organic molecules of small size that remain there permanently. In order to check this assumption, carbon samples No. 1 and No. 2 were subjected to the following kinds of treatment: boiling in water, heating at temperature +190°C, ultrasound treatment, extraction treatment with the solution of sodium hydroxide and chloroform. After the treatment carbon samples were used to estimate their structural and adsorption characteristics in terms of p-chloroaniline adsorption. The results of experiments are also presented in Table 1 also.

In addition, after the treatment with water, the total content of organic matters in COD units was determined in wash-outs from the samples. The results of experiments are presented in Table 2.

Table 2. The content of organic matters in wash-outs from AC after different methods of treatment

Types of treatment	Content of organic matters in water outwash, mgO/l
Boiling:	
Sample 1	146.2
Sample 2	17.2
Washing by alkali solution:	

Sample 1	182.2
Sample 2	79.2
Ultrasonic:	
Sample 1 (1 min)	39.7
Sample 1 (3 min)	95.6

The analysis of data presented in Tables 1-2 allows us to come to the following conclusions. As a result of the carbon treatment with water at temperature 100°C, the effective specific surface area of mesopores was actually fully recovered in the carbon samples from the upper part of the column, unlike the case of natural bioregeneration, when the degree of recovery was 82-84%. The degree of liberation of the adsorption space in the upper part of the column also increased to 66%, while in the lower part it essentially decreased. This also correlates with the data specifying the content of organic matters in wash-outs from AC samples taken in the upper and lower parts of adsorption column. One can assume that the composition of biofilm in various parts of the column is different due to different amounts of the substrate supplied. This results in inferior solubility of products of vital activity of microorganisms in the lower part of the column at t =100°C as compared with that in the upper part. Different quantitative and qualitative composition of the biofilm on AC after the service during one year was established in paper (Persson, 2005) where it was shown that the average amount of biomass in the lower layers of sorbent was approximately four times less than in the upper layers. Earlier we showed (Klymenko, 2005) that under conditions of the substrate deficit the biofilm formed on the AC surface contained a great number of exopolysaccharides of different chemical origin, hydrocarbons, proteins, and live and dead microorganisms.

Though heating of carbon samples to temperature 190oC enhances the efficiency of recovery of adsorption characteristics of AC in the upper part of the column, in fact, it does not result in liberation of micropores. In order to check the assumption on filling carbon micropores with water, thermogravimetric curves were measured on a Derivatograf Q-1000 (Fig. 2).

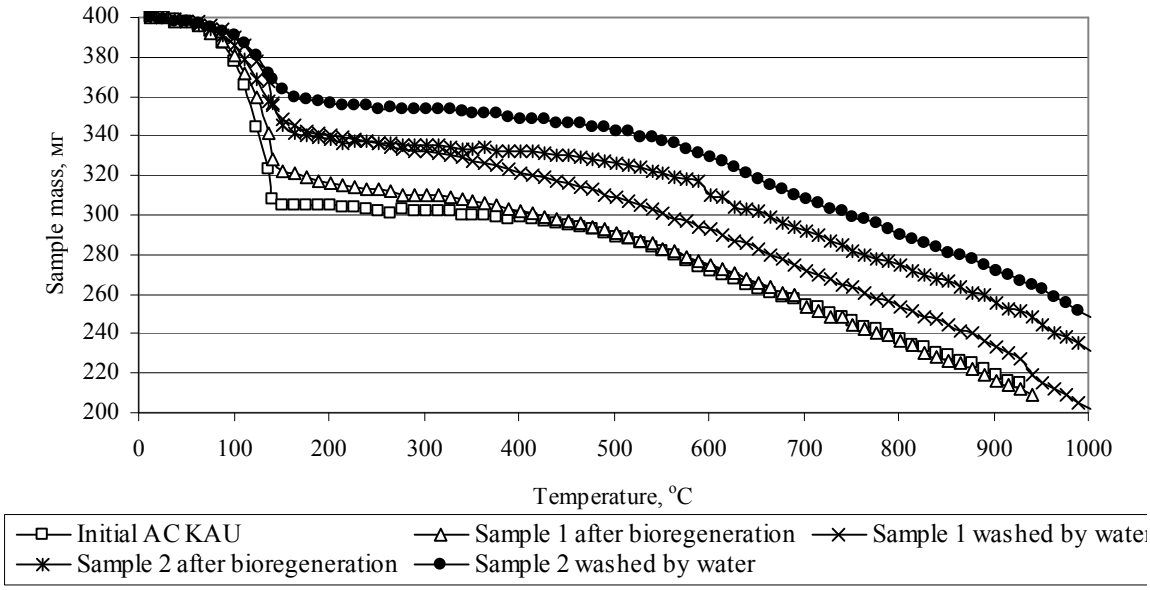


Fig 2. Termogravimetrical curves of investigated AC sample (samples weight 400 g)

Since the samples were subjected to preliminary air-drying, the water in liquid state should not be present at their surface, that is why up to the heating temperature of 100ºC all carbon samples hardly lose their mass. In the temperature range 120-140°C all carbon samples actively lose their mass that according to our guess is determined by the liberation of AC porous space from the water adsorbed in it. This is corroborated by data in Table 3.

As can be seen from Table 3, the specific loss of sample mass during heating to temperature 140ºC well correlates with changes of the adsorption volume of liberated pores. This confirms our

assumption about filling of the volume of AC porous space with a certain amount of the water in a special state close to solid one. It can be also seen that in the majority of cases the difference between the adsorption volume of liberated pores and the volume of occupied micropores is equal to specific mass loss by a sample during its heating to 140ºC. This can be explained by the fact that the microporous space of carbon is not liberated during heating even to 190°C as was noted above.

Table 3. Comparison of AC specific mass loss under heating and pore adsorption volume

Sample	Loss of sample mass at heating to 140°C, mg	Specific loss of sample mass at heating to 140°C, g/g	Adsorption volume of non-occupied pores of AC, cm^3/g	Micropores volume, cm^3/g
Initial KAU	105.6	0.26	0.39	0.12
Sample 1 after bioregeneration	87.2	0.22	0.23	0.02
Sample 1 washed by water	60.0	0.15	0.16	0.02
Sample 2 after bioregeneration	61.6	0.15	0.20	0
Sample 2 washed by water	44.0	0.11	0.25	0.01

Recovery of S_{me} and V_a in the upper and lower parts of adsorption column correlates with the content of organic matters in water wash-outs from these AC samples. And only ultrasound treatment of AC samples during 1 min allows us to liberate up to 37% of the microporous space (Table 2). Thus, the resultant data favor the assumption that the main reason of deterioration of AC adsorption characteristics is blocking of access to supermicro- and microporous volume of AC by microorganisms and products of their vital activity. However, it should be noted that ultrasound treatment results in quite significant losses of carbon due to its disintegration to pulverized state. They constitute 12.5% and 10.4% for the times of treatment 3 min and 1 min, respectively. Therefore, in order to use ultrasound for regeneration of AC after biofiltration, it would be necessary in future to determine the rational conditions of its application.

Additional flushing with water and sodium hydroxide under static conditions only reduces the degree of liberation S_{me} of the carbon sampled in the upper part of the adsorption column, but at the same time it increases S_{me} in the lower part of the column. In the case of using chloroform the reverse is true: we can observe the reduction of S_{me} in the samples of carbon taken from the bottom layer. These fluctuations correlate with V_a variations for carbon KAU. Data specifying variations of the degree of recovery of AC adsorption characteristics in the upper and lower parts of the column during the AC treatment with hot water or during the heating and treatment with sodium peroxide are also noteworthy. In the first case, the degree of recovery of adsorption characteristics achieves 96-100% in the upper part of the column, while in the second case this value is 97% but in the bottom part of the column. This is, probably, related to different conditions of forming biofilm in the upper and lower parts of the adsorption column as was shown in paper (Persson at al., 2005).

Guesses made on the efficiency of AC bioregeneration progress and variation of the degree of liberation of AC surface during its flushing would have been incompetent without objective verification of the measured values by independent methods. To this end, we conducted a comparison of the effective specific surface area S_{ef} and the volume of pores for samples of carbon KAU determined by the BET method using adsorption of p-chloroaniline from the aqueous solution and by nitrogen adsorption on the vacuum adsorption plant. Comparison of the results obtained is presented in Table 4.

As it is seen from Table 4 difference between S_{ef} value, which was determined by different methods, for tested AC sampled does not maintain ±5%.

Table 4. Comparison of structural-sorption characteristics AC KAU measured by adsorption of p-chloraniline from water solution and nitrogen adsorption

Activated carbon	S_{BET} by nitrogen, m2/g	S_{BET} by p-chloraniline, m^2/g		Divergence between S_{BET} value, %	
		$0.05<p/p_S<0.35$	Total interval		
Initial KAU	890.4	876.5	903.8	– 1.56	+1.50
Sample 1 after bioregeneration	796.6	787.0	758.4	– 1.21	– 4.80

4. CONCLUSIONS

1. Variations of the structural and adsorption characteristics of activated carbon were investigated for the carbon used in the drinking water treatment for the period of 3.5 years.
2. It was established that as the result of biological activity of biofilm formed during the filter service, the processes of spontaneous bioregeneration occurred on the AC surface. In this case, the degree of recovery of the effective specific surface area of AC constituted 83-85%, while that of the adsorption volume of pores was 51-59%. In fact, AC micropores were not actually liberated during spontaneous bioregeneration.
3. During the carbon treatment with water at temperature 100°C the effective specific surface area of AC in the upper part of adsorption column recovered practically completely, while the degree of liberation of adsorption space increased to 66%. In the lower part of the column these figures were essentially inferior with respect to the case of spontaneous bioregeneration. This is determined by different compositions of the biofilm in the upper and lower parts of adsorption filter.
4. It was shown that part of the AC porous space was irreversibly filled with water being in a special state close to solid. Heating of carbon to temperature 140°C resulted in liberation of the adsorption space of AC pores and a certain degree of liberation of micropores (up to 8.3%). Ultrasound treatment of carbon made it possible to liberate up to 33-37% of the volume of micropores; however, this treatment is accompanied by the loss of carbon due to its disintegration to the pulverized state.
5. Essentially complete recovery of adsorption properties of AC could be achieved by combining its spontaneous bioregeneration during long-term service with low-temperature thermal treatment at temperatures 120-140°C and flushing with an alkali solution in the lower part of adsorption filter.

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INTEGRATION OF SATELLITE REMOTE SENSING, GIS, MODELLING AND WIRELESS-SENSOR NETWORK FOR MONITORING AND DETERMINING IRRIGATION DEMAND IN CYPRUS

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Abstract: This paper presents the methodology and results of a research project that integrates technological tools for developing a complete system for monitoring and determining irrigation demand on a systematic basis in Cyprus such as multi-spectral satellite remote images, GIS, evapotranspiration models and micro-sensor technology. The main aim is to estimate evapotranspiration in Cyprus and furthermore to undertake the required measures for an effective irrigation water management for the whole Island. Evapotranspiration (ET) is difficult to be determined since it combines various meteorological and field parameters while in literature quite many different models for estimating ET are found. A wireless sensor network will act as a monitoring tool for providing measurements of the necessary required parameters such as meteorological, climatic data and other auxiliary parameters required to be used in the irrigation models for determining the irrigation demand. Reflectance will be determined directly from the satellite images. GIS will be used to create a database for all the collected data.

Available methods found in the literature for determining irrigation demand using satellite remotely sensed and other auxiliary data are presented. Afterwards, the methodology is briefly described, and some results of the FAO Penman-Monteith method by combining satellite images (ASTER and Landsat 7 ETM+) and meteorological station data are shown.

Key words: evapotranspiration, irrigation, remote sensing, wireless sensors.

1. INTRODUCTION

The agricultural sector is one of the major consumers of water, consuming more than 70% of the world's fresh diverted water use from rivers and groundwater. Thus the use of irrigation water plays a significant part in increasing land productivity (Ahmad et al 2008; Hongjun et al 2008). One of the main components of the water balance is the water loss via evapotranspiration, which is proportional – among others – to the vegetation cover of a region. Remote sensing is a very useful research tool to identify irrigated area and classify the vegetation coverage. The objective of this research project is to describe and apply a procedure for monitoring and improving the performance of on-demand irrigation networks, based on the integration of remote sensing techniques and simulation modeling of irrigation water in Cyprus, which is facing a severe drought the last five years. Multi-spectral satellite images are used to infer crop potential evapotranspiration, which is the main input for water balance simulations. The need for estimating ET in Cyprus is imposed in order to determine the exact quantity of irrigated water needed for each specific crops. The over use of water for irrigation has resulted in eliminating the water resources in the whole island. The determination of ET for irrigation purposes will be used as a vital tool for supporting the decision-making process in the management of water resources, on a technocratic level, and on the other hand will have a positive effect on the rest of water resources of Cyprus.

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2. LITERATURE REVIEW

Evapotranspiration is referred as the combination of two different processes where in the first water is lost from the soil and plant surfaces by evaporation and in the second from the crop by transpiration (FAO 1998). The actual estimation and calculation of evapotranspiration, is a basic and crucial need for climate studies, weather forecasts, hydrological surveys, ecological monitoring, and water resource management (Hoedjes et al 2008). Evapotranspiration, as mentioned, constitute a main component of the hydrological cycle and its estimation demands auxiliary hydro-meteorological data. Many formulas have been developed by scientists to calculate evapotranspiration.

In the past decades the estimation of evapotranspiration combining conventional meteorological ground measurements with remotely-sensed data has been widely studied and several methods have been developed for this purpose. Reference evapotranspiration values, can be calculated by measuring weather conditions and typical reference crops using specialized instruments the lysimeters. Today, it has been recognized that estimating evapotranspiration is quite accurately, by using both equations and remote sensing data. Climatic data, e.g. solar radiation, air temperature, relative humidity and wind run, can also be used in these estimations remote sensing data. These kind of data, from different remote sensing satellites, is also a useful tool to provide information over various temporal and spatial scales (e.g. NOAA (Gieske and Wouter, 2005), Landsat/ETM+ (Hafeez et al. 2002; Sobrino et al 2004; Stathopoulou et al. 2007; Qin and Karnieli 2001), Aster (French et al. 2002). Remote sensing has several advantages instead of field measurements since remote sensing covers a large area and information can be spatially represented in maps through Geographic Information Systems (G.I.S.), revealing spatial characteristics that are often not apparent when information is provided other form e.g. tables, text (Bastiaanssen and Bos 1999). Different methods and models have been developed by scientist, to use this information from remote sensing in surface flux estimation schemes. It is difficult to classify all these methods, since their complexity. These models are based either on empirical or physically methods. Nevertheless, three main and general categories can be identifying (Courault et al. 2005): a) empirical direct methods, b) residual methods and c) indirect methods

The FAO Penman-Monteith method, which was derived from the Penman-Monteith method, has recently been recommended as the sole standard method. It is a method with strong likelihood of correctly predicting ETo in a wide range of locations and climates. In brief this methodology consists of estimating crop evapotranspiration (Etc) for a crop canopy using a reference evapotranspiration (Etr) and a crop coefficient (Kc). The reference evapotranspiration is calculated over a grass under optimum soil moisture conditions with a constant value of the surface canopy resistance considering then the grass as a single big leaf (FAO 1998). Although in reality the surface resistance can vary according to the day, the weather conditions and also the determination of crop coefficients is also debatable.

3. RESOURCES AND METHODOLOGY

3.1 Case study

The Paphos district area has been selected as the project case study (see Figure 1). The irrigation supply of the Paphos area is provided from: Asprokremmos Dam reservoir, 24 boreholes drilled in the gravel aquifers of the major river beds Dhiarizos, Xeropotamos and Ezousas rivers and the coastal calcarenite aquifer. The irrigated area near the Asprokremmos dam will be used as a pilot study in this project.



Figure 1: Landsat TM image of Cyprus: Area of interest is marked

3.2 Resources

For estimating evapotranspiration using FAO Penman-Monteith method, auxiliary meteorological data where used. Air temperature, atmospheric pressure, wind velocity and other data where collected from an automatic weather station, located nearby our case study i.e. at the Paphos International Airport (see Figure 2). The necessary meteorological data will be the base for validating the data collected from the wireless sensors network which is applied in the field of interest.

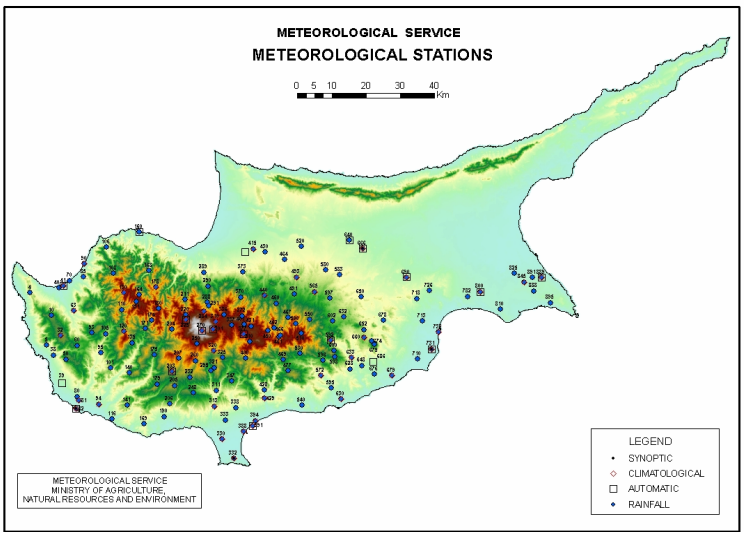


Figure 2: Map of the meteorological stations in Cyprus.

Furthermore, multi-spectral, ASTER and Landsat 7 ETM+ satellite images, for the past two years have being used. For the pre-processing and processing of the satellite images the ERDAS IMAGINE software was used. Sun-photometer and spectro-radiometric measurements were also taken in situ (see Figure 3).

More specific a five-channel handheld Microtops II sun-photometer (Solar Light Co., http://www.solar.com/) operated from 380 to 1020 nm is used. The sun-photometer will be used to derive the AOT and the water vapour concentrations during the satellite overpass so as to assist the effective removal of atmospheric effects. The GER1500 field spectro-radiometer is also used in this research project (http://www.spectravista.com). The GER1500 field spectro-radiometers is a light-weight with high performance spectral reflectance's accuracy, covering the ultraviolet, visible and near-infrared wavelengths from 350 nm to 1050 nm. GER 1500 uses a diffraction grating with a silicon diode array which has 512 discrete detectors and provides the capability to read 512 spectral

bands. The instrument is very rapid scanning, acquiring spectra in milliseconds. The spectro-radiometer is used to derive the spectral reflectance values of various crops in the selected agricultural fields and to support the derivation of reflectance values from medium and high resolution images.



Figure 3: (a) In-situ spectro-radiometric and (b) sun-photometer measurements acquired at the area of interest in Paphos.

The Wireless Sensor Network (WSN) is consisting of a number of approximately twenty wireless nodes placed in our case study (see Figure 4). The WSN acts as a wide area distributed data collection system deployed to collect and reliably transmit soil and air environmental data to a remote base-station hosted at Cyprus University of Technology (at the Remote Sensing Laboratory) (Hadjimitsis et al. 2008). The micro-sensors are deployed using ad-hoc multi-hop communication protocol and transmit their data to a gateway which is responsible to collect, save and forward them to a remote database through a GPRS connection. The solar powered gateway is equipped with various meteorology sensors required to assist the indeed research project such as rain, wind, barometric pressure, temperature etc, which give additional information to the system. The gateway also hosts a GPS sensor for identifying the exact position of the WSN an event-driven smart camera for acquiring real-time pictures of the area and also a GPRS modem for communicating with the remote server. In the future a multi-parameter decision system running on the remote server would be able to process the sensor data and produce valuable information about watering different vegetables and create early notifications and suggestions which are then distributed to farmers and water management authorities. The system would be able to process multi parameter data collected from different sensors such as soil moisture; soil temperature leaves wetness and temperature, humidity, rainfall, wind speed and direction and ambient light.

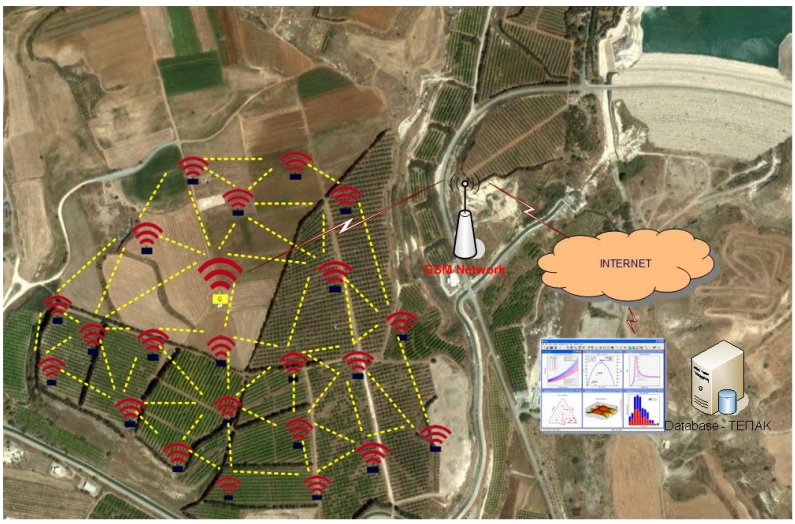


Figure 4: The Wireless Sensor Network

3.3 Methodology

The overall methodology is shown in Figure 5. Further to the data collection, the project consists pre-processing and post-processing steps.
Pre-processing and processing of the satellite images, includes geometric correction (second order polynomial transformation), georeferenced of the image using ground control points (WGS 84 / UTM), radiometric correction in order to obtain the same comparable units for each image, and finally atmospheric correction by employing the darkest pixel method (histogram minimum method), to get the true reflectance of each pixel of the image (Hadjimitsis et al. 2003). Finally Liang's equation (Liang 2000) for deriving albedo was employed in this research for ASTER images. For Landsat images, weighted coefficients and irradiance were used for calculating albedo for each image (Zoran and Stefan 2006).

After preprocessing, the satellite data is ready for extracting the necessary inputs for the irrigation demand model in our case study. According to FAO Penman-Monteith (FAO 1998) method, the reference evapotranspiration can be calculated from the following equation:

$$ETo=\frac{0.408\Delta(R_n-G)+\gamma\frac{900}{T+273}u_2(e_s-e_a)}{\Delta+\gamma(1+0.34u_2)} \quad (1)$$

where:

ETo reference evapotranspiration [mm day^{-1}],
Rn net radiation at the crop surface [MJ m^{-2} day^{-1}],
G soil heat flux density [MJ m^{-2} day^{-1}],
T air temperature at 2 m height [°C],
u_2 wind speed at 2 m height [m s^{-1}],
es saturation vapour pressure [kPa],
ea actual vapour pressure [kPa],
es-ea saturation vapour pressure deficit [kPa],
Δ slope vapour pressure curve [kPa $°C^{-1}$],
γ psychrometric constant [kPa $°C^{-1}$].

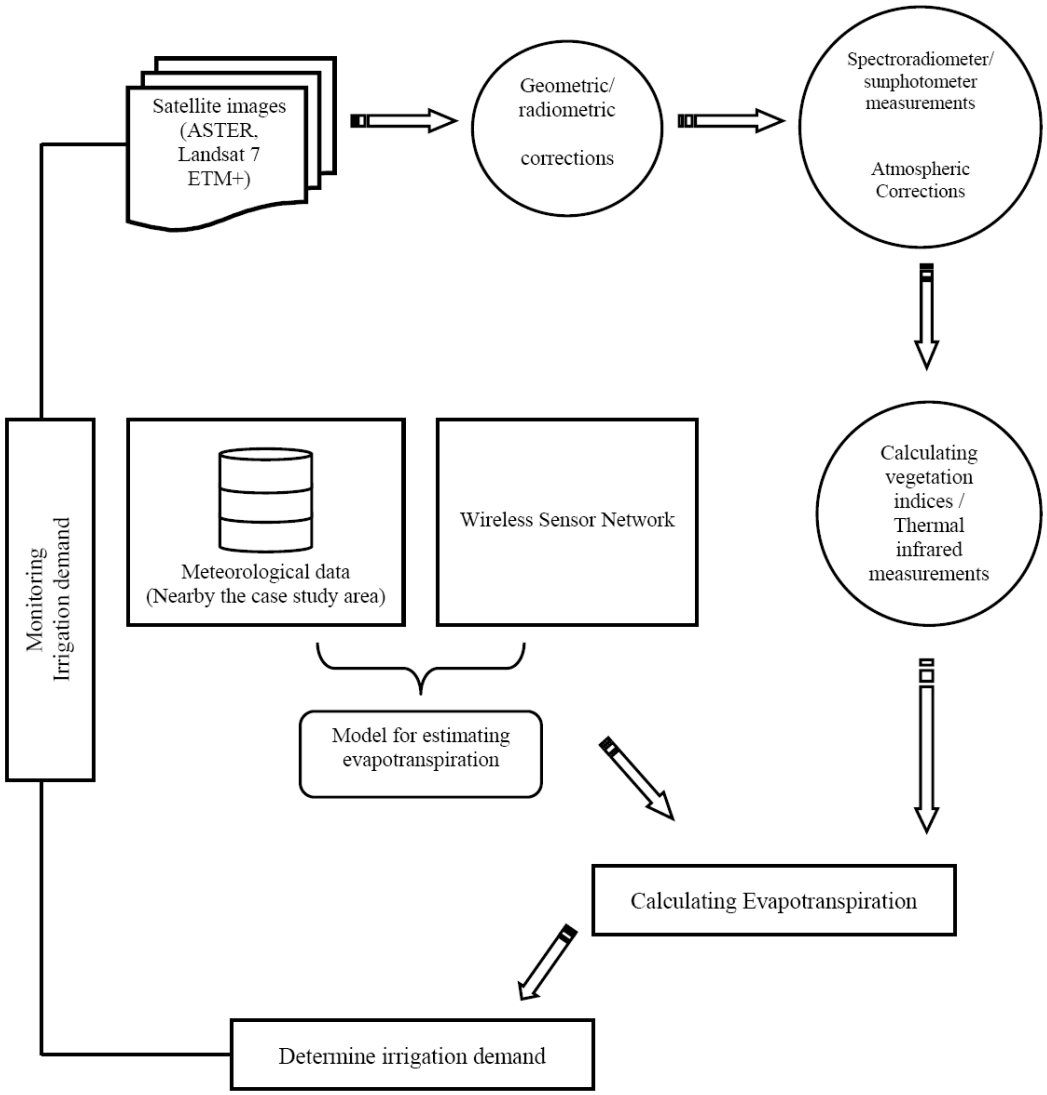


Figure 5: Project Methodology

A4. RESULTS

FAO Penman-Monteith method, a well accepted method for estimating evapotranspiration, has been applied. The final results have been tabulated and shown in *Table 1* and graphically in *Figure 6.* Atmospheric correction has been found to be very critical for estimating ETo. Approximately an error of 5% less than the calculated 'Eto' mean was recorded due to atmospheric effects. *Figure 7* shows how a GIS can assist the presentation of evapotranspiration.

Table 1: Results of FAO Penman-Monteith application for estimating Etc

No	Date of Satellite image	Satellite	ETo mean	ETo max	ETo min
1	17-12-2007	ASTER	2.33	2.36	2.29
2	24-12-2007	ASTER	2.34	2.38	2.27
3	26-02-2008	ASTER	2.25	2.27	2.22
4	28-07-2008	LANDSAT 7 ETM+	2.78	2.85	2.74
5	13-08-2008	LANDSAT 7 ETM+	2.84	2.88	2.77
6	29-08-2008	LANDSAT 7 ETM+	3.03	3.07	2.99
7	04-12-2008	ASTER	2.65	2.69	2.60

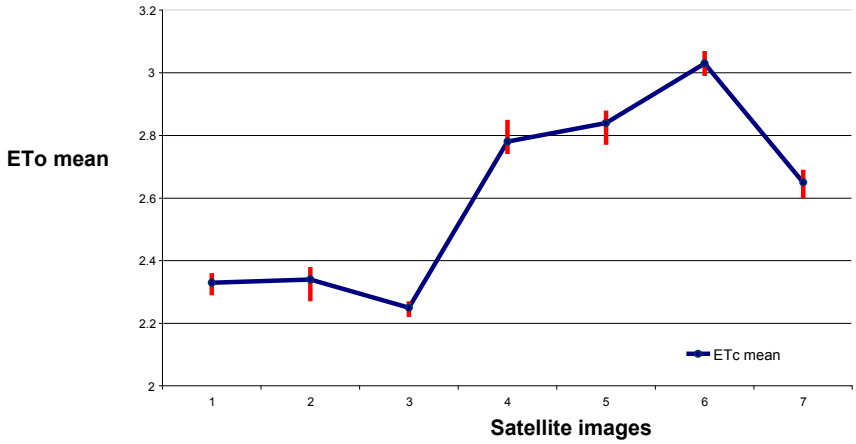


Figure 6: ETo mean diagram for each date, with their variations

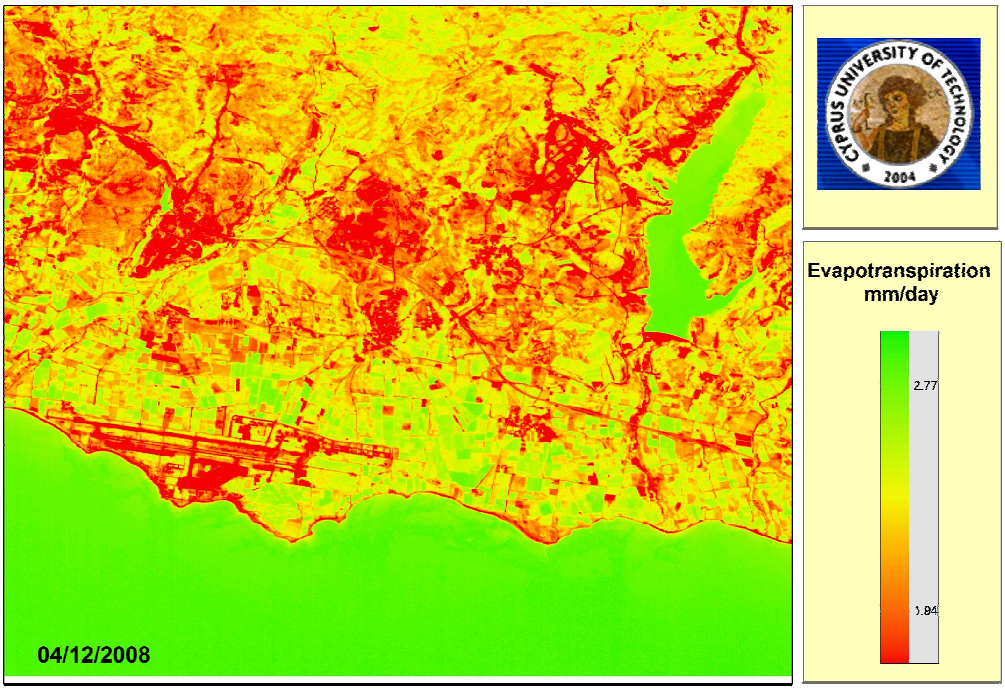


Figure 7: Map showing the ETo as calculated for ASTER images using GIS software.

The results are acceptable since as it is indicated ETo is increasing as we move towards spring and summer time, which is reasonable. The first two images show similar results since they are in the same date of time. This difference was calculated less than 0.01 mm/day and moreover no significant variations of evapotranspiration have been observed. Furthermore the results indicate that ETo of the early irrigated vegetables which are cultivated in the area, variable from 2.22 to 2.27 in February, and a slightly decreased of evapotranspiration was recorded. During the summer season of 2008, where leafy vegetables are cultivated, an increase of 20% of evapotranspiration was recorded, compared to ETo during winter season. As expected the highest evapotranspiration mean measurement was recorded summer time. Finally a dropped of the reference evapotranspiration can be seen at 2.65 mm/day.

5. CONCLUSIONS

This paper shows the overall methodology that has been developed to support this research project. Using the FAO Penman-Monteith method, evapotranspiration results have been found.

Meteorological data and satellite images (Landsat 7 ETM+ and ASTER) have been used in this preliminary application of the FAO method.

These promising results show how remote sensing data can be used in order to calculate evapotranspiration. Future work consists of further validation of the results, not only using Penman-Monteith method for other cultivations but other available methods and models for estimating evapotranspiration. The use of spectro-radiometric and sun-photometer measurements during the satellite overpass will provide further accuracy assessment of the existing evapotranspiration models and methods.

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DESIGN AND IMPLEMENTATION OF AN INTEGRATED HIGH RESOLUTION IMAGING GROUND PENETRATING RADAR FOR WATER PIPELINE REHABILITATION

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Abstract: Accurate and timely detection of leaks in water supply pipes is a significant environmental issue. Development of efficient non-invasive methods would lead into significant water saving and prevention of health hazards introduced by water leakage. This paper shows the ability to use a High Resolution Ground Penetrating Imaging Radar (GPIR), to be used as an instrument for water pipeline rehabilitation situations, parallel to the existing industrial equipment. In the framework of the European research initiative WATERPIPE, a Decision Support System (DSS) for the rehabilitation management of the underground water pipelines has been developed, parallel to the implementation of the GPIR.

Key words: ***Ground Penetrating Imaging Radar (GPIR), Decision Support System (DSS), Ultra Wide Band (UWB)***

1. INTRODUCTION

Many EU cities are experiencing increasing problems with their water pipeline infrastructure. The cost of replacing these old, worn-out systems, if left to deteriorate beyond repair, is astronomical and clearly beyond the resources of many communities. Replacement, however, is not the only choice as many of these systems can be rehabilitated at 30 to 70 percent of the cost of replacement. Accordingly, resources are now increasingly being allocated to address pipeline rehabilitation management issues. Due to the emphasis on sustainable management, risk-based approaches for the rehabilitation management of the water supply network need to be developed (Eiswirth, 2001, Farley, 2001).

Rehabilitation decisions should be based, inter alia, on inspection and evaluation of the pipeline conditions. Yet, utilities cannot locate a number of their old pipes and current inspection technologies typically do not provide the needed detailed information on pipeline damage (Ellyin, 1986, Sorensen 1988).

Thus, in the framework of the European Union funded "WATERPIPE" (SUSTDEV – 2005 – 3.II.3.3 - Global Change And Ecosystems) project, a radar system to work parallel to a decision support system are being currently under development to improve the capability of imaging of water supply pipelines, for better rehabilitation policy (Hunaidi,1992).

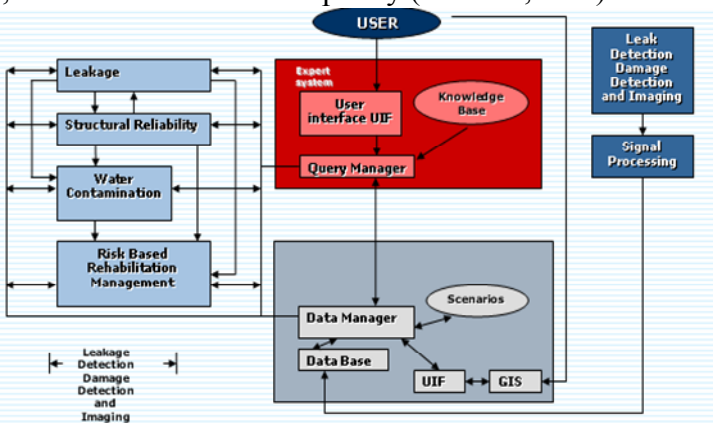


Fig. 1: Integrated System for the Rehabilitation Management of the Trunk and Distribution Water Mains

The objectives of WATERPIPE project can be summarized as follows:

1. To develop a novel, high resolution Imaging Ground Penetrating Radar (GPIR) for the detection of pipes, leaks and damages and the imaging of the damaged region and evaluate it at a test site (Lymperopoulos, 2003).

2. To produce an integrated system that will contain the equipment, as described before, and a Decision-Support-System (DSS) for the rehabilitation management of the underground water pipelines that will use input from the inspections to assess, probabilistically, the time-dependent leakage and structural reliability of the pipelines and a risk-based methodology for rehabilitation decisions that considers the overall risk, including financial, social and environmental criteria (Camarinopoulos, 1999, Hosoi 2001).

3. To field test the equipment and the DSS.

2. OPERATION PRINCIPLES

The pulsed signal with zero baseband spectral components is generated by a digitally controlled generator. The Ultra Wide Band (UWB) signal shapes have various type shapes such as a one period sinusoid or two opposite and inverted triangular pulses. The Short Pulse Generator output has amplitude of the order of 10 V and drives to fixed position wide band antenna as shown in figure 2. A sample reference signal is also being driven to Correlator Receiver to achieve synchronisation between the receiver and transmitter units.

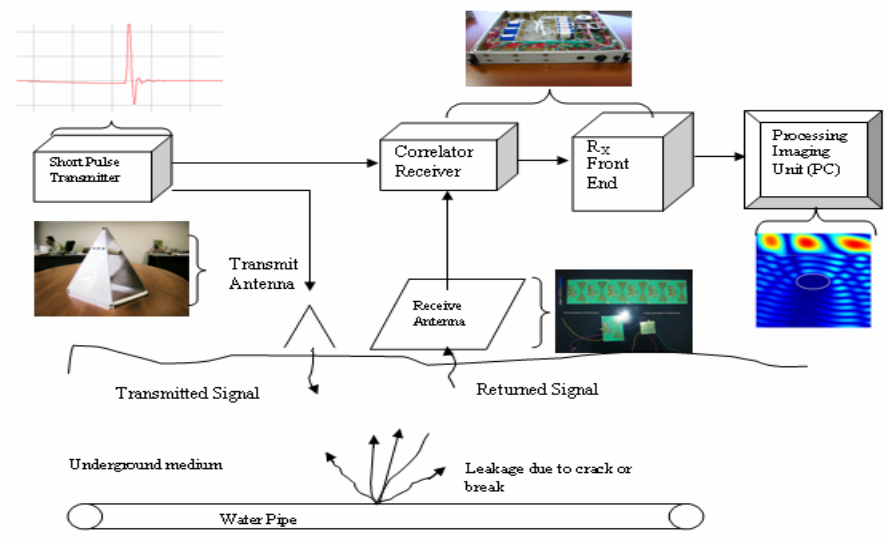


Fig. 2: GPIR Operation Detailed Analysis

The receiver chain consists of a mechanically scanning antenna in an area of 1x1 m2. For each pulse transmission a number of 8-64 pulses will be integrated to improve the Signal to Noise (SNR) power ratios. The number of points for the scanning antennas is 20x20 leading into 400 measurements which lasts approximately 8 seconds. Therefore the pulse repetition frequency of the transmitter is 3.2-52 kHz. The received signals are amplified and then correlated with the reference pulse signal to obtain the envelope of the signals reflected by the ground medium. The use of UWB signals provides extensive information for the imaging of the underground target medium.

At the end of each receiver scanning antenna, 400 reflected signal waveforms are available to the processing unit. The reconstruction algorithm of the underground medium complex dielectric constant (two real numbers for each frequency spectral component) is obtained using a time domain version of the Method of Auxiliary Sources (MAS) which has been developed in the past. Essentially the reconstruction of the complex dielectric permittivity of the underground medium

with the embedded water supply pipe will pose an “Inverse Scattering” problem whose solution leads into imaging of the medium under measurement (Kechribaris, 2000).

The Ground Penetrating Imaging Radar operation principle is based on the use of UWB Signals. In the following, details of the system are described by explaining the individual functioning of the subunits of the system to be developed.

The subunits constituting the GPIR are described in the following paragraphs.

2.1 Transmitter Unit (including Antenna)

An UWB signal generator is utilized to produce sub-nanosecond pulse (500-3000ps). The peak pulse amplitude is of the order of 10 Volt. This pulsed signal is conveyed to the transmitting antenna which is a Transverse Electromagnetic Flared Horn antenna. The transmitting antenna is fixed contrary to the receiving antenna which is of scanning type. In order to decrease the total sampling time of the reflected signals from the underground, the pulse repetition frequency (rate) should be up to 10MHz. The use of TEM horn guarantees wide band matching of the antenna of pulse generator.

Table 1. Transmitter Specifications

Specification	**Value**	**Units**
UWB Spectrum Centre Frequency	800	MHz
Total Spectral Bandwidth	300-2000	MHz
Pulse amplitude (pk-pk)	5-20	Volts
Pulse duration (min/max)	0.5/1/2/3	nsec
Repetition Frequency	10	MHz

The pulse shaping circuit consists of a crystal oscillator with an oscillation frequency of 10 MHz and an amplitude in the range of ± 1 Volts. This signal is then amplified by an RF amplifier to 20-30 Volts pk-pk and it is driven into the step recovery diode (SRD

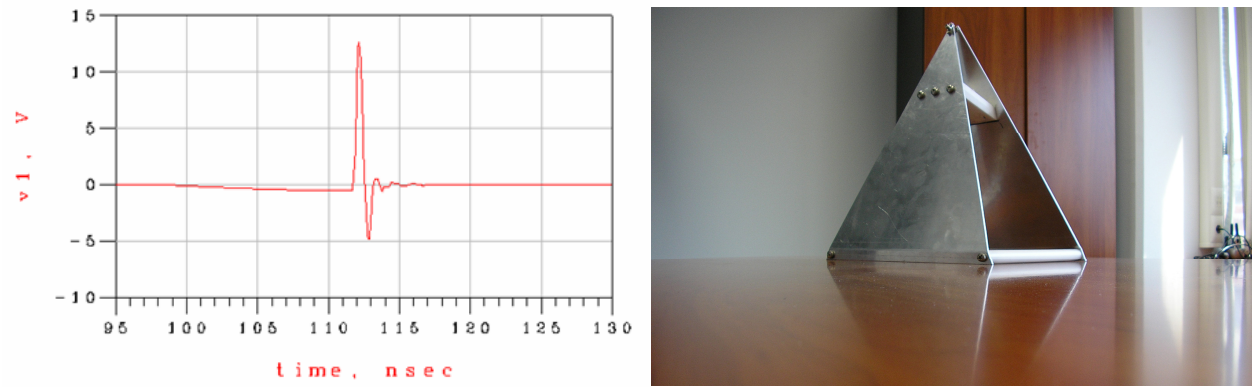


Fig. 3: Left: Transmitted Pulse of the GPIR Right: Photo of the transmitter antenna

The antenna to be used to radiate the pulsed signal towards the ground medium have a bandwidth of at least of 1600 MHz with a center frequency of 800 MHz and should not present dispersion such as the case of log-periodic antennas. The radiation pattern is down looking and at a distance of 20-30 cm from the ground surface should cover a distance of ± 50cm.

2.2 Receiver Front End (Analog Part)

Signal conditioning of the scanning antenna should be carried out using a Wide Band Low Noise Amplifier-LNA (0.2-2GHz) and an Anti-Aliasing Filter. The LNA has a maximum gain of 40dB

gain. The received echo pulse signals reflected by the ground structure are amplified up to a level that is sufficient to convert into digital signals. Direct Analog to Digital (A/D) conversion to achieve robust signal detection of echo signals. The following specifications are set up concerning of the receiver front end:

Table 2. Receiver Specifications

Specification	Value	Units
Frequency bandwidth	2,000	MHz
Gain	20-40	dB
Gain Ripple	+/-1.5	dB
Gain control	20	dB
Noise Figure (max)	4	dB
Input/Output impedance	50	Ohms

In figure 4 the receiver Rx unit is shown. The microstrip bow-tie antenna dipole is clearly shown out of the shielding metallic box. Inside the metallic box all the receiver circuitry is included. The bow-tie antenna permits the reception in a very wide bandwidth of over 1GHz.

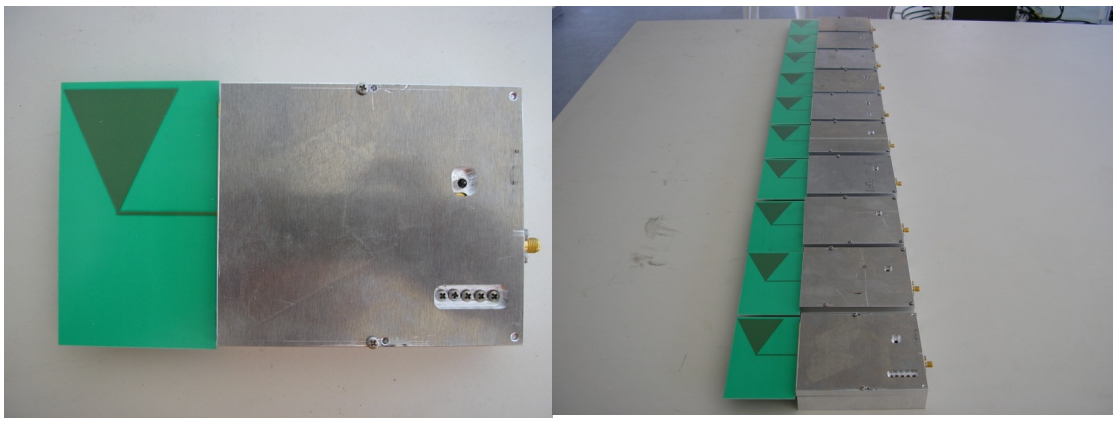


Fig. 4: Left: Photo of the Receiver System Right: Photo of the 10-receivers of WATERPIPE

2.3 Scanning Receiver Antenna

The receiving antenna is also a TEM horn but with minimum size. The antenna scans mechanically parallel to earth surface above the earth surface approximately 15-20cm. The scanning is of raster type that provides scanning of 1x1 m2 horizontal area in the Cartesian coordinate system. During the scanning period at 400 (20X20) points data are collected and converted into digital format. Signal averaging are implemented to improve the signal to noise ratio at the receiver output. The time needed for the antenna to scan the area of 1X1 m2 is approximately 8 seconds at maximum.

2.4 Signal Digitizer

The received echo signals, after their amplification, are driven to an ultra high speed A/D converter with the following basic specifications:

- Conversion Speed: 1.5 GSample/second
- Output Resolution: 8 bits
- Input Bandwidth: 2 GHz

The acquired data after averaging (8-64 bits) are stored at the RAM of the host computer where the imaging software will develop the 2 or 3 Dimensional images.

2.5 Computer and Image Display

The Computer where the processing of the recorded echo signals take place is a 3GHz Pentium. The primary development will focus on the development of imaging software which is based on the Method of Auxiliary Sources (MAS). This method is based on the solution of electromagnetic material media interaction using auxiliary sources within the excited media. The early concepts of MAS have been developed in the famous Tbilisi Mathematical School and during the last 10 years this method has been used extensively by the Laboratory of Applied Electrodynamics of the Tbilisi State University and the Microwave and Fiber Optics Laboratory of the Institute of Communication and Computer Systems in Athens (Zaridze, 1998a, b).

The imaging technique is based on correlating the measured time domain data at the 400 points with the Auxiliary Sources assumed to within the ground media. The number of sources is 400 also providing a sufficient resolution capability. In order to determine the complex amplitudes of the Auxiliary sources the mean square error technique is used by comparing the measured data and the Auxiliary source distribution inside the ground medium. The source field distribution in the volume of interest provides the required images with an expected resolution of 5-10 cm.

2.6 Mounting Structure of GPIR on a car

In figure 5 the vehicle that carries the whole system is shown. The transmitting antenna is shown on the left figure as well as the ten receiver units which are moved back and forth through the use of step motors which is computer controlled. On the table on top of the vehicle there is a laptop computer along with ultra fast digitizer card and other controller cards which will acquire and control the useful signals and present them to the user in a meaningful way.

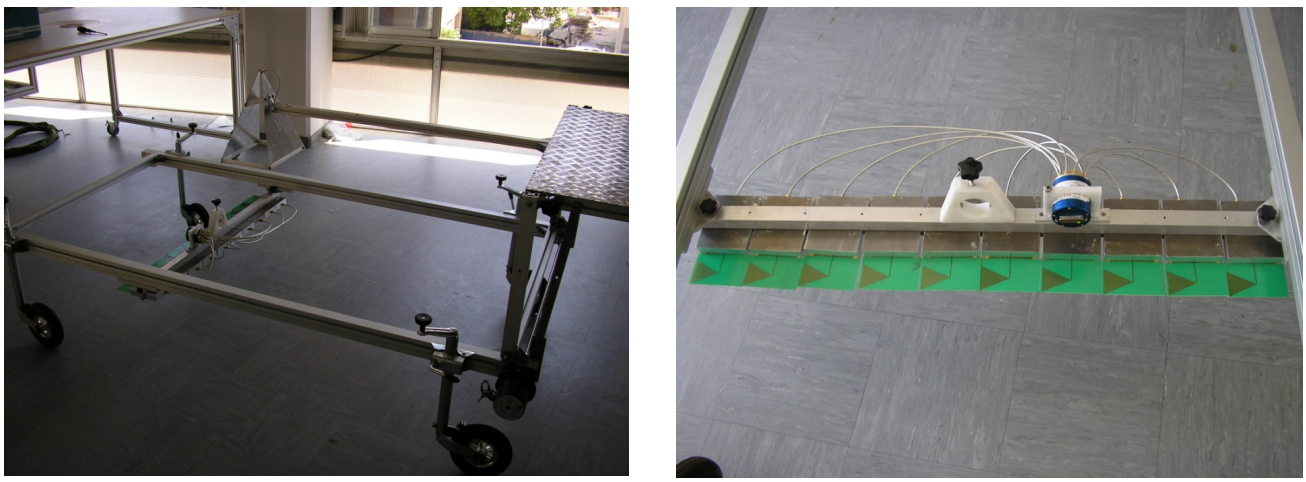


Fig. 5. Photos of the System vehicle

3. RESULTS AND DISCUSSION

The implemented system is currently under testing. Hopefully, the results will be gathered and analysed within the following months and presented in the conference. Meanwhile there are some initial measurements, by using raw data from another system and performing imaging with our software algorithms.

The figure 6 represents the 2-D data imaging of a pipeline (diameter of 50 cms) buried in a depth of 75 cm in relation to depth. As you can see from the figure, there is a very good approximation of the pipeline.

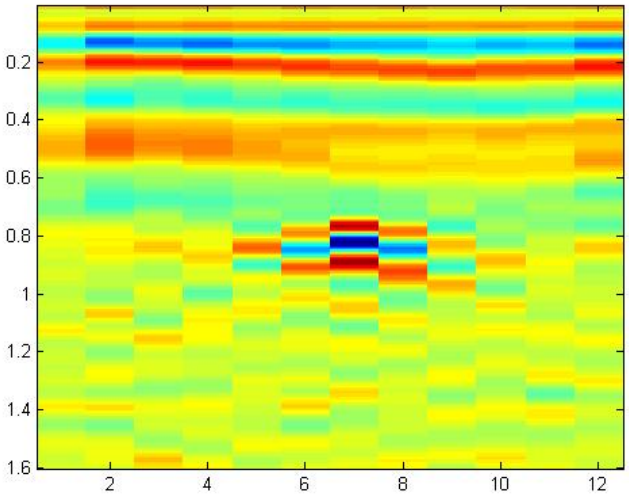


Fig. 6. 2-D data representation by using our software algorithms

The following figures show the application of the 3D reconstruction algorithm on these data. Since the developed reconstruction algorithm is inherently three dimensional, data were replicated along the y-direction in order to generate a 3D data set. These images show the excellent ability of the algorithm to depict

By using these equations, we obtain the 3-D graph as presented in the right figure, where we can see a pipeline buried in 0,875 meters, which is quite close to 0,75 meters (actual depth).

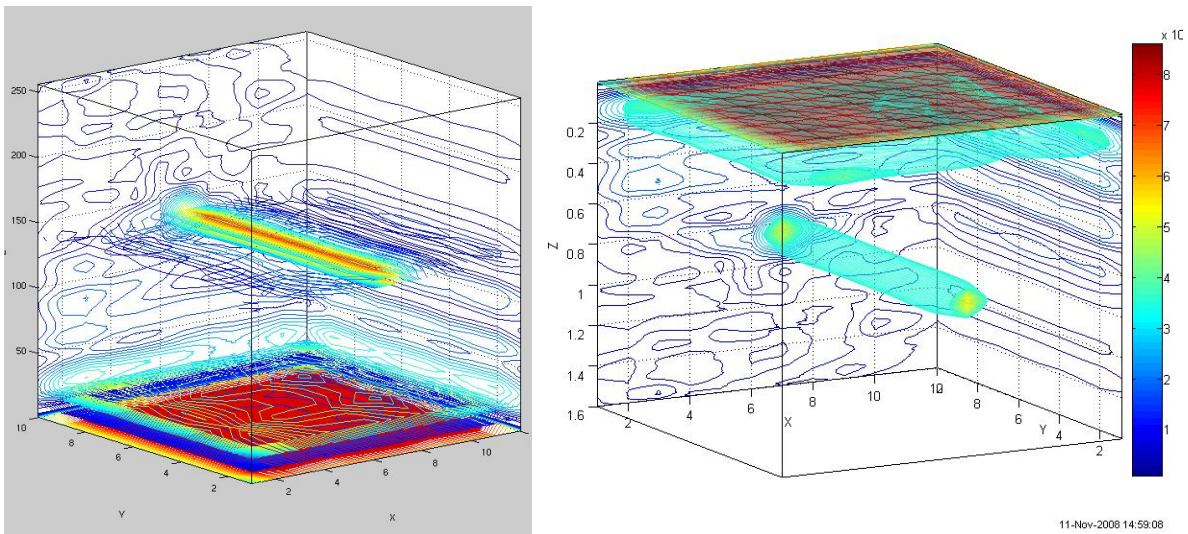


Fig. 7. 3-D data representation by using our software algorithms

4. CONCLUSIONS

The proposed UWB radar is also has been constructed to find out the most efficient system in detecting the water leaks. This system is capable of performing imaging of the underground and the pipeline as well. Finally, the use of an efficient Decision Support System makes the whole system very attractive and useful tool for most of the Water Utility companies, since the outcome would suggest an integrated solution for the water leak detection and the rehabilitation system. At a latter

phase of the project, the authors will attempt to integrate the proposed system with a Continuous Wave Doppler Radar- LEAKING System that was implemented in the 5th framework of EC projects. This system is able to find water leaks with very good accuracy without having any imaging capabilities at all.

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THE MULTIDISCIPLINARY AND MULTINATIONAL DATA AND INFORMATION SYSTEM DAISY – THE KEY FOR AN IWRM-PROJECT IN THE LOWER JORDAN VALLEY (ISRAEL-JORDAN-PALESTINE)

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Abstract: During the last 2 years DAISY, an interdisciplinary and multinational database and exchange platform was developed at the Helmholtz-Centre for Environmental Research – UFZ (Germany) and within the framework of the integrated water resources management project SMART. Within SMART, the most important issues of the Levant: the characterisation and the sustainable use of all available water resources is studied and evaluated. For that purpose, 13 parties from Germany, Israel, Jordan and Palestine are involved and cover different interdisciplinary scientific sectors. The aim of DAISY was to fulfil the need of central available datasets for the trans-boundary work of the projects partners and to feed the decision support system, which is simultaneously under development. As yet, about 2 million datasets are gathered, processed, implemented and are for all project partners and interested public users available. The key issue was to bridge societal gaps and trenches in order to allow real trans-boundary project work.

Key words: SMART, integrated water resources management, DBMS, DAISY, Lower Jordan Valley.

1. INTRODUCTION

SMART is based on the concept of an Integrated Water Resources Management (IWRM), which, by definition, needs to take into account e.g. the environment, eco-systems, socio-economy, water supply and demand management and the need to preserve the water resources for future generations. In the case of the Lower Jordan Valley (LJV) it needs also to take into account subjectively conflicting interests, as the project area is shared by three different political entities, with often acute problems of communication.

Beside different socio-economical and technical aspects, the planned IWRM has to deal with the physical characteristics of the area, such as climate, geology, hydrology (surface and subsurface), land-cover, etc. All these variables are connected. To achieve the management of the LJV-water resources and the resulting decisions, all these natural and anthropogenic effects and variables have to be calculated and evaluated.

The accompanied decision making in natural resources management is always based on the assumption that the decision maker had the required cognitive and integrating capabilities. However, the degree of complexity of the system LJV and the need to evaluate the interconnection and –action makes a computational decision support system (DSS) necessary.

Despite of what architecture the DSS is, the data and information management is the fundamental component (Fig. 1). The database allows people and institutions from different backgrounds and political environments to talk the same language. Besides being integral part of the DSS, DAISY also administrates data and information, which are needed for the scientific project work that is the base for each IWRM. Data here include spatially distributed, time varying and geo-referenced information. In this paper, the authors want to present the alphanumerical part of DAISY and its accessibility.

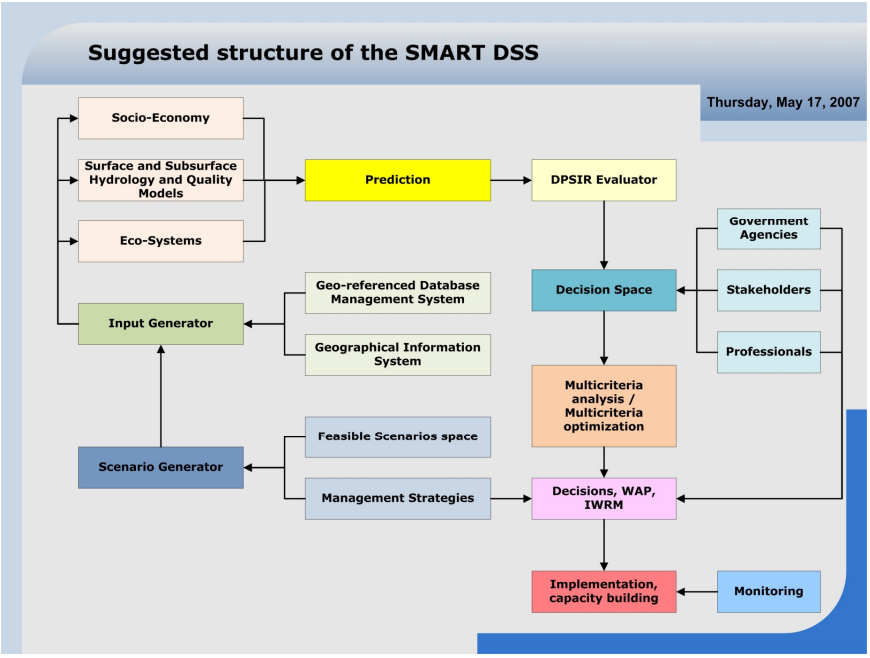


Fig. 1: suggested structure of the SMART DSS and the integral function of the DBMS (Wolf et al. 2009)

2. OBJECTIVES AND REALISATION

As a result of the IWRM-approach of SMART, the aim to develop an adapted DSS for the LJV, the interdisciplinarity and complexity of data and information which are needed from former and parallel projects and those which will be gathered during the project phase, the SMART database management system (DBMS) was necessary. That comprehensive database is developed to efficiently supply all partners with input-data for established systems and systems, which are under way (model builder or geographical information systems - GIS).

The DBMS is designed to be a data platform for collecting, storing and sharing data between partners, in order to ensure proper, efficient and fast supply of data and information needed for the execution of the various project tasks.

Therefore, project (basin) areas and borders were determined and subdivided into sub-areas of more homogeneous problems and characteristics also to allow the focus on more specific local problems that could be then integrated into the project stream.

To fill in relevant historical and recent data, an integral part was to gather, compile, and organize all the available data on the LJV that is relevant and necessary to the project, including:

- Agriculture: land-cover and soil maps by satellite images, vectorised in 1:100,000
- Socio-Economy: inventory of the cities and villages in the LJV, their current population and expected growth trends, water consumption (e.g. by sectors of activity), current output of wastewater and treatment levels and utilization
- Geography: up to date topographical and thematic maps in different scales (1:50,000 – 1:500,000) and vectorised thematic maps

- Remote Sensing: aerial and satellite images from different periods, gathering/buying particular remote sensing data (high resolution for necessary small areas: aerial photographs), time series images for wider areas in respect to characterize development of urban/rural areas
- Morphology: up to date digital elevation model for the study area, computed by combining different sources (Israel Geological Survey, ASTER, SRTM), vectorised contour lines
- Wells: characteristics (location, depth, casing, pump rates, lithologies, etc.); time series of piezometric heads and quality like chloride, nitrate and other elements that exist in any of the institutional data warehouses
- Springs: characteristics (location, elevation, type, aquifer etc.), time series of discharge and quality (chlorides, nitrates and other elements)
- Climatic data: gauges characteristics (location, elevation, type of gauging device) and time series of rainfall data of various aggregations (storm aggregation for the analysis of runoff processes and monthly aggregation as input to the groundwater flow and transport models). Time series for precipitation and for other climatic data (e.g. temperature, air-moisture, solar radiation, wind-speed, etc.) in respect to specific and regional climatic conditions (e.g. storm events…)
- Water distribution system: layout and characteristics (diameter, discharges etc.)
- Sewage network: size, layout, treatment plants, type of treatment and capacity etc.
- Water retaining and storing facilities for the collection of runoff: size, capacity, location.

All data stored in previous works and those delivered by project partners have been reviewed and classified according to database regulations and their relevance to the project The purpose was to get a logical data structure (variables, units, parameters, etc.) and to avoid redundancy.

In cooperation with project partners gaps of knowledge and information that need to be bridged in the frame of the project were partly be detected, in order to meet the project objectives. Making all these information and data available DAISY assisted project partners to determine additional investigations (including field work) that are necessary to achieve the strategic objectives of the project.

All implemented data are incorporated into the integrated relational DBMS (Fig. 2), which is based on an Oracle 10.1 system at the UFZ. The web based graphical user interface (GUI) of DAISY makes them available via personal internet-portal and an interface to GIS platform. DAISY is mentioned to be the central inquiry location for all data requests and problems of the project. DAISY is designed to answer on different issues:

- DBMS – Oracle: storage and providing alphanumerical data
- ArcSDE: storage and providing raster- and vector data
- Transfer-drives: being a secure platform for transfer, exchange and storage od all types of data between working groups, project members and institutions
- DAISY-harvester: GUI to request alphanumerical data of Oracle
- ArcIMS/UMN: interactive GUI to plot raster- and vector data and design maps

Furthermore, via a special Citrix-Gateway every partner is able to use special software applications like Erdas Imagine, ArcGIS, Mathematica, Statistica, ModFlow, etc (Fig. 2).

Additionally, it is a platform for focused and detailed public relation of SMART with a high developing potential. At the same time, the concept of DAISY allows the access for public users to get informed about project outcomes and to download the available data from former, running and future projects especially in the Middle East.

Because it is accessible by the public, it is designed on several distinct security layers, which either keep public out and/or shields sensitive data against everyone. Hence, a key issue was to address data rights and establish a system for preserving rights and privileges to regulate the use and dissemination of data.

Partners do have personal access (login/password) to the web site for downloading and querying the DBMS. Hence, the download- and data-sharing interface is strictly password protected, administrator-controlled, copyright sensitive and working-group specific. Visibility and downloads of data is regulated by the permission of the data owner.

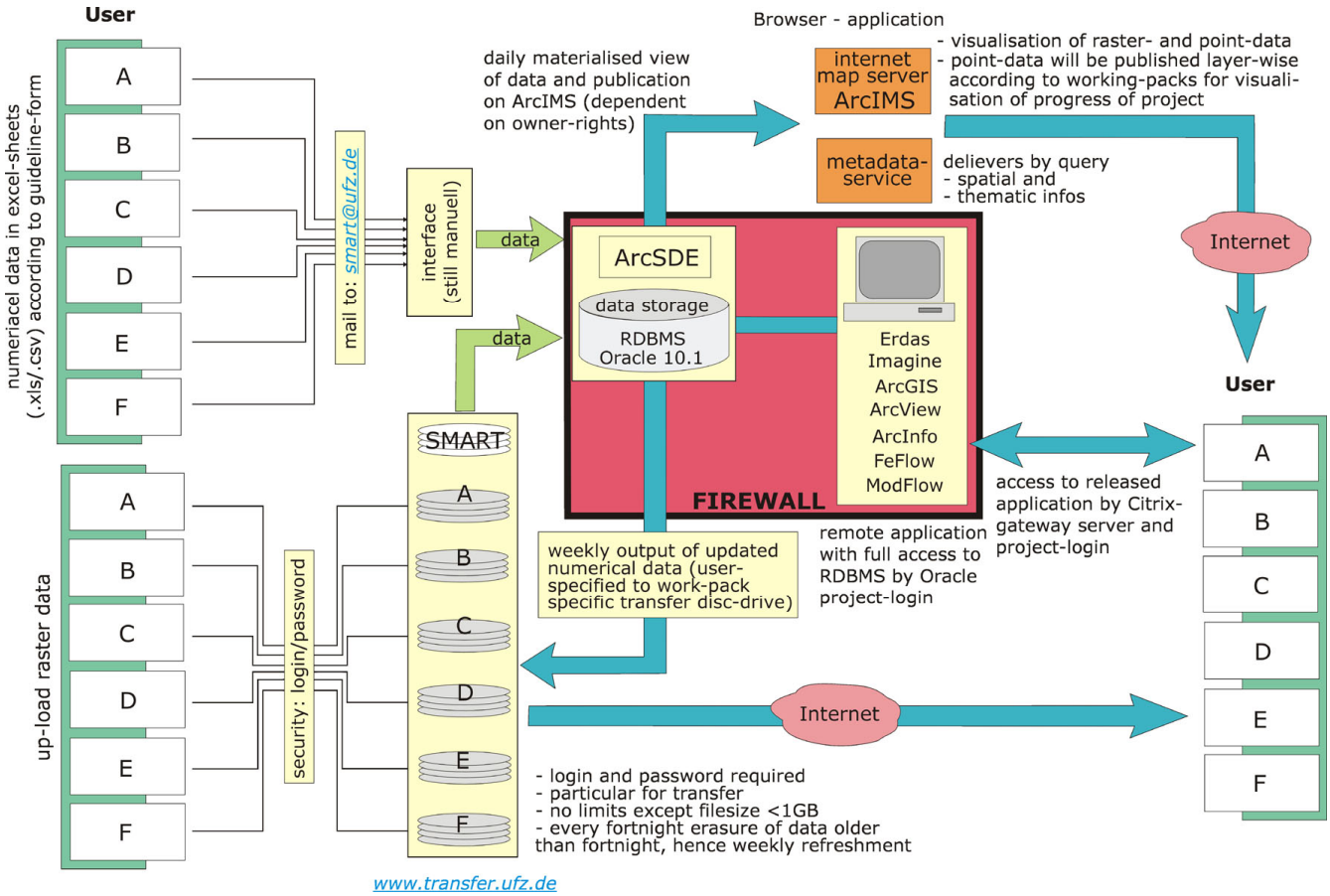


Fig. 2: the design of the SMART data and information system DAISY at the UFZ

3. THE GRAPHICAL INTERFACE

After the first phase of developing DAISY, an easy to use and web based graphical user interface (GUI) was developed (Fig. 3), to make the access for all users as easy as possible. Out of this mask, every user decides, what he is interested in and what is the target he wants to surf to.

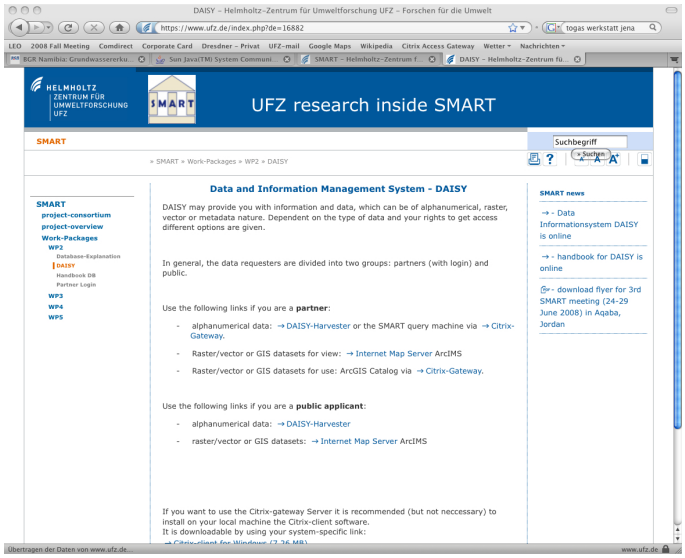


Fig. 3:
the entry to the Data and Information System – DAISY

For alphanumerical data, the DAISY-data harvester as a comfortable web based front-end (Fig. 4) to check out location, types, time series and availability of datasets of external projects or of partners of the SMART-consortium.

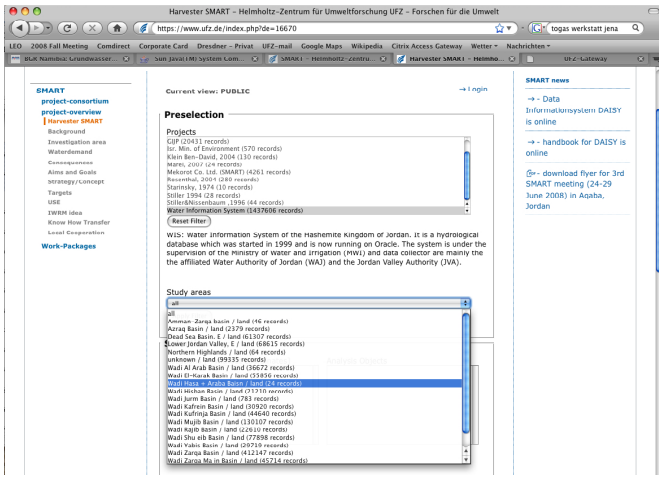
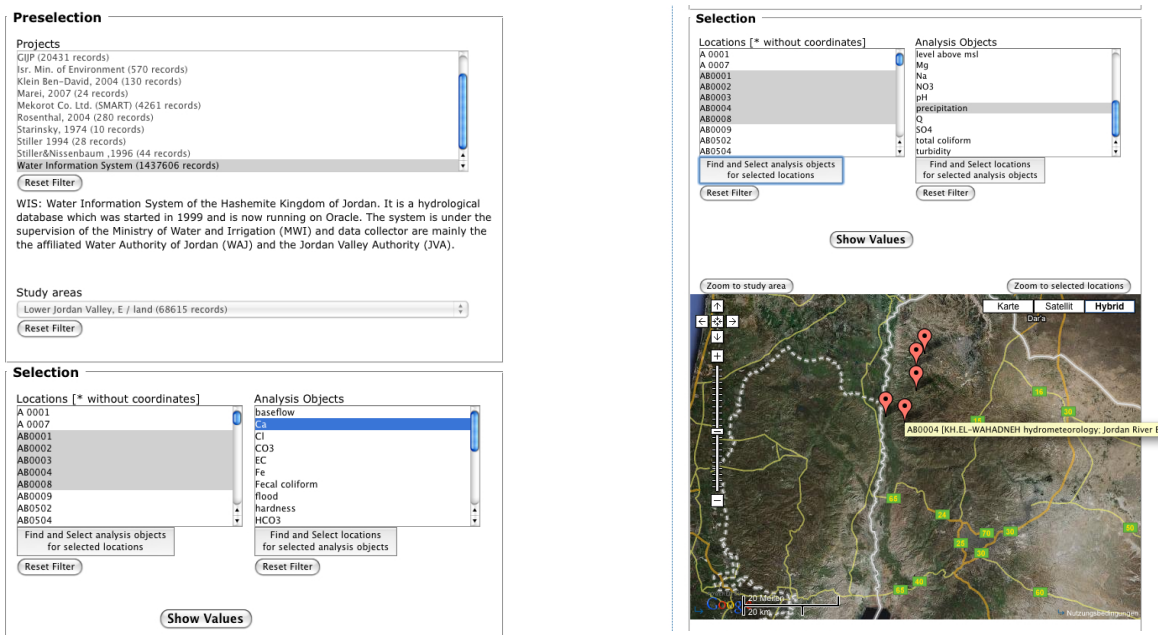


Fig. 4:
entrance to the DAISY-harvester, where in the preselection those projects are selectable, which spent data to the RDBMS and in the next step, the location of interest will be refined

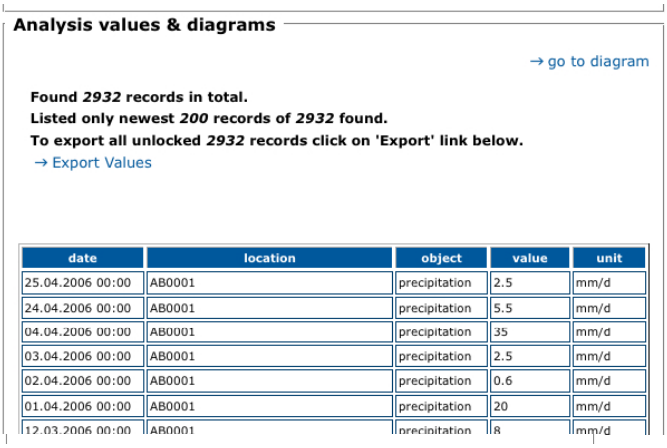


(a) (b)

Fig. 5: a) shows the preselection and selection window and the choise the user have and b) the plot of selected locations inside the Google Earth application.

Two levels of visitors are allowed to use the harvester, the publicity and project members with LDAP-passwords. The public level allows the user to surf through the data, which are released by data-owner as public domain. Such data are either already published, from authorities or gathered out of literature and other databases. When SMART-members want to login, they use the login link and type in their respective LDAP-login.

The preselection starts with data owners and projects of which a short description and the scientifical content are given below the first list. When the area of interest is selected, the type of data and the specific locations will be defined by cross filtering and selection by locations and analsyses objects (Fig. 5a). The location of the selected data points is displayed in a Google Earth application where red drops indicate selected locations (Fig. 5b).



Analysis values & diagrams

→ go to diagram

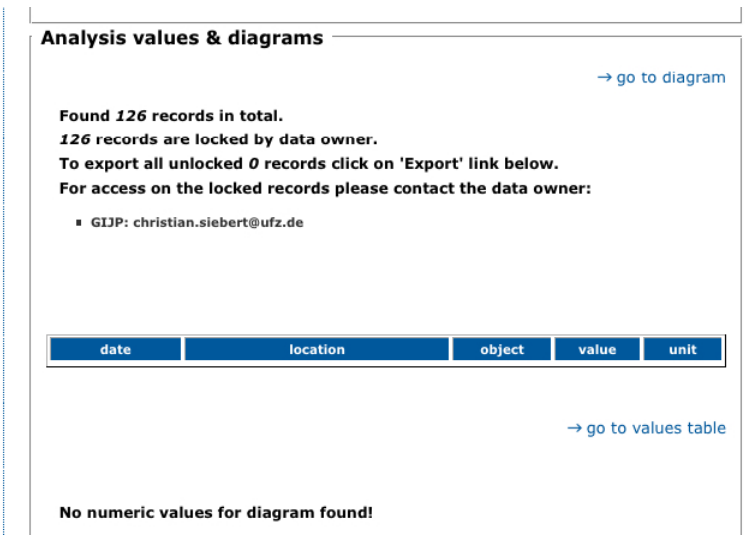
Found *2932* records in total.
Listed only newest *200* records of *2932* found.
To export all unlocked *2932* records click on 'Export' link below.
→ Export Values

date	location	object	value	unit
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24.04.2006 00:00	AB0001	precipitation	5.5	mm/d
04.04.2006 00:00	AB0001	precipitation	35	mm/d
03.04.2006 00:00	AB0001	precipitation	2.5	mm/d
02.04.2006 00:00	AB0001	precipitation	0.6	mm/d
01.04.2006 00:00	AB0001	precipitation	20	mm/d
12.03.2006 00:00	AB0001	precipitation	8	mm/d

Fig 6: database answer on selected locations and parameters can be displayed as table (a) or diagram (b)

The output of the data is possible as table (Fig. 6a) and as diagram (Fig. 6b). In the diagram option, the user is able to select one of the requested locations (if multi-location request was sent) or parameters (if multiple-parameter request was sent). Additionally the plotted frame of the time series can be chosen and exported into Microsoft Access format (.mdb) or as csv-file.

However, if data are sensitive, no other users than the owner will see the content of the query results, except the amount of records that are stored in the database and the note to contact directly the owner (Fig. 7).



Analysis values & diagrams

→ go to diagram

Found *126* records in total.
***126* records are locked by data owner.**
To export all unlocked *0* records click on 'Export' link below.
For access on the locked records please contact the data owner:

- **GIJP: christian.siebert@ufz.de**

date	location	object	value	unit

→ go to values table

No numeric values for diagram found!

Fig. 7: The database answer on requests when selected datasets are confidential. It shows the user, how many datasets are available, how much of them are locked and who is the person of charge to ask for (contact is given in the project explanation in the preselection area – on top of the page.

The second types of data, which are stored in the database, are raster- and vector datasets. However, Oracle as database for alphanumerical data is not able to handle spatial coded raster or vector data. For that purpose, an ArcSDE (one application in the ESRI-family) translates and administrates these data for Oracle. The direct access to these data via ArcCatalog standard application is restricted to project members and their institutional co-workers.

4. CONCLUSION

The concept of and DAISY itself as integral part of the SMART project is on the way to be a successful multi-purpose, -discipline and –national data and information system for integrated water resources management approaches and for all accompanied activities. To get access to DAISY, public and project internal users type into their browsers: http://www.ufz.de/daisy.

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THE USE OF HIGH RESOLUTION SPRING DISCHARGE DATA SETS TO ESTIMATE TRAVEL TIMES THROUGH THICK VADOSE ZONES IN A SEMI-ARID AREA

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Abstract: The aim of the presented approach is the temporal and spatial identification of groundwater recharge in karst aquifers with thick unsaturated zones. The approach requires the measurement of time series of discharge / water level at springs and boreholes with high temporal resolution. It is based on time derivative(s) of hydrographs and recession coefficient(s) of reservoirs (Geyer et al. 2008). Information about the characteristics of the unsaturated zone is not required. The results are compared to recharge assessments based on soil water balance approaches. The study area is located on the western graben shoulder of the Jordan Rift Valley northwest of Jericho (West Bank) and can be subdivided into two aquifer systems. The aquifers are composed of karstified cretaceous carbonate rocks. During the rainy winter season precipitation percolates through the vadose zone, which is mostly several hundreds of meters thick, and recharges the aquifers. The mean annual precipitation is about 600 mm/y in the western part of the study area and decreases towards the east to about 150 mm/y. The upper aquifer section drains via several large springs. During the dry summer season the few perennial streams are only fed by spring water. Spring water is used further downstream for domestic water supply as well as for irrigation. The proposed approach is demonstrated at the location of two of the largest springs in the West Bank: Auja spring and Sultan spring. The springs exhibit different discharge characteristics. Whereas Sultan spring possesses a rather constant discharge, Auja spring displays high fluctuations linked to precipitation events. Therefore different recharge mechanisms are assumed for the two springs, rapid and concentrated recharge for the Auja spring and slow and diffuse recharge for the Sultan spring. After consecutive years with less than average precipitation Auja spring has even been observed to dry out. This cease of flow has severe implications on the agricultural areas relying upon the spring water. This technique has large potential in the reduction of ambiguity, both in the interpretation of flow processes as well as for modelling groundwater flow in these highly complex aquifers.

Key words: Groundwater discharge, aquifer, semi-arid, SMART.

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GROUNDWATER RESOURCES IN WESTERN JORDAN: A HYDROGEOLOGICAL INVESTIGATION OF THE INFLUENCE OF COMPLEX GEOLOGICAL FEATURES ON GROUNDWATER FLOW PATHS AND STORAGE

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Abstract: The Jordan Valley is a part of the Jordan Dead Sea Wadi Araba Rift Valley, which extends from the Red Sea to lake Tiberias and beyond with a major 107 km sinistral strike-slip fault between the Arabian plate to the east and the northeastern part of the African plate to the west. Due to extensional forces a topographic depression was formed. As a result of an arid environment it is filled with evaporites, lacustrine sediments, and clastic fluvial components. Due to its proximity to the active plate boundary the geology of the study area is rather complex. All available geological and geophysical data set along with several field surveys were used to generate a 3-D geometric geological model, which incorporates all important geological features, such as faults, fold and thickness variations of the different geological strata. On the basis of these findings a numerical 3-D flow model integrating all important geological features of the hydrogeological system was developed. The numerical model was implemented by using the object- and process-oriented GeoSys/Rockflow model developed at the Department of Environmental Informatics at the Helmholtz Centre for Environmental Research/ Germany.

Key words: 3-D geologic model, GeoSys, Jordan Valley, Jordan, Numerical modelling, SMART.

1. INTRODUCTION

The purpose of this paper is to assess the influence of complex geological features on groundwater flow. The challenging aspect of this study is the application of a numerical flow to a steep and complex geometry of different geological layers. Possible outcomes of this numerical flow modelling study could be the identification of areas who have either a potential for groundwater storage (infiltrating both excess surface water and treated waste water) or areas for further groundwater abstraction (well drillings) or both. The numerical simulations are conducted with the scientific software GeoSys/Rockflow.

1.1 Geography and Climate

The study area (the subsurface catchment of Wadi Kafrein) lies on the western margin of the Asian continent and belongs geographically to the Eastern Mediterranean Basin and is located in the lower Jordan Valley/ Jordan (*Figure 1*). The centre of the study area is located at WGS84 coordinates 31° 55' N and 35° 46' E, and has an area of 150.66 km^2. The maximum extension of the study area is 14 km in east- west and around 20 km in north- south direction. The highest elevations are located on the eastern border of the study area. The highest point is located to the west of the city of Sweileh at an elevation of 1,096 m a.m.s.l. The lowest point in the study area is located to the west of the study area, next to a large surface water reservoir (Wadi Kafrein Dam) at an elevation of -190 m a.m.s.l. Two major cities are located in the eastern part of the study area: the capital Amman an the city of As-Salt. The deeply incised valleys of Wadi Kafrein carry perennial

flow. However, during and after the rainy events, they also drain floodwater down to the Jordan Valley. The floodwater carries sediments and sometimes even rocks forming fan deposits in the Jordan Valley.

According to Köppen (1931) the study area can be classified as a Group B Dry (arid and semiarid) climate, since precipitation is less than the potential evaporation. Morphology has the largest influence on the prevailing climate. A subdivision into three different climatic zones can be made: the Highlands area, the Western Slopes of the East Bank, and the Jordan Valley. The climate in the Highlands is of Mediterranean type. It is characterized by long, hot, dry summers and short, cool, rainy winters. Towards the west, the climate undergoes a rapid change to semi-arid and arid climate in the Jordan Valley. The western Slopes act as a transition zone between the Mediterranean climate along the Highlands in the east and the arid climate in the Jordan Valley in the west.

Rain falls only during the winter months. Then the climate changes abruptly from dry hot summer conditions to humid, cold, and stormy conditions. Usually it starts to rain in November and rainfall continues until the end of April, whereby 70 % of the annual precipitation falls between November and February. Snow falls once to twice a year in the Highlands. The average annual precipitation on the western ridges of the Highlands (Jordan University, Amman) is around 505 mm/y, on the eastern part of the Western slopes (Naur) 395 mm/y, and in Jordan Valley (South Shuneh) the long term average is around 166 mm/y. Thus a strong correlation with altitude and climate data for the area exists.

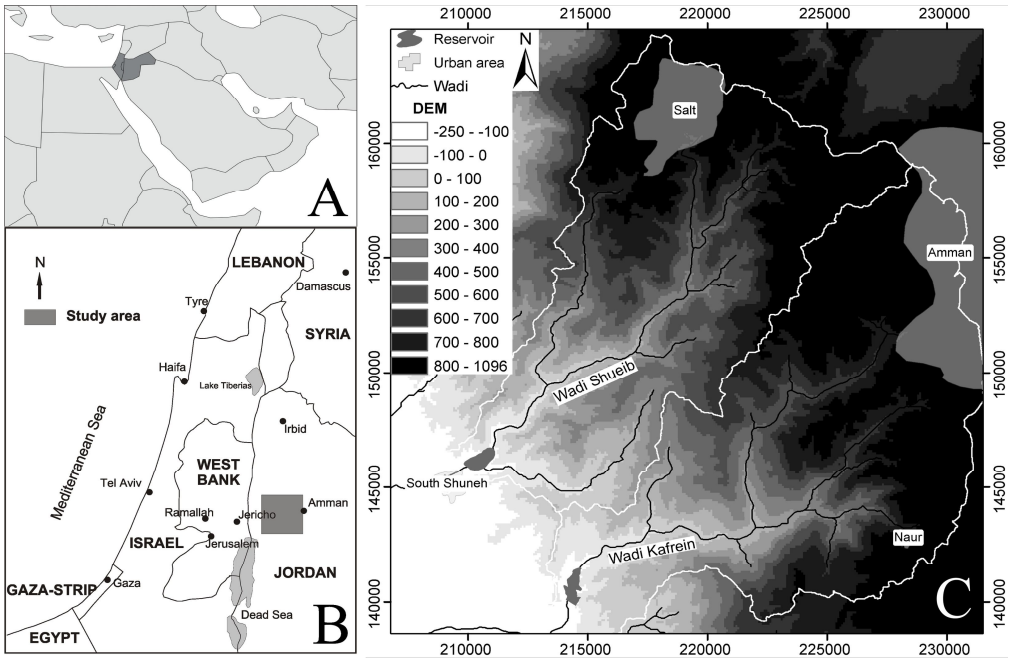


Figure 1*: Location of the study area. A: Middle East; B: Study area in the regional context; C: Study area (the surface water catchment boundaries of Wadi Kafrein and Wadi Shueib are highlighted by a white line).*

1.2 Geology

Regarding the geology of the study area: three unconsolidated groups exist in the area. Above the early Cretaceous transgressive unconformity, the sandy Kurnub Group (Neocomian), the limestone-marl sequences of the Ajlun Group (late-Albian to end of Turonian) and the predominantly chalky Belqa Group (Coniacian to Eocene), follow. The fluvial to marginal marine sand-stones of the Kurnub Group increase in thickness northwestward attaining 180 to 200 m at the NE end of the Dead Sea and 250 to 300 m in the Wadi Zarqa area. They form the Kurnub aquifer containing water of low salinity.

The lower Ajlun Group (Naur, Fuheis, Hummar, Shueib Formations; A1 to A6) with 300 m of subtidal to peritidal marls, nodular limestones, and massive limestones, reflect several transgressive to highstand cycles on a permanently submerged carbonate platform which, however, end with a

late Middle Turonian lowstand generating the upper Shueib Formation with shallow water dolostones and marls grading southward into supratidal gypsiferous claystones (Schulze et al 2003). Thus, because of the presence of the Fuheis and the Shueib marls and claystones, the lower Ajlun (A1 to A6) forms a multilayered aquitard - locally even an aquiclude - with embedded aquifers such as the massive limestones of the upper Naur and of the Hummar Formations. Contrastingly, the overlying, ca 100 m thick Wadi as-Sir Formation (A7) consists in the area mainly of well bedded and massive limestone of a prograding and agrading carbonate platform. It forms together with the basal Belqa Group the important Upper Aquifer (A7-B2 aquifer).

After an episode of non-deposition, the pelagic chalks of the Wadi Umm Gudran and Amman al Hisa cherty limestone formations (Belqa Group) were deposited and, on the west-facing slopes, are preserved in synclines. Only on the crest of the Transjordanian Mts. and beyond, they form a gently eastward sloping continuous layer overlain by bituminous marls and chalks of the B3 aquitard (Muwaqqar Form.).

Pre-Cretaceous structural imprint on the subsurface rocks of the area is virtually unknown. Triassic and/or Jurassic phases of extension are indicated by a number of basaltoid dykes and sills in the Ramtha and Azab Groups, which did not intrude the Cretaceous, cover (Bandel and Khoury 1981). The structures most pertinent to groundwater flow probably originated during the late-Cretaceous to Eocene Syrian Arc folding episode and they were modified during the following Miocene to present-day deformation associated with shear on the N-S-trending Dead Sea Transform Fault (DSTF) which in the area runs close to the River Jordan. The Syrian Arc folding produced the following fold- and fault- structures exposed in the study area, between Wadi Zarqa and the eastern shore of the Dead Sea. They are listed from N to S: a) the NNE-trending Kafrein normal fault and associated faults bounding the Kafrein syncline to the W; b) the Kafrein asymmetric syncline (bCr ca -400 to –800 m); c) the NE- to ENE- trending Amman flexure and associated faults facing toward NW; d) the gently warped dip slope SE of the Amman flexure where bCr rises again to about +300 m a.s.l.

These structures are modified or enhanced by sets of normal faults trending NNW and NW, which may be regarded as syn- and antithetic to the subsiding Jordan Valley depression.

The network of wadis flowing down the western slopes of the Transjordanian Mountains toward the Jordan Valley depression seem to be fairly well controlled by the above structures of the Cretaceous rocks. For example wadis follow synclines (Wadi Shueib, Wadi Kafrein etc). They also follow a combination of structural dip with NW-trending antithetic faults on the dip slope SE of the Amman Flexure (Wadi Hisban/Ar Rama, Wadi al Muhtariqa, Wadi al Hiri E of Sweimeh). This may reflect some of the controls geologic structure exerts on subsurface flow paths of groundwater in the consolidated rock sequences. A detailed description of the study area, its geography, geology, hydrogeology can be found at Toll 2007.

2. DELINEATING THE COMPLEX GEOMETRY OF THE AQUIFER SYSTEMS

2.1 Base contour Map

On the base of a structure contour map (base Kurnub Group) provided by Dr. Till Heinrichs (Göttingen University) in the western area and the base Kurnub Group contour map, prepared by the BGR/WAJ (1994), for the eastern area, a 3-D geological model for the area of the consolidated rocks was prepared. In order to fit the borders of the study area both maps were adjusted and extended to the north and south. It should be noted, that the geological model is intended to represent the large surface water catchment area of Wadi Shueib, Wadi Kafrein, Wadi Hisban and the Dead Seas Wadis. Therefore only large structural features are included within this model. Small scale models, where e.g. small detachment faults or inclined faults etc. are included, require much more input data and a more detailed mapping program, which is beyond the focus of this study. The base of the Kunrub Group was chosen since it is the base of the lowermost aquifer modelled within

this study. The elevation of younger formations on top of the Kurnub are based on the Kurnub base structure contour map. *Figure 2* shows the subsurface catchment of Wadi Kafrein delineated from the base contour map of the Kurnub Group.

The western part, provided by Dr. T. Heinrichs, is based on the geological map of Lenz (1999), mapped at a scale 1 : 10,000, the geological map of Karameh, published by the National Resource Authority of Jordan (NRA), mapped at a scale of 1 : 50,000 (Shawabkeh 2001), and lithology logs of some wells in the area, provided by the Ministry of Water and Irrigation of Jordan (MWI). Due to the detailed mapping of Lenz (1999) a relatively detailed picture of the structural features around the western end of Wadi Kafrein could be attained. Future detailed mapping of the relatively complicated area would be desirable in order to get a better picture of the structural features.

The eastern part of the of the consolidated strata is based on the maps provided by BGR/WAJ (1994). This part of the structure contour map is mostly based on well log data. Only very large scale features can be seen on this map, since the density of well logs is poor, especially in the area between the Mountain Highland and the Jordan Valley.

The structure contour map was digitised and transformed via triangulated irregular networks (TIN) into a raster data set by using the commercial ArcGIS 9.0 software package (ESRI Inc.). In order to represent the large and steep structural features of the study area a cell size of 50 m was chosen.

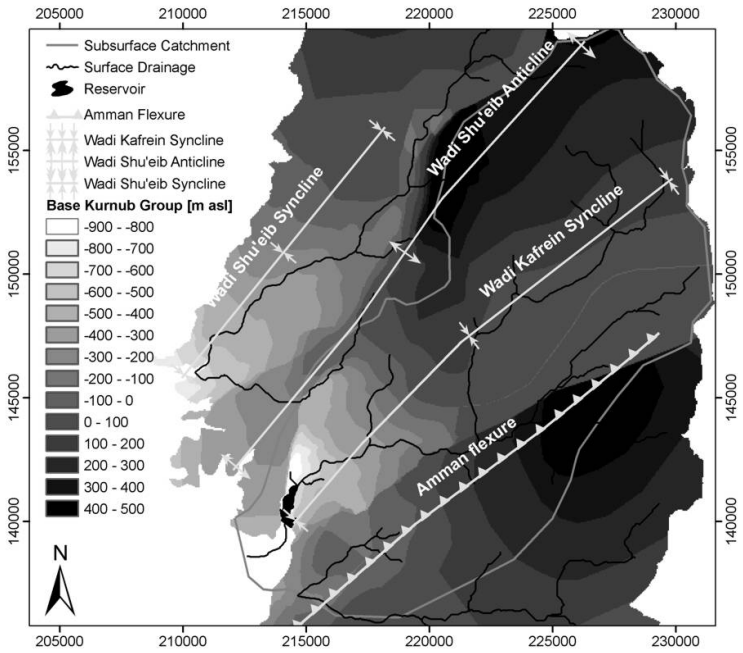


Figure 2: *Structure contour map of the base of the Kurnub Group.*

2.2 Construction of the remaining strata

Since the thickness of the different formations cannot be added perpendicular to the dip of the formation, the exaggerated thickness has to be calculated for every cell of the grid. The exaggerated thickness can be easily calculated by using the formula:

$$t' = t / \cos d \quad (1)$$

Where t' = exaggerated thickness, t = true thickness, and d = true dip of the unit. The true dip of every grid cell can be obtained by calculating the slope of the raster. When calculating the exaggerated thickness with the help of the raster calculator, the radian values of the different dip angles have to be used. A dip of up to 5° can be neglected since the exaggerated thickness almost equals the true thickness. By using the Raster Calculator of the Spatial Analyst Tool the exaggerated thickness value for each grid cell can easily be calculated and be added on top of the respective raster cell. Too little is known about faults in the area. For most of the wider area of

interest the shape, dip, and continuation through the different strata is either not known or insufficiently described, or drawings on the geological maps contradict each other. Only for some selected areas information exist. Therefore it was assumed, that faults mapped on the surface continue through every formation in a vertical direction. Furthermore it was assumed, that no thickness variation happens in areas of deformation. That is no thickness variation as a result of layer compressibility exists. The elevation added of each grid cell depends also on the thickness distribution of the different strata in the area.

2.3 Error discussion

One problem of the geological reconstruction described above is the quality of the structure contour map of the Kurnub Group. This can be clearly seen in *Figure 3*. On the left side of *Figure 3* the top view of the reconstructed geological map and on the right side the geological map published by the National Resources Authority of Jordan can be seen. It should be noted, that the white areas within the wider area of interest (*Figure 3* left) represent outcrop areas of formations older than the Lower Cretaceous Kurnub Group. Discrepancies can be seen mostly in the eastern part of the geological reconstruction. The eastern part of the reconstruction is based on the structure contour map compiled by the BGR WAJ (1994). This contour map is based on information of well data only and was used for a regional model of the geology of northern Jordan. Therefore its suitability is limited for a geological reconstruction of smaller areas. Taking existing geological maps and undertaking further field mapping will significantly increase the accuracy of the geological model. As a consequence, although the Amman Formation is the youngest hardrock formation, its reconstructed top is in some areas lower than the actual topography.

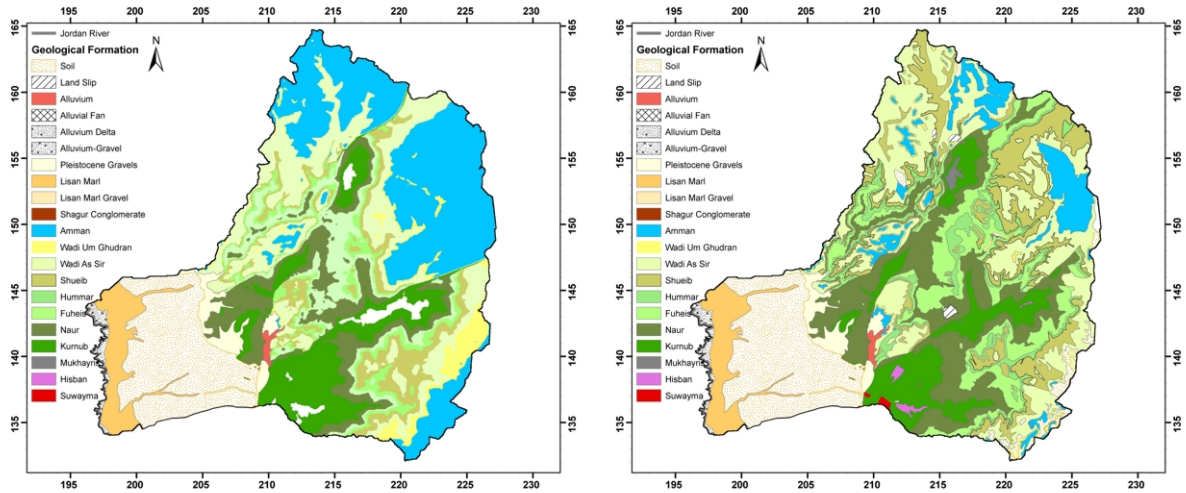


Figure 3: Left: *Results of the structural 3-D Model of some major wadis in the wider area of interest (based on the structure contour map of Figure 2). The empty areas are the outcrop areas of the Triassic Zerqa Main Group.* ***Right:*** *Geological Map of the modelled area (NRA open files).*

3. IMPLEMENTING THE COMPLEX GEOLOGICAL MODEL INTO A NUMERICAL 3-D FLOW MODEL

GeoSys/RockFlow scientific modeling software is used in developing a numerical groundwater flow model in the Wadi Kafrein area. GeoSys is an object-oriented program to simulate flow, mass, heat transport and deformation as well as chemical reaction processes in porous and fractured media (Wang and Kolditz 2007, Kolditz et al. 2008). The implementation of numerical methods involving geometric description of geosystems with the application of finite element analysis is of vital importance for the investigation and modeling of complex coupled nature or man-induced

processes. The main concept of GeoSys is the separation of geometrical, topological and physical data. More important, it also allows for an easy extension to more complex applications.

3.1. The 3D structural model

The 3D structural model is of significant importance in order to accurately simulate the aquifer systems and the groundwater flow. The construction of the 3D structural model for the area is challenging because of the complex geometry of the subsurface strata. The general procedure of developing the 3D structural model in GeoSys is as follows (Figure 4):

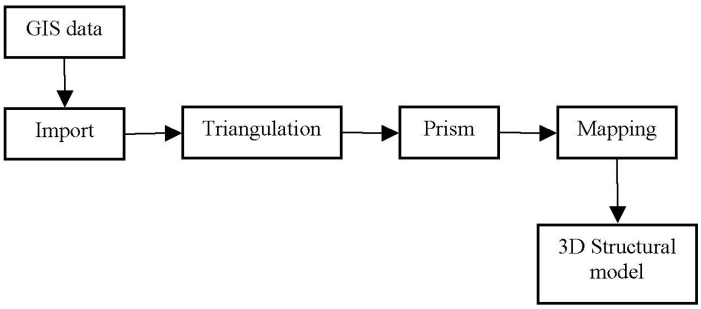


Figure 4*: The workflow of creating the 3D structural model.*

3.1.1 GIS data

All required GIS data for the modeling is converted to raster format using the ArcGIS software following the approach of Chen et al. 2005. The subsurface catchment and the flow path are saved as shape files, which serve as the geometric boundary of the modeling area and the assignment of the source terms (i.e. infiltration). However, the Digital Elevation Model (DEM) of the study area and the geological features of the layers for the study domain need to be converted into ASCII files for the mapping purpose.

3.1.2 Data import

At first, the shape files are imported into GeoSys. Therefore, the geometric objects including points, polylines, and surfaces are created and saved in a (gli) file. Based on the geometric objects (e.g. domain boundary), a finite element mesh has to be created.

3.1.3 Triangulations and Prisms

Based on the surface created from the subsurface catchment, triangulation is carried out to obtain the 2D mesh. Special attention was paid to coincide the nodes of the surface layer of the mesh with the valleys (drainage), since water will recharge the aquifer along these episodic streams. That means that the nodes must be created along the surface water flow path (Figure 5 left). Prismatic elements can be generated from 2D triangles by specifying the number of layers and layer thickness. For the study area, the geological formations are divided into eight hydrological units. Accordingly, prismatic elements for eight layers (Figure 5 right) are created and layer topography can be changed afterwards by using mapping techniques.

3.1.4 Mapping

The most demanding procedure is surface mapping. During this process the mesh is made to conform with stratigraphic irregulations, i.e. thickness and orientation of the mesh slices (Chen et al. 2009). It is also the basis for creating a successful 3D mesh for the subsurface modelling. As mentioned above, there are many areas which are marked as NODATA when raster data is converted into a ASCII file. The original mapping function designed to deal with continuously data did not work for such complex geological formations. Therefore, the mesh elements including

NODATA points should be deleted directly from the layers, because they do not exist in the real world.

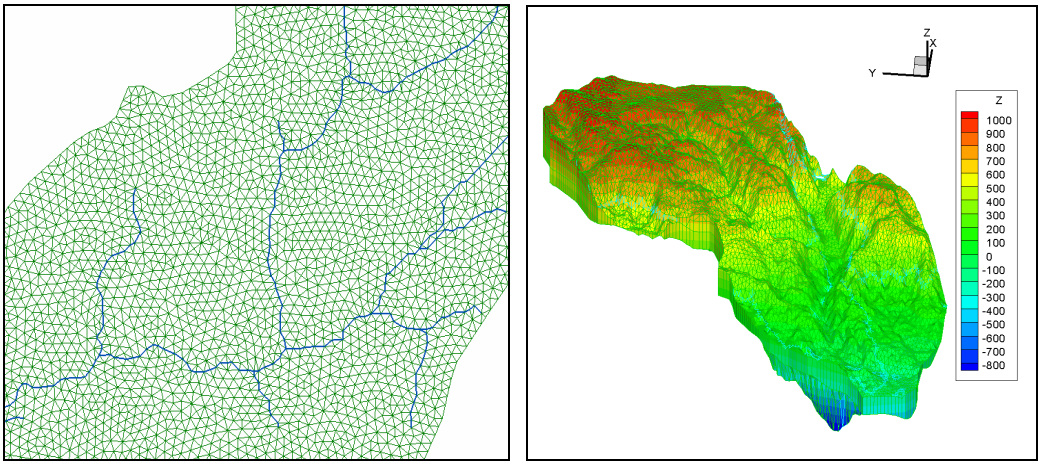


Figure 5: Left: *The 2D triangulation mesh of the Wadi Kafrein area.* **Right:** *The 3D prismatic and tetrahedral elements of the Wadi Kafrein area (unmapped model).*

The detailed description of the mapping implementation method is as follows. First, mapping is processed using the original mapping function. The elevations of nodes are interpolated from the raster data. In this step, the nodes are divided into two categories, the normal nodes and the NODATA nodes. The corresponding nodes are marked with true or false. True is defined where four raster points around a node have normal elevation values, false is defined at nodes where at least one raster point in the surrounding is a NODATA point. Secondly, some special measures are taken in order to check, remove and adjust involved elements including NODATA nodes. This is the most important part to obtain accurate mapping result. **1) Node reassigning:** It is assumed that all data inside of the catchment of the base layer exists. Beginning the loop from the bottom layer to the surface layer, when the node mark is detected as false, the value is replaced with the elevation of the layers under it. This is based on the fact that the elevation of all nodes are referencing the same zero surface. **2) Element checking, removing and adjusting**: After checking every element layer by layer, four different measures are taken to process these elements according to the number of the nodes needed to be deleted in a single element. That is to say, normal elements (no change), one node need to be deleted in a single element (the old prism element is adjusted to a tetrahedron element and a new tetrahedron element is created), two nodes need to be deleted in a single element (the old prism element is adjusted to a tetrahedron element), and three nodes need to be deleted in a single element (the old prism element is deleted). The exception of this type is that the node values of vertical counterparts do not exist in other layers except for the bottom layer and the surface layer. The elements of such type of nodes need to be processed very carefully. Finally, topology of the mesh is re-established based on the above described procedures of removal and adjustment.

Afterwards the eight layers are mapped to their corresponding elevations. As a result, a mesh file is created containing the node geometry and the element topology as well as the material group connected with these elements. This mesh file will be used as one of the input files for the simulations of the groundwater flow and transport processes.

3.2 Numerical flow model

Due to the scarcity of data it is difficult to reproduce a precise regional distribution of groundwater flow. Nevertheless, considering these limitations, groundwater models have been successfully developed for the lower Jordan Valley area (Hötzl et al. 2009).

As far as the material parameters are concerned, no pumping test data of the wells in the area exist. However, a literature study of the different hydrogeolgical formations for the area of middle and northern Jordan has been performed. The hydrogeological units and its characteristics applied

for the simulations were taken from Abu Ajamieh (1998), Margane et al (1999), Parker (1969), Salameh and Udluft (1985), and Toll (2007).

There are two kind of boundary conditions set in the groundwater flow model. A flux boundary condition is applied to all vertical nodes of the southwest area boundary and no-flow boundary conditions are set to other part of the catchment borders.

The recharge to the aquifers by infiltration of surface water is given by flux boundary conditions for all surface nodes. The average annual precipitation of the study area is assumed to be 300mm/yr, according to the available data of the different area distributions. The recharge of different area widely varies depending on rainfall distribution, topographical situation, soil cover, karstification, etc. Because no data was available for the recharge rate in the study area, an infiltration of 30% of the rainfall is assumed in the model. In order to reach a balance between infiltration and outflow, the corresponding outflow flux is estimated.

The hydraulic system depends upon the recharge and discharge conditions to or from the model area. Figure 5 right shows the hybrid elements. A preliminary simulation has been carried out. The next step for the groundwater modeling is to develop more realistic infiltration and recharge conditions based on observations.

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HIGH ALTITUDE TEMPERATURE VARIATIONS OVER THE LAST 50 YEARS IN GREECE

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Abstract: The scope of this study is to investigate the temperature (minimum, mean and maximum) trends in high altitude agricultural regions and forests of Greece, where no significant rural development has occurred since 1960. The variations of temperature values can affect plants phenological behaviour and growth. The dates of growth initiation or the length of the growing period are defined by the minimum temperature. On the other hand, the values of maximum and mean temperature have a direct effect on water availability, playing a significant role in growth and development rates especially during the water-stressed summer period in rain fed agricultural or natural ecosystems. Data available from 9 meteorological stations at altitudes 520 – 1310 m in Greece, covering the time period 1960 – 2006, are analyzed for temperature variability. The stations are located in agricultural and forest environments with no significant land use changes over the study period. Trends and slopes of minimum, maximum and mean temperature are calculated on a monthly as well as a yearly basis for different levels of confidence. Generally, significant yearly decreases of mean temperature were observed. Averaged for all sites, the temperature decrease rate is – 0.021 oC y-1. In all months and seasons, the average trends of mean temperature are also negative, with November becoming coolest (– 0.069 oC y-1) and autumn showing the greatest temperature decrease (– 0.038 oC y-1). In comparison with mean temperature, an even greater decreasing rate is observed for minimum temperature (– 0.027 oC y-1). Monthly minimum temperature trends are similar to those of mean temperature. On a seasonal basis, however, it is in spring that minimum temperature shows maximum negative trend (– 0.040 oC y-1). On the contrary, maximum temperature has a positive annual trend, with a site-average of + 0.011 oC y-1. The greatest increasing rate (+ 0.037 oC y-1) is observed in summer with June becoming warmest. In some months, even maximum temperature becomes cooler with November, again, becoming coolest (– 0.054 oC y-1). Conclusively, high altitude sites in Greece generally become cooler, with reference even to maximum temperature in most cases.

Key words: Mann Kendall, Sen slopes, temperature, trends.

1. INTRODUCTION

Temperature increase and global warming is considered to be a universal phenomenon (Keim et al. 2003, Trombulak and Wolfson 2004, Chmielewski et al. 2004, Todisco and Vergni 2008) although there are cases where opposite variation is recorded (Gadgil and Dhorde 2005, Giles and Flocas 1984, Makroyannis et al. 1998). According to the Third Assessment Report of the Intergovernmental Panel on Climate Change (IPCC, 2001), the global average surface temperature has increased by about 0.6 ± 0.2 ^{o}C over the 20th century, when rainfall appears to decrease in the Northern Hemisphere. Climate change as an impact of urbanization and population growth (Jones et al. 1990, Fujibe 1995, Karaca et al. 1995) is more intense in major cities (Wigley and Farmer 1982, Colacino and Rovely 1983, Karaca and Aslan 1993, Founda et al. 2004). In Greece such increasing rates in temperature is confined in rural developed areas (Founda et al. 2004). In natural ecosystems and agricultural regions, climate change appears to differ, being affected less by urbanization and land use changes (Proedrou et al. 1997, Philandras et al. 1999).

In this work a study of temperature data from 9 high altitude (greater than 500 m) agricultural and forest meteorological stations is attempted, through the statistical investigation of their trends and slopes. Average minimum, maximum and mean values of temperature play a significant role in

vegetation development and therefore their changes can affect the species populations in plant societies, along with their spatial and temporal distribution.

2. MATERIALS AND METHODS

Commonly used meteorological data were established for this study. The monthly values of mean (T_{mean}), mean maximum (T_{max}) and mean minimum (T_{min}) temperatures, from 9 meteorological stations at different altitudes (520 – 1310 m) in Greece, were analyzed on a monthly, seasonal (winter: December – February, spring: March – May, summer: June – August and autumn: September – November) and annual basis. The time series cover the period 1960 – 2006. Where gaps in data, no seasonal or annual averages were extracted. The geographical characteristics of the meteorological stations along with their vegetation coverage are presented in Table 1, whereas the mean climatic conditions for each site in Figure 1. All meteorological stations, either forest or agricultural, are located away from highly populated cities, where significant urbanization and land use changes have occurred since 1960.

Table 1. Meteorological stations geographical characteristics and their vegetation coverage. The stations are operated either by the [1]National Agricultural Research Foundation or by the [2]National Meteorological Service of Greece.

Station	Latitude	Longitude	Altitude	Vegetation coverage
Arnea[1]	40° 29’ 35’’	23° 36’ 09’’	520 m	*Querqus frainetto*, *Fagus sylvatica*
Artemisia[1]	37° 05’ 49’’	22° 14’ 04’’	710 m	*Pinus nigra* ssp *nigra*
Krania[1]	39° 53’ 51’’	21° 17’ 02’’	930 m	*Pinus nigra* ssp *nigra*
Vamvakou[1]	37° 14’ 42’’	22° 33’ 17’’	970 m	*Abies cefalonica*
Fournas[1]	39° 03’ 25’’	21° 53’ 16’’	1100 m	*Abies borisii-regis*
Metsovo[1]	39° 47’ 07’’	21° 09’ 34’’	1310 m	*Pinus nigra ssp nigra*
Aspropotamos[1]	39° 38’ 57’’	21° 20’ 25’’	1200 m	*Abies borisii-regis*
Tripolis[2]	37° 32’ 05’’	22° 24’ 19’’	652 m	*Agricultural crops*
Kalavrita[2]	38 ° 02’ 05’’	22° 06’ 10’’	731 m	*Agricultural crops*

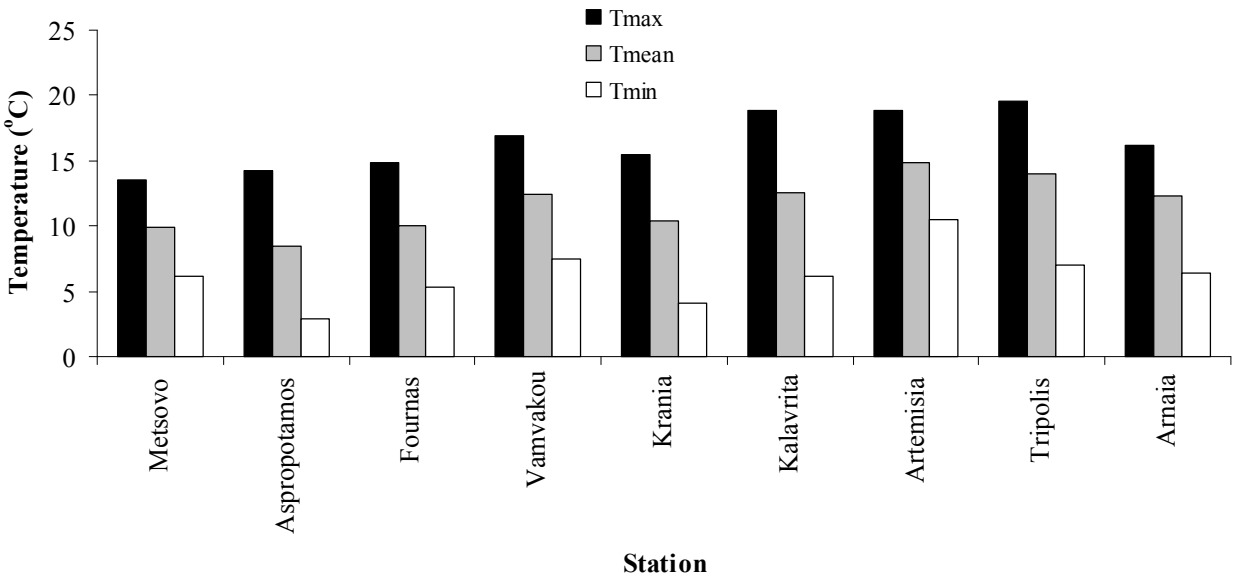


Figure 1. Annual mean temperatures (minimum, mean and maximum) in nine high altitude (520-1310 m) meteorological sites of Greece.

In order to define trends for each parameter the non parametric Mann-Kendall test (Mann 1945, Kendall 1975) for different confidence levels (α=0.001, 0.01, 0.05 and 0.1), is used. The Mann-Kendall test is widely used for climatic and environmental time series trends evaluation (Hirsch et al. 1982, van Belle and Hughes 1984, Zetterqvist 1991, Zhang et al. 2001, Yue and Wang 2002, Gemmer et al. 2004, Zhang et al. 2006, Chen et al. 2007, Proutsos et al. 2008, Tigkas 2008),

because as a non-parametric method requires only independence of the data and is tolerant to outliers. It is a reliable method to identify monotonic linear and non-linear trends in non-normal data sets (Helsel and Hirsch, 1992). The slope of each trend evaluated from the climatic data, is identified by the method of Sen (here referred as Q Sen slope), as a median of all possible slopes (Sen 1968, Helsel and Hirsch 1992). The variability of trends and slopes with altitude is also examined and a further investigation is conducted through the categorization of the stations as north (Arnea, Krania, Fournas, Metsovo and Aspropotamos) and south (Artemisia, Vamvakou, Tripolis and Kalavrita).

3. RESULTS AND DISCUSSION

3.1 Annual and seasonal variations of temperature

On an annual basis, the average slopes, from data available for all sites, indicate generally negative trends in T_{min} and T_{mean} with slopes – 0.021 and – 0.027 $^{o}C\ y^{-1}$, respectively. On the other hand, the averaged slopes of T_{max} appear to be positive (+ 0.011 $^{o}C\ y^{-1}$). Specifically, according to the geographical site position, in Northern Greece the T_{mean} and T_{min} slopes have much greater negative values than the ones in Southern Greece (Figures 2 and 3). Though the T_{max} averaged slope in Northern Greece is slightly negative (– 0.013 $^{o}C\ y^{-1}$), positive trends are recorded in South (+ 0.040 $^{o}C\ y^{-1}$).

The T_{min} and T_{mean} trends (Table 2) generally indicate negative slopes. For T_{min}, specifically, negative slopes characterize six stations, four of which are statistically significant even at a level of confidence greater than $\alpha = 0.001$. For T_{mean}, respectively, negative slopes were calculated for seven stations and in fact significantly (level of confidence greater than $\alpha = 0.05$) for four of them. T_{max} trends, on the contrary, were positive in four stations, in two of which significantly ($\alpha = 0.01$).

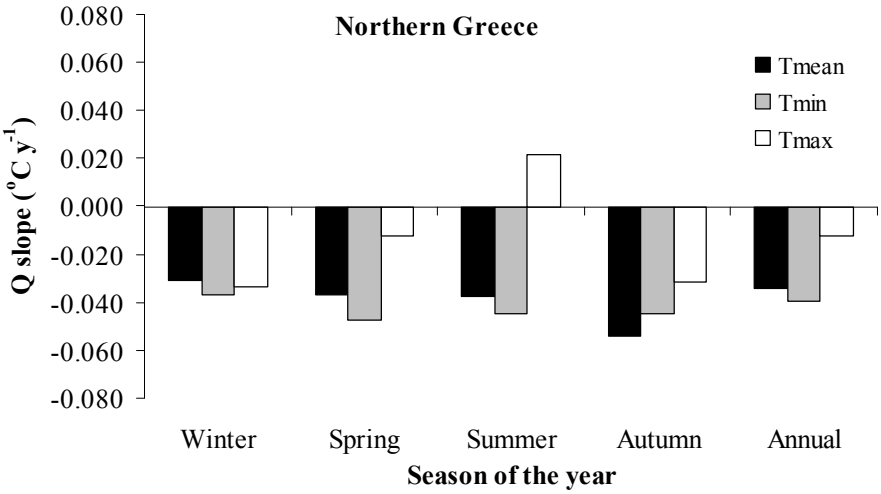


Figure 2. Average seasonal and annual variation of Sen slopes (Q) of mean, maximum and minimum temperature from 5 meteorological stations in Northern Greece.

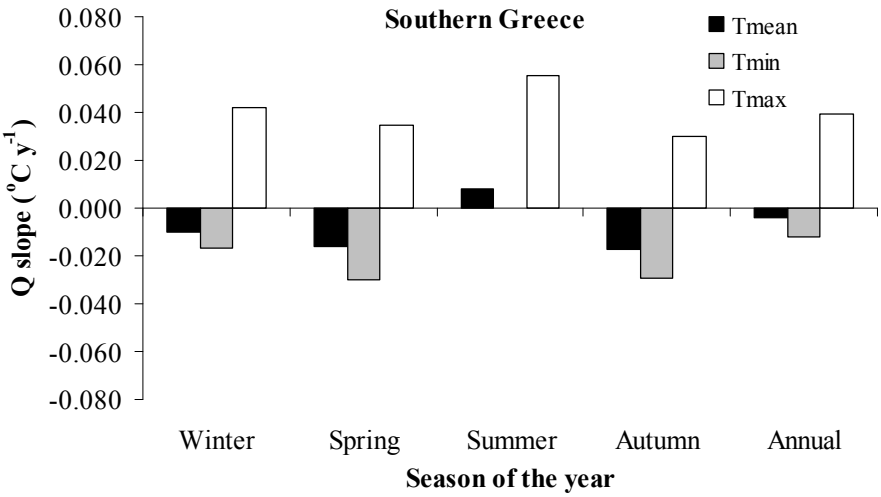


Figure 3. Average seasonal and annual variation of Sen slopes (Q) of mean, maximum and minimum temperature from 4 meteorological stations in Southern Greece.

Table 2. Seasonal positive (+), negative (-) and unvaried (0) trends of minimum, mean and maximum temperatures in 9 forest-agricultural meteorological stations at different significant levels (a= 0.001, 0.01, 0.0 and 0.10). The 0 trend refers to absolute slopes less than 0.009 °C y-1.

Station	Altitude (m)	Winter			Spring			Summer			Autumn			Annual		
		T_{min}	T_{mean}	T_{max}	T_{min}	T_{mean}	T_{max}	T_{min}	T_{mean}	T_{max}	T_{min}	T_{mean}	T_{max}	T_{min}	T_{mean}	T_{max}
Metsovo	1310	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Aspropotamos	1200	-	-	-	-	-	-	-	-	0	-	-	-	-	-	-
Fournas	1100	+	-	0	+	-	-	+	-	+	0	-	-	+	-	0
Vamvakou	970	-	-	0	-	0	+	0	0	0	-	-	-	0	0	0
Krania	930	-	-	-	-	-	+	-	-	+	-	-	0	-	-	+
Kalavrita	731	0	0	+	-	-	+	-	0	+	-	-	+	-	-	+
Artemisia	710	0	0	+	+	+	+	+	+	+	+	0	+	+	+	+
Tripolis	652	-	-	0	-	0	+	-	0	+	-	-	0	-	-	0
Arnea	520	-	-	0	-	-	+	0	0	+	-	-	-	-	-	+

α=0.001 α=0.01 α=0.05 α=0.10

On seasonal basis, since 1960, T_{mean} and T_{min} generally show decreasing rates throughout the year, whereas T_{max} remains almost unvaried in all seasons except summer, when high positive trends indicate warming at an average rate + 0.037 °C y^{-1}. Specifically, in Northern Greece (Figure 2), with regard to T_{mean}, the greatest cooling rates appear in autumn (– 0.054 °C y^{-1}), whereas reference to T_{min} in spring (– 0.047 °C y^{-1}). Finally, the average seasonal slopes of T_{max} indicate greatest cooling in winter (– 0.034 °C y^{-1}) and greatest warming in summer (+ 0.022 °C y^{-1}). In Southern Greece (Figure 3), T_{mean} slopes are negligible or slightly negative in all seasons. Spring has the greatest negative T_{min} slope. T_{max} slopes are positive all year round and highest is summer (+ 0.055 °C y^{-1}).

Also shown in Table 2 are the station-specific trends for each season. Generally, T_{max} positive trends are identified in spring and summer, whereas negative T_{min} and T_{mean} trends in almost all seasons and in most of the stations. All temperature parameters show negative signs in the stations with the highest altitudes (Metsovo, Aspropotamos). On the other side, the southest Artemisia shows generally positive trends, especially in spring and summer.

3.2 Monthly variations of temperature

The mean monthly slopes of T_{mean}, T_{min} and T_{max} indicate most decreasing rates in November (– 0.069, – 0.063 and – 0.054 $^{o}C\ y^{-1}$, respectively). The mean slopes of T_{max} in summer months are positive with greatest rates in June (+ 0.048 $^{o}C\ y^{-1}$) and July (+ 0.044 $^{o}C\ y^{-1}$). Northern Greece temperature trends do not differ from the general trends (Figure 4), although in Southern Greece (Figure 5) T_{mean} is decreasing in winter, with the greatest negative slope in November. However, the summer months present positive trends, with the greatest slope in July (+ 0.022 $^{o}C\ y^{-1}$). Respectively, the mean slopes of T_{min} are negative in almost all months, being most negative in November. The T_{max} mean slopes are positive in all months and become maximum in July (+ 0.076 $^{o}C\ y^{-1}$).

Trends of T_{mean}, T_{max} and T_{min} for each station at different levels of confidence are presented in Tables 3, 4 and 5. The most significant ($\alpha = 0.001$) trends for T_{mean} were detected in November in four of the five N-stations and were all negative. T_{min} monthly trends at a high significant level (α=0.001) appear seven months of the year in Metsovo (1310 m), indicating a gross minimum temperature decrease in the region since 1960. A T_{max} increase is detected for almost all months in Artemisia (the southest site), and is most significant ($\alpha = 0.05$) during the water stressed period (May to October), leading to continuously dryer conditions since 1960 (Proutsos et al. 2008).

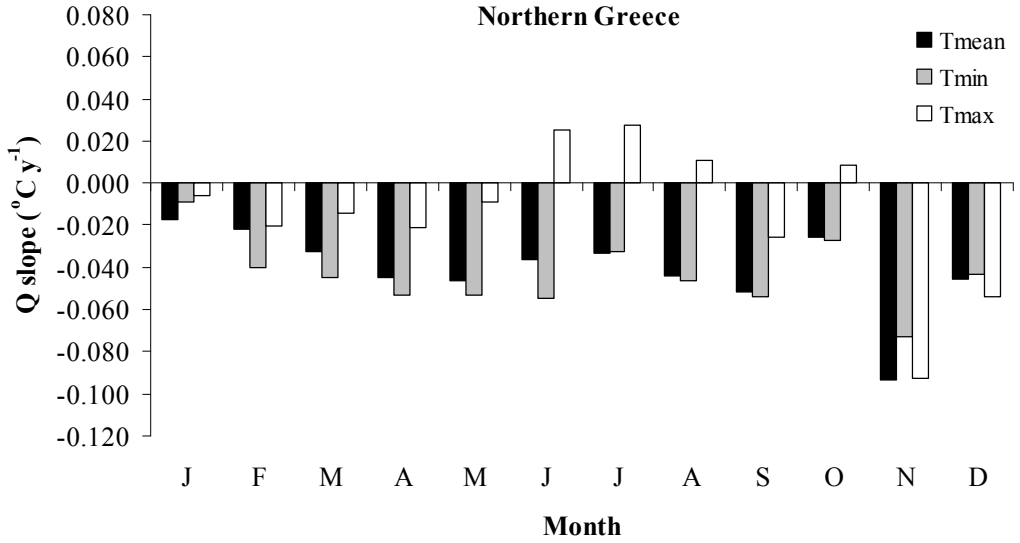


Figure 3. Average monthly variation of Sen slopes (Q) of mean, maximum and minimum temperature from 5 meteorological stations in Northern Greece.

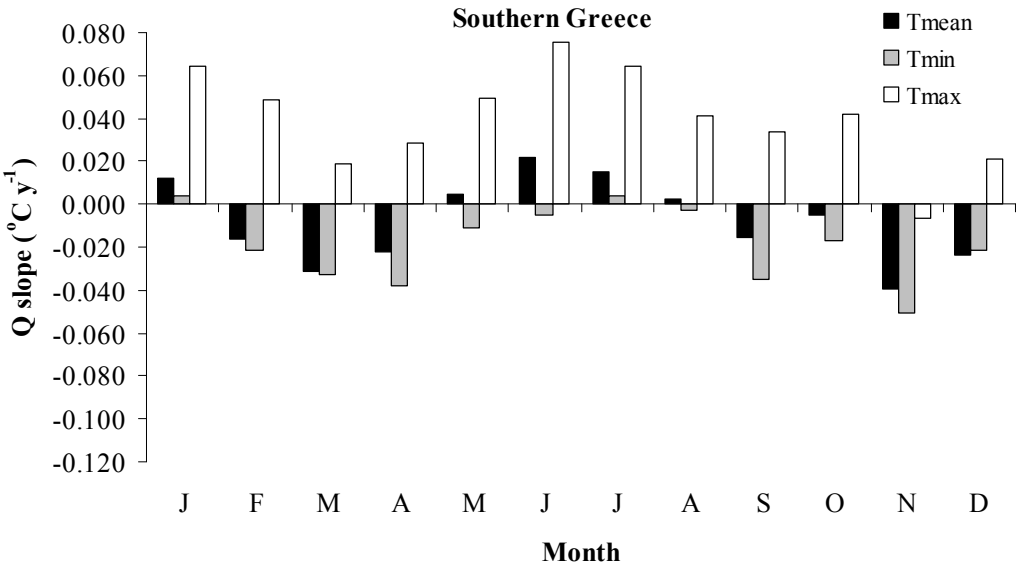


Figure 4. Average monthly variation of Sen slopes (Q) of mean, maximum and minimum temperature from 4 meteorological stations in Southern Greece.

Table 3. Monthly positive (+), negative (-) and unvaried (0) trends of mean temperatures in 9 forest-agricultural meteorological stations at different significant levels (a= 0.001, 0.01, 0.0 and 0.10). The 0 trend refers to absolute slopes less than 0.009 $^{o}C\ y^{-1}$.

Station	Altitude (m)	Month											
		J	F	M	A	M	J	J	A	S	O	N	D
Metsovo	1310	-	-	-	-	-	-	-	-	-	-	-	-
Aspropotamos	1200	-	-	-	-	-	-	-	-	-	-	-	-
Fournas	1100	0	0	-	-	-	-	-	-	-	-	-	-
Vamvakou	970	0	-	0	-	-	0	+	-	-	-	-	-
Krania	930	-	-	-	-	-	-	0	-	-	0	-	-
Kalavrita	731	+	-	-	-	-	+	0	+	-	-	0	0
Artemisia	710	0	0	0	-	+	+	+	+	+	+	-	-
Tripolis	652	0	-	-	0	+	+	0	-	0	-	-	-
Arnea	520	0	-	0	-	-	-	-	0	-	-	-	-

α=0.001 α=0.01 α=0.05 α=0.10

Table 4. Monthly positive (+), negative (-) and unvaried (0) trends of minimum temperatures in 9 forest-agricultural meteorological stations at different significant levels (a= 0.001, 0.01, 0.0 and 0.10). The 0 trend refers to absolute slopes less than 0.009 $^{o}C\ y^{-1}$.

Station	Altitude (m)	Month											
		J	F	M	A	M	J	J	A	S	O	N	D
Metsovo	1310	-	-	-	-	-	-	-	-	-	-	-	-
Aspropotamos	1200	-	-	-	-	-	-	-	-	-	-	-	-
Fournas	1100	+	+	+	+	0	0	+	-	+	0	-	-
Vamvakou	970	-	-	-	-	0	0	+	0	0	+	-	-
Krania	930	-	-	-	-	-	-	-	-	-	-	-	-
Kalavrita	731	+	-	-	-	-	-	-	-	-	-	-	0
Artemisia	710	+	+	+	0	+	+	+	+	+	+	-	0
Tripolis	652	-	-	-	-	0	-	-	-	-	-	-	-
Arnea	520	0	-	-	-	-	0	0	+	-	0	-	-

α=0.001 α=0.01 α=0.05 α=0.10

Table 5. Monthly positive (+), negative (-) and unvaried (0) trends of maximum temperatures in 9 forest-agricultural meteorological stations at different significant levels (a= 0.001, 0.01, 0.0 and 0.10). The 0 trend refers to absolute slopes less than 0.009 $^{o}C\ y^{-1}$.

Station	Altitude (m)	Month											
		J	F	M	A	M	J	J	A	S	O	N	D
Metsovo	1310	-	-	-	-	-	-	-	-	-	-	-	-
Aspropotamos	1200	-	-	-	-	0	0	+	-	-	0	-	-
Fournas	1100	+	0	-	0	-	+	+	0	+	0	-	-
Vamvakou	970	+	-	+	+	0	+	+	-	-	0	-	0
Krania	930	0	0	+	0	+	+	+	+	0	+	-	-
Kalavrita	731	+	+	0	+	+	+	+	+	+	+	+	+
Artemisia	710	+	+	+	+	+	+	+	+	+	+	-	+
Tripolis	652	+	0	0	+	+	+	+	0	+	+	-	-
Arnea	520	+	+	+	-	0	+	+	+	-	+	-	-

α=0.001 α=0.01 α=0.05 α=0.10

3.3 Altitudinal variation of mean temperature slopes

The mean temperature slopes can be well correlated with altitude (Table 6). Best correlation coefficients were extracted for August and June slopes, but satisfactory correlations were found for the seasonal values in autumn and summer.

Generally, the values of mean temperature slopes appear to decrease with altitude. For the annual values, the average rate is 0.0066 $^{o}C\ y^{-1}$ decrease for every 100 m altitude increase. The slopes become zero at altitudes around 500 m. At lower altitudes the slopes generally become positive (data not shown).

Table 6. Coefficients from altitude and mean temperature Sen slopes correlation.

Month	**J**	**F**	**M**	**A**	**M**	**J**	**J**	**A**	**S**	**O**	**N**	**D**
R^2	0.34	0.15	0.08	0.22	0.44	0.63	0.50	0.70	0.57	0.32	0.48	0.29
Season	**Winter**		**Spring**		**Summer**		**Autumn**				**Annual**	
R^2	0.49		0.17		0.67		0.72				0.50	

4. CONCLUSIONS

The following general conclusions can be drawn for the temperature behavior during the latest half century in high-altitude regions:

- On an annual basis, T_{mean} and T_{min} were decreasing and T_{max} increasing. With respect to space variability, in Northern Greece, T_{mean} and T_{min} decreased and T_{max} stayed almost unvaried. In Southern Greece, the T_{mean} and T_{min} slopes were not significant but T_{max} appeared to increase.
- On seasonal basis, T_{mean} and T_{min} decreased in all seasons but T_{max} remained rather constant, except for summer, where it increased. In Northern Greece, T_{mean} and T_{min} were decreasing throughout the year, whereas T_{max} was increasing. However, in Southern Greece T_{max} increased in all seasons, with the greatest slopes appearing in summer.
- On a monthly basis, the most decreasing rates of temperature were recorded in November. In Northern Greece, greatest negative slopes were found in November for all temperature parameters. T_{min} and T_{mean} decreased all year round, but T_{max} showed positive trends, greatest in June and July. On the other hand, in Southern Greece T_{mean} in summer was increasing.
- The T_{mean} slopes are well correlated with altitudes, diminishing when altitude increases. High correlations were found for June, July and August. On a seasonal basis, summer and autumn are best correlated, whereas the annual correlation coefficients can be considered as satisfactory.
  High-altitude temperature behavior is rather opposite to temperature trends at lower region.

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PRACTICABILITY OF INTEGRATED WATER RESOURCES MANAGEMENT (IWRM) CONCEPTS IN ARID AND SEMIARID REGIONS

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Abstract: Water is a limited natural resource and a public good fundamental for life and health. This essential sentence of UN human rights entitles everyone to sufficient, safe, acceptable and affordable water for personal and domestic use. However, in semiarid and especially in arid areas the general natural scarcity of water restricts the availability of these resources tremendously. While in the past the people, living in arid areas as relatively small group of mainly nomads, were adapted from their whole way of life to these conditions, now the freshwater resources are under increasing pressure from partly extreme population growth, economic activities and intensifying competition for available water resources among users. Exacerbating the problem, human activities often contaminate the limited freshwater resources, making them unavailable for further human use and threatening the health of the river, aquifers and ecosystems. The above problems are intensified by shortcomings in water management, which together with lack of regulation leads to mismanagement of the resources.

To support the suffering countries, the concept of IWRM was developed and attracted particular attention following the international conferences on water and environmental issues in Dublin 1992. IWRM is a systematic process for the sustainable development and management of water, land and related resources in the social, economic and environmental context. Integrated management means that all the different uses of water resources are considered together. However IWRM has neither been unambiguously defined nor has the question of how it is to be implemented been fully addressed. Particularly for arid areas the question of sustainability has to be raised with regard to the scarce and partly not replenished resources. Socio-economic evaluations have to be carried out to define priorities between the opposed interests of urban and rural population as well as between the competitive ambitions of agriculture, industry and tourism. What has to be integrated and how is it best done? The report will try to show the options, but also the limitations of the water management of the Lower Jordan basin, where an exploding population on one hand and on the other hand the increasing drought conditions due to climate changes cause a situation, where the natural resources are completely insufficient.

Key words: IWRM, arid, semiarid, SMART.

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QUANTITY AND QUALITY ASSESSMENTS OF KAFREIN RESERVOIR IMPACT ON GROUNDWATER SYSTEM

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Abstract: The Wadi Kafrein catchment represents one of the drainage systems extending from the high mountain region around Amman down westwards to the Jordan Valley. Wadi Kafrein has two major tributaries: Wadi Es Sir and Wadi Na'ur. These wadis have permanent base flow supported by many springs and seepages scattered in the catchment area, the floods during winter and by the treated waste water which is discharging from Wadi Es Sir Treatment Plant. The whole catchment area is drained by Wadi Kafrein stream (a perennial stream) which is barred at the emergence to the Jordan Valley by the Kafrein reservoir. Kafrein reservoir was constructed on Wadi Kafrein in 1968, it intended to store winter flows for downstream irrigation as well as to recharge the groundwater system. This study as part of SMART Project is aimed to assess the quality and quantity impacts of the reservoir stored water (different qualities) on groundwater system down stream. About 45 water samples were taken from surface and groundwater sources in the catchment area and analysed for chemical composition and isotope contents. The online long term monitoring of groundwater level and the Kafrein reservoir storage records were studied. The results show that the stored water in Kafrein reservoir is a mixture of three types of water; sulphate, chloride and bicarbonate water and it can be classified as chloride water in winter and early summer but it becomes sulphate water in late summer just before rain starts. While the groundwater down stream of the reservoir is of bicarbonate water type in the shallow aquifer and chloride water in the deep aquifer. The stored water quality meets the Jordanian standards for artificial groundwater recharge and the infiltrated water shows minor impact on groundwater quality. The analysis of the dynamics of groundwater level variations and the Kafrein reservoir storage shows a direct relation, pointing to a direct recharge from the reservoir to groundwater system. The calculation of the reservoir water budget during the years 2003-2008 suggests that around 5 MCM/year in average is entering the groundwater system from the reservoir. In the framework of integrated water resource management, the Kafrein reservoir demonstrates the benefits for managed aquifer recharge in maintaining quality and quantity of groundwater resources.

Key words: Artificial recharge, Infiltration, Kafrein Reservoir, Quality and quantity impacts, SMART.

1. INTRODUCTION

The Hashemite Kingdom of Jordan is facing a chronic imbalance in the water demand-water resources equation. The total available water resources in 2007 for all uses amount to 867 MCM, while the water demand was 1526 MCM. The annual per capita water availability was 160m3/y in 1997 and decreased to145 m3/y in 2007. This is far below the international poverty line which amounts 500 m3/y. The water supply in 2007 for all uses was 940 MCM and the water resources-water supply deficit (73MCM) was covered mainly by over pumping of groundwater resources (MWI 2008).

Groundwater is considered to be the major source of water in Jordan, and the only source in some areas. Some of the renewable groundwater resources are presently exploited to their maximum capacity and in some cases beyond their safe yield. Overexploitation of groundwater aquifers, in excess of the annual recharge, has contributed significantly to the degradation of the water quality in the exploited aquifers and endangers the sustainability of these resources for future use. Because of the limited availability, water resources have become one of the main government priorities and development targets with major efforts on utilizing it in the most efficient and productive way. Various measures were taken to ease the wide-spread shortages. These include the

construction of reservoirs on wadis with relatively high rainfall to enhance irrigation, to recharge the groundwater system and to mitigate the ground water depletion. One of these reservoirs is the Kafrein Reservoir, which is located at Wadi Kafrein mouth in the Jordan Valley.

The Wadi Kafrein catchment represents one of the drainage systems extending from the high mountain region around Amman down westwards to the Jordan Valley with total catchment area of about189 km2 (Figure 1). The elevation varies from about 1000 masl in the north-eastern parts to 200 mbsl at the outlet site in the southwest. Wadi Kafrein has two major tributaries: Wadi es Sir and Wadi Na'ur. These wadis have permanent base flow supported by many springs and seepage scattered in the catchment area and by the floods during winter.

In 1996, the Wadi Es Sir Waste Water Treatment Plant was built in Wadi Kafrein to serve all communities in western Amman living within Wadi es Sir catchment with full capacity of 4000 m3/d. Currently about 3000m3/d of treated waste water is discharging into Wadi Kafrein and it mixes with the base flow of Wadi es Sir and Wadi Na'ur. The whole catchment area is drained by Wadi Kafrein (a perennial stream) which is barred at the emergence to the Jordan Valley by the Kafrein Reservoir.

The Kafrein Reservoir was constructed in 1968 and in 1996 its storage capacity was raised to 8.5 MCM. The area of the reservoir is about 1km2 and has north-south orientation. The reservoir intended to store winter flows for downstream irrigation as well as to recharge the groundwater in the area. The water for irrigation is taken from off-takes in the valve tower and passes via twin 500mm diameter pipelines through a concrete lined tunnel to the Hisban - Kafrein irrigation project. Al Batous pumping station was constructed in 1978 to collect the seeping surface water from the reservoir and pump the excess water after irrigation needs back to the reservoir.

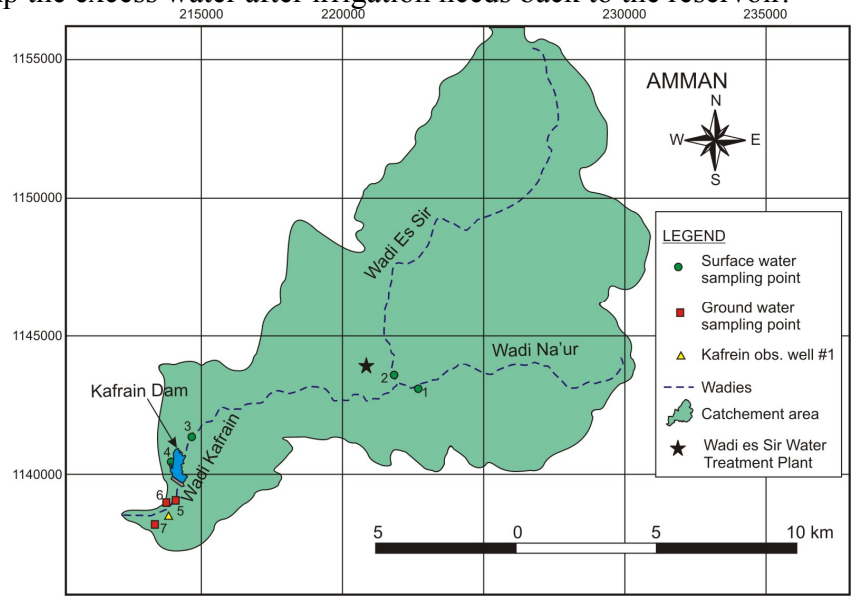


Figure 1. Wadi Kafrein Catchment

2. WATER QUANTITY ASSESSMENTS

The records of Kafrein Reservoir storage since the year 2000 and the water level fluctuations records of the Kafrein observation well-1 down stream (MWI data base 2008) were used to assess the quantity impact of the stored water on the groundwater in the area.

2.1 Kafrein Reservoir Storage

The stored water records of Kafrein Reservoir during the years 2000-2008 showed that the highest total storage (8.2 MCM) was recorded in the wet year 2003 when the reservoir was almost full for about two months and the lowest was in the year 2001 when the reservoir was dry for about

four months of the year. The stored water in the reservoir starts to increase in December as the rain season starts and reaches its maximum in April each year. After the rain stopped (April-May), the stored water starts to decrease due to evaporation, infiltration and irrigation activities down stream. The reservoir storage fluctuations are shown in figure 2.

2.2 Kafrein Observation well-1

Kafrein Observation Well was drilled down stream of Kafrein Reservoir and equipped with water level recorder to monitor groundwater level fluctuations since February 2000. The recorded data showed that the highest water level was recorded in April 2002 and April 2004 while the lowest was recorded in November 2004. The water level fluctuation during the whole period of records is described in figure 3.

2.3 Reservoir Storage and Groundwater level Relation

During the years 2000-2008, the reservoir storage and the groundwater level records show in general a similar trend on both monthly and yearly bases fluctuations. In wet years when the reservoir storage is high and there is enough water pumped from the reservoir into Hisban-Kafrein irrigation system down stream, the farmers do not need to pump groundwater from their own wells. So, the ground water level increased or at least kept its level. On the contrary, in the dry years when the reservoir storage was low and no enough water for irrigation, the farmers pumped groundwater for irrigation and lowered the water level. The comparison between the average of reservoir storage and the groundwater level in Kafrein observation well-1 shows a direct relation, pointing to direct recharge from the reservoir into the groundwater (Figure 4).

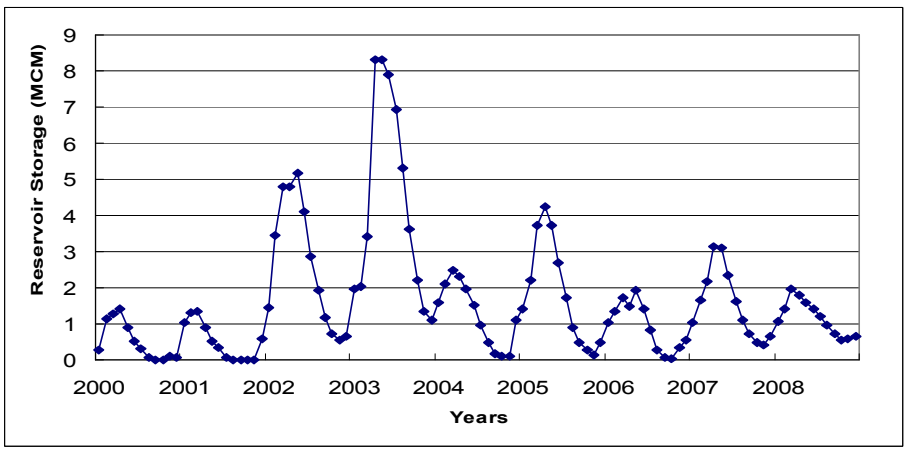


Figure2. Kafrein Reservoir storage fluctuations

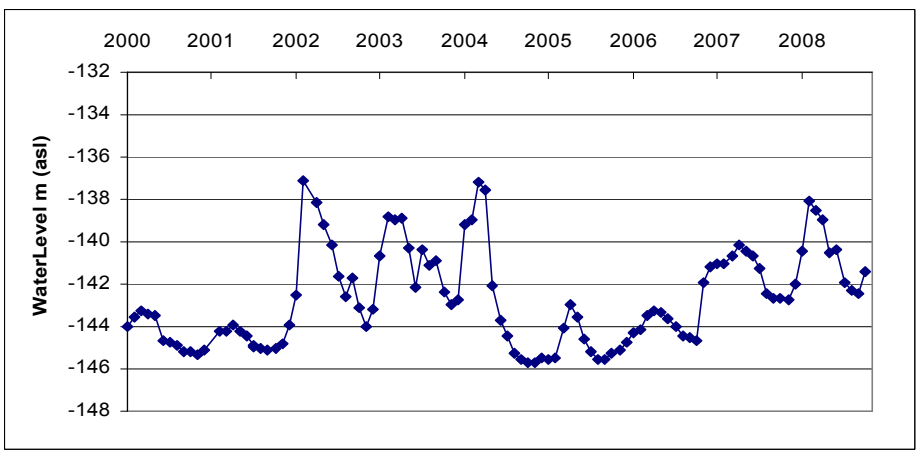


Figure 3. Water level records of Kafrein Observation Well-1.

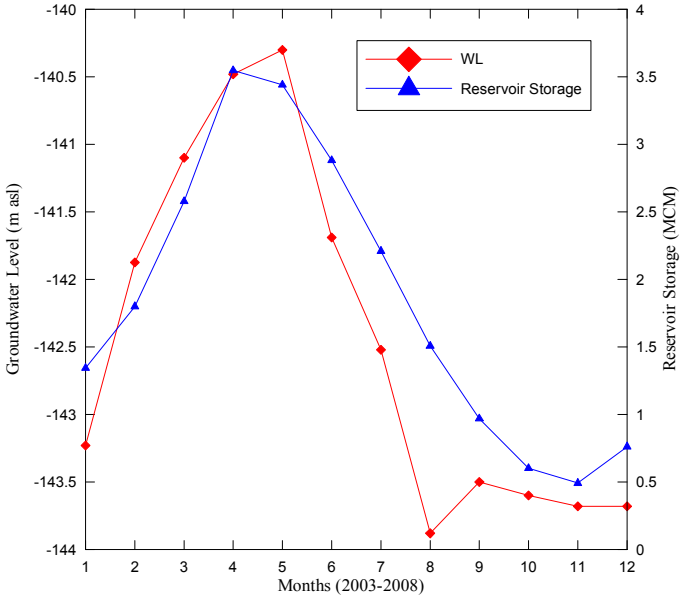


Figure 4. The average correlation between reservoir storage and groundwater level l in Kafrein Observation Well-1

2.4 Kafrein Reservoir Detailed Water Budget

Kafrein reservoir water budget can be calculated according to the change in reservoir storage. The total inflow to the reservoir consists of inflow from Kafrein Stream (Current meter), rain and pumped water from al Batos station back to the reservoir (Flow meter). The total outflow from the reservoir includes evaporation (Evaporation Pan), pumping from the reservoir for irrigation (Flow meter), surface seepage from the reservoir (V-notch weir) and infiltration into groundwater (Calculated). The daily records of all components of the reservoir water budget are available in JVA data base 2008. The data of the years 2003-2008 is used to calculate the monthly and the yearly infiltration amounts from the reservoir to the groundwater down stream (Table 1). The year 2003 has the highest infiltration amount (7.4 MCM) while the year 2006 has the lowest (3.5 MCM). March has the highest values of infiltration except in 2003 May has the highest. The average infiltration for the period is 5.3 MCM/year.

Table1. Monthly and yearly infiltration amounts during the years 2003-2008 (m^3)

Date	Yr 2003	Yr 2004	Yr 2005	Yr 2006	Yr 2007	Yr 2008	Average
January	782424	387112	732939	392726	488844	556582	556771
February	664480	669635	827632	559063	895205	735988	725334
March	800312	985353	1037603	623067	991540	795985	872310
April	818882	598025	1025622	657675	984719	516881	766967
May	837452	520849	1002588	529523	813729	367224	678561
June	811384	415351	598140	272094	671849	338032	517808
July	729186	163498	198162	108368	477443	172154	308135
August	650179	44975	42420	43240	254768	242199	212963
September	607856	46541	16851	6745	141235	201131	170060
October	301303	1021	37526	10515	60069	169382	96636
November	168517	14722	53706	71367	79283	254925	107087
December	231409	611059	166131	274614	348928	206405	306424
Total	7403384	4458141	5739320	3548997	6207612	4556888	5319057

The monthly averages of infiltration from the reservoir during the years 2003-2008 (Table 1) are plotted against the monthly averages of reservoir storage (Figure 5) and the monthly averages of groundwater level downstream (Figure 6). The figures show the direct relation between the

infiltration and the reservoir storage and the groundwater level fluctuation. The groundwater levels continue rising because the infiltrated water takes about two months to reach the groundwater observation well (Lenz 1999). The lateral distance to the observation well is about 1.5km. Also in summer, water level decreases sharply due to high pumping from wells for irrigation in addition to the lower infiltration rate.

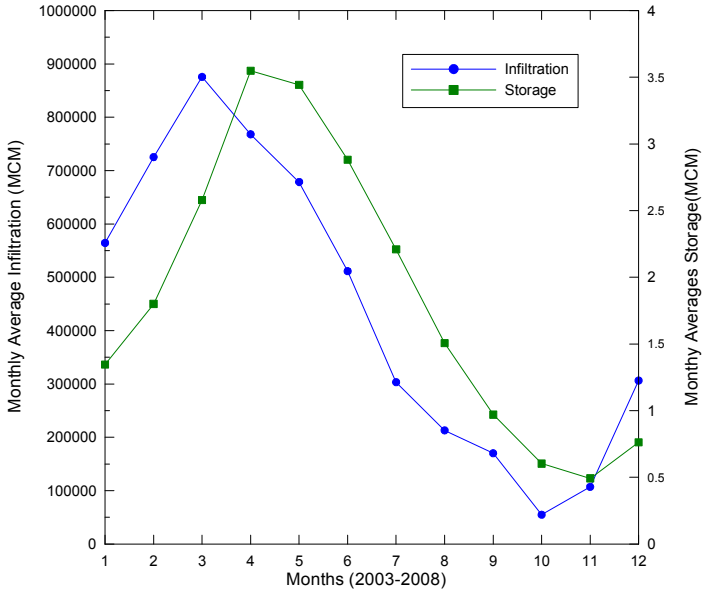


Figure 5. Monthly averages relation between reservoir infiltration and reservoir storage

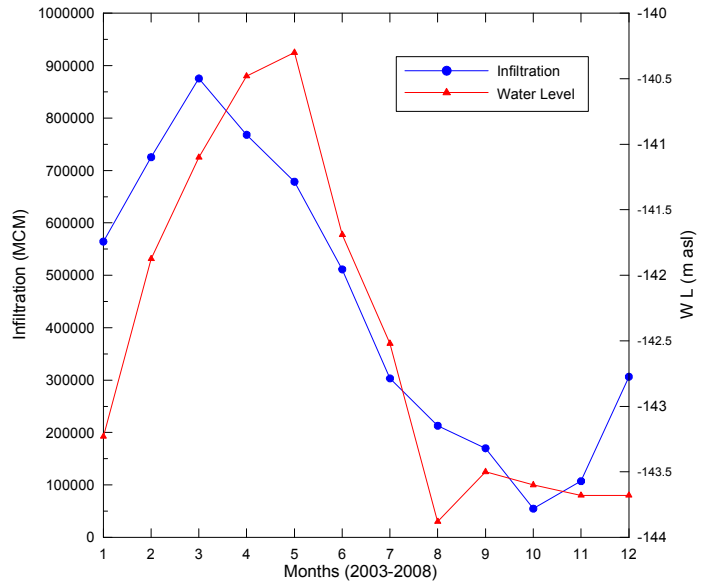


Figure 6. Monthly averages relation between reservoir infiltration and groundwater level in Kafrein Obs. well-1

3- WATER QUALITY ASSESSMENTS

3. 1 Sampling points and analyses

To assess the impacts of the infiltrating reservoir water (different qualities) on groundwater quality down stream, seven water sampling points were selected. Three points along the surface flows, one from the reservoir and three from the groundwater, shallow and deep aquifers (Figure 2). The sampling points (3-7) were sampled four times during the period; July 2007- April 2008 and analyzed in Water Authority Labs for cations and anions, biological oxygen demand, chemical oxygen demand and Isotopes. About 45 samples were taken from all sampling points. The analyses

show that the electric conductivity of Na'ur, es Sir and Kafrein streams ranges between 800 to about 1400 µs/cm depending on the time of the year especially in wadi Kafrein stream. Groundwater of the shallow aquifer has EC values between 1000 and 1100 µs/cm, while in groundwater of the deep aquifer the EC values range from 4200 to 4700 µs/cm. The BOD and COD analyses show that Wadi Kafrein stream and Kafrein Reservoir water as well as the groundwater have <20 mg/l of COD and <10 mg/l of BOD. This means that the treated waste water is having enough dilution by the fresh water base flow in the wadi, therefore the stored water quality in the reservoir meets the Jordanian standards for artificial groundwater recharge (<20mg/l). The high Nitrate (NO_3) concentrations in the analyses are reflecting the long-term fertilizing practice which was carried out for several years with inorganic as well as organic fertilizers. Also, nitrate can be derived from the mineralization of organic nitrogen in cultivated soils or the decomposition of urea in urine.

3.2 Isotope Contents

The isotopic content of the samples (from this study and from previous studies) were plotted and shown in figure 7. It can be seen from the figure that all samples are located between the Global and the Mediterranean meteoric water lines. Isotope samples from the reservoir are distributed in two areas, samples with highly depleted in Oxygen-18 in the left lower cluster of points, which originate from the precipitation that took place in the high mountains and has a long flow path and samples in the upper right which are enriched, meaning they originate from lower altitudes. The figure also shows that most of the groundwater samples plot in the same area as several samples from the reservoir, which indicates that there is mixing between the two types of water. This is taken as evidence of recharge from the reservoir stored water to the groundwater system.

Tracing levels of radioactive tritium in different types of water can provide an estimate of the age of the recent water. A study done by Al-Saudi 2003 showed that the average tritium content of the water from the reservoir is 4.7 TU and from groundwater is 5 TU. The similar values suggest that the water from the reservoir and ground water have the same age or, in other words points towards direct recharge to the groundwater system.

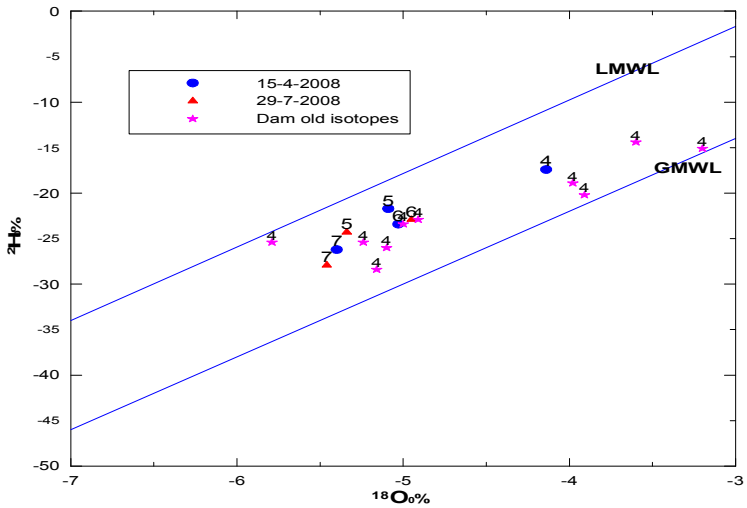


Figure7. Stable Isotope content of Kafrein Reservoir and groundwater wells

3.3 Water Resources Types

The chemical analyses results were plotted on the triangular diagram (Figure 8). The figure shows that Wadi Na'ur flow is of bicarbonate water (sample 1) while water from Wadi es Sir flow (sample 2) can be described as chloride water due to slightly mixing with saline water springs discharging from the lower aquifer. Wadi Kafrein flow (sample 3) is a mixture of Wadi Na'ur, Wadi es Sir flows and the treated waste water discharging from Wadi es Sir Treatment Plant into

the wadi as well as the flood water during winter. Therefore, the water quality in the wadi can be described as carbonate water in winter, chloride water in summer and sulphate water in November (just before winter) when the majority of the flow in Wadi Kafrein is treated waste water. The reservoir water (sample 4) is of chloride water type in winter and summer but it becomes sulphate water in November for the same reason. The figure also shows that groundwater of the shallow aquifer (samples 5 and 6) is of bicarbonate water type and groundwater of the deep aquifer (sample 7) is of chloride water type throughout the year, despite the increase in chloride and sulphate concentrations in November analyses of shallow groundwater samples due to the infiltration from the reservoir.

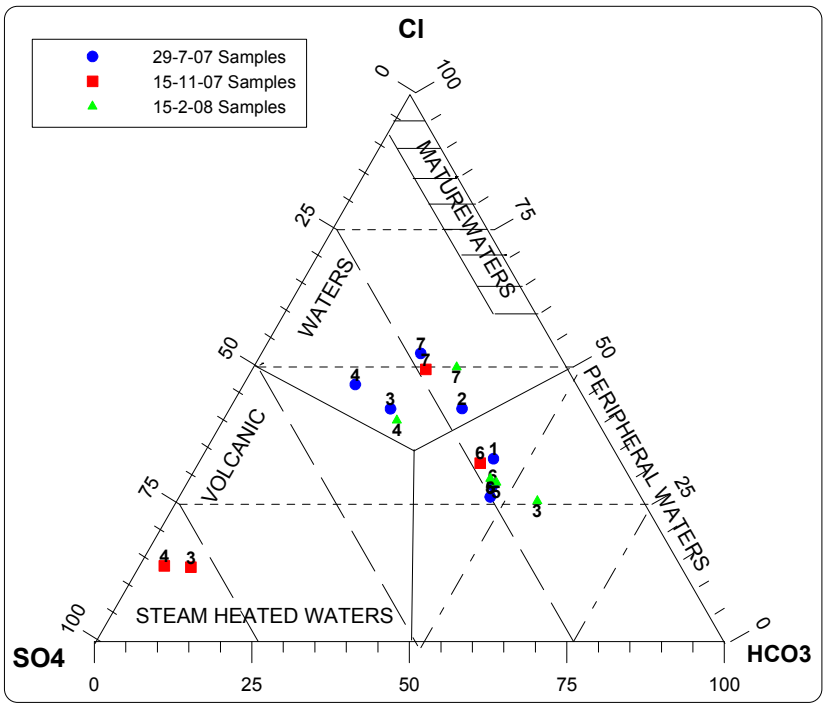


Figure 8. The chemical analyses (samples 1-4 surface water and samples 5-7 groundwater) on the triangular diagram

4- CONCLUSIONS AND RECOMMENDATION

As a result of quality and quantity assessment several conclusions were obtained:

1- Different water sources are flowing in Wadi Kafrein stream; groundwater base flow supported by springs and seepages, treated waste water and flood water.

2- The stored water in Kafrein Reservoir is a mixture of three types of water; sulphate, chloride and bicarbonate water and it can be classified as chloride water in winter and early summer but it becomes sulphate water in late summer (just before rain) when the major source of stream flow is the treated waste water.

3- Quality of the stored water in the reservoir meets the Jordanian standards for groundwater artificial recharge.

4- Groundwater down stream of the reservoir is of bicarbonate water type in the shallow aquifer and chloride water in the deep aquifer.

5- Groundwater quality show minor impact by the infiltrated water from the reservoir through the aquifer material.

6- The long term average of the groundwater level and the reservoir storage shows a direct relation, pointing to a direct recharge from the reservoir to groundwater system.

7- The calculation of the reservoir water budget during the years 2003-2008 show that about 5 MCM /year in average is entering the groundwater system from the reservoir

For more quality assessment of all stored water resources it is highly recommended to have more sampling points like the pool of al Batous pumping station and to install an Ec diver in Kafrein Observation well and to analyse for other compounds.

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The book also contains contributions by the SMART Research Consortium whose scientific coordination meeting on Sustainable Management of Available Water Resources with Innovative Technologies (SMART) was held within and in parallel with EWRA's conference.



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