WATER RESOURCES MANAGEMENT: NEW APPROACHES AND TECHNOLOGIES

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Sustainable Water Services and Sustainable Water Resources over Time Separation or Interaction?

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Abstract: With the merging large metropolises, the issue of water services interacts with water resources allocation. In this paper, we go back to the end of the 19th century in Europe to follow the parallel developments of technologies, institutions and socio-economic issues in water services provision. We sketch it through the 3 successive engineering approaches: civil, sanitary, and environmental. Each development led to a crisis which in turn opened the way to a new development. Municipal involvement in water services is a key factor of success in Europe, and should be considered by the international institutions facing the Third World issues. Yet today, large metropolises call for new solutions away from 'municipalisme': multi-level governance, and demand management implying citiezns' involvement.

Keywords: Capacity building, municipal water services, privatisation, public participation, public participation, sustainability

1. INTRODUCTION: THE THREE AGES OF WATER TECHNOLOGY AND SERVICES

Metropolis is one of the most striking phenomena since the end of the 20th century, and it has direct consequences on water policy: many of these metropolises are so big that they have a complex interaction with nature. Paris is far too big for the Seine! The increasing interaction between large cities and nature gives rise to a reflection on 'urban water': water resources and water services[1], which had become two separate issues thanks to the prowess of water transport and treatment technologies, are now increasingly interfering. We cannot take nature for granted anymore, and we need to

[1] Water services usually include public water supply (PWS) and sewage collection and treatment (PSCT), and in large cities stormwater control. In Brazil, they are included in the notion of *saneamento básico* (basic sanitation), with additionally solid waste collection.

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protect the resource, if only for reducing the long term cost of transporting and treating water. Metropolisation leads to re-consider the separation between water management and land use planning. The same issue has developed in developing countries, e.g. in the large metropolised area around Sao Paulo and Rio de Janeiro: there is indeed a growing competition for water resources between water services and electricity, while the three systems are strongly linked (no electricity, no water).

Having worked on water management in the 15 member States (Barraqué, 1995), in this paper we would like to sketch the history of water industry (chiefly PWS and PSCT), under three "ages" separated by "crises", in order to gain some intelligence of what is a sustainable water management approach. Looking back to the last century and a half, we can describe 3 socio-technical systems which successively developed under civil engineering, sanitary engineering, and environmental engineering perspectives. Indeed, these 3 approaches developed in Europe partly as an answer to sustainability crises, along with innovation and changes in the professionalisation of engineers; in particular, with the post 1960 rise of environmental engineering, and its superseding the more traditional sanitary engineering. In other parts of the world, relative abundance of water made it possible to postpone the adoption of sophisticated technologies and water is used in larger quantities because it is much cheaper. In large Third World cities, there is an additional problem: dramatic urban in-migration of poor people make it more difficult to catch up with good water services. Can the European path to good water services be of use in the global debate about good water management?

Of particular interest is that some European countries are still in an intermediate position in terms of water services quality. Portugal and Spain had authoritarian governments until 1974-75, and they ended up with water services lagging behind, but with large hydraulic projects developed for the sake of hydroelectricity and irrigated agriculture. Return to democracy meant decentralisation, and for a while it was difficult to imagine some level of regional policy to solve complex territorial issues. However more recently, governments in several countries set up a complex multi-level system to efficiently channel National and European subsidies towards improvement of local water and sewer services. In the most advanced cases, water policy is driven by a complex arrangement of interactions between institutions of various government levels. In France for instance, the well known agences de l'eau are both in charge of improving water resources protection and allocation at river basin level, and of mutualising the large investment needs for water services to implement National and increasingly, European Directives.

However, today in Europe, there are signs of a new crisis, which might make the 'good water services' unsustainable. Clearly water prices are going up: renewal of ageing infrastructure must be done without anymore subsidies, and comes on top of environmental policies. Strangely enough, a "spiralling down" evolution could take place, partly due to the low acceptability of the full cost recovery doctrine. European water services might then face the same issues as those of emerging countries. Then comes the time when territory could resolve what technology cannot any more, and

this is why the above mentioned separation between water services and water resources is increasingly blurred, as soon as resources re-allocation (or conservation and protection), end up cheaper than treatment technology sophistication.

2. THE AGE OF WATER QUANTITY AND LONG DISTANCE TRANSFERS

In the 19th century, or rather until the discoveries of Koch and Pasteur were popularised, public water supply (PWS) developed on the assumption that water should be drawn form natural environments far from the cities. The trend had been set in the previous centuries by kings and landlords building aqueducts for their castles or for their capital cities. With the industrial revolution and urbanisation, large cities in particular would have to get water from further and further. This was made possible by the possibilities of municipalities to obtain "cheap money", in particular from the early popular savings banks which they controlled. However, obtaining a water right or a concession on distant water sources requested at least the legal intervention of the national government, who had sovereignty on water resources and their allocation. This created a de facto centralisation. On top of this, the transition from the old feudal world to the capitalist world was slow, and, according to the feudal legal system[2], while kings and landlords could mobilise water resources with aqueducts, they had to respect the right to common use of water by peasant communities; common people in turn thought that they should have access to water resources for their domestic needs for free. Thus the idea that water should be paid for as a service was not popular, and this vision was supported by the quasi-absence of operations costs. All the expenses were with the installation of the infrastructure, which was paid for by public money.

Yet, in the 19th century private companies imagined that water could be delivered in private residences, including in condominiums, against payment of a bill. In many cities of the time, authorities would consider this as a luxury, and they did not really want to get involved. They granted concessions to the companies, which often had to deliver water at a cheap price to fountains and public needs, most of the profit coming from private subscribers of 'private' water. Companies were left by themselves to produce and distribute water, having to solve several technological problems (leak control, metering etc.). Despite their skills (in particular in England thanks to the profile of the 'mechanics'), these initial ventures usually lacked both enough capital and political support to be able to generalise the service. The initial model of the concession in turn maintained a feeling of distrust between companies and the population and the authorities. Then municipalities became convinced that water supply was not a luxury, but a fundamental public health issue. If operators took

[2] Broadly separating the rules of *eminent domain* applying to landlords and the common law (or *dominio util* vs *dominio eminente* in Spain) ruling the use of natural resources by the peasant communities.

water from nearby rivers, it was of bad quality, but they had even less financial and legal possibilities to get it from a distance, and on top of this, central government intervention was needed.

3. A TRANSITION TOWARD THE SECOND AGE: MUNICIPAL MANAGEMENT

The country which played the largest role in these developments was indeed England, where water supply and sewerage were generalised earlier than on the continent, where innovation took place, in an overall context of decentralisation and rise of what should be termed municipalism (rather than municipal socialism as depicted in France, or as water & gas socialism as derided in the UK itself). Indeed, the 'welfare state' and central government involvement in the economy was anticipated by at least two generations of municipal welfare policy in England, followed by other cities in Europe.

With the emergence of a middle class and of qualified working class, also came the savings banks, which would eventually loan at cheap rates for local welfare achievements. Municipal bonds were found attractive by the public, and on top of this the Government would subsidise projects. This is why in most cases, private concessions were not renewed and even terminated to be replaced by municipal direct management. But overall, in this initial development and extension of systems phase, the idea prevailed that water services should be covered by local taxation, i.e. by rates proportional to the renting value of the house. This proved to have important redistributive effects and played a significant role in the acceptance of domestic services. In Montreal, Quebec, where a private company had started a water system as early as 1798, but met severe profitability problems, the municipality terminated the contract soon after being authorised to incorporate by the British crown; but it was soon to discover that billing water services was far from covering the needed costs for expanding the service. Around 1850, bargaining with the crown resulted in authorisation to compel all citizens to connect, and to charge them for water in local taxes. 15 years later, everybody was connected (Fougères, 2004).

Glasgow offers another good illustration of this transition, opting for municipalisation of water supply: There was a private company, delivering rather bad quality water, and unable to find the capital to extend the service. The city council decided to take over the service, and indeed it was able to find money. However, the technology chosen to produce the needed water was in conformity with pre-pastorian preference for distant sources: "Loch Katrine was located in the Perthshire Highlands, some 55 km from Glasgow, and thus well away from the polluted city (…) Loch Katrine was unquestionably the prime municipal showpiece for the city, combining the wonders of Victorian technology with the nurturing quality of pure Highland water" (Maver, 2000). Joel Tarr (1996) illustrated this broad approach in the US: getting cleaner water from further, and using the rivers as sewers on the other, the

latter decision relying on the assumption of natural dilution and self purification of rivers.

However in Britain, there was not so much water available, and many cities had to take it from rivers; so they had to invent some form of purification; then, the overall problematique of water quantity, long distance transfers and reluctance to treat it could be partly replaced by a quality one. But the first type of approach has remained dominant in the New World, and was also extended in the rest of the world after the Second World War, due to the co-occurrence of International Financing Institutions offering cheap money, and of various forms of support for National Governments' intervention in infrastructure provision (Keynesian or socialist). The American federal government's involvement in large hydraulics for irrigation and cities offered a long lasting example. Today, still, many States in developing countries base their water policy on large water transfers, so as to indirectly subsidise the production of irrigated cash crops to integrate in the world market. In some cases the ceiling of extractable water resources has been reached, and the present crisis offers the possibility to check the unsustainability of these past policies. California, which has become indeed the "largest artificial river basin in the world", thanks to enormous State and Federal investments in infrastructure, is a leader in experiencing this need for change.

4. THE SECOND AGE OF WATER TECHNOLOGIES: FROM FURTHER QUANTITIES TO NEARER QUALITIES

In NW Europe, industrialisation and urbanisation led to a rather rapid crisis of the 'civil engineering age': as cities grew in population, water would eventually be fetched from further and further, and this would generate conflicts with communities deprived from their resources, and even with other cities competing for water. Besides, for reasons of political conflicts related to the centralisation vs decentralisation issue, cities would eventually become reluctant to depend on the central governments' legal and financial support. Then water contamination was discovered a major cause of diseases in the 1880's, even distant and pure water would eventually be concerned. It was decided that water should be filtered (end of XIXth century), and soon disinfected by chlorine, activated carbon, UV or ozone (around the First World War), which ever source it came from. And this discovery a contrario supported the idea to take water from rivers closer to the cities and treat it anyway. This is how sanitary engineering developed to complement civil engineering in the solution to the first crisis of water services.

Paris offers a perfect example of this change, with the 30 years debate in Paris Council about an 1890 project for an aqueduct from Geneva Lake which would 'solve all the City's water quantity problems forever'. The project was discarded after the First World War for geostrategic reasons, but indeed it had been made superfluous after the inauguration, in 1902, of the slow filtration plant in Ivry, operating with

Seine water just upstream Paris; still in use today, this plant itself was an answer to a typhoid epidemics in 1899, which was found to be due to the contamination of the Loing springs; yet these then were the furthest and supposedly cleanest water sources for the city.

So in that still early period, large European cities changed strategy from investments to increase quantities to investments to improve quality. But treating water would of course induce a serious rise in operation costs. And the latter are usually more visible to the public and to the city councils. Therefore it is easy to imagine that even publicly provided supplies could be at least partly covered by water bills. It was certainly also an important change for the public, and charges were usually limited to operations costs and did not cover investments depreciation. Slowly then, delivery of pressure water within the homes changed status, from a luxury good to a normal comfort, and made water billing normal. In Europe, only Britain (and the Republic of Ireland) kept until recently the ancient charging system based on rateable housing values (but also many municipalities in the US, like New York, and in Canada).

Increasing the self financing capacity of water services really helped to increase their sustainability. It was found so attractive, that in many continental countries, it was decided to pass on the sewerage charges in the drinking water bills, despite the compulsory character of sewer connection, which would normally entail coverage of the costs by local taxes. Eventually, increased self financing capacity reinforced the legitimacy of local authorities as services providers, or at least organisers.

Conversely, we can hypothesize that the deepest problem for the generalization of water services in most developing countries, is indirectly due to the adoption of a centralised model, under the Welfare State influence after WW2, to base economic development upon water mobilisation, in particular in large ‘multipurpose’ schemes. Mobilising water in hydroelectricity and irrigation was the priority. And this maintained the influence of the civil engineering and the first age vision, shared with many occidental and soviet experts, and many funding institutions like the World Bank. What was not readily seen is that these policies resulted in concentrating agriculture in fewer hands and mechanising it, pushing a dramatic number of ‘landless’ people towards cities. It then became very difficult to really improve the connection rates, among other reasons because poor people settled in improper areas. But that is not all. In many of these countries, the post-colonial situation was that of very inequitable societies and strong hierarchies. There was little solidarity of the rich and even the middle class to the poor, at the precise level where it was most needed, i.e. at city level. In Europe, municipalism was built upon the very concept of solidarity, and despite a commodification process (i.e. replacement of funding by taxes with funding by bills), this never implied an incapacity of the poorer population to access. Indeed, public money and subsidies remained largely used until quasi everybody would be connected. In Brazil for instance, typical anglo-saxon despise for local public management combined with the anti-democratic military regime into the centralisation of water services at Federated States’ level. After an initial period of

success, the model progressively went into a crisis: it had to face the connection of poorer and poorer people, with interest rates always going up. In that situation, it is usually less risky to invest in additional heavy infrastructure than into local services. It is difficult to say that a maintained municipal or joint board model, like in most European countries, would have done better. But it is sure that the lack of transparency in the regional approach and in particular in the real degree of cross subsidization drove the population to a complete lack of confidence in the system; in turn this made it impossible to raise the service prices and to reach a minimal self financing capacity.

5. THE CRISIS OF MUNICIPAL WATER SERVICES IN EUROPE: TOWARDS THE THIRD AGE

However, the sustainability of the European model might be challenged in the future: how is it possible to maintain a good service quality on the long run, even once everybody is connected, if full cost recovery is imposed?

In France like in other northern European countries, important efforts were made on city sewerage from the 1950's onward, and on sewage works from the 1970's. To make the investment funding easier, it was decided to change the status of PSCT from an imposed system to a service, and then to have it paid within the water bills. But in the same period, PWS itself became a mature business, i.e. it had to face the issue of renewing ageing infrastructure without any more subsidies. This is the fundamental reason why European municipalism had to adapt in various ways, but chiefly through returning to legal private status: traditional public accounting rules were embedded in a money culture which was reluctant depreciate the assets and to make renewal provisions, while private accounting could, and even had to. PWS, and later PSCT, slowly turned towards depreciation and provision practices, and this of course meant a rise in water bills. With economists arguing in favour of full cost pricing, subsidies are phased out. In turn water bills rise dramatically, and an increasing number of large users (industry, services) either quit PWS, change their processes, or fight their leaks. This explains the recent stagnation of volumes sold. In some countries even domestic consumers have reduced their demand for PWS, through changing fixtures and domestic facilities, different garden design, and also with rainfall storage or other alternative sources of water for non drinking uses. Now slowly, we discover that this demand reduction should not go too fast, because it would then threaten the already fragile financial balance of the services!

At the same moment, water engineers discover that it's going to be ever harder to comply with the drinking water standards (DWS) all the time at reasonable costsThe multiplication of criteria is slowly bringing the situation into over-complexification: chlorination byproducts give cancer (for a history of drinking water criteria and the present result, see Okun, 1996). There are many other examples, and anyway, year after year, the media can report a growing proportion of people receiving non-

complying water, even though the treatment is improving on the long run. To lower the risk of being unable to make it, along with local, national and European authorities, water supplies turn to a new strategy: land-use control on water resources protection areas, usually implying agriculture re-extensification, and farmers compensation programs. This policy eventually turns out to be cheaper than water treatment sophistication, and to be a positive sum-game with farmers. Yet it seems to be a long way to a new and more sustainable equilibrium. And worse, in the meantime, the drinking water criteria are being reinforced regularly as new risks are discovered, and the spiral of treatment costs and loss of consumer confidence is going on.

Costs are becoming so high, and the future is so uncertain, that technologies of the first age, based on large mobilisation of surface water (Llamas, 2001), are questioned: the present time is not appropriate for big long water transfers. What happened in California starting in the 1970's is now general in Europe: it is getting ever harder to build dams, because environmental movements have been joined by economists and liberals who advocate full cost payment of water infrastructures by their beneficiaries. But creating "water markets" seems very difficult. Then the new slogan seems to be "conserve water first and manage the demand, there is no cheap money in sight for subsidised water transfers" (Barraqué, 2000).

6. SOME FEATURES OF ENVIRONMENTAL ENGINEERING AS PARADIGM OF THE THIRD AGE

A history of the yet short existence of environmental engineering (Hendricks *et al.*, 1990) shows its origin in sanitary engineering. But environmental engineering aims at protecting not only populations from negative environmental factors, but also global and local environments from potentially dangerous human activities. Of course, improving the knowledge of natural processes was an important issue, but a large part of the programs were focusing on urban issues and technical systems. Also it was agreed that the core education of environmental engineers should include chemistry and biology, but also systems analysis, applied social science, and project design in practice. Water services were still concerned, but in a more global approach than in traditional sanitary engineering. Indeed, in the two first "ages" engineers focussed on supply-side solutions, while necessarily environmental approaches have to consider what is termed demand side.

When urban systems started, they were called infrastructure, because they were hidden from the people, for both technical and social reasons. Some systems like water-supply were essential to improve health and reduce (fire) risks. But in the end, all can be seen as having brought apparent degrees of freedom to the urbanites, as time and space saving devices. Until recently, however, urbanites underestimated the importance of these systems, because their vision of the world was centred on landownership. It was not the infrastructure, but the plot of built or buildable land,

which provided political legitimacy. City aldermen and elected representatives were competent on issues like valorisation/devalorisation of urban land, and they did not know much about the systems. In turn, this constrained public procurement of services within municipalism led to the oblivion by people of the importance of systems: “out of sight, out of mind” and NIMBYism characterise the public’s attitude and is at odds with a conservation attitude (Melosi, 1983): the elected authority would gather the demand of the citizens for urban technology, and they would pass it globally on to the operators, engineers and professionals who remained out of the political space as most as they could; operators just had to match the demand with more or less invisible infrastructure and that was it.

And this is how we get the best potable water for all the uses we need in unlimited quantity, while only 5 to 10% really need to pass the increasingly numerous drinking water quality standards! With our "systemic" eyes of today, we can see how the municipalist model comes to a crisis. We can propose new solutions. But it is not a purely market based rationalisation as claimed by environmental economists. When we move to demand side solutions, in fact we improve the interaction between supply and demand.

7. REDEFINING THE OPERATOR

This leads the reflection towards redefining the very notion of "operator" in ecological terms. If supply and demand permanently interact, then the operator needs a direct contact with the public, at least to know in detail what the needs are, and even to bring the users of the system to use it more efficiently. Besides, many of the decisions to be taken involve planning and zoning measures, which are highly political decisions, just as well as differential pricing for incentives. Basically, the traditional operator cannot do it alone, and he needs a much stronger interaction with the authority. Too often, local authorities have decided to privatize some urban infrastructure, thinking that they would be rid of a too difficult or too costly problem. But this type of privatization often fails, because conversely, it was increased local authority implication that was needed, in the definition and the control of the service if not in the financing.

In the end the new grid-based systems operator is a complex mix of people and institutions who can only master the networks effects when they interact. In some cases, the users themselves become part of the operator, if real time information allows them to react; then one can argue that demand management indeed means to develop citizens’ participation.

A new generation of grid-based systems planners and operators will also need an intelligence of the limits of urban technologies, which means reverting parts of technical systems to other forms of service delivery, for the sake of extending the services to new areas. A perfect example of this is decentralised sewerage systems. In most European countries official statistics never mention explicitly what happens to

the population which is not connected to sewers, just as if these were bad polluters to eliminate shortly. Yet in most rural areas, and in low density suburbs, it is economically unreasonable to connect everybody to a sewer. Besides, biology technologies allow us to use the soil as a reactor to eliminate bacteriological pollution better than small treatment plants. But once the habit has been given to most urbanites not to care about their wastes, they tend to feel that a septic tank is a low grade technology. What we need to invent then is a service in- between the costly and heavy traditional centralised sewerage system, and the full autonomy of rural people, who have to do everything themselves. Some countries have developed public services for the control and the emptying of the tanks, but it is still exceptional to have "public" septic tanks installed and operated by a utility in private homes, the users paying exactly the same volumetric charges as sewered people. In that case the network has been extended into a service, which saves money and provides equal opportunity to citizens.

Lastly, managers of urban technologies will learn to develop non technological solutions, in order to get out of the cycle of over-complexification. One good example is the development of new catchment-based land use policies to protect water resources devoted to potabilisation from diffuse agricultural pollution. Increasingly, in Germany, the Netherlands, and to a lesser degree in Denmark and France, utilities or their organising authorities develop contracts with farmers or groups of farmers, to obtain the re-extensification of their practices, with much less fertilisers and biocides, and they give them appropriate compensation for the corresponding loss of revenues (Brouwer *et al.*, 2003). In some German Länder this policy is funded by "waterpenny" taxes on water bills of domestic consumers, which makes much more money available for contracts with farmers. This policy is criticised by some economists and ecologists together, who argue that the polluters and not the victims should be the payers. Yet ongoing experiments are rather positive.

8. CONCLUSION

Innovation in non conventional water sources, and demand management, allows to re-think the issue of water services in developing countries and in developed countries (low density areas for instance) together: new types of public-private partnerships are needed to develop low capital water services with alternative management and technical formulas. The Mexico WW forum newly argued for capacity building at local level to generate solidarity at a more appropriate level than with State policies. This is the centennial experience of Europe: organising sustainability of a great innovation (systems for water services) first at local level, but with the help of government money. It results impossible to rationalise what is still under rationing. Recent outcomes of privatisation attempts in various parts of the world, starting with British water companies, show the limits of a system which just forgot that infrastructure's life time is not in tune with the time horizon of bankers today. Water

services in that case really need special skills from environmental engineers. Yet, presently, water and city engineers in third world countries, having received no training to the socio-economic dimensions of their activity, just try to develop the technological part of the occidental model of cities, with no socio-economic basis to do so. Worse: together with large consulting and international contractors, they still promote the realisation of large water transfers, thinking that these project will lower the cost of water and make it really available for the poor. But the poorest cannot afford to pay any water at all; this leads journalists and NGOs to mix up access to water resources and access to water services, without realising that this conceptualisation is embedded in the previous colonial (feudal) period, where the absence of civil rights should be 'compensated' by free services (Jeter, 2002).

The expression "three ages of engineering for water industry" should not be taken literally, because of course they overlap and do not come in sequence. There are projected water transfers today which are still quite necessary and sustainable. And if sanitary engineers of African cities stop chlorinating potable water on the basis that by products can have carcinogenic effects on the long run, they will have far more casualties from dysenteria than from cancer … (Okun, 1996). But the aim is to develop an STS approach with historical dimension, so as to show what is needed for engineers to have a good vision of what is at stake in water services provision today. Note that a similar evolution could be traced concerning water resources integrated management, and the need for social sciences in what we call "hybrid forums" where stakeholders are confronted to conflicting scientific or technical representations within the scientific community, and can eventually build up alternative and innovative "advocacy coalitions" to lead more sustainable water policies. There is a need for conservation, demand side management, but in a more applied manner than in neo-classical economics: it's more the systemic interaction between demand and supply which is at the heart of the problem, and should therefore mobilise interdisciplinarity in environmental engineering. We think that may be the most important is to give engineers a "socio-economic and historical culture" of their action.

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Flood Risk Assessment: A Methodological Framework

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Abstract: The concepts of hazard, vulnerability and risk have been extensively used in various disciplines with a different meaning, impeding cross-disciplinary cooperation for facing hazardous events. Even for natural hazards, such as floods, no unique definitions and assessment procedures have been widely accepted. In this paper we propose a comprehensive way for defining and assessing flood risk and vulnerability in the flood-prone areas. The suggested methodology follows a three-step assessment approach: a) annualised hazard incorporating both probabilities of occurrence and the anticipated potential damages b) vulnerability (exposure and coping capacity) in the flood-prone areas and c) annualised flood risk (estimated on annual basis). The methodology aims to assist water managers and stakeholders in devising rational flood protecting strategies.

Keywords: Annualised flood risk, flood hazard, flood risk assessment, vulnerability.

1. INTRODUCTION

The assets at risk from flooding can be enormous and include private housing, transport and public service infrastructure, commercial and industrial enterprises, and agricultural land. The International Committee on Large Dams (ICOLD, 2003) conducted a survey to determine the social and economic impacts of the floods in 20 countries, where are located about 80% of the total number of the world's largest dams. The greater number of victims due to floods is produced in Asian countries, as it is shown in the Table 1.

In addition to economic and social damage, floods can have severe consequences, where cultural sites of significant archaeological value are inundated or where protected wetland areas are destroyed. Regarding floods in Europe, two trends point to an increased flood risk and to greater economic damage from floods. First, the scale and frequency of floods are likely to increase in the future as a result of climate change, inappropriate river management and infrastructure development in flood risk

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areas. Second, an increase in vulnerability has been noted due to the number of people and economic assets located in flood risk zones. Therefore the coming decades are likely to see a higher flood risk in Europe and greater economic damage.

Table 1. "Mean" number of victims per year (ICOLD, 2003).

Victims	Countries
0 – 10	Argentina, Australia, Brazil, Canada, France, Ireland, Italy, Netherlands, Norway, South Africa, Sweden, Russia
10 – 20	Spain
50 – 100	Indonesia, USA
100 – 150	Japan
>150	Korea (250), Bangladesh (200), India (1500), China (2000-3000)

On 18th of January 2006 the European Commission proposed a Directive on the assessment and management of floods (COM, 2006). Its aim is to manage and ultimately to reduce the risks that floods pose to human health, environment, infrastructure and property. Under the proposed Directive the Member States are obliged to deliver the following for river basins and sub-basins:

1. Preliminary flood risk assessment
2. Flood risk maps
3. Flood risk management plans

The provision of structural flood defenses can have a major impact on the environment and there has been an expression of concern by many members of the public for the degradation of river corridors. Therefore, it is becoming common practice for central and local governments to subject flood management plans to public discussion (COM, 2006).

It is obvious from the above that concepts such as hazard, risk and vulnerability are the most commonly used terms to describe the potential threats that natural disasters pose to human life, the environment and the infrastructure. Additionally, these terms are used to question the capacity of various structural and non-structural measures, which are applied for protection from these threats. In the absence of regulatory establishment of a common accepted terminology platform, the confusion on the context of these terms grows. Furthermore quantification of the terms is not an easy task. It is possible that some parameters affecting the above concepts are beyond quantification and also that these parameters vary in space and time (Brauch, 2005; Thywissen, 2006).

However, in order to accomplish a comprehensive and participatory approach to flood risk management, it seems that a clear comprehension of the processes of hazard, vulnerability and risk perception is necessary. The primary focus of this paper is to clarify these concepts and to highlight a methodology for the assessment of flood hazard and flood risk. Particular attention is also given to the concept of vulnerability with regards to its social nature and the factors on which it depends.

2. DEFINING TERMS AND CONCEPTS

2.1. Flood Hazard

Hazard may be defined as:

- a source of potential harm.
- a threat or condition that may cause loss of life or initiate any failure to the natural, modified or human systems.

The initiating causes of a hazard may be either an external (e.g. earthquake, flood or human agency) or an internal (defective element of the system e.g. an embankment breach) with the potential to initiate a failure mode. Hazards are also classified as either of natural origin (e.g. excessive rainfalls, floods) or of man-made and technological nature (e.g. sabotage, deforestation, industrial site of chemical waste). Regarding hazard identification and estimation, two approaches can be identified based on the ANCOLD Guidelines (2003) and the ISDR principles (2004):

a. *Traditional deterministic approach:* a first level estimation of the potential adverse consequences, if the hazard occurs, in order to classify the system under threat, identify the necessity or not of further investigation. This approach is also the most comprehensive way of estimating man-induced and /or technological hazards, e.g. a forest fire hazard that can not be captured by a probability distribution.
b. *Probabilistic approach*: it is based on the theory of probability and regards hazard estimation as the estimation of the probability of occurrence of a particular natural event with an estimated frequency within a given period of time. It can be applied on hazards of natural origin and it represents a very common method used in most flood plain delineation studies when the potential for loss of life is considered negligible in terms of historical floods. The probabilistic approach tends to assume that events in the future are predictable based on the experience of the past.

Concentrating on the flood hazard, it can be supported that the capture of the natural phenomenon requires the frequency of the flood events as well as their magnitudes (and thus their anticipated flood damages) (Alexander, 1991). Since the magnitudes of flood events can be modeled by a probability density function, flood hazard can be estimated by the probability that the flood damage that occurs in any one year, in the case of a flood event with specified frequency, will not exceed a specified value x_1. If X is the random continuous value of flood damage then the cumulative distribution function of the variable X will be:

$$F_X(x_1) = P(0 \le X \le x_1) = \int_0^{x_1} f(u)du = F(x_1) - F(0) \qquad (1)$$

where $f(u)$ is the probability density function (p.d.f.). This way, the probability of occurrence of various potential magnitudes of flood damages $X \in [0, x_1]$ can be

expressed by a single number by summing the products of all possible values of flood damage and their probability of occurrence.

However, due to the random nature of flood damage it is impossible to predict the exact value of damage that would be incurred or prevented in any year. Therefore, flood hazard estimation could be based on long-term statistical averages, also known as expectations that give a measure of the location of the p.d.f. The expected value E(X) of annual inundation damage *X* is computed as:

$$E(X) = \int_{-\infty}^{\infty} x \cdot f(x) dx \quad \text{(for a continuous random variable X)} \tag{2}$$

Finally, the variance of the annual inundation damage X, var(X), is a measure of the spread of the p.d.f about its mean and it can be calculated as follows:

$$\text{Var}(X) = E(X^2) - \{E(X)\}^2 = \int_{-\infty}^{\infty} x^2 f(x^2) dx - \{\int_{-\infty}^{\infty} xf(x) dx\}^2 \tag{3}$$

Regarding the derivation of the damage-probability function can be estimated via the method illustrated by Figure 1 (USACE, 1996). Initially, a discharge-probability function (Figure 1a) is developed. If stage and discharge are uniquely related, a rating function (Figure 1b) can be developed and the discharge-probability function can be transformed through this rating function in order to develop a stage probability function. Similarly, if stage and damage are uniquely related, a stage-damage function (Figure 1c) can be developed, and the stage-probability function can be transformed through that function to yield the required damage-probability function. Finally, to compute the expected damage, the resulting damage-probability function is integrated.

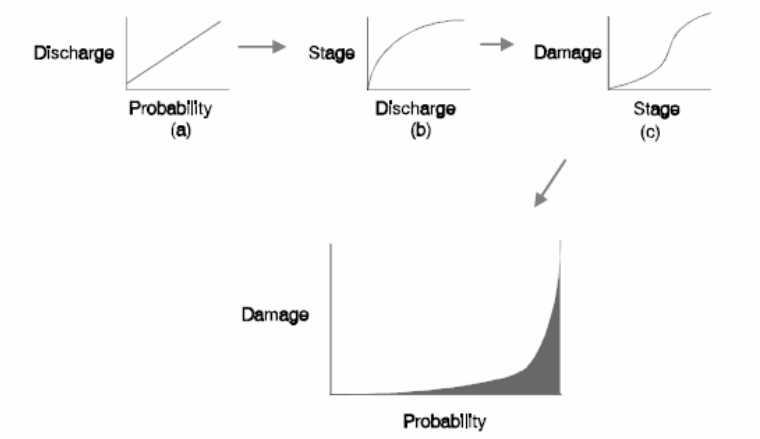


Figure 1. Transformation for traditional expected annual damage computation (USACE, 1996).

2.2. Vulnerability in the Flood-Prone Areas

One of the best-known definitions of vulnerability was formulated by the International Strategy for Disaster Reduction (ISDR, 2004), which regards it as "a set of conditions and processes resulting from physical, social, environmental and economical factors, which increase the susceptibility of a community to the impact of hazards". A basic consensus has emerged, that the concept of vulnerability addresses a double structure consisting of an external side (exposure) (Bohle, 2001), and also that vulnerability (Pelling and Uitto, 2001) is:

- multi-dimensional and differential (varies across physical space and among and within social groups).
- scale-dependent (with respect to time, space and units of analysis, such as individual, household, region, system).
- dynamic (characteristics and driving forces of vulnerability change over time, certainly exceeding that time of the extreme event itself).

Generally, the vulnerability of a system against a certain hazard is not easily assessed. Three routes for the assessment can be distinguished: a. economic, b.social and c. cultural.

The vulnerability function could be treated as a function between 0 and 1 (see Figure 2). The most appropriate approaches for the case of vulnerability of the society and the cultural heritage are thought to be qualitative (Laoupi and Tsakiris, 2007). Given the above thoughts, vulnerability assessment in flood prone areas depends on the following factors (Tsakiris, 2006):

- the Exposure of the system (E).
- the initial coping capacity (resources availability) of the system (S).
- the magnitude and intensity of the hazardous event (Qmax).
- the social response of the system (early warnings, indigenous experience, public awareness etc) (SF).
- the fuzziness of the interrelated sides of vulnerability (coping capacity & exposure) (I).

and therefore in mathematical terms vulnerability can be expressed as the following function:

$$V = f(E, S, SF, Q_{\max}, I) \tag{4}$$

A vulnerability analysis in the event of a flood hazard considers the population and structures at risk within the affected area. In the start of the analysis, a reference level of the system's vulnerability should be determined that usually refers to existing flood protection systems of the affected area. The vulnerability analysis evaluates the potential costs of flooding in terms of damages to buildings, crops, roads, bridges and critical infrastructure. Normally the analysis is carried out for various probabilities of flood occurrence, and an elevation-damage curve is plotted.

The aim of the additional reclamation and protection works is to reach a lower level of the system's vulnerability. Consequently, this analysis is valuable for

reaching at decision about the desired level of flood protection. Ideally, this analysis should initiate a public process to establish the "acceptable level of risk" that would refer to the flood discharge magnitude appropriate for the delineation of the affected areas. A comprehensive measure of the improvement of a system is the ratio of anticipated consequences after the improvement divided by the initial potential consequences.

Figure 2 depicts a simplified representation of vulnerability and its reduction versus the magnitude of the hazardous phenomenon. As it can be seen the improvement of the capacity of the system is schematically represented by a shift of the vulnerability curve to the right.

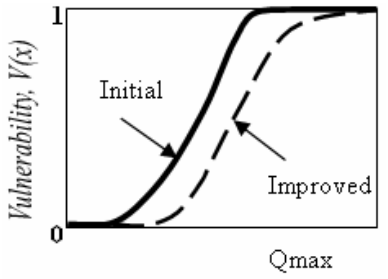


Figure 2. Vulnerability vs magnitude of the phenomenon for the initial and the improved capacity of the system (Tsakiris, 2006).

2.3. Flood Risk

According to EU Directive (COM, 2006) for flood management, *"flood risk"* is the likelihood of a flood event together with the actual damage to human health and life, the environment and economic activity associated with that flood event. In this context flood risk can be considered as the actual threat, in other words the real source of flood hazard to the affected areas. The quantification of flood risk results either in monetary units or in loss of life units, if the losses are measurable, or in qualitative terms (e.g. allocation in classes) in the case of intangible damages (social, environment, cultural) to the affected areas.

In general, risk as a concept incorporates the concepts of hazard {H} (initiating event of failure modes) and vulnerability {V} (specific space/time conditions). It is customary to express risk (R) as a functional relationship of hazard (H) and vulnerability (V).

$$\{R\} = \{H\} \square \{V\} \tag{5}$$

in which the symbol □ represents a complex function incorporating the interaction of hazard and vulnerability. Consequently in mathetical terms it can be expressed as:

$$R = \{H\} \times \{V\} \tag{6}$$

Since vulnerability is a dimensionless quantity (Villagran, 2006), risk could be measured in the same units as hazard. In quantitative terms, annualised risk can be estimated as the product of probability of occurrence of the hazardous phenomenon and the actual consequence, combined over all scenarios. According to the methodology of estimating average (annualised) hazard, the expected value of flood risk can be calculated as follows:

$$E(X) = \int_{-\infty}^{\infty} x \cdot V(x) \cdot f(x) dx \qquad (7)$$

where X is the actual flood damage caused by the flood hazardous phenomenon, $f(x)$ is the p.d.f. that describes this phenomenon and $V(x)$ is the vulnerability of the system towards the corresponding magnitude of the phenomenon. It is obvious that such an estimation involves major restrictions such as:

- can be applied only on hazards of natural origin due to probabilistic analysis,
- although it abides to a general methodological framework, it is highly case-specific,
- highly dependable on expert's judgment.

3. A METHODOLOGY FOR ASSESSING FLOOD RISK

Based on the ANCOLD Guidelines 2003 there is a basic consensus towards the methodological framework of risk identification and estimation. In this context, the general methodological framework for risk assessment can be, more or less, determined and it is given by the following steps:

1. *Risk identification*: refers to the identification of the hazard source.
2. *Risk analysis or risk estimation*: refers mostly to the probabilistic quantification of the average annualised risk and it is measured in the same units as hazard. It involves the estimation of the probability of occurrence of the hazardous phenomenon, the estimation of the actual consequences and the vulnerability estimation of the affected system over the selected hazard scenarios.
3. *Risk evaluation*: refers to the identification of the local society's tolerable risk policies and criteria as well as to the comprehension of the local society's perception of the hazard impacts by the decision makers. One's willingness to pay for risk reduction is controlled by the perceived and not the actual risk. Simultaneously the perceived risk reflects the human attitude towards various kinds of risks and it is therefore of high importance to assess it.
4. *Risk assessment*: refers to the evaluation of the tolerability of the estimated risks based on the local society's acceptability criteria. The comparison of the estimated risks with acceptable ones results in the decision of what risk will be

acceptable in the particular affected system and what risk reduction measures will be applied; if needed.

Under the auspices of INTERREG IIIC - Project OCR NOE - sub project DISMA, it was decided, by the local stakeholders, that the flood risk management framework will be mainly oriented towards non-structural measures (e.g. land use planning, flood warning systems, evacuation plans, insurance policy); that is towards measures that are mainly driven by the need of cultural heritage protection and also by the socio-economic conditions of the Eastern Attica Prefecture. In this context the authors developed a workflow chart, in order to apply the aforementioned methodology over flood hazard scenarios to the specific case-study areas of special cultural interest within the same Prefecture. Figure 3 depicts the applied methodological framework for flood risk assessment.

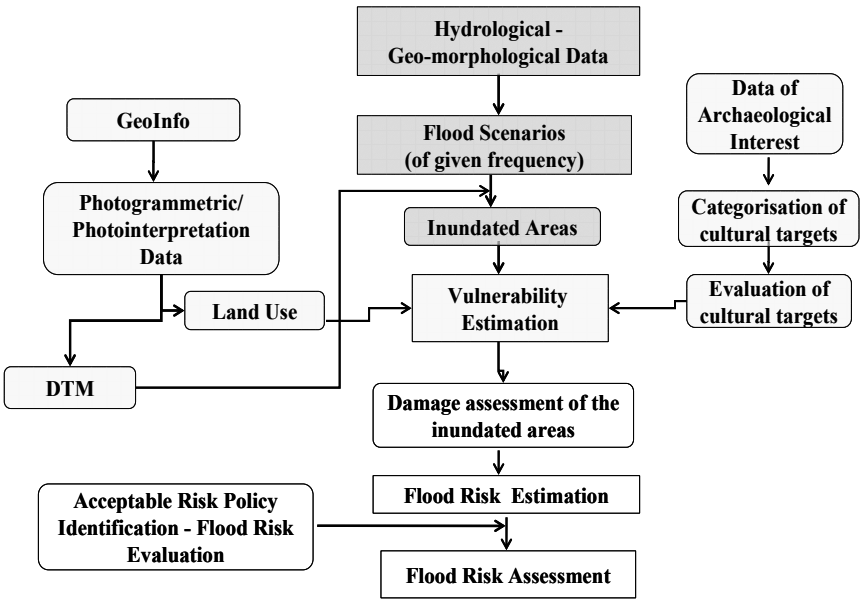


Figure 3. Methodological framework for flood risk assessment.

The procedure for the assessment of flood risk is developed by allocating the workload into three categories of steps that can be initiated independently and represented with different colour. The first category (coloured yellow) refers to the task of collection and processing the necessary data in order to determine the boundaries of the system under risk (in this case particular cultural sites) and subsequently to identify the socio- economic conditions of the surrounding areas (for instance to detemine the local land use establishment). At this stage the identification of the system's vulnerability takes place and that is mainly the definition of the system's exposure to a potential flood hazard and the system's carrying capacity to cope with flood events. The second category involves (coloured blue) the

development of hazard scenarios, the estimation of their probability of occurrence. At each scenario the production of the respective floodplain mapping delineation is given in Arc-GIS environment so as to identify the flood-prone areas and therefore the cultural sites that are under the inundation threat.

The third category in the workflow depends on the results of the two previous ones and so it is always performed last. It assesses the expected damage of the affected system and consequently it estimates the annualised flood risk in monetary units, if possible. Moreover, in this last stage, an attempt is made so that the local acceptable flood risk policy and the public risk perception are identified. The comparison between the acceptable flood risk setting and the estimated flood risk figures can significantly assist the decision makers to apply a series of effective flood risk reduction measures. Such measures can support the process of prioritising, justifying and targeting investments and developing sustainable policies and strategies and subsequently such measures can support flood risk management plans, spatial planning and emergency plans that can be acceptable from the majority of the local society.

4. CONCLUDING REMARKS

The EU Directive recognises the need for methodologies for the effective assessment and management of flood risk. In this paper an attempt was made to contribute to the ongoing scientific discussions on these issues. It is well-known that both the probabilistic risk based methodology and the deterministic methodology face heavy criticism and questioning regarding their applicability on a wide range of systems under threat. The proposed procedure incorporates elements from both of the above methodologies highlighting the significance of the vulnerability analysis within the flood-prone areas with the aim to contribute to a more rational assessment of risk.

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Drought Variability in Western Iran

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Abstract: An analysis of drought in western Iran from 1965 to 2000 is presented using monthly mean precipitation data observed at 170 gauges uniformly distributed over the area. Drought conditions have been assessed by means of the Standardized Precipitation Index (SPI). To study the long-term drought variability a Principal Component Analysis was applied to the SPI field computed on 12-month time scale. Results suggest that the area is characterized by drought variability with multi-year fluctuations. By applying an orthogonal rotation to the first four principal component patterns, it has been found that three distinct sub-regions having different climatic variability may be identified. In particular, the northern part of the study area is characterized by a climatic linear trend toward drier periods from the eighties onward, in agreement with recent studies on drought variability at global scale.

Keywords: Drought variability, principal component analysis, SPI.

1. INTRODUCTION

Drought is 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). In areas prone to drought, rainfall deficit has an intense effect on water resources and, in many cases when a water shortage occurs over an extended period of time, meteorological drought is followed by agricultural, hydrological or socio-economic drought (Wilhite, 1997).

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 indexes have been

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developed to evaluate the water supply deficit in relation to the time duration of precipitation shortage (see Keyantash and Dracup, 2002; Heim, 2002 and references therein). Among them the most commonly used for drought monitoring is Standardized Precipitation Index (SPI, McKee *et al.*, 1993). In fact, 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 standardize nature. The SPI complies with this requirement. It is a standardized index that can be computed on different time scales, so as to allow monitoring most of drought types. The SPI computation for any location is based on the long-term precipitation record cumulated over the selected time scale. This long-term record is fitted to a probability distribution (usually a Gamma distribution, Guttman, 1999), which is then transformed through an equal-probability transformation into a normal distribution (Bordi and Sutera, 2001). Thus, because of the simple computational requirement, the spatial comparability and the possibility to monitor different kinds of drought, the SPI is a widely used drought index. Many authors, including Hayes *et al.* (1999), Lloyd-Hughes and Saunders (2002), Vicente-Serrano *et al.* (2004), Tsakiris and Vangelis (2004), Bordi and Sutera (2001) used the SPI to monitor drought in many regions, while Paulo *et al.* (2005) have used the SPI to predict drought class transitions adopting Markov-chain and Loglinear models.

Studies on climate variability are important for the design and management of water resource systems. However, such analysis may be difficult when data from many stations are used. The Principal Component Analysis (PCA) is a convenient and useful method to reduce inter-correlated variables into a few linearly uncorrelated ones. Many authors, such as Bordi and Sutera (2001, 2002), Bonaccorso *et al.* (2003), Bordi *et al.* (2004a; 2004b), Vicente-Serrano *et al.* (2004) and Bordi *et al.* (2006), applied the PCA to the SPI for analyzing the temporal and spatial variability of drought. On these grounds, we carried out an analysis of drought variability in western Iran by decomposing the SPI time series into principal components. The motivation is twofold:

i. Precipitation in Iran has a high spatial and time variability. There are regions in the south of Caspian Sea, which receive up to 2000 mm of annual precipitation, whereas a portion of central and eastern part of the country gets less than 50 mm. Furthermore, except in northwest and Caspian Sea regions, where maximum precipitation occurs in spring, most of the precipitation in Iran falls during the winter and autumn seasons, due to the prevalence of humid western winds of Mediterranean origin (Domroes *et al.*, 1998; Dinpashoh *et al.*, 2004; Raziei and Azizi, 2007). These features make difficult the management of water resources in the region, especially during water shortage periods;
ii. Iran experienced recurring drought events. For instance 10 out of the 28 provinces were affected by one of the worst and prolonged droughts in 1998–2001 period, leaving an estimated 37 million (over half the country's population) vulnerable to food and water shortage. Twenty provinces experienced precipitation shortfalls during winter and spring 2001 (Agrawala *et al.*, 2001). At

the best of our knowledge, while some efforts have been done to study the precipitation variability in Iran (Domroes *et al.*, 1998; Raziei and Azizi, 2007), no comprehensive study on drought (i.e. deviation of actual precipitation from a historically established norm) has been performed for the region.

The paper is organized as follows. In section 2 there is a description of data and methods used for the analysis. Section 3 shows the main results obtained and section 4 contains the conclusions.

2. DATA AND METHODS OF ANALYSIS

Monthly precipitation data from 196 stations in western Iran were made available by the Iranian Water Resources Institute and the Iranian Meteorological Organization. Statistical tests were applied to the annual precipitation. Homogeneity of the median and variance was evaluated using Mann-Whitney homogeneity test. Trend and independence were checked using Mann-Kendall and Kendall autocorrelation tests, respectively (Helsel and Hirsch, 1992). Results showed that 26 stations had to be discarded due to low data quality and having more than 5% missing values. The remaining 170 stations, covering the period October 1965 - September 2000, were used for the present analysis. Figures 1a, b show the geographic location of the study area in relation to Iran and the rainfall network stations respectively.

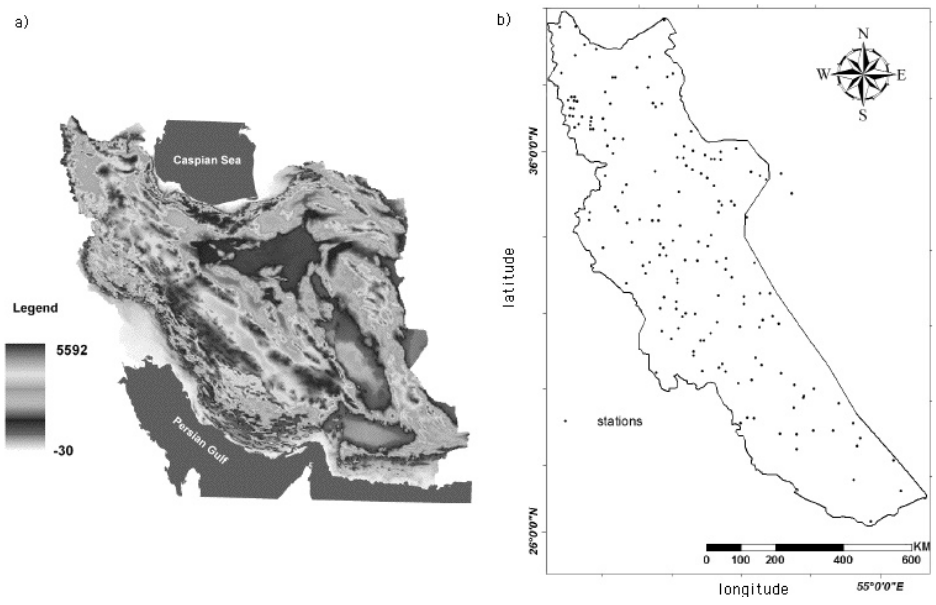


Figure 1. a) Topographic map of Iran and b) stations network across the study area.

Among the SPI 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 (say 12 and 24 months) it is more suitable for water resources management purposes. In this paper, results concerning the 12-month time scale (SPI-12 hereafter) 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). Notice that, using this time scale we avoid the seasonal cycle, while the memory effect associated with the inter-annual variability is still accounted for. The SPI time series were computed following the method described by McKee *et al.* (1993).

To capture the patterns of co-variability of drought at different stations, the S-Mode PCA (Richman, 1986) was applied to the SPI-12 time series. The method consists in computing the covariance matrix of the SPI data with the corresponding eigenvalues and eigenvectors. 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-12 time series at single stations) and the corresponding principal component time series. The first four principal components, explaining about 61.5% of total variance, were considered based on a sharp break point on the screeplot. Moreover, in order to extract more localized spatial patterns, the retained loadings were rotated using Varimax rotation (Von Storch and Zwiers, 1999). The time variability of drought events across the study area was determined by the four rotated PC score time series.

Finally, the obtained rotated spatial patterns of drought variability across the study area were compared with the subdivisions found by Raziei and Azizi (2007). In that paper, the authors have regionalized the study area into five homogenous subdivisions as Azarbayjan (AZ), Western Zagros (WZ), Eastern Zagros (EZ), Persian Gulf (PG) and Pars-Hormoz (PH) (see Figure 2a).

3. RESULTS

Based on the screeplot (Figure 2b), the first four loadings and the corresponding PC scores of the SPI-12 have been considered. Figures 3 and 4 show the first four un-rotated loadings and the corresponding PC scores time series respectively. The first loading, which explains 38.3% of the total variance, shows high positive values over the whole area with the exception of some stations in AZ region in the north. This means that there is a high positive and spatially homogeneous correlation between drought variability in all stations and the corresponding PC score. The first PC score (PC-1) shows multi-year fluctuations and remarkable dry events are noticeable around 1971, 1974, 1984, 1986, 1991 and 1999. The second loading, which explains 14.1% of the total variance, has positive values mainly in AZ subdivision in the north of the

study area. The corresponding PC score shows a multi-year variability though embedded on a linear trend towards negative values (i.e. dry periods) from the eighties onward.

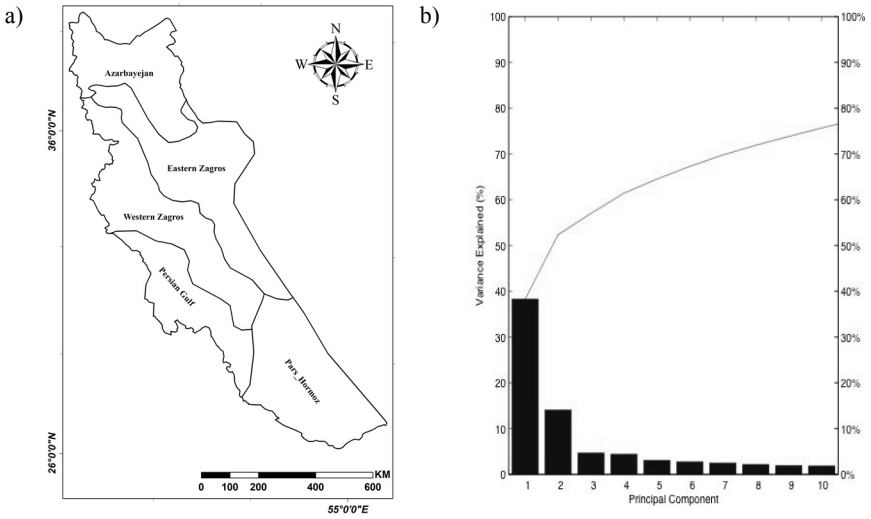


Figure 2. a) Climate subdivisions of the study area by Raziei and Azizi, (2007) and b) the scree plot of PCA (first ten components, solid line denotes the cumulative variance).

The period 1994-1995 was characterized by a severe wet event. In the third loading (4.7% of explained variance) positive values are localized in few stations in northeast of the study area, say EZ subdivision. The time behaviour of PC-3 shows a high temporal variability; years 1968, 1994 and 1998 were characterized by severe dry spells. The fourth loading explains 4.4% of the total variance and has positive values in few stations in the west of the study area (WZ and AZ subdivisions). The corresponding PC score shows multi-year fluctuations superposed on a cyclic-like pattern with a long-duration drought from 1976 to 1985 preceded and followed by two long-duration normal or wet periods. In the following we will show how an orthogonal rotation applied to the four loadings can provide more localized areas of drought variability.

The four loadings above were rotated using Varimax rotation. The rotated loadings 1, 2 and 3 explain most of the total variance (54.0%) with 25.2, 15.6 and 13.2%, respectively. The fourth rotated loading accounts for 7.5% of the total variance. Rotated loadings and PC scores are shown in Figure 5 and 6 respectively. The first rotated loading shows positive values in the south of the study area, including all of PH and PG subdivisions and southern parts of WZ and EZ subdivisions. The corresponding rotated PC score (RPC-1) suggests that several severe drought periods occur there: the drought of 1966 is followed by 1971, 1974, 1984-1986, 1994 dry events.

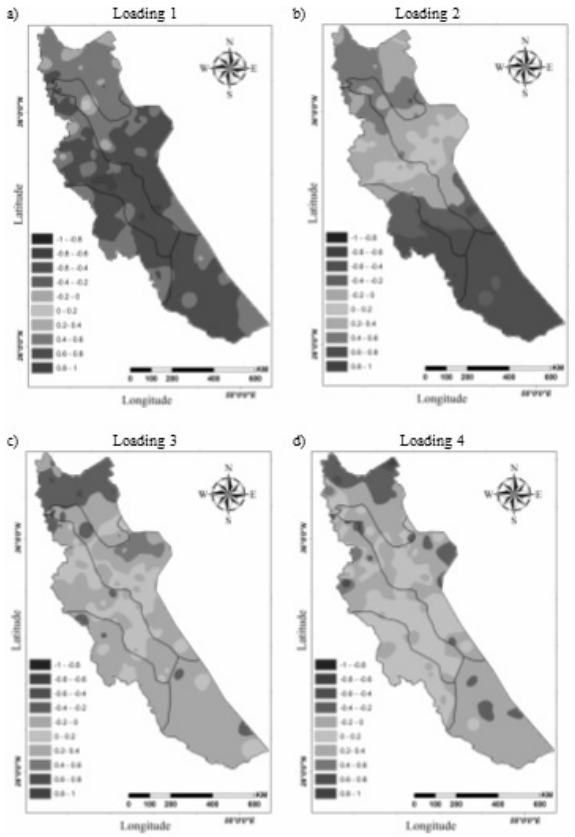


Figure 3. First four loadings of the SPI-12.

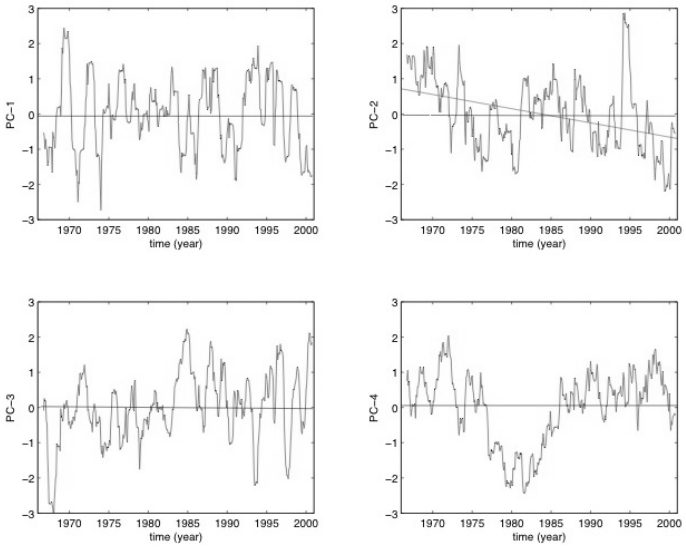


Figure 4. First four PC scores. Straight line denotes the linear trend.

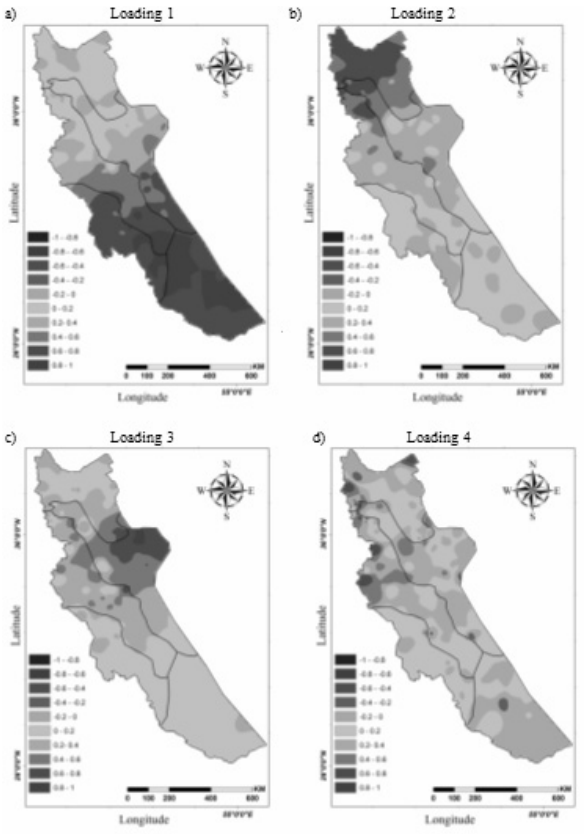


Figure 5. Varimax rotated loadings.

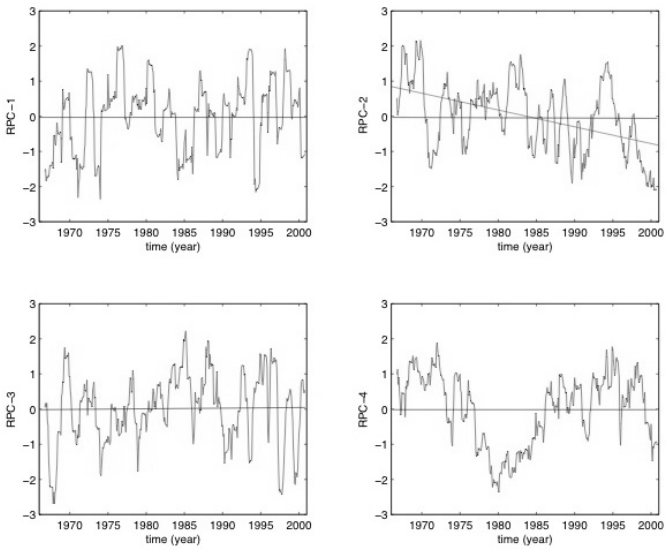


Figure 6. Rotated PC scores. Straight line denotes the linear trend.

The second rotated loading has high positive values in AZ region and the northernmost of WZ and EZ subdivisions in the north of the study area. The rotated PC-2 score shows a time behaviour characterized by multi-year fluctuations superposed on a linear trend towards negative values from the eighties onward. Positive values of the third rotated loading are mainly localized in northeast of the study area (EZ subdivision and some parts of AZ subdivision) and the corresponding rotated PC score is related with frequent drought occurrences. Finally, the fourth rotated loading has high positive values in few stations on the west; say WZ and AZ subdivisions. The corresponding rotated PC score resembles the forth un-rotated score shown in Figure 4: except a long dry period from 1976/77 to 1984/85, the rotated PC-4 characterizes near normal or wet conditions.

Thus, the rotated loadings seem to be well localized in space, identifying four specific sub-regions of the study area. However, it should be noted that the fourth rotated pattern characterizes only a few stations in the west side of the study area and explains a small percentage of the total variance (7.5%). This suggests that the main areas of drought variability are characterized by the first three rotated loadings. The time variability of drought in such sub-regions is given by the time behaviour of the corresponding rotated scores. In particular, it appears that the northern part of the study area is characterized by a climatic trend towards drier periods in agreement with the results obtained for the global scale by Bordi *et al.* (2006) using NCEP/NCAR re-analysis data. Finally, we can say that, as expected, the precipitation based regionalization (Raziei and Azizi, 2007) does not apply for drought. This is because drought is a relative measure; say the deviation of actual precipitation from a historically established norm.

4. CONCLUSIONS

The time and space variability of drought in western Iran was studied by using rain gauge data from 170 stations distributed uniformly over the area. Drought conditions were assessed through the SPI computed on 12-month time scale, while their variability was analyzed applying the PCA to the index time series. Results suggest that the first loading has high positive values over the whole area. This means that there is a high positive and spatially homogeneous correlation between drought variability for all stations and PC-1 score, which is characterized by multi-year fluctuations. The second loading, instead, identifies the north of the study area as the region where the SPI-12 time series are characterized by a linear climatic trend towards drier periods though embedded on multi-year variability.

Applying a Varimax rotation to the first four loadings we found more localized areas of drought variability. In particular, it seems that there are three sub-regions having different climatic variability. Also in this case, the northern part of the study area is characterized by a climatic linear trend toward drier periods from the eighties onward, in agreement with recent studies on drought variability at global scale.

Thus, for an efficient water resources management under dry conditions and in planning measures for mitigating the adverse impacts of future drought occurrences, these three sub-regions should be separately considered.

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Drought Risk Analysis in Water Supply Systems using Genetic Algorithms and Montecarlo Simulation

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Abstract: The evaluation of risk of shortages due to drought is a necessary task both in the planning and in the operation of water supply systems. A methodology for unconditional (planning) and conditional (operation) risk evaluation is presented in this study. The evaluation is carried out by means of a Montecarlo simulation model which uses a stochastic generation model to generate hydrological input and an optimization model based on genetic algorithms to determine triggers for drought management levels. Results from simulations are used to compare, in a probabilistic way, effects on the system of given mitigation measures. The Genetic Algorithms enables to optimize monthly thresholds for shifting among three states of the system (normal, alert and alarm) to which correspond different mitigation measures such as water demand rationing, additional supplies from alternative sources or reduction of release for ecological use. For unconditional risk evaluation a long time horizon has been considered (40 years) while the conditional risk evaluation is performed on a short time horizon (2-3 years). The analysis of the results includes consolidated indices of performance (reliability, resilience and vulnerability) and frequency analysis of shortages of given entity in correspondence of different planning/operation policy. A multiuse water system has been used as case study including competing irrigation and industrial demands.

Keywords: Drought, risk evaluation, triggers for drought plan, water management.

1. INTRODUCTION

Risk analysis plays a key role on decision making process especially for potentially drought-prone systems and a methodology which includes evaluation of risk as a tool for defining effective triggers for a predefined set of mitigation measures of a given drought mitigation plan is a desirable goal. One of the most difficult task on the definition of a general methodology for evaluating risk of shortages due to drought in water supply systems is strictly related to the vagueness of the concepts involved on the evaluation. Indeed the definition of risk is itself not unique and the definition of

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risk due to droughts in a water system is even a more fuzzy concept.

The aim of this paper is to set up a methodology to assess the risk of shortage due to drought for a multifacility system with different demands. The study is carried out by means of an heuristic optimization model based on genetic algorithms, a stochastic model for the generation of hydrological series and a Montecarlo simulation approach to evaluate the performance of the proposed methodology. The goal is sought through the heuristic optimization of triggers based on the status of the system to activate levels of alert and alarm which operational policy is characterized by pre-defined mitigation measures to contrast the effects of incoming or ongoing operational drought. In particular the assessment of operational drought risk for the system is carried out analyzing results of simulations through traditional indices of performance and probability distribution of shortage belonging to different classes of severity for different demands including those for environmental purposes.

2. METHODOLOGY

Unconditional risk assessment (planning) has the objective of the evaluation and the selection of preferred drought mitigation alternatives through the Montecarlo simulation over a long time horizon (30-40 years) of the system behaviour by using generated series. The risk is evaluated in terms of a synthetic assessment of failure based on the analysis of the satisfaction of consumptive demands (both in time and volume), as well as of meeting some specified objectives such as the satisfaction of ecological requirements.

The results of the Montecarlo analysis enable to verify whether the system exhibit an acceptable risk of shortage under the given configuration, set of operating rules and implementation of mitigation measures triggered by opposite optimized thresholds. If this is not the case, the procedure can be repeated by analyzing different configurations and/or set of mitigation measures (Cacelliere *et al.*, 2007).

The proposed procedure for conditional (operation) risk assessment has the objective of evaluating of the shortage risk in a short time horizon by using generated series. The procedure makes use of the same basic tools (namely stochastic data generation, water system simulation and synthetic assessment of performance), but in this case the analysis is performed with reference to a shorter time horizon (2-3 years) and by taking into account the initial state/conditions of the system. Thus, the preliminary results will be revised as new data is available.

Therefore, such procedure should be carried out at given time steps (e.g. every month) during the operation of the system, in order to identify potential failures in the future and to implement the necessary measures. Application of the proposed methodology enables to assess, in a probabilistic way, the short term risk of failures considering the actual condition of the system, thus giving the opportunity to explore effects of different policies of management and mitigation measures (Nicolosi, 2006).

Among heuristic programming models, genetic algorithms (GA) have gained a

growing importance in water management applications. GA are based on natural selection and genetic evolution mechanisms and mainly use three heuristic processes, namely reproduction, crossover and mutation in a probabilistically manner to binary coded discrete variables repeatedly modifying a population of individual solutions to approach the optimal solution. At each step, the GA selects individuals at random from the current population to be parents and uses them produce the children for the next generation. Over successive generations, the population evolves toward an optimal solution. One of the advantages of genetic algorithms is that can be used to solve a variety of optimization problems that are not well suited for standard optimization algorithms, including problems in which the objective function is discontinuous, nondifferentiable, stochastic, or highly nonlinear.

In the present study GAs have been used to optimize breakpoints of piecewise annual curves of triggers used as thresholds among three different states of the system (normal, alert and alarm) by linking the genetic algorithm with a simulation model of the system. Constraint terms have been used in order to maintain a given degree of satisfaction of two different competing goals of optimization and to express constraints on values of triggers for alert conditions to be higher than those assumed by triggers for alarm conditions.

3. CASE STUDY

3.1. The Acate Water Supply System

The proposed methodology has been applied to a water supply system located in the Acate river basin in the south-eastern Sicily. It includes the Ragoleto dam located on the higher part of the Acate river and the Mazzarronello diversion on the Mazzarronello stream tributary of the Acate.

Ragoleto dam and Mazzarronello diversion watersheds, respectively 117.5 km^2 and 70.1 km^2 are characterized by a generalized low permeability typically giving a torrent-like behaviour to the Acate river and its tributaries. Available hydrological data include historical (1964-2003) monthly flows for the Acate river at Ragoleto reservoir and for Mazzarronello stream at the diversion. Ragoleto reservoir has been built during 1961-62 and has a net design capacity of 21.7 hm^3 which is currently limited, due to sediment transport phenomena, to a maximum storage of 20.4 hm^3. Linked to the Mazzarronello diversion, built during the 80's, there is a small reservoir of about 600000 m^3 where water for irrigation use is stored.

The Acate water system supplies both irrigation and industrial demands respectively of 6.0 hm^3/year (Land Reclamation Consortium no. 8) and 6.5 hm^3/year (National Hydrocarbon Agency ENI).

The Ragoleto reservoir supplies both the uses, the mentioned industrial demand mainly deriving from a petrochemical area that is also supplied by a desalination plant for a maximum water production of 611 l/s and part of the irrigation demand of the

LRC 8. The entire releases from the Mazzarronello diversion are devoted to the irrigation demand of the LRC 8.

Both, Ragoleto reservoir and Mazzarronello diversion, release downstream flows for ecological use. In particular the instream flow requirements (IFR) for the two sections have been calculated following a procedure that takes into account a constant base flow function of the watershed extension corrected on the basis of the annual mean precipitation and the inter-annual variability of the natural streamflows.

3.2. Stochastic Generation of Streamflow Series, Mitigation Measures and Hydrological States

Generation of synthetic streamflow data has been performed by means of the software SAMS (Sveinsson *et al.*, 2003). The lag 0 monthly cross correlation between the two streamflow series (Ragoleto inflow and Mazzarronello streamflows) is significant in several months, thus the stochastic modeling of the two series must be carried out by means of a seasonal multivariate model, able to take into account the cross correlations, as well as their seasonal variability from month to month. The proposed generation scheme firstly provides an annual and monthly data transformation in order to reduce skewness, then annual data are generated by means of multivariate autoregressive model (CARMA 1,0) and, finally, monthly data are generated by means of an appropriate disaggregation scheme.

Simulation of the water system has been carried out using the specifically developed software SIMDRO (Simulation of systems under drought conditions) able to simulate the system in correspondence of the implementation different mitigation measures triggered by thresholds established by the user. Through triggering thresholds three different conditions of the system, namely normal, alert and alarm can be defined as a function of stored volumes in reservoirs. For each simulation step SIMDRO compares the value of the variable chosen as significant of the general state of the system with the monthly thresholds in order to establish in which condition the system lies and, as a consequence, what is the mitigation measure to activate. The general framework of priorities used to manage the Acate water system consists in:

1. Fulfilment of instream flow requirement (IFR) for Ragoleto and Mazzarronello
2. Release from Mazzarronello diversion to the irrigation demand
3. Release from Ragoleto reservoir to the irrigation demand
4. Release from Ragoleto reservoir to the industrial demand
5. Release from desalination plant to the industrial demand

For the Acate water system the mitigation measures chosen refer to reductions of releases that grow in percentage in correspondence of worsening hydrological conditions and/or a larger production of desalinated water for the industrial use (see Table 1). Management policy during the different conditions of the system under study have been expressed in terms of water priority allocation among different uses, percentage of demand reduction for consumptive and ecological uses, increased access to desalinated water.

Table 1. Mitigation measures adopted in correspondence of different drought conditions for the Acate water system.

Drought condition	Sequence of actions
Alert	20% reduction of IFR for Ragoleto and Mazzarronello 20% reduction of the release from Ragoleto to the irrigation demand 35% reduction of the release from Ragoleto to the industrial demand
Alarm	50% reduction of IFR for Ragoleto and Mazzarronello 35% reduction of the release from Ragoleto to the irrigation demand No release from Ragoleto to the industrial demand

The values of thresholds for the different condition of the system that trigger the above mitigation measures have been defined through an optimization based on genetic algorithms (GA) considering the historical series as hydrological input. In particular the following parameters have been selected to implement the genetic algorithm technique i) Population size: 10000; ii) Generations: 10000; iii) Elite parents: 200; iv) Crossover fraction: 0.8; So, given the population size of 10000, the elite parents group of 200, and the crossover fraction of 0.8, the numbers of each type of children in each new generation is as follows: 200 elite children, there are 9800 individuals other than elite children, so the algorithm counts 0.8*9800 = 7840 of crossover children while the remaining 1960 individuals are mutation children. Maximum number of generation (10000) has also been used as stopping criterion for the algorithm.

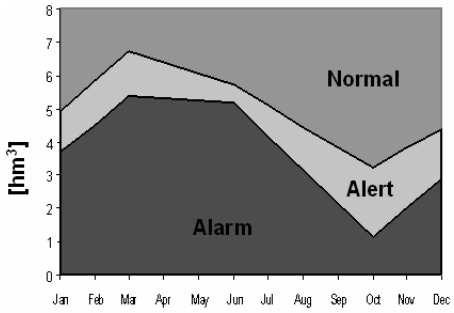


Figure 1. Monthly thresholds of the stored volume in the Ragoleto reservoir corresponding to the best trade-off solution of the constrained GA optimization.

In order to define which value of the objective function should be considered the best trade-off between the satisfaction of the two competing demands a curve obtained considering as objective function the squared sum of irrigation deficits respecting constraints related to the maximum value of the squared sum of industrial deficits has been analyzed.

The values of the thresholds, obtained by means of the GA optimization, corresponding to the pre mentioned optimal solution are shown in figure 1. Optimized thresholds are strictly related to the demand patterns showing a maximum on March as precautionary of the irrigation season starting on May and a minimum on October

corresponding at the end of the irrigation season.

3.3. Results of Unconditional and Conditional Risk Evaluation

The evaluation of the unconditional risk of shortages due to drought for the Acate water system has been carried out by means of two set of simulations. In the first set the system has been simulated considering only the management policies described for the normal condition, that is without trigger any mitigation measure. In the second one the system has been studied considering the activation of the described mitigation measures based on the value of thresholds set for the different conditions. As hydrological input for the simulations 500 stochastic generated series of streamflows of the same length of the available historical series (40 years) have been considered.

Table 2 compares some significant performance indices of the water system simulated with and without the triggers of the mitigation measures for alert and alarm conditions estimated on demands and on minimum in stream flow at the two sections. Figure 2 shows that implemented mitigation measures are effective on both demands. In particular, even if triggering mitigation measures has risen the probability to have deficit for some months of the irrigation season, nevertheless those deficits mainly belong to the 0-25% class and, just in some cases, to the 25-50% class obtaining an important reduction of severe deficit and a more frequent realization of mild deficits that are more tolerable by the crops.

Table 2. Indices calculated by simulating the Acate water system without (a) and with (b) the triggering of the mitigation measures for alert and alarm conditions (unconditional risk).

	Time based reliability		Volume based reliability		Mean deficit length [month]		Sum of deficit² [hm³]	
	a	b	a	b	a	b	a	b
Irrigation use	0.79	0.72	0.81	0.92	2.86	2.97	56.97	9.54
Industrial use	0.87	0.82	0.97	0.95	3.33	3.98	1.11	1.75
IFR Ragoleto	0.81	0.59	0.93	0.86	1.52	4.24	-	-
IFR Mazzarronello	0.63		0.78		1.86		-	-

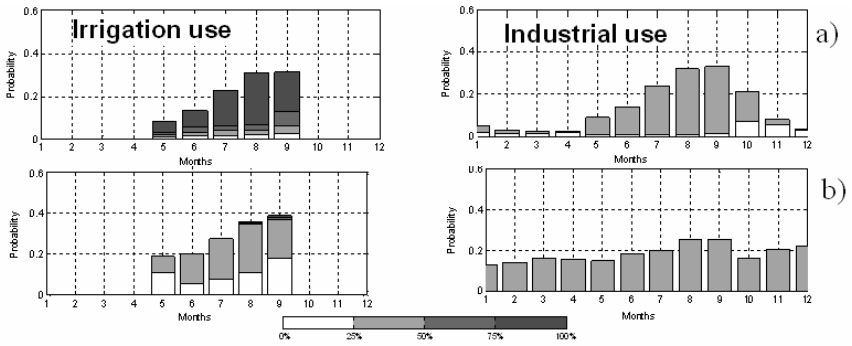


Figure 2. Probability of monthly deficit for irrigation and industrial use without a) and with b) mitigation measures for unconditional risk assessment.

To the general advantage obtained on the irrigation demand corresponds an increased probability of deficit on industrial demand. Those deficit could be reduced by increasing the volume of desalinated water produced by the desalination plant.

The evaluation of the shortage risk during the current operation of the system (conditional risk) been carried out considering as hydrological input 500 stochastic generated series of 36 months and initial conditions equal to those of January 2000 (4.1 hm³ stored in Ragoleto) corresponding to the beginning of a severe recorded drought period.

Firstly the simulations have been carried out without triggering any mitigation measure; then the previously described mitigation measures have been triggered based on the comparison between the stored volume in Ragoleto and the determined thresholds. Table 3 compares performance indices of the water system with and without the triggers of the mitigation measures.

Table 3. Indices calculated by simulating the Acate water system without (a) and with(b) the triggering of the mitigation measures for alert and alarm conditions (conditional risk).

	Time based reliability		Volume based reliability		Mean deficit length (month)		Sum of deficit² (hm³)	
	a	b	a	b	a	b	a	b
Irrigation use	0.89	0.84	0.91	0.95	1.09	1.46	2.04	0.42
Industrial use	0.94	0.90	0.98	0.97	1.42	1.79	0.04	0.07
IFR Ragoleto	0.83	0.72	0.96	0.92	1.20	1.33	-	-
IFR Mazzarronello	0.67		0.84		1.75		-	-

Figure 3 shows the probability of deficit belonging to the different considered classes in the case at the beginning of the drought event starting on January 2000 the system has been managed with or without the triggering of the mitigation measure.

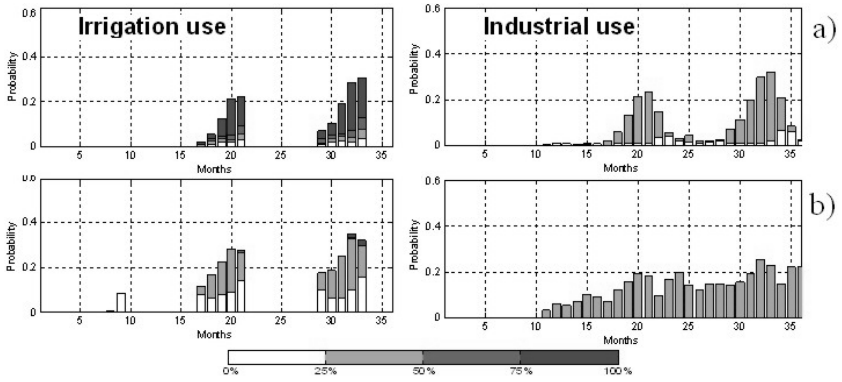


Figure 3. Probability of monthly deficit for irrigation and industrial demands without a) and with b) mitigation measures for conditional risk assessment.

The effects of the proposed mitigation measures on the following 36 months are generally to increase the probability of mild deficit avoiding deficit belonging to the

class 50-100% occurred in the case of no mitigation measures application. For industrial demand the probability of deficits belonging to the class 0-50% is distributed in a almost uniform manner during the 36 future months avoiding the variability characterizing the management without any mitigation measure.

4. CONCLUSIONS

A methodology has been proposed for the evaluation of risk of shortages due to drought considering unconditional risk, typical of the planning stage and characterized by long time horizon and conditional risk, typical of the operational stage, strongly conditioned by the initial state of the system and characterized by a short time horizon. Application to a real world case study has shown that the proposed methodology based on optimization of thresholds corresponding to a given set of mitigation measures, stochastic generation of streamflow series, Montecarlo simulation and probability analysis of deficits of given degree constitutes a valid aid in order to evaluate the risk of shortage due to drought for a water system at long and short term. Further research should be focused on the social-economic assessment of the results by means of multicriteria analysis techniques able to take in to account aspects of the problem otherwise difficult to compare.

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Efficacy of Palmer Drought Indices in Hydrological Drought Evaluation

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Abstract: This study evaluates the efficacy of Palmer drought indices to monitor hydrological droughts in river discharges and soil moisture in selected watersheds with varying geomorphologic characteristics in region of Thessaly, Greece. The Palmer four indices (PDSI, Weighted PDSI, PHDI and the moisture anomaly Z-index) were used as indicators of meteorological drought severity. The hydrological drought severity was evaluated from the outputs of the monthly UTHBAL conceptual water balance model. The UTHBAL was calibrated with the available observed runoff data to extend, reconstruct and produce runoff and soil moisture timeseries for the hydrologic period 1960-2002 at the study catchments. The produced hydrologic variables were normalized through Box-Cox transformation and standardized to normal distribution. The standardized hydrologic variables were used as an indicator of hydrological drought severity and were compared with the Palmers' drought indices timeseries estimated by basin-wide meteorological data. The results showed that the Weighted PDSI and the moisture anomaly Z-index were found to better represent river discharges and soil moisture respectively, for all study watersheds irrespectively to their area, geophysical, and hydroclimatic characteristics.

Keywords: Drought indices, hydrological drought, meteorological drought, PDSI.

1. INTRODUCTION

In many Mediterranean countries environmental policy and water resources and drought policies have been badly structured and organized and continue to negatively impact water resources in most countries. Water Framework Directive (WFD) in the European Union, sets the general framework for water management in member states and requires "good ecological status" to be achieved in EU waters by 2015. It promotes the integrated management of water resources to support sustainable water use and reduce problems associated with excessive water abstraction, pollution,

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floods and droughts. For effectively preventing droughts and increase resilience to its impacts, it is indispensable to achieve a rational use of water, even in rainy years. It is worth mentioning that a better management of water resources and demand is listed as the first among seven priorities identified by the Mediterranean Strategy for Sustainable Development finalized in 2005 and adopted by the 21 Mediterranean countries and the European Union in the framework of the Barcelona Convention.

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 the precise quantification of drought is a difficult geophysical endeavor. Important efforts for developing methodologies to quantify different aspects related to droughts have been made, such as the spatial differences in the drought hazard (Loukas and Vasiliades, 2004; Vicente-Serrano and Begueria, 2003), the prediction of droughts by means of the use of atmospheric circulation indices (Lloyd-Hughes and Saunders, 2002), and the mitigation of drought effects (WMO, 2000). However, more efforts have been made to develop drought indices, which allow an earlier identification of droughts, their intensity and surface extent. During the twentieth century, several drought indices were developed, based on different variables and parameters (Heim, 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 (WMO, 2000).

The different response of hydrological systems to precipitation can vary noticeably as a function of the time scales. McKee *et al.* (1993) indicated that usable water resources include soil moisture, groundwater, snowpack, river discharges, and reservoir storages. The time period from the arrival of precipitation until usable water is available differs considerably. Thus, the time scale over which precipitation deficits accumulate becomes extremely important and functionally separates different types of drought: hydrological, environmental, agricultural, urban, etc. 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.

This paper analyses the usefulness of Palmer drought indices to monitor droughts in river discharges and soil moisture. Moreover, the monthly differences in the response of both hydrologic sub-systems to droughts were also analyzed. The study was carried out in selected watersheds located in Thessaly region, Greece, where there is a high complexity of hydrological processes with a marked seasonal and interannual variability. The objective is to determine the most adequate drought indices to monitor droughts in two basic water usable sources: river discharges and soil moisture. This is useful for forecasting and monitoring hydrological and soil moisture drought severity and improving the assessment of the availability of water resources by means of monthly meteorological data.

2. STUDY AREAS

Thessaly is a plain region surrounded by high mountains. The Thessaly plain is one of the most productive agricultural regions of Greece. 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 the Thessaly region is about 700 mm, varies from about 400 mm at the central plain area to more than 1850 mm at the western mountain peaks and it is distributed unevenly in space and time. Generally, rainfall is rare from June to August. The mountain areas receive significant amounts of snow during the winter months and transient snowpacks develop.

Seven (7) small medium and large watersheds with areas ranging from 132 to 6591 km^2 located in the western region of Thessaly have been selected to demonstrate the efficacy of Palmer drought indices to monitor hydrological drought (Figure 1). Processed monthly precipitation data from a large network of precipitation stations (Figure 1) for the period October 1960 to September 2002 were used. The monthly areal precipitation of each watershed 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 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 the seven water-gauging stations were used for the calibration of the hydrological model. The records of the water gauging stations were discontinuous with large gaps.

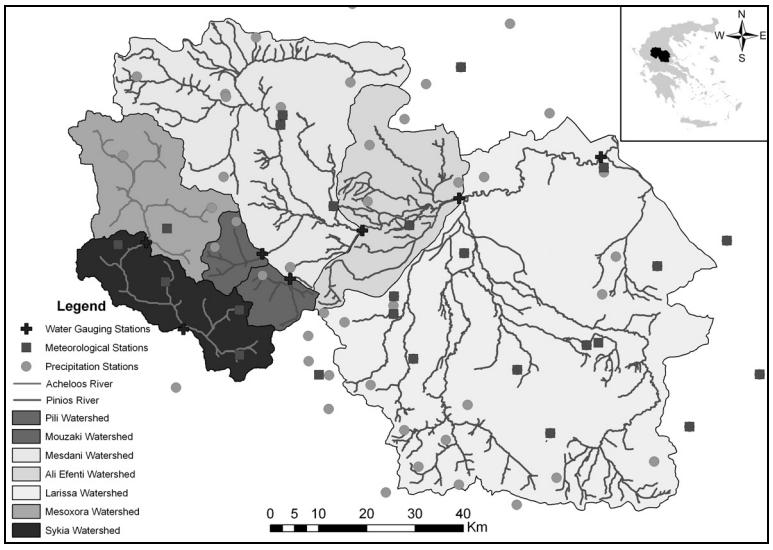


Figure 1. Study area and database.

3. METHODOLOGY

This study evaluates the efficacy of meteorological drought indices to monitor hydrological droughts in river discharges and soil moisture in selected watersheds in the region of Thessaly, Greece. The Palmer four indices (PDSI,Weighted PDSI, PHDI and the moisture anomaly Z-index) were used as indicators of meteorological drought severity. The hydrological drought severity was evaluated from the standardized outputs of the monthly UTHBAL conceptual water balance model. Standardised data of river discharges and soil moisture were compared with the Palmers' drought indices considering three approaches: continuously (considering all the months as a continuum), monthly and for selective historical drought events. For comparison the Pearson correlation coefficient was used. In the next paragraphs the Palmer drought indices and the UTHBAL model are presented.

3.1. Meteorological Drought Indices

The Palmer Drought Severity Index (PDSI) was introduced by Palmer (1965) for the assessment of meteorological drought. Although PDSI is referred to as an index of meteorological drought, however, the 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 PDSI measures 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. Complete description of the equations of the PDSI calculation procedures can be found in the original paper (Palmer, 1965).

The PDSI varies roughly between -6 (drought periods) and +6 (wet periods) with the zero value indicating the normal conditions. The PDSI is standardized for different regions and time periods to facilitate direct comparisons of the PDSI between different regions (Guttman *et al.*, 1992). Despite several assumptions used in the water balance calculations, other limitations and deficiencies, and the empirical nature of some of the standardized coefficients, the PDSI can be a useful tool for both research and operational drought assessment (Rao and Voeller, 1997; Heim, 2002).

In the present study four Palmer indices were included in the analysis. These 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 (Weighted PDSI). The modification was

made by the National Weather Service Climate Analysis Center for operational meteorological purposes (Heddinghaus and Sabol, 1991). The PDSI calculates three intermediate parallel index values each month. Only one value is selected as the PDSI drought index for the month. This selection is made on the basis of probabilities. If the probability that a drought is over is 100%, then one index is used. If the probability that a wet spell is over is 100%, then another index is used. If the probability is between 0% and 100%, the third index is assigned to the PDSI. The modification incorporates a weighted average of the wet and dry index terms, using the probability as the weighting factor. The Weighted PDSI 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.

Timeseries of monthly precipitation and temperature were used for the calculation of Palmers' indices. These timeserieses were estimated using the methods presented in the previous paragraphs. The estimation of the available water capacity was taken from the UTHBAL model for all study watersheds.

3.2. Hydrological Drought Assessment

For the assessment of hydrological drought severity (surface runoff and soil moisture) because most discharge river measurements are intermittent, the monthly conceptual water balance UTHBAL model (Loukas *et al.*, 2006) has been used. The water balance model allocates the watershed runoff into three components, the surface runoff, the interflow runoff and the baseflow runoff using a soil moisture mechanism. 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 (WMO, 1986).

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. The optimization was performed using the Simplex Downhill Algorithm and the Nash-Sutcliffe Model Efficiency was used as the objective function. The UTHBAL model was calibrated with the available observed runoff data to extend, reconstruct and produce runoff and soil moisture timeseries for the period of analysis 1960-2002 for all study watersheds. The synthetic runoff and soil moisture timeseries were used for the estimation of hydrological drought. The synthetic runoff and soil moisture were normalized through Box-Cox transformation and standardized to normal distribution. The transformed and standardized runoff and soil moisture timeseries were used as an indicator of hydrological drought severity and were compared with the Palmers' drought indices timeseries estimated by basin-wide meteorological data.

4. APPLICATION AND RESULTS

Table 1 shows the Pearson correlation coefficients between the continuous standardized series of river discharges, soil moisture and the four Palmer indices. For river discharges and soil moisture, correlations are positive, but there are important differences with regard to examined meteorological drought indices. Considering river discharges, higher correlations have been obtained with the Weighted PDSI (average R = 0.80 for all watersheds) followed by the PDSI (Table 1) for all study watersheds. Considering soil moisture, higher correlations have been obtained with the moisture anomaly Palmer Z-index (average R = 0.86 for all watersheds). Graphical examination of temporal evolution confirms that the Weighted PDSI and the Palmer Z-index are representatives of river discharges and soil moisture for all study watersheds.

Table 1. Correlation between continuous meteorological drought indices and the standardized hydrological variables.

Watershed	Palmer Z-index	PDSI	Weighted PDSI	PHDI
River discharge (runoff Z-index)				
Mouzaki	0.68	0.72	0.77	0.68
Pili	0.69	0.76	0.80	0.70
Mesdani	0.73	0.76	0.79	0.68
Ali Efenti	0.73	0.78	0.81	0.71
Larissa	0.66	0.76	0.78	0.70
Mesoxora	0.72	0.77	0.81	0.71
Sykia	0.74	0.77	0.80	0.71
Soil moisture (soil moisture Z-index)				
Mouzaki	0.82	0.68	0.72	0.61
Pili	0.85	0.73	0.75	0.62
Mesdani	0.89	0.72	0.74	0.61
Ali Efenti	0.88	0.74	0.76	0.65
Larissa	0.86	0.75	0.76	0.66
Mesoxora	0.86	0.73	0.76	0.63
Sykia	0.87	0.74	0.75	0.63

The monthly patterns follow the results of the continuous correlation analysis. Again the Weigthed PDSI and the Palmer Z-index are the best indices for assessing river discharges and soil moisture, respectively. The monthly correlation coefficients are better for autumn and winter months and worse for spring and summer months. A possible reason could be the accumulation of snow and snowmelt, which is not accounted in the calculation of the Palmer indices, and causes redistribution of runoff especially in the spring months.

Previous studies (Loukas and Vasiliades, 2004) have shown that significant drought events have been established in the years 1976-1977, 1988-1990 and 2001. Figure 2 presents the comparison of Palmer indices with the drought event 1988-1990 and the drought event 1976-1977 for Larissa watershed. For these historical drought events the weighted PDSI followed by the PDSI are the best indices for depicting

hydrological drought of river discharges (Figure 2a). Soil moisture drought is best described with the Palmer Z-index. However, the response of the other Palmer indices for assessing soil moisture is improved (Figure 2b). Similar patterns are observed and for the other identified historical drought events for all study watersheds.

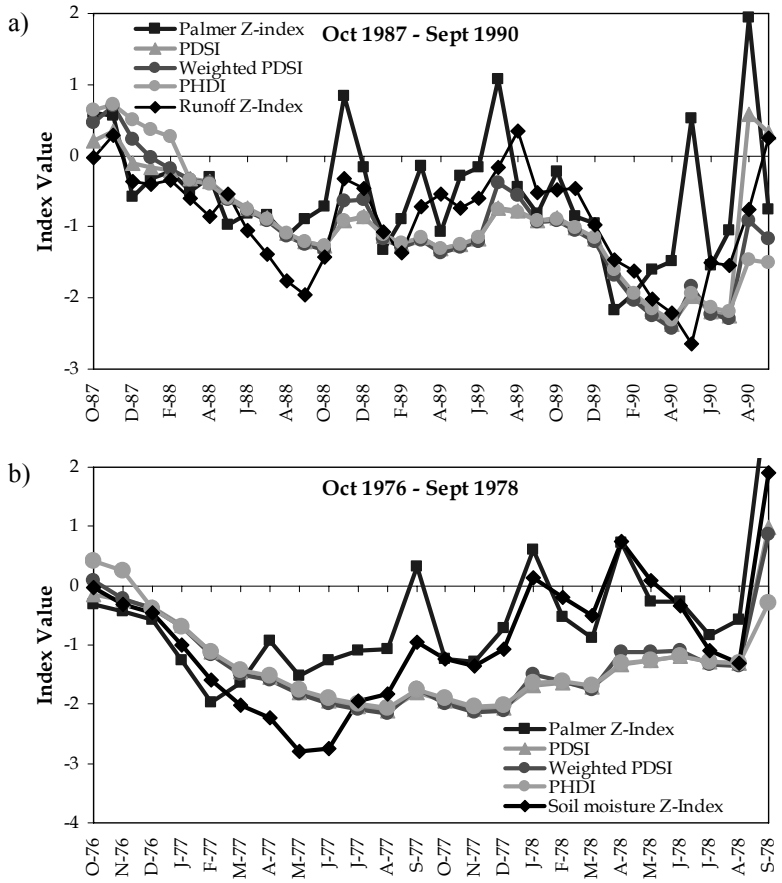


Figure 2. Comparison of Palmer indices (rescaled to the scale of hydrologic variables) with standardized a) river discharges and b) soil moisture for Larissa watershed.

5. CONCLUDING REMARKS

This study analyses the usefulness of the four Palmer drought indices to monitor droughts in river discharges and soil moisture. The study was carried out in selected small, medium and large watersheds located in the Thessaly region of Greece, where there is a high complexity of hydrological processes with a marked seasonal and

interannual variability. The results showed that the weighted PDSI and the moisture anomaly Z-index are the most adequate drought indices to monitor droughts in two basic hydrologic variables: river discharges and soil moisture. This is useful for forecasting and monitoring hydrological and soil moisture drought severity and improving the assessment of the availability of water resources by means of monthly meteorological data.

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Spatial and Temporal Drought Analysis in Greece

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Abstract: Monthly rainfall data for 32 selected stations in Greece, covering the period (1957 2001), are used for a spatial regionalization of the country, according to rainfall characteristics by using principal component analysis. These data are also used in the computation of the SPI9 and SPI12 indices, as developed by McKee *et al.* (1993), for two time series, (1957-2001) and (1974-2001), in order to investigate the effect of the length of record on obtaining both indices. Probability density functions are also calculated for estimating drought occurrence on 9-month and 12-month time scales. Spatial regionalization of drought in Greece is achieved by performing principal component analysis on both SPIs, followed furthermore, by a cluster analysis to SPI12 loadings.

Keywords: Cluster analysis, drought analysis, principal component analysis, standardized precipitation index.

1. INTRODUCTION

Drought is a current phenomenon with impacts on many fields of the social development and human life in general. There are many drought indices developed to monitor and define hydrological drought but the standardized precipitation index is one of the most simplest widely used by scientists. This index is very easily handled as only precipitation data for at least 30 years are required for its computation. Many recent works based on this index have successfully described drought (Lana *et al.*, 2001; Keyantash *et al.*, 2002; Bonaccorso *et al.*, 2003; Bordi *et al.*, 2004; Tsakiris and Vangelis, 2004; Sonmez *et al.*, 2005; Wu *et al.*, 2005; Livada and Assimakopoulos, 2007). In this study, monthly rainfall values are handled through principal component analysis for grouping the 32 Greek stations according to their rainfall patterns. The standardized precipitation indices, SPI9 and SPI12 are calculated for each station and an attempt is made to investigate the effect of the length of the period of record on the calculation of both SPIs. The corresponding probability density functions are also

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estimated. Finally, SPI_9 and SPI_{12} are spatially regionalized by means of principal component analysis followed by cluster analysis.

2. ANALYSIS AND RESULTS

Monthly rainfall data, covering the period (1957-2001), obtained from the National Meteorological Service for 32 sites representing various climatic conditions in Greece are used in the analysis. The selected sites are presented in Table 1.

Table 1. Rainfall's principal components loadings for the stations of the study.

Station	LAT	LONG	ALT (m)	PC_1	PC_2	PC_3	PC_4	PC_5
Anogeia	35º17	24º53	740.0	0.907	0.139	0.121	0.034	0.036
Iraklio	35º20	25º11	37.0	0.887	0.184	0.180	0.050	0.043
Shteia	35º12	26º08	114.5	0.882	0.170	0.128	0.076	0.063
Chania	35º30	24º02	62.0	0.860	0.157	0.164	0.058	0.082
Ierapetra	35º00	25º44	10.0	0.823	0.224	0.134	0.095	0.226
Rhodes	37º24	28º05	34.7	0.707	0.327	0.119	0.171	0.306
Tympaki	35º00	24º46	5.0	0.724	0.342	0.221	0.064	0.311
Milos	36º44	24º26	164.0	0.658	0.236	0.302	0.050	0.450
Naxos	37º06	25°23	9.0	0.632	0.214	0.329	0.105	***0.463***
Kithira	36°08	23°01	165.8	**0.589**	0.422	0.268	0.060	0.415
Kerkira	39º37	19º55	2.0	0.304	0.771	0.176	0.276	0.112
Agrinio	38º37	21º33	24.0	0.251	0.750	0.249	0.276	0.161
Ioannina	39º40	20º51	483.0	0.179	0.747	0.121	0.432	0.158
Andravida	37º55	21º17	11.1	0.331	**0.726**	0.297	0.215	0.306
Argostoli	38°11	20°29	21.2	0.382	**0.699**	0.284	0.254	0.228
Pyrgos	37°40	21°26	12.0	0.303	0.663	**0.348**	0.117	0.334
Tripoli	37°32	22°24	652.4	0.342	0.554	0.357	0.211	***0.416***
Methoni	36°50	21°42	53.0	0.502	0.533	**0.390**	0.048	0.312
Aghialos	39°13	22°48	12.2	0.248	0.197	**0.787**	0.271	0.054
Lamia	38º51	22º24	14.3	0.274	0.246	0.745	0.210	0.167
Larisa	39º39	22º26	71.0	0.100	0.199	0.672	0.481	0.093
Trikala	39º33	21º46	108.6	0.253	0.458	0.645	0.321	0.062
Aliartos	38º23	23º06	110.0	0.466	0.219	0.596	0.143	***0.254***
Elliniko	37°54	23°45	10.0	0.407	0.282	0.467	0.151	***0.436***
Serres	41°05	23°24	34.0	0.055	0.153	0.096	**0.815**	0.253
Kozanh	40°17	21°47	625.0	-0.011	0.198	0.287	**0.740**	-0.137
Mikra	40°31	22°58	4.0	0.082	0.169	0.399	**0.706**	0.182
Florina	40°48	21°25	689.0	0.093	**0.403**	0.371	0.603	-0.022
Kavala	40°56	24°23	60.0	0.121	0.290	0.067	0.619	**0.488**
Mytilini	39°04	26°36	4.2	0.427	0.400	0.137	0.217	**0.619**
Chios	38°21	26°09	4.0	0.426	0.379	0.176	0.175	**0.611**
Alex/lis	40°51	25°26	2.5	0.203	0.418	0.050	0.475	**0.497**

As a first part of this study, an S-mode principal component analysis is applied with orthogonal varimax rotation (Richman, 1986) to monthly rainfall totals for the period 1957-2001 to separate the total area into groups of stations with unique rainfall regimes. Table 1 also shows the 5 principal components with their corresponding rotated factor loadings relative to the initial variables. It is evident that Greece is divided in 5 regions of different rainfall characteristics. The first group consists of areas in southern Greece as Anogeia, Iraklio, Siteia, Chania, Ierapetra, Rhodes, Tympaki, Milos, Naxos, Kithira. The second group includes Kerkira, Agrinio,

Ioannina, Andravida, Argostoli, Pyrgos, Tripoli, Methoni, which comprise the western part of Greece, which is more prone to heavy rainfall. The third group and the fourth group consist of Aghialos, Lamia, Larisa, Trikala, Aliartos, Elliniko in central Greece with moderate rainfall values and Serres, Kozani, Mikra, Kavala, Florina respectively in the north part of the country. The last group is formed by the eastern islands Mytilini, Chios and the coastal Alexandroupolis which present moderate precipitation values. This analysis has slightly different results from the ones obtained by Loukas *et al.* (2001) due to a different period of records (1950 – 1981), and the fact that the set of stations is not common for the two studies.

The standardized precipitation index is calculated (according to McKee *et al.*, 1993; 1995) from monthly precipitation of the period (1957 – 2001), for the mentioned 32 stations, (evenly spread throughout the country) to give a representation of abnormal dryness over Greece. 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. The SPI can track drought on multiple timescales and for the present study SPI is calculated for each month, at 9- month and 12-month timescales, reflecting long-term precipitation patterns. Mean monthly and mean annual values of the SPI12 are estimated. In order to investigate the effect of the length of record for precipitation on the computation of a specific SPI, SPI9 and SPI12 values are calculated for two periods (1974 – 2001) and (1957 – 2001). A regression analysis is performed for each station between the values of SPI9 (or SPI12) for the period (1974-2001), estimated from the short period and the values for the same period (1974-2001) that origin from the longer period of computation (1957-2001). The statistical results of this approach are expressed by the correlation coefficient (R), the index of agreement (I) (which is more sensitive to differences between two variables), the standardized mean bias error (MBE) and root mean square error (RMSE), presented in Table 2. All stations present high values of R and I for both SPI9 and SPI12, showing a very good correlation of the two different period-based calculations of each SPI and in most cases the R, I values are lower for SPI12 than for SPI9 .

Figure 1 presents the two differently calculated time series of the SPI_{12} index values for the stations Argostoli (I = 0.8933) and Methoni (I = 0.9920). As a following step, the probability density functions of the SPI9 and SPI12 values are calculated and are shown in Table 3.

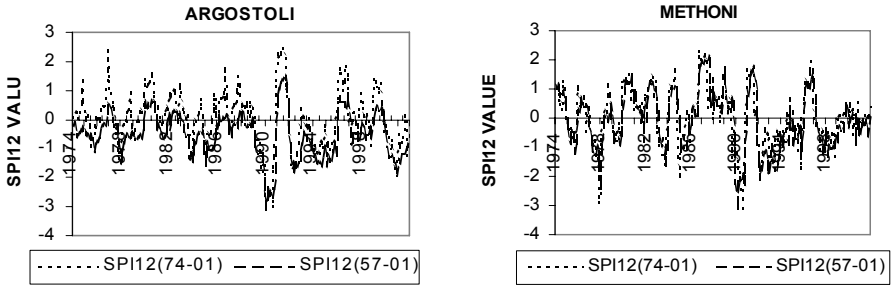


Figure 1. SPI values derived from the long and short length precipitation record over the common period 1974 – 2001 for a 12-month timescale.

Analysis for SPI12 shows that all stations have an occurrence probability of mild, moderate, severe and extreme drought episodes of about 30% - 40%, 10%, 4% and

2% respectively. The major probability for a mild drought to occur, is 40% in Aliartos (referring both to SPI_9 and SPI_{12}), for moderate drought 13% in Kavala (referring to SPI_{12}), for a severe 7% in Chania (referring to SPI_9) and for extreme 4% in Pyrgos (referring to SPI_{12}). The majority of the inland stations suffer statistically more than the coastal ones from mild drought, which is in accordance with the findings of Sonmez *et al.* (2005) for Turkey.

Table 2. Regressions of the SPI values between the two different time periods calculated series for SPI9 and SPI12.

Station	SPI_{12}				SPI_9			
	R	I	MBE	RMSE	R	I	MBE	RMSE
Tympaki	0.9966	0.9946	-0.0828	0.0205	0.9924	0.9932	-0.0821	0.0261
Shteia	0.9833	0.9905	-0.0562	0.0367	0.9849	0.9918	-0.0393	0.0317
Ierap/ra	0.9940	0.9741	-0.2907	0.0953	0.9934	0.9811	-0.2441	0.0718
Iraklio	0.9891	0.9881	-0.1214	0.0399	0.9863	0.9888	-0.1110	0.0385
Anogeia	0.9846	0.9829	-0.1821	0.0619	0.9893	0.9881	-0.1534	0.0429
Rhodes	0.9714	0.9721	-0.1980	0.0936	0.9746	0.9788	-0.1536	0.0735
Chania	0.9965	0.9713	-0.3184	0.1077	0.9957	0.9759	-0.2870	0.0903
Kithira	0.9931	0.9894	-0.1607	0.0383	0.9915	0.9904	-0.1394	0.0348
Milos	0.9750	0.9866	-0.0175	0.0511	0.9823	0.9902	-0.0190	0.0377
Methoni	0.9957	0.9920	-0.1450	0.0311	0.9805	0.9860	-0.1233	0.0545
Naxos	0.9844	0.9890	0.0543	0.0453	0.9831	0.9892	0.0445	0.0447
Elliniko	0.9832	0.9909	-0.0035	0.0342	0.9794	0.9896	0.0006	0.0373
Tripoli	0.9867	0.9787	-0.2238	0.0749	0.9865	0.9826	-0.1917	0.0617
Pyrgos	0.9918	0.9806	0.2345	0.0849	0.9915	0.9828	0.2130	0.0714
Chios	0.9905	0.9732	-0.2829	0.1016	0.9933	0.9795	-0.2591	0.0804
Arg/li	0.9928	0.8933	-0.5543	0.3622	0.9912	0.9119	-0.5141	0.3074
Andr/da	0.9593	0.9609	-0.2504	0.1467	0.9666	0.9714	-0.2023	0.1064
Lamia	0.9922	0.9853	-0.1090	0.0460	0.9921	0.9893	-0.0908	0.0347
Aliartos	0.9926	0.9813	-0.2393	0.0739	0.9870	0.9809	-0.2181	0.0754
Agrinio	0.9930	0.9835	-0.2054	0.0562	0.9931	0.9860	-0.1891	0.0488
Mytilini	0.9734	0.9756	-0.1783	0.1002	0.9762	0.9804	-0.1501	0.0789
Aghialos	0.9924	0.9824	-0.2227	0.0632	0.9865	0.9836	-0.1906	0.0619
Larisa	0.9887	0.9913	0.0342	0.0326	0.9817	0.9891	0.0133	0.0413
Trikala	0.9909	0.9878	-0.1188	0.0403	0.9909	0.9890	-0.1126	0.0374
Ioannina	0.9867	0.9511	-0.3828	0.1736	0.9869	0.9632	-0.3287	0.1324
Kerkira	0.9901	0.9367	-0.4416	0.2288	0.9921	0.9533	-0.3833	0.1682
Kozani	0.9857	0.9278	-0.4198	0.2445	0.9923	0.9443	-0.3801	0.1895
Alex/lis	0.9857	0.9608	-0.3437	0.1435	0.9866	0.9691	-0.2986	0.1132
Kavala	0.9928	0.9777	-0.2385	0.0753	0.9895	0.9805	-0.2132	0.0670
Mikra	0.9734	0.9746	-0.2157	0.0978	0.9864	0.9834	-0.1910	0.0618
Florina	0.9872	0.9404	-0.4188	0.2126	0.9917	0.9531	-0.3876	0.1730
Serres	0.9735	0.9364	-0.3894	0.2186	0.9814	0.9509	-0.3393	0.1654

In Figure 2 one may observe the number of stations for each month of every year between 1957 and 2001 where SPI_{12} depicts severe and extreme drought events. For both SPIs the largest number of stations that presented severe and extreme drought events were mainly observed in 1989 – 1990 and then in 1992 – 1993 and 2000 – 2001 and a satisfying number of stations presented severe and extreme drought characteristics in 1977.The dry period after 1989 is detected also by Livada and Assimakopoulos (2007) and by Tsakiris and Vangelis (2004).

In the second part of the analysis, an S-mode principal component analysis (PCA) is performed over the SPIS values using orthogonal varimax rotation in order to obtain a spatial regionalization and the temporal behaviour of the SPI9 and SPI12

indices referring to hydrological drought. Seven principal components (PCs) are derived from the PCA.

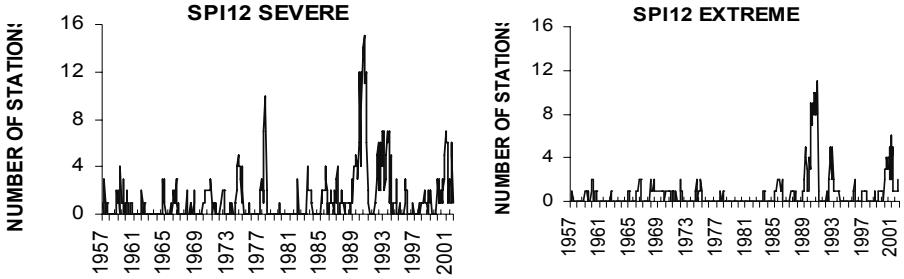


Figure 2. Number of stations that present severe and extreme drought events on 12-month timescales for the period 1957 – 2001.

Table 3. Probability density functions of the SPI9 and SPI12 values for each station.

Variance for Rotated Loadings of SPI_{12}			Variance for Rotated Loadings of SPI_9		
Component	% of Variance	Cumulative %	Component	% of Variance	Cumulative %
1	15.148	15.148	1	14.430	14.430
2	12.341	27.489	2	14.219	28.650
3	12.211	39.700	3	11.784	40.434
4	10.107	49.807	4	10.399	50.833
5	8.104	57.911	5	7.689	58.522
6	7.844	65.754	6	6.866	65.388
7	7.582	73.336	7	6.441	71.830

Table 4 shows the variance results of PCA and it is obvious that the first 7 components explain 71.83% of the variance while the first 3 components explain 51% of the variance. In Figure 3 the spatial distribution of the loadings of 6 (out of 7) principal components for SPI_{12} are plotted in six maps to determine the patterns governing drought occurrence based on $SPI_{12.}$ The first component explains 15.148% of the variance and the highest loadings are at stations situated mainly in the west part of Greece (Kerkira, Ioannina, Agrinio, Andravida, Argostoli), which present heavy rainfall amounts during the rainy period that increase from the coastal parts to the inland eastly, where the southern winds from the Mediterranean meet the chain of Pindus. These sites are influenced also by the depressions directed from west to east where in the mountainous areas larger rainfall amounts are received in the western parts as compared to the eastern ones. According to the corresponding mean monthly SPI_{12} values, these stations suffer greatly of mild drought mainly in winter with the greater SPI_{12} values in Argostoli in December (-0.062), in Ioannina in December (-0.058) and in Kerkira in December (-0.042). Florina is added to this group of stations and this can be explained from its rainfall regime. PCA analysis in rainfall, already gave high PC2 loading value for Florina (Table 1). The second component explains 12.341% of the variance having the highest loadings on the southern part of Greece (Crete, Rhodes, Naxos, Milos). North-coming winds in the east part of the Mediterranean, mainly in the Aegean sea cause dryness especially in the summer period. This region presents not very significant values for mild drought as the worst one appears in Chania in March with an SPI_{12} value –0.037. The third component explaining 12.211% of the variance shows high loading values at stations situated in the northeast part of continental Greece (Alexandroupolis, Kavala) and the islands

nearby the west Turkish coasts (Mytilini, Chios). These areas experience significant mild drought having the greater SPI_{12} value in Alexandroupolis in December (-0.049). The fourth component explains 10.107% of the variance and the highest loadings are concentrated mainly in the central part of the Greek area (Larisa, Trikala, Lamia, Elliniko). Pyrgos is added to this group and this is obvious from its PC3 value in Table 1. These stations experience the worst mild drought with Larisa having the greater SPI_{12} value in December (-0.077). The fifth component explaining 8.104% of the variance show high coefficients in the areas Tripoli and Aliartos in the central Greece presenting very local features and present significant SPI_{12} values with the greater in Aliartos in March (-0.067). The sixth one explaining 7.844% of the variance presents high loadings in areas of north Greece (Serres, Mikra, Kozani) where there are considerable rainfall amounts distributed through all the year. Mild drought is not significant by means of $SPI_{12.}$ The last component (which is not mapped in Figure 3) explains 7.582% of the variance with high loadings in stations Methoni and Kithira in southern Greece, reflecting local features and not showing significant mild drought occurence. This classification may be generally explained by the climate of Greece as proposed by Mariolopoulos (1938).

Table 4. Explained variance and cumulative explained variance, in percentage, from the PCA for spatial regionalization of SPI9 and SPI12.

Station	SPI 9				SPI 12			
	mild	moder	sever	extr	mild	moder	sever	extr
Tympaki	0.3687	0.0926	0.0463	0.0110	0.3798	0.0981	0.0481	0.0147
Shteia	0.3611	0.0759	0.0501	0.0239	0.3130	0.1203	0.0463	0.0202
Ierap/ra	0.3519	0.1056	0.0388	0.0165	0.3945	0.0722	0.0555	0.0165
Iraklio	0.3278	0.1223	0.0461	0.0055	0.3408	0.0926	0.0575	0.0073
Anogeia	0.3242	0.1093	0.0406	0.0147	0.3593	0.0963	0.0388	0.0220
Rhodes	0.3075	0.1018	0.0611	0.0147	0.3019	0.1054	0.0631	0.0073
Chania	0.3630	0.0628	0.0741	0.0110	0.3594	0.0648	0.0575	0.0238
Kithira	0.3574	0.0889	0.0424	0.0220	0.3298	0.1038	0.0333	0.0257
Milos	0.3594	0.0814	0.0443	0.0165	0.3686	0.0870	0.0408	0.0183
Methoni	0.3482	0.0834	0.0332	0.0238	0.3317	0.0888	0.0368	0.0202
Naxos	0.3446	0.0833	0.0313	0.0313	0.3371	0.0852	0.0330	0.0293
Elliniko	0.3278	0.1092	0.0554	0.0055	0.3075	0.1168	0.0368	0.0183
Tripoli	0.3278	0.1019	0.0518	0.0165	0.3148	0.1018	0.0481	0.0203
Pyrgos	0.3650	0.0537	0.0462	0.0331	0.3648	0.0554	0.0351	0.0405
Chios	0.3445	0.0889	0.0537	0.0183	0.3500	0.0963	0.0574	0.0110
Arg/li	0.3593	0.0962	0.0425	0.0183	0.3428	0.1111	0.0312	0.0202
Andr/da	0.3520	0.0928	0.0425	0.0165	0.3408	0.0833	0.0368	0.0220
Lamia	0.3778	0.0703	0.0330	0.0238	0.3613	0.0798	0.0312	0.0238
Aliartos	0.4055	0.0610	0.0295	0.0348	0.4000	0.0667	0.0203	0.0385
Agrinio	0.3390	0.0758	0.0497	0.0275	0.3425	0.0926	0.0330	0.0348
Mytilini	0.3742	0.0760	0.0555	0.0147	0.3742	0.0798	0.0387	0.0312
Aghialos	0.3577	0.0871	0.0350	0.0220	0.3315	0.1058	0.0480	0.0092
Larisa	0.3389	0.0852	0.0294	0.0312	0.3373	0.0833	0.0313	0.0330
Trikala	0.3278	0.1038	0.0535	0.0202	0.3258	0.1186	0.0407	0.0238
Ioannina	0.3426	0.0981	0.0333	0.0296	0.3389	0.1001	0.0368	0.0293
Kerkyra	0.3703	0.0703	0.0535	0.0238	0.3778	0.0815	0.0593	0.0183
Kozani	0.3389	0.1110	0.0221	0.0257	0.3667	0.0946	0.0239	0.0275
Alex/lis	0.3835	0.0742	0.0498	0.0147	0.3519	0.1132	0.0443	0.0165
Mikra	0.3297	0.1057	0.0648	0.0055	0.3259	0.0999	0.0368	0.0202
Florina	0.3908	0.0629	0.0313	0.0350	0.3650	0.0723	0.0276	0.0368
Kavala	0.3351	0.1130	0.0368	0.0110	0.3223	0.1371	0.0295	0.0110
Serres	0.3686	0.0759	0.0388	0.0238	0.3610	0.0870	0.0240	0.0293

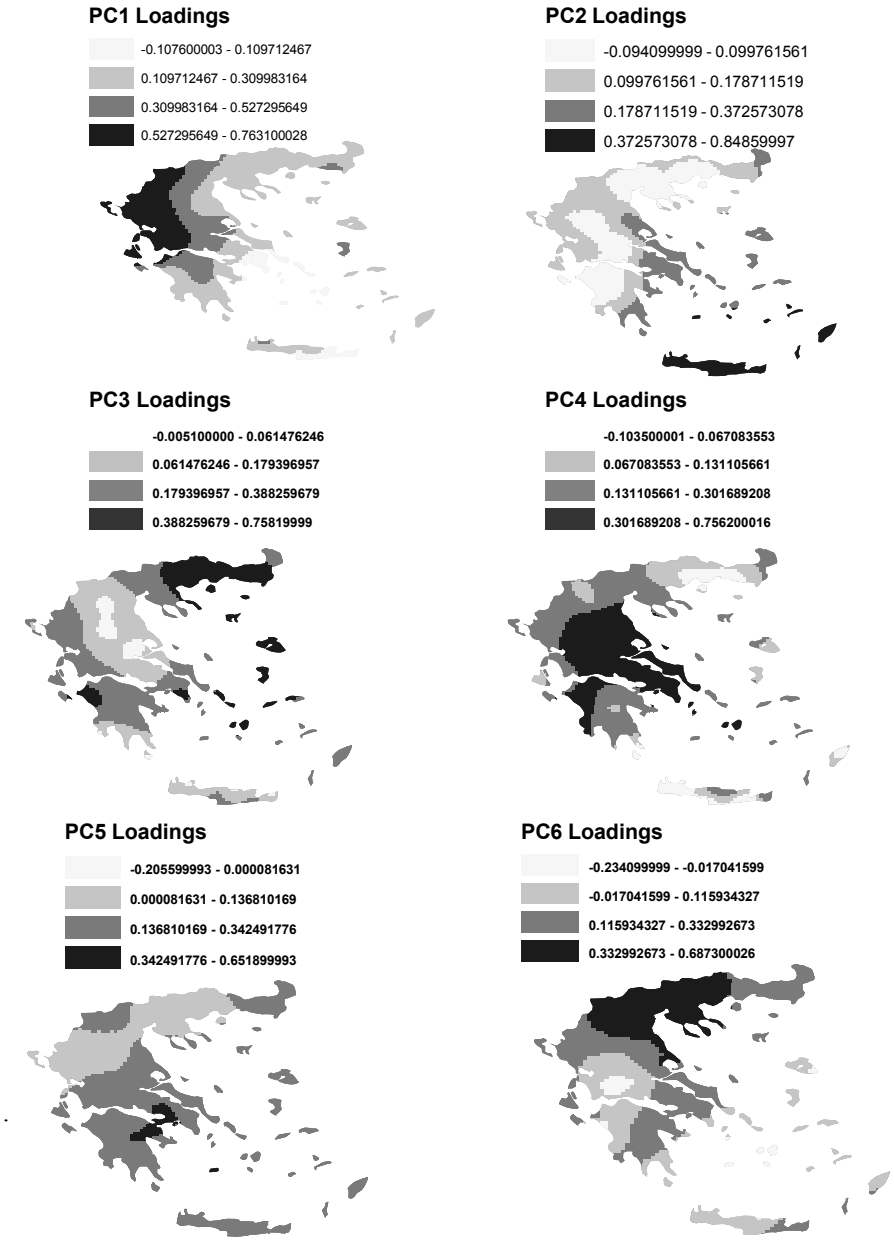


Figure 3. Maps of the distribution of the loadings of the 6 first principal components of SPI_{12} over the Greek area.

Figure 4 presents the 6 first PC scores for SPI_{12} throughout the 45-year period, describing the temporal behaviour of regions as a whole. The scores of PC1 for SPI_{12} that correspond to variations of drought of the stations of west and northwest Greece are more intense and longer during the period 1989-2001 and also intense in the period 1977-1981, showing two dry periods and one more humid period (1957-1976).

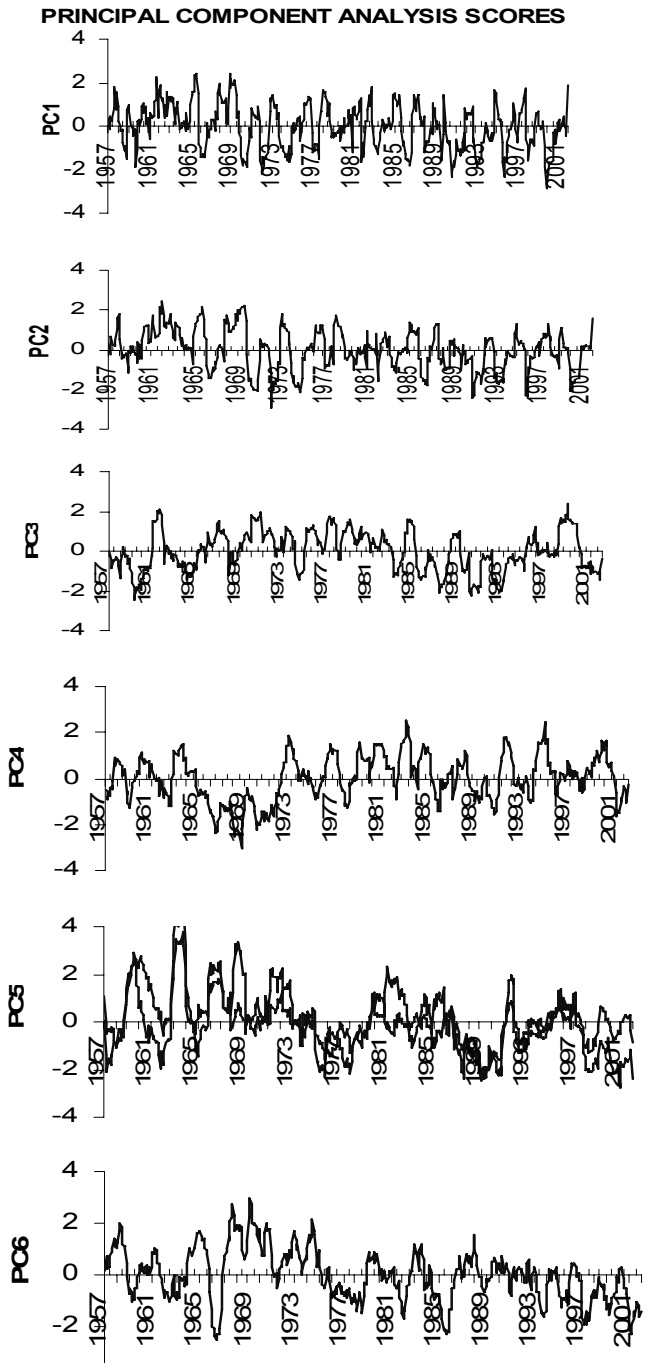


Figure 4. Time series of the 6 first principal component analysis scores for SPI12.

Positive scores reflect non-drought events on a 12-month scale while the negative ones depict drought events. The scores of PC2 show great negative and great positive values throughout the whole period and it is not possible to distinguish a dry or humid period. According to the scores of PC3 that refer to the northeast part of Greece, two dry periods are detected, one between 1985-1995 and another in 1957-1961. Examining the PC4scores, PC5scores, PC6scores and PC7scores it is evident that long dry periods appear during 1965-1973, 1974-1981 and 1987-2001, 1957-1967 and 1973-2001, 1989-2001 respectively.

The final regionalization based on SPI_{12} is achieved when cluster analysis with Ward algorithm is applied to the rotated factor loadings of the new 7 variables that describe the original set, resulting in grouping all stations in Greece in 6 unique areas of different behaviour according to drought. The first, second and third clusters consist of (Kerkira, Agrinio, Ioannina, Andravida, Argostoli, Florina), (Anogeia, Iraklio, Siteia, Chania, Ierapetra, Rhodes, Tympaki, Kithira), (Mytilini, Chios, Alexandroupolis, Kavala), respectively. The fourth, fifth and sixth clusters include (Aghialos, Lamia, Larisa, Trikala, Pyrgos, Methoni), (Tripoli, Aliartos, Elliniko, Naxos, Milos) and (Serres, Kozani, Mikra). This grouping of stations is in a very good agreement with the one obtained only according to rainfall in the first part of the study, which depicts that SPI is a good indicator of rainfall shortage or excess.

3. CONCLUSIONS

The present study shows that spatial regionalization of the Greek territory is successfully achieved by means of principal component analysis whether it is based on the rainfall patterns or in drought occurrence expressed by the SPI indices. The five groups of stations that were distinguished according to monthly precipitation data describe very well the climate in Greece and are quite the same with the more refined seven groups that principal component analysis presents when is performed on monthly SPI values. The final regionalization of Greece in six groups of regions accomplished by cluster analysis on the basis of the obtained SPIs loadings show a good agreement with the rainfall regime over Greece.

The temporal analysis indicates six different appearances of mild drought with the most intense ones in the Thessaly region, western coastal Greece and Ionian islands, northeast Greece and central parts of the country (Aliartos, Tripoli). In all cases the worst droughts appeared during the rainy period. In addition, analysis shows that there is not significant effect on the computation of the SPI_{12} series when the data series is less than 45 years and the results are reliable even in this case, as can be seen from the high indices of agreement and correlation coefficients for each station of the study that are of the order of 0.98. Finally, during the period of interest the more intense and long drought events appear mainly during 1989 – 2001which is in total agreement with previous studies over the Greek territory and the neighbour area of Europe.

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Assessing the Areal Extent of Drought

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Abstract: Drought is a three dimensional natural phenomenon characterised by its severity, duration and areal extent. The study deals with the areal extent of drought proposing two ways of its illustration and assessment: Discretised severity maps and "or more" cumulative curves of the affected area in relation to severity level. Using a reference period (e.g. year) instead of duration and a severity index such as the Standardized Precipitation Index (SPI) or the Reconnaissance Drought Index (RDI), discretised maps show the areas affected by the corresponding severity of drought whereas the "or more" cumulative curves produce directly the percentages of areas belonging to each class of drought severity. Finally an illustrative application of both ways of drought areal extent estimation is presented for Eastern Crete, an area facing frequent droughts.

Keywords: Areal extent, drought indices, "or more" curves, reconnaissance drought index, standardized precipitation index.

1. INTRODUCTION

Drought is a recurring natural phenomenon characterised by its sevirity, duration and areal extent. It is therefore a three-dimensional phenomenon, which is difficult to assess. Significant work has been done over the last decades for devising indices, which represent the severity of drought (Yevjevich, 1967, Yevjevich *et al.*, 1983, Rossi *et al.*, 1992). Although the severity of drought could be rationally assessed from the anticipated consequences, drought indices have been widely used due to their simplicity, which they offer to both scientists and water management professionals.

Some of the widely used drought indices are the PDSI (Palmer, 1965), the Deciles (Kinninmonth *et al.*, 2000), the SPI (McKee *et al.*, 1993) and many others. Comprehensive reviews on these indices may be found in specific papers and publications (Richard and Heim, 2002; Hayes, 2004; Tsakiris *et al.*, 2005, etc.). Recently a new index was proposed by Tsakiris (2004), the RDI – Reconnaissance Drought Index – which uses potential evapotranspiration in conjunction with

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precipitation as the variables, which the drought assessment is based upon. In its standardized form, in most of the cases, the RDI responds in a similar way as the SPI and the various thresholds representing the borders of severity classes are the same (Tsakiris and Vangelis, 2005; Tsakiris *et al.*, 2006).

By fixing the time reference of analysis, the "duration" dimension may be omitted. If for instance the hydrological year is selected as the reference period (that is from October to next September for the Mediterranean countries), drought can be assessed for each year of the historical record based on the selected drought index and its spatial distribution.

2. OVERVIEW OF DROUGHT INDICES

For the purposes of this paper two drought indices were used: the SPI and the RDI. The Standardized Precipitation Index (SPI) was developed for the purpose of defining and monitoring drought based on a single meteorological determinant, the precipitation (McKee *et al.*, 1993).

The SPI calculation for any location is based on a series of accumulated precipitation for a fixed time scale of interest (i.e. 1, 3, 6, 9, 12,… months). Such a series is fitted to a probability distribution, which is then transformed into a normal distribution so that the mean SPI for the location and desired period is zero (Edwards and McKee, 1997). Positive SPI values indicate greater than median precipitation, and negative values indicate less than median precipitation. Because the SPI is normalised, wetter and drier climates can be represented in the same way, and wet periods can also be monitored using the SPI.

The RDI (Reconnaissance Drought Index) is based on the ratio between two aggregated quantities of precipitation and potential evapotranspiration. It appears in three forms: the initial value a_k, the Normalised (RDI_n) and the Standardised (RDI_{st}). For real world applications if RDI is calculated as a general index of meteorological drought it is advisable to use periods of 3, 6, 9 and 12 months. In its initial formulation RDI for a 12 month period could be directly compared with the Aridity Index produced for the area under study. If a_{12} for a certain year is lower than Aridity Index calculated according to UNEP (1992) then the area is suffering from drought during this year. The Standardised RDI (RDI_{st}), behaves similarly to the SPI and therefore the interpretation of the results is similar since the same thresholds as SPI can be used. The equasions for calculating RDI are presented in the Appendix.

3. DROUGHT SEVERITY MAPS AND CUMULATIVE "OR MORE" CURVES

Drought severity maps have been used in many studies during the last decade (Kim *et*

al., 2002; Tsakiris and Vangelis, 2004; Loukas and Vasiliades, 2004). Based on a descretisation procedure, the area under study can be divided into a number of squares, at which the meteorological variables are transferred. Various methods for this transfer may be used (e.g. the weight of each station is taken as a reciprocal of square distance between the centre of the square in question and the location of the meteorological station in operation). Altitude correction should be also introduced if it is nessessary. Then, the selected drought index is calculated at each square for the reference period. Using different colours for each severity class a drought severity map is produced. Drought maps are very important tools for delineating the particular parts of the area under study, which are affected by drought. Although drought maps are useful, they exhibit two major drawbacks:

a. They do not assess drought at the river basin level, at which water resources management is practised.
b. They do not illustrate the areal extent of drought in relation to the threshold value of "the critical area percentage".

In order to correct the first drawback an additional drought map can be produced by a spatial integration, so that drought is expressed at the river basin scale. For this purpose it is advisable to transfer the values of the meteorological determinants to the river basin and particularly at the actual mean elevation of the basin. In case the basin is very large subbasins could be used as territorial units.

As regards the second drawback, this work proposes to use the cumulative "or more" curves (ogives), also known as "drought severity – areal extent" curves, which express directly the percentage of the area under drought, which then can be compared with the critical area percentage. It is customary to compare the areal extent of drought with a preset "critical area" percentage. These curves can be produced by plotting the severity of drought (y-axis) versus the percentage of the affected area (x-axis). The severity of drought is presented by a drought index and the area refers to that affected by at least the corresponding severity level. This type of graphs can be used not only for the characterisation of drought and the determination of its areal extent, but also for comparisons with the critical area percentage (related to severity) directly. Clearly, more than one thresholds referring to the percentage of critical area can be used defining different levels of severity. Since each class of drought severity has a different threshold, it is obvious that various critical area percentages could be simultaneously adopted for characterising a drought episode in relation to its areal extent.

4. APPLICATION

For the application of the proposed methodology, the area of Eastern Crete - Greece (Prefectures of Heraklion and Lassithi) was selected. Both annual SPI and RDI were calculated, for the period of 30 hydrological years. Maps for one wet and one dry year (1964-65 and 1969-70 respectively) appear in Figure 1. As expected SPI and RDI

give comparable results. However discripancies occur due to the fact that RDI is using an additional meteorological determinant (PET) apart from precipitation.

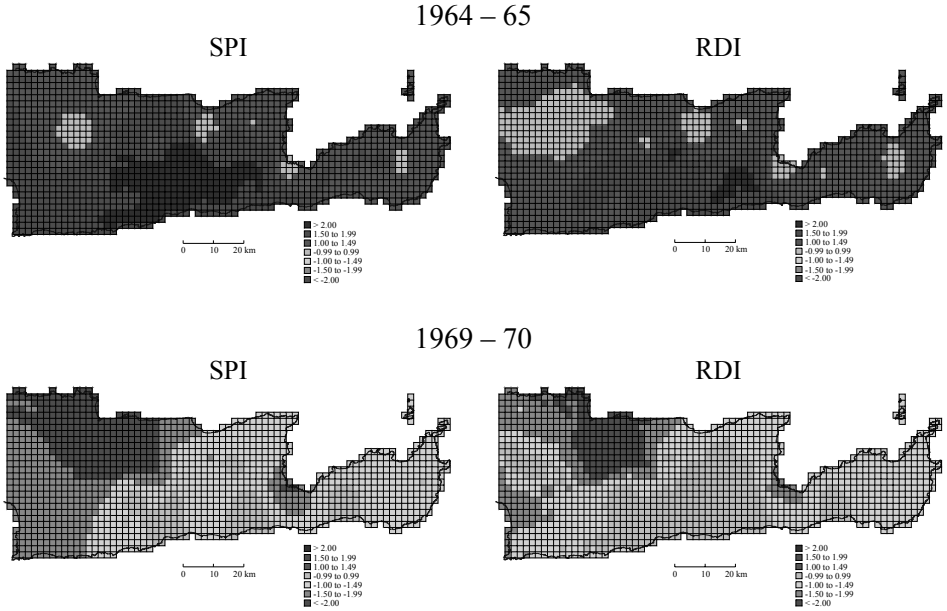


Figure 1. Drought Severity Maps for Eastern Crete for one wet (1964-65) and one dry year (1969-70), based on the severity indices SPI and RDI.

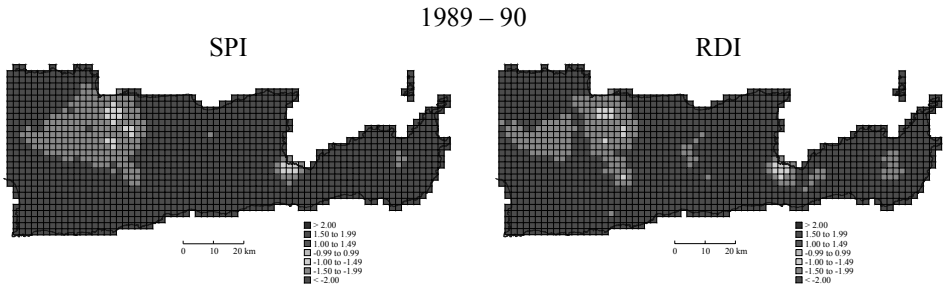


Figure 2. The most dry year (1989-90) during the examined period 1962-63 – 1991-92 for Eastern Crete based on the SPI and the RDI.

Further in Figure 2 the most dry year (1989-90) during the examined period from 1962-63 to 1991-92 is presented. In Fig.3, the cumulative "or more" curves for both SPI and RDI are presented for the wet year 1964-65 and the dry year 1969-70. The boundaries of the drought severity classes [0,-1] normal to dry, [-1, -1.5] moderate dry, [-1.5, -2] severe dry and [<-2] extreme dry appear also in Figure 3. Focussing on

the dry year 1969-70 it may be observed that various percentages of "critical area" may be adopted and may be directly compared with the areal extent of each drought level. If the RDI is considered it can be seen that about 10% of the total area is under extreme drought, whereas 25% of the area is under extreme or severe drought and 70% is is under at least moderate drought. In other words, 10% of the area is under extreme drought, 15% under severe drought and 50% of the area is under moderate drought.

Finally in Figure 4 the cumulative "or more" curves for the most dry year (1989-90) during the period examined are presented. From Figure 4 it can be concluded that more than 80% of the area is under extreme drought.

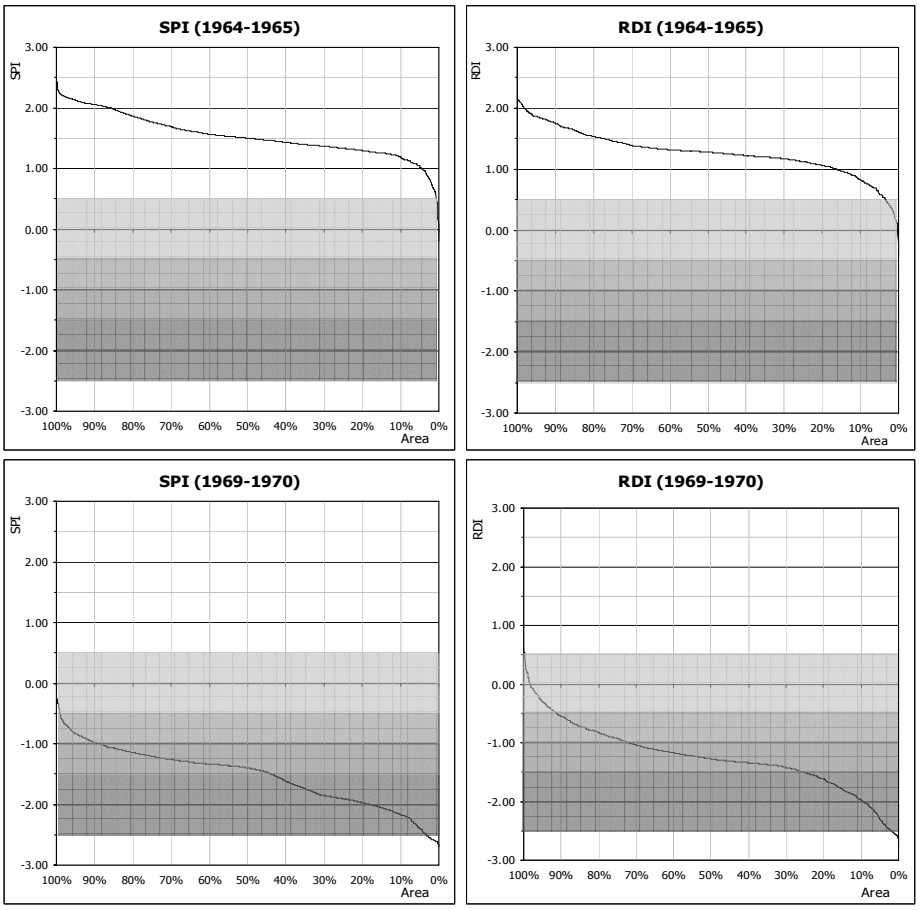


Figure 3. The cumulative "or more" curves for both SPI and RDI for the wet year 1964-65 and the dry year 1969-70.

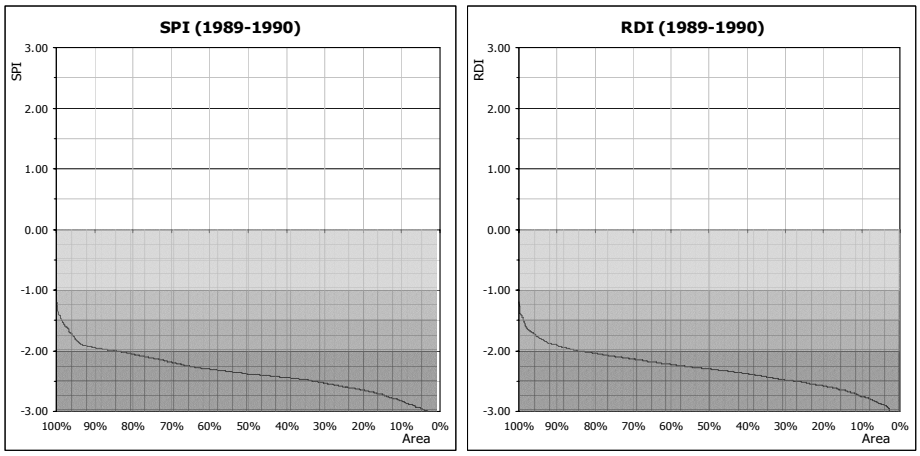


Figure 4. The most dry year 1989-90 during the period 1962-63 – 1991-92.

5. CONCLUDING REMARKS

The proposed methodology showed that Drought Discretised Maps and Cumulative "or more" curves can be presented to illustrate the severity of drought and its areal extent. In particular, the cumulative "or more" curves describe the areal extent of drought in direct percentage figures, which may be compared with the adopted critical area percentage for each class of drought severity. Based on this information, a comprehensive analysis of past droughts can be assessed and sound preparedness plans can be devised for facing future droughts.

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APPENDIX - THE CALCULATION OF RDI

The new index called “Reconnaissance Drought Index – 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 the end month (k) during a year, is calculated by the following equation:

$$a_k = \frac{\sum_{j=1}^{j=k} P_j}{\sum_{j=1}^{j=k} PET_j} \tag{A1}$$

in which P_j and PET_j are the precipitation and potential evapotranspiration of the j-th month of the hydrological year. The hydrological year for the Mediterranean region starts in October, hence for October k=1 and for the following September k=12.

Equation A1 may be calculated for any period of the year. It can be also written starting from any month of the year different from October if it is necessary.

The Normalised RDI (RDI_n), is computed using the following equation:

$$RDI_n(k) = \frac{a_k}{\overline{a}_k} - 1 \tag{A2}$$

Finally the Standardised RDI (RDI_{st}) is computed following a similar procedure to the one that is used for the calculation of SPI:

$$RDI_{st}(k) = \frac{y_k - \overline{y}_k}{\hat{\sigma}_k} \tag{A3}$$

in which y_k is the $\ln a_k$, $\overline{y}_k$ is its arithmetic mean and $\hat{\sigma}_k$ is its standard deviation.

It should be mentioned that standardisation is achieved by assuming that a_k follows a lognormal distribution.

For the standardised RDI (RDI_{st}), the severity thresholds of SPI can be used.

Detection of Water Leaks in Supply Pipes Using Ground Penetrating Radar Technologies

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Abstract: This paper shows the ability to use a Continuous Wave (CW) Doppler Radar, operating at 2.45 GHz, as an instrument to detect water leaks. The operation principle is the detection of the Doppler frequency shift of the electromagnetic wave when it is reflected by slightly moving water that leaks out of the pipe. The comparison between the prototype radar system and prevailing commercial methods proves the efficiency of the proposed system. These promising results of this radar trigger off the authors to enhance the detection capabilities and provide an integrated system by using an UWB radar for detecting additionally the extent of the water leaks and a Decision Support System to point out the need to repair the pipelines.

Keywords: Continuous Wave (CW) Radar, doppler shift, ground-penetrating radar (GPR), Ultra Wideband (UWB).

1. INTRODUCTION

Accurate and timely detection of leaks in water supply pipes is a significant environmental issue. Development of efficient non-invasive methods can lead into significant water saving and prevention of health hazards introduced by water leakage. Implementing electromagnetic detection techniques is an innovative approach in locating leaks in water supply pipes. The prevailing methods for detecting water leaks are the followings: a) acoustic leak sounding system, b) multi-point correlation systems and c) ground microphones (Rott, 2000). The main drawback of the mentioned methods is their inefficiency in detecting water-leaks at non-metallic pipes (Hunaidi *et al.,* 2000).

Thus, to overcome this drawback, and in the framework of the European Union funded "LEAKING" (GROWTH PROGRAMME GRD1-2001-40284) project, passive (radiometric) and active ground penetrating radar technologies are developed to improve the detection capability of leaks in water supply pipes. Two specific active techniques using wide-band frequency modulation-continuous wave (FM-CW) and Continuous Wave (CW) are utilized. The innovative CW radar is operated at the high

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frequency 2.45 GHz, compared to the most common GPR techniques for leak detection, which have been presented so far (Hunaidi *et al.*, 1998, Haemi *et al.*, 2000). The three independent modalities being supplementary provide better detection performance in diverse environmental conditions (high detection probability and lower false alarm rates), and their sensors are able to operate simultaneously.

In Figure 1 a block diagram of the proposed leakage detection system is presented and the ground E/M model. This paper analyzes the ability of the CW radar to detect leaks. It also discusses the possibilities of using UWB GPR for the same purpose.

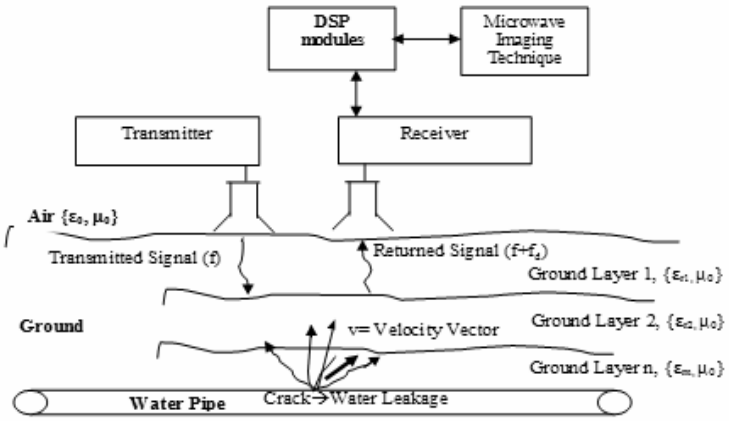


Figure 1. Schematic diagram of the proposed leakage detection system and ground E/M model.

A prototype version of the radar system has already been constructed and tested both in test site and real conditions. Its operation principle, the system design and the characteristics of the RF parts of the radar system and some indicative test results of the measurements carried out using the CW radar in a test site are presented in Sections 2, 3 and 4 respectively. At Section 5 of this paper, the current development of a new Ground Penetrating Imaging Radar is presented, along with a Decision Support System (DSS), in order to provide an integrated solution for Water Utility Companies. Finally some conclusions and future research work plans are given.

2. OPERATION PRINCIPLE

The proposed method is based on the principle of measuring echo signals and the frequency shift of returning signals from a material media, when there is a movement within the three dimensional space to be observed. When an electromagnetic wave of frequency fo hits a moving object, in this case the water leaking out of the pipe, the frequency is shifted according to the Doppler phenomenon. The Doppler frequency, f_d, is proportional to the relative radial velocity, v_r, of the object with respect to the transmitter following the law (Skolnik, 1980):

$$f_d = 2 \cdot \frac{v_r}{u} \cdot f_o \tag{1}$$

where u denotes the speed of electromagnetic waves inside the earth medium. The signals scattered back after reflection from the leaking water are:

$$f_o + f_d = f_o + \frac{2v \cdot \hat{t}}{u} f_o \tag{2}$$

The returned signal at the receiver has frequency f_0+f_d, and the down converted signal at the baseband part of the receiver has the following form:

$$s(t) = \alpha \cdot \cos\left(f_d \cdot t\right) \tag{3}$$

Assuming that the value of the operational frequency equals to 2.45 GHz, the speed of electromagnetic waves inside the earth medium is approximately $\upsilon \cong 1.5 \times 10^8$ m/s and the mean value of leaking water velocity is less than 0.01 m/s, the maximum value of the shift frequency f_d due to Doppler effect, as resulted by the first equation, is 0.3 Hz. Thus, the signal described from the above equation can be easily detected and processed by implementing Digital Signal Processing techniques (Balanis, 1989).

3. SYSTEM DESCRIPTION

The above short analysis indicates that a CW radar is suitable for utilization as detector for leakages. The operational frequency of the radar system was chosen to be 2.45 GHz, since this frequency combines a relatively good penetration depth with a high spatial resolution (Lymperopoulos *et al.,* 2003; 2004a). The block diagram of the system is given in Figure 2.

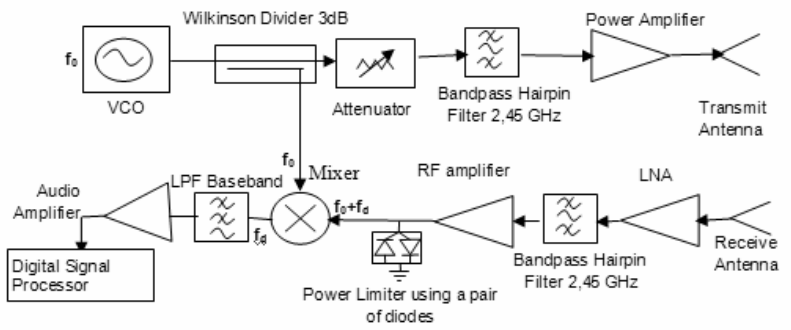


Figure 2. Block diagram of the proposed CW radar.

A CW signal, generated by a Voltage Controlled Oscillator (VCO), is filtered, amplified and transmitted by an antenna. The same VCO is used as local oscillator in the receiver. An independent transmit/receive prototype patch antenna approach is employed to achieve the necessary isolation between transmitter and receiver of the

radar. This antenna has an optimized radiation pattern and a satisfactory gain and efficiency (Balanis, 1997; Bimpas *et al.,* 2005).

On the receiver part, a Low Noise Amplifier (LNA) following a microwave bandpass filter and another amplifier brings the signal to a power level that can be processed by the mixer. The digital processing enhances the performance of the filters. The 2.45 GHz CW radar sensor main characteristics are the following:

- Power Output: 2.5 W
- Power Consumption < 5 W
- System Noise Figure < 2.1 dB
- Receiver Sensitivity: -100 dBm
- Processing: A/D Conversion at 75Hz sampling rate, 4096-point FFT

4. TEST RESULTS

4.1. Measurement at a Test Site in United Kingdom

The test results presented below have been carried out in an experimental site in Wales in United Kingdom, and several features of leak detection were evaluated. All the constructed systems (radiometer, FM-CW and CW Doppler radar systems) were tested in this site, but only the results of CW are presented in this paper (Lymperopoulos *et al.,* 2004b).

Previous experience has shown that the spectrum of this waveform has a fundamental component reaching no more than 0.3 Hz for the case of water leaks, and the signal has a sinusoidal form in time domain (Bimpas, 2004). These facts are exploited to develop a detection criterion.

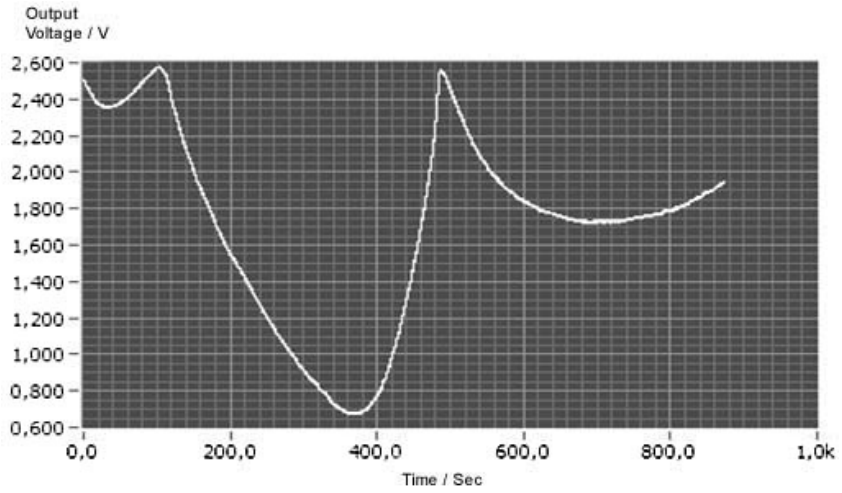


Figure 3. CW radar measurements on leak point: 2V p-p signal in time domain.

In the graph of Figure 3 a screenshot of a measurement taken on the leak point is presented. A relatively powerful (approximately 2V p-p) sinusoidal signal waveform in time domain is indicate due to the existence of a water leak in the specific test

point. . On the other hand, in the graph of figure 4, there is a screenshot representing the non-leakage case. It is obvious that the signal is very weak (approximately 0.1V p-p) and noisy in time domain. To conclude, at the second non leakage case, the output of the radar system is not as "smooth" as in the waterleak case, the amplitude of the waveform is much lower and there are quite a lot components composed in the sinusoidal waveform.

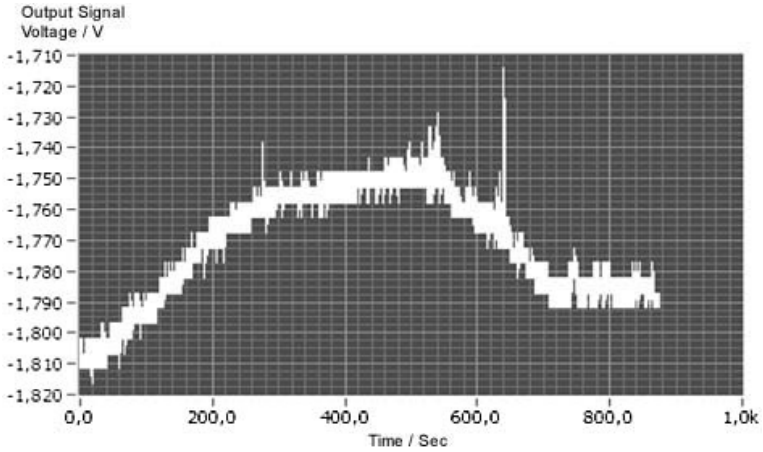


Figure 4. CW radar measurements away from leak: 0.1V p-p signal amplitude.

4.2. Comparative Measurements in an Urban City of Germany

In this section, the results of some indicative measurements, which took place mainly in Chemnitz (Germany), are presented. A multi-point correlator was compared to the CW radar system. SEWERIN was at head throughout the whole measurement procedure.

In Table 1, the main characteristics of fifteen different leakages in real conditions, along with the results of the measurements using the correlator and the CW radar system, are given. In the second and the third column of this table, the pipeline material and the type of soil (underground propagating media) are given respectively. The measured distance (in meters) from a reference point resulted from the tested equipment (correlator and CW radar system) are presented in the fourth and fifth column of Table 1. The measurements given with the bold characters indicate the results, which were considered satisfactory from the SEWERIN system operators. The dash where appear in some of the cells of these two columns means that the specific equipment did not show any indication for water leakage. Finally the sixth column shows the actual distance from the reference point in which the leakage was found when the excavation took place by SEWERIN. In all cases, the water pipelines were buried in the ground at about 1.1 meters up to 1.6 meters and the leakage size varied from 1 m/h up to 4 m^3/h.

Judging from these results, both the equipment had an excellent performance in ten (10) cases out of the total fifteen (15) measurements. This percentage may be considered very satisfactory, especially for an experimental equipment such is the proposed CW radar system.

As far as the water leakages are concerned in cases of PVC pipes (where the commonly used acoustic methods encounter difficulties), the superiority of CW radar was obvious considering not only the accuracy in measurements, but also the fact that its indications were unambiguous

Table 1. Comparative measurements between correlator and the proposed CW radar system, in real conditions of water leakages.

No	Material	Soil	Correlator (m)	CW (m)	Actual (m)
1	cast iron	clay	9.61	10	No leak
2	steel	clay, wet	**13.79**	-	14
3	cast iron	clay, wet	**10.96**	**10.8**	10.9
4	steel	sand, wet	**21.25**	20	21.4
5	steel	clay, wet	**31.67**	**32**	32.2
6	cast iron	stone, wet	**3.34**	1.8	3.4
7	steel	clay, wet	-	**0**	0
8	PVC	clay, wet	**2.79**	**2.9**	2.9
9	PVC	stone, wet	**2.02**	**2.40**	2.0
10	PVC	sand, wet	-	**17**	17.1
11	PVC	sand, dry	0	-	2.0
12	cast iron	clay, wet	18.42	**17.4**	17.4
13	cast iron	clay	**27.72**	23.4	27.4
14	cast iron	sand, dry	**3.56**	**3.8**	3.6
15	PVC	clay, dry	**3.62**	3.7	3,6

5. ENHANCEMENT OF THE ABOVE SYSTEM

The cost of replacing 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. Based on the promising results presented in the previous section, the authors decided to enhance and expand the CW radar in order to implement an integrated system that will offer apart from better efficiency and accuracy in water leak detection, additional information about the need to repair the pipeline under investigation.

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. The main objectives set by the authors are:

1. To develop a novel, high resolution ground penetrating imaging radar (GPIR) for the detection of pipes, leaks and damages and the imaging of the damaged region and evaluate it at a test site. The main characteristics of this radar is an Ultra Wide Bandwidth pulse shape signals with zero dc component, the bandwidth region is 500-1000 MHz and the duration of the pulses will be of the order of 2-

3ns. The Penetration Depth for Imaging Capability will be less than 2m for any natural ground environment, the spatial resolution will be approximately 5cm and the system will be Real Time (RT) operated.

2. To produce an integrated system that will contain the equipment in '1' 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.
3. To field test the equipment and the DSS.

In the Figure 5, the integrated proposed system is presented with the main features under development. This work is currently being performed under the EC funded project WATERPIPE (6th FW) and the first results will be presented in a follow up paper.

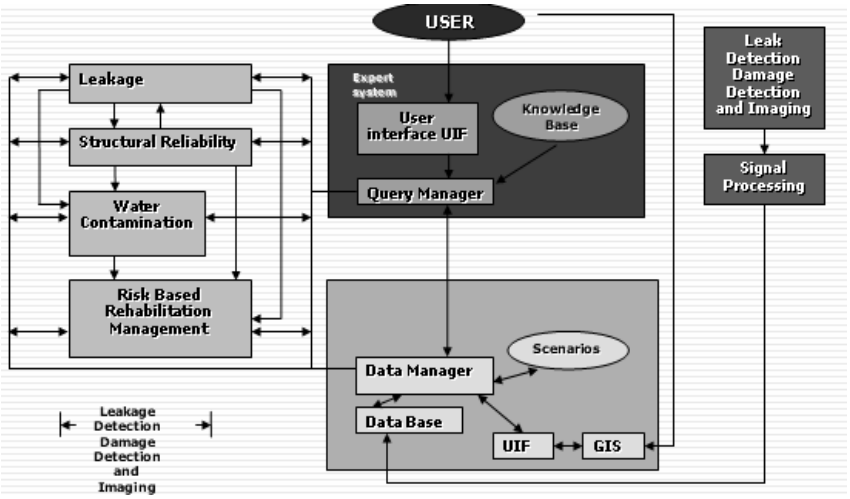


Figure 5. Integrated system for the rehabilitation management of the trunk and distribution water mains.

6. CONCLUSION

The use of a CW Doppler radar sensor and advanced signal processing techniques allows the detection of extremely weak signals. The measurements have shown the efficiency of the CW radar in detecting the very small Doppler shift of the moving water. Additionally it was made clear that the systems' performance would greatly benefit from the utilization of any advanced signal processing technique. This procedure is under progress and the first results will be presented in a follow up paper. The development of an UWB radar is also under development and both systems will be compared, in order to find out the most efficient system in detecting the water leaks. The latter system will also be capable of performing imaging of the

underground and the pipeline as well. Finally, the use of an efficient Decision Support System would make 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.

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Managing the "Repair or Replace" Dilemma on Water Leakages

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Abstract: In light of the increasing and pressing need to efficiently manage scarce water resources there has been renewed interest by water distribution network owners to develop and implement water management strategies and tools that would assist in the integrated and automated management of those networks. Such asset management strategies should assist the network owners evaluate the condition of the water distribution network, assess historical incident data (leakage or breakage) and risk of failure, visualize areas of high risk, propose "repair or replace" strategies and prioritize the work based on the inherent risk and cost of action. The work presented outlines a risk assessment and management ("repair or replace") system based on analytical and numerical modeling techniques and supplemented with geographical distribution systems (GIS). The goal is to enable water utilities to better manage condition assessment information, process historical records with a number of analytical and numerical models, identify underlying data patterns with artificial intelligence techniques and eventually assess the corresponding risk of failure of each network element and visually disseminate this information via geographical information systems. This report is based partially upon work supported by the National Science Foundation (under Grant No. CMS-0118376) and upon work supported by the Cyprus Research Foundation (under Grant No. AKGEN/1204/01).

Keywords: Risk assessment, water leakage, pipeline asset management, GIS.

1. INTRODUCTION

As the ongoing globalization and urbanization has increased in pace and magnitude, urban utility owners are faced with increasing demands on resource usage and reliability of provided services that, in turn, put a highest strain on their resources, their infrastructure and the level of service to their customers. This sudden increase in

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demand for scarce resources and reliable services is even more pressing in developing countries and countries with limited resources.

In the case of urban water distribution networks (especially 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, due to the lack of alternatives and the pressing need to provide these resources to the public. After all, in the case of water distribution networks the resource managed and provided to customers (water) is the essential element for life and no substitute can fulfil this resource's role. Furthermore, in the case of most developing countries the managed distribution networks are based on ageing and neglected infrastructure that is highly unreliable and cost-inefficient. 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, 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 is one of the most important dilemmas facing water distribution -organisations. Should an organisation repair or replace ageing and/or deteriorating water mains and what, in any case, should the sequence of any such repairs be as part of a long-term network rehabilitation strategy?

The work presented outlines an integrated methodology and a decision support system for arriving at such "rehabilitate-or-replace" decisions, as part of a long-term pipeline asset management program that could be undertaken by a water utility to improve on the reliability of the water distribution networks. The International Water Association's Water Loss Task Force has been advocating and promoting four basic leakage management activities for leakage reduction, namely: pressure management, active leakage control, speed and quality of repairs and pipeline asset management, maintenance and renewal (Lambert and Mckenzie, 2002). The methodology and support system outlined in this paper can form an integral part of a leakage management strategy and provide a useful decision making tool.

2. STATE OF KNOWLEDGE IN LEAKAGE MANAGEMENT

To-date a number of studies have been undertaken on infrastructure assessment and deterioration modelling, with the intent to assist owners of such systems improve their understanding of a system's behaviour over time, its deterioration rate and its reliability with respect to presumed risk factors. The intent has always been to assist owners and operators of water distribution networks in arriving at "repair-or-replace" decisions on a more scientific basis. 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 (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 (1987) to predict the probability of subsequent breaks given that at least one break had already occurred. Finally, Kleiner *et al.* (1998; 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 (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 (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.* (2006) in a developed framework for integrated GIS-based management, risk assessment and prioritization of water leakage actions.

3. INTEGRATED WATER LEAKAGE MANAGEMENT SYSTEM

3.1. The Distribution Network

The water distribution network currently under review (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 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 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 (Charalambous, 2005). 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.

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 so as to be able to account efficiently and accurately for all water produced (Figure 1) or "lost" (non-revenue water). Reduction and control of water loss was achieved through the application of a holistic strategy based on the approach developed by the Water Loss Task Force of the International Water Association. Integral part of this approach is the establishment of a strategy for pipe break incidents ("pipe breaks policy"). The policy further envisions the prioritization of the repair/replacement actions on the basis of risk of failure, life-cycle costing, social and financial impacts.

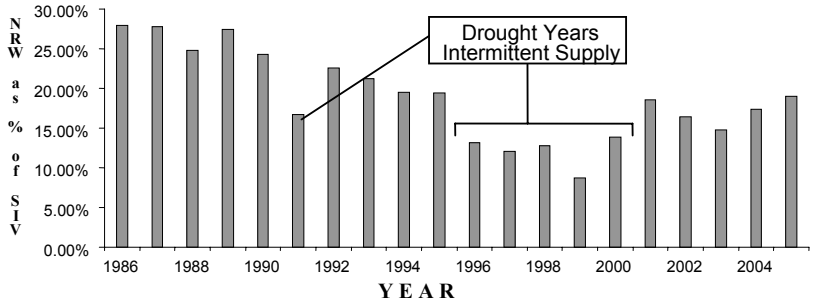


Figure 1. Non-revenue water.

In addition to the reported bursts, the Water Board of Lemesos through its strategy for active leakage control maintains records of unreported bursts located by means of acoustic leak localizers. Typical results are shown in Table 1 (Charalambous, 2002). Details of all such breaks are maintained and will be included together with the reported bursts in the analysis.

Table 1. Unreported bursts – identified, located and repaired.

Type of pipe	Number of bursts		Percentage per type	
	1999	2002	1999	2002
20mm MDPE	7	7	39%	44%
32mm MDPE	9	8		
100mm AC mains	10	10		
150mm AC mains	12	9	61%	56%
200mm AC mains	3	0		
TOTAL	41	34	100%	100%
Estimated water saved	140,000 m³	110,000 m³		
Worth of water saved	US$ 45,000	US$ 35,000		

3.2. The Proposed Integrated Pipeline Asset Management System

3.2.1. System's framework

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 analytical and numerical analysis tools and supplemented with a geographical information system (GIS) with the aim of delivering to water distribution network owners and administrators with a complete decision support system (DSS) that can improve on the management of such water distribution networks.

Starting with the premise that historical data on pipe break incidents, reaction methodologies, social and financial impacts and life cycle costing are important ingredients in the puzzle of "repair or replace" strategies and action prioritization, the proposed system envisions the integration of all these key elements in the delivery of one integrated management system which will help utilities manage their distribution networks more efficiently. As such, the proposed system encompasses:

- 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.

3.2.2. System's modus operandi

The database structure (snapshot shown in Figure 2) is the heart of the DSS, comprising the knowledge repository of the distribution network over time, as each data event is date/time-stamped to provide a chronology of events related to the risk of failure of each component. The databases are relational and integrated to minimize entry points by the users, standardize input, minimize risk of errors in data handling, and maximize automation of data analysis and reporting.

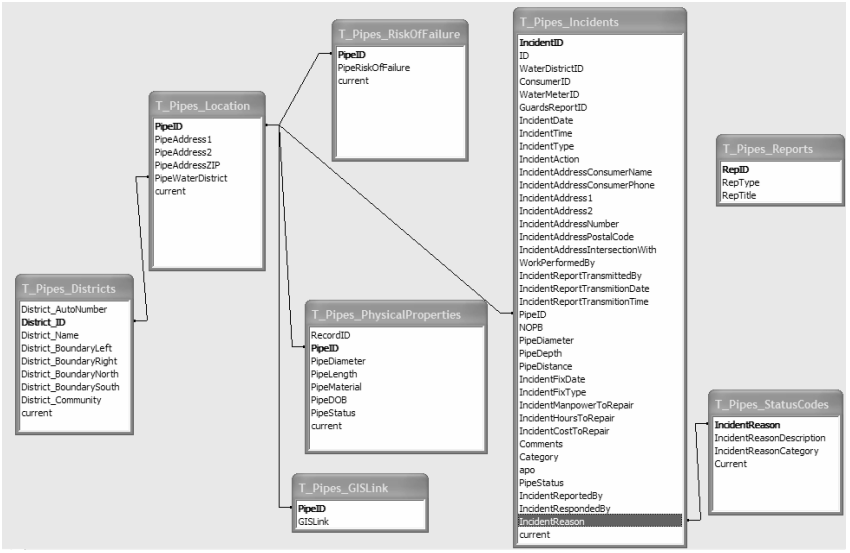


Figure 2. Implemented leakage database management system.

The data is processed by means of a combination of decision support tools, such as survival analysis, statistical analysis, artificial neural networks and fuzzy logic. This analysis aims at the identification of possible risk factors for pipe breaks and rank according to their severity and causal effect. Christodoulou *et al.* (2006) and Deliyanni (2006) reported on the severity of a number of presumed factors (such as pipe diameter, material, length, age, number of previously observed breaks, etc.) and proceeded in listing the factors by means of a neurofuzzy system. A subset of the derived fuzzy rules is tabulated in Table 3.

A significant output of the described management system is a ranking of the system's components (pipes) in terms of risk of failure (Figure 3), so that users can assess the probability of failure and, in conjunction with associated costs, decide what the proper management strategy should be (replace or repair, and corresponding time horizon). This computation is achieved by means of a combination of statistical analysis and survival analysis techniques (Vanrenterghem, 2003; Aslani, 2003; Christodoulou *et al.*, 2003), and can be visually represented by means of a customized GIS-based software that enables visual representation of the risk of leakage and quick assessment of the probability of failure and the corresponding action strategy (repair

or replace). Degrees of the risk of failure are colour-coded and continually updated as time-related data is processed by the system (sample shown in Figure 3).

Table 3. Fuzzy logic rules describing expected behaviour of water pipes.

If …					Then …
Diameter	Number of	Length	Material	Traffic	Break (1)
Small	Small	Small	2	1	0
Small	Small	Small	1	1	1
Large	Large	Small	4	2	1
Medium	Small	Small	4	2	0
Small	Small	Medium	-	1	0
Small	Small	Large	1	0	0
Medium	Medium	Small	4	2	1
Large	Large	-	2	-	1
-	Small	-	1	-	0
S: 4 – 30"	*S: 0 – 2*	*S: 0.25 – 5.50*			
M: 20 – 48"	*M: 1 – 4*	*M: 4.50 – 14.0*			
L: 40 – 72"	*L: 3 – 9*	*L: 10.0 – 21.0*			



Figure 3. GIS-based leakage management system for water distribution networks.

4. CONCLUSIONS

The paper reports on the development of an integrated GIS-based decision support system for assessing risk of failure and managing water distribution networks in urban environments. The work, which is still under development, is now piloted for implementation in two cities in Cyprus. The water utilities involved aspire, through this implementation, to reduce water losses in their water distribution networks and to improve on the reliability of their systems. The underlying knowledgebase and integrated decision support tools (statistics, ANN, fuzzy logic, GIS) aim to support

these utilities in their endeavours and benefit their consumers the most.

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New Leakage Detection Techniques Using Various Performance Indicators

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Abstract: City of Calgary - Water Services instigated a leakage study of certain parts of the city to estimate the time period to compete a survey of Calgary. This paper explains the technique used and the findings of the study. The paper shows that regardless of the level of water losses reported, it is still possible to perform Active Leakage Control (ALC) and develop new techniques to increase water loss savings. Some of the findings from this study are as follows: There is a correlation between an acoustic numeric value was recorded on the ground microphone and the size/type of water leak; the size and type of leak can be estimated by the by the analysis of the data sets and from confirmed leaks. The propagation of sound values (using the AS3) on the non-metallic material mains and infrastructure appear to much less to that of metallic, therefore without the addition of supplementary data sets taken from confirmed leak types on this material type the methodology established cannot be supported. This technique has proved successful and more investigation is required to obtain a generic footprint for non metallic pipe-work. This shows that some form of ALC can be performed with success from an un-skilled operator with minimum training.

Keywords: Acoustic, ALC, CARL, ILI, UARL.

1. INTRODUCTION

Hydrosave was commissioned by City of Calgary - Water Services to assist with the development of a bespoke leakage detection methodology anticipated to complement the existing detection resources with the addition of a robust procedure, which if proven successful could be utilised by an unskilled operative in order to successfully identify potential areas of loss from the below ground infrastructure.

These potential 'areas of interest' (AOI) could then be recorded and passed to a skilled operator for further investigation and the confirmation of leak position carried out with the assistance of specialist acoustic equipment.

The real losses currently being estimated from its below ground assets in Calgary are recorded as a percentage of volumetric input, this is currently estimated as being

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some 12 per cent of the total water into supply. No calculation of the ILI has been completed for the city as a whole but has been for smaller temporary DMA's, and these results are provided later in this paper.

Although 'percentage of volume input' is the method currently being used by City of Calgary - Water Services to measure the real losses, it is important to note that the authors do not support the use of the 'percentage of volume input' as the sole PI for real losses but rather as one of several PIs that should be used collectively. The City of Calgary – Water Services also does not agree with "percentage of water input" being the sole PI for water loss and will be undergoing a full IWA water audit in late 2007 and 2008. The following PI's are suggested as being more reliable PI:

- litres/service connection day or l/km/day (if service connections density is > 20)
- ILI
- % of losses (If % of losses is used in any circumstances it should be understood it is the least reliable and the IWA WLTF advises that should not be used as an indicator for Real Losses but if anything at all only as a comparison of the performance of the system from previous years providing that the operational and distribution systems remain unchanged)

1.1. Survey Scope

The survey scope was to develop a robust methodology utilising an acoustic listening device based upon a leak noise value represented by a numeric reading and where possible obtain an acoustic "footprint" for the supply area. The equipment of choice chosen for this purpose was the Aqua Scope AS3 manufactured and supplied by Gutermann Limited the reason being it displays a numerical reading for the acoustic noise heard. The data captured during the process was analysed to identify the correlation between the numeric value and material type and, where possible, substantiated with recorded values taken from known leak positions. Survey times were also monitored and measured so that an approximate estimate of manpower, projected timescales and geographical coverage could be ascertained.

The survey methodology itself and subsequent data collection and assessment were aligned with the International Water Association's (IWA) & American Water Works Association (AWWA) principles and practices

1.2. Survey Areas

The water supply network coverage within the chosen areas for the development of the methodology is approximately 120 square kilometres. These areas are sub-divided into city districts that are represented by a variety of topographic and demographic property types. Each district incorporates about 30 square kilometres of water supply network and integrates a proportionate mix of both material and fitment types. The network is generally constructed on a generic grid-based design with a singular strategic supplying main. In some instances the areas are independently fitted with localised pressure management facilities.

Four districts in total were chosen which concentrated principally upon the following residential areas: Lake View, Glenmore, Bowness and Spy Hill.
The real losses for these survey areas were unknown as flow monitoring by area is currently limited, however flow data was gathered 12 months prior to the survey.

Fitments, pipe-work and service valves within these areas are mostly accessible and in some instances situated within unmade access routes sited toward the rear of properties. Above ground fire hydrants are generally positioned within the carriageway and are clearly visible with servicing valves installed controlling the fire hydrants.

1.3. Analysing the DMA's

One of the main goals of establishing a DMA is to answer the first question of water management, "Where is the water coming from?" and, "Where is it going to?" Once the DMA is isolated and the flow meter(s) installed, the first question can be answered. The second question is answered by monitoring the incoming flow and completing the IWA water balance.

Snapshot ILI Bowness
CARL/UARL=ILI
50/12=4.1
ILI=4.1
Snapshot ILI Lakeview
CARL/UARL=ILI
10/1.5=6.6
ILI=6.6

This was considered to small an area to have a valid ILI calculation performed.

2. EQUIPMENT

The acoustic equipment utilised during the survey (Gutermann AS3) was selected by City of Calgary - Water Services Company and the methodology was to be developed using this brand of equipment.

2.1. Methodology – General Principal

The primary survey methodology was to incorporate the sounding of all available fitments. A secondary survey option was also identified which concentrated on the use of fire hydrants and fire hydrant controlling valves only.

Each survey option was completed under comparable circumstances by a two-man team comprising a team leader and an assistant with the data collection process defined by the following categories: Asset type; Asset number; Numeric reading; Material type and Geographical reference.

The primary survey option was completed by the sounding of every available fitment with the AS3. In some instances access to below ground servicing valves was partially obstructed and could only be made by the breaking and removal of the compacted debris around the proximity of the valve chamber and cover.

The secondary survey option was to concentrate on the sounding of the fire hydrant and fire hydrant controlling valve in order to determine if a correlation between the two methods could be obtained and if the survey option was successful in identifying high volume losses.

In both instances the survey equipment was configured with filters off, a volume setting of 75% of the total and held directly to the fitment surface for a minimum period of at least 20 seconds. No headphones were used and numerical values taken only, this was so no sound interpretation was taken into account from an experienced leakage engineer.

The numerical representation of generated system noise was captured from the digital indicator on the AS3 and duly recorded. In all instances the lowest numerical reading captured by the equipment was used.

2.2. Numerical Clustering

On the completion of each sub-division, the representative numerical data captured for each fitment surveyed would be overlaid onto a network schematic plan detailing asset position, size, type and distinct reference code. From this process a numerical footprint of the sub-division was obtained and an associated assessment completed. This highlighted any obvious numerical values in close proximity of the defined "intervention denominator" associated around a singular geographical point.

Prior to any further analysis, each high reading point was evaluated against an asset location. This was done as the asset may be in the location of a known noise, for example a pressure reducing valve or main incoming supply valve.

Each AOI would initially involve the re-capture of the numerical value recorded on the fitment at least 2 hours later in case the value was that of domestic draw-off. Should the numerical values be comparable with those initially captured then a more robust leak location process should be instigated with the utilisation of standard leak noise correlation processes and above ground acoustic listening techniques.

The AOIs may only then be removed from the schedule by the confirmation of leak position or quantification of the numerical value as legitimate consumption.

The overall numerical value data range captured throughout the project is presented at Table 1 and Table 2 respectively.

Table 1. All fitments survey results.

Primary survey – All fittings sounded	
Total numerical range	Material type
25 – 43	PVC
26 – 70	Cast Iron
27 – 60	Ductile Iron

Table 2. Fire hydrants only survey results.

Secondary survey – Fire hydrants & controlling valves only	
Total numerical range	Material type
29 - 35	PVC
21 - 40	Cast Iron
30 - 36	Ductile Iron

2.3. Intervention Denominators

The numeric data captured was generally found to be proportional across all material and fitting types particularly where no leaks were present. In places where several leaks were located, a significant increase in the numerical value was recorded. This increase in the numerical value can not indicate the size of the leak recorded for a known type of pipe material without further assessment of data captured from known losses. In an attempt to validate the numeric values captured and establish a leak noise threshold by material type, a simulation of a known loss was replicated from operation of a fire hydrant and the impact of the surrounding numeric values recorded on all valve types situated in close proximity.

Firstly a simulation of a known loss was replicated from the operation of a fire hydrant and the measurement of the subsequent impact of this upon surrounding numeric values recorded on all available valve types situated within a close proximity, the results of which are presented within Table 3

Table 3. Numerical values obtained during test.

Asset type	Material type	Distance from source	Numeric value[a]	Numeric value[b]	Numeric value[c]
Sluice Valve	Cast Iron	57 m	34	35	39
Sluice Valve	Cast Iron	57 m	31	37	39
Sluice Valve	Cast Iron	51 m	37	36	49
Fire Hydrant	Cast Iron	0 m	42	78	82
Sluice Valve	Cast Iron	38 m	31	39	61
Sluice Valve	Cast Iron	43 m	35	38	69
Sluice Valve	PVC	43 m	36	32	38

[a] Reading captured prior to test
[b] Reading captured during test at 0.2 litres per second
[c] Reading captured during test at 1.0 litres per second

From the above it was found that the amount of induced losses had little impact on the numeric values recorded on fitments directly situated on the non metallic pipe. Only those fitments situated on the metallic pipe recorded any “uplift” in numeric activity which was found to be both distance and flow proportionate.

Below are numeric values recorded during this exercise and any singular point identified by a numeric clustering greater than 40 was initially highlighted as an AOI and duly scheduled for further investigation, the outcome of which has been summarised below in Table 4.

Table 4. Area of interest summary.

Area reference	Numerical profile range	Investigative outcome
Bowness	40 - 59	PRV location no leaks found
Bowness	37 - 57	Incoming supply no leaks found
Bowness	46 - 60	Burst service pipe at ferrule connection
Glenmore	56 & 69 & 70	3 no leaking service valves found

Completion of the investigative outcomes identified a number of leaks, the predominant being from a numeric clustering of between 46-60 over a 200m distance. These leaks arose from a combined failure of a copper service pipe and associated

ferrule connection situated at a cast iron main. Following excavation and visual confirmation prior to repair, the loss was estimated at some 3 litres per second.

The remaining leaks identified were found to be general losses from service valves, predominantly due to the erosion of packing material from around the operating gland or the general corrosion to the body and bolt sets. These leaks shown readings of between 56 –70 when the AS3 was placed on the fitting but there was little or no transmittal of these leak noises to the surrounding fitting.

Unfortunately the impact on the numeric values obtained from the confirmed leak position did not incorporate a proportional or representative amount of plastic pipe material within the immediate peripheral vicinity. Therefore additional information should be obtained before a definite conclusion can be drawn.

In considering the relationship between numerical value and material type it is suggested that the following intervention denominators at Table 5 be considered however as mentioned earlier in this paper a calibration exercise should be carried out to obtain the intervention level as many characteristics can have an effect on the noise transmittal i.e. mains pressure, pipe material, granular backfill to name a few.

Table 5. Numerical intervention denominators.

Pipe material	Numeric intervention denominator value
PVC	40
Cast Iron	45
Ductile Iron	45

2.4. Survey Cycle

The survey cycle was completed in accordance with the two methodologies outlined in the section above. As previously mentioned, the survey areas comprised a variety of topographical, social and economic property types thus allowing a realistic productivity ratio to be ascertained. From the data obtained it can be reasonably concluded that:

- In carrying out an all fitment survey, an average timescale between fitments of 5.51 minutes can be expected; Average daily distance listening on all fittings was 5.8 km/day.
- In carrying out a fire hydrant & controlling valve survey only an average timescale between fitments of 5.0 minutes can be expected. Average daily distance listening on fire hydrants only was 14.2 km/day
- overage of mains length listening on fire hydrants only was 230% greater than doing that of all fittings.

2.5. Numeric Footprints

In analysing the numeric data sets recorded during the survey it is possible to build a graphical representation of each district meter area. Each representation may then be considered as a numeric footprint of the water supply characteristics and utilised as a reference point when carrying out any future survey, especially if any significant time period has elapsed.

On the whole it was found that the footprint for most supply mains situated within the survey areas and constructed of a metallic composite such as Cast or Ductile Iron recorded a comparable numeric value, this allowed AOI to be easily recognisable.

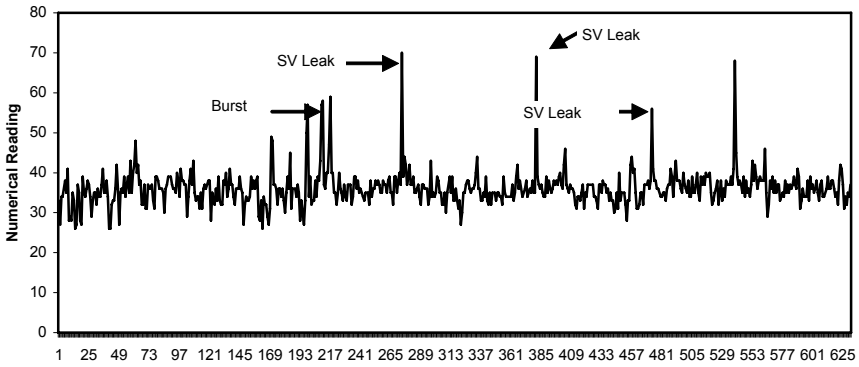


Figure 1. Material footprint for Cast Iron.

It can be seen from Figure 1 after the data has been cleansed that Areas of Interest (AOI) are easily identified. However the footprint obtained for non-metallic composites such as PVC was recorded at a consistently lower numeric value; this is due to the inherent low dispersal of sound through non-metallic composites and the absorption of the acoustic leak noise into the pipe wall. No Areas of Interest were found when completing this listening procedure on non metallic pipes and further investigation is required for this type of pipe line.

3. CONCLUSIONS

There is a correlation between a numeric value and the size of water leak and pressure. A calibration exercise should be carried out in each area to establish the acoustic noise transmittal and the intervention denominator. In assessing the impact of these leaks on the surrounding infrastructure, a numeric value greater than 40 is the intervention point The propagation of sound values (using the AS3) on the non-metallic material mains and infrastructure appear to be much less to that of metallic mains. Therefore without the addition of supplementary data sets taken from confirmed leak types on this material, the methodology established cannot be supported. Although the secondary survey option may be useful in identifying a catastrophic failure within a survey cycle, the primary survey option to incorporate all fitments is suggested as perhaps a more robust method for inclusion into a routine pro-active control plan. Above ground fire hydrants were found to be generally susceptible to additional ambient and surrounding noise levels.

Fire hydrants where water was replaced by air gave a lower numeric value as the noise transfer was totally reliant on the wall material of the main. It is therefore

considered that noise transferral is greatly enhanced when a main is pressurised with water and that sounding should only be carried out on fittings that are connected to a pressured system. Variation in local climatic conditions could in some instances impact on the numeric recordings obtained. In the case of high background noise, a numeric reading offset should be applied to compensate for the "uplift" in ambient conditions around the equipment.

When carrying out an all fitments survey option, an average timescale between fitments of 5.51 minutes per fitting for each two-man team may be anticipated under average conditions. Average daily distance listening on all fittings was 5.8km/day.

When carrying out a fire hydrant only survey option, an average timescale between fitments of 5.0 minutes per fitting for each two-man team may be anticipated under average conditions. Average daily distance listening on fire hydrants only was 14.2 km/day. This technique has proved successful and more investigation is required to obtain a generic footprint for non metallic pipe-work. This shows that some form of ALC can be performed with success from an unskilled operator with minimum training.

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Best Practice Strategies to Leakage Management – District Metered Areas and Pressure Control

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Abstract: It is evident that water is a limited resource in arid and semi-arid regions of the world, a situation that has highlighted, among other things, the need to reduce leakage from urban water distribution systems to levels that are considered economically acceptable. This paper reviews international experience in the application of the District Metered Areas (DMAs) as a leakage management technique for reducing water losses from distribution networks and describes the methodology applied and experiences gained at the Water Board of Lemesos (Water Board) in striving to reduce leakage to economically acceptable levels and to maintain these levels through the adoption of a proactive leakage management strategy. An account of the work carried out in redesigning DMAs at the Water Board is given together with the results obtained in further reducing real losses from the distribution system by pressure reduction and control. A comparison is made of the results achieved at the Water Board with the results reported from similar international case studies.

Keywords: Active leakage control, DMA, leakage reduction, pressure optimisation.

1. INTRODUCTION

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

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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.

2. DMA LITERATURE REVIEW

A District Metered Area (DMA) is defined as an area of the supply network having ideally about 3000 properties supplied preferably from a single entry point which is totally metered (water entering and leaving) and pressure controlled. 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. 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".

3. 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 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 zones 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.

Distribution of water to the DMAs is effected through dedicated ductile iron mains from the storage reservoirs. Each pressure sector 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.

4. 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.

5. 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 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.

6. RESULTS

6.1. 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.

6.2. 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.

7. 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 8% 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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Valuing Benefits from Wastewater Treatment and Reuse

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Abstract: This study aims to first elicit residents' Willingness to Pay (WTP) for the provision of central wastewater treatment and second farmers' WTP for recycled water. A case study for a rural and seaside tourist area, the Municipality of Kissamos located in North-West Crete is presented. Therefore, a contingent valuation study was conducted and members from 450 households were face to face interviewed. Mean WTP for the provision of wastewater treatment was 21.02€ to be paid as an additional charge on the quarterly water bill. Farmers' mean WTP for irrigating their olives with recycled water coming from the treatment of the wastewater was 61.2% the price of fresh water, which corresponds to 0.0872€. 84.70% of them would use the oil produced after irrigating with recycled water.

Keywords: Contingent valuation; cost benefit analysis; logit; tobit; wastewater treatment; water recycling.

1. INTRODUCTION

Wastewater treatment is essential for the development of semi-urban municipalities, of Greece, where tourism, agricultural and urban development have to co-exist eliminating any environmental impacts coming from untreated domestic wastewater. Such impacts include the sea and groundwater pollution, which respectively may discourage visitors and affect the quality of agricultural products, which are the main income sources. The construction of a wastewater treatment plant (WWTP) should eliminate such impacts, but it has to be "approved" by the residents who should also be willing to pay for its construction and operation. If projects of this kind are not fully accepted by residents, their operation can be upheld (Tsagarakis *et al.*, 2001).

The Contingent Valuation (CV) method has been applied to estimate how much

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people value benefits coming from the clean environment in the absence of a market value. In the case of wastewater treatment projects, people value benefits that include cleaner seawater, cleaner underground reservoirs and a cleaner environment in general. The construction of wastewater management facilities (sewerage network, central or decentralised treatment plants and disposal structures) are projects that can eliminate environmentally adverse impacts. There are many CV studies valuing benefits coming from the treatment of wastewater, which may differ according to the level of treatment the impacts of the pollution already caused and the development level of the country. Focusing on Greece, a CV study was undertaken by Kontogianni *et al.* (2003) who studied the individuals' willingness to pay (WTP) for ensuring a full operation of Thessaloniki WWTP which would result in improvements in the water quality of Thermaikos Bay (adjacent to Thessaloniki). Genius *et al.* (2005) studied the WTP for the construction and operation of a WWTP of three neighbouring municipalities of Crete (Greece).

Concerning WTP for treated effluents, a survey has been undertaken by Menegaki *et al.* (2007) who studied the value farmers place on recycled water and the value consumers place on products irrigated with recycled water.

The present paper seeks the WTP for the wastewater treatment of Kissamos municipality located in the northwest part of Crete. Kissamos is a tourist seaside area and does not have any centralized wastewater treatment facilities at present. Domestic wastewater is disposed into septic tanks or absorption pits. The study area is characterized by an increasing tourist development but agriculture remains a main source of income. The WWTP will be installed in the area of "Sfagia". It will receive wastewater from the city of Kissamos and the villages Agios Antonis, Kaloudiana, Drapanias, Koleni and Gramboussa. The proposed WWTP is an extended aeration activated sludge system of 10,000 population equivalent (p.e.), design flow rate 2,912 m^3/d and organic load 600 Kg BOD_5/d. The design year for the facility is 2021. The plant's winter p.e. for the target design year is 57% of the full capacity. The effluent quality limits are less than 20 mgBOD_5/L, 30 mgSS/L and 8 mgTN/L. Effluent will be chlorinated with NaOCl solution.

CV studies should clearly state, what the payment vehicle is, i.e. how the money required will be paid. All wastewater bills in Greece are directly related to the water bill and it is essential to study further the tariff structure of the municipality, as it is the water bill, which is the payment vehicle for charges for wastewater. For more on water bill structures see Tsagarakis (2005) while for tariff practices in Greece see Safarikas *et al.* (2005). The water tariff can influence the residents' WTP for wastewater treatment (Genius *et al.*, 2005). Kissamos municipality imposes a two-part tariff consisting of a fixed part and a second being an increasing block. Table 1 gives the price charged per m^3 according to semi-annual consumption. However, for the subsequent calculations quarterly (three month) conversion is made for comparison reasons, since in almost all municipalities in Greece, water charges are made 4 times per year. No water company has been established yet and water supply is directly managed by the municipality.

Table 1. Prices per m³ for the area under consideration (as for 2006).

Pricing component		Charge
1st part:	Fixed charge	23.27 €
2nd part:	Cost per m³ for 0- 60 m³ consumption per semester	0 €
	Cost per m³ for additional 61-120 m³ consumption per semester	0.32 €
	Cost per m³ for additional 121- 400 m³ consumption per semester	0.44 €
	Cost per m³ for >400 m³ consumption per semester	0.59 €

2. QUESTIONNAIRE DESIGN AND RESEARCH METHODOLOGY

A two-part questionnaire was designed to elicit residents WTP for wastewater treatment provision and for farmers buying recycled water. For the first part a similar questionnaire structure was used as by Genius *et al.* (2005) concerning the wastewater treatment part. In the second part the farmers' WTP was asked for buying the recycled water in order to use it for irrigation.

At the beginning of the interview, the identity of the research was given, the importance of giving sincere answers was stated and the anonymity of the respondents was ensured. Initially some warming-up questions concerning the environmental profile of the respondents were posed. Then the respondents' house attributes, possible wastewater disposal problems, septic tank evacuation cost, and water bill costs were recorded.

An information section that included flow charts, drawings and pictures for making the respondent understand the existing onsite wastewater treatment units (i.e. septic tank and absorption pit) but also the kind of project to be constructed was provided at the beginning of the interview. The absorption pit flowchart depicted the potential impact on the quality of the ground water. This information was essential in order to be sure that the respondent had the basic knowledge for the project about which he was asked to state his WTP. Then was asked the water bill and followed the WTP question for the construction of a WWTP. This would be part of the drinking water bill. Respondents with zero WTP had to answer a filter question to find out if this is a zero bid or a protest vote.

Afterwards, respondents who were also potential users of recycled water were selected by asking if they had agricultural land to irrigate. In case they had not, they were not asked further questions about water recycling. Only respondents who owned agricultural land were included in the second CV study. Olive trees are the main crop type irrigated in the study area and they are safe to be irrigated with recycled water of the expected quality of the WWTP under consideration. Farmers were asked at which price fresh water used for irrigation was bought. Then an information session concerning benefits and risks for reuse of treated effluents was provided and photos of large-scale wastewater reuse projects were shown. The respondent was asked his

willingness to pay and, if zero, he had to answer a filter question to locate protest bids. Then, they were asked whether they would consume olive oil irrigated with recycled water.

Finally, personal and general socio-economic characteristics such as respondent's sex, age, income, education level and income were asked.

3. WTP ELICITATION FORMATS

3.1 Elicitation Format for Wastewater Treatment

The WTP was sought through a double bound dichotomous choice (DB-DC) format with three different WTP starting amounts (in three different versions of the questionnaire). The question was: "Suppose that the wastewater treatment plant, which will increase the quality of the environment, were to be constructed. Would you be willing to pay *x* €?" (where *x* was calculated as 80% of the water bill for the first, 100% for the second and 160% for the third version of the questionnaire respectively). Those who had answered positively were further asked a higher bid: "Would you be willing to pay *y* €?" (where *y* was calculated as 100% of the water bill for the first, 140% for the second and 200% for the third version of the questionnaire respectively). Those instead, who had answered negatively, were further asked a lower bid "Would you be willing to pay *z* €?" (where *z* was calculated as 60% of the water bill for the first, 80% for the second and 120% for the third version of the questionnaire respectively).

The model under estimation is of the form $ln(WTP_i)=Y_i^*=X_i\beta+\varepsilon_i$, where i is the consumer index and $\varepsilon_i \sim N(0,\sigma^2)$. This relationship does not fall under the usual simple regression framework, because Y_i^* is not observed, namely we do not observe the respondent's real WTP. What we observe is whether consumers say yes or no to a certain bid. Consumers have the following four answer options to the two consecutive bids offered to them:

$$\begin{aligned} &\text{Response for bid 1: } Y_{1i}=1 \text{ if } \ln(WTP_i)=Y_i^* > t_{1i} \\ &\qquad\qquad\qquad\quad Y_{1i}=0 \text{ if } \ln(WTP_i)=Y_i^* \le t_{1i} \\ &\text{Response for bid 2: } Y_{2i}=1 \text{ if } \ln(WTP_i)=Y_i^* > t_{2i} \\ &\qquad\qquad\qquad\quad Y_{2i}=0 \text{ if } \ln(WTP_i)=Y_i^* \le t_{2i} \end{aligned} \qquad (1)$$

t_{1i} and t_{2i} stand for the natural log of the two bids. For the probabilities corresponding to the four options see Genius *et al.* (2005).

3.2 Elicitation Format for Water Recycling

The WTP question for buying the recycled water coming from the wastewater treatment should be replied only by those who would be using it. These are respondents who are full-time or part-time farmers, who were also able to provide the

price of irrigation water. Irrigation water is provided by the Organization of Development of Western Crete (OADYK in Greek) and other private individual water borehole owners. Irrigation water prices reported ranged from 0.10 to 0.50€ with an average of 0.1425€. This price should be born in mind when responding to the question "What is the maximum amount you would be willing to pay for recycled water for irrigating your olive trees?" The open-ended elicitation format was preferred since the price of a superior substitute good, which is the freshwater, was available. This allows the use of an open-ended format in contrast to a close-ended question format for the analysis. We did not use the maximum WTP as the dependent variable because this is related to the price of fresh water. Instead, we have used the percentage over the price of fresh water the respondent would be willing to pay. The regression analysis of the question, though, cannot be treated in the typical ordinarily least squares regression framework, as it is framed to zero. This is the tobit regression model and the observed data *y* are given by:

$$Y_i = \begin{cases} 0 & if \quad Y_i^* \le 0 \\ Y_i^* & if \quad Y_i^* > 0 \end{cases}$$

This applies to the $Y_i^* = X_i\beta + \sigma\varepsilon_i$ latent variable regression model, where Y_i^* is the latent variable, X_i is the vector of the i explanatory variables, β is the vector of estimated coefficients, and σ is the scale parameter (EViews 4 Users Guide, 2000).

4. RESULTS AND DISCUSSION

A total of 450 face to face interviews were conducted but 435 were retained for further analysis which represent a 10.7% sample of the population. Five interviewers who possessed the basic technical information on wastewater treatment and reuse were employed. This research took place from November 2005 to March 2006.

4.1. WTP for Wastewater Treatment

Mean and median WTP were estimated using the following formulae (Cameron, 1988).

$$\text{Estimated } WTP_{mean} = \exp\left(\bar{x}\hat{\beta} + \frac{\hat{\sigma}^2}{2}\right) \tag{2}$$

$$\text{Estimated } WTP_{median} = \exp\left(\bar{x}\hat{\beta}\right) \tag{3}$$

where,

$\bar{x}$: vector of mean values of the explanatory variables,

$\hat{\beta}$: vector of estimated coefficients, and

$\hat{\sigma}$: estimated σ.

The estimated values and confidence intervals are given in Table 2. Knowledge of mean WTP is necessary for performing a cost-benefit analysis for the construction of the WWTP since it provides the value people attribute to the benefits they will get from it. We calculate the mean WTP percentage residents are willing to pay as an increase to their water bill. It is estimated that this percentage is 93% and corresponds to 21.02€ (=0.93x[22.62€], i.e. the average bill). The estimated median amount residents are willing to pay is 20.14€. The delta method was used to obtain the confidence intervals (Greene, 1993). The amount reported has been calculated quarterly since almost all water companies in Greece charge water for this period.

Table 2. WTP statistics in €.

Statistic	WTP	95% Confidence interval
Mean	21.02	19.87-22.17
Median	20.14	19.15-21.14

A total of 18 real zeros (4.0%) have been recorded in the WTP for wastewater treatment. Respondents claimed "I cannot afford it" (8 responses), "I do not believe that the construction of the WWTP will cause any environmental improvement" (4 responses) and "I believe there is no problem with wastewater disposal" (6 responses). The survey also includes 17 (3.8%) protest responses. Respondents claimed verbatim: "I am entitled to have this service anyway (13 responses) and "I am tired of paying for the municipality" (3 responses). Also 15 respondents (3.3%) did not answer the WTP question, as they were against the construction of the WWTP. Almost all (14) of them were residents of he proposed area for the construction of the WWTP and they were probably afraid of any impacts from the WWTP operation and devaluation of their land. The last mentioned 15 questionnaires were dropped from the analysis.

4.2. Factors Explaining WTP for Wastewater Treatment

The explanatory variables used in the DB-DC model followed by their desriptive statistics are described in Table 3. It is worth noticing that the average amount respondents are already paying per month for empting their septic tanks is 11.88€. INCO1, INCO2, COPT2, SEX, STCOST and GRPO were significant at 1% significance level. COPO and EDU1 were significant at 10%. Also, CHIL, with a t-statistic greater than one, was kept in the final model. With the likelihood ratio test we kept dropping insignificant variables until we reached the version presented in Table 4.

Residents who recognized the groundwater reservoirs pollution as an important issue (variable GRPO) appeared as willing to pay higher amounts, which is to be expected since the groundwater is used for drinking and irrigation. Respondents who recognized the coastal pollution (variable COPO)

as a major problem are willing to pay on average higher amounts. This is due to the fact that tourism arrivals development has increased the residents' income lately and if the coastal pollution increases, tourism will decrease.

Table 3. List of variables and descriptive statistics explaining WTP for wastewater treatment (n=435).

Variable	Description	Mean	St Dev
GRPO	I value groundwater pollution as: 0=Not important, 1=Important	0.497	0.501
COPO	I value costal pollution as: 0=Not important, 1=Important	0.329	0.470
COPT	Consumption in m³ per month per family	20.749	7.494
STCOST	Monthly cost for emptying septic tank in €	11.880	14.982
SEX	0=Male, 1=Female	0.393	0.489
CHIL	Have children: 0=No, 1=Yes	0.795	0.404
EDU1	1= Junior and senior high school graduate, 0= Otherwise	0.609	0.488
INCO1	Monthly family income: 1= ≤1200€, 0= Otherwise	0.384	0.487
INCO2	Monthly family income: 1= 1201-1800€, 0= Otherwise	0.308	0.462

Table 4. Estimation WTP results for wastewater treatment and water recycling.

	Wastewater treatment			Recycled water		
Variable	Coefficient	t-Statistic	Probability	Coefficient	t-Statistic	Probability
C	2.2819	31.351	0.0000	0.5995	8.356	0.0000
GRPO	0.2933	6.067	0.0000			
COPO	0.0886	1.943	0.0520			
COPT	0.0238	13.278	0.0000			
STCOST	0.0032	3.182	0.0015	-0.0014	-1.246	0.2129
SEX	0.1214	2.883	0.0039	0.0722	1.978	0.0480
CHIL	0.0678	1.407	0.1593	-0.0756	-1.744	0.0812
EDU1	0.0783	1.911	0.0560	-0.0949	-2.620	0.0088
INCO1	-0.2335	-4.441	0.0000	-0.1711	-4.036	0.0001
INCO2	-0.1519	-2.708	0.0068	-0.0547	-1.248	0.2121
COO	-	-	-	0.4581	9.229	0.0000
σ	0.2922	15.240	0.0000	0.3289	25.663	0.0000
Log likelihood	-393.23			-137.72		

People who consume the higher amounts of water (COPT) are willing to pay more. This is contrary to our expectations because their percentage contribution

according to the bill is higher. However, this can be explained by the fact that these residents belong to larger families with children who consume more water but also are willing to pay higher bids in order to protect them and from the fact that drinking water is undervalued. We conclude that water is undervalued as the average water bill is 22.62€ in contrast to 32.51€ reported by Genius *et al.* (2005) for municipalities about 60 Km away. Residents who pay higher amounts for emptying their septic tank (STCOST) are willing to pay on average higher amounts. This is reasonable because the construction of a WWTP will make them stop using their septic tanks and hence stop paying for it.

The female population (SEX) is willing to pay on average higher amounts than the male one. Thus it appears that women are more sensitive to environmental issues than men. People with children (CHIL) are willing to pay on average higher amounts. This is because they have developed a sense of duty for safeguarding the environment and bequeathing it to their descendants.

Secondary school graduates (EDU1) are willing to pay more. This is due to the fact that outside urban centers the highest level of education achieved is usually the secondary one. So the "educated" people in that area saw the benefit in constructing a WWTP and were more willing to say yes to higher bids. University graduates however (EDU2) was not statistically significant and was omitted from Table 4, this is probably due to the fact that university graduates are not affected by the water pollution caused by the untreated wastewaters. This finding about education verifies findings of Genius *et al.* (2005).

People with incomes 1200-1800 €/month (INCO2) wish to pay on average lower bids ($\hat{\beta} = -0.1519$), comparing with others, while people with incomes less than 1200 €/month (INCO1) wish to pay on average much lower ($\hat{\beta} = -0.2335$) comparing with others, which is consistent with the economic theory.

4.3. Factors Explaining WTP for Recycled Water

The explanatory variables for the tobit model are described in Table 5. The mean and standard deviation from these variables do not differ much from those of the other WTP question of Table 2. The new variable (COO) was derived from the question, "*Would you consume olive oil irrigated with recycled water*", with values 0: certainly no and maybe no, and 1: certainly yes and maybe yes. It is worth noticing that 84.7% of the respondents would use olive oil irrigated with recycled water.

The estimation results for water recycling are presented in Table 4. Seven variables were significant for the model. Six of them had also been included in the DB-DC model. The following variables were significant at 1% significance level: INCO1 and COO. SEX was significant at 5%. CHIL was significant at a 10% level. Also, INCO2 and STCOST, with a t-statistic greater than one, were kept in the final model.

Table 5. List of variables and descriptive statistics explaining WTP for recycled water (n=372).

Variable	Description	Mean	ST Dev
STCOST	Monthly cost for emptying septic tank	12.001	15.516
SEX	1=Female, 0=Male	0.355	0.479
CHIL	Have children: 0=No, 1=Yes	0.798	0.402
EDU1	1= Junior and senior high school graduate, 0= Otherwise	0.616	0.487
INCO1	Monthly family income: 1= ≤1200€, 0= Otherwise	0.371	0.484
INCO2	Monthly family income: 1= 1201€-1800€, 0= Otherwise	0.315	0.465
COO	Consume olive oil irrigated with recycled water: 0: certainly no and maybe no, 1: certainly yes and maybe yes	0.847	0.361

Sex and income have the same sign with the WTP for wastewater treatment and the same interpretations can be given. On the other hand, different signs have resulted for CHIL, EDU1 and STCOST. The respondents with children (CHIL) want to pay less for recycled water, in contrast to their WTP for wastewater treatment. This can be attributed to parenthood sensitivity and aversion to any risk their children could be exposed to. Secondary educated people (EDU1) want to pay on average less, which means that although they want to pay more for the wastewater treatment, they pay less for recycled water. Similarly, people who pay high amounts for emptying their septic tanks (STCOST) are willing to pay on average less for buying recycled water. These last two findings can be explained by the fact that they have already given high bids for the treatment and believe that they have already paid the cost for buying it. It is concluded that 92.5% of them would be willing to pay for recycled water. On average they offered 61.2% (C.I. 46.3%-76.0%) of the freshwater price (0.612x0.1425€= 0.0872€). This high percentage over the price of an inferior good can be explained by the very low price of irrigation water. From those who did not wish to pay for it, 6.4% were real zeros and believed that recycled water is not clean, were afraid of illnesses or through that there is no need of using it since there is plenty of fresh water. On the other hand 1.1% were protest votes that claimed that did not trust that the effluent would be of the appropriate quality or through that it should be provided free of charge.

5. CONCLUSIONS

This paper performs in an integrated manner the valuation of environmental benefits form wastewater treatment and the benefits derived from the use of recycled water in Kissamos Municipality. Almost all residents (96.7%) agree with the construction of a wastewater treatment plant in the region, it is a common understanding that this project should be implemented. In order to obtain public acceptance of additional wastewater charges, it is of great importance to explain beforehand that these charges

are necessary in order to obtain a high living standard. Wastewater charges are directly related to the water bill. The water in the latter is undervalued and therefore in this framework wastewater charges cannot reflect their economic cost. Residents are willing to pay for wastewater treatment on average 93.0% (=21.02€) the money they pay for their drinking water bill. Also farmers that participated in the research are willing to pay for recycled water on average 61.2% (0.0872€) the freshwater price.

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Public Participation in the Implementation of WFD, Case of Anthemountas River Basin District

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Abstract: The Water Framework Directive 2000/60/EC establishes a water protection framework considering water as a heritage which must be protected, defended and treated as such. The WFD defines the requirements for the production of river basin water management plans in order to achieve this aim. Moreover it defines the necessity for motivating the active involvement of all the interested parties during the implementation of this Directive, in particular in the production, review and updating of basin management plans. In this work the methodology of ensuring the public participation and implementing the public participation procedure for the production of the water management plan in Anthemountas river basin in Thessaloniki prefecture in Greece is presented.

Keywords: Public participation, river basin, WFD, water management plan.

1. INTRODUCTION

Anthemountas river basin is situated at the south-eastern part of Thessaloniki Prefecture (Figure 1), covering an area of 320 km^2. Due to its vicinity to Thessaloniki urban centre, the area of the basin experiences an increasing urbanisation trend, which mostly affects the Municipality of Thermi and the other two Municipalities of the Basin (Vasilika and Anthemountas) to a smaller degree. In terms of economic activities, current trends portray a marked increase in the tertiary sector. However, the primary sector is still dominant.

The urbanization combined with the economic activities resulted in the increased demand for water of sufficient quantity and good quality. The most important problem regarding the management of water resources in the Anthemountas' Basin is that water demand by far exceeds the available renewable water recourses. Water needs are almost exclusively met through groundwater. Although surface runoff is estimated at 41 million m^3/yr, while demand is approximately equal to 22.5 million m^3/yr, the annual infiltration is estimated to be approximately 5.1 million m^3/yr

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(ANATOLIKI, 2006a).

Several years ago, Region of Central Macedonia, local Authorities, institutes and agencies began an effort to examine the area in order to protect the basin's water. Within this effort, Development Agency of Eastern Thessaloniki, "ANATOLIKI s.a.", has undertaken many initiatives the latter one of which resulted to the approval of the project LIFE -Water Agenda that is aiming at the pilot implementation of WFD in Anthemountas river basin. The project partner scheme consists of ANATOLIKI s.a. (beneficiary), Region of Central Macedonia, Municipality of Thermi, National Technical University of Greece, National Center of Environment and Sustainable Development, Autonomous University of Barcelona and Municipality of Milan.

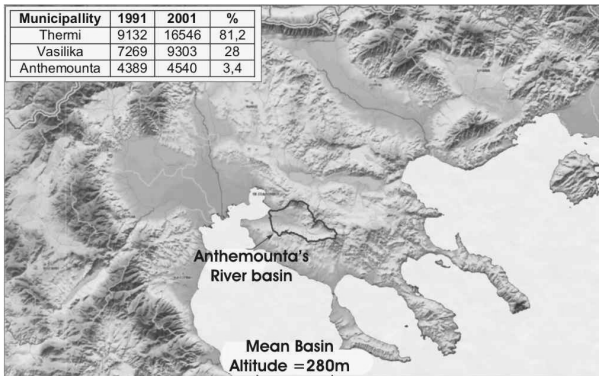


Figure 1. Anthemountas river basin - Population of the river basin.

The LIFE project refers to the integrated water resources management of Anthemountas river basin. It includes the development and application of a sustainable water resources management policy according to the provisions of WFD and the principles of Agenda 21 (UN, 1992b). The project started in October 2004 and will end in October 2007.

2. WATER MANAGEMENT POLICY

2.1. Water Management Scenarios

In the frame of the LIFE project, different water management scenarios for the basin district were produced, considering environmental, social and economical parameters, which have a strong impact on basin's water within the next 15 years. The aim of the above scenarios is the provision of alternative storylines regarding the freshwater demand and availability within the basin by the year 2020. These scenarios will be used as the key tools during the public participation procedure (EPEM., 2006):

"Business as Usual, BAU" scenario: This is the scenario that examines the basin's water resources status in case of the current trends being the same in the future. There will be no change in the political planning and the technical

intervention and the agricultural sector will be formed mainly by the economic extension/view of the new Common Agricultural Policy.

"Engineering, ENGI" scenario: The potential technical interventions are envisaged and designed in this scenario. Infrastructures in order to maximize the supply of water quantity as well to protect and improve its quality are proposed. The water demand in the agriculture sector is controlled by the economic factors introduced by CAP and the water supply provided by the infrastructures (reservoirs, wastewater treatments, sewerage networks e.g.).

"Hand in Hand, HIH" scenario: Within this scenario, the water demand and supply are governed by a number of management actions in a political point of view. In agricultural sector the water demand is driven by the economic as well as environmental factors of CAP. Additionally, WFD 2000/60/EC significantly contributes in the water supply conditions in Anthemountas basin. The agro-environmental regulations as well as the agro-land forestation measures as alternative agricultural land use are also factors. A number of infrastructures are also foreseen to increase the water supply.

2.2. Public Participation Methodology

A Public Participation (PP) Procedure between stakeholders, consumers and civil society was established in the area in order to attain a social consensus that will lead to the formulation of a social local agreement on water resources management. The developed methodology for the PP procedure (Figure 2) was based on the existing bibliography and similar case studies. Additionally, the social, economic and environmental characteristics of the area were taken into account. The proposed approach consists of 5 stages that are described in Figure 2 (European Commission, 2003).

2.2.1. Identification of thematic areas

The project team identified the main thematic areas that the deliberation had to follow, during the designing of the PP's methodology. However, their content could be finalized under the contribution of the participants in order to enhance the thematic areas and to ensure that the local society's worries and thoughts will be taken into account. Apart from the thematic areas, a corresponding flexibility will also be applied on the procedure by providing the participants with the opportunity to jointly decide upon it.

2.2.2. Identification of social partners and their contribution in the stages of participative procedure

A crucial point on the designing of the procedure was the identification of the social partners to participate. In PP cases, the type of participants differs according to the project subject, the implementation area and the activities that will be executed. The

participants should be approached by having in mind their personal characteristics. Due to the different interests (usually conflicting) among the participants, a customized approach needs to be carried out in order to adequately appraise their needs and priorities.

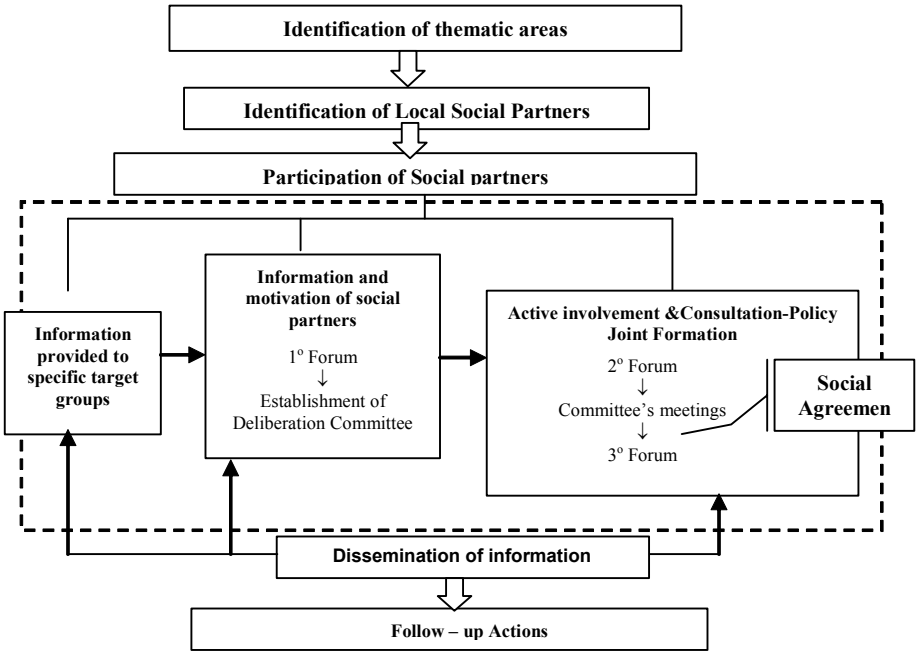


Figure 2. Stages of public participation procedure.

Initially the list of social partners was intentionally long and as the procedure was moving on, the target groups became more specific and the number of participants was limited. A general categorization of them is the following (Ntavos *et al.,* 2002):

- Political parties, local authorities and social groups
- Groups of economic interests (chambers, organizations of tourist offices, etc.)
- Local Environmental Groups and NGOs
- Public Organisations (Local, Regional, National, International) and Individuals
- Research groups, scientists and professionals etc operating in the area.

2.2.3. Participation of Social Partners in the Procedure

In an ideal situation and according to Agenda 21 directions, the social partners should be involved into all the project's stages that is from identifying the problem to designing the method for handling, implementing and finally evaluating it. However, in every project that includes a public participation procedure, adjustments are made according to decisive factors such as the budget and the duration. In the present project, the PP procedure, which is estimated to last 14 months, is structured in the following stages (ANATOLIKI, 2006b).

Table 1. Stages of public participation procedure.

Stages	Period
1st stage. Information provided to specific target groups	March '06 - April '06
2nd stage. Information and Motivation of local society – Establishment of a Deliberation Committee	May '06 - October '06
3rd stage. Active involvement, Consultation and policy making – Social Agreement	November '06 - April '07

1st Stage: Information provided to specific target groups.

The scope of this stage was the information of the local society about the objectives and aims of the LIFE project, the status of river basin's water bodies, the value of the public involvement in the decision-making and the methodology of the PP procedure in the case of Anthemountas river. The main target groups were:

- farmers, mainly members of local agricultural cooperatives,
- opinion leaders, individuals in the region whose attitudes influence those of the citizenry at large, and
- the students of the elementary and high schools.

The information actions focused to these targets groups, aimed at growing the idea of PP among them and the recording of their proposals and response in order to improve the PP procedure as well as the formation of the water management policy. In addition these groups are considered as the main supporters and dissemination factors of the procedure as well as the members of the deliberation committee. The information and the awareness of the students is crucial in order to change the consumer habits and to achieve the rational water use from the future generation, according to the theory of sustainability, and also to multiply - by them - the information procedure in the society.

2nd Stage: Information and motivation.

During this stage the more general information is provided to all the potential interested parties in the targeted and greater area of Anthemounta basin. Moreover the motivation for active involvement in the PP procedure is reinforced. The focal point of this stage was the 1st Forum of the social consultation for Anthemountas river water management (5th June, World Environment Day). During this event the scope and the stages of the consultation procedure were presented. In addition the participants of the forum were invited to contribute in the operation of a Deliberation Committee. The first Committee's meeting has taken place in June 2006 and its operation was continued, mainly in the period from November 2006 to April 2007. The members of the committee represent the local community and they bring in the PP procedure the opinions and proposals of the local community.

3rd Stage: Active involvement - Consultation and Policy making.

In the beginning of this stage the 2nd Forum took place. Within it, the following issues was presented:

- Conclusions of the 1st forum and 1st meeting of deliberation Committee.
- Critical presentation of Water Scenarios.

- Economic analysis, cost recovery of water services in Anthemountas Basin.
- Presentation of the Committee's Members and the consultation method.

After this event the Committee continue its operation in accordance to a "Give and Take process" (San Diego, 2001). In the frame of its operation workgroups was formed aiming at the active involvement of them and to the elaboration on different water issues in cooperation with the Life-project's partner scheme. As it was mentioned the main responsibility of the committee is the formation of an integrated water management policy, and expected to be accomplished at the end of May 2007. The agreed water policy will be presented in the final forum in order to be wholly or partly approved by the participants by setting-up a Social Local Agreement. A number of informative actions about the proposed water management policy will be organized and implemented focusing mainly on the local society aiming at the achievement of an agreement. These preparatory actions will also motivate and encourage the citizens to participate in the final Forum on June 2007. The final Forum is the closure of the PP Procedure. This Forum is the outcome of the overall procedure, the output of which is the proposed policy. All the participants shall sign a Water Protocol. Its main issues will be:

- Water uses (Infrastructures, rational water use, water pollution precaution, new technologies) and Pricing Policy.
- Environmental targets for a fifteen years period.
- Water Basin users' commitments and Monitoring Committee.

2.2.4. Dissemination of information and results

The main objective of this task was the identification of the most proper and effective dissemination means, having always in mind the special characteristics of the local society. Due to the wide span of the groups the following means were selected as the most effective: Advertisements and articles in the local newspapers, News releases, notices of information boards and use of posters in facilities of the area (the municipal offices, local buses etc), Telephone contacts, Internet and special events. The above means was applied during the whole PP procedure (Dryzek, 1997). By adjusting the means, the local population was informed, not only for the project and the water resources current state, but for the development of the PP procedure as well.

2.2.5. Follow-up actions

The PP procedure in the environmental management is a nonstop process that should not end once the initial targets are achieved. It should not be bound to achieving high numbers of participants while being executed but maintain the experience and the results in the long-term (European Commission, 2003). In the case of Anthemountas, the follow-up actions that will be implemented relate to the establishment of a Monitoring Committee for the monitoring of the Water Policy implementation and the use of the project results and conclusions in the implementation of WFD in the 11th

and 12th hydrological departments of Greece by the Region of Central Macedonia (Water Directorate), which is also a project partner.

3. CONCLUSIONS

Although a well-prepared informative campaign took place, the participation in the 1st Forum was limited. More specifically, even though more than 100 posters were posted and 700 invitations were sent, as well as many news releases were published in local newspapers, only 92 persons participated, the unequal geographical distribution of whom is also of special notice. In particular 55,5% of the participants came from the area of the Anthemountas basin with 64,5% coming from the Municipality of Thermi, 27,5% from the Municipality of Vasilika and only 8% from the Municipality of Anthemountas. The remaining 45,5% came from the area outside of Anthemountas basin. The distribution of the participants to the identified categories of social partners is shown in Figure 3.

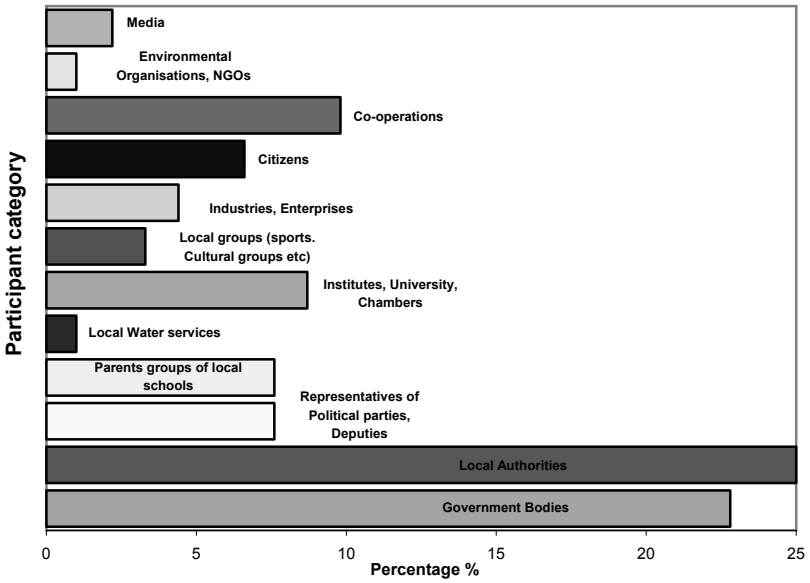


Figure 3. Public participation in the participative procedure.

ACKNOWLEDGMENTS

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Water Resources Management in the International River Basin Districts within the WFD 2000/60/EC: Problems and Perspectives

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Abstract: Water Framework Directive (WFD) 2000/60/EC inserts new patterns, criteria, institutions and procedures for the integrated water resources management on river basin level and can contribute significantly to the management of international European rivers and lakes. When Member States of E.C. or Candidate Countries are involved, the general approach and the detailed intentions of the Directive constitute a perfect base for international cooperation, so that frictions, intensities and conflicts that threaten the world peace can be avoided. However, for the European States which are not obliged or won't be in the near future to apply the E.C. legislation, this Directive may constitute a "reference system" for the improvement of their international waters management, by applying sequential steps to lead them to integrated water resources management. This paper examines; (a) the requirements of the WFD on the implementation of its objectives in the international river basin districts, (b) the perspectives that are opened up by its application, (c) the problems and the shortcomings that can result from its implementation and finally, certain proposals are made for their confrontation.

Keywords: Directive 2000/60/EC, hydrodiplomacy, integrated water resources management, international river basin district.

1. INTRODUCTION

Freshwater resources and their management have increasingly captured the attention of the international community, because of the lack of access to safe drinking supplies and sanitation for much of the developing world. These conditions combined with competitive demands, depleting groundwater resources and the water stocks's pollution worldwide have motivated greater international movement in water

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management issues, particularly those concerning international basins.

Water is shared among nations, areas and societies, without identifying administrative or national boundaries at its distribution among various users. The management of international freshwater systems is complicated because of the need for co-operation among the riparian states, a problem that is exacerbated when manifold countries are involved. Although, freshwater supplies are clearly limited, for most people water scarcity is caused by political, technological and economic barriers that limit the access to water and by competition among water users (Falkenmark and Lundqvist, 1998). From international experience, it is concluded that a complex problem is created when it comes to the water resources management, especially at areas where they are limited and political, nationalistic and religious differences don't allow, complicate or make impossible the cooperation among states. The threats that ambush by the misconduct of international water resources and the deficiency of cooperation and conventions among states, which share common water systems are numerous. There are currently 263 international basins worldwide, accountings for approximately 60% of global freshwater flow. Two persons out of five live in an international basin. This demonstrates how important it is for riparian states to co-operate in a basin's water resources management. Also, it is important that the international community and governments of all states that are shared international river basins must be activated on a collective basis, taking the appropriate steps.

Water resources management must be performed within water's natural limits, as they are shaped by its river basin, national or international, and not by political or administrative boundaries. So, when the river basin covers the territory of more than one state, water resources management plans must be provided for the whole basin, so that conflicts, intensities and frictions that threaten the peaceful co-existence of people will be avoided. Nowadays, the water resources management of international basins is one of the most important problems. Within the past few years, for the international water resources management, specific principles and methods have been followed, which have been taken by the science of International Relations and Diplomacy. Thus, a new scientific area has been formed, called Hydrodiplomacy (Vlachos, 1998; Elhance, 2000; Turton, 2002).

A waterway or a lake is international where its basin area and use are shared among two or more states. If a river basin extends across international boundaries, the WFD (WFD, 2000) specifically requires to be assigned to an International River Basin. WFD requires the collaboration of Members - States that are shared international river basins and encourages their collaboration with the third States, when river basin extending beyond the boundaries of the Community. Consequently, according to WFD river basin has an international character and the river basin management goes beyond the political, social and cultural boundaries. River Basin District (national or international) is the administrative unit for the river basin management and thus, all the stakeholders within the river basin need to take an active role in this framework. A total of 69 international river basins are in Europe, which include more than 194 International River Basin Districts. By them a

percentage of 68% belongs to the Member – States of the European Union and the rest to the Non Member States. Special attention needs to the international river basins in which some specific problems have to be overcome: legal and political discrepancies, communication problems, social, cultural, historical differences and economic issues. WFD indicates the year 2009 as a deadline to produce all the river basin management plans. However, WFD is less severe and somewhat ambiguous in its demands about the international river basin districts compared to its demands for national basin districts. The above observations motivate to look into the WFD requirements about the international water resources management. This paper examines; (a) the WFD's requirements on the implementation of its objectives in the international river basin districts, (b) the perspectives that are opened up with its application, (c) the problems and the shortcomings that can result from its implementation and finally, certain proposals are made for their confrontation.

2. INTERNATIONAL RIVER BASINS

The WFD defines the river basin as the area of land from which all surface run-off flows through a sequence of streams, rivers and, possibly, lakes into the sea at a single river mouth, estuary or delta (WFD, 2000). Also, according to WFD, a river basin is considered international if any tributary crosses the political boundaries of two or more nations. Wolf *et al.* (1999) made the distinction that the flow of this tributary must be perennial, which means that water must flow year-round. In the 1978 assessment of the United Nations, as international river basins were considered the ones that communicated directly to the final recipient of the water, which could be an ocean or a lake or a closed inland sea, in order to make a distinction between them and the tributary basins (U.N., 1978). This distinction is still valid today (Gleick, 2000).

In the United Nations assessment of 1978 (U.N., 1978), 214 international river basins were identified. However, another research made by Wolf *et al.* (1999), Wolf and Giordano (2002) by using Digital Elevation Models (DEM), led to the conclusion that the number of international river basins is 263 (not including Antarctica). The absolute numbers of international basins, as well as the nations through which they traverse, change over time in response to alterations in the world political map. For example, the break-up of the Soviet Union and of Yugoslavia led to the internationalization of the Dnieper, Don and Volga basins, as well as to changes in the political composition of existing international basins (e.g., the Danube, Ob, and Aral Sea basins). In contrast, the unification of Germany and Yemen resulted in the nationalization of two formerly international basins (Weser and Tiban). The 263 international river basins cover nearly 45.3% of the earth's land surface, they are considered of vital importance for the survival of the 40% of world's population and they generate roughly 60% of global freshwater flow. However, it is the political composition of these shared water systems, that highlights their vulnerabilities. Table

1 presents the number of the world's international river basins per continent.

Table 1. Number of international river basins per continent (Wolf and Giordano, 2002).

Continent	Africa	Asia	Europe	North America	South America	Total
Number of international river basins	59	57	69	40	38	263

There are 145 countries, from which part of their territory belongs to an international river basin (Wolf *et al.,* 1999; Gleick 2000). In particular, 15 of these countries lie entirely within the international river basins (e.g. Austria, Hungary, Slovakia, Switzerland etc.), 19 of them in a percentage greater than 95% (e.g. Czech Republic, Luxembourg, Andorra, Romania, Lithuania, etc.), while 17 others in a percentage greater than 80% (e.g. Latvia, Slovenia, Bulgaria, etc.).



Figure 1. International river basins of Europe.

As concluded by Table 1, most of international river basins lie in Europe. This is because of the breakups of U.S.S.R. and Yugoslavia into smaller nations and because of the division of Czechoslovakia into two independent states. Another important factor is that, since 1980, there have been new potentials for more accurate earth impression and several cartography methods have been developed in high rates, due to the technological progress, so that the borders' determination can be more specific compared to the past, when these procedures were more difficult. Figure 1 shows

international river basins of Europe (Wolf *et al.*, 1999). Percentage of 54% of Europe land surface is included within international river basin, while the percentages for Africa, South America, North America and Asia are 62%, 60%, 35% and 39%, respectively (Wolf *et al.*, 1999).

3. PROBLEMS OF INTERNATIONAL WATER RESOURCES MANAGEMENT

These problems can be divided into two main categories, as following: a) the ones caused by natural reasons and connected to the runoff and b) the ones caused by human interventions and activities (Yannopoulos and Giannopoulou, 2005). More details with regard to the conflict and the collaboration of the international river basins are given by Wolf (1998) and Yoffe *et al.* (2003).

3.1. Problems caused by natural reasons with respect to water discharges

These problems are: (a) floods, (b) erosions and depositions, and (c) riverbed changes.

3.2. Problems caused by human activities and interventions

The interventions and the activities coming from upstream countries can cause unfavorable effects to downstream countries, either by changing the quantity of water (by using dams, diversions and water abstractions) or by the continuous degradation of the river water quality or both of them. Generally, the problems that could be faced by downstream countries due to human interventions and activities coming from upstream ones are; (a) water constraint from the upstream country through dam construction, (b) water diversion from an international to a national basin, (c) inability of downstream country to plan in a long-period, (d) causing floods to downstream country due to deforestation at upstream one, (e) derangement of ecological balance and (f) transboundary pollution.

However, downstream countries may also cause problems to upstream ones by; (i) overpumping common groundwater (recession of water table level of the upstream country) and (ii) constructing dams nearby borders, so that the river's discharge can be changed and many areas of the upstream country will flood.

4. INTERNATIONAL CONVENTIONS

The physical, economic and social disparities among riparian nations that share river basins make their management complex. International treaties and agreements serve to provide structure that allows nations to address these disparities within a legal framework. This structure can provide joint management and monitoring of the resources in order to support sustainable development of water resources, including

management of water flow, water quality, and infrastructure development. Care must be taken to ensure that disparities do not translate into unfair inequalities. Consequently, from a geographical point of view, a river basin should be developed and managed as a whole, regardless of political divisions. Thus, international cooperation, either on information and research sectors, or on the construction and conservation of the infrastructure of international rivers and lakes, is mandatory. Therefore, it should be stressed that the management, planning and development of water resources of international river basins is one of the most important problems of the 21st century. Furthermore, it constitutes one of the most important long - dated or short - dated reasons for disputes and conflicts among the riparian states of international basins.

At international level, there is not legislation laying down legally binding rules. However, the international community, in order to set a general framework of principles for cooperation and confrontation of the conflicts and the intensities concerning water resources management of international river basins, has incorporated a wide variety of international instruments, of which the most eminent legislative tool is the Convention of United Nations for the "Protection and Use of Transboundary Watercourses and International Lakes" of 1992, which has been valid since October 6, 1996 (Bosnjakovic, 1998). The basic principles of this convention, also known as "Helsinki Rules" or "Helsinki Convention", are summarized as follows (article 2) (U.N. 1992); (a) protection of transboundary waters (including surface and ground) by preventing, controlling and reducing pollution, (b) restoration and maintenance of ecosystems, (c) reasonable and equitable use of transboundary waters and (d) conservation and restoration of ecosystems. In order to achieve these targets, states are particularly obliged to prevent, control and reduce the pollution of transboundary waters, caused by point and diffuse sources, as close to them as possible. Additionally, the Helsinki Convention inserts and accepts specifically the "precautionary principle" and the "polluter pays principle" in the implementation of such measures. Also, it recognizes the requirement that the water resources management should meet the needs of the present generation, without compromising the ability of future generations to meet their own needs.

Also, a relevant Convention of the United Nations is the "Law of non-navigational uses of international watercourses" (U.N., 1997). This Convention has not yet entered into force pending its ratification by 35 countries. This Convention, which is known as "New York Convention", incorporates three main principles: (a) "Equitable and reasonable utilization and participation". This principle requires the utilization and the development of an international watercourse to be "optimal and sustainable", by taking into consideration the interests of the concerned watercourse states. (b) "Obligation not to cause significant harm". This principle enjoins watercourse states in utilizing an international watercourse in their territories, to take all appropriate measures to prevent the causing of significant harm to other watercourse states. Nevertheless, if significant harm is caused, the state or states whose use causes such harm, shall take all appropriate measures to eliminate or mitigate this harm and,

where appropriate, to discuss the question of compensation. (c) "General obligation of cooperation". According to this principle, watercourse states shall, on a regular basis, exchange readily available data and information on the condition of the watercourse and in particular these of a hydrological, meteorological, hydrogeological and ecological nature and related to the water quality, as well as related to forecasts. It is obvious that all these principles, on which the Convention is based, have a great potential for dispute prevention. However, according to McCaffrey (2001), this Convention should be viewed as an instrument that reflects a general starting point for the negotiation of agreements related to specific international watercourses, and, because of the absence of any applicable agreement, sets basic parameters governing the conduct of riparian to those watercourses states.

Because of lack of treaties among the riparian countries and due to deliberate procedure to the Helsinki Convection confirmation, International Law Association, in 2004, has forwarded the review of Helsinki Rules and has added new rules and directives, which had been left out from the first edition (I.L.A., 2004). This review combines the international environmental legislation to the national and international laws about water, while simultaneously incorporates the demands of United Nations Convention for "the law of non-navigational uses of international watercourses" (Mylopoulos *et al.*, 2005).

5. DIRECTIVE 2000/60/EC: WATER RESOURCES MANAGEMENT OF INTERNATIONAL RIVER BASINS

Directive 2000/60/EC establishes a framework for the European Community in water policy sector and it constitutes a great challenge, because it aims at sustainable management and protection of all water resources, i.e. inland surface waters, transitional waters, coastal waters and groundwater. Member States and Candidate Countries had to harmonize the WFD provisions into their national legislation until December 2003.

The objective purposes of WFD are the following; (i) prevention of further deterioration, protection and enhancement of the status of water resources, (ii) promotion of sustainable water use based on a long-term protection of available water resources, (iii) aim at enhancement protection and improvement of the aquatic environment through specific measures, (iv) ensuring the progressive reduction of groundwater pollution and planning for its prevention and (v) contribution to mitigating the effects of floods and droughts. Therefore, WFD contributes to; (a) the provision of the sufficient supply of good quality surface water and groundwater as needed for sustainable, balanced and equitable water use, (b) a significant reduction in pollution of groundwater, (c) the protection of territorial and marine waters and (d) achieving the objectives of relevant international agreements.

A very important concept of WFD is the demand to establish integrated river basin management plans, with the aim of achieving the Directive environmental

objectives and especially, the achievement of "good status" in terms of quality and quantity for all Community waters by 2015. The basic thinking behind the term "good status" is that water can be used by humans as long as the ecological function of the water body is not significantly impaired.

Three main aspects of international water resources management are central in the WFD, which they can be considered as decisive steps for the improvement of international management:

- *Establishing collaboration*: WFD (article 3 (3)) requires Member States to ensure that a river basin covering the territory of more than one Member State is assigned as an International River Basin District (IRBD). Also, it requires that each Member State shall ensure the appropriate administrative arrangements, including the identification of the appropriate competent authority, for the application of the rules of the WFD within the portion of any international river basin district lying within its territory.
- *Data and monitoring*: River basin district authority had to provide all details requested in Article 5 of WFD on the natural characteristics of each river basin, up to December 2004. In the long run, this requirement will achieve the need for organizing these processes, gathering more useful and relevant information and rendering the exchange process as much as possible efficient and easy using modern technology.
- *Plans of Measures*: WFD (articles 11 and 13) requires a management plan to establish for each river basin district, the first one shall be published until 2009. This work will include measures that can be taken in order to achieve the environmental qualitative objective (ecological, quantitative, chemical status and protected areas) of "good status".

6. COMMENTS

Directive 2000/60/EC is based on the concept of integrated river basin management and consequently, it can contribute to the introduction of "best" management practices concerning international waters, even the ones of Non Member States of the European Community. Territorial management based rather upon physical than administrative boundaries is an important point stressing the innovative character of the WFD. Moreover, WFD encourages Member States to pursue the collaboration with the Third Countries, when a river basin extends beyond the European Community's borders. Consequently, the achievement of WFD environmental objectives concerning the management of international river basins requires international management.

WFD proposes that each Member State must ensure cooperation for producing one single International River Basin Management Plan (IRBM) for the International River Basin District (IRBD) falling within the territories of the European Community. However, at the same time, in a manner somewhat ambiguous, WFD indicates that failing to produce such a plan, the Member States should at least set up a plan for the part of the IRBD falling in their own territory. If IRBD extends beyond the European

Community territories, then WFD encourages Member States to cooperate with the Third Countries, so that water resources management should be carried out within the whole international river basin.

However, WFD is much less severe and somewhat ambiguous on its claims for these IRBDs compared to its claims for the national districts. Consequently, these "soft" requirements involve a risk to undermine the WFD intentions for water resources management of the river basins. In this way, it is concluded that WFD does not impose the international water resources management in a rigorous way, but it pursues to be a useful supporting frame for integrated management and collaboration of the riparian states.

According to Grimeaud (2001), and due to rather confused formulations of WFD, there is a risk to have multiple interpretations by Member - States in its implementation. Also, this case was mentioned by Macrory and Turner (2002), who pointed out that, although the international dimensions of WFD are clearer than the previous directives, the requirement for close cooperation of Member States for the water resources management on international river basins and, also, the strict legal requirements, in order to accomplish this common management are, rather, weak. As pointed out by Chave (2001), the intervention level which is materialized through WFD and mainly, the program of measures which demands great effort, in order to improve the cooperation. Also, Savenije and van der Zaag (2000), dispute whether the management of international river basins requires a common legal framework and, in this way, whether WFD may be able to serve as a common framework and enforce the joint international management or not.

Another arising issue is the extend and the way with which this joint management will be done. Possibly, it depends on the available water resources and the willingness for collaboration of the riparian states. Also, a very important matter is, whether the riparian states will choose to follow the aims of WFD and to harmonize their activities to the whole river basin (international level) or if they will cut down the implementation only at their political borders (national level). Consequently, WFD's intension to manage all E.C. water resources according to their hydrological borders and not to the political ones may not be realistic. However, the WFD may contribute substantially to the improvement of international European rivers' and lakes' management, especially, in cases in which Member or Candidate States are involved. The general approximations and the detailed demands of WFD constitute a "perfect" base for the management of the environmental status of European water bodies.

7. CONCLUSIONS

According to the aforementioned analysis, the conclusions are: (a) WFD encourages Member States to cooperate with Non-Member States for the water resources management of international river basins, in a river basin scale. This cooperation is of high importance and is related directly to the security and peace maintenance, because

lack of collaboration may cause tensions and conflicts among the relevant states; (b) WFD is less severe and somewhat ambiguous of its demands about the international river basin districts compared to its demands for national basin districts. Consequently, in these cases, there is a risk to undermine the WFD intentions for water resources management by river basins; (c) WFD intentions for the water resources management of international river basins may have different interpretations by Member States concerning the assessment and the management of international river basin districts and (d) there is uncertainty about the way with which Member States will define and apply WFD's demands about the international basin districts, because there are still unclear points in the Directive. Therefore, it is necessary that working groups will be created, in order to follow a unified strategy for the WFD implementation, which will examine the particular subject.

Finally, the mean level pursued by Directive, especially, the one concerning the programs of measures in the whole international river basin, demands a lot of work (preparation of guidelines on a range of technical issues), in order to improve the cooperation among the interested states. Many examples of these co-operations for the Europe concerning international basins are the cases of the rivers Danube, Sava and Rhine and the cases of lakes Geneva and Sevan.

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Technical and Economical Evaluation of an Urban Water Distribution Network-Upgrading Project

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Abstract: When a Water Utility manager is called to decide whether a specific project, aiming to upgrade the network's performance level, will have the desired results, is usually forced to trust the project's ex-ante evaluation. Ex-ante evaluation is what is available; ongoing evaluation is usually a failure-correcting process; ex-post evaluation is what is needed. But who can fast-forward the time? Experience form similar projects implemented in networks having more or less the same characteristics can show the way. The paper shares this kind of experience resulting from the ex-post evaluation of a project, implemented in Greece.

Keywords: Technical and feasibility analysis, water distribution networks.

1. INTRODUCTION

When a Water Utility manager is called to decide whether a specific project, aiming to upgrade the network's performance level, will have the desired results being feasible at the same time has to relay on the project's ex-ante evaluation that is based mainly on assumptions. This is why he usually hesitates, having second thoughts. The project's ex-post evaluation is the only way to judge whether its approval was a wise think to do or not. Ex-ante evaluation is what is available; ongoing evaluation is a failure-correcting process; ex-post evaluation is what is needed. But who can fast-forward the time? Experience form projects implemented in networks having similar characteristics can be used as the most-desired time machine. The paper shares this kind of experience resulting from the ex-post evaluation of a project implemented in Thessaloniki (GR) including PRVs installation and launching of a Public Awareness Campaign, to decelerate pipe brake rates; and decrease urban water consumption.

Studies showed that the most effective ways to decelerate pipe brake rates are on time pipe replacement (when its economical lifetime has expired and its repair is just money loss) and lifesaving interventions projects implementation. The present paper follows this path. At first, a methodology developed by Kanakoudis and Tolikas in

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2001 (used by several researchers since then) that determines the optimum replacement time of water pipes is briefly presented and applied in the network of Thessaloniki, to show what would have been the case regarding pipe replacement needs if no lifesaving interventions project had been implemented by the local Water Utility on time. Then, based on the project's results (reduced pipe break incidents), the feasibility analysis of this project takes place in order to verify or not the decision of the Water Utility.

2. OPTIMUM REPLACEMENT TIME OF A PIPE

To determine the optimum replacement time of an urban water delivery pipe network, the pipe-break rates forecasting models, have to be developed based on the available failure data records, and all kinds of costs related to a pipe failure should be estimated.

2.1. Pipe-Break Rates Forecasting Models

The first attempts to develop pipe-break rates forecasting models based on the available failure data records, took place in late 70's. The same period experts started to study the several parameters affecting the characteristics of a pipe break (Lane and Buehring, 1978). Results showed that break rates were not as strongly correlated to the age of pipe as it was initially supposed (O'Day, 1982). This was strongly doubted by a sequence of more detailed studies, regarding the quality of break data records (Kettler and Goulter, 1985). These studies revealed a strong correlation between the size of the pipe and the type/rate of its breaks. Specifically, delivery pipes (∅<400mm) use to suffer from beam breaks (circumferential) due to their static attitude. On the contrary, water mains (∅>400mm) usually experience ring breaks (longitudinal), crushing-hole blowouts, and joint failures. At first the break rates of delivery pipes were thought to be higher than those of mains. This was strongly doubted by the results presented by Walski and Peliccia in 1982. In 1981, Clark combined pipe break rates with the size, pressure conditions, exterior loads and neighboring conditions of the pipe. In 1985, Kettler and Goulter examined the break rates variations in conjunction with the pipe's material, analyzing the types of breaks for various materials. In 1988, Goulter and Kazemi, proved the time and space clustering of breaks near an initial failure location, using the data available for the city of Winnipeg, Canada. After all these years of dedicated efforts, researchers came up to several types of break rate forecasting models (e.g. exponential, linear, logarithmic), depending on the quality and quantity of the available data records. The present study uses the exponential model:

$$N(t) = N(t_o) \cdot e^{A(t-t_o)} \quad (1)$$

where, t is the time in years; A is the growth rate coefficient (1 year^{-1}); N(t) is the number of pipe breaks per km in year t; and t_o is the pipe installation date (actual date for an

individual pipe or mean time for a pipe group regarding its material, use, size and type). This model was initially presented by Walski and Peliccia (1982) and was further developed by Kanakoudis and Tolikas (2001), in order to utilize the results of other studies (Cabrera *et al.*, 1995; Clark and Goodrich, 1989; Shamir and Howard, 1979).

2.2. Pipe Replacement and Repair Costs

A pipe's replacement cost includes general expenses and abnormal costs that can be accurately determined considering the pipe's material, dimensions (length/size) and installation site (road/pavement), using analytical tables available. On the contrary, a pipe's repair cost is not that easy to determine as it depends on the break's characteristics (magnitude/significance) and the repairing method applied. The total repair cost includes costs directly related to the repair works (labor, transport, equipment, repairing materials, landscaping, supervision, general and abnormal costs) and costs that quantify the effects of the break related to the Water Utility and its customers. These indirect costs result from the Water Utility expenses regarding the use cycle (intake/treatment/supply) of the water losses during breaks (non reciprocal costs) and also from the facts that during the repairs: a) the Water Utility fails to (fully or partially) satisfy the water needs resulting in lost revenues; b) the fire extinguishing water supplied pressure fails; c) there are damages to third parties; d) there is a social cost (multiplication factor D) due to the related public annoyance/dissatisfaction. Studies showed that this cost can be two (D=2 for small pipes) to four (D=4 for supply mains) times the actual repair cost (Clark *et al.*, 1981; Walski-Pellicia, 1982; Cabrera *et al.*, 1995). In Greece, studies proved that in Athens D=4 (Kanakoudis-Tolikas, 2001) while in Thessaloniki D=2 (Kanakoudis-Charavgis, 2005). Regardless of the basic guidelines, the D factor should be calculated for each case study network.

2.3. Determining the Optimum Replacement Time of a Pipe

The analysis uses the exponential pipe age-break function and refers to a specific type of break (regarding its restoration time), ensuring that the pipe length involved in a break, its repair and replacement costs do not vary with time, resulting in a constant repair unit cost UC_{Rr} (€/m). The repair cost of breaks in year t is thus:

$$C_{Rr}(t) = UC_{Rr} \cdot N(t) = UC_{Rr} \cdot N(t_o) \cdot e^{A(t-t_o)} \quad (2)$$

If t_p is the present year, then the present value (PV) of the repairs in year t is:

$$PV\left[C_{Rr}(t)\right] = \frac{C_{Rr}(t)}{(1+R)^{(t-t_p)}} = \frac{UC_{Rr} \cdot N(t)}{(1+R)^{(t-t_p)}} = \frac{UC_{Rr} \cdot N(t_o) \cdot e^{A(t-t_o)}}{(1+R)^{(t-t_p)}} \quad (3)$$

where R is the mean annual inflation rate. Considering that the replacement takes place in year t_r, then the PV of the total cost for the previous repairs t_r-t_p is:

$$PV\left[\sum C_{Rr}(t)\right]=\sum_{t=t_p}^{t_r}\frac{C_{Rr}(t)}{(1+R)^{(t-t_p)}}=\sum_{t=t_p}^{t_r}\frac{UC_{Rr}\cdot N(t_o)\cdot e^{A(t-t_o)}}{(1+R)^{(t-t_p)}} \quad (4)$$

The replacement cost has a constant unit value UC_{Rm} (€ per km) and present value:

$$PV\left[C_{Rm}(t_r)\right]=\frac{UC_{Rm}}{(1+R)^{(t-t_p)}} \quad (5)$$

It is obvious that as the PV of the total repair cost increases with t_r and the PV of the replacement cost decreases with t_r, the PV of the total maintenance cost is therefore:

$$PV\left[C_{tot}(t_r)\right]=PV\left[\sum C_{Rr}(t_r)\right]+PV\left[C_{Rm}(t_r)\right]=\left[\sum_{t=t_p}^{t_r}\frac{UC_{Rr}\cdot N(t_o)\cdot e^{A(t-t_o)}}{(1+R)^{(t-t_p)}}\right]+\frac{UC_{Rm}}{(1+R)^{(t_r-t_p)}} \quad (6)$$

Concluding, the optimum replacement time t_r^* is:

$$t_r^* = t_o + \frac{1}{A}\cdot\ln\left[\frac{UC_{Rm}\cdot\ln(1+R)}{UC_{Rr}\cdot N(t_o)}\right] \quad (7)$$

which corresponds to the year in which the annual increase in repair cost is equal to the annual decrease in replacement cost.

2.4. Optimum Replacement Time of Thessaloniki Pipes

Thessaloniki is the second biggest city in Greece. Its network consists of 1,556km pipes, serving 1,000,000 customers. The annual water consumption is 94,000,000m^3. Pipe materials include asbestos (58%, ∅80-350mm); cast-iron (16%); PVC (15%, ∅50-150mm); iron (11%). The pipe failure records being kept in the existing 8 pipe service centers were used to specify the breaks per pipe material. PVC pipes were not further examined as they experience too many annual breaks, regarding their age (installed in early 90's). This resulted from the lack of know-how and construction experience and also from the fact that some PVC pipes (connection ones) were constructed by the residents themselves (in order to satisfy their own needs) and not by trained personnel, aiming only at cost reduction instead of quality of service.

To calculate the optimum replacement times (t_r^*) for all types of pipes the first step was to develop their break age forecasting models. For the case-study network considering the mean installation date of each pipe type (Cast-iron: 1973, Iron: 1975; Asbestos: 1980), the models developed are for Asbestos: $y=0.6763e^{0.0341x}$; Cast-iron: $y=0.5258e^{0.0341x}$; Iron: $y=1.4608e^{0.0341x}$. The lack of long time break data records resulted in the same break growth rate for all pipe materials (3.47%). This "mean" value is comparable both to the values that Kanakoudis and Tolikas, reported in 2001 for the water delivery network of Athens, Greece (1.18% for asbestos, 9.33% for cast-iron); and to the values reported in similar studies worldwide (1%-15%) (Shamir and Howard, 1979; Walski and Pelliccia, 1982). As expected, iron pipes suffered more initial breaks ($N(t_o)=1.4608$), compared to asbestos (0.6763) and cast-iron pipes (0.5258), due to the highly corrosive soil. The next step was to determine the actual

pipe replacement and repair costs. Since 2000, trouble-causing pipes (<∅110mm) are being replaced by ∅110mm PVC pipes. Based on the Water Utility's detailed pipe maintenance records, the total replacement cost is 39.70€/m, while the mean repair cost is 773€/break. Kanakoudis and Charavgis, reported in 2005 that in Thessaloniki, the social cost related to a pipe break is two times bigger than the actual repair cost (D=2). Therefore, the total mean repair cost of a break is 2,320€. The last two rows of Table 1 present the optimum replacement time for the pipes, when the social cost is considered or not respectively. These values are smaller compared to those reported by Kanakoudis and Tolikas, in 2001 for Athens network (232 years for asbestos; 57 years for cast-iron pipes when the social cost is considered), due to the fact that in Thessaloniki the repair costs growth rate is higher than the replacement costs growth rate, compared to Athens. All iron pipes should be replaced, as their optimum replacement time has already passed. Although the optimum replacement time for asbestos and cast-iron pipes is 2022, 50 km asbestos and 43 km cast-iron pipes should be replaced as soon as possible.

Table 1. Estimation of the optimal replacement time for the network pipes (<∅400mm).

	t_o	1.1. N(t)	1.2. $N(t_o)$	1.3. A	1.4. UC_{RM}	1.5. UC_{Rr}	1.6. $(D+1)*UC_{Rr}$	$t_r^*-t_{o(D)}$	$t_r^*-t_o$
Asbestos	1980	$0.6763e^{0.0341x}$	0.6763	0.0341	39.70 €/m	773 €	2,320€	42	75
Cast-Iron	1973	$0.5258*e^{0.0341x}$	0.5258	0.0341	39.70 €/m	773 €	2,320€	49	83
Iron	1975	$1.4608*e^{0.0341x}$	1.4608	0.0341	39.70 €/m	773 €	2,320€	19	53

3. EVALUATION OF THE PROJECT IMPLEMENTED

In Thessaloniki 65,000 to 83,000 m^3 of water are being lost every day (30% of the total water supplied). This fact forced EU to "suggest" that the Water Utility should find a way to minimize those water losses in order EU to co-finance the construction of a water supply aqueduct from Aliakmonas river to the city (studies proved that this water work will supply the city with enough water to cover its needs till 2050). Another presupposition set by EU was that a more rational use of the water should be achieved by implementing an effective water demand management policy. This forced the Water Utility to apply a huge research project based on water pressure monitoring, in order to fully understand the way that its network operates. Results revealed that in many network sections the water pressure was way more than 40m, which is considered satisfactory for an 8 floors building without a boosting device. Based on these findings, the installation of 5 PRVs in specific "nodes" of the network that took place in 2000, was considered to be the most cost effective thing to do.

The first step for the project's ex-post evaluation was to define its positive impacts on the pipe break rates. Prior to the installation of the PRVs (2000), the pipe break growth rate was 3.47%, while since 2000 breaks that actually occurred were reduced by 38.27% (1,202 less breaks) compared to those that would have occurred in 2005 if no measures were taken (Figure 1). The water losses during a break were estimated to

be 1,357 m^3 (∅200mm pipe; water velocity: 2m/sec; break duration: 5h). Thus, the reduced breaks from 2000 to 2005 (3,960 less breaks in total) resulted in savings of 5,373,720 m^3 (are expected to reach 7,224,668 m^3 by the end of 2006, as the total breaks are expected to be 5,324 less than those that would have occurred since 2000).

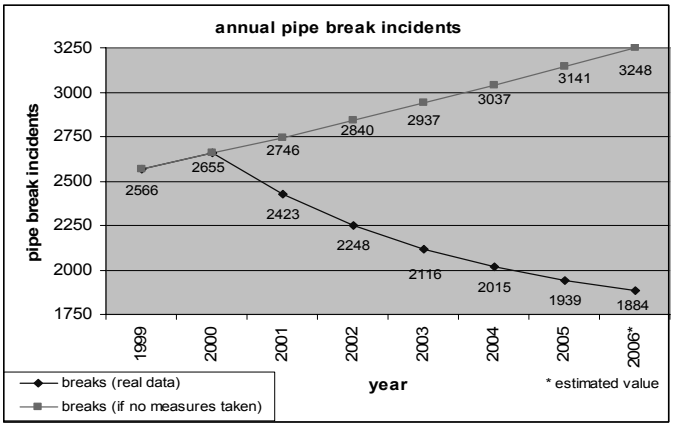


Figure 1. Annual pipe break incidents.

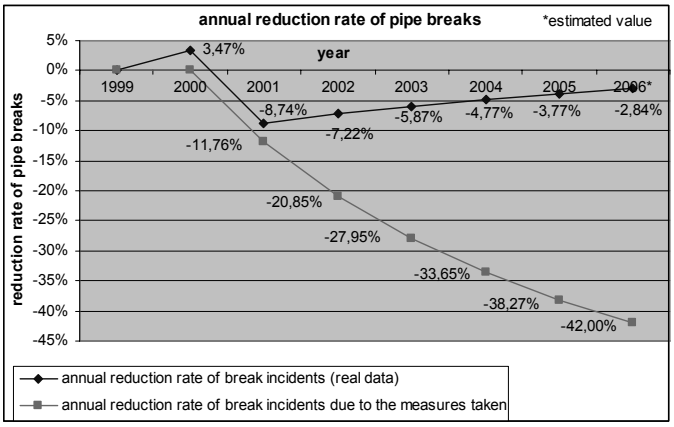


Figure 2. Annual pipe break rate.

3.1. Environmental Aspect

The project had an important water saving outcome (7,224,668 m³ less water being lost by the end of 2006). Based on the Water Utility records, till 2000, although the annual growth rate of its customers was 1.5%, the respective growth rate of the water consumption was 1%. The "water savings" of 0.5% resulted from a public awareness campaign, lunched by the Water Utility. The need for immediate water conservation was urgent due to the severe water shortage problems the city is facing during the last decade. Since 2000, although the annual growth rate of the customers was also 1.5%,

the total water consumption was annually reduced by 1% (0.68 Mm^3/year that is 4.76 Mm^3 for the period 2000-2006). This reduction was a joint result of the PRVs installation project; an Aggressive Water Pricing Policy (increased water rates, Inclining block rates); and an Aggressive Public Awareness Campaign, adopted by the Water Utility. As a result, the annual water consumption was decreased by 1% per year instead of the 1% annual increase expected. This means that since 2000, the water utility charged 1.36Mm^3 less water per year (9.52 Mm^3 since 2000), than it would have sold, if no water saving measures were taken. These water savings must be added to the water savings due to the reduced pipe break incidents occurred in the same period (7,224,668 m^3). So the environmental importance of the entire project is strongly justified, as the total water savings since 2000 reached 16,744,668 m^3.

3.2. Economical (Financial) Aspect

At first it should be noted that the whole evaluation process "uses" the pipe break rates and has nothing to do with the (doubtful) pipe break social cost, as this cost is only used to determine the optimum replacement time of a pipe. The latter does not affect the evaluation process. The evaluation concept is very easy as it compares the estimated costs and benefits (on monetary terms) related to the project implemented.

3.2.1. Costs vs. benefits

Regarding the direct costs related to the project, the five water pressure reduction devices cost 54,514€ (purchase: 17,814€, installation 36,700€), while their annual operation/repair cost is 10,000€. The total fee of the technical consultant was 1,003,670€. There was also an annual fee of 50.000€ for its supervising services. Since 2000, the local Water Utility "sold" 1.36 Mm^3 less water per year (9.52 Mm^3 in total), than it would have sold, if no water saving measures were taken. Considering a mean water price of 0.60 €/m^3, the lost revenues are estimated to be 5,712,000€. Finally the Public Awareness Campaign cost 126,000€ for 2000 and 50,000€ for each year forth. Concluding, the total cost of the water saving project was 7,556,184€.

Regarding the direct benefits related to the breaks prevented since 2000, each one costs 773€ (the social cost is not considered, in order the evaluation process to be as "socially" independent as possible). So, for the 5,324 breaks prevented, the Water Utility saved 4,115,452€. Water savings due to the breaks prevented since 2000 are 7,224,668 m^3. Considering the relative water supply/treatment costs (0.22€/m^3), the Water Utility saved 1,589,427€. Additionally, since 2000, the Utility sold 1.36 Mm^3 less water per year (9.52 Mm^3 in total), than it should have sold, if no measures were taken. Considering the water supply/treatment costs mentioned above, the Utility saved 2,094,400€. Concluding, the total benefits of the project were 7,799,279€.

3.2.2. Cost-benefit analysis

The Analysis (Figure 3, Table 2) proves that the project is feasible. It has a 7-years

Payback or Return of Investment period. Finally, there is the environmental profit involved along with the fact that the Utility will be able to postpone the search for additional water supplies and the respective costs.

Table 2. Cost – Benefit Analysis of the project.

	COSTS per year (€)						
	Water not being sold (m^3)	Lost revenues (water not sold)	PRV's-related costs	Technical Consultant Fee	Public awareness campaign cost	Total annual cost	Total project cost (€)
Unit values	1,360,000	0.60 €/m^3					
2000	1,360,000	816,000	54,514	1,003,670	126,000	2,000,184	2,000,184
2001	1,360,000	816,000	10,000	50,000	50,000	926,000	2,926,184
2002	1,360,000	816,000	10,000	50,000	50,000	926,000	3,852,184
2003	1,360,000	816,000	10,000	50,000	50,000	926,000	4,778,184
2004	1,360,000	816,000	10,000	50,000	50,000	926,000	5,704,184
2005	1,360,000	816,000	10,000	50,000	50,000	926,000	6,630,184
2006	1,360,000	816,000	10,000	50,000	50,000	926,000	7,556,184

	BENEFITS per year (€)							
	Breaks annually prevented	Cost not paid for breaks' repairs	Water savings-less breaks (m^3)	Money savings-less water used	Water not being sold (m^3)	Money savings-less water sold	Total annual benefit	Total project benefit (€)
Unit values		773€/break	1,357 m^3/break	0.22€/m^3	1,360,000	0.60 €/m^3		
2000	0	0.00	0.00	0.00	1,360,000	299,200	299,200	299,200
2001	323	249,679	438,311	96,428.42	1,360,000	299,200	645,307.42	944,507.42
2002	592	457,616	803,344	176,735.68	1,360,000	299,200	933,551.68	1,878,059.10
2003	821	634,633	1,114,097	245,101.34	1,360,000	299,200	1,178,934.34	3,056,993.44
2004	1,022	790,006	1,386,854	305,107.88	1,360,000	299,200	1,394,313.88	4,451,307.32
2005	1,202	929,146	1,631,114	358,845.08	1,360,000	299,200	1,587,191.08	6,038,498.4
2006	1,364	1,054,372	1,850,948	407,208.56	1,360,000	299,200	1,760,780.56	7,799,278.96

	NPV of the project (€)
2000	-1,700,984
2001	-1,981,676.58
2002	-1,974,124.90
2003	-1,721,190.56
2004	-1,252,876.68
2005	-591,685.60
2006	243,094.96

* all values are considered as present values

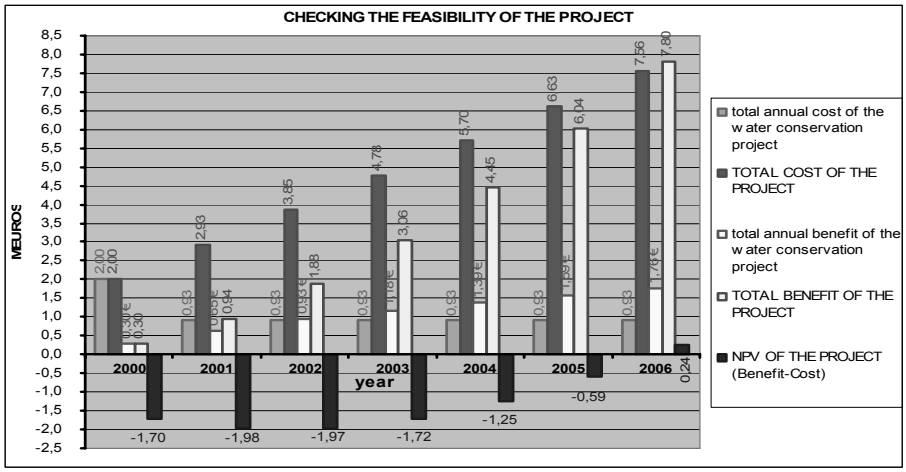


Figure 3. Checking the feasibility of the project.

4. CONCLUSIONS – SUGGESTIONS

The suggestions to the local Water Utility regarding the water losses are focused in prevention and reduction/minimization of their impacts. The pressure management project should be regularly updated following the changing needs of the network. It is important to form the quality standards of the constructions done by the customers themselves. Also crucial is to set the quality standards of the maintenance works regarding the supply mains. A lot of breaks were caused by third parties (service staff of other public utilities for energy/telecommunications). As the Water Utility's policy towards such failures is very loose, it is advised to set a significant fine-penalty so as to force third parties service staff become more careful. Last but not least is the better knowledge/management of the network that could be achieved through: a) installation of a SCADA system, providing continuous network data monitoring; b) improvement of the GIS system and; c) development of a hydraulic simulation model. All these can form an EDSS for Crisis Management. In order the Water Utility to suppress the water losses or minimize their impacts it is crucial to improve the level of alertness of its staff as well as to simplify the bureaucratic procedures followed nowadays when a private company is called for help to repair a break. The Water Utility should consider developing a mobile tool house so as to reduce the repairing time of a break and the water losses/money involved. Equally important is the implementation of new techniques regarding break repairs/services. Staff capacity building will prove to be lifesaving! If the Water Utility proceeds to the adoption of this Policy, it is more than certain that it will make its customers change their negative attitude towards its future plans. This is the most important benefit of all nowadays.

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Human-induced Water related Hazards and Cultural Heritage. An Overview

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Abstract: Water related hazards represent a serious multi-level threat with direct and indirect impact, periodic reappearance and complicated socio-economic profile. Apart from the natural water-related hazards (flooding, tsunami, tropical cyclone-driven storm surges, landslides, debris flow and soil liquefaction, temperature & moisture variations (periodic freezing, ice, heat and rain, existence of saline waters near the monuments), various human-induced hazards (construction of dams, loss of the wetlands, acid rain, weathering, sea-level rise / coastal changes and socio-economic factors related to water disasters) threat the cultural landscapes. The aim of this paper is to identify and categorise these potentially destructive factors in a flexible and comprehensible methodological framework, hat enhances public awareness and sensitivity towards the evaluation and salvation of the Landscapes of Memory.

Keywords: Cultural heritage, human-induced hazards, landscapes of memory, water-related disasters.

1. INTRODUCTION

The presence of water hazards was always been emphasised in the survival mentality of human communities, playing a crucial role in the transformation of human civilization. On the other hand, cultural landscapes are extremely vulnerable to the disasters related to water. Apart from the fact that these events are increasing, institutions and countries take action after damage has occurred. But rather than financing relief, it is essential to think of risk management as a coherent suite of actions including cultural assessment, prevention, monitoring, preparation, intervention and sustainable reconstruction. Although not all disasters can be avoided, preventive measures should play an important role and have proven to be cost effective. In order to increase the coping capacity of cultural heritage, and decrease its vulnerability to water hazards, policies, either in local, regional, national or

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international level, should be less dependent on environmental extremes. Detailed analyses of the cultural landscapes and integrated vulnerability assessment of the patrimony can function as a buffer action, in order to absorb natural or human-triggered shocks like floods, tsunami, pollution / contamination, sea-level rise and coastal changes. Water -related disasters take an enormous bill not only on human lives and economic losses, but also on environmental, social and cultural losses all over the world. The driven forces of water hazards may be generally categorised into three main groups: a) hydro-meteorological phenomena (HM), b) hydro-geological phenomena (HG) and c) anthropogenic causes (A). Their impact may also be assessed either as direct, or as indirect / long-term process. Although there are many methods of evaluation, categorisation and analysis of such phenomena, an archaeologist / heritage manager would probably like to know the main factor of risk and its spatio-temporal distribution. Thus, it would be wiser, for practical reasons only, if we could estimate the role of man in the appearance or aggravation of disastrous phenomena that destroy, degrade, mutilate or alter the cultural landscapes. The list presented below is not exhaustive, but rather explanatory and representative, in order to give various examples of cultural heritage undergone damage related to water phenomena.

2. WATER HAZARDS WITH HUMAN-INDUCED TRIGGERING MECHANISMS

2.1. Inundation / Flooding

Many areas of the world experience periodic inundation or are prone to flooding due to local geomorphological and hydroclimatic conditions. It is also evident that multiple secondary factors increase their vulnerability to flooding, growing population, denser occupancy of flood plains and other flood-prone areas, along with the expansion of unwise forms of watershed land use, being among them. Moreover, massive deforestation and urbanisation reduce water storage capacity and amplify flood waves. Noteworthy is the fact that dryness may turn to be a normal state to which humans have adapted, while floods strike unprepared populations suddenly. This category includes heavy rainfall cases and river or lake over flooding. Dam and sea-generated flooding is analysed as separated category for various methodological reasons. The study areas of Marathon & Brauron (E. Attica) that are under evaluation within DISMA sub-project (INTERREG IIIC -NOE framework), are separately presented by another paper in this session of the Conference(see also Tsakiris, 2006).

2.2. Sea-Level Rise (SLR) and Coastal Changes

Global or local event, SLR reflects the interaction between various parameters, such as eustatic or isostatic processes, tectonic mechanisms, local geological phenomena or accumulation processes, and characterises many stages in the geological history of

our planet (i.e. Ryan and Pittman, 2000; Bailey and Parkington, 1988; van Andel, 1989; Butzer, 1983). Coastal evolution after the Last Glaciation Maximum and especially during the Late Pleistocene and the Holocene, is related to the amelioration of the climatic conditions, the active sedimentary dynamics of marine, tidal, fluvial and lacustrine environments, extensive freshwater runoff, rise of vegetation cover and pedogenetic processes. Coastal flooding is also due to severe windstorms and tsunami.

The circum-Mediterranean area is an interesting case of changing coastal landscapes that are registered in the local geoarchives (i.e. Bintliff, 2002; Pirazzoli, 1991; Masters and Flemming, 1983). Ancient authors (Herodotus, Thucydides, Plato, Aristotle, Theophrast, Strabo, Pausanias, Titus Livius) had realised these coastal changes, their repeated transformative impact on human societies, but also the human factor (deforestation, urbanisation) that aggravates various hydrogeological phenomena like soil erosion and flooding. Coastal regression or transgression affected cities' prosperity and longevity. Well-known are the examples of ancient Mediterranean harbours like Piraeus, Thessaloniki, Ephesos, Myous / Miletus and Oiniadai (Mac Kil, 2004; Vött *et al.,* 2004).

On the other hand, the Giulia - Venezia region in Italy is under high social stress from flood hazards apart from the serious health hazards from long-term freshwater and coastal water pollution. Venice and its lagoon were made a UNESCO World Heritage Site in 1987. The entire city is an architectural masterpiece in which even the smallest building contains works by some of the world's greatest artists. Refugees from the Po Estuary fleeing foreign invaders, founded the city in the 5th Century AD.

In later centuries Venice became part of the Byzantine Empire, then an independent city-state and a major maritime power in its own right. Venice extends across 118 small islands in a marshy saltwater lagoon at the N.W. corner of the Adriatic Sea. A series of barrier islands protect this low-lying city from' normal' storm surges. But a correlation of differently triggered phenomena (subsidence, storm surge flooding, tidal flooding, SLR in the Adriatic Sea and oil gas extraction offshore, not too far from the Laguna Veneta) induced either by nature (local geomorphology, wind patterns, seas, moon) or by man himself, repeatedly aggravates the situation. In the famous flood of 1966 waters rose to almost two metres above sea level, and flooding while occurs regularly these days between September 2000 and March 2001 alone, more than 40 floods were recorded. As the time went by, the historic centre of Venice and the entire lagoon have become more and more exposed to flooding.

Today, with a tidal level of + 60 cm on the marigram of Punta della Salute, water begins to invade Piazza San Marco, one of the lowest areas of the city. With a tide of + 110 cm the 12% of the city is blocked with water and raised wooden walkways must be placed along established pedestrian. Thus, Venice provides an interesting example of excellent documentation and close interaction between man and the environment. Due to his geographical position close to the sea level, the city is extremely sensitive to climate changes and could, in the future, risk to be submerged (Enzie and Camuffo, 1995).

2.3. Construction of Dams

Large dams are constructed in river basins where people have lived for long periods of times, from the prehistoric past to the present They may also impact very large areas which encompass many cultural regions and tribal or indigenous groups There is a strong scientific consensus that the construction of dams has led to incalculable loss, destruction and damage of cultural resources, ranging from shrines of local communities to world heritage monuments. Cultural Heritage Management (CHM) should be a flexible methodological tool with internationally acceptable criteria and guidelines for an integrated assessment of the natural and cultural landscapes (World Commission on Dams, 2000). The famous case of the construction of the Aswan High Dam (1953 ff.) which radically changed our knowledge of Egyptian archaeology with the Nubia campaign, was vital for safeguarding Egypt against increasing water, energy and land demands. Similarly, the dam at Akosombo on the Volta River, which created the largest man-made lake in Africa (officially opened in 1966), was a catalyst for Ghanaian archaeology. On the other hand, over the last forty years the lands impacted by the Siberian hydroelectric projects (Ob, Yenisey, lena, Amur and other rivers & tributaries) have witnessed a tremendous amount of cultural protection and conservation, for all the kinds of monuments have been united into a single 'cultural and historical heritage' protected by the state. Nowadays, flood control and mitigation is normally a high priority under International River Basin Commissions in bilateral arrangements on trans-boundary rivers. Egypt and Sudan have a long-established cooperation agreement for preparedness and operational management of the Nile summer floods. In reverse, the controversial Three Gorges Projects (China), the largest reservoir project in the world, due to be completed by 2009, experienced lack of funding, shortage of trained personnel and serious problems related to administrative organisation, logistical requirements and political constraints. The lack of an overall research design and the failure to incorporate a sampling scheme based on issues of significance resulted in the damage and destruction of archaeological and historical sites, along with the total disfiguration of past cultural landscapes. Although the overall number of the reports and the identification of the sources made by Chen Shen (Royal Ontario Museum & University of Toronto, Canada)and his salvage team, are not available, this effort focused on the assessment of the degree to which the cultural heritage of the region is being properly protected.

2.4. Loss of the Wetlands

Wetlands are included in the Natural Heritage's list of every country in the world and are protected via the Ramshar Convention, but as Humans have transformed the landscapes and the ecosystems in which they live during their long history on the planet, wetlands and biotopes are experienced tremendous anthropogenic impact. Various forms of pollution and contamination affect the ecological equilibrium of the environments, their integrity, identity and value. Except from romantic, aesthetic and

cultural reasons, the protection of biodiversity / biotopes is a necessity for the future sustainable development of nations (Cylinder *et al.*, 2004; Dafis and Gerakis in Goulandris, 2004; Azous and Horner, 2001; Carlisle, 1998; Mitsch and Gosselink, 1993). Lands with shallow fresh, brackish or saline waters are characterised as wetlands, including the marshes, lakes, lagoons, rivers, streams, peat bogs, bushes, shallow coastal zones, mangroves, meadows covered by waters, estuaries, springs, salinas and rice fields. Potable water, water for irrigation needs, lush vegetation, abundance of game animals, increased food -gathering possibilities, neighbouring pasture lands, were the main reasons of why the first sedentary human societies preferred similar environments to live and reproduce. Myths and legends, customs and technical works related to the wetlands, all testify their role in human civilisation.

For thousands of years, lake Voiveis (Karla) - E. Thessaly was a paradise among the Eastern Mediterranean wetland environments. It was drained by 1962, after local farmers' request for more cultivated land. Human intervention solved some problems and caused others more serious. So, twenty years later, the children of those who asked for lake's drainage, wanted their wetland environments back. Its restoration began in 1999. The sustainable use of aquatic resources seems to be the only safe way to the maintenance of biodiversity and the long-term development of local economies. Within the framework of the technical works in the area, significant archaeological evidence came to light, 14 human settlements dated from the Late Neolithic Period onwards. Thus, excavational activities confirmed the philological evidence (Homer, Iliad B 711-3; Herodotus, VII.129; Euripides, Alcestis 588; Strabo, IX.5 *et al.*), along with this important project of environmental 'reconstruction'.

2.5. Erosion of the Monuments

Long-term changes affect the cultural sites and monuments in climates that experience interannual variability, in areas that are characterised by intense seismotectonic activity, in cases of uncontrolled human action or total abandonment of the cultural heritage. Temperature / humidity variations, periodic hydrometeorological effects (rain, snow, freeze-thaw cycles, moisturising winds), apart from air pollutants, UV radiation, salt crystallization and biological weathering interrelate each other in a vicious cycle of short- and long-term damage. These phenomena are often misregarded by heritage managers, even if they play crucial role in the overall status of the monuments, or by archaeologists who abandon the sites to their fate without permanent monitoring controls, iterative assessment processes and strategic plans for sustainable touristic development.

Weathering mechanisms of stone material (i.e. marbles and granites) that are exposed to marine environments, are a constant threat to the coastal / island archaeological sites. The monuments of Delos (Cyclades island - Aegean) are damaged by the dissolution and crystallization of salts, occurring in cycles (succession of transfers in liquid & gas phases), by a seaspray deposit mechanism, without taking into account biological degradation and hydrological or thermal

dilatation (Fitzner *et al.*, 2002; Chabas and Jeannette, 2001; Fitzner and Heinrichs, 1998; Fitzner *et al.*, 1997).

The case of the Diolkos of Corinth is another sad combination of human negligence and environmental degradation. The Diolkos was an 8-km long paved trackway across the Isthmus of Corinth, over which ships could be hauled overland between the Gulf of Corinth and the Saronic Gulf, to save them sailing around the Peloponnese. It has ruts with a gauge of 1.52 m cut for the trolleys onto which the ships were loaded, and has been called the world's first railway. First built probably by Periander (625-585 B.C.), it is mentioned by Thucydides as something ancient. Nowadays it has been superseded by the modern Corinth Canal. It is a monument of first-class importance for the history of technology, and for the Greek achievement, generally. The western end of the Diolkos, excavated between 1956-1962, lies today in a serious state of degradation. In 1985, the monument was already heavily eroded.

2.6. Acid Rain

Acid Rain, or more precisely acid precipitation, is the word used to describe rainfall that has a pH level of less than 5.6. This form of air pollution is currently a subject of great controversy because of its worldwide environmental damages. For the last ten years, this phenomenon has brought destruction in the United States, Canada and parts of Europe.

One serious anthropogenic threat for the monuments is related to the atmospheric circulation of SO_2, which is released from the combustion of sulphur-contained mineral fuel. Acid rain causes damage on the fluvial and lacustrine waters, on the soil and agriculture, as well as on the surface of the buildings. Acid rain contributes to the corrosion of metals and to the deterioration and soiling of stone and paint on buildings, statues and other structures of cultural significance. Limestone and marble turn to a crumbling substance called gypsum upon contact with acid. The damage inflicted on cultural objects is especially costly, since a loss of detail seriously depreciates the objects' value to society. They say that ancient buildings and sculptures in a number of cities have weathered more during the last 20 years than in the preceding 2.000 (available online at: http://pubs.usgs.gov/gip/acidrain/contents. html; http://www.epa. gov/acidrain). Many monuments and beautiful historic artefacts of great importance all over the world suffer from air pollution, Parthenon - Acropolis, Greece and Taj Mahal - Agra, India being among them. Another interesting case is also the Maya ruins in the Yukatan Peninsula - S.E. Mexico / Central America. The Maya monuments are being considered as a whole, Monuments to Mankind by UNESCO. However, there is great concern over the unwanted threat potential of acid rain on them Effects of acid deposition on monuments are well known and demonstrated either by natural weathering or by man made pollutants (Bravo *et al,* 2006; Bravo *et al.*, 1998; Camuffo, 1992).

2.7. Water Drilling

Water- drilling due to meteorological (lack of precipitation), hydrological (lack of water in streams and aquifers) or agricultural drought (when conditions are unable to sustain agricultural and livestock production) represents a serious long-term hazard for the cultural landscapes. Water tables are falling fast under the North China plain. In 1997, almost 100.000 wells were abandoned but 221.900 new wells were drilled (Brown 2001). Analogous desperate quest for water is observed in Mesogeia Plain (Attica - Greece). Similar activities threat buried antiquities, by damaging the stratigraphies (therefore future excavation processes) and the protection of items from unprepared exposure to atmospheric phenomena and from illegal exchange.

2.8. Socio-Economic Side-Effects

Water hazards and water-related disasters cause various damages on the agricultural production, the local infrastructures and the communications network, apart from the bioclimatic changes (i.e. spread of diseases such as malaria), the food crises, the war conflicts and the general societal upheaval. All the above-said results make the cultural landscapes to suffer, because the sites and the monuments are destroyed or severely damaged, the touristic exploitation is stopped, the communication network is chaotic, the scientific research is blocked, the heritage is vulnerable to illegal activities, political / religious / social / commercial restrictions and constraints are imposed and the agenda of the governments / international organisms finds no room for cultural issues. In addition, the displacement of people who are directly or indirectly affected by those hazards break the linkage between local populations and their cultural roots, hurting the plurality of identities, cultures and memories.

3. CONCLUSIONS

Water hazards with human-induced triggering mechanisms (e.g. flooding, sea-level rise and coastal changes, construction of dams, loss of the wetlands, erosion of the monuments, acid rain, water drilling, socio-economic side-effects) have a significant share in risk profiles of various geopolitical systems around the world. On the other hand, marine and coastal environments (lagoons, river deltas, mangrove landscapes, dunes, fluvial routes, wetlands, islands, shorelines), apart from their natural and cultural significance, are also rich in archaeoenvironmental information, which is vital for a huge spectrum of scientists. Another important aspect is that parts of many contemporary seas were dry land during Palaeolithic and Mesolithic times (until LGM) and were then inhabited. Many remains of these habitations are preserved in the sediments of the seabed. These underwater settlement sites are unique in an international context. Obviously, all these scientific treasures call for a strong protection. Our wish is that global visions and strategies should be coordinated with

the awareness and preparedness of local communities in order to preserve the landscapes of memory for the generations to come, without isolating the natural changes from other environmental and societal influences.

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Assessing Vulnerability in Cultural Landscapes

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Abstract: The topic of the cultural heritage management was not generally regarded by governments as a high priority till recently, so it will come as no surprise that it has not been subjected to much in-depth study or analysis. This paper focuses on the elaboration of an integrated methodological framework that could assess vulnerability's parameters concerning various categories of cultural heritage. Case studies have shown that the evaluation and analysis of cultural landscapes may be significantly improved, when different socio-economic, environmental and technical patterns are explicitly organised into grouped questionnaires. The four-part analysis (IESO) provides flexible criteria for hazard assessment of cultural targets (memory institutions, open-air sites & monuments, elements of maritime heritage, natural landscapes, caves with paleontological/ archaeological interest, landscapes of memory), and is built on the following scheme: a) Intrinsic parameters (describing the condition of the cultural asset), b) Environmental parameters (describing the natural setting), c) Socio-economic parameters (describing the living community) and d) Organisational / Institutional parameters (describing various functions of the State).

Keywords: Cultural heritage, Eastern Attica, flood hazard, vulnerability assessment.

1. INTRODUCTION

Although "the right to a cultural heritage is an integral element of humanity" (Article 27 of the United Nations' Universal Declaration of Human Rights) and "the diversity of such resources is essential for sustaining the ability to cope with the past, present and future", the colossal magnitude of the loss and damage of cultural heritage due to various worldwide natural phenomena and human-induced hazards diminishes the pool of knowledge and wisdom from which we draw our strength and resilience. Unfortunately, in the majority of cases, we cannot rehabilitate or restore what has been lost, but we can prevent a further loss of cultural heritage by*:* a) the integration Cultural Heritage Management (CHM) assessment with environmental assessment to elucidate long-term interactions between living and past populations and their

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environments, b) the elaboration of a more synergetic, effective, satisfactory and internationally accepted legislation and policy, c) the incorporation of costs for CHM into national budgets, d) the involvement of direct and active participation of local communities in all stages of CHM, e) the enforcement of public education, f) the coordination of international efforts to secure funding and ensure compliance with international legislation and guidelines, as well as compliance with professional standards.

2. WHERE TO BEGIN? DEFINING TERMS AND CONCEPTS

2.1. Cultural Heritage

As cultural heritage can be assigned any kind of evidence related to human action, any 'product' of human creativeness and expression, widely accepted for its scientific, historic, artistic and anthropological value. On the other hand, natural landscapes are also included in the lists of patrimony objects that must be protected. The international meetings can provide all the terms needed for further analysis (e.g. UNESCO General Conference, 17 October - 21 November 1972, Paris -Convention for the Protection of the World Cultural and Natural Heritage, NATURA 2000 network, Directive 92/43/EEC, Rio Convention, 1992 etc.).

Natural features (physical or biological formations), geological and physiographical formations, natural sites and protected natural areas (marine parks, national parks, aesthetic forests, protected monuments of nature, game reserves and hunting reserves, eco-development areas), along with the four types of biodiversity (genetic, species, habitat, landscape), are unified under the umbrella of this category.

On the other hand, cultural landscapes include places, features, objects, memories and perceptions related either to natural or man-made environments, ranging from those that are lost or 'mythical', to those with numerous surviving features. Some are living landscapes, but their usage has altered them considerably, while others are largely unchanged. Sometimes, 'fossil landscapes' (e.g. Pompei, shipwrecks on the sea floor of Black Sea) are unusually well preserved due to various environmental conditions or geological / physical processes.

Monuments, caves of archaeological interest, groups of buildings, archaeological sites (open air areas, subterranean, submarine or coastal), mobile objects, archival material, scientific works, paleontological & paleoanthropological remains, industrial sites and landscapes of memory (e.g. languages, oral traditions, sacred and mythical landscapes), museums and collections, all are prone to diverse water hazards, the impacts of which can demand extremely expensive restoration programmes.

Finally, the category of the archaeological remnants underneath the surface of earth or water, usually being transformed into geological features (e.g. buried sites under river courses, cultivated lands, estuaries, layers of various sediments, e.t.c.) cry out for their complete protection from all kind of physical disturbance.

2.2. Hazard

Torrential rainfalls, overflooding and other water hazards are among the environmental activity that occur in Nature, independently of the human presence on Earth, even if they are considered as negative inputs of ecosystems' stability. But, the unexpected or uncontrolled / inevitable natural events of unusual magnitude, that threaten the life and activities of humans, have also some special characteristics*:* a) reform the natural and cultural landscapes, b) intensify the degradation's processes, especially when human factors play a prominent role, c) may provoke a broad spectrum of losses within human society (http://www.naturalhazards.org/discover/index.html· http://www.unesco.org/science/earth/disaster/about_disaster.shtml· Burton *et al.*, 1978).

Concequently, as hazard may be defined 'every source of potential harm, every situation with the potential to cause damage, and every threat or condition with the potential to create loss of lives or to initiate any failure to natural, modified or human systems'. The causes of water hazards may be external (e.g. flooding) or internal (e.g. defective section of levees), and their triggering mechanisms may be natural (e.g. storm event) or man-made (e.g. deforestation) (Tsakiris, 2006).

2.3. Vulnerability

The terms Vulnerability, Resilience and Adaptive Capacity, are relevant in the biophysical realm as well as in the social realm. In addition, they are widely used by the life sciences and social sciences with different foci and often with different meanings, blocking the communication across disciplines. Depending on the research area, Vulnerability's concept has been applied exclusively to the societal subsystem, to the ecological, natural, or biophysical subsystem, or to the coupled socio-economic systems (SES), variously referred also as target system, unit exposed, or system of reference. Vulnerability, according to Adger (2006) is most often conceptualized as being constituted by components that include exposure to multi-scaled perturbations or external stresses, sensitivity to perturbation, and the capacity to adapt. Vulnerability is also thought of as a susceptibility to harm, a potential for a change or transformation of the system when confronted with a perturbation, rather than as the outcome of this confrontation (Gallopín, 2006). A system (i.e., a city, a human community, an ecosystem) may be very vulnerable to a certain perturbation, but persists without problems insofar as it is not exposed to it. Respectively, the vulnerability to water related hazards includes three interdependent parameters (exposure to stress, high intensity of hazard and limited coping capacity. Although measuring vulnerability is a difficult task, the need for its assessment is obligatory. In an attempt to propose a simplified procedure, Tsakiris (2006) presented a component approach corresponding to economic, environmental, social and patrimonial damages. For the economic component, a function between 0 and 1 has been proposed.

3. VULNERABILITY ASSESSMENT OF CULTURAL HERITAGE

We accept that cultural landscapes represent systems. So, we define ther vulnerability as the degreee of susceptibility to damage from hazardous water related phenomena (Tsakiris, 2006). However, in the case of cultural heritage, vulnerability can not be analysed referring to the entire system, but it is necessary to disaggregate the system into a number of components and perform a detailed analysis on each of them. Consequently, the vulnerability of patrimony, based, initially, on various methodological analyses of social indicators for measuring community's vulnerability to natural or technological hazards (i.e. Krumpe 2006; Dwyer *et al.*, 2004; Pelling, 2003; Heijmans, 2001; Buckle, 2000; King and MacGregor, 2000; Crichtonl, 1999; Morrow, 1999; Buckle, 1998; Cobb and Rixford, 1998; Jasanoff, 1998; Neuman, 1997; Buckle, 1995; Firschhof *et al*, 1978; Andrews and Withery, 1976), should be studied according to a four-part analysis (IESO): a) Intrinsic parameters (describing the condition of the cultural asset), b) Environmental parameters (describing the natural setting), c) Socio-economic parameters (describing the living community) and d) Organizational / Institutional parameters (describing various structures & functions of the State). Although a further analysis with ranked questionnaires is presented in the relevant reports of the sub-project DISMA, a brief but explicitly- structured presentation is also necessary for further discussion and improvement. A number of elements contribute to the understanding of the conservation requirements of heritage assets. The process is the following:

a. Intrinsic parameters. The general condition of the cultural asset may be extracted from the data that identify the condition and the integrity of heritage assets. These data include indicators for recovery, context within which heritage items function, operability level of the cultural asset, integrity of the asset and carrying capacity of the cultural target: i. coordinates, ii. extended area of the cultural site / dimensions of the monuments, iii.altimeter of the site asl, iv. distance of the site from the nearest coast, v. ground relief, vi. kind of the open-air cultural unit, vii. chronology of the cultural asset, viii. future excavation / other intervening activities, ix. kind and value of finds, x. 'manager' of the site, xi. running programmes (restoration, excavation, e.t.c.), xii. participation in technical and other development projects, xiii. previous technical interventions on the site, xiv. accessibility in case of hazard, xv. capacity of receiving visitors, xvi. number of visitors per years, xvii. medical assistance inside or near the site, xviii. vicinity to other cultural sites, xix. vicinity to modern settlements / other touristic destinations, xx. existence of touristic facilities.

b. Environmental parameters. The natural setting should be identified in the best possible way by an interdisciplinary scientific team, in order to describe the pressure experienced by the asset and the living community in which this heritage belongs or exists. The data that may contribute to a better understanding of the

spatio-temporal distribution of water hazards within the cultural landscapes, include: i. predictability of the hazard, ii. certainty of the hazard (Descriptors of standard AS/NZS 4360: 1999 + World Heritage Convention Criteria + Disaster Management Planning for Archaeological Archives*:* IFA 2004), iii. duration of exposure to the hazard, iv. periodicity of exposure to the hazard, v. reversibility of the hazard, vi. other existing hazards or factors of magnification, vii. severity of consequences in case of damage (referring to the cultural unit), viii. destruction level in case of damage, ix. definition of risk level (Model 1 = destruction level + certainty of hazard), x. definition of risk level (Model 2 = magnitude + frequency of hazard), xii. detection of past catastrophes.

c. **Socio-economic parameters.** The living communities play a significant role, too, in the perception, assessment and management of hazards, either referring to contemporary landscapes or to heritage ones. Information that describes the range & significance of conservation values of heritage assets includes: i. rarity of a heritage asset, ii. originality of a heritage asset (duration of use) iii. existence of legal, economic, national / regional or other response framework in case of hazard, iv. elaboration of economic assessment concerning the damage profile of cultural asset, v. awareness of the social side-effects after a damage on cultural patrimony, vi. awareness of the environmental side-effects after a damage on cultural patrimony, vii. awareness of the technical level (infrastructures, personnel, disaster plan, disaster simulation techniques) for intervention in case of damage on cultural patrimony, viii. awareness of the cultural structure of the community, within which the cultural asset exists, its function / role and significance, ix. awareness of the scientific 'value' of the cultural asset, its role and potentiality in the current or future research, x. awareness of the aesthetic / artistic 'value' of the cultural asset, its role and significance in the modern landscapes, xi. awareness of the economic 'value' of the cultural asset, its role and potentiality in the current or future development planning (i.e. eco-tourism, archaeo- tourism, contemporary cultural happenings), xii. overall prioritization level for rescue in case of hazard /damage (what to save first and why).

d. **Organizational / Institutional parameters.** The afore-mentioned preparedness' framework should be coordinated by the authorities, local, national or international, in an explicitly - defined scheme, that takes under consideration the management options and the expectations available, in order to build a long-term, flexible and resourceful response to any possible hazard threatening the cultural landscapes.The estimation of this situation is further analytical and examines: i. whether an asset is on a protective listing, ii. whether an asset is subject to agreed agency standard operating procedures, iii. whether an asset receives recurrent conservation funding, iv. whether an asset is subject to asset management planning guidance, v. whether knowledge of an identified asset is subject to a standard inventory, regularly updated and maintained, vi. whether an asset is subject to an assessment criteria and process consistent with industry standards, and endorsed at senior management level, vii. whether an asset is subject to a risk management strategy addressing the

threat to a place by in-house staff, viii. whether an asset undergoing conservation is subject to conservation management planning guidance, tied to the relevant ICOMOS (International Council on Monuments and Sites) charter, ix. whether management of an asset is subject to cyclical maintenance plan works, x. whether an asset is subject to an agreed process of presentation directed by conservation heritage values, xi. whether an asset is subject to a monitoring regime integrated with asset management planning requirements and predetermined performance measures. Consistent with the four-part systematic analysis of the cultural landscapes, an overall assessment of their vulnerability is rather unappropriate and should be avoided. Instead of it, we suggest a four-part vulnerability assessment for each group of the afore-said parameters through a filtering proceess of the answers, according to which the level of vulnerability profile could be rated as: 1. High, 2. Moderate or 3. Low. In the case of cultural heritage, though, we can not proceed further into a unification of vulnerability's sublevels, because each group of questionnaires present different 'entities' that are equally present but not equally expressed and measured.

4. THE CASE STUDY OF EASTERN ATTICA

DISMA sub-project is a pilot case study carried out by Eastern Attica Prefecture in collaboration with the Centre for the Assessment of Natural Hazards and Proactive Planning (CANHA)- NTUA, within INTERREG IIIC -NOE framework. The areas of Marathon and Brauron (Attica, Greece) that are under evaluation, experience iterative phenomena of flooding that are aggravated by the lack of sufficient infrastructures, the overpopulation expansion, the extensive agricultural land patterns, and other local environmental factors (e.g. geomorphology and climate conditions that favor the formation of marshy lowlands, the transgression of the sea, high sedimentation rates due to riverine alluvia) that trigger these events since early Antiquity. In the area of Marathon, Charadros river, notorious from theClassical Period, has a steady annual water runoff, characteristic that triggered the formation of the big marshy lake in the plain of Nea Makri (Pausanias, I.32.7). The transgression of the marshes in the area was the main cause for the spreading of malaria, an endemic disease in the archaeoenvironments of Eastern Mediterranean, disease with hardly recognisable overall impact on the population rates in the ancient world. Equally notorious for his destructive action, was another river, Erasinos, in the area of Brauron (Strabo, VII.5.VII-VIII cap. 371). The life of the famous temple irrevocably stopped during the 3rd cent. B.C., when extended flooding from the river brought so much alluvial deposits, that the whole geomorphology of the area changed dramatically. Insofar, the cultural landscapes of the above-mentioned areas seem to take a high score in each of the vulnerability's assessemnt groups, proving that societal, economic, geographical and environmental parameters are interrelated to each other.

5. CONCLUSIONS

The magnitude of water related hazards, the rapid environmental and geopolitical changes, as well as the inner character of the cultural heritage which loses its initial coherence through time, make it extremely vulnerable to many spatio-temporal variations. The management of cultural heritage presupposes the elaboration of an integrated, flexible and innovative methodology, which would be able to analyse and evaluate all the interrelated parameters in order to reduce the vulnerability of the patrimony in face of water related hazards. The IESO systematic framework provides a satisfactory analytical tool that is suitable for the assessment of vulnerability in complex socio-economic, geographic and cultural units, within which patrimonial assets coexist today.

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Water Resources CyberManagement of Temporary Rivers

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Abstract: Environmental cyber-infrastructure is applied for the acquisition of water quality data, real-time assessing of data quality, use of data as inputs to water quality models and posts the results on the internet in a real time environment. Physical and chemical quality sensors, data and access nodes for real time measurement of gage height, water temperature, conductivity, pH, dissolved oxygen, turbidity, nitrate, sediment temperature and moisture have been installed. Methods are being developed for data mining. Data are being assessed, stored and used in water quality models.

Keywords: Cyber-infrastructure, real time monitoring, temporary rivers, water resources management.

1. INTRODUCTION

The Mediterranean region is characterized by its own micro-climate and a clear separation of seasons that is characterized by water deficits. Unique features of the geomorphological regime of the region are the temporary rivers. Temporary is a general term for all intermittent (streams that carry water a considerable portion of the time, but that cease to flow occasionally or seasonally because transmission losses and evapotranspiration exceed the available water supply), ephemeral (the stream channel that carries water only during and immediately after periods of rainfall or snowmelt) and episodic streams (the stream channel that carries water only during very short periods (hours) of heavy rainfall) (Uys *et al.*, 1997). Thirty percent of the surface water bodies world wide can be characterized as temporary waters. This number will increase in the future as a consequence of human impacts such as climate change and water demands. In Greece, it has been estimated that at least 42% of the

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Greek territory is comprised of temporary rivers (Tzoraki *et al.*, 2006) and as a whole in the south Mediterranean terrestrial area temporary rivers comprise about 26% of the area (Froebrich *et al.*, 2002). The main characteristic of temporary streams is the alternation between dry and wet periods. During the dry period, there is lack of water due to different factors such as: droughts, extensive evapotranspiration, water abstraction, high irrigation demand, overexploitation of groundwater and large vertical water exchange (e.g. downwelling into alluvial or karstic aquifer). With local climate, geology and topography driving ephemerality at the local level and regional climate gradients, catchment area, abstraction and agricultural and waste return flow influencing regional river ephemerality (Tzoraki and Nikolaidis, 2006). Future increases in the demand for water and increased climatic extremes are likely to add further pressure to stream ephemerality and quality, with increased upstream abstractions reducing flow and downstream discharges enhancing nutrient load.

Environmental monitoring is not a new research field. Up until recently, it was mostly based upon loggers which recorded data at specific intervals and required manual downloading by a maintenance team. The introduction of wireless technology in environmental sensing has tremendous benefits, and environmental scientists are becoming increasingly aware of its potential. A wireless sensor system is specifically designed to transmit data from an array of sensor probes to a data repository with minimum effort and latency. The objective of this research is the development and implementation of an environmental cyber-infrastructure for the acquisition of water quality data, real-time assessing of data quality, use of data as inputs to water quality models (such as AQUATOX, HSPF or RS-tempQ models (Tzoraki *et al.*, 2007)) and posting the results on the internet in a real time environment. This work addresses the inability of current technology to capture environmental information when and where problems arise, and integrate them in a high performance computing environment where they can be used in detailed process modeling, development of predictive models, and, ultimately, in driving decisions about detection, prediction, and response to environmental change and societal protection. There are two parallel applications, one in the US at the Iowa River and the other in Crete at the Koiliaris River. The main goal is the improvement of system ability to forecast (in near real-time) critical events that transport water, sediment, and nutrients at the catchment scale in Koiliaris river in the Municipality of Armenon, Chania, Crete. The preliminary results from the Greek site are reported here.

2. METHODOLOGY

The overall methodology used in this research is the following:

1. **Development of Cyber-infrastructure:** Physical and chemical quality sensors, data and access nodes for real time measurement of gage height (InSitu Inc., Aqua Troll 200), water temperature(UTECO, Pt 100, conductivity (Hanna, HI3001), pH (Hanna, HI1000), dissolved oxygen (Oxyguard, model 420),

turbidity (Global Water, WQ710), nitrate (pHionics, NO3140), ammonia (pHionics, NH3020), chloride ((pHionics, Cl3060), sediment temperature and sediment moisture(DELTA-T,SM200) have been installed at the gauging station.

2. **Development of Data Mining Tools:** Even-though real-time water quality measurements are possible, there is a need for development of methods to correct the drifting of probes with time. We will estimate the drifting with time and develop correction methods and guidelines for use of the probes.
3. **Modeling and Systems Analysis:** The acquired data will be used to drive the simulation of the selected water quality model such as AQUATOX or RS-tempQ models. Automated techniques will be used for the calibration and real-time simulation of the model. The results will be projected in the web page as a way of communication with the public.
4. **Development of Environmental Management Strategies:** We will develop management strategies and utilizes mathematical models to evaluate the impacts of management strategies on water and ecological quality.
5. **Decision Making Process and Involvement of Stakeholders:** We have developed a web site (www.koiliaris.tuc.gr) that reports the real time results. We will actively involve the stakeholders in the region. The transparency of data and water needs will provide the framework for decision making.

3. DEVELOPMENT OF THE CYBER-INFRASTRUCTURE

The implementation of an efficient and reliable real – time system for in-situ monitoring of environmental and ecosystem processes imposes some critical issues that need to be addressed. Recent advancements in wireless communications and electronics have enabled the development of low cost highly sophisticated sensor networks for data acquisition, analysis and storage. In such a system there are many constraints that need to be dealt.

Network Communication: One of the key features of sensor networks is the communication layer. In the sensor network of our gauging station deployed in an isolated uninhabited area, we are referring to wireless communication. Communication demand depends on the level of data processing and control in the nodes, but the communication layer must be able to cope with high and even burst data (worst-case) transfers, especially when real-time tasks are involved.

Energy efficiency: Since we are referring to autonomous sensor nodes, each node must have the ability to maintain balanced (if not minimal) energy consumption. Obviously, the power consumption of a system is closely related to its computational task (Schmitz *et al.*, 2001).

System deployment: The sensor nodes must present low environmental disturbance. They must blend into the environment. This may not always be an easy task since the miniaturization of subsystems has to be balanced: the limiting factors are still battery size and radio power requirements (Martinez et al, 2004).

Scalability: The system must be able to integrate and cope with a variety of sensor probes and not just be limited to a fixed number of sensors. Sensors may need to be added. Of course there should be an upper limit to the number of sensor probes that a

remote node can cope with since the deployment of a large number of probes can have a major impact on various critical subsystems of the node, like power consumption and communication bandwidth.

Remote Management: System maintenance and thorough checks can not be performed on a regular basis, especially if the sensor nodes are deployed in isolated locations. For this reason, the system must be able to provide status indications through remote management. The developed cyber-infrastructure can be partitioned into the following major subsystems: the data collection platform (sink node), the autonomous sensor node, and the base station (Figure 1).

Sink Node (Data collection platform): The data collection platform doubles as a data logger collecting measurements from the probes, physically attached to it and from the remote sensor node, and as a communication medium for transmitting these data to the base station server in the lab upon request.

- *Data logger:* The Delta-T DL2e Logger is a programmable data logging device, capable of taking readings and storing data from a wide variety of sensors. It is independently powered, capable of operating under wet conditions, and at high and low temperatures. It has an RS232 interface with the communication subsystem. The data-logger (with the use of its built-in real-time clock) initiates data retrieval from the sensors periodically, every 10 minutes and as a further process, it estimates the mean value of these measurements on an hourly basis. These values are then stored in a 128K built-in RAM. It may seem that the memory size of the data logger is rather small but by taking into account that (a) the amount of data stored are only a few tens of bytes per day and (b) data is transmitted to the base station on a daily basis, we have estimated that the system can continue the data collection and storing process even in case of communication failure and thus isolation, for a whole year.

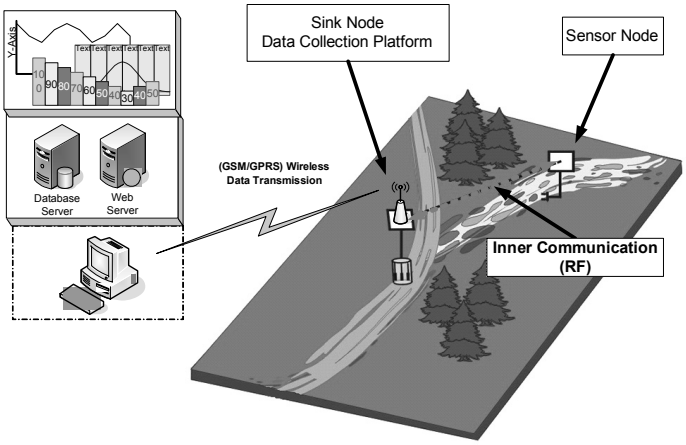


Figure 1. Schematic representation of the proposed cyber-infrastructure design.

- *Sink – Node Inner-Communication subsystem:* The communication between the sink node and the sensor node via RF transceivers with omnidirectional antennas. This is ideal for short – range communications. The sink, issues control or data requests to the remote node. The sink node is responsible for maintaining a fault

– tolerant network. Fault tolerance is the ability to sustain network functionalities without any interruptions due to sensor node failures. The sensor node may fail or be blocked due to lack of power, have physical damage or interference. It is the sink's role to keep the reliability of the cyber-infrastructure network by excluding the affected node from the network.

- *Sink – Base Communication subsystem:* (b) The communication between the base station and the data collection platform. This is performed through a GSM (Global System for Mobile communications) modem (Wavecom, M2M modem Fastrack GSM/GPRS M1306B). The base station sends a request to the sink node to commence the data acquisition process. The communication system forwards this request to the data logger, through a serial interface. As a result, the data logger starts sending the stored data to the base station in the lab. GSM is an open, digital cellular technology used for transmitting mobile voice and data services. The reason for selecting a GSM service instead of GPRS is mainly because GSM is circuit switched. When the base station sends a request for data retrieval, a channel is reserved (just like for voice communication) and data is transmitted. This channel remains active only for the duration of the data exchange process. It then remains idle, consuming minimum energy. GPRS, on the other hand, is "always on" and packet switched. The communication baud rate between the base station and the sink node is set at 9600 bps.
- *Power Management:* The data collection platform is equipped with a 12V rechargeable lead battery. The battery is recharged by a small solar panel (Shell, ST20). Even with the presence of a solar panel, the need for maintaining a strict low-power consumption policy is imperative. As a safety feature, the data logger is equipped with a backup battery (6 internal AA alkaline cells) which enables it to retain information stored in its memory in the absence of other power supplies (i.e. power failure) for approximately 24 hours continuous operation/readings (500,000 readings) using the keypad/LCD or RS-232interface, or 12 months' quiescent operation.

Sensor Node: Most of the sensor probes are physically attached to the data collection platform but others are placed in the nearby area. These distant probes are controlled by the sensor node equipped with its own power unit and communication module (RF transceiver). It isn't equipped with a data logging device since it acts as a slave in respect to the sink node. If the sink node requests new measurements, the sensor node starts gathering data from its attached probes and directly forwards them.

Base Station: It is actually a pc (in the laboratory vicinity) with a twofold server role. It acts as a database and web server for storing and publishing the environmental measurements respectively. The data storage procedure is initiated by the base station (as mentioned earlier). This is an automated process and is performed once per day without any user intervention. This scheduled process performs a call to the GSM modem in the sink node. After the communication is established, the base station receives the transmitted data measurements. In case of an error in the communication phase, the base station is programmed to perform two additional attempts to "talk" to the remote sink node. This is very often the case since during wet and windy conditions the communication link can be quite unstable. Thereafter, the received data are analyzed (model & system analysis), categorized and classified with the use of an open source database server (MySQL). A web server (open-source apache HTTP) is

installed in the base station. The deployed website contains custom server-side scripts (developed in PHP) to allow dynamic web contents of the data stored in the database.

4. DATA MINING: PRELIMINARY RESULTS

The installation of station was deployed on the 4 of October, 2006. Field campaigns were conducted during 14-30/11/2006 and 8/2-29/3/07 which represent late autumn and winter periods respectively (Figure 2). During the field campaigns, the telemetric station sensors were calibrated and field samples were collected for independent laboratory analysis. In the first phase, the water temperature, pH, D.O., Conductivity and turbidity sensors were deployed and in the second phase the sensors of nitrate, ammonia and chloride. For the verification of station measurements the pH, temperature, D.O., and conductivity were measured independently in the field using the following instruments: Orion 9107 pH meter, Orion 081010 D.O. meter, and Orion 011050 Conductivity meter. Water samples were obtained and were analyzed in the lab for nitrate, ammonia and chloride using spectrophotometric techniques.



Figure 2. Koiliaris River telemetric monitoring station.

During the deployment periods, the weather was dry for the first period and successively dry to wet and vice versa for the second period, Temperature was varied from 12.9 to 15 °C. The flow of the river was comprised primarily from the Stylos

karstic springs with small additions from groundwater originating from the valleys alluvial deposits. Figure 3 presents the telemetric water quality parameters data and their comparison to laboratory measurements. Sensors exhibited four different operating modes, that is, stability with minor drifting (pH, D.O. and conductivity), small adjustable drifting (nitrate probe), major drifting not adjustable (ammonium and chloride) and finally inoperable mode (turbidity). The pH probe functioned properly and it was confirmed by laboratory measurements (Figure 3c).The pH of the water was very stable ranging between 8.05 and 7.95 for the first deployment period Increases in the pH was observed after a strong rainfall at 24/2/07 (Figure 3a). The karstic spring pH was higher than the groundwater pH (value and date measured) supporting in this way the above argument. The conductivity probe exhibited steady function (Figure 3d). Conductivity field measurements showed lower values for the same period the pH decreased (at 24/2/07). Conductivity measurements for groundwater exhibited higher values and it supports the observation that lower conductivity values are associated with rainfall and successive karstic supply of the river. The D.O. measurements were fairly constant and oversaturated by 10%. The comparison between the theoretical dissolved oxygen saturation concentration and the station data suggests that air temperature is the main driving force and that primary production plays a secondary role during daytime. This is also supported by the fact that no-bio-fouling was observed on the probes during deployment. The RMSE (Root Mean Square Error) between the probes and field measurement were 0.5 for pH, 0.8 mg/L for D.O. and 31 mV for Conductivity. The nitrate probe (RMSE of 0.3 mg/L for nitrate-nitrogen showed fairly stable function (Figure 3e). Small drifting was adjusted according to calibration curves obtained in successive field measurements. The karstic spring nitrate was lower than the groundwater nitrate due to agricultural land fertilizer impacts. The chloride and ammonium probes exhibited major instability (Figure 3f). Long term deployment is necessary to identify drifting patterns in the probes.

5. CONCLUSION – FUTURE WORK

In the first phase of this work, a telemetric water quality station was deployed successfully that was designed to measure parameters that are appropriate for the evaluation of temporary rivers. Future work will address the development of:

- Reconfigurable resources (FPGAs) can provide field programmable capabilities by introducing high speed and efficiency in computationally demanding tasks, evolving in this way the sensor nodes from plain data loggers to intelligent, autonomous, decision – making systems (Efstathiou *et al.*, 2006).
- Procedures for automated data mining methodologies that will assess remote data, correct drifting, and provide information on probe failure.
- Automated techniques for real-time simulation of watershed models.

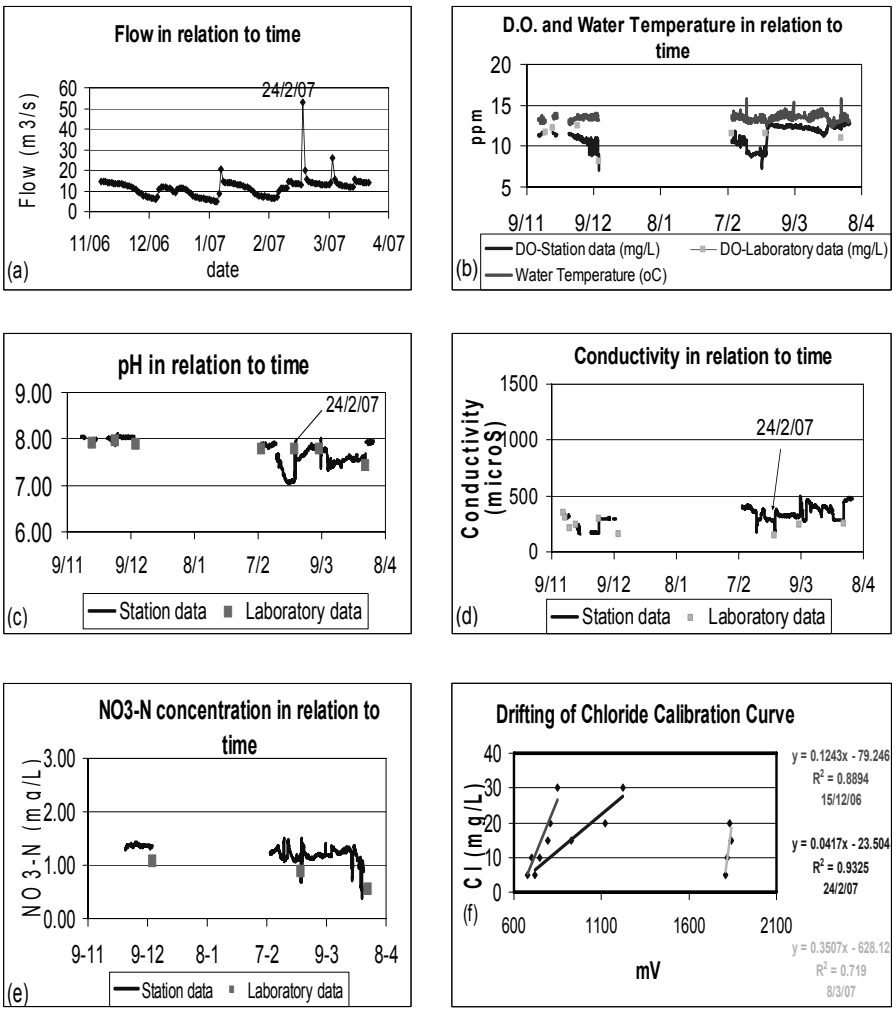


Figure 3. Comparison of field to lab measurements of water quality parameters.

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The Role of Remote Sensing in Water Balance Modeling in Small Watersheds

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Abstract: Application and intercomparison of three deterministic water balance models for the estimation of monthly runoff in a small rural watershed is presented. The three models are: Abulohom, GR2M and Xiong-Guo. In first approach, monthly regional precipitation and mean monthly air temperature derived from the meteorological stations of the region, as well as potential evapotranspiration based on NOAA/ AVHRR satellite data, are used as inputs to the models. In the second approach, the air temperature derived from NOAA/ AVHRR land surface temperature (LST) satellite images is used. Time series of the above parameters for 13 years (1981-1993) are employed; a part is used for model calibration (1981-1988) and the remaining data (1989-1993) for verification. The models are applied at Portaikos river basin of Thessaly, upstream of Pili site, covering 132 km^2. The results refer to simulated runoff for the same time period. Also, a number of statistical parameters for the intercomparison between employed models are used and show satisfactory results.

Keywords: Blaney-Criddle, remote sensing, water balance models.

1. INTRODUCTION

After more than 40 years of classical hydrological model development, there is an increasing trend in hydrology to use new tools, which provide distributed information of the catchments characteristics. Hydrological models must be reliable and robust as these qualities influence all applications based on model output. The hydrological response of watershed area depends on rain characteristics, on initial moisture conditions and also on landscape characteristics such as topography, soil, geology, land cover and hydrography. The most basic modelling approaches use lumped models but tend to over simplify the watershed heterogeneity (Christodoulaki *et al,* 2003). Simple deterministic water balance models, parametric or empirical, which unify the heterogeneous characteristics of the watersheds and simulate the rainfall-runoff process, are, in general, easy to be used and require a number of inputs (Tsakiris, 1995; Dalezios, 1987).

The first water balance model using monthly data was developed in the 1940s, by

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Thornthwaite (Thornthwaite, 1948). Since then deterministic models have been developed and applied in a large spectrum of hydrological problems. In recent years, such models have been applied in order to find out the consequences of climate change, and the long-term runoff forecasting. Although, in these applications hourly or daily hydrological models are used, these models require significant data and use many more parameters as compared to the monthly models. (Hondronikou, 2004; Hondronikou, 2006). In many hydrological problems a remarkable effort has been made for the development of new models and techniques, using monthly data. Such models focus on their complexity of the algorithms and the number of their parameters. Although a long experience has been acquired through many models, there is always a need for validation and verification of these models for regionalization their applicability in other regions. (Hondronikou *et al.*, 2005; Hondronikou *et al.*, 2006).

The aim of this paper is to apply, compare and evaluate several monthly models in a small drainage basin of Pili in Thessaly, using surface precipitation, temperature and potential evapotranspiration. Innovation is introduced by incorporating remote sensing techniques to obtain regional estimates of the meteorological parameters. Comparison of results of three models, namely Abulohom *et al* (2001), GR-2M (Makhouf and Michel, 1994), and Xiong-Guo (1999) is implemented in this research. The main differences of these models are in evapotranspiration, soil moisture recharge of the aquifer and routing, which result in a variation of the number of parameters, used in each model.

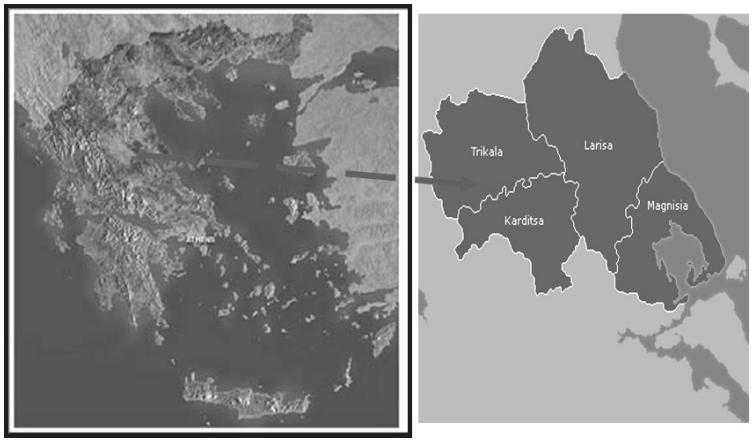


Figure 1. Location of Pili watershed in Thessaly.

2. STUDY AREA AND DATABASE

This research is applied in the Pili watershed, located in Thessaly, central Greece. The Pili basin is drainaged from Portaikos River, which is a tributary of Pinios River in Thessaly. The Pili basin drains the southern part of Thessaly district, and specifically

the south part of the region of Trikala (Figure 1). This basin is mainly mountainous with a large percentage of woodlands and grasslands. In Table 1 topographical and hydro-meteorological characteristics of the study watershed are presented.

Table 1. Topographical and hydro-meteorological characteristics of the study watershed.

Pili characteristics	
Area	132.0 km^2
Mean elevation:	946.66 m
Maximum elevation:	1683.26 m
Minimum elevation:	301.94 m
Receiving water body:	River Portaikos / River Pinios
Mean slope	38.36%
Mean annual precipitation	1564 mm
Mean annual runoff	1042 mm

The geological formations at the watershed under study are sandy flysch, flysch colluvium, hard limestones and alluvial materials, the major land cover in the watershed is forest and the terrain form is mountainous. The climate of the study area is typical Mediterranean, with hot, dry summers and mild winters. Data from four precipitation stations and three meteorological stations were used for the period October 1981 to September 1993. The data set concerns mean monthly precipitation and temperature values. Mean monthly runoff data measured at the mouth of the watershed were also available. The precipitation (Argithea, Drakotripa, Stournareika and Elati) as well as the meteorological stations (Vakari, Pahtouri and Polineri) are showed in Figure 2a. In Figure 2b the location of runoff measurement station is indicated. In addition, mean decadal air temperature and Blaney-Criddle potential evapotranspiration of all Thessaly district, derived from NOAA/AVHRR satellite data set, are used for the same time period (Kanellou, 2006; Kanellou *et al.,* 2006; Hondronikou, 2006). Examples of satellite images used are presented in Figures 3 and 4.

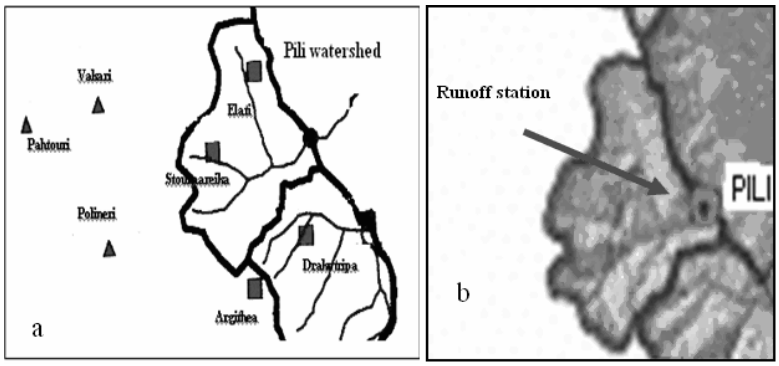


Figure 2. Locations of Precipitation, meteorological and runoff stations.

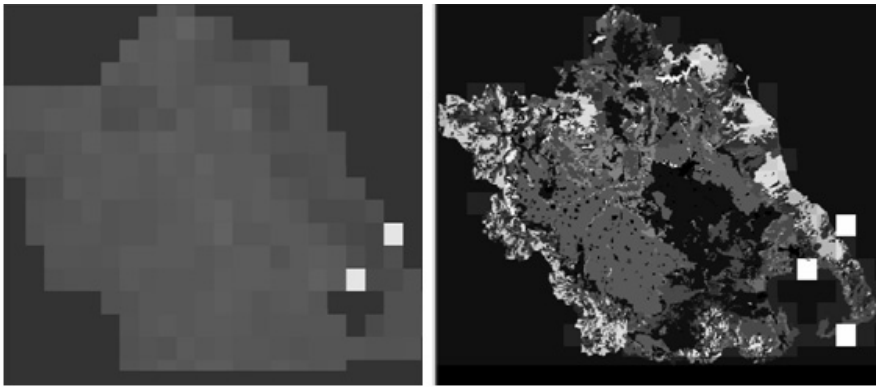


Figure 3. Air temperature satellite image and Potential Evapotranspiration satellite image.

3. THE WATER BALANCE MODELS

The three employed models in this work are the following:

3.1. Abulohom *et al.* (2001) model

The Abulohom model (Abulohom *et al.*, 2001) has been recently developed and is a model with five parameters. Abulohom model divides the total runoff into two components, the direct runoff and the basic runoff, using precipitation and potential evapotranspiration. Therefore parameters of the model are considered as catch all parameters representing indirectly the characteristics of the basin study i.e. soil type, vegetation cover, and geology. In this model the values of three parameters are identified through calibration. The values of the other two parameters take discrete values also for each drainage basin only one value is used for each parameter.

3.2. GR2M model

The model GR2M (Makhouf and Michel, 1994) consist of two tanks, which store water. Every month there are input elements, namely rainfall and potential evapotranspiration. The output of the model is the total monthly runoff. The GR2M model has five parameters, two of which can take discrete values. As this model has been applied in ninety-one drainage basins in France, it has been estimated that for a certain area, the minimum capacity is A = 200 mm and the other parameter, which is the ground water runoff, is equal to the 20% of the total inflow. These values can change, due to topography, the geological base and the climate. Only two parameters are calibrated, if the appropriate values are given to the other parameters.

3.3. Xiong and Guo (1999) model

The by Xiong and Guo (1999) model has two parameters, which are used for the simulation of the runoff in seventy basins, in Southern China. This model is a simple one as compared to the other models that use more parameters. Compared to the other

models another difference is that actual evapotranspiration is used. The areas where this model can have positive results are according to Xiong-Guo moist and semi-moist basins.

4. METHODOLOGY

Two methodological approaches are used of input data sets for each model. Firstly, the input data of these models are the mean monthly rainfall, the mean decadal potential Blaney-Criddle (1950) evapotranspiration (Ep), the temperature values from meteorological stations and the stored moisture at the beginning of the current month (or the end of previous) Sn-i. Secondly, satellite images of mean decadal air temperature are used placing the meteorological stations' temperature values which are used in first application. The other inputs variables remain the same. The current output is the simulated monthly runoff Qn, for all time series and from all applied models. The use of remote sensing data in these models is an innovative approach. The aim is the examination of use of satellite data and of the comparison between observed and simulated runoff extracted from the two different ways of methodology.

The water simulation of the surface runoff, carried out with the Abulohom, GR2M and Xiong- Guo models. The water balance models were calibrated using the available runoff data for the period October 1981 to September 1987 (7 hydrological years), which is called calibration period, and verified for the period October 1987 to September 1993 (6 hydrological years), which is called verification period. The water balance model parameters were optimized during calibration using the Simplex Downhill Algorithm with an objective function as defined by Nash-Sutcliffe (1970):

$$Eff = 1 - \frac{\sum_{i=1}^{n} (Qobsi - Qsimi)^2}{\sum_{i=1}^{n} (Qobsi - \overline{Qobs})^2} \quad (1)$$

where,
Qobsi: observed runoff of month i,
Qsimi: simulated runoff of the month i,
Qobs (uperliing): mean monthly observed runoff.
Apart from the Nash-Sutcliffe Model Efficiency statistic, three more error statistics are used to validate the performance of the water balance model namely:
- Coefficient of determination R^2.
- RMSE (Root mean square error)

$$\text{RMSE} = \sqrt{\frac{\sum_{i=1}^{k} (Qobsi - \overline{Q}obspi)^2}{K}} \quad (2)$$

where,
Qobsi: observed runoff of the month *i*,
$\overline{Q}$ *obspi*: mean of the observed runoff for the period, calibration or verification,

K: number of the cases
- BIAS

$$BIAS = \frac{1}{N}\sum_{i=1}^{N}(Q_{obs} - Q_{sim}) \quad (3)$$

where, *Qobs*: observed runoff of the month, *Qsim*: simulated runoff of the month, N: number of the cases.

5. RESULTS

After calculating the monthly runoff, diagrams of monthly-estimated and observed runoff for the period Oct. 1981-Sept. 1993, are extracted for both approaches. Also the scatter graphs along with regression lines are created for the above data. In Tables 2 and 3 the results of statistic parameters computed after the methodology applied, are presented, for calibration, verification period and for all the timeseries of data. In Table 2 the results concern about the statistical parameters from first approach, in which air satellite temperature is used. The results of second approach where air temperature for meteorological stations is used are shown in Table 3.

Table 2. Statistic values for the region of Pili (Thessaly) for calibration (Cal.) period, verification (Ver.) period and for all the period of database, using conventional air temperature and Blaney- Criddle evapotranspiration.

Statistics	Abulohom			GR2M			Xiong-Guo		
	Cal	Ver.	Period	Cal.	Ver.	Period	Cal.	Ver.	Period
Eff	0.69	0.79	0.72	0.72	0.82	0.79	0.64	0.76	0.67
R^2	0.70	0.81	0.72	0.70	0.80	0.72	0.64	0.78	0.69
RMSE	59.27	33.93	49.15	60.10	33.49	49.55	63.56	36.75	53.56
BIAS	6.58	8.72	7.57	-4.6	-1.0	-2.89	-7.80	-7.80	-7.80

Table 3. Statistic values for the region of Pili (Thessaly) for calibration (Cal.) period, verification (Ver.) period and for all the period of database, using satellite temperature and Blaney- Criddle evapotranspiration.

Statistics	Abulohom			GR2M			Xiong-Guo		
	Cal	Ver.	Period	Cal.	Ver.	Period	Cal.	Ver.	Period
Eff	0.69	0.75	0.71	0.71	0.78	0.78	0.63	0.72	0.65
R^2	0.69	0.76	0.72	0.69	0.77	0.72	0.72	0.64	0.67
RMSE	59.68	37.43	50.58	68.74	35.39	50.30	65.19	39.97	54.93
BIAS	6.58	8.74	7.59	-4.75	-1.06	-3.04	-8.18	-8.29	-8.18

The results, as they are presented in tables above, show that all the three models applied appear a satisfactory simulation of runoff in the study area. The model efficiency statistic's values vary between 0.65 and 0.79 for all the period of data set which are well enough. The GR2M model appears better simulation of the runoff, compared with the other models, as it shows from all the statistics. The use of the satellite data in the second methodological approach, gave also very well results, which are approximate with these from the firstly approach. This fact proves that the remote sensing techniques can be positive in water balance models.

6. CONLUSIONS

This paper proposes three water balance model for runoff modeling and two different methods for input data. From the results, as are shown in Tables 2 and 3, the GR2M and Abulohom models appear satisfying application in database of the study area. The Xiong- Guo model though appears a good simulation, differs from the other two models regarding the simulation results. The GR2M model has higher similarity in form and scale between simulated and observed runoff, and has better simulation in Pili than Abulohom or Xiong-Guo models. The statistic parameter of Model Efficiency in GR2M model has higher stability in the application in different values of runoff.

The results of statistical parameters of simulated runoff using NOAA/ AVHRR satellite data of temperature and evapotraspiration in the first way of methodology do not snow significant differences with the results, which were produced using earth measurements of temperature. This application shows that the remote sensing provides credible data and information and can be used in similar applications and therefore is capable to be used in areas which have no ground measurements or meteorological stations.

Furthermore, regionalization of the other model parameters may lead to the application of the water balance models either lumped or distributed, in ground ungauged watersheds.

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DSS Application at a River Basin Scale, Taking into Account Water Resources Exploitation Risks and Associated Costs: The Ribeiras do Algarve River Basin

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Abstract: The increase on water demand most commonly lately verified for the different sectors, namely for domestic use, has been a major incentive towards the improvement of DSS use and capabilities. In the context of a European Project, a DSS tool was developed and applied to the Algarve region, having as major purpose, the sustainable management of the water resources existing in the whole region. However, this macro-analysis can only be considered valuable if, at the local level, water resources management is also performed in a sustainable way. In that context, the work undertaken and more specifically the strategies developed in order to minimize the existing and foreseen water deficits had to take into account all the individual specificities of each water source, namely through the determination of the sustainable levels of exploitation. Moreover, having in mind the requirements specified in the Water Framework Directive (WFD), an economic analysis was also performed. Some results of the strategies developed will be briefly referred and special emphasis will be given to the specific analysis of the Querença- Silves aquifer and to the determination of environmental costs used in the economic analysis.

Keywords: Decision support system, environmental costs, Querença-Silves aquifer.

1. INTRODUCTION

The Algarve region, the southern region of Portugal, which embraces the Ribeiras do Algarve river basin and also a small part of the Guadiana river basin, benefits from favourable weather conditions. In this region, the increase in water demand mostly for agriculture and public water supply has been, for almost two decades, one of the main problems to the balanced economic and tourist development, as this increase in water demand is not always compatible with the existing water resources. In the specific case of golf courses for example, the major problem is associated with the fact that

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most of the investors want to implement golf courses in the coastal area, where the existing aquifers already present important water quantity and quality problems. Adding to that, the agriculture sector is still representing the major water consumer in the region, with more than 65% of the total water consumption volume. All these factors, obviously lead to some conflicts between the different water users. Until the 90's, domestic water supply in the region was almost exclusively assured by municipal boreholes. Moreover, agriculture and industry also used groundwater supply, obviously leading to a severe decrease in correspondent water levels and also to quality problems difficult to control and evaluate. In that context, in the late 90's, the solution proposed was to implement two primary water supply systems (one for the east part of the region and the other for the western part), based on surface water sources, which would assure the conveyance of water from the storage reservoirs supply sources to the municipal reservoirs, of treated water. Each Municipality is then responsible for the operation and management of the secondary water supply systems. More specifically, these storage reservoirs are (Figure 1): the Bravura, Arade and Funcho dams, also used for agriculture, in the western part, and the Odeleite-Beliche dams system in the eastern part (Guadiana river basin). Although three storage reservoirs are located in the western part, the water availability is lower than in the eastern one as their capacity is reduced. In fact, the primary water supply system was planned and implemented considering that a new water source (Odelouca dam) would start operating in 2006 to supply the eastern part. Regrettably, the schedule was not fulfilled (and the construction was even put at risk) as some issues regarding environmental aspects had to be discussed and the dam is now only expected to start operating in 2012 (Maia, 2006). The two systems were physically connected though a reversible pumping station, and a single water utility company, Águas do Algarve, S.A. (AdA), was created. More recently, the company faced some difficulties in fulfilling the increasing water demand due to the dry period verified between 2003 and 2005, which led to low water levels in the different storage reservoirs, strongly limiting the water abstractions.

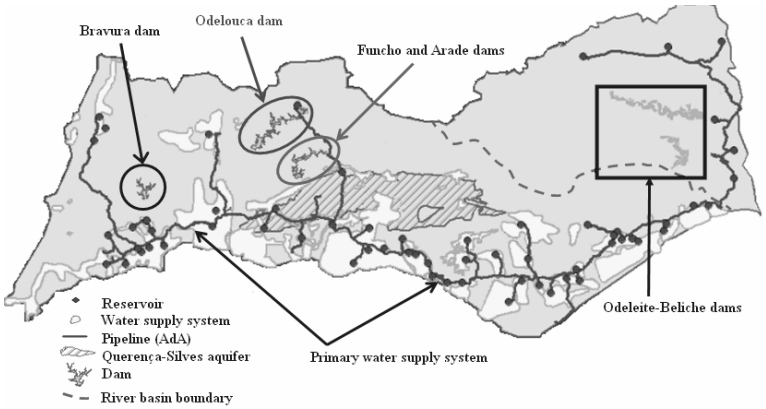


Figure 1. The primary water supply system and water supply sources of the Algarve region.

This situation led to the necessity of finding alternatives on a very short term to guarantee good quantity and quality characteristics in public water supply. On a short term basis, the executable solution found jointly with the different Municipalities, was to connect to the water supplied by AdA some groundwater from the former municipal boreholes that were abandoned with the implementation of the primary water supply system. That way, the different Municipalities were able to mitigate the expected water shortages originated by the dry period.

After the re-activation of the former municipal boreholes, AdA, with the previous agreement of the National Water Institute (INAG), equipped and started exploiting also new boreholes initially constructed for irrigation purposes but not being used. These new boreholes are located in the western part of the most important aquifer of the Algarve region, the Querença-Silves aquifer. This situation led to the necessity of urgently defining a first version of a contingency plan for public water supply in drought situations, revealing, once more, the imperativeness of considering more severe hydrological scenarios in the planning and management of existing water resources. This contingency plan, among other situations, considered the re-activation of the former municipal boreholes and different scenarios corresponding to important limitations in surface water abstraction volumes. In that context, FEUP was requested by AdA to simulate the risk exploitation management of the Querença-Silves aquifer under different water availability conditions.

2. THE WSM-DSS TOOL

By the time of the drought period, FEUP (Faculdade de Engenharia da Universidade do Porto) was carrying out, with the collaboration of AdA and other regional and national institutions, an European Project (Water Strategy Man Project, under the Fifth EU Framework Programme) using a Decision Support System tool developed during that project, in order to test and evaluate the introduction of different structural and non-structural options in the region for water deficit mitigation.

This tool, the WSM-DSS (Water Strategy Man Decision Support System), integrates GIS capabilities, handling both vector and raster information and treating geo-referenced and time-referenced data. It enables the user to evaluate the current situation and analyse the impacts of the introduction of different water management options in the study region. One of the main purposes of this tool is to allow decision makers to evaluate the impact of different water management options simulated considering different hydrological and demand scenarios. This evaluation can be performed through a multidisciplinary approach provided by the tool (Maia and Schumann, 2006).

3. QUERENÇA-SILVES AQUIFER RISK EXPLOITATION MANAGEMENT

3.1. Considerations and Scenario

The simulations performed using the WSM-DSS tool are performed at the regional scale, but the results can be individually discriminated for each water source for example. As groundwater resources, and more specifically the Querença-Silves aquifer, are considered strategic water sources, special attention was given to the results obtained for that particular aquifer.

In a first set of simulations, it was assumed that the existing water deficits for urban use would be solved by increasing water abstractions in the aquifer. The scenario considered corresponds to combining an availability scenario that repeats the hydrological sequence verified in the last 35 years ("Normal" scenario) and a demand scenario based on the current trends of the region ("BAU" – Business as Usual scenario), as defined in the River Basin Plan (PBHRA, 2000). This scenario will be referred as the BAU+Normal scenario. Moreover, in order to perform these simulations, according to the description made in section 1 and considering the current operation of the primary water supply system, some limitations had to be introduced concerning the volume available for abstraction in the other existing water sources of the region and also an operational limitation in the pumping station allowing transferring water from the east to the west and vice-versa. More specifically, it was considered that: (i) in the Funcho storage reservoir, the maximum volume available for abstraction for domestic water supply purposes would respect the following rule: 12 hm^3 in dry and very dry years, 17 hm^3 in normal years and 23 hm^3 in wet and very wet years; (ii) in the Bravura storage reservoir, a maximum water abstraction of 4hm^3 per year would be allowed; and, (iii) the pumping station would transfer 7hm^3 per year until 2010. This volume would be then reduced in 0.5hm^3 every 5 years until 2030.

Considering that set of limitations, special attention was given to the Querença-Silves aquifer, aiming that it would be the alternative supply source. It was admitted that only 80% of the recharge of this aquifer should be used, trying to prevent depletion in the aquifer and consequently quantity and quality problems that could be not reversible. As no consensus existed concerning the annual recharge volume of the Querença-Silves aquifer, in a normal year, two different values were considered, translating the existing discrepancies: 70 hm^3 and 86 hm^3.

3.2. Querença-Silves Exploitation Index Evolution

The results hereafter presented correspond specifically to the groundwater exploitation index (GEI - ratio between abstractions and recharge) of the Querença-Silves aquifer, where no limitation in water abstraction was imposed. These results are compared with the denominated "reference case" that represents the situation

where no changes were introduced in the water system operation. Figure 2 a) and b) show the results for the two values of annual recharges volumes presented above.

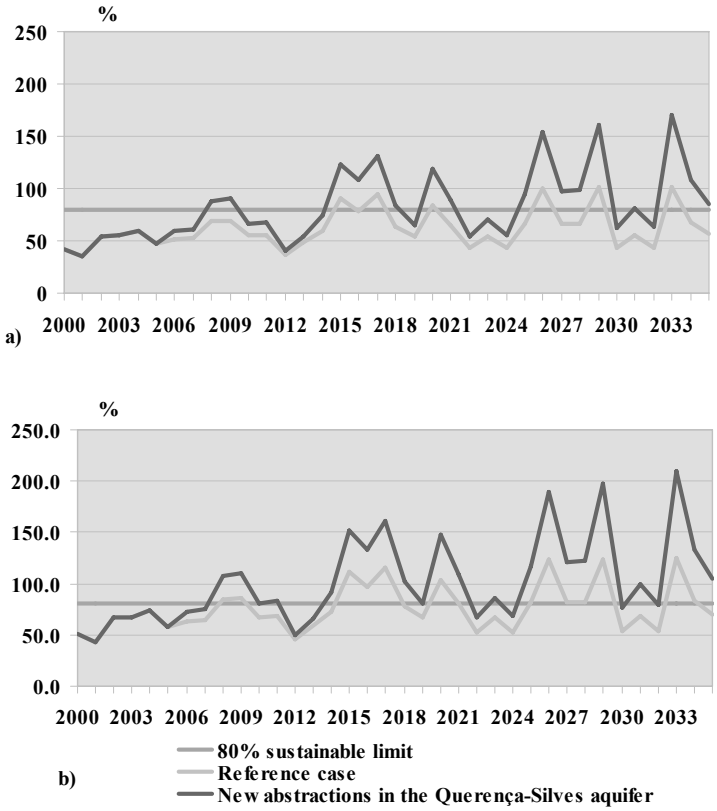


Figure 2. Evolution of the GEI of the Querença-Silves aquifer, considered the major alternative water source, for the BAU+Normal scenario and considering an annual recharge volume of: a) 86 hm³, b) 70 hm³.

In both Figures, the GEI reaches values above 150%, even 200% for the lowest value of recharge (Figure 2b). However, if considering the average value of this index, over the full time period, one verifies that: (i) for the 86 hm^3 annual recharge volume: the average GEI is 62% and 82% for the reference case and the new abstractions in the Querença Silves aquifer respectively. In Figure 2b, these GEI reach 76% for the reference case and more than 100% when groundwater abstractions are intensified. However, if only the last 15 years are considered, the GEI is almost always above 100%, in both situations, with increasing amplitude of the higher peaks. Having that in mind, and the idea of a sustainable management of water resources, it seems reasonable to consider that this solution could only be used until 2015 or 2012 considering 86 hm³ and 70 hm³ of annual recharge respectively.

These results were unfortunately confirmed by the decrease observed in the piezometric levels of the Querença-Silves aquifer leading to serious danger of salinization, even with limited volumes of water abstraction, revealing that this measure cannot be considered, in an isolated way, as a medium or long-term solution. Moreover, the other aquifers existing in the region are still suffering the consequences of the overexploitation verified until the 90's, in terms of quantity and quality aspects, and strong limitations concerning water abstractions were imposed. That way, the different institutions involved in water resources management, and particularly AdA, had to consider other alternative options for water deficit mitigation.

4. STRATEGIES FORMULATION AND ANALYSIS

4.1. Formulation of the Strategies

Maia and Schumann (2006) presented two different strategies for mitigating the water deficits that were identified simulating a 35 years period. The strategies, defined taking into account stakeholders' opinions, were an attempt of considering two different approaches for solving the water deficits that could be expected when considering two different global scenarios: (i) The BAU+Normal scenario, described in 2.2.2; and (ii) a more severe availability scenario characterised by a High frequency of Dry years (HD), obtained reducing in 10% the precipitation, every year, combined with the same demand (BAU) scenario, as in 3.1. This second combination will be referred as the BAU+HD scenario. That way:

- strategy 1 is a combination of the exploitation of surface water sources (considering the implementation of the Odelouca dam) and the introduction of other measures like network enhancements and losses reduction, representing the current most common solutions, as defined in the River Basin Plan;
- strategy 2 is a tentative to emphasize the interest of non-structural water management options, preferably at the local scale, like water re-use, as well as alternative options like desalination plants that are not common solutions in the region. More importantly, this strategy intended to focus on the sustainable use of surface and groundwater in order to preserve water resources.

In each strategy, the volume to be abstracted has been limited, in accordance with the water deficits identified in the reference case and the set of options introduced.

4.2. Global Results

Table 1 presents the results considering effectiveness (relative performance index for demand coverage) of the two strategies, under the two scenarios and the comparison with the reference case, which represents the simulation of the 35 years period without the introduction of any water resources management option of any kind.

Table 1. Strategies'evaluation table regarding effectiveness

	Reference case	Strategy 1	Strategy 2
BAU+Normal Scenario	0.300	0.572	0.638
BAU+HD Scenario	0.300	0.566	0.668

Considering the hypothesis explained in Maia and Schumann (2006), strategy 2 achieves better effectiveness results than strategy 1.

4.3. The Querença-Silves Exploitation Index Evolution

The strategies defined and very briefly explained in 4.1 represent the global intervention, at the regional level. Following the study performed in 3.2, the evolution of the GEI of the Querença-Silves aquifer has been analysed under the two different scenarios' conditions and for the two strategies considered (Figure 3).

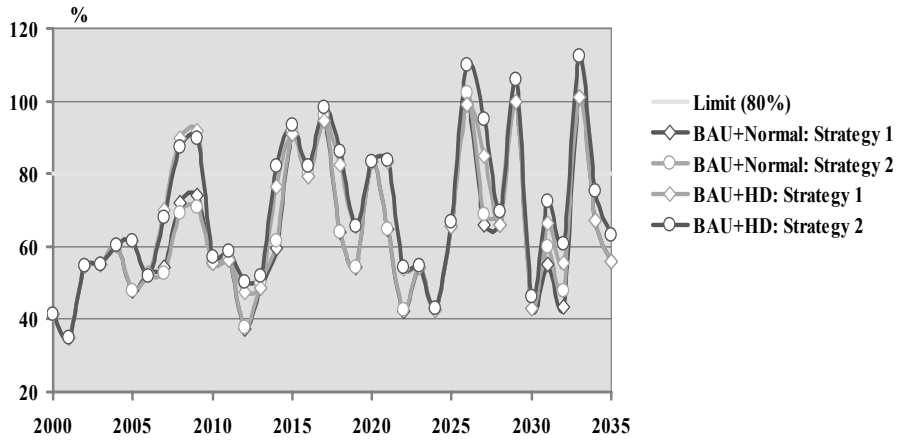


Figure 3. Evolution of the GEI of the Querença-Silves aquifer.

In each case, values above 80% and sometimes reaching more than 100% can be observed. However, if considering the average values for the entire period, the GEI is below 80% for the four different combinations considered with a minimum of 62% for strategy 1 under BAU+Normal scenario and a maximum of 70% for strategy 2, under BAU+HD scenario, showing that in an inter-annual basis, the satisfactory results of the two strategies proposed are also influenced by the behaviour of the Querença-Silves aquifer. In the last 10 years of the simulation period, the highest peaks, reaching a maximum of 110%, are observed but, each time, the aquifer recovers to values below the 80% limit, translating a sustainable use of the aquifer.

5. THE COST ANALYSIS

5.1. Importance of the Cost Analysis

Another very relevant analysis, which can be performed through the WSM-DSS tool, is the economic analysis. The work here presented will only focus on direct and environmental costs definition and evaluation which are main goals of the WFD.

5.2. Determination of Direct and Environmental Costs

5.2.1. Direct costs

According to Maia (2006) the direct costs considered for the Algarve region have been defined based on the evaluation of the:

- depreciation of capital costs associated to past and new investments, for both domestic irrigation and industrial uses.
- operation and maintenance costs of new and existing infrastructures, for domestic, irrigation and industrial purposes.

All the necessary data for the definition of direct costs was collected from the different national and regional institutions, official reports and the different stakeholders consulted.

5.2.2. Environmental costs

The estimation of environmental costs is much more difficult than direct costs estimation as they cannot be directly measured and estimated as investment for example. That way, the approach here chosen is in accordance with the WATECO Guidance document principles (WATECO, 2002). This estimation took into account the following conditions:

- the environmental costs were evaluated considering a relevant example existing in the region and afterwards extrapolated for the entire region,
- all the users would pay the environmental cost corresponding to their use of the natural resource: principle of equity.
- the environmental costs considered the associated costs of an alternative structure whose main purpose would be to supply the maximum share of non-sustainable water abstracted in the existing water source.

In the Algarve region, the environmental costs were estimated considering the aspects and limitations explained above concerning water sources. That way, for groundwater resources, the example chosen for this evaluation was the Querença-Silves aquifer due to its dimension and water supply strategic importance in the region. As previously referred, it seems reasonable to consider, for a "Normal" availability scenario, that approximately 20% of the annual recharge has to be left aside for assuring natural discharges. Having that in mind, the environmental cost was estimated as equal to the construction, operation and maintenance costs of a desalination plant, designed to guarantee the same volume of water. The

correspondent estimated value was set equal to 0.15 €/m³ and applied to every abstraction from any aquifer located in the Ribeiras do Algarve River Basin.

For surface water, the Funcho-Arade storage reservoirs were chosen as they are currently the most important water source in the western part of the Algarve region. Considering the estimated domestic demand in 2035 for the western part of the region (37 hm³), two alternative supply sources were considered in this case: (i) construction of a dam with an overall storage capacity of 48 hm³; and (ii) construction of two desalination plants (total capacity of 37 hm³). The environmental cost average of 0.10 €/m³ for surface water abstractions was considered.

Finally, in what concerns pollution originated by effluent discharges, environmental costs have been set equal to the operating expenses of a secondary wastewater treatment plant (0.20 €/m³). Moreover, a secondary and tertiary treated effluent discharge bonus of 0.20 €/m³ was estimated.

5.3. Evaluation of the Strategies Concerning Direct and Environmental Costs

Table 2 presents the results of the cost analysis the results obtained for direct and environmental costs for each strategy considered under both scenarios. As verified for the effectiveness parameter, strategy 2 performs better than strategy 1 under these two scenarios, with lower values of direct and environmental costs.

Table 2. Direct and environmental costs estimation for strategies 1 and 2.

	PV (M€)	Reference case	Strategy 1	Strategy 2
BAU+Normal Scenario	Direct Costs	1690	2361	2252
	Environmental Costs	692	700	667
BAU+HD Scenario	Direct Costs	1665	2360	2245
	Environmental Costs	687	699	667

5.4. Re-Evaluation of the Strategies under Cost Recovery Strategy

Finally, the last aspect to be taken into account is the implementation of a cost recovery strategy, another capability of the WSM-DSS, which is implemented in order to achieve one of the goals defined in the WFD. In this context, Maia and Schumann (2006) determined a differentiated increase in water prices for domestic users (every two years from 2005 to 2015) in order to achieve 100% recovery of direct costs in 2020 and a minimum of 75% of recovery for environmental costs by 2025. The results obtained showed that the increase would logically be smaller for strategy 2, 65%, against 78% for strategy 1.

6. CONCLUSIONS

The DSS application proved to be efficient at both regional and river basin scales. The

tool allowed associating to each water source the maximum water available for abstraction and defining the sustainable exploitation limits taking into account exploitation risks and associated costs. These costs were evaluated according to WFD requirements, with environmental costs being estimated taking into account the referred analysis of sustainable exploitation of water sources. These considerations enabled to frame alternative strategies to the ones traditionally used, as exemplified by the two strategies' results presented.

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Assessing the Water Potential of Karstic Saline Springs: The Case of Almyros (Heraklion – Crete)

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Abstract: The estimation of useful water potential of sources with varying water quality is based on the identification of the period at which all element concentrations are within the acceptable limits which are dictated by the quality requirements of the use. Karstic saline springs often exhibit strong quality seasonality, with each element or ion following its own seasonal pattern. To facilitate the rational estimation of the useful water potential two fuzzy set methodologies are employed. The first is the use of the min intersection as the fuzzy aggregator and the second is the use of fuzzy multicriteria filtering method. Application of these methods have been used for assessing the quantity from the karstic saline spring of Almyros (Heraklion - Crete) which is suitable for human consumption or irrigation.

Keywords: Almyros spring, fuzzy sets, irrigation water potential, karstic saline spring, potable water potential.

1. INTRODUCTION

Water resources management is considered as a continuous process for fulfilling the needs for living and development under the obligation of conserving adequate resources for the future and protecting sources and the environment.

The frequent occurrence of drought episodes during the last decades and the anticipated conditions under the foreseen climatic instability or change warn water authorities for further adverse conditions in the future.

Most Mediterranean cities face problems for securing water quantities even for potable water whereas other uses are under high pressure. Under the conditions of water scarcity, water authorities are obliged to assess all viable alternative solutions in a systematic way usually called “preparedness plan”.

The preparedness plan is usually conceived as a process following a set of actions, which include the technocratic component, the administrative organization, and the implementation process all based on the public involvement. According to Tsakiris

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(2007) the main actions comprising a water shortage preparedness plan may be summarized as in Table 1.

Table 1. The water shortage preparedness plan as a process (Tsakiris, 2007).

	water shortage preparedness plan process
Public Involvement	• Establish objectives and standards • Conduct inventory (documentation) • Analyze data - prepare forecasts • Calculate risk (annualized risk) • Formulate alternatives • Compare alternatives (multicriteria approach) • Select solutions • Prepare an integrated plan • Prepare a plan implementation programme (administrative organization) • Implement plan

Among the possible solutions to face water shortage the first priority is usually given to the better use of existing water resources resulting in better efficiency and effectiveness of the water system. Demand management is usually known as the set of actions (water pricing policies etc.) that result in lower water demand.

Apart from these actions additional water resources, which are not utilized, could possible enhance the available water potential. Sources of this kind of secondary water resources include brackish water, recycled water, rainwater harvesting etc.

It is widely known that is most Mediterranean islands there exist significant sources of brackish water mainly karstic springs which could be used as viable solutions to face water scarcity problems.

The main disadvantage of these karstic springs is that water is of varying quality but following a more or less predictable seasonality. The most complicating fact is that the concentration of each element is following its own time pattern which is not the same for all major elements.

The paper addresses the problem of the assessment of water volumes from these type of brackish springs which are acceptable for use in various sectors. Needless to say that different volumes of water can be estimated based on the quality requirements of each use.

The methodology used for this estimation and presented in this study is the Fuzzy Set aggregations whereas the case study examined is the Almyros brackish karstic spring – Heraklion, Crete.

2. BASIC NOTIONS

As known each type of water use has different quality requirements. In case of water intended for human consumption quality requirements are very strict for European countries and they have been reported in various EU Directives. In essence each element or ion encountered in the potable water can have a concentration approximated by the guide value (GV) whereas its concentration should be lower than its maximum acceptable concentration (MAC). Figure 1 shows graphically the range

of concentrations of an element j, which is acceptable so that water is considered appropriate for human consumption.

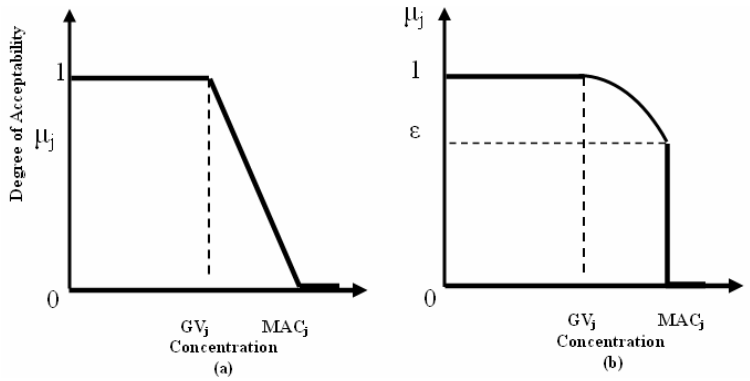


Figure 1. Graphical representation of water acceptability for human consumption based on element.

If all the elements have concentrations from 0 to GV then the water is appropriate for human consumption. However if some elements have concentrations (Z_j) greater than their MAC value, the water is not acceptable for human consumption. Figure1 presents two alternatives representing the acceptability of concentration of each element. The main difference of the two alternatives lies in the fact that the degree of acceptability follows different expressions between GV and MAC values.

Similar considerations could be followed for other uses. However less strict constraints can be adopted for other uses such as irrigation. For instance there might be also a possibility for irrigating more resistant crops in relation to the element, which has a high concentration. Therefore the formulation of the problem for estimating the volumes which are acceptable for each use could be different, dependent on the specific quality requirements of each use.

3. THE METHODOLOGY OF FUZZY SETS

In general, the methodology of fuzzy sets comprises a mapping from a general set X to the closed interval [0, 1] which is described by its membership function. The following equations for the membership functions of Figure1a and 1.b may be written respectively:

$$\mu_j(x) = \begin{cases} 0 & \text{if} \quad Z_j(x) \geq MAC_j \\ 1 & \text{if} \quad Z_j(x) \leq GV_j \\ 1 - \dfrac{Z_j(x) - GV_j}{MAC_j - GV_j} & \text{if} \quad GV_j \leq Z_k(x) \leq MAC_j \end{cases} \tag{1}$$

$$\mu_j(x) = \begin{cases} 0 & \text{if} \quad Z_j(x) \geq MAC_j \\ 1 & \text{if} \quad Z_j(x) \leq GV_j \\ \sqrt{1 - \dfrac{(1-\varepsilon^2)}{MAC_j - GV_j}\left(Z_j(x) - GV_j\right)} & \text{if} \quad GV_j \leq Z_j(x) \leq MAC_j \end{cases} \tag{2}$$

where,

$\mu_j(x)$: the membership function of the jth element

$Z_j(x)$: the concentration of the jth element

GV_j : guide value of the jth element

MAC_j : maximum accedable concentration of the jth element

ε: a predetermined level between 0 and 1.

In this paper the second equation was introduced for the calculations since it simulates better the quality acceptance conditions since values between GV and MAC are considered acceptable. For this reason ε-level is selected near to one.

Having all the individual membership functions of all major elements, the problem is to determine a global evaluation of x, through a fuzzy operator M_w with respect to all indicators. Most environmental standards are based on the non-compensatory combination rule which in general form can be written:

$$\mu(\mathbf{x}) = \min(\mathbf{x_1}, \ldots, \mathbf{x_K}) \tag{3}$$

in which min is the classical *intersection.* This is a very conservative approach (e.g Silvert, 1997) for classifying environmental impacts since any quality indicator that falls below the threshold of acceptability causes the failure of the global quality indicator. The global membership function μ(a) becomes:

$$\mu(a) = \min(\mu_1(a), \mu_2(a), ..., \mu_K(a)) \tag{4}$$

in which K is the number of the quality parameters and a refers to the sample.

Due to the fact that in most cases we have weakly rather than daily measurements for all quality indicators, we have to interpolate the global quality indicator between two time points.

The same approach may be adopted on the quality of irrigation water. In addition the choice of salt tolerant crops should also be investigated due to the fact that they have less rigidly GV and MAC values. Another possibility in case of irrigation use is to adopt an acceptable reduction in yield due to the high concentration of certain elements in the irrigation water (e.g. Ayers and Westcot, 1985).

It should be clear that the use of min intersection for estimating the water volume for irrigation is rather a very rigid approach. Another point of view is that we should investigate several categories due to the fact that there are several crops with different tolerance in salt, conductivity etc.

For these reasons the fuzzy Multicriteria filtering method based on concordance and non-discordance principles was selected in order to categorize the sample to several categories. Actually several categories for irrigation water quality have been proposed. Based on previous studies and considering the Greek conditions and practices the following quality thresholds have been adopted in this study

(Cristiansen, 1977; Maas and Hoffman, 1977; Tsagarakis *et al.*, 2004; Antonopoulos and Tsakiris, 2006). However, the proposed categories take into account the local socio- economic and water conditions. The categories are set a priori and are characterized by thresholds F_i serving as reference points, each representing a boundary between two categories (Table 2).

Table 2. Quality threshold thresholds for irrigation water.

Quality elements	F_2	F_3	F_4	F_5	F_6	F_7
$EC_w\,(dS/m)$	0.7	1.1	2.1	3	5.3	12
SAR	3	6	9	15	16	30
Cl (mg/L)	110	350	600	1100	2130	5000
B (mg/L)	0.5	1	2	3	3.4	15

Between F_1 and F_2 the best category is defined. Between F_2 and F_3, F_3 and F_4, F_4 and F_5, F_5 and F_6, F_6 and F_7, the nearly excellent, very good, good, positive, and acceptable for special crops (salt tolerant crops) and not acceptable categories are defined respectively. The F_0 is a fictitious threshold. The above values, must be normalized by using the values of column F_7 as follows:

$$F_{ij}(\text{normalized}) = \begin{cases} 1 - \dfrac{F_{ij}}{F_{7j}} & \text{if } F_{ij} \le F_{7j} \\ 0 & \text{otherwise} \end{cases} \tag{5}$$

Obviously, the score of each criterion should be normalized both for the samples and the thresholds with the same manner by using Eq.5.

This multicriteria filtering method is based on the strict preference relation (Perny, 1998). The statement aPb means "a is strictly better than b". The assertion aPb implies that the score difference a - b is large enough to make a definitely preferable to b on the basis of criterion j. In this work the strict preference relation is used (e.g Perny, 1998 and Fodor and Roubens, 1994). The concordance and discordance principle is given as follows: When the majority of criteria dictate that a is to be strictly preferred to F_i and there is no criterion which uses its veto against the proposition and a is strictly preferred to F_i, then the a is considered overall a strict preferred to threshold F_i.

The two major criteria selected as the concordance criteria are SAR and conductivity. Four criteria were selected for the discordance principle, which are SAR, conductivity and concentration of both Chloride and Boron. According to this approach the concentration of Cl and B have only the opportunity to cancel the total quality indicator and therefore, SAR and conductivity are the main criteria. The score of each criterion should be normalized both for the samples and the thresholds. The criteria of SAR and conductivity may be considered as equivalent. The indifference threshold is selected to be equal to zero, while the preference threshold was selected to be indifferent to zero to express the vagueness at the boundaries.

The main idea of the discordance principle is that it takes account of strongly conflicting minorities. The overall discordance relation measures with respect to the

fuzzy relation P (strict preference), $D_P(a,F_i)$, measures the degree to which there exists at least one discordant criterion (minority principle), whereas the overall concordance relation with respect to P, $C_P(a,b)$, measures the overall importance of the concordant coalition (majority principle). Thus the disjunctive aggregative operator is mainly chosen for the overall discordance relation, while average operators are chosen for the overall concordance relation (Perny, 1998). The overall relation P becomes:

$$P(a,F_i) = \min(C_p(a,F_i), 1-(D_p(a,F_i))) \quad (6)$$

in which min is the fuzzy intersection.
Then the rule for the assignment of alternatives may be characterised as follows: 'When considering a valued or crisp preference relation P defined on A × F which uses a concordance and non-discordance principle, the assignment rule which is selected is defined on the basis of the equation below:

$$\mu_{Ci}(a)=\min(P(a, F_i),1-P(a, F_{i-1})) \quad (7)$$

Furthermore, the investigation of several scenarios is proposed. Obviously, the global quality indicator is different according to the use of water. For every scenario the time period (quantity) in which the quality is appropriate for several uses should be estimated in order to determine the mean annual usable volume of the source.Consequently the following alternatives could be investigated for utilizing brackish water:

- drinking water,
- irrigation water (several crops),
- irrigation water (choice of salt tolerant crops),
- irrigation water (choice of salt tolerant crops with a 10% reduction in yield due to the high brackishness, e.g Ayers and Westcot, 1985).

4. CASE STUDY: ALMYROS KARSTIC SPRING

The development of karstic springs is of great importance for regions such as many areas around the Mediterranean Sea, where due to the warm and dry climate, there is a great need for water. Unfortunately, many of them are brackish or periodically brackish, as the limestone of their basin is adjacent to the sea. For example, most of the major karstic springs of eastern Crete are brackish or periodically brackish. Among these springs the most important is Almyros karstic spring.

The Almyros spring is located 8 km west of the city of Heraklion, at an elevation of about 4m and at a distance of 1 km from the sea. It is a periodically brackish karstic spring. Its discharge, according to data based on measurements between the years 1968 and 1990, fluctuates between 3.5 m^3/s in the summer and 40 m^3/s in the winter following heavy rainfalls on the Psiloritis Mountain. The water salinity varies from values higher than 5000 mg / L to practically the lowest values in the area of 100 mg / L. The annual spring discharge approaches $240.3x10^6$ m^3 (Paritsis, 2007). At low

discharge, the spring water is brackish because of seawater intrusion into the spring reservoir.

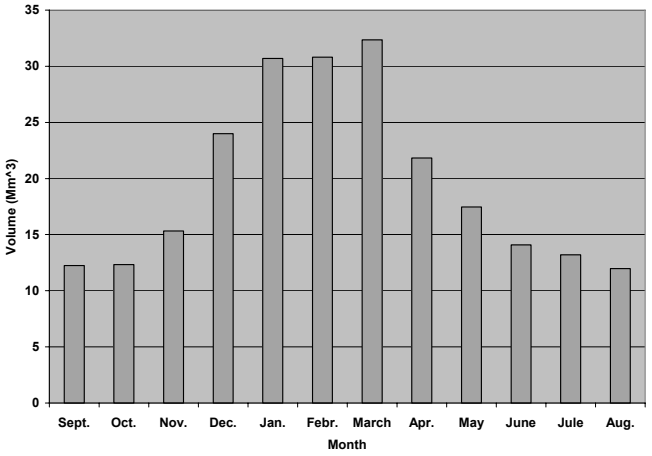


Figure 2. The mean monthly volumes of Almyros spring (1970 -71 to 1998 – 99).

Almyros brackish spring discharges into the sea most of the groundwater potential found within Mesozoic carbonate mountain complex of Idi in central Crete. The groundwater flow field in the eastern section of the Idi complex, with a saturated thickness exceeding 1000 m is funnel shaped to the north east (Almyros spring) as it is bounded to the north by a nappe of impermeable metasediments and metavolcanics and the east by a deep blackballed through filled by Neogen marly deposits.

Despite hydrogeological investigations mainly pertinent to critical aspects of prevailing groundwater conditions on which the mechanism of saline water intrusion is dependent, the operational mechanism is not yet fully understood despite of the significant progress which has been achieved during the last decades (Breznik, 1973; Arfib *et al.*, 2001; Monopolis *et al.*, 1998; Maramathas *et al.*, 2004; Paritsis, 2007).

At first the use of spring water as drinking water was investigated. Unluckily we have not daily measurements and therefore the results used were obtained by interpolation from the weekly data. By applying Eq. 2 as concerning the water quality indicators, in this case the conductivity, the concentration of Chloride, Boron, Magnesium, SO_4 and NO_3 and the hardness, were selected. The min fuzzy intersection method described by Eq. 4 was used for representing the total quality indicator. Using the data of 1999 – 2000 this methodology resulted in an approximate figure of 21x 10^6 m^3 as the potable water (Figure 3).

Another alternative is the use of brackish water for irrigation. Since the quality standards are less rigid for the irrigation use, it is expected that a bigger quantity from Almyros spring could be used for irrigation. By applying Eq. 2 as concerning the water quality indicators, the conductivity, the SAR and the concentration of chloride and boron were selected. Concerning the total quality indicator an approximately use of 28×10^6 m^3 for irrigation use is expected, based on data of 1999 - 2000. The results are graphically shown at Figure 4.

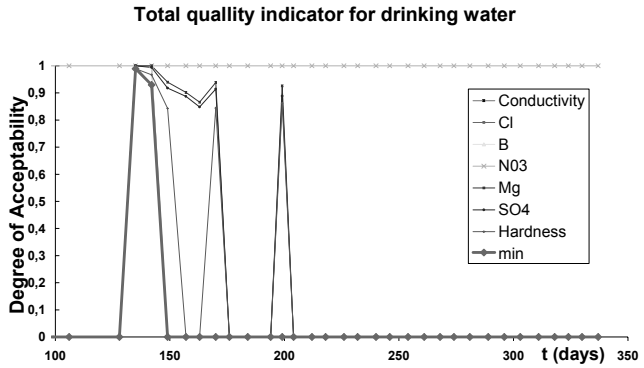


Figure 3. Total quality indicator for potable water of Almyros spring water during the year 1999 – 2000.

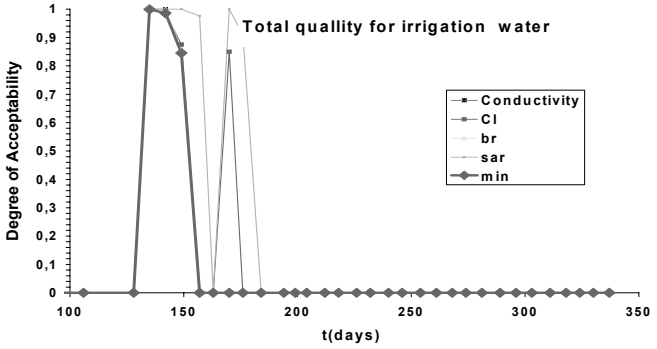


Figure 4. Total quality indicator for irrigation water of Almyros spring water during the year 1999 – 2000.

Further one more alternative the one of yield reduction up to an acceptable level could be investigated (Antonopoulos and Tsakiris, 2006; Tsakiris and Spiliotis, 2006). The second method presented in this paper is the Fuzzy Multicriteria filtering method based on concordance and non-discordance principles to classify the total water quality into categories related to the irrigation use. The results are shown in Table 3. In all other time periods the water quality is not acceptable for any irrigation use. The results of the Multicriteria filtering method seem to be comparable to those of the min intersection method and are presented in Table 4.

Interesting but also constraining factor for the utilization of Almyros spring water is the fact that Boron concentration is unexpectedly high. The reasons of this should be carefully studied. The quantities estimated above correspond to the year 1999 – 2000. The frequency analysis of the annual volumes from the spring show that the discharge of this year has a probability of non - exceedance of about 20% (probability of failure). Therefore the estimated quantities are quite safe in 4 out of 5 years.

Table3. Categorization of brackish water for irrigation use 1999 – 2000.

Time	C_1	C_2	C_3	C_4	C_5	C_6
13/1/2000	0	0	0	0	0	1
20/1/2000	1	0	0	0	0	0
27/1/2000	0	1	0	0	0	0
3/2/2000	0	0	0	1	0	0
11/2/2000	0	0	0	0	0	1
17/2/2000	0	0	0	0	0	1
24/2/2000	0	0	0	0	1	0
2/3/2000	0	0	0	0	0	1

Table4. Quantities for several categories of irrigation use 1999 – 2000.

Categories	C_1	C_2	C_3	C_4	C_5
Quantities ($\times 10^6$ m^3)≈	14	21	21	28	44

5. CONCLUSIONS

Two fuzzy set methodologies were used for assessing the water potential of springs with varying water quality. The min intersection rule seems more strict and more suitable for the estimation of the water potential for human consumption whereas the multicriteria filtering method provides a more flexible method for classifying annual water quantities into quality classes which correspond to quality requirements of the various uses.

The two methods were applied to Almyros karstic saline spring of Heraklion – Crete producing comparable results. As expected the annual water quantity from the spring, suitable for irrigation use is much greater when compared to the quantity suitable for human consumption.

ACKNOWLEDGEMENTS

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Estimation of Evaporation in Lake Vegoritis, Greece

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Abstract: Vegoritis Lake, in Northern Greece, is under a continuous deficit in its water balance for up to four decades. A main component in both the water and energy budgets of lakes and a primary process of water loss for most of them is evaporation from their surface. A one-dimensional, eddy diffusion model of lake's water temperature distribution simulation is used to evaluate Lake Vegoritis daily evaporation values as a residual of the energy budget and for the years of 2003 and 2004. Estimated evaporation rates show a characteristic annual cycle with the highest values at summer and relatively low values at spring. Evaporation values estimated by the model are used for determining the mass transfer coefficient.

Keywords: Evaporation, lake Vegoritis, mass transfer coefficient, model, temperature simulation.

1. INTRODUCTION

Evaporation is a primary process of water and heat loss for most of lakes and thus a main component in both its energy and water budgets. 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. Furthermore the discussion of the last decades on the climate changes and the assessing of relation between climate variation and fluctuations in the size of a lake, require methods for estimating evaporation rates that are sensitive to climate variation.

Evaporation from a free water surface such that of a lake, 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 solar radiation, temperature, relative humidity and wind velocity. Besides climate, lake characteristics as its size, shape, depth, water quality and circulation, even its location can affect the rate of evaporation. Dependence of evaporation on many climatological and geological parameters makes it a unique feature of each lake and inhibits the development of a clearly theoretical method for its estimation. Thus the estimation or measurement of a lake's evaporation is a very difficult task, for which a

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multitude of methods exist, which can be grouped into the following categories: 1) water budget, 2) measurement, like evaporation pans and eddy correlation technique, 3) energy budget, 4) mass transfer and 5) combination methods. The energy budget and eddy correlation techniques are considered to be the most accurate methods. As eddy correlation method is only suitable for short-term studies, the energy budget is the preferred technique for accurate, long-term monitoring (Sturrock *et al.,* 1992; Assouline and Mahrer, 1993; Sacks *et al.,* 1994; Winter *et al.,* 2003; Lenters *et al.,* 2005).

The purpose of this paper is to estimate daily evaporation values for Lake Vegoritis. A one-dimensional, eddy diffusion model using the equilibrium temperature approach is used to determine the lake's water temperature distribution and the net energy available at the lake's surface, thus allowing the use of the energy budget method for the evaporation estimations. The estimated evaporation values are then utilized for determining the mass transfer coefficient for Lake Vegoritis.

2. LAKE VEGORITIS DESCRIPTION

Lake Vegoritis is a very important natural impoundment in Greece. It is located in the northern part of Greece at northern latitude of 40047′ and at an altitude of 510 m (nowadays). The volume of the lake has been decreasing since the mid fifties. About 65% of the lake's water has been lost. The surface level has dropped from the elevation of 540 m (in 1950) to 508 m (in 2002) (Antonopoulos *et al.*, 1996; Papamichail and Antonopoulos, 1998). Lake Vegoritis exhibits great quantitative, qualitative and ecological problems. A huge amount of water discharged out of lake's catchment to satisfy hydroelecrical demand between the years of 1956 and 1985 was the main reason for the decrease in the lake's volume. Moreover the lake's water has been used for irrigation and to supply a number of thermo-electrical power stations in the area. The natural reasons which influence the water balance of the lake include the 'katavothras' (sinks), which drain the lake and is a characteristic of it's limestone bed (Hutchinson, 1975). Lake Vegoritis has been contaminated mainly with nitrogen by the Pentavrissos stream, which accepts industrial and urban waste of the surrounding area. The trophic status of the lake has been changed from oligotrophic at the beginning of the 80's to oligomesotrophic later during the 90's (Antonopoulos *et al.,* 1996).

3. MATHEMATICAL MODELLING

The energy budget (Bowen) method is based on the conservation of energy law, which accounts for incoming and outgoing energy, balanced by the amount of energy stored in the system. The energy budget of a lake can be expressed as (Bowie *et al.,* 1985; Sturrock *et al.,* 1992):

$$q_t = q_s - q_{sr} + q_a - q_{ar} - q_{br} - q_c - q_e + q_b + q_v \qquad (1)$$

where,
q_t is the change in the energy (thermal) content of the water body, q_s and q_a are the short-wave radiation incident to water surface and the incoming long-wave radiation from the atmosphere respectively, q_{sr} and q_{ar} are the reflected short-wave and long-wave radiation respectively, q_{br} is the back (long-wave) radiation emitted from the body of water, q_c is the energy conducted to or from the body of water as sensible heat, q_e is the energy utilized for evaporation, q_b is the heat transfer between the lake water and the bottom sediments and q_v is the net energy advected into the body of water.

Exchanges of energy occur and through precipitation, withdrawal of evaporated water, chemical and biological reactions in the body of the water, conversion of kinetic to thermal energy, but in small amounts so that they can be omitted. In many cases, and especially in large and deep lakes, the components q_b and q_v are enough small to be neglected. Many researchers agree that omitting the energy budget components with small values doesn't significantly affect the results (Bolsenga, 1975; Sturrock *et al.,* 1992; Sacks *et al.,* 1994; dos Reis and Dias, 1998; Winter *et al.,* 2003). In our case we constrain the energy budget to the surface of the lake, using Equation 1 with the following form:

$$q_n = q_s - q_{sr} + q_a - q_{ar} - q_{br} - q_c - q_e \tag{2}$$

where,
q_n is the net energy available at the lake's surface.
Equation 2 is used to calculate lake evaporation as a residual of the energy budget. The basic formulation which describes the energy consumed for evaporation (q_e) is:

$$q_e = \rho_w L_v E \tag{3}$$

where,
ρ_w is the water density, L_v is the latent heat of vaporization of water and E is the water evaporation.
The flux of sensible heat (q_c) is related to the evaporative heat flux (q_e) through the Bowen ratio (β) (Henderson-Sellers, 1984):

$$\beta = \frac{q_c}{q_e} = \gamma \frac{T_s - T_a}{e_{sw} - e_d} \tag{4}$$

in which T_s is the surface water temperature (ºC), T_a is the air temperature above the water surface (ºC), e_{sw} is the saturation vapor pressure at water temperature (mbar), e_d is the air vapor pressure above water surface (mbar) and γ is the psychrometric constant (in mbar/ºC). Combination of Equations 2, 3 and 4 results in the following equation for evaporation estimation:

$$E = \frac{q_s - q_{sr} + q_a - q_{ar} - q_{br} - q_n}{\rho_w L_v (1 + \beta)} \tag{5}$$

The energy terms q_s and q_a are estimated from meteorological data, q_{sr} and q_{ar} are fixed portions of q_s and q_a and q_{br} is estimated from the lake's surface temperature with the use of the Stefan-Boltzmann law. For determining the net energy available at the lake's surface (q_n) and the water temperature of the lake the mathematical model QUALAKE-DOT is used. QUALAKE-DOT is a one-dimensional, eddy diffusion, finite element, temperature stratification and oxygen prediction model which was developed to simulate the seasonal temperature cycle and oxygen distribution in Lake Vegoritis (Antonopoulos and Gianniou, 2002; Antonopoulos and Gianniou, 2003).

The submodel of heat transport is based on the non-linear heat transfer equation using daily weather parameters as input data. The one-dimensional vertical transport of heat is described by the diffusion equation of the form (Henderson-Sellers, 1984; Chapra, 1997):

$$\frac{\partial T}{\partial t} = \frac{1}{A}\frac{\partial}{\partial z}(KA\frac{\partial T}{\partial z}) - \frac{1}{A}\frac{\partial (Aq)/\partial z}{\rho_w c_{pw}} \tag{6}$$

in which $T(z,t)$ 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 (m²), K is the eddy diffusion coefficient (m²/day), $q(z)$ is the internal distribution of heat sources due to solar radiation absorption inside the water column (cal/m^{2}day), ρ_w is the water density (kg/cm³) and c_{pw} is the specific heat of water (cal/kgºC).

The internal distribution of solar radiation is described with the net short wave solar radiation q_{sn} absorbed at the surface, which is calculated from the upper solar radiation q_o. The boundary condition at the water surface (z=0) denotes that the heat diffused away from the lake surface equals the net surface heat flux (q_n), as described by the equation:

$$\rho_w c_{pw} K \frac{\partial T}{\partial z}\Big|_{z=0} = -q_n \tag{7}$$

As estimation of evaporation (E) and consequently of evaporative heat flux (q_e) is the aim of this study, q_n cannot be evaluated via the energy budget equation. A simple linear approach to estimate q_n offers the equilibrium temperature theory proposed by Edinger *et al.* (1968) and expressed by the equation:

$$q_n = H(T_e - T_s) \tag{8}$$

where,

H (cal/cm^2-day-ºC) is the surface heat exchange coefficient which represents the rate of water temperature changes under the influence of heat exchange procedures and T_e is the equilibrium temperature (ºC), defined as that water surface temperature which for a set of meteorological conditions causes the surface heat flux to equal zero. The surface heat exchange coefficient and equilibrium temperature are calculated according to Edinger *et al.* (1974) approach. The bottom boundary condition is defined as one of zero heat flux across this boundary.

4. MODEL APPLICATION IN LAKE VEGORITIS

The proposed model is applied to Lake Vegoritis to simulate evaporation on a daily basis and for the climate conditions of 2003 and 2004. Simulations started on February with isothermal conditions (5ºC) and continued in daily time steps until the next February. The model estimates the daily distribution of lake water temperature with depth, the surface temperature (T_s), the net energy available at the lake's surface (q_n) and the energy budget components and from them the daily evaporation. The parameter set of the mathematical model QUALAKE-DOT was calibrated and verified against water temperature measurements during the years 1981 and 1993 (Antonopoulos and Gianniou, 2002; Antonopoulos and Gianniou, 2003). During the application when calculated Bowen ratio values (β) are close to -1 extremely inaccurate values for latent heat flux (q_e) were estimated. A simple way of solving this problem, as proposed by Tanner *et al.* (1987), is to reject the data for which $-1.25<\beta<-0.75$. In Figure 1 the monthly simulated evaporation rates for Lake Vegoritis for the years of 2003 and 2004 are presented.

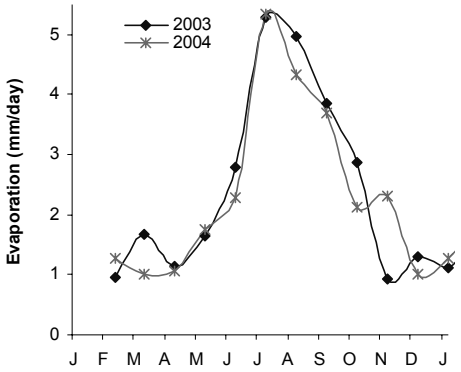


Figure 1. Monthly variation of evaporation values for Lake Vegoritis during 2003 and 2004.

The mean rate of evaporation for the simulation period is 2.50 mm/day for the year of 2003 and 2.45 mm/day for the year of 2004. Evaporation rates are low from the onset of calculations and throughout springtime. At summer evaporation increases rapidly reaching its highest values at July and August. During autumn evaporation decreases, still retaining up to the end of the year higher than the springtime values. Negative evaporation values (condensation) emerge exclusively at spring since at this time of the year air temperature increases more rapidly than lake surface temperature due to the large heat capacity of the lake. Additionally atmospheric instabilities and frequent transitions of temperature and vapour pressure declination over the lake are common during springtime.

Net radiation during spring and early summer is utilized mainly for the lake's heating. Reversely during autumn, even though air temperature and net radiation continuously decrease, evaporation rates are high because lake releases as latent (and in a minor proportion as sensible) heat the energy stored in its volume.

5. MASS TRANSFER EQUATION

One of the most common and oldest methods for estimating evaporation is the mass transfer (aerodynamic) method. The mass transfer method is based on Dalton's law, according to which transfer of water vapor from an evaporating surface is proportional to the wind velocity and the vapor pressure deficit, expressed by the general equation (Singh and Xu, 1997):

$$E = f(u)(e_{sw} - e_d) \tag{9}$$

where,
$f(u)$ is a function of the wind speed at some specified elevation above the water surface (Henderson-Sellers, 1984). Most commonly the function $f(u)$ takes the following form:

$$f(u) = a \cdot u \tag{10}$$

In the above equation a is an empirical coefficient. Its value is site specific and depends on many factors, like the shape and size of the lake, the height and location of meteorological measurements, the climatological conditions and more. Since a generally valid equation for $f(u)$ doesn't exist, many different mass transfer equations have been proposed and used. For determining the mass transfer equation coefficients the evaporation estimations of another accurate method are necessary. In most cases the energy budget method is utilized as the independent method (Sturrock *et al.,* 1992; Sacks *et al.,* 1994).

In the case of Lake Vegoritis the daily energy budget evaporation rates for the two years of the simulation (2003 and 2004) are used as the depended variable and the value of the coefficient a of Equation 10 is determined with regression analysis using the statistical program SPSS. The value determined from this technique is a=0.1656 when E is expressed in mm/day, e_{sw} and e_d in mbar and u is in m/sec and measured at 3 m above the ground. In this way the mass transfer equation derived for Lake Vegoritis has the form:

$$E = 0.1656 u_3 (e_{sw} - e_d) \tag{11}$$

The correlation coefficient for Equation 11 is 0.74. The evaporation rates resulting from the application of the above equation for the two years 2003 and 2004 are presented as monthly values in Figure 2. In this figure the estimations of the mass transfer equation are compared to that of the energy budget method.

Evaporation rates calculated by the mass transfer equation follow the variations of the mass transfer product $u(e_{sw}\text{-}e_d)$ and exhibit less extreme values compared to the energy budget method. Equation 11 is a very useful tool for simplified and fast estimations of lake evaporation and it should be preferred to other equations derived for different lakes and different climatological conditions.

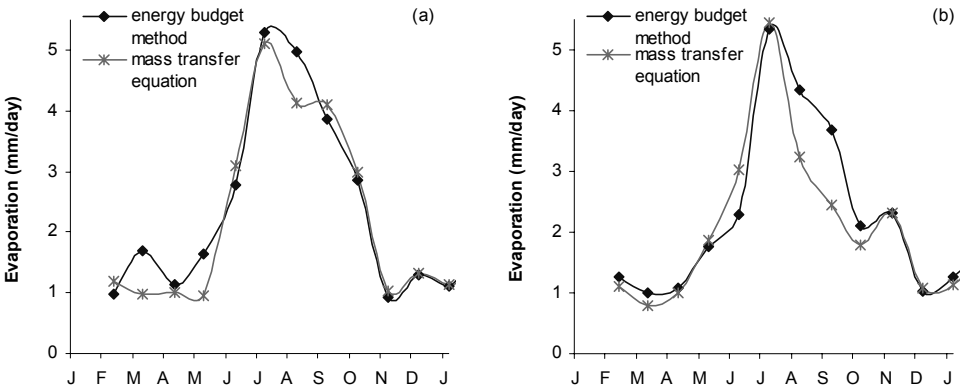


Figure 2. Monthly variation of evaporation correlated to the mass transfer product u(esw-ed) for Lake Vegoritis during the years 2003 (a) and 2004 (b).

6. CONCLUSIONS

The one-dimensional temperature stratification and dissolved oxygen prediction model QUALAKE-DOT, which uses the equilibrium temperature approach, was combined with the energy budget method to estimate daily evaporation values of Lake Vegoritis for the years 2003 and 2004. Lake Vegoritis is under a continuous deficit in its water balance for up to four decades. Therefore the development of a reliable method for evaporation estimates is essential for water budget and lake management studies.

Evaporation shows a characteristic annual cycle with low values at spring and the highest values at summer. During autumn evaporation decreases, still retaining up to the end of the year higher than the springtime values. Negative evaporation rates emerge exclusively at springtime. The mean rate of evaporation for the two years of the simulation period is 2.50 and 2.45 mm/day for 2003 and 2004 respectively. The daily energy budget evaporation rates were used to determine the coefficient for the mass transfer equation, which can be applied for simplified and rapid estimations of Lake Vegoritis evaporation.

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Application of a Continuous Rainfall – Runoff Model to the Basin of Kosynthos River

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Abstract: In this paper, the application of a continuous rainfall – runoff model to the basin of Kosynthos River (district of Xanthi, Thrace, northeastern Greece), as well as the comparison of the computational results with field discharge measurements are presented. The rainfall losses are estimated by the widely known Soil Conservation Service (SCS) model, while the transformation of rainfall excess to direct runoff hydrograph is made by using the dimensionless unit hydrograph of SCS. The routing of the direct runoff hydrograph from the outlet of a sub-basin to the outlet of the whole basin is achieved by the Muskingum – Cunge model. The baseflow is computed by applying an exponential recession model. In order to calibrate the whole composite model, a sensitivity analysis is made.

Keywords: Continuous rainfall – runoff model, Kosynthos river, Muskingum – Cunge model, soil conservation service.

1. INTRODUCTION

In this paper, the application of a continuous rainfall – runoff model to the basin of Kosynthos River (district of Xanthi, Thrace, northeastern Greece), as well as the comparison of the computational results with field discharge measurements are presented.

In the 60's, the need of a more realistic simulation of the process of rainfall – runoff, based on physiographic characteristics (i.e. land use, soil cover and slope) in complex larger basins, was urgent. Then, models as Stanford IV and Sacramento River were developed for this purpose. However, the plethora of the data demanded led to no realistic results (Crawford *et al*., 1966; Burnash *et al*., 1973). This lack of a one to one relationship between model and reality was the spark of a research attempt in hydraulic and hydrologic institutes [Danish Hydraulic Institute (Denmark), Institute of Hydrology (U.K.) and SOGREAH (France)], which led to the development of a

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conceptual model. This model included the principal differential equations (continuity and momentum equations) which connect the subprocesses of the model by adjusting boundary conditions. Principal differential equations such as Richards equation for infiltration, Darcy's equation for groundwater flow and Saint Venant equations for routing flood waves were available. The product of this joint effort is the "SHE" model (Système Hydrologique Européen) (Abbot *et al*., 1986), which is milestone for all these simulation attempts of the rainfall-runoff process.

One basic part of this process is the flood routing. The models can be divided into two main categories, the hydrologic models as the Muskingum model (Dooge *et al*., 1982) and the hydraulic models as the kinematic, diffusion, hydrodynamic wave models (Price, 1973; Keefer and McQuivey, 1974).

The rainfall-runoff models are classified into various categories, that depend on the kind of the equations, the time and spatial step etc. The most common categories of the models are the continuous and event models, as well as the lumped and distributed models. The US Army Corps of Engineers delevoped the VISUAL HEC and its evolution the HEC-HMS software, which is used in this paper.

2. A BRIEF THEORETICAL MODEL DESCRIPTION

2.1. Rainfall Excess Model

According to Soil Conservation Service (SCS) method, the rainfall excess is computed by:

$$h_R = \frac{(h_r - 0.2S)^2}{h_r + 0.8S} \tag{1}$$

where,
h_R: rainfall excess [mm],
h_r: rainfall [mm], and
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, 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 simplified 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 q_p is

$$q_p = \frac{CA}{T_p} \tag{3}$$

where,
C: 2.08,
A: basin area [km^2], and
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. Routing Model of Muskingum – Cunge

The routing of the direct runoff hydrograph from the outlet of a sub-basin to the outlet of the whole basin is enabled by means of Muskingum – Cunge model. 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:

$$Q_{i+1}^{k+1} = C_o Q_i^{k+1} + C_1 Q_i^k + C_2 Q_{i+1}^k \tag{5}$$

where Q is the direct runoff discharge, i designates the space step Δx and k designates the time step Δt.

$$C_o = \frac{c\lambda - 2x}{2(1-x) + c\lambda}, C_1 = \frac{c\lambda + 2x}{2(1-x) + c\lambda}, C_2 = \frac{2(1-x) - c\lambda}{2(1-x) + c\lambda}, K = \frac{\Delta x}{c} \text{ and } \lambda = \frac{\Delta t}{\Delta x} \tag{6}$$

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 $v_h = q_o / 2S_f$ with the numerical diffusion coefficient $v_n = c\Delta x(0.5 - x)$ of the numerical scheme, the

value of the parameter x is obtained:

$$x = \frac{1}{2}(1 - \frac{q_o}{S_f c \Delta x}) \tag{7}$$

where,
c: small wave celerity,
q_o: reference discharge per unit width (from the inlet hydrograph), and
S_f: energy slope.
By using Equation (7) the parameter x is computed by means of the physical characteristics of the routing stream (Δx, S_f, c, q_o).

2.4. Exponential Recession Model

The applied composite model includes an exponential recession model to represent the time variation of baseflow (Chow *et al*., 1988). The recession model is described mathematically by the following equation:

$$Q_t = Q_0 k^t \tag{8}$$

where,
Q_t: baseflow at any time t,
Q_o: initial baseflow (at time zero), and
k: exponential decay constant; it is defined as the ratio of the baseflow at time t to the baseflow one day earlier.

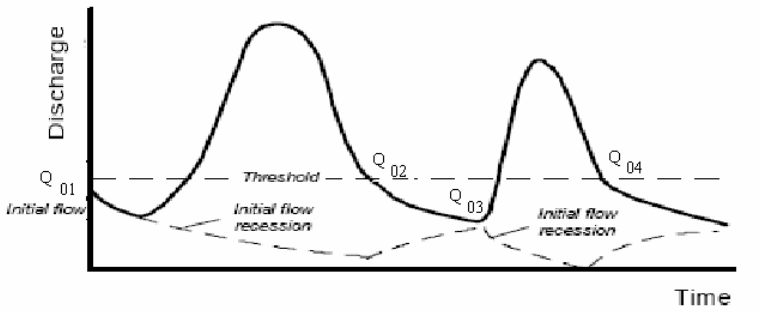


Figure 1. Recession with multiple runoff peaks.

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). Equation (8) is applied twice to a hydrograph: (a) to simulate the initial flow (baseflow) recession; in that case, the value Q_{01} lies on the discharge axis, and (b) to simulate the total runoff recession; in that case, the value Q_{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_{03} corresponds to the end of the first hydrograph, while the value Q_{04} is defined as the value Q_{02} in the first hydrograph.

3. APPLICATION

The above model was applied to simulate a continuous hydrograph of Kosynthos River basin (district of Xanthi, Thrace, northeastern Greece) of the years 2005-2006. The basin of Kosynthos River has an area of about 237 km^2 consisting of forest (74%), bush (4.5%), urban area (1.5%) and an area with no significant vegetation (20%). The highest altitude of the basin is about 1700 m. The length of the main stream of the basin is about 35 km. For more precise calculations, the basin was divided into ten natural sub-basins (Figure 2). The structure of the basin, especially for the model operation, is also depicted in Figure 2. For the estimation of the model parameters, rainfall data (time step 5 min) from the meteorological station of Oreo, as well as topographical, geological, main stream and soil cover maps were used. The meteorological station of Oreo is located in the centre of gravity of the basin. Moreover, the Laboratory of Hydrology and Hydraulic Works, Civil Engineering Department, Democritus University of Thrace, carried out five field measurements of discharge in October, November, December 05 and four measurements in June 06.

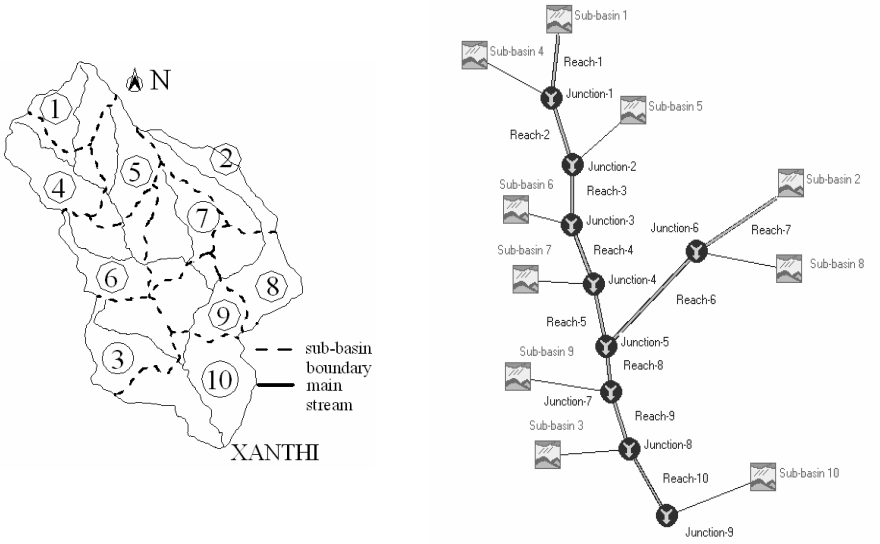


Figure 2. Main stream map (left); basin structure for the model operation (right).

3.1. Determination of the Model Parameters

The curve number (CN, parameter of the SCS rainfall excess model) was estimated for each sub-basin by processing land use and soil cover maps. It should be mentioned that the model is very sensitive to the CN parameter; therefore, after the first estimation of the above parameter, the model was calibrated with field discharge measurements.

The parameter of the SCS model for the transformation of rainfall excess to runoff

hydrograph, which should be estimated, is the concentration time. The concentration time was computed by various empirical equations (Giandotti, Kirpich, Ramser, SCS, Fortbend). After the comparison of the computed discharges with the field measurements, it was proved that the most suitable empirical equation is the formula of Giandotti and that the model is very sensitive to the concentration time.

The determined parameters of the Muskingum – Cunge routing model are:

a) the main stream length of each sub-basin (main stream map),

b) the average slope of the main stream of each sub-basin (contour map),

c) the geometric characteristics of the cross sections (estimated),

d) the roughness of the stream bed.

In Table 1, the area, the curve number and the concentration time, resulting from the Giandotti formula, are given for each sub-basin. In Table 2, the length, the bed slope, the bottom width, the shape of the cross section and the Manning coefficient are given for each routing reach.

Table 1. Area, curve number and concentration time values for each sub-basin.

Sub-basin	Area [km^2]	Curve number	Concentration time [min]
1	17.58	65	129
2	21.53	70.72	171
3	25.79	62.58	276
4	23.68	64.90	102
5	21.93	66.75	108
6	25.43	71.14	353
7	23.33	70.22	101
8	22.18	79.22	104
9	20.48	78.88	133
10	35.35	68.18	394

Table 2. Length, bed slope, bottom width, cross section shape and Manning coefficient for each routing reach.

Routing reach	Reach length [m]	Reach bed slope	Bottom width [m]	Cross section shape	Manning coeff [s/m$^{1/3}$]
1	3930	0.0637	30	Rectangular	0.12
2	2320	0.0216	30	Rectangular	0.12
3	3740	0.0401	20	Rectangular	0.12
4	2960	0.0169	20	Rectangular	0.12
5	2000	0.0238	50	Rectangular	0.12
6	2000	0.0238	50	Rectangular	0.12
7	4720	0.0212	50	Rectangular	0.12
8	4200	0.0238	50	Rectangular	0.08
9	1000	0.0112	80	Rectangular	0.08
10	8910	0.0112	80	Rectangular	0.08

The decay constant *k* of the baseflow model has obtained the value 0.85, while the threshold discharge was determined as equal to 25% of the peak discharge. The determination of both values was enabled by means of calibration and sensitivity analysis.

4. QUANTITATIVE RESULTS

In Figure 3, the computed discharge hydrograph (direct runoff + baseflow) of October – December 2005 and the measured discharge values at the basin outlet are depicted. In Figure 4, the computed discharge hydrograph (direct runoff + baseflow) of June 2006 and the measured discharge values at the basin outlet are illustrated.

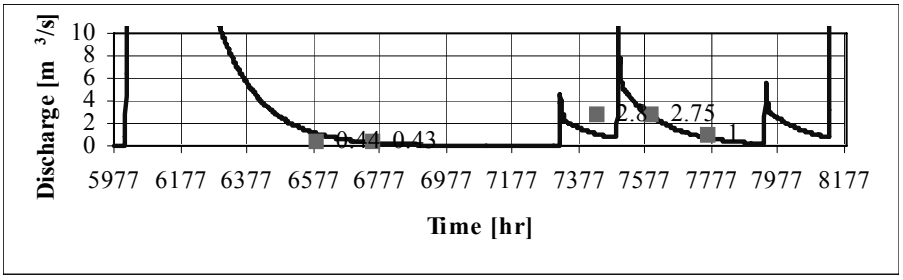


Figure 3. Discharge hydrograph of October – December 2005.

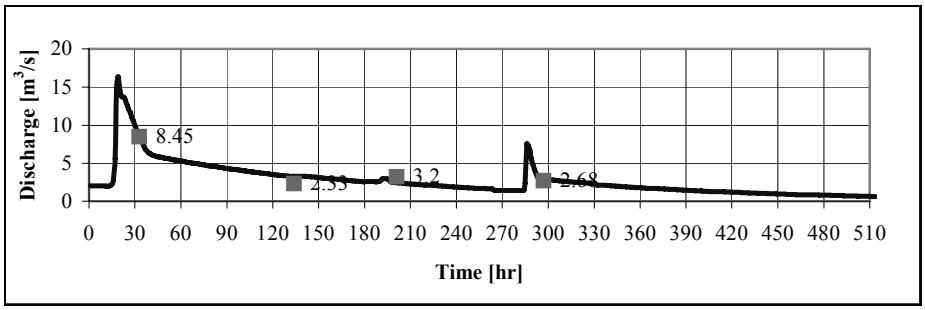


Figure 4. Discharge hydrograph of June 2006.

The comparison between computed and measured discharge values was performed on the basis of the "root mean square error" (RMSE). The value of RMSE amounts to 0.75 m^3/s. The degree of linear dependence between computed and measured discharge values is expressed by the correlation coefficient. This coefficient, as a result of linear regression analysis, is equal to 0.95.

5. CONCLUSION

From Figures 3 and 4, as well as from the RMSE-criterion, it is concluded that the deviation between computed and measured discharge values is not considerable. Additionally, the degree of linear dependence between computed and measured discharge values is very high. However, it has to be noted that the number of the measured discharge values is quite small. Therefore, the values of RMSE and correlation coefficient are not widely representative.

ACKNOWLEDGEMENTS

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Remotely Sensed Severity Assessment of Hailstorms

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Abstract: Severe storms in the form of hailstorms are considered natural environmental hazards, which may have significant impact to environment and natural resources. In this study the severity of hailstorms is assessed and identified by developing a three-stage methodological procedure, which is based on empirical functional relationships between satellite METEOSAT IR (infrared) parameters and weather radar parameters. In particular, in the first stage an empirical functional relationship is developed, for hailstorms between only METEOSAT IR parameters, namely cloud - top temperature (CTT) and areal extent of storm (AREA). In the second stage similar empirical relationship is developed for hailstorms between METEOSAT IR and weather radar parameters. Specifically, height of echo top (ETOP) from weather radar is plotted versus AREA, ETOP versus CTT, maximum reflectivity (MXRFL) from weather radar versus AREA and MXRFL versus CTT. Finally, in the third stage only weather radar parameters are used, namely MXRFL versus ETOP. The third stage is basically used to validate and evaluate the storm severity assessment procedure, where the main storm severity criterion is MXRFL. The proposed procedure has been validated using only METEOSAT IR parameters. This validation test showed that the proposed scheme can be operationally employed to assess the storm severity and also to serve as early warnings during the initial stage of storm evolution.

Keywords: Hailstorms, METEOSAT, radar.

1. INTRODUCTION

Early identification and assessment of storm severity is useful in many applications including natural hazards, nowcasting and forecasting and better support of cloud seeding operations for hail suppression (Boe and Jung, 1990) and/or rain enhancement, among others. Storm severity can also be identified and assessed by the magnitude of disastrous impact of storm related phenomena to human activities in urban and rural regions. High – accuracy precipitation estimates on a variety of temporal and spatial scales are also desirable for a wide range of applications including climate monitoring, drought assessment, water balance and hydrological modeling, severe weather and flash flood warnings. Several methods have been

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developed to estimate precipitation from satellite observations, which rely, in general, on the identification of specific cloud types in the visible (VIS) or infra - red (IR) wavelength of spectrum or observe the emission or scattering of microwave radiation by raindrops. Description of the range of techniques can be found in several publications (Barret and Martin, 1981; Barret, 1990; Janowiak, 1992; Negri and Adler, 1993; Adler *et al.*, 1993; Grassotti and Garand, 1994; Porcu *et al*., 1999; Buchroithner, 2002), just to mention a few. Nevertheless, there is a direct linkage between the rain itself and the radiances only in the microwave frequencies (Liu and Curry, 1992; Adler *et al*., 1993), but such data are not yet readily and operationally available.

In this paper, the severity of storms is identified and assessed on the basis of radar parameters such as maximum reflectivity (MXRFL), height of echo top (ETOP) and echo diameter (EDIAM). Specifically, discrimination between hail and rain is commonly based on threshold values of radar maximum reflectivity. Moreover, hail detection is also based on radar maximum reflectivity and height of echo top and empirical relationships have been established between maximum radar reflectivity and hail damages in the field (Thompson, 1991). The objective of this paper is to develop a hailstorm severity assessment scheme based on geostationary satellite parameters, such as minimum cloud top temperature (CTT) and areal extend of cloud cover (AREA), which are considered hailstorm severity indicators and to use radar characteristics to evaluate the hailstorm identification procedure. Application is attempted in Greece. The paper is organized as follows: the first section covers the background information; in the following section the data environment and data processing is described; and then the development of the hailstorm identification scheme is presented. Finally, the performance of the hailstorm identification scheme is tested against independent data and discussed.

2. DATA ENVIRONMENT

There are many regions where conventional weather radars are used and in several cases, such as Greece, there are not continuously recorded digital radar data (Dalezios and Spanos, 1995). However, the routinely available information from meteorological satellites can be sufficient and satisfactory in developing hailstorm severity identification schemes, which utilize weather radar data to evaluate the developed quantitative functional relationships from satellite data. The main sources of data for this study are the National Hail Suppression Program (NHSP) (GNHSP, 1993).

The NHSP of Greece has been designed as an operational cloud seeding program to reduce hail damage over three distinct agricultural areas, namely two areas A1 and A2 in northern Greece and one area A3 in central Greece (Figure1) (Dalezios and Spanos, 1995). The NHSP participated, among others, on airborne cloud seeding operations and has been under the auspices of the Hellenic Agriculture Insurance Organization (ELGA). About half of the selected data referred to area A3 (Figure1), which is covered by an S-band radar at Larisa (LR) (Figure 1). The program has been

operational continuously from 1984 up to now with the exception of 1991 and 1994. Besides operations, a 5-year (1984-88) randomized experiment in one of the areas (A1) (Figure 1) has been designed. Seeding operations have been carried out in these agricultural areas ranked among the first in the list of annual hail damage insurance payouts (Dalezios and Spanos, 1995). The three protected areas are orographically similar in that they are mostly surrounded by complex mountain terrain. This characteristic has a major effect on storm forecast, radar monitoring and seeding techniques.

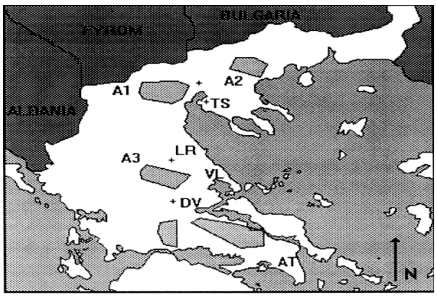


Figure 1. Map of Greece showing the three target areas for seeding operations. (A) indicates hail suppression. Radar positions are denoted by crosses.

Satellite data have been recorded every 30 minutes (VL in Figure1). The satellite data used in this study consisted of METEOSAT IR images with a resolution of 5x5 Km^2, which have been examined in detail by using cloud-top temperature analysis. Radar data in the form of PPI and RHI images were regularly recorded and archived in the three project areas during the 1993 operational season (April-September) of the NHSP (Rudolph and Robitaille, 1994).

3. COMPUTATION RADAR AND SATELLITE PARAMETER

Weather radar data have been used in this analysis and correspond to isolated convective cells, which have been recorded and digitized during the summer of 1993 in Central Greece. There is a time lapse by no more than 15 minutes between the selected radar recordings and the corresponding satellite IR images. Radar echo tops (ETOP) and diameters (EDIAM) as well as the maximum radar reflectivity (MXRFL) have been determined from recorded RHI images. ETOP has been determined at a constant reflectivity level of 15 dBz. EDIAM has been measured in the horizontal distance scale of the RHI images at the same reflectivity level of 15dBz. According to the recording rules, RHI images usually scan the area of maximum reflectivity in the PPI. MXRFL of the radar echo has been recorded at any level below freezing level, i.e., negative temperature, to avoid bright band effects and beam occultation. Freezing level has been determined from sounding data.

Satellite images are analyzed in order to determine the areal extent of individual

storms (B.U.R.L., 1991). A pixel-by-pixel navigation procedure is used to locate the radar echoes, i.e. the storm center, on the satellite grid following the steps:

1. The radar site is determined on the satellite image by using geographical coordinates. Thus, the radar site grid is determined as (iradar, jradar).
2. A cartesian system with origin the radar site and orientation of X axis East-West and Y-axis North-South is then used.
3. The coordinates of the storm center, in the defined Cartesian system, are calculated by the radar measured echo distance (ED) and azimouth (AZIM) as: X_{storm}=ED*sin(AZIM) and Y_{storm}=ED*cos(AZIM)
4. The satellite pixel where the storm center is located is then determined by: $5*(i_{storm} -1) < X_{storm} <5* i_{storm}$ and $5*(j_{storm} -1) < Y_{storm} <5* j_{storm}$

Cases of great uncertainty due to navigation errors and tilting of vertical echo axis have been excluded from the analysis. The cloud area (AREA) is defined as the areal extent of negative brightness temperatures. In cases of more complex negative-temperature areas with neighboring cells, isotherms are drawn around the minimum temperature and the area boundaries of individual cells are determined in the pixels, where the temperature gradient is inversed. In each individual cell the minimum temperature was extracted and considered as cloud top temperature (CTT).

4. HAILSTORM SEVERITY IDENTIFICATION AND ASSESSMENT

In Figures 2 and 3, scatterplots and fitted curves are shown. AREA in Figure 2 is inversely proportional to CTT. The cloud top area is sharply decreasing when minimum cloud top temperature is increasing. Similarly, EDIAM is inversely proportional to CTT (Figure 3) and an analogous proportionality is evident between ETOP and CTT (Figure 3).

In Figure 4, MXRFL is directly proportional to ETOP. The maximum reflectivity - echo top relationship is strongly dependent on the time of the year. Usually not only higher echoes occur in summer, but also for the same echo tops the maximum reflectivities are higher.

The development methodology follows a three-step procedure. The following equations have resulted with the corresponding coefficients of determination (in brackets).

4.1. Step 1: Empirical Functional Relationships Between Satellite Parameters

$$AREA_{hail} = -34.826 + \frac{1378.64}{CTT} \qquad (R^2 = 0.937) \qquad (1)$$

where AREA is the areal extent of storm from satellite picture elements (pixels), CTT

is the minimum cloud top temperature from satellite (°C). In Step 1 the empirical functional relationship between satellite parameters are shown based on the fitted models of Figure 2.

4.2. Step 2: Empirical Functional Relationships Between Radar and Satellite Parameters

$$\mathrm{MXRFL}_{hail} = 34.0188 + 1.34437 * \sqrt{AREA} \qquad (R^2=0.758) \qquad (2)$$

$$\mathrm{MXRFL}_{hail} = 35.3671 + \frac{112.734}{CTT} \qquad (R^2=0.824) \qquad (3)$$

$$ETOP_{hail} = 1.796 + 1.875 * \ln(AREA) \qquad (R^2=0.943) \qquad (4)$$

$$ETOP_{hail} = 17.467 - 3.132 * \ln(CTT) \qquad (R^2=0.945) \qquad (5)$$

$$EDIAM_{hail} = -0.440186 + 2.31068 * \ln(AREA) \qquad (R^2=0.640) \qquad (6)$$

$$EDIAM_{hail} = 19.3657 - 4.02559 * \ln(CTT) \qquad (R^2=0.697) \qquad (7)$$

where, EDIAM is the storm diameter from radar, (km), ETOP refers to storm height, from radar data, (km), MXRFL is the maximum radar reflectivity, (dBz).

In Step 2 the empirical functional relationships between radar and satellite parameters are shown based on the fitted models of Figure 3 for Eq. (2)-(7).

4.3. Step 3: Empirical functional relationships between radar parameters

$$\mathrm{MXRFL}_{hail} = 23.7009 + 2.29483 * ETOP \qquad (R^2=0.703) \qquad (8)$$

In Step 3 the empirical functional relationship between radar parameters are shown based on the fitted models of Figure 4 for equation 8. The derived equations are meaningful and physically justifiable and can be used to make inferences in simulation and forecasting studies and applications. Inferences about storm severity can be made by relating satellite parameters to radar parameters and rain rate. According to those criteria, hail occurred from echoes, which extended up to at least 9Km and had maximum reflectivity between 41 dBz and 54 dBz with a 40% chance of occurrence, and a 90% chance when reflectivity equaled or exceeded 55dBz.

5. DISCUSSION

Several uncertainties were encountered during the data selection and interpretation.

Difference in time recording of weather radar and satellite data resulted in a considerable reduction of the selected data sample. The extracted curves are the expected ones based on previous processing of radar and satellite parameters (Dalezios and Spanos, 1995).

For validating the equations extracted, a new dataset with weather radar parameters was utilized independent and different from the dataset used for the development of the equations. The dataset was extracted by the NHSP (GNHSP, 1993). The satellite parameters are highly correlated and the extracted relationship of Eq. (1) can be used for the estimation of the AREA from CTT. Eqs. (4) and (5) can be used for the estimation of the ETOP and the estimated values can be used as input in Eq. (8) for the estimation of MXRFL

6. CONCLUSIONS

A procedure for identification and assessment of storms severity was presented by establishing physically meaningful empirical functional relationships between satellite parameters, which also indicated a prognostic potential of storm severity. Radar parameters were also extracted and used to develop the relationships between the radar and satellite parameters and radar parameters alone. The severity of storms was defined on the basis of radar parameters, such as MXRFL, ETOP and EDIAM.

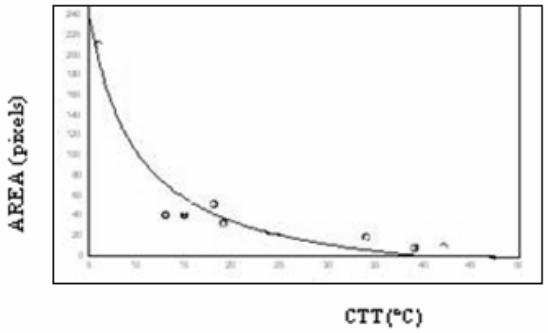


Figure 2. Scatterplot and fitted lines between Cloud Top Temperature (CTT) and cloud Area.

The analysis indicated linear and square-root relationships between MXRFL and ETOP for hailstorms. Moreover, logarithmic equations were found to relate ETOP and CTT from satellite data. Radar EDIAM was related to CTT from satellite data with a logarithmic and relationship for hailstorms. Logarithmic relationships were derived between radar EDIAM and satellite AREA for hailstorms. Furthermore, the analysis of results indicated a reciprocal relationship between AREA and CTT from satellite data for hailstorms. The relationship between maximum reflectivity and the two satellite parameters was also derived.

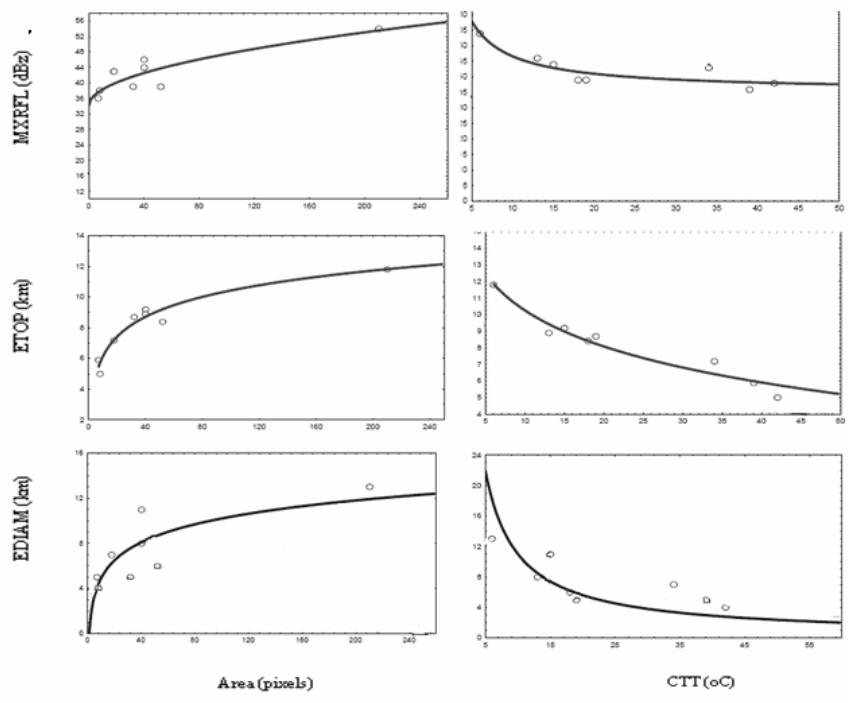


Figure 3. Scatterplot and fitted lines between weather radar and satellite parameters.

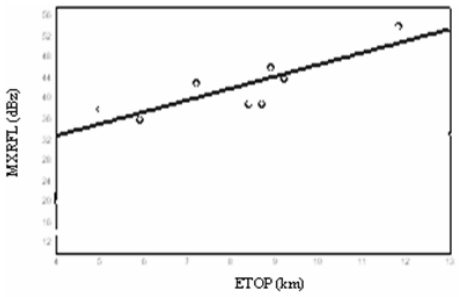


Figure 4.: Scatter diagram between computed Maximum Reflectivity (MXRFL) and Echo Top (ETOP).

The first relationship (Eq. (1)) can be used in the estimation of cloud area from satellite IR temperature and can be used as input to Eq. (6) for the estimation of echo diameter. Two radar parameters related to storm severity, i.e., echo top and echo diameter, could be directly estimated from cloud top temperature using the Eqs (5) and (7). By employing the Eqs (2) and (3) maximum reflectivity of the echoes can be estimated and inferences can be made about hail occurrence. Some of the developed equations were evaluated against independent datasets. Lack of simultaneous weather radar and satellite data restricts the validation of all the developed equations. The developed relationships can be applied operationally to assess the storm severity.

ACKNOWLEDGEMENTS

The authors wish to express their gratitude to Hellenic Agricultural Insurance Institute (ELGA) for providing the radar data. Recording and storage of satellite data started on July 1993 when a Meteosat/PDUS (Primary Data User Station) was installed at the Laboratory of Agrometeorology of University of Thessaly in Volos, Greece.

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Use of Remote Sensing and GIS to Analyze Lineaments as Indicator to Groundwater Exploration in Lebanon

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Abstract: In Lebanon, several approaches have been utilized to determine potential groundwater accumulation zones. However, many of them failed due to the improper application of study. Remote sensing in combination with GIS is a viable alternative, however, where linear features *(lineaments*) are analyzed from satellite images and entered into a GIS. Lineaments represent fracture systems that often govern groundwater storage. The lineament maps in this study were obtained from *Landsat 7 ETM+* images, and were processed using *Erdas Imagine* software. The objective of this paper is to find out a relationship between the location of productive water wells and the geometric properties of lineaments. The use of GIS techniques allows the plotting of potential wells on the lineament map in order to determine any relationship. The highest number (82%) of the potential water wells in the area of study were found to be located on/or at distance less than 250 m from a fault-lineament. Thus, fault-lineaments can be used as a reliable indicator for groundwater exploration, notably in a terrain dominated by fractured carbonate rocks, such as the case in Lebanon.

Keywords: Lebanon, lineaments, satellite images, wells.

1. INTRODUCTION

Due to the increase in population in Lebanon, the water supply is under increasing demand. Groundwater has become a major source of water, notably in areas where river courses are far aways from human settlements. In this respect, a large number of water wells have been drilled, many of which are privately owned and randomly dug. Neverthless, a large number of known wells have failed as either the yield was not enough to cover the cost of digging or sometimes they were found to be dry. However, as a normal practice, geologists follow the aligement of faults to locate favorable sites for groundwater borehole drilling. Nevertheless, the elements of this

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assumption are not understood and a reliable procedure must be developed to locate new water wells.

The development of remote sensing has provided an essential tool for terrain analysis, notably to identify hydrogeological signatures that are able to give first hand information for futher detaled investigation and field survey. Mainly these signatures are the linear features that can be observed on satellite images, and termed lineaments.

Analysis has shown that many of the lineaments can be attributed to rock fractures (Woodruff *et al.*, 1982; Degnan and Clark, 2002). Moreover, linear features were proved to have a great significance in creating groundwater-transporting routes, as well as being high water recharge potentials zones (Shaban *et al.*, 2006).

This study tackles selective zones from Lebanon (Figure 1), and aims to establish relationships between lineament characteristics and groundwater potential zones. In a broad sense, it is to understand the influence of lineaments on groundwater storage in different aquiferous rock formations.

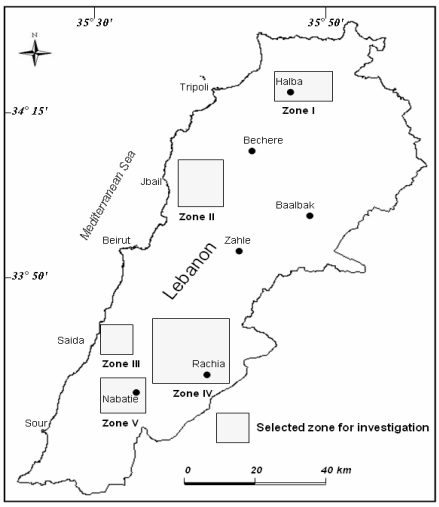


Figure 1. Location map of the selected zones for investigation.

1.1. Lineaments

A lineament is a mappable, simple or composite linear signature of a terrain surface, whose parts are aligned in a rectilinear or slightly curvilinear relationship (O'Leary *et al.*,1976). However, many features on terrain surfaces have the same geometric properties as lineaments, thus creating confusing signatures "*false lineaments*". In order to minimize this error, a GIS application can be applied by overlaying the lineament maps with the relevant topographic maps, taking notice of linear features such as roads, pipelines and terraces. Any coincidence with linear features is

interpreted as being non-geologic.

Several geological studies have utilized lineament maps, especially where fissure systems are developed (Edet *et al.* 1998; Gustafsson, 1994; Shaban, *et al.*, 2006). Yet, there is debate on distinguishing lineaments as fissure systems (Berger, 1984) or as fault aligenments (Waters *et al.*, 1990). In combination with field verification, discrimination of lineaments on satellite images was carried out as follows:

1. Faults on satellite images or aerial photos have acute and sharp shadows, which do not exist in fissures,
2. Faults always occur over relatively long distances, i.e. several kilometres, whereas fissures do not exceed 2 km,
3. Faults may cut two different lithologic units, while fissures are normally found within the same lithology,
4. Faults tend to follow preferred orientations and sets, e.g., same direction and parallelism, cut within valleys, etc.,
5. Long faults show disconnected delineation and appear as linear segments following the same orientation,
6. Fissures terminate at rigid structures or lithologies, while faults do not.

1.2. Fractures and Groundwater

The two types of lineaments, i.e, fissures and faults have two different hydrologic characteristics with respect to the groundwater regime. The groundwater regime is represented either by groundwater flow or storage, and sometime by both. The coincidence between groundwater storage and fissure traces on the terrain surface of carbonate aquifers is often cited, and in particular that the fissures are underlain by zones of localized weathering that increases permeability and porosity of the carbontes (Rauch and LaRiccia, 1978; Bazynski *et al.*, 1983; Teeuw, 1995). Therefore, great attention was given to the coincidence between the location of fissure lineaments and sucessful water wells. Erroneous results often exist and fissure lineaments are found to be poorly related to the productivity of water wells. Some workers have rejected this assumption (Parizek, 1967). In many studies, the erroneous results were attributed to the incorrect identification of lineaments from remotely sensed data, or due to the failure of the procedure of lineament analysis, such as the calculation of their density, morphometric characteristics, etc.

Many studies on groundwater exploration using remote sensing tools were dependant on manipulating different components. In this respect, the lineaments component was frequently the major element, while the other components used were slope, lithology, karst, drainage, soil and land use/cover. Furthermore, many supplementary factors were added to support this groundwater exploration approach (El-Shazly *et al.*, 1983; El-Baz, 1992; Per Sandra *et al.*, 1996; Das, 2000; Shaban *et al.*, 2006).

A fault lineament has two roles in the hydrologic regime. Firstly, it can serve as an hydraulic boundary, notably between two different rock lithologies, e.g., permeable and impermeable. Secondly, a fault may work as a subsurface conduit transporting

water for long distances, for example in Lebanon (> 30 km).

Several hydrological elements must be considered when locating groundwater wells. Groundwater exploration is not completely dependent on the existence of lineament whether they indicate fault or fissure systems. For example, in many places of dense fissured surface terrain, surface water recharge is high, but the acute dip (inclination) of rock bedding planes serves to transfer water to another place where no fissure zone exists on the terrain surface (Shaban, 2007). Furthermore, if a fault lineament is situated between two rock lithologies of similar type, notably massive carbonates, it will not serve as a hydraulic boundary and water along this fault will diffuse in a chaotic direction opposite to its alignment.

2. METHOD OF ANALYSIS

For lineament mapping, Landsat 7 ETM+ (30 m x 30 m resolution) images were processed using ERDAS Imagine software, which has the ability to detect "edge" features, i.e., lines of small and large fractures. Several optical and digital procedures were applied, especially directional filtering, contrast and sharpness. In addition, single band and multi-band enhancement were carried out by interrelating each three bands as one set. Often, the combination of bands 2, 5, and 3 of Landsat 7 ETM+ better identifies linear features. In addition, the thermal band, i.e., band 6 (120 m x 120 m resolution) was applied to inspect wet horizons that reflect fracture zones.

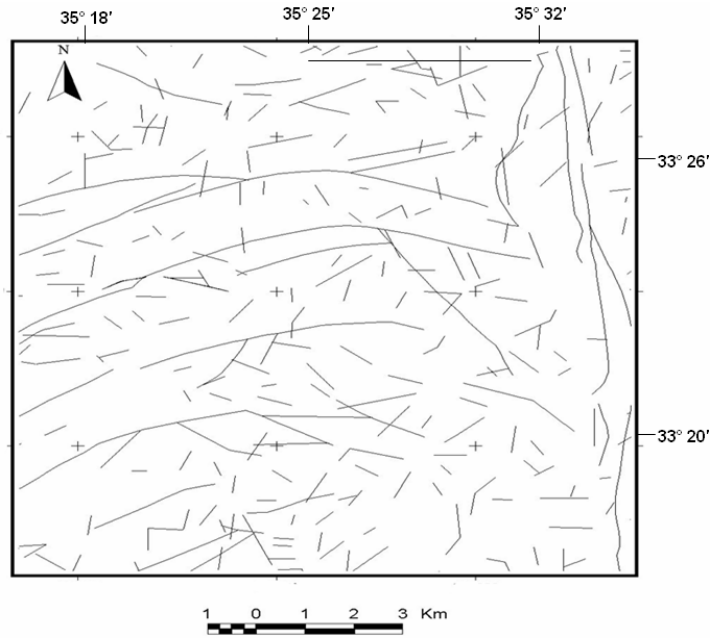


Figure 2. Example showing a lineament map for the selected zone V.

The non-geologic linear features were eliminated from the lineament map (Figure 2). For a more perfect reliability, a field survey was carried out to verify different lineaments. This was accomplished in combination with previous geologic maps for the area (Dubretert, 1953).

A map showing the distribution of water wells was prepared for five selected zones in Lebanon. These zones represent different physiographic and geologic regions. Among these zones, 134 water wells of different groundwater productivity were recorded.

2.1. Lineament Analysis

From the lineament map, a detailed geometric and morphometric analysis was carried out in order to obtain criteria for defining lineaments that can be used to compare with the location of productive wells after the morphometry has been established. This needs a diagnostic analysis of the lineament map. The majority of these criteria fall into three lineament properties. These are: the location of productive well with respect to:

- Different lineament density zones
- Different lineament frequency zones
- Fault aligenment

Several authors have incorporated these criteria in their hydrological studies and proved their use indiagnosing lineament behavior (Teeuw, 1995 and Edit *et al.*, 1996) and including lineaments frequency (Greenbaum, 1985· El-Baz and Himida, 1995· Shaban *et al.*, 2007). Lineaments density (L_d) is the sum of lineament length in a specific area. It is expressed as:

$$L_d = \sum \frac{L_l}{A} \qquad (1)$$

where,
ΣL_l: total length of lineaments,
A: the area in km^2
While Lineament frequency (L_f) is the visible number of lineaments per unit area (Greenbaum, 1985) and is expressed as:

$$L_f = \sum \frac{L_n}{A} \qquad (1)$$

where, ΣL_n : the total number of lineaments

In order to present zones with different lineament density and frequency on a map, a graphic method of analysis was followed to produce maps with contour lines that express lineament density and frequency. This was obtained by dividing the area of study into grids from which a frame is produced. The size of a frame depends on the spatial distribution of the linear features. In this study, a frame of 5 km x 5 km (i.e., 25 km²) was selected. The total length of lineaments (for density) in each frame was

measured, while the number of lineaments in each frame was calculated (for density). Consequently, the resulting values were plotted in the middle of the frame. For each four neighboring frames, the average values were again calculated, and plotted in the mid-point of the composited frame. The resulting midpoints represent the value of the lineament density/ or frequency. These points are used to illustrate the contour lines that express the lineament density or frequency (Figure 3). For the density and frequency criteria, a five-class map was obtained. Each class represents a range of lineament (Table 1). In addition, the distance from a fault lineament was also classified into five buffer zones (Table 2).

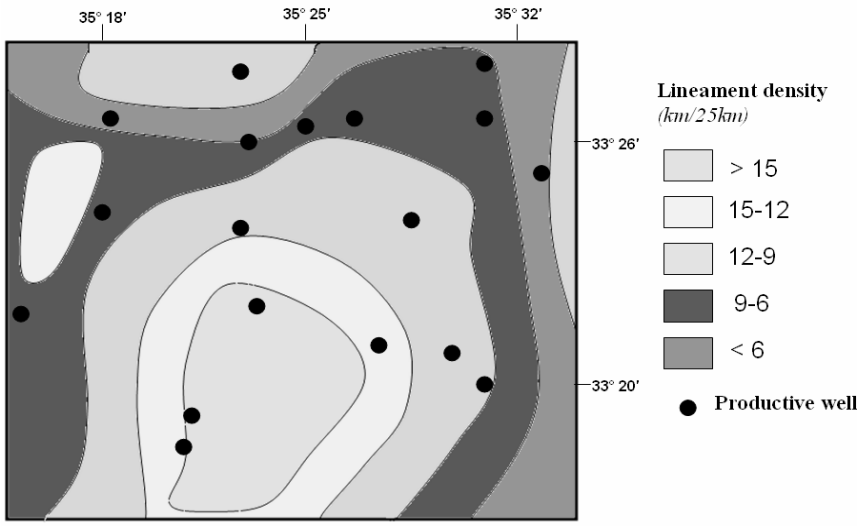


Figure 3. Example showing the expression of lineament density in the zone V.

The Fault-lineament (Fl), is another criteria to be compared with the location of productive wells, and was determined from the lineamenet map. These fault lineaments were discriminated from fissure lineaments following the procedure described in Section 1.1.

Table 1. Lineaments density and frequency classes.

Lineament density *(km/25 km^2)*	Lineament frequency *(segment/25 km^2)*	Description
>15	>20	Very high
15-12	20-15	High
12-9	15-10	Moderate
9-6	10-5	Low
<6	<5	Very low

Table 2. Fault lineament buffer areas.

Distance from a fault lineament *(m)*	Description of buffer
< 100	Very close
100-250	Close
250-500	Relatively close
500-750	At a range
>750	Away from the fault

2.2. Comparative Analysis

Well productivity was tabulated to show different water discharge from wells in the five selected zones (Table 3). These classifications (Tables 1, 2 and 3) were obtained from the maximum and minimum values that exist in the selected region. The classifications were also combined with previous works. The comparison of these maps was undertaken following a superimposing procedure using GIS (Figure 3) with *Arc-Info* software.

Table 3. Classification of different water well productivity.

Well productivity *(l/sec)*	Description
≥20	Very high productivity
19-15	High productivity
14-10	Moderately productive
9-5	Low productivity
<5	Very low productivity (or Wildcat)

3. RESULTS AND CONCLUSION

The 134 water wells in the five selected zones of varying productivity, were plotted on different lineament maps.. Of this number, there are 78 wells with high and very high productivity levels. The geographic distribution of these productive wells was found to be within specified zones with respect to lineament criteria. However, no reliable coincidence was found between the location of productive wells and lineament density and frequency. Therefore, only 28 and 36s well were located in areas of high and very high lineament density and frequency respectively. This equals 36% and 46% for each of these criteria, which is insufficient to be used reliably.

The fault-lineaments, which are numerically available, were expressed graphically to show their relation with productive well locations. Figure 4 shows the relationship between water well productivity and their proximity to faults.

It is obvious that there is a direct relationship between both components. Thus, 29 wells, with very high productivity, are located at a distance less than 100m from a fault lineament; 35 wells, (wells with high productivity), are located at a distance between 100 and 250 m from a fault lineament, thus totalling 64 wells, which is an equivalent of 82% of the total number of productive wells. As a conclusion and from a hydrogeological point of view, it can be consided that lineaments related to with faults have an influence on groundwater storage.

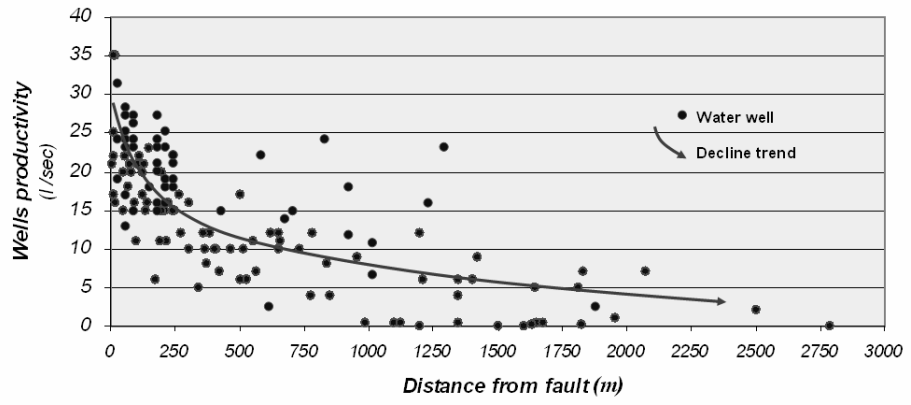


Figure 4. Water wells productivity and their distance to fault lineament.

Those lineaments attributed to fissure systems are not related to the productive wells and their influence relates only to the recharge of water into the deep aquifers (Shaban *et al.*, 2006). This is mainly attributed to the fact that faults in Lebanon are often of the wrench (strike slip) fault type, which creates a lateral displacement and resulting in a a contact between different litologies. Most of these lithologies are of different permeabilities such as limestone and marl. Therefore these faults because of the lithologies serve as hydraulic barriers or as groundwater storage zones. This explains why the high productivity wells in Lebanon are found at a short distance from faults.

This study introduces a reliable relationship of lineament occurrence as indicative elements in groundwater exploration. It also shows the importance of remotely sensed data in geological studies in hydrogeology.

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Trends in Actual and Potential Evapotranspiration over Greece

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Abstract: To determine the influence of global changes on surface hydrological fluxes the feedback mechanisms between the land surface and atmosphere within the framework of Bouchet's complementary relationship between potential and actual evapotranspiration are investigated. Using meteorological observations and the advection-aridity model actual, potential and wet environment evapotranspiration are estimated for 28 or 16 stations in Greece for the period 1974 - 2001 or 1957 - 2001. Trends in actual and potential evapotranspiration are not increasing or decreasing uniformly over Greece. Directions of trends in actual and potential evapotranspiration (being in most cases opposite) are successfully used to offer observational evidence of the complementary relationship.

Keywords: Actual, complementary relationship, evapotranspiration, potential, trends.

1. INTRODUCTION

The decreasing trend in pan evaporation, observed in several countries (Chattopadhyay and Hulme, 1997; Golubev *et al.,* 2001; Lawrimore and Peterson, 2000; Roderick and Farquhar, 2004, 2005) has captured attention, especially because the trend seems contrary to concurrent increasing trends in ET (Szilagyi *et al.,* 2001), temperature (Folland and Karl, 2001), and precipitation and cloudiness (Karl *et al.,* 1996). These increases in precipitation and cloud cover could only occur if surface evaporation, as the only source of atmospheric water vapour also increased. From a surface in which there is abundant moisture, actual evapotranspiration ETa equals potential evaporation ETp. Potential evapotranspiration is defined as the amount of evapotranspiration from a large uniform land surface with adequate moisture such that available energy is the limiting factor. In a region without adequate moisture, ETp cannot be sustained, and as a result ETa is less than ETp. The decrease in ETa is expected to have a negligible effect on net radiation and it will rather, primarily affect the temperature, humidity, and turbulence of the air near the ground. The energy that is not used for evaporation manifests itself as sensible heat flux and, in the absence of local oasis effects, results in an increase in potential

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evapotranspiration in an amount equal to the energy given up by ETa with no change in the overall energy budget.

This complementary relationship between actual and potential evaporation, first proposed by Bouchet (1963) and applied by Morton (1975) in estimating evapotranspiration from climatological observations, is the basis for the Brutsaert and Parlange (1998) solution to the evaporation paradox, which turns on the relation of ETpan to ETa. Depending on moisture availability in the region around the pan, these two variables may be nearly identical or very different, but they are nevertheless related. The evapotranspiration rate in purely energy-limited areas is referred as wet environment evapotranspiration ETw and it is related to ETa and ETp. ETw and ETp may be expressed as functions of the radiative budget Qn and the advective budget indicated by the drying power of the air Ea as:

$$ETw = f(Qn) \text{ and } ETp = f(Qn, Ea) \quad (1)$$

The temporal trend in ETa (denoted by δETa) can then be expressed as a combination of trends in its component budgets (δQn representing the time-rate of change in the radiative energy budget and δEa representing that in the advective or vapor transfer budget (i.e., in the regional drying power of the air) as follow:

$$\delta ETa = \left(2 \frac{\partial \mathrm{ET}\ w}{\partial Qn} - \frac{\partial ETp}{\partial Qn} \right) \delta Qn - \frac{\partial ETp}{\partial Ea} \delta Ea \quad (2)$$

ETw and ETp are obtained –typically, from some variation of the Priestley –Taylor equation and Penman equation (Brutsaert and Stricker, 1979), respectively. In traditional paradigms, ETa is estimated as a monotonically increasing function of ETp or of ETp and moisture availability, implying that trends in ETa are a positive function of trends in ETp and therefore similarly in the driving power of the air Ea.

In this study, actual evapotranspiration is computed for the period 1974-2001. Trends in ETa are examined for 28 stations over Greece. The corresponded trends of the original components of ETa (radiative and advective budget) are also examined. The complementary relationship is examined as consistent with climate trends in potential evapotranspiration, precipitation, temperature, and water vapor concentration.

2. ANALYSIS AND RESULTS

Daily measurements of temperature, humidity wind velocity and sunshine hours obtained from the National Meteorological Service for 28 stations in Greece are used for estimating actual (ETa), potential (ETp) and wet environment (ETw) evapotranspiration for the period 1974-2001. Table 1 presents the characteristics of the stations and the mean annual values of ETa, ETp, and ETw. It is obvious that there are not great differences between the estimates obtained from daily or monthly values of the meteorological parameters. Thus, monthly values of the parameters available for 16 of the 28 stations for a longer period 1957-2001 are used in order to investigate trends in ETa, ETp, ETw, solar radiation (RS), long wave radiation (RL), relative humidity (RH), vapour pressure (E), wind velocity (U) and precipitation. Table 2 presents analytically the slopes of the trends of yearly values of the mentioned parameters bETa, bETp, bETw, bRS, bRL, bRH, bE, and bU, respectively for all stations. Table 3 shows the number of stations with negative or positive trends of the considered parameters and the corresponding ones with the significant trends at the

95% level. Incident solar radiation (RS) is examined, since it provides the major energy input to any evaporation process, and is an excellent estimator of Qn. A significant positive trend is observed in only one station for the period 1974-2001. RL has a similar behaviour. Humidity is a key factor in Ea, and thus RH and E are taken into account. RH does not indicate a certain trend, while E and U seem to be rather slightly increasing. However, in order for humidity to reveal anything about ETp or ETa, it must be multiplied by a functional measure of the speed of the over passing air, generally in the form of an empirical function of wind speed f (U), revealing Ea, the advective budget. Thus, trends in Ea can be due to trends in wind speed U, vapor pressure deficit or some combination of both.

Table 1. Characteristics of the stations and annual potential (ETp), actual (ETa) and wet environment (ETw) evapotranspiration obtained from monthly and daily measurements during the period 1974 – 2001.

Stations	Lat.	Long.	Alt. (m)	Monthly			Daily		
				ETp	ETa	ETw	ETp	ETa	ETw
				mm/yr	mm/yr	mm/yr	mm/yr	mm/yr	mm/yr
Serres	41° 31'	22° 58'	34.0	1153.14	932.58	1043.24	1167.34	915.31	1041.33
Mikra	40° 31'	22° 58'	4.0	1348.53	791.28	1070.36	1374.57	781.29	1077.94
Alex/poli	40° 51'	25° 26'	3.5	1242.17	869.36	1055.36	1253.86	863.96	1058.78
Kozani	40° 17'	21° 47'	625.0	1193.70	946.66	1070.11	1144.68	880.19	1012.44
Kerkira	39° 37'	19° 55'	2.0	1276.73	1042.48	1159.53	1294.69	1056.16	1175.43
Ioannina	39° 40'	20° 51'	483.0	1135.66	944.58	1039.85	1139.67	924.57	1032.12
Trikala	39° 33'	21° 46'	108.6	1341.63	921.88	1131.65			
Larisa	39° 39'	22° 26'	71.0	1260.16	905.05	1083.14	1269.05	914.28	1091.67
Aghialos	39° 13'	22° 48'	12.2	1339.70	889.18	1114.56			
Mitilini	39° 04'	26° 36'	4.2	1519.97	882.24	1201.00	1548.41	914.59	1231.50
Agrinio	38° 37'	21° 33'	24.0	1226.14	1106.21	1166.32	1238.53	1118.49	1178.51
Lamia	38° 51'	22° 24'	14.3	1178.95	1030.36	1104.64			
Adravida	37° 55'	21° 17'	11.1	1300.06	1128.51	1214.28	1326.76	1252.56	1239.66
Argostoli	38° 11'	20° 29'	21.2	1394.96	1054.45	1224.43	1424.32	1075.36	1249.84
Chios	38° 21'	26° 09'	4.0	1508.36	909.00	1208.92	1561.19	918.10	1239.64
Tripoli	37° 32'	22° 24'	652.4	1303.45	865.05	1084.31	1303.58	872.98	1088.28
Elliniko	37° 54'	23° 45'	10.0	1633.89	761.17	1197.46	1668.21	771.34	1219.78
Kalamata	37° 04'	22° 06'	222.0	1348.02	1075.18	1211.65	1373.78	1090.21	1231.99
Naxos	37° 06'	25° 23'	9.0	1658.16	841.73	1249.61	1675.13	885.23	1280.18
Methoni	36° 50'	21° 42'	53.0	1424.01	1060.07	1241.77	1443.41	1082.28	1262.84
Milos	36°44'	24° 26'	164.0	1691.52	765.99	1228.23	1688.26	830.25	1259.25
Kithira	36° 08'	23° 01'	165.8	1597.02	866.47	1231.62	1605.33	895.63	1250.48
Chania	35° 30'	24° 02'	62.0	1473.54	1058.10	1266.15	1218.56	1369.49	1294.02
Rhodes	37° 24'	28° 05'	34.7	1600.53	1006.34	1302.65	1615.87	1020.70	1318.29
Iraklio	35° 20'	25° 11'	37.0	1678.49	857.90	1268.52	1720.47	870.71	1295.59
Ierapetra	35° 00'	25° 44'	10.0	1898.00	769.09	1333.42	2038.53	705.13	1371.83
Sitia	35° 12'	26° 08'	114.5	1594.79	987.07	1291.04			
Tymbaki	35° 00'	24° 46'	5.0	1693.75	902.61	1298.49			

Only 5 of the 28 annual ETp show the decreasing ETp trend reported widely in the literature (e.g. Chattopadhyay and Hulme, 1997; Brutsaert and Parlange, 1998; Roderick and Farquhar, 2005) at the significant level. Eight ETp of the 17, have significant increasing trends. The distribution of negative and positive ETa and ETp trends across the quadrants of Figure 1 reflects the spatial distribution of their trends with opposite

direction. Overall, 10 stations lie below the x-axis and 9 stations lie to the left of the y-axis reflecting the predominantly decreasing ETp and increasing ETa or vice versa.

Table 2. Trends (slopes) of annual actual (ETa), potential (ETp), wet environment evapotranspiration (ETw), solar radiation (RS), long-wave radiation (RL), relative humidity (RH), vapor pressure (E), wind speed (U), and precipitation (P) for the period 1974-2001.

Station	b ETa mm/yr^2	b ETp mm/yr^2	b ETw mm/yr^2	b RS mm/yr^2	b RL mm/yr^2	b RH	b E mbar	b U m/sec	b P mm/yr^2
Serres	3.668	-0.347	1.661	-0.686	-1.208	0.144	**0.063**	-0.011	-1.288
Mikra	0.372	2.486	1.391	1.066	0.511	**-0.170**	**0.019**	-0.014	-3.025
Kozani	-3.402	**4.508**	0.540	0.000	0.821	**-0.262**	-0.012	0.001	-3.518
Kerkira	0.872	1.781	**1.332**	0.471	-0.164	0.003	**0.039**	0.000	-4.511
Ioannina	**-6.165**	**8.077**	1.011	1.325	0.577	-0.032	0.009	**0.056**	-8.785
Trikala	**-3.639**	**4.895**	0.635	0.000	-0.686	-0.010	0.012	**0.027**	-3.621
Larisa	-0.668	1.628	0.496	-0.821	-0.339	-0.128	0.005	-0.005	-2.147
Aghialos	**-3.668**	**6.391**	1.354	0.993	0.142	-0.045	**0.023**	**0.036**	-4.963
Mitilini	**13.823**	**-6.990**	**3.435**	1.978	**-1.292**	**0.338**	**0.114**	**-0.032**	-4.388
Agrinio	**3.727**	-1.263	1.237	0.241	-0.314	0.014	**0.034**	**-0.020**	-7.203
Lamia	-1.011	3.665	1.321	0.157	-0.332	-0.142	**0.020**	**0.010**	0.432
Adravida	**3.402**	0.748	**2.128**	1.723	-0.263	0.085	**0.054**	0.004	-7.005
Argostoli	1.464	1.927	**1.723**	1.851	0.088	**0.106**	0.038	0.014	-4.476
Chios	**5.891**	-3.391	1.259	0.310	-0.157	0.000	0.039	**-0.049**	-2.093
Tripoli	-0.124	-0.252	-0.164	1.252	1.347	**-0.217**	**-0.037**	**-0.025**	-7.253
Elliniko	1.964	0.230	**1.037**	-0.219	-0.281	-0.055	**0.020**	**-0.023**	-2.297
Kalamata	**3.037**	**-2.548**	0.168	-1.285	-0.661	-0.036	**0.021**	**-0.033**	-2.735
Naxos	0.318	**3.267**	**1.832**	**2.719**	0.792	-0.033	**0.019**	0.002	-4.710
Methoni	**4.497**	-1.099	**1.661**	1.935	0.310	0.026	**0.033**	**-0.027**	-2.871
Milos	**-6.023**	**6.862**	0.358	0.000	**0.690**	**-0.216**	-0.009	-0.004	-0.834
Kithira	**5.037**	**-2.263**	**1.351**	0.183	-0.402	0.030	**0.037**	**-0.035**	-1.637
Chania	**-8.483**	**9.125**	0.350	0.398	**1.807**	**-0.131**	-0.001	**0.049**	-3.319
Rhodes	**9.636**	**-4.990**	**2.332**	1.077	-0.931	**0.169**	**0.087**	**-0.033**	0.945
Iraklio	1.106	1.416	1.245	1.770	0.599	-0.040	0.014	-0.008	-2.235
Ierapetra	**5.632**	-2.307	**1.690**	0.752	-0.420	-0.029	**0.035**	**-0.044**	**-8.314**
Sitia	**-7.187**	**9.512**	1.172	**2.873**	0.511	**0.120**	0.015	**0.100**	**-5.297**
Tymbaki	**7.921**	**-5.902**	1.026	0.398	-0.719	**0.173**	**0.044**	**-0.032**	**-6.814**

Table 3. Number of stations with positive or negative trends of actual (ETa), potential (ETp), wet environment evapotranspiration (ETw), solar radiation (RS), long-wave radiation (RL), relative humidity (RH), vapor pressure (E), wind speed (U), and precipitation (P) for the period 1957-2001 or 1974 –2001. (*Sunshine hours are missed in 1957- 1973 period).

	1957-2001 (16 stations)		1974-2001(28 stations)	
	negative	positive	negative	positive
ETa	10 (5)*	6 (4)*	10 (6)	18 (10)
ETp	7 (4)*	9 (5)*	11(5)	17 (8)
ETw	10 (5)*	6 (2)*	1 (0)	27 (11)
RS			4 (0)	21 (1)
RL	8 (6)*	8 (1)*	15 (0)	13 (2)
RH	9 (8)	7 (3)	17 (6)	11 (6)
E	11 (6)	5 (3)	4 (1)	24(16)
U	10 (6)	6 (5)	17 (11)	11 (6)
P	26 (11)	2 (0)	26 (3)	2 (0)

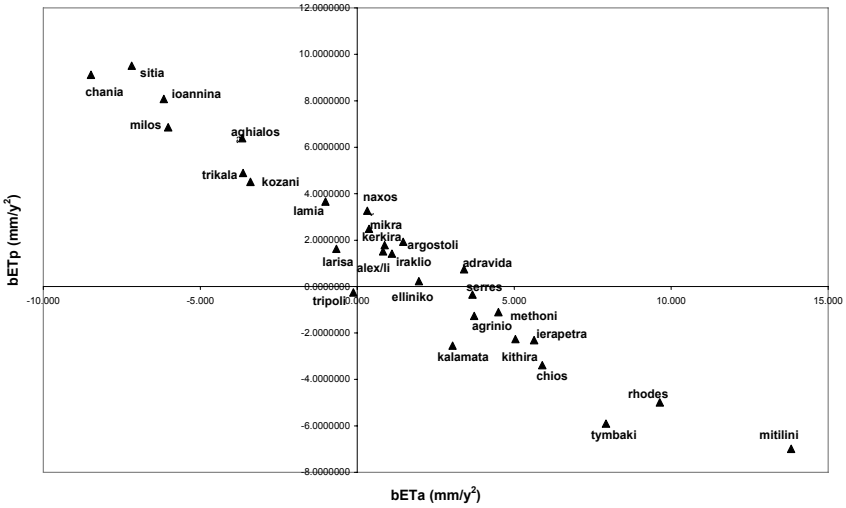


Figure 1. Trends in mean annual actual (ETa) and potential (ETp) evapotranspiration.

The lower right and the upper left quadrant where the majority of the stations fall, the directions of the trends are in conflict. Figure 1 helps for explaining the relation of ETp to ETa. Decreasing ETp is clearly not an indication of decreasing ETa. To the contrary, decreasing ETp is often associated with increasing ETa and this can only be explained in the context of the complementary relationship. Figure 2 shows the trends in mean annual ETa and ETp spatially interpolated from monthly time-series. The complementarily of ETp and ETa is also obvious in most areas of the country.

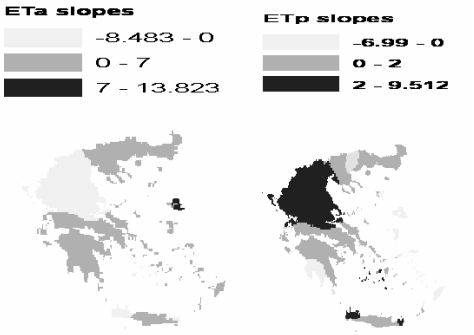


Figure 2. Trends in mean annual actual (ETa) and potential (ETp) evapotranspiration spatially interpolated from monthly time- series expressed in mm/y².

The relationship between ETp and ETa is apparent in Figure 3a which contains 28 data pairs, each consisting of an annual measure of ETp and an annual measure of ETa corresponding to its certain value of precipitation. The highest values of ETp occur at the left of the graph, in water-limited environments, and are matched with the lowest values of ETa. Moving to the right, precipitation increases, the water-limitation on the evaporative progress gives way to an energy-limitation, and ETp decreases as ETa increases .In general, ETp and ETa rates converge in the wettest areas. The evapotranspiration rate in

pure energy –limited areas referred to here, as wet environment evaporation (ETw) is also shown in Figure 3. Figure 3 closely matches the theoretical shape of the complementary relationship between regional–scale ETp and ETa (Bouchet, 1963). All available energy not taken up by ETa, goes to heat and dry the over passing air, driving ETp above ETw by amount that ETa falls below it. A potential humidity index (PHI) is defined as the ratio of annual precipitation to wet environment evapotranspiration and the data pairs of the ETa and ETp are also plotted as a functions of PHI in Figure 3 (b). ETp and ETa, in Figure 3 so closely displaying complementarily, provide strong evidence for the complementary relationship hypothesis.

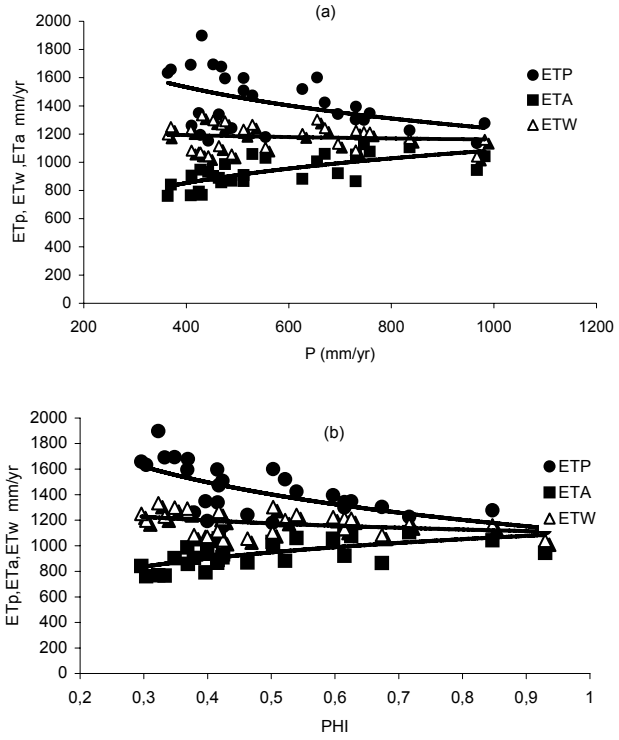


Figure 3. Point annual potential (ETp), actual (ETa) and wet environment (ETw) evapotranspiration of precipitation P (a), or potential humidity index (PHI) (b).

Figure 4 presents the relationship between average annual values of ETp, ETw and ETa as functions of time for selected stations (Mitilini, Tymbaki) in (a) and (b) respectively. The general structure is symmetrical as ETp - ETw decreases, ETa - ETw becomes increasingly more positive, yielding the complementary structure similar to that proposed by Bouchet (1963) and already illustrated in Figure 3. It is interesting to note that for the last few years of the study period ETp is nearer to saturated conditions (ETp ≈ ETw), suggesting that evapotranspiration from saturated surfaces under conditions of minimal advection is similar to what literature suggests. Figure 4 depicts the behavior that has been characterized as paradoxical, wherein decreasing ETp is matched by increasing ETa. The trends in the driving components of ETa and ETp have already been indicated in Table 2. Although the data pairs plotted in Figure 4 correspond to different years, each pair

can be considered as showing in the expected instantaneous response of ETp and ETa to changes in moisture availability. This is so because the magnitude of ETw is almost relatively constant. Clearly Figure 4 shows that the complementary relationship is also observable for each individual area. This is in accordance with the findings of Ozdogan and Salvucci (2004). In theory, a decreasing radiative budget (Qn), due to increasing cloudiness, for example, lowers the dashed lines representing both ETa, while increasing Qn raises them. A decreasing advective budget (indicated by the drying power of the air, Ea), due to increasing precipitation, for example, moves a datum pair conergently to the right along their respective dashed lines, upwards for ETa and downwards for ETp whereas increasing Ea moves a datum pair divergently to the left, downwards for ETa and upwards for ETp. To understand trends in ETp, trends in ETa (the complement to ETp as indicated in Figures 3 and 4) are examined.

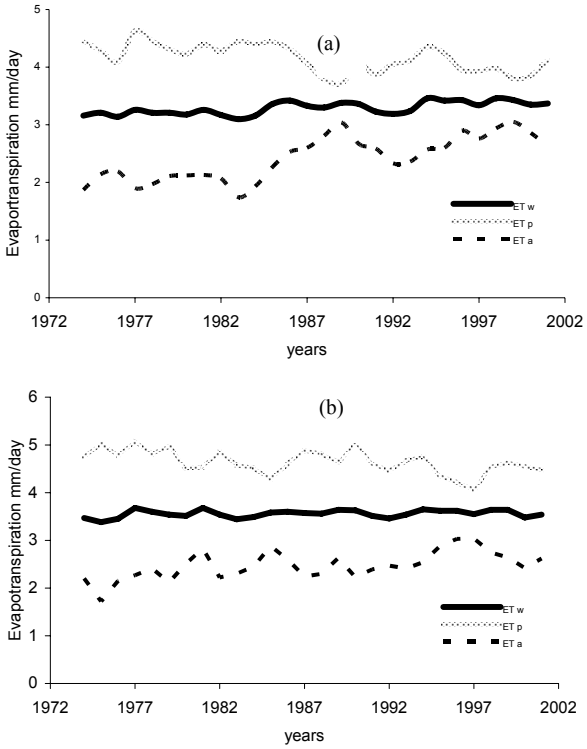


Figure 4. Mean annual potential (ETp), actual (ETa) and wet environment (ETw) evapotranspiration as functions of time in Mitilini (a) or Tymbaki (b).

3. CONCLUSIONS

Based on the analysis presented here it is concluded that an increase or decrease in long term actual evapotranspiration is not detectable uniformly over Greece. Directions of trends of actual and potential evapotranspiration are mainly opposite over Greece providing observational evidence of the complementary relationship. To understand trends in potential evapotranspiration one must examine ETa, since ETa is the

hydrological flux of primary interest (Ohmura and Wild, 2002).

An estimation procedure that ignores the complementary effects of regional advection would fail to represent the physical processes involved in a trend in ETa, and may result in misdiagnosis of the cause of the trend. Although ETp is a very useful concept, it can be misleading if used by itself to indicate climatic trends.

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Small Scale Modelling of River Subcatchments: the Kleine Ronsebeek Case Study

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Abstract: Different simulation software tools were evaluated for microscale modelling the Kleine Ronse brook. The Infoworks modelling package was selected based on the objectives that were put forward by Leiedal, the intermunicipal society of the Kortrijk region. The Kleine Ronse brook was implemented in the software. Initial calibration of the model was based on a 1 month measurement campaign (August-September 2006) and a limited number of measurements was performed in the winter period (February 2007). Flow rate, ammonium concentration and COD concentration were measured at 3 points in the catchment area. Further, rain data recorded in July 2006 was used. Additional measurements will be necessary before the model can be used for scenario analysis, although initial calibration and simulation results look promising.

Keywords: Decision support, InfoWorks, river basin modelling, small scale.

1. INTRODUCTION

Domestic effluent is still directly discharged to many Flemish rivers. This leads to an increase of pollutants in these rivers. Intensive agriculture causes a further increase of pollutants. River modelling is an efficient tool for sustainable management of rivers and for the evaluation of possible protective measures.

In Flanders, the northern part of Belgium, computer models exist for the larger non-navigable rivers. These are denoted as category 1 and 2 rivers and the models for these rivers are constructed for the Flemish Environment Agency (www.vmm.be). Further computer models for the Flemish sewer network are constructed by Aquafin (www.aquafin.be). Brooks, which are generally maintained by the Flemish cities and communities, have dimensions that lie between the size of non-navigable rivers of

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category 1 and 2 and the size of sewers. These brooks are categorized as non-navigable rivers of category 3. Up to now, no models have been developed in Flanders for these category 3 rivers, although such models could serve as valuable support tools for decision making, planning and performing of integral water management by the local government. For example, model simulations can quantify the effect of certain pollution loads on water quality.

In order to investigate the feasibility of the development of such a microscale model for category 3 rivers, the Kleine Ronsebeek brook, which is part of the Scheebeek brook catchment was selected as a case study. This Scheebeek is part of the Interfluvium region, a region situated between the river Scheldt and the river Leie. A digital terrain model of the Kleine Ronsebeek valley is depicted on Figure 1.

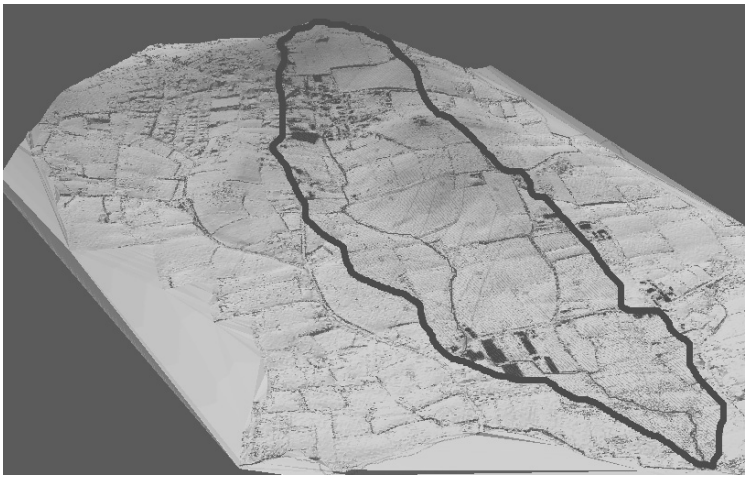


Figure 1. Digital terrain model of the Kleine Ronsebeek valley.

The objectives of the case study were threefold:

1. The selection of an appropriate software for modelling and simulation of the Kleine Ronsebeek.
2. The implementation of the Kleine Ronsebeek in the software.
3. Application of the model for scenario analysis, based on objectives put forward by the local government.

This paper describes the obtained results.

2. OBJECTIVES OF THE DEVELOPED MODEL

The following objectives were put forward by Leiedal, the intermunicipal society of the Kortrijk region:

1. Digitalization of all the water related data.
2. Indication of high flood-risk areas by a hydraulic module.

3. Evaluation of different measures such as extra buffer volume on water drainage.
4. Planning of the maintenance of sewers.
5. Evaluation of different measures to increase water quality.
6. Modelling of ground water flow.

3. SOFTWARE SELECTION

The following water quality and water quantity modelling software tools were considered for possible use:

1. Isis / pdm-isis
2. Infoworks rs
3. Infoworks cs
4. Mike11/nam
5. Delft3dflow
6. Floodworks
7. Simcat
8. Sentwa
9. Septwa

An evaluation of the available commercial software, listed above, resulted in the selection of CS for the case study. An extension with Infoworks RS was recommended for the future, but not implemented so far. This combination is able to store water related data in a systematic and digitized way in a centralised master database. This data can easily be accessed and altered by Infoworks and other applications, like GIS. Possible flood areas can be identified by using historic precipitation data and by coupling the software with the software Floodworks. Simulations can easily be visualised and certain decisions can be evaluated by the software. Finally, water quality can also be calculated, although this water quality module is very difficult to calibrate.

The downside of the InfoWorks package is that no ground water module is incuded. As such, other simulation software such as Modflow should be used when studying ground water flow. Further, planning of the maintenance of sewers can also not be integrated either in the model, but the effect of sewer maintenance can be evaluated.

4. IMPLEMENTATION OF THE KLEINE RONSBEEK IN INFOWORKS CS

The Kleine Ronsebeek was implemented in Infoworks CS (Devoldere, 2007). A network of 50 nodes and 26 flow surfaces was defined. Input for the model was collected by different governmental agencies such as the Flemish Environment Agency, the Database for the Subsoil of Flanders, the Royal Meteorological Institute of Belgium, the National Geographical Institute of Belgium, the Flemish Hydraulic Information Centre and the Flemish Agency for Geographical Information. The fact that the necessary information is so dispersed was identified as a major bottleneck for modelling of small scale subcatchments.

4.1. Measurement Campaign

In order to calibrate the model a 1 month measurement campaign was performed in the summer period (August-September 2006). A limited number of measurements was performed in the winter period (February 2007). Flow rate, ammonium concentration and COD concentration were measured at 3 points of the catchment area (denoted with number 1, 3 and 6) in Figure 2. Further, rain data recorded in July 2006 was used.

These measurements allowed the calibration of the base flow and the rainfall with flooding during the dry months and the rainfall without flooding during the wet months. However, additional measurements will be necessary in the future to calibrate events such as a first flush.



Figure 2. Map of the Kleine Ronsebeek valley.

4.2. Calibration results

First, the heavy rain data was used to calibrate flooding during the dry months and to identify flood risk areas. Simulations revealed that points A, B, C and D indicated on Figure 2 were the main flooding locations. This was in accordance with records from the local government. Resulting calculated flow rates in the measurement points are indicated on Figure 3.

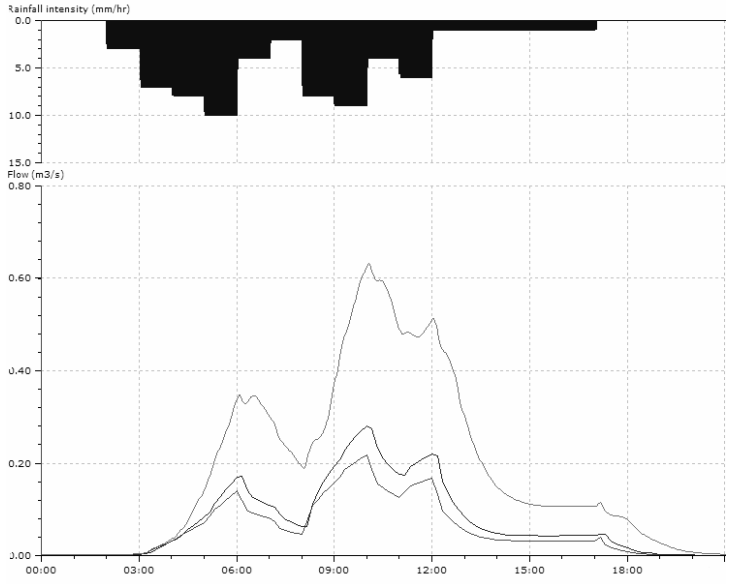


Figure 3. Resulting flowrates at the measurement points (1, the lowest curve; 2, the middle curve; 6: the highest curve) during the flood event in July 2006.

A rain event without flooding that occurred in February 11th 2007 was also simulated. Resulting calculated flow rates in the measurement points are indicated on Figure 4.

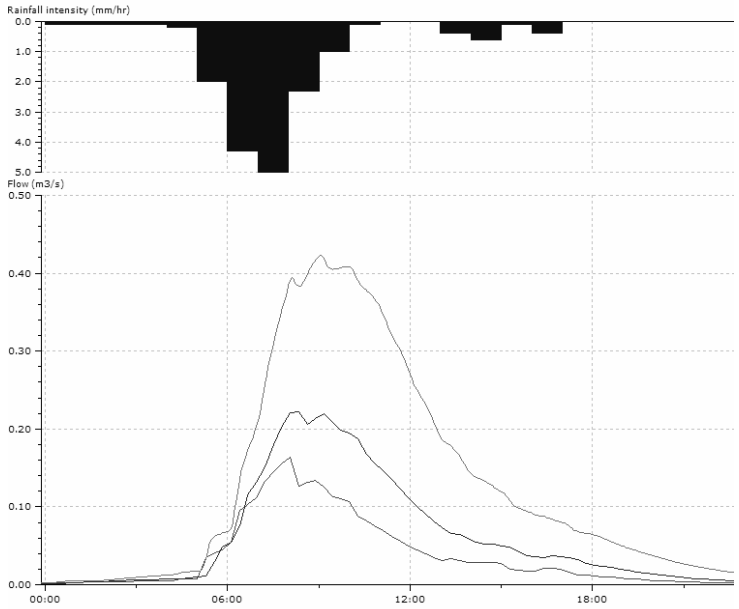


Figure 4. Resulting flowrates at the measurement points (1, the lowest curve; 2, the middle curve; 6: the highest curve) during the rain event in February 2007.

Measurements of the flow rate at points 1, 3 and 6 were performed at 14:30, 13:30 and 13:00. Calculated and measured flow rates at these points are compared in Table 1.

Table 1. Comparison between calculated and measured flow rates at the measurement points 1, 3 and 6 during the rain event in February 2007.

	Measured flow (l/min)	Calculated flow (l/min)
1	16000	10800
3	3800	3000
6	1200	1800

In Figure 5 the calculated and measured COD concentrations at the measurement points 1 and 6 during the measurement campaign in August-September 2006 are compared. At measurement point 1, certain measured concentrations are above the simulated range.

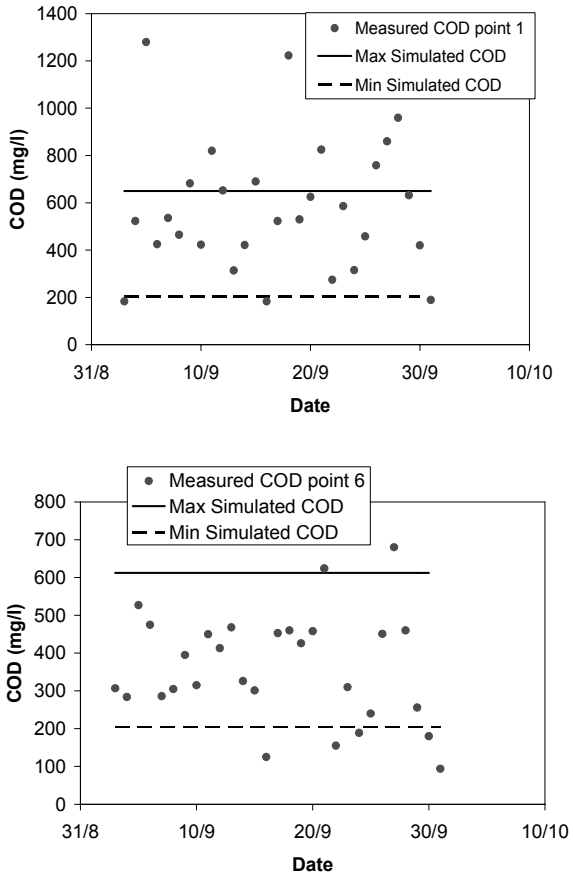


Figure 5. Comparison between calculated and measured concentrations at the measurement points 1 (left) and 6 (right) during the measurement campaign in August-September 2006.

Due to the small flow rate (50 l/min) the concentration at this point is influences by an increased use of chemicals (soap and detergents) on for example a Saturday morning. The COD concentration at measurement point 6 is less vulnerable for an increased chemical use because of the higher flow rate (150 l/min).

4.3. Model Simulations

So far, no scenario analysis was performed as several parts of the model are not calibrated. Future efforts will be directed towards further calibration of for example first flush phenonema. Once the model is fully calibrated, different simulations will be performed in order to evaluate different environmental actions such as the placement of treatment plants for individual houses.

5. CONCLUSIONS

Different simulation software tools were evaluated for microscale modelling of the Kleine Ronse brook. Infoworks was selected based on the objectives were put forward by Leiedal, the intermunicipal society of the Kortrijk region.

The Kleine Ronse brook was modelled. Initial calibration of the model was based on a 1 month measurement campaign period (August-September 2006). A limited number of measurements was performed in the winter period (February 2007). Flow rate, ammonium concentration and COD concentration were measured at 3 points of the catchment area, heavy rain data recorded in July 2006 was used.

Additional measurements will be necessary before the model can be used for scenario analysis, although initial calibration and simulation results look promising.

ACKNOWLEDGEMENTS

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Estimation of Evapotranspiration Using Fuzzy Rules and Comparison with the Blaney-Criddle Method

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Abstract: Evapotranspiration is a fundamental parameter in water balance fluctuation. There are a number of methods suggested to estimate evapotranspiration in the absence of direct measurements. The main disadvantage relates to the fact that most of them require a number of factors that are not usually available. Moreover, their performance varies among areas with different climate characteristics. In this research the possibility of predicting potential evapotranspiration using fuzzy logic theory with temperature as input is studied. The fuzzy logic model is being trained using a series of temperature and evapotranspiration values. The results are being compared to measured values and with the ones obtained using the Blaney-Criddle method. The results show a high efficiency in calculating and predicting the potential evapotranspiration values.

Keywords: Blaney Criddle, evapotranspiration, fuzzy rules, prediction.

1. INTRODUCTION

The most common method to deal with the uncertainties was probability theory, until 1965, when Zadeh introduced the fuzzy set theory. Fuzzy logic is an effective tool for handling the ambiguity and uncertainty of the real world systems. The Fuzzy Rule-Based (FRB) systems or Fuzzy Inference Systems (FIS) originate from fuzzy logic and the fuzzy set theory in general. FRB systems provide an effective way to capture the approximate nature of the real world processes, due to the rule formulation. The relationships can be described verbally instead of using mathematical equations. The input and output variables are related using fuzzy IF-THEN rules, where IF relates to the vector of fuzzy premises and THEN to the vector of the consequences, which has the form of a fuzzy set as well. The linguistic formulation of the rules resembles the human reasoning, since it is based on "IF-THEN" expressions and this is why fuzzy logic is such an effective tool in describing ambiguity. FRB models provide answers

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to problems where the procedures are complex and difficult to define mathematically.

In recent years many applications using fuzzy logic theory appeared, since it is an alternative and effective tool for studying complex phenomena. Fuzzy logic models can give answers to practical problems, without being time consuming. Fuzzy rule systems have been used successfully in reservoir management (Panigrahi and Mujumdar, 2000), rainfall-runoff problems (Nayak *et al.*, 2005; Yu and Chen, 2005) and in parameters of groundwater flow (Tzimopoulos *et al.*, 2004; Moutsopoulos *et al.*, 2005; Chalkidis *et al.*, 2006).

In the present paper, an attempt is made to apply FRB to simulate evapotranspiration values using temperature data. A brief description of fuzzy logic theory is given in sections 2 and 3, followed by a description of the Blanney–Criddle method in section 4 and the estimation of evapotranspiration (E_p) values together with the comparison to those predicted by the Blanney-Criddle method in section 5.

2. FUZZY LOGIC

In Boolean logic the boundaries of a set are clearly defined and it is evident whether an object belongs to a set or not. It is described by a binary function, the characteristic function, taking the value 0 when an element x doesn't belong to the set A and the value 1 when it does. On the contrary, in fuzzy logic, it is possible for elements to belong partially to a set. Fuzzy sets are a generalization of the classical sets since they don't have clearly defined boundaries. If X is a collection of objects denoted generally by x, then a fuzzy set $\tilde{A}$ in X is a set of ordered pairs:

$$\tilde{A} = \left\{ \left(x, \mu_{\tilde{A}}(x) \right) \middle| x \in X \right\} \quad (1)$$

where $\mu_{\tilde{A}}(x)$ is called the membership function or grade of membership of x in $\tilde{A}$. The degree to which an element belongs to a fuzzy set is shown by the membership function, which takes values from the [0, 1] interval. The closer the membership function $\mu_{\tilde{A}}(x)$ is to 1, the more the element x belongs to the fuzzy set $\tilde{A}$.

3. FUZZY RULE SYSTEMS

3.1. Fuzzy system function

A fuzzy system R is defined as the collection of rules consisting of fuzzy premises $A_{i,k}$ with membership functions $\mu_{Ai,k}$ and fuzzy responses $B_{i,k}$ with a form of a fuzzy set also. A fuzzy rule has the following general form:

$$\text{If } a_1 \text{ is } A_{i,1} \otimes a_2 \text{ is } A_{i,2} \otimes \ldots\ldots \otimes a_K \text{ is } A_{i,K} \text{ then } B_i$$

All rules use the same variables as fuzzy premises and the same variable as fuzzy response. A fuzzy rule is composed of linguistic variables connected together with logical operators. The most usual ones are the AND, OR represented by the $\otimes$ symbol above.

In fuzzy systems the partial fulfillment of a rule is possible. The degree to which a rule is true is called Degree of Fullfillment (DOF). Zero DOF value denotes no fulfillment of the rule and unitary value means that the rule is fully applicable. The values within the interval [0, 1] show the degree of applicability of the rule. Since the premise part of the rule is applicable to some degree, the same degree stands for the antecedent part too. If there is more than one fuzzy premise, then the membership values should be combined, in order to obtain the DOF. The AND operator corresponds to the union of the fuzzy sets and the OR operator to the intersection. AND operator is interpreted using a t-norm and OR is interpreted using a t-conorm. There is a great variety of t-norms and co-norms suggested in the literature (Zimmermann, 1995). The most usual one is the product inference method.

AND operator

The DOF results from the membership functions of each fuzzy premise:

$$\nu_i = \prod_{k=1}^{K} \mu_{A_{i,k}}(a_k) \tag{2}$$

OR operator

$$\nu(A_1 \text{ OR } A_2) = \mu_{A1}(\alpha_1) + \mu_{A2}(\alpha_2)) - \mu_{A1}(\alpha_1) * \mu_{A2}(\alpha_2)) \tag{3}$$

Overlap between premise membership functions is necessary in order to reduce the error sensitivity. As seen in figure 1 overlap exists when the domains between neighboring fuzzy numbers partially cover each other. This means that when a crisp number is imported to the system, it belongs partially to more than one fuzzy numbers. Each premise fuzzy set corresponds to a different rule. When a crisp number belongs to an overlapping section then there are as many rules triggered as the fuzzy sets belonging to the particular section. Crisp values are assigned using a training set (to train the rules) and a verification set (to test model performance).

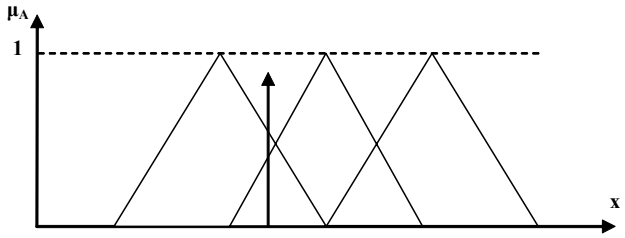


Figure 1. Fuzzy numbers with overlap.

It is obvious that in this case every rule has a different DOF and leads to a different response for the same inputs. All fuzzy responses are being aggregated to a

fuzzy set which results from all the fuzzy premises and takes into account also the DOFs. There is a number of methods suggested to aggregate fuzzy responses. The most usual ones are the maximum combination method, the minimum combination method and the additive combination method (Bárdossy and Duckstein, 1995).

The fuzzy set obtained from the above procedure is necessary to be replaced by a crisp number. This number must be the most representative of the fuzzy set, especially when a solution to practical problems is sought. Defuzzification is called the procedure where the fuzzy set obtained from the rule responses is being transformed into a crisp number. There is a plethora of defuzzification methods. The most usual one is the fuzzy mean method. This is the number for which the left part of the membership function of this number is in equilibrium with the right side. This yields (Bárdossy and Duckstein, 1995):

$$\int_{-\infty}^{M(B)} \left(M(B)-t\right)\mu_B(t)dt = \int_{M(B)}^{+\infty} \left(t-M(B)\right)\mu_B(t)dt \qquad (4)$$

3.2. Rule construction

Rule construction is a procedure where information and available data are being transformed into fuzzy rules. The rules can be constructed empirically or by using available data. Usually the procedure is quite difficult, owing to relationships between the variables that cannot be easily understood. In these cases a training set is used. A training set consists of input and output data. There are many techniques suggested to utilize the training set and export the fuzzy rules. The most common ones are (Bárdossy and Duckstein, 1995):

1. Counting and weighted counting algorithm
2. Least squares algorithm
3. ANFIS algorithm (Adaptive Neuro-Fuzzy Inference System), (Jang, 1993). This method has similar function with artificial neural network and adapts the membership function parameters to minimize the system error.

In order to measure the system performance it is essential to use a verification set. The verification set consists also of input and output data. Input data are imported into the rule system and the responses are compared with the measured output values.

4. EVAPOTRANSPIRATION

Evapotranspiration results from the combination of evaporation from the ground surface and the transpiration of the plants. Evaporation is called the phenomenon where the soil moisture and the surface water are being transformed from liquid to vapor water. Transpiration is called the phenomenon where water returns to the atmosphere from the leave stomata. The factors that mostly influence evapotranspiration are air temperature, sun radiation, wind speed, humidity, etc.

At first there is a distinction between potential and actual evapotranspiration. Potential evapotranspiration corresponds to the amount of water that would return to the atmosphere in case of moisture redundancy. Actual evapotranspiration represents the actual water volume that returns to the atmosphere under real circumstances.

The concept of actual evapotranspiration is easy to understand but the calculation is difficult and complicated. It is also difficult to find measured values for all the factors that contribute to actual evapotranspiration especially in Greece. Measured evapotranspiration values are rare, hence one of the existing methods is used. Some of them require firstly the calculation of potential evapotranspiration.

4.1. The Blaney-Criddle method for potential evapotranspiration

Blaney-Criddle suggested the following empirical formula:

$$E_P = K\frac{32 + 1.8T}{3.94}P \quad (\text{mm/month}) \tag{5}$$

where K is a crop coefficient, T the mean monthly temperature (^oC) and P the daylight percentage, given from tables (Tsakiris, 1995). The above empirical formula is the first one proposed by Blaney-Criddle in 1950. A more precise calculation is possible by using emendatory coefficients so that more factors are taken into account. It was found (Xanthopoulos, 1990) that the Blaney-Criddle method approaches efficiently the results obtained by using Penman method to calculate pan evaporation.

The above method was modified in 1977 by Doorenbos and Pruitt resulting in the following formula:

$$E_p = a + b\left[p(0.46T + 8.16)\right] \text{ mm/day where,} \tag{6}$$

$$a = 0.0043\left(RH_{min} - \left(n/N\right) - 1.4\right) \tag{7}$$

where, RH_{min} the minimum relative humidity, n/N the relative daylight duration, T the mean monthly temperature, b a parameter depending on n/N and the wind speed. Owing to data shortage, equation 5 was used in this application.

5. APPLICATION OF FUZZY LOGIC TO ESTIMATE EVAPOTRANSPIRATION

The feasibility of predicting the evapotranspiration values of Kalamoto meteorological station was studied using fuzzy logic theory. Kalamoto station (220 m a.m.s.l) is located in North Greece, into Mygdonia hydrological basin. The available data concern the daily evapotranspiration values, covering the period from March 2002 to February 2004. The results are compared with those given by the Blaney-Criddle method.

5.1. Rule System

The fuzzy model operates based on "If-Then" rules, where "If" relates to the vector of fuzzy premises regarding the mean weekly temperature data of Kalamoto station. The "Then" relates to the vector of fuzzy response, which is the mean weekly evaporation. Rule construction was implemented using the ANFIS technique, contained to MATLAB. The model was executed using mean weekly temperature in combination with other fuzzy premises, like relative humidity and relative daylight duration. The results did not result to remarkable improvement and finally only one fuzzy premise was used. Several runs were executed in order to find the most efficient number of membership functions. The model seemed to approximate the observed values quite well when using 15 triangular fuzzy numbers. The procedure of premise membership function construction is highly subjective and is implemented using the trial and error method in order to find the functions that represent the system efficiently. Fuzzy premise membership functions cover the entire observation set.

5.2. Training Period

The entire set of observations is divided into a training set and a validation one. The training set is used when the rules cannot be directly formulated by the experts and is used to construct them. The training set consists of the mean weekly values of temperature at Kalamoto station and the mean weekly values of evaporation at the same station. The training period starts April 2002 and ends on February 2003. This means that it consists of 44 values, regarded enough for the rule system construction.

5.3. Verification Period

The verification period is covering the period from March 2003 to August 2003 also consisting of the mean weekly values. The input is imported into the rule system and obtains the mean weekly evaporation values. The outputs are being compared to the measured evaporation values of Kalamoto station. In figure 2 the observed and the calculated values using the fuzzy logic model are shown. The results were also transformed to mean monthly values and compared with the ones obtained using the Blaney-Criddle method for potential evapotranspiration. Model performance is measured using the correlation coefficient and the reduced mean square error:

$$reduced\ \mathrm{MSE} = \frac{1}{\mathrm{n}} \sum_{i=1}^{n} \left(\frac{\mathrm{Y_{obs}} - \mathrm{Y_{calc}}}{\mathrm{Y_{obs}}} \right)^2 \tag{8}$$

where Y_{obs} is the observed value, Y_{calc} the one calculated from the model.

The reduced mean square error between mean weekly observed evaporation values and calculated using the fuzzy logic model is 0.042 and the correlation coefficient is 0.9146 for the training period. For the prediction period the reduced MSE is 0.0582 and the correlation coefficient is 0.8877.

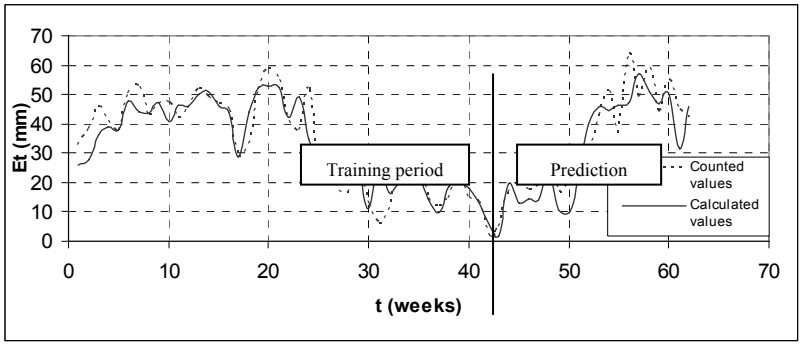


Figure 2. Measured and calculated with the fuzzy logic model mean weekly evaporation values.

The results were also converted to mean monthly values and compared to the respective values computed using the Blaney-Criddle method. The reduced mean square error between mean monthly-observed evaporation values and calculated using fuzzy logic is 0.0294 and the correlation coefficient is 0.9673. The reduced mean square error between mean monthly-observed evaporation values and those calculated using the Blaney-Criddle method is 0.1107 and the correlation coefficient 0.902.

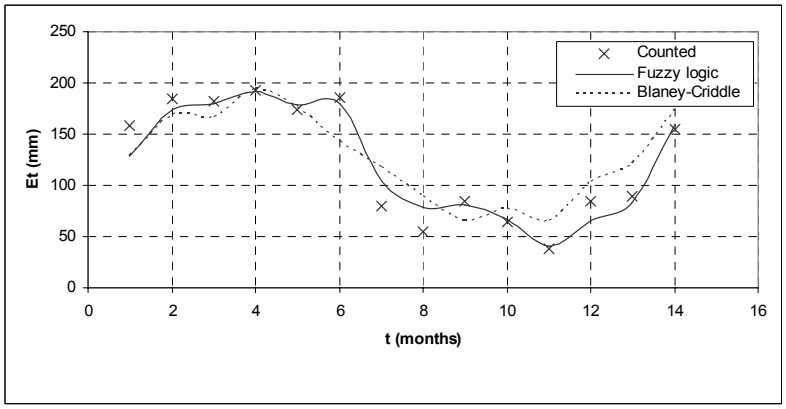


Figure 3. Counted and calculated mean monthly evapotranspiration values using the fuzzy logic model and the Blaney- Criddle method.

6. CONCLUSIONS

In this research the capacity of fuzzy logic theory to be used for evaporation prediction was investigated. The model was constructed using ANFIS method and fifteen triangular fuzzy numbers. The Fuzzy logic model seems to have the ability to reproduce the evaporation values quite well. The construction and execution of the model are not time consuming and they can be used for prediction too. The only requirement involves the existence of a small series of measured temperature and

evaporation values. The rule system is constructed in order to be used for each case separately. On the contrary conventional models require more parameters to be taken under consideration and follow the same methodology for all regions and climate conditions.

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What Went Wrong With Stochastic Groundwater Hydrology? Randomness Versus Incomplete Information

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Abstract: Stochastic models had dominated, from 1975 to the mid- nineties, groundwater hydrology but have recently faded. This article explores the reasons for the decline of stochastic groundwater hydrology. Contrary to arguments, such as, of lack of understanding of sophisticated mathematics or computational needs that were not fulfilled, the reasons for its decline were more basic and directly related to its premises. First the framework and major assumptions, which are used in stochastic analytical and numerical groundwater models, are presented. Next the major arguments are presented about the failure of stochastic groundwater models. The reasons for their failure were: (i) the lack of the distinction between randomness and incomplete information and the non-plausibility and non-verifiability of assumptions; (ii) the non-incorporation of prior information; and (iii) the lack of predictive power and the non-correspondence between a model's complexity and available raw data. The arguments are expounded using a Bayesian view of Probability Theory as reasoning under incomplete information, developed by Jeffreys (1939), Polya (1954) and Jayes (2003). It is concluded that the primary task for the hydrologic community is the collection of data, the incorporation of prior information, and the use of more primitive models, in correspondence to the limitations of data.

Keywords: Bayesian, groundwater hydrology, inductive reasoning.

1. INTRODUCTION

The importance of what model to use to describe and predict the groundwater flow and the transport of contaminants in the soil hardly merits any emphasis. The *U.S.* Resource *Conservation and Recovery Act (RCRA), Title D: Protection of the Environment for Municipal Solid Waste Landfills*, specifically calls for the use of *Groundwater Impact Assessments* (GIA), the prediction of any potential migration of contaminants from the area of a landfill, and for 100-years after closure of a landfill. The corresponding *European Directive 1999/31/EC* does not explicitly refer to GIA

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but, in practice, many Member States of the EU require the use of models to assess the impact of existing landfills on the environment and the sitting of new landfills. Both the U.S. Act and the European Directive rely on the "Polluter pays" principle, which interjects the results of a model into the legal system.

At the initial stage of application of these laws, modeling was used in the legal process in order to bring some semblance of scientific validity into the litigation process. Although, deterministic models had their problems due to the lack of data, the nature of their assumptions, and the occasional misuse to conform to the desired line of arguments, they were based, on similar assumptions, and hence provided a common basis for comparisons.

With the advent of stochastic groundwater and transport models competing conceptual frameworks were presented in the courts of law. Concepts of uncertainty and relative interpretation of results created confusion to legal and regulatory bodies. This has led to the situation where technical expertise has been cast in doubt and winning or not a case rests primarily on the "plausibility" of the arguments and not on the scientific merits of a case. It will be shown here that, far from being a negative development, reliance on reasoning is the correct approach.

At the same time, the failure of the stochastic models to provide any strong evidence of predictive power has led to the decline of this research. Unfortunately, at least in USA, this has translated to doubts and to a corresponding decline to research funding for the whole area of subsurface hydrology. Although, stochastic models seem now to have faded, no cohesive explanation has been given for their failure. This article provides some thoughts on what went wrong with stochastic subsurface hydrology.

2. STOCHASTIC CONCEPTUAL MODEL

We give an exposition, first, of how the development of a stochastic conceptual groundwater model evolves in order to evaluate later on the plausibility of its assumptions. Stochastic conceptual models consider, similarly to deterministic models, the steady state equation

$$-\nabla \cdot \underline{q}(\underline{x}) + f(\underline{x}) = 0 \quad \text{for} \quad \underline{x} \in D \tag{1}$$

and Darcy's law:

$$\underline{q}(\underline{x}) = -K(\underline{x}) \ \nabla \ h(\underline{x}) \tag{2}$$

subject to the general boundary conditions

$$\begin{aligned} h(\underline{x}) &= H(\underline{x}) \qquad \text{for } \underline{x} \in \Gamma_D \\ -\underline{q}(\underline{x}) \cdot \underline{n}(\underline{x}) &= Q(\underline{x}) \quad \text{for } \underline{x} \in \Gamma_N \end{aligned} \tag{3}$$

Although some authors have considered other types of flow (transient,

unsaturated flow, etc.) for simplicity we will make the arguments, which apply equally well to other situations, utilizing the steady state model. Here, underlined symbols represent vector quantities, $\underline{q}(\underline{x})$ is the flux vector, $f(\underline{x})$ is a forcing scalar term, $h(\underline{x})$ is the hydraulic head, and $\underline{\bar{K}}(\underline{x})$ is the hydraulic conductivity, all of which have a *dependence* on position $\underline{x}$. Other symbols include: D, the flow domain, Γ_D and Γ_N, the prescribed head and flux boundaries, respectively, and $n(\underline{x})$ the unit vector normal to Γ_N. In our discussion we will ignore details on forcing terms and the boundaries, concentrating on the case that has attracted, the most attention, which is of an infinite domain with uniform mean hydraulic gradient J equal to a constant.

All the stochastic groundwater theories start by stipulating that the hydraulic conductivity K, *due to its spatial variability,* and *insufficiency of data*, which are termed, collectively, as *uncertainty on K*, can be described, at each point $\underline{x}$ of the flow field, as a scalar *random variable,* r.v. The point of reference in the stochastic literature is the article by Freeze (1975), who was the first to popularize the transfer of stochastic techniques in the field of subsurface hydrology. The variability of K has been observed in the field since the 1960's and K can vary several orders of magnitude within short distances.

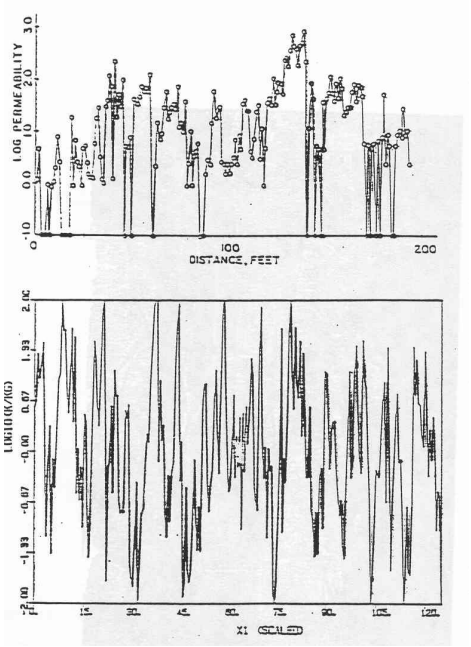


Figure 1. Similarity of results between a physical and an artificial stochastic process.

The conceptual transformation from the presence of variability and the depiction of K-data by an apparent non-smooth function in space to the conceptualization of K as a r.v. is, usually, defended in the stochastic literature by graphs similar to the one

shown in the next page. Figure 1 is reproduced by Gelhar (1993) and at the top the data of log permeability with distance are plotted, whereas, at the bottom, the results of an artificial stochastic simulation are shown. The graph was used to exhibit the *similarity of results* between a physical and an artificial stochastic process and to provide an *ad hoc* justification of the use of the stochastic framework in groundwater problems.

The introduction of an infinite number of random variables K's necessitates the use of the concept of a collection of all of these r. v., which is designated as a random function. The infinite random variables K are, now each, characterized by a probability density function (pdf) and its corresponding moments. This translates to a joint density function with an infinite number of marginals, a formidable task, which is bypassed in the stochastic development by invoking some form of weak stationarity, i.e., that all the random K's share the same pdf, which is the same with the joint density function.

The majority of the stochastic results have, actually, been developed by invoking a stronger form of argument, that of second order stationarity, i.e., that the means of each K, which are $m(\underline{x})$ - since the random variables are position dependent so are their means - are equal to each other and to a constant, $m(\underline{x}) = M$; and that the covariance function between two r.v. K, which are separated by a distance d does not depend on the positions of the conductivities but only on the magnitude of the separation distance.

Introduction of a stochastic model for the hydraulic conductivities turns the variables of head and flux in equations (1) and (2) also in random functions and the equations into stochastic partial differential equations. There has been an impressive arsenal of mathematical techniques that has been employed to solve these equations over the last 30 years, and for reasons of completeness these are described briefly below.

The analytical stochastic method that has been most widely used in studies of flow in porous media is the small-perturbation expansion method (Dagan, 1989; Gelhar, 1993). Accordingly, the variables and parameters of the problem are decomposed into a mean and a first-order perturbation (with minor efforts in some cases to include corrections for second terms). Although the results are therefore applicable only to, relatively, homogeneous media, stochastic groundwater theories have extended them to strongly heterogeneous soils by invoking different hypotheses, with no physical or mathematical justification given for such extension. The most popular conjecture (named after Landau and Lifshitz) in studies of the effective hydraulic conductivity is to consider linear expressions as a Taylor series expansion of the corresponding exponential function and extend the results to strong heterogeneities.

Some authors (Zhang, 2002) have argued that, in principle, one does not have to be restricted to only first order terms but can proceed to a series expansion that includes higher order terms. Including all of these terms into the governing flow equations allows the development of a set of independent partial differential equations

of variable order. Since this set of statistical moment partial differential or integro-differential equations cannot be solved analytically, what is envisioned as the next challenge of stochastic research is a development in numerical techniques that can address this type of equations.

Statistical moment equations have also been derived through spectral methods (Gelhar, 1993; Paleologos *et al.,* 1996) for stationary variable fluctuations or for special cases of non-stationary fluctuations (Zhang, 2002), or with a Green's function method. Our arguments, however, do not rest on the appropriateness of a specific mathematical technique but on the premises of all stochastic groundwater methods.

In contrast to developing parallel codes to solve stochastic integro-differential equations, the preferred numerical tendency in stochastic groundwater hydrology was that of Monte Carlo simulations. Numerical studies of flow in heterogeneous formations relied on Monte Carlo simulations of a "large number", n, of realizations of the hydraulic conductivity field. Each realization was generated based on the same underlying statistics of the K-field and each represented an equiprobable depiction of the real aquifer. This allowed the generation of n equiprobable flow fields and the evaluation of the statistics of the variables of interest (head, flux). Monte Carlo approaches were less restrictive than the theoretical methods, since the whole pdf of the output variables could be obtained, and necessitated the use of fewer assumptions. However, Monte Carlo approaches still utilized the basic premises of stochastic hydrology, which were that the variables and parameters were r.v., some form of stationarity, of an ensemble of aquifers, and of ergodicity.

But, where does one get the statistics of the, with one assumption or another of stationarity, random function K? After all, despite the measurement errors, problems of scale etc., what is estimated at a location is a single value of K. The connecting thread to pass from the impossibility of repeated sampling at one location and the need for summary statistics was accomplished through the concepts of the ensemble and ergodicity.

The concept of an ensemble defined not the one, existing in reality, single aquifer but many, equally possible aquifers that exhibit the same statistical structure for the K-data; the real aquifer being a single realization of this ensemble. Since in reality one always deals with a specific geologic formation rather than a fictitious ensemble of formations, the statistics of the real data that are measured (or estimated) at different points in space cannot be taken to represent ensemble statistics without invoking the hypothesis of ergodicity. Ergodicity means that the probability law governing a random function can be derived either from repeated sampling at a given point of an ensemble of statistically equivalent media, or at different points within a single medium.

3. ARGUMENTS

We proceed to argue now that the framework on which stochastic groundwater

hydrology relied on was based on unsupportable premises. The arguments are given in a qualitative form in order to allow access to most hydrologists but one can certainly develop a more formal framework in the lines of a Bayesian analysis or of the Probability Theory as an extension of Inductive Logic, as expounded by Cox (1925), Polya (1954), Jeffreys (1939) and Jaynes (2003).

3.1. Non-Plausibility and non-Verifiability of the Stochastic Premises in the Field

Central to the development of stochastic groundwater hydrology is the idea of a random variable, of the "randomness" of the parameters and variables of models described by equations such as those in (1)-(3). It is reminded that until the 1930's, the notion of probability was defined through two approaches. In the "frequentist" approach if an event appeared, *a posteriori,* after a large number n (infinite asymptotically) of repeated, similar experiments p times then the probability of its occurrence was defined as p/n. In the "classical" approach if an event could be defined, *a priori*, that it could appear p times out of n equally probable ways then its probability was defined as p/n. Both of these are problematic in the definition of the "randomness" of K, h, or q.

One can, potentially, bypass the physical impossibility of sampling for K at the same location, without destroying the medium and the "same conditions" that are required for defining a random variable at each location, by claiming that the data that are actually used are not from a single location (that is irrevocably disturbed) but from the whole domain, which only needs to be sampled once. This is termed as the ergodicity assumption. An argument that is used to defend the use of ergodicity in stochastic groundwater hydrology reads accordingly. If one were to throw in the air a single coin repeatedly and calculate its (ensemble) statistics wouldn't then these be the same with the statistics that would be calculated by throwing in the air a very large number of coins only once (spatial statistics)? The plausibility of such an argument for a non-defective coin is indeed high. The single coin is a physical reality and repeated experiments with a single coin can be conducted by independent observers who can verify the results of an ensemble statistic. Additionally, one can certainly collect, physically, a large number of coins and throw them in the air at the same time (and thus define a spatial statistic). Again, the results and the correspondence of ensemble and spatial statistics are verifiable, experimentally. In the case of a real aquifer, however, this concept becomes an artificial abstraction of convenience. In the case of an aquifer, neither the ensemble statistics at a location can be experimentally defined, nor any correspondence of ensemble and spatial statistics can be verified. Similarly, no conclusions can be reached on the stationarity or not at a specific location.

The idea of repeated experiments and of a r.v. becomes even less defensible if one were to consider for example some well-defined variable such as the hydraulic head. In a steady state situation as equations (1)-(3) refer to, repeated measurements

of the head at a monitoring well, by definition of the steady state are not random and should yield the same result. The observed variability of the head at a location is the result of *temporal* variations that are not part of the system (1)-(3) or of a change in the *boundary conditions* or the *forcing terms* of the experiment (barometric changes, restructuring of the solid matrix etc.) that again invalidate the definition of random variable.

Leaving aside the validity or not of ensemble or ergodicity the question now rests (even accepting that one has a large number n of data at one's disposal in order to obtain reliable spatial statistics) of whether the "same conditions for the experiments" that enters the probabilistic concepts is satisfied even for the spatial data. One should notice that, in the coin analogy, the conditions and the medium (the air), where the experiments take place is the same in both situations. One would have a hard time though arguing about the equivalence of the results, if the conditions were substantially different, for example if the air in different locations where the coin is thrown was more or less turbulent etc. But, even if one were to use a much weaker statement of "somewhat similar conditions" of the experiments this stipulation cannot be defended for groundwater conditions. Especially, since the stochastic subsurface hydrology had focused on problems of heterogeneity, *both the geologic structure and the hydrologic conditions at different points of a field of interest are, by definition of the heterogeneity, dissimilar*.

The stochastic premises in groundwater hydrology do not arise from physical processes but are used simply as convenient mathematical tools. However, in probability theory most important developments were accomplished through physical considerations. For example, the normal distribution did not appear simply as a curve that fits data but as the required shape of the function that describes the distribution for errors in measuring the position of a star (Herschel-Maxwell derivation) or again in astronomy by Gauss for estimating the position of a star given a number of observations etc. The infinite set of random variables that are introduced are not specified through a limiting processes and in Probability theory several paradoxes exist that have arisen precisely from the misuse of infinite sets.

Of course one can claim that what is actually meant by randomness in stochastic models is not randomness per se but lack of information, due to limited data and knowledge of the complex processes, which govern the hydro-geologic problems. This line of thought lands one in the area of the Bayesian approach to probability. It will be seen below that the Bayesian approach also refutes the premises of stochastic groundwater hydrology.

3.2. Non-Incorporation of Prior Information

The last fifty years the field of Probability Theory has seen a considerable transformation. R.T. Cox, H. Jeffreys and E.T. Jaynes have redefined the concept of probability in terms of the theory of Inductive Logic (or Polya's term of Logic of Plausible Arguments). This point of view has resolved many paradoxes, reproduced

all known results and provided numerous contributions in fields from astrophysics, genetics, epidemiology, engineering, medical technology etc. In a very broad sense the Theory of Probability has moved away from the quantification of the frequency of occurrence of "random" events to a structured procession of arguments that is done with rules of Logic. The term plausibility of the arguments enters because in contrast to the rules of deductive reasoning by Aristotle, scientific progress operates with much weaker statements where no proof is attained initially but only the plausibility of a general statement can be evaluated. As a simple illustration, the Bayes's theorem and marginalization is one of the tools that has proven to be useful to evaluate the power of a hypothesis in light of prior information and measurements:

$$\text{Prob(hypothesis} \mid \text{data, I)} = \text{Prob(data} \mid \text{hypothesis, I)} \times \text{Prob(hypothesis} \mid \text{I)} \quad (4)$$

where,
I is the prior information and Prob(hypothesis | I) is the *prior probability* of a hypothesis about the nature of a process (and before we analyze the data but given whatever prior information I we have); Prob(data | hypothesis, I) is the *likelihood function*, which is modified by the experimental measurements; and Prob(hypothesis | data, I) is the *posterior probability* of the hypothesis in view of both the prior I and the data.

The point is that for groundwater flow problems there exists a wealth of information apart from conductivity estimations. This consists of geologic or hydrologic knowledge of a qualitative nature, a good sense of the stratigraphy, past studies etc. None of this is utilized in the stochastic framework (and this is true for many deterministic models as well). Indeed, not even the major geologic features of the formations at a site, which control the hydraulic conditions, are usually part of a stochastic model, which often views the site as a generic porous medium, with the same degree of heterogeneity everywhere. Even the limited K-data are not used per se (in the unconditional studies) but mostly to extract the mean, variance, and covariance function. *This completely eliminates any sense of prior probability since only the conductivity data are used (in the conditional case) in the model.*

Even the term of the likelihood function is problematic because the term I does not exist in stochastic treatments and only the *perceived* structure of the data is taken into account. The perceived is stressed here because in many cases other than the lognormal distribution have fitted the data; the covariance model is chosen with considerable liberty to the fitting of data (and in most analytical studies the exponential form is picked a priori for convenience); the data could have been obtained in different formations and therefore may represent different statistical populations; and the data on conductivity are not directly measurable but inferred indirectly through the flux and head, and rely on the model of flow used for the estimation of K. One is referred to a discussion in Lerche and Paleologos (2002) on the dependence of K-values on the control function (model) that is used to infer these values. Any sense of a posterior probability that evaluates a hypothesis is therefore not applicable and stochastic groundwater hydrology has no means of evaluating its

hypotheses in light of both data and prior information.

3.3. Predictive Power and Complexity of Model

One can argue that the power of a theory lies in its predictive capabilities. Of course this argument is not completely true because no understanding of the physical world would have been attained: one would simply have a compilation of situations where some results and not others applied. Scientific progress cannot be made without any rational mechanism for the continuous assessment of the assumptions of a model, the range of their applicability, and eventually either rejection of a hypothesis or establishment of the specific conditions under which a particular model and the assumptions it invokes applies. Stochastic groundwater hydrology does not allow for its basic hypotheses to be tested for their validity and rests its case strictly on whether it can predict "better" than deterministic models. But also here stochastic subsurface hydrology has not demonstrated any advantage and has not managed to add any substantial development in the theory of groundwater flow. The same can be said for the prediction of flow and transport at specific sites, where no practical advantage has been seen to have been gained. The problem here, apart from the non-plausibility of the stochastic assumptions, lies in the implicit assumption of the modeling community that a more involved, mathematically complex model can do better than a simpler one.

It is fair to characterize the structure of a stochastic groundwater model as much more complex than a traditional deterministic model. On the other hand, the data that are available are either extremely sparse - given the extent of the field analyzed (even for deterministic applications in most cases data are collected only in a few locations), the complexity of the geologic background through which flow takes place, and the complexity and interaction of several process that are involved - or of questionable value. Jaynes, Jeffreys, and Polya have all argued that the complexity of a model can dominate the information from data, to the extent that when new data arrive, if they are to be seen "…in terms of old ideas (that is old models which are not questioned)…" then they "…*cannot lead us out of the old ideas*…" Jaynes has argued that even Bayesian methods are more speculative if there exist few data, whereas maximum entropy "…is the appropriate (safest) tool when we have very little knowledge beyond the raw data; it protects us against drawing conclusions not warranted by the data." H. Jeffreys (1939) noted that "an argument which makes it clear intuitively *why* a result is correct is actually more trustworthy, and more likely of a permanent place in science, than is one that makes a great overt show of mathematical rigor unaccompanied by understanding." John von Neumann (1954) in his discussion about the failure of mathematics in economics had similarly voiced the opinion "…the empirical background of economic science is definitely inadequate. Our knowledge of the relevant facts of economics is incomparably smaller than commanded in physics at the time when the mathematization of that subject was achieved."

It seems to us that similarly to the situation described for economics the hydrologic community has overreached in the mathematization of the field and that the primary task at hand at this point is for the collection and utilization of all relevant information about a hydrogeologic problem, and for solution of these problems through primitive models, in correspondence to the limitations of data, and not for models with a complex structure.

4. CONCLUSIONS

There is a large body of basic objections, which address the very nature of what constitutes a stochastic framework in groundwater hydrology. This article expands on several arguments why the mathematization of the field of groundwater hydrology, attempted through stochastic efforts was premature, and instead finds that current efforts should be focused on attempts to include all prior information in a model, the collection and understanding of more data, and the application of simple models and case studies to increase our level of understanding.

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Comparison of two Groundwater Numerical Simulators Using Different Geostatistical Methodologies for the Solution of Complex Environmental Design Problems

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Abstract: Mathematical expressions have been widely used to describe and simulate complex physical processes encountered in nature. For the majority of simple physical problems an analytical solution can be obtained using well known conventional analytical methodologies whereas numerical methodologies are used to obtain solution in complex linear or non-linear design problems. In the present work, a comparison of two different computational methods, focusing on Finite-element and Finite-difference methods is presented for the solution of complex environmental design problems. An analysis and comparison of the main geostatistical methodologies available in the two graphical user interfaces (GUI) for spatial data interpretation is also discussed.

Keywords: Geostatistical methods, groundwater simulation models, numerical methodologies.

1. INTRODUCTION

Wide range of engineering problems such as land subsidence, solute and energy transport, flow in fractured and porous media, oil, gas and geothermal reservoir simulation are governed by first and second order partial or ordinary differential equations. For a large number of problems that are described by linear or quasi-linear equations and relative simple problem geometries, an analytical solution could be employed by separation of variables or the use of complex variable techniques, such as Laplace transformation or Green's function approaches. The mathematical solution of these problems could be obtained analytically and is valid for specific solution domain, boundary and initial conditions. On the other hand analytical solutions can not be obtained for linear and nonlinear problems with irregular geometries. For such problems the use of a numerical method has proved to be a very powerful tool to obtain an approximate solution. For the solution of these types of problems and the simulation of complex physical systems various numerical methods such as the finite

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difference, the finite elements, the collocation, the boundary element and the method of characteristics have been widely reported in the literature (Huyakorn and Pinder, 1983).

San Juan and Kolm (1996) used the ARC/INFO GIS software to first manage and analyze their data and then used the MODFLOW simulator a finite-difference numerical simulator, to characterize and conceptualize the Jackson Hole hydrologic system in order to predict the water table elevation and to produce a mass balance. Mylopoulos *et al.* (2006) developed a finite-difference model for the simulation of a complex lake aquifer system in Northern Greece. Their hydrogeological analysis and the simulation results proved that there is not interaction between the lake and its underlying aquifers. Improvements of the classical numerical methodologies have also been reported in the literature in order to overcome difficulties associated with local grid refinement and accuracy in simulating irregular boundary geometries. Mehl and Hill (2002) proposed a new method of local grid refining for two-dimensional block-centered finite difference meshes that couples the grids by sharing nodes and iteratively updating the matrix equations to ensure that the state variables are consistent between the two grids. Recently, Loudyi *et al.* (2006) developed a two-dimensional finite volume model for flow in porous medium that was able to improve the flexibility and accuracy in simulating irregular boundary geometries compared with the standard finite difference methods.

This work focuses on the comparison of two widely used numerical methods, the finite difference and the finite element methods, in groundwater modelling. These numerical methods are incorporated in two very popular groundwater numerical simulators, the Princeton Transport Code (PTC) and MODFLOW, with quite different operations regarding data interpretation, subject of further assessment. The aim of this study is to remark the advantages and disadvantages of the two numerical simulators regarding numerical methodologies, groundwater modelling and data interpolation. Thus, the user will be informed of each numerical simulator's abilities before its application.

2. COMPUTATIONAL METHODOLOGIES

In a simulation model, a set of mathematical functions is used to accurately describe the physical system. Numerical methods such as Finite Element (FE) and Finite Difference (FD) methods are used for the solution of the governing set of partial differential equations. This work focuses on the development of a simulation model of a coastal physical system using two different numerical approaches, a Finite Element and Finite Difference algorithms. Both the two numerical simulators, PTC and MODFLOW, are linked to graphical interface system software that allows the organization, quantification and interpretation of hydrological, meteorological and geological data. Using the interface system abilities, spatial distributed parameters could be easily analyzed and imported into the two simulation models.

2.1. Finite Element Method

The finite element method is a numerical methodology that solves differential equations by "piecewise approximation". The domain of interest is subdivided into

large number of interconnected sub-regions called finite elements. The finite element analysis of a physical problem can be summarized as follows (Huyakorn and Pinder, 1983):

1. Discretization of the domain: The physical system is discreatized into a series of finite elements that are connected at a discrete number of nodal points.
2. A matrix expression is developed to relate the nodal variables of each element creating an element matrix.
3. The element matrices are combined to form a set of algebraic equation that describes the entire global system
4. Imposed boundary conditions along the boundaries are incorporated into the assembled matrix equation.
5. The resulting set of simultaneous algebraic equations is solved.

2.2. Finite Difference Method

Whereas in the finite element method the discrete equations are applicable over an element, in the finite difference approximations of a differential equation the discrete representation of the equation is applied to a point. The finite difference method is normally used on rectangular grids. As it is shown in Figure 1, two kinds of finite difference discreatization are available, one that has nodes at the corners of the element and the other that has nodes in the centre. It should be noted that at each node a finite difference approximation for the equation is written (Pinder, 2002).

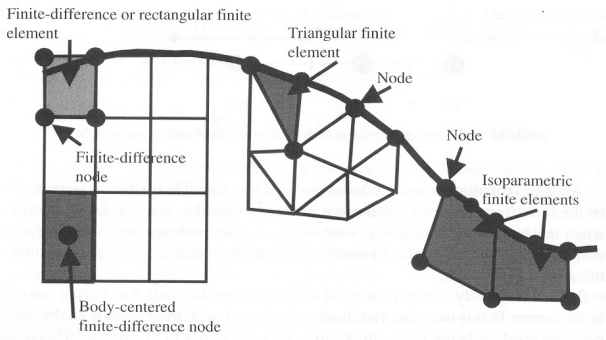


Figure 1. *Finite-element, finite-difference and body-centered finite difference meshes (Pinder, 2002).*

2.3.1. The princeton transport code (PTC)

The Princeton Transport Code (PTC) is a numerical simulator that solves a system of partial differential equations in order to accurate represents the subsurface flow, the velocities and the contaminant mass transport of the simulated aquifer. PTC employs a unique splitting algorithm for solving the fully three-dimensional equations, which significantly reduces the computational burden. The algorithm discretizes the domain into approximately parallel horizontal layers and within each layer a finite element discretization is used allowing for the accurate representation of irregular domains. The layers are connected vertically by a finite difference discretization. This hybrid coupling of the finite element and finite difference methods enables the application of

a splitting procedure according to which, during any given time iteration, all the computations are divided into two steps: a) In the first step all the horizontal finite element discretizations are solved for each layer, b) In the second step, the vertical equations which link the layers, are solved using the updated information from the horizontal equation solution (Babu and Pinder, 1984). For the purposes of this analysis the PTC-ArgusOne simulator was applied (Pinder, 2002).

2.3.2. MODFLOW

MODFLOW is a three-dimensional ground-water flow simulator that solves the governing equation using a finite-difference approximation. The domain of interest is subdivided into blocks in which the properties of the medium are assumed to be uniform. In plan view the blocks are made from a grid of mutually perpendicular lines that may be variably spaced. Model layers could be varying in thickness. A flow equation is written for each block, called a cell. Several solvers are provided for the solution of the resulting matrix problem and the user could choose the best solver for the particular problem. Flow-rate and cumulative-volume balances are computed for each time step (Hill *et al.*, 2000). For the purposes of this analysis the Visual MODFLOW 3.1 simulator was applied (Visual MODFLOW 1995-2003).

3. GEOSTATISTICAL METHODOLOGIES

The solution of complex physical problems such as the subsurface flow in a porous media requires the collection and analysis of spatial distributed data for soil and water parameters (i.e hydraulic and electrical conductivity, porosity, contaminant concentration). Spatial data interpretation and analysis is performed over the entire domain of interest using common geostatistical approaches.

In this analysis, the **Inverse Distance to a power method** is used by both simulators and it is a very fast and efficient, weighted average interpolation method. The weighted factor applied to the data, depends on the distance of the point from the grid cell and is inversely proportional to the distance squared. Consequently, the larger the distance of the data point from the grid node, the smaller the influence on the calculated value (Deutsch and Journel, 1998). Another choice regarding data interpretation, available only in MODFLOW simulator, is the **Kriging method**. Kriging is a geostatistical method that is used for interpolation based on irregularly spaced data. The fundamental is that the value at the estimation point is a result of a linear combination of its neighbors. The estimated value comes from the optimization of a statistical measure such as the minimization of the square estimation error. Anisotropy and underlying trends suggested in the data can be incorporated in an efficient way through Kriging. The application of the Kriging method requires the calculation of the semi-variogram for the determination of the data spatial dependence (Journel, 1989). The **Nearest contour method** is widely used by Argus One the graphical user interface linked with PTC simulator for spatial data interpretation, where an evaluated point returns the value of the contour it lies in or that of the contour nearest to it.

4. FIELD SITE APPLICATION

For the purposes of this analysis, two simulation models of the Hersonissos coastal aquifer have been developed, one based on the PTC-ArgusOne simulator and the other based on the Visual MODFLOW simulator. The aquifer of interest is located in the northern part of Heraklio Prefecture in Crete and covers an area of 17.9km^2 extended approximately 4.7km inland in the N-S direction with a coastal line of 3.8km. The topographic terrain is quite mild, and high elevations are observed towards the southern boundary. Hydro-geological data obtained from in-situ geophysical measurements at two locations were used to represent the geological stratification of the aquifer. The main geological formations present in the aquifer are karstified limestones and marls, while high permeability alluvial deposits are present along the coastal line. Analysis of soil composition, porosity, electrical conductivity, temperature and density measurements were used to determine representative values of hydraulic conductivity and porosity necessary for the development of a representative physical model of the Hersonissos coastal aquifer.

5. RESULTS-DISCUSSION

PTC-ArgusOne and Visual MODFLOW are two numerical simulators used for the simulation of the subsurface flow in a coastal aquifer. The most important differences and similarities among the two models are related to data processing and analysis and solution accuracy.

The elevation data input in the area of interest is based on interpolation methods. Regarding the Visual MODFLOW simulator the interpolation is obtained using the inverse distance to a power method as it is the only available. On the other hand the PTC-ArgusOne simulator includes a wide choice of methods for the same application such as the inverse distance and the nearest contour methods. The input of the hydraulic conductivity parameter is also necessary for the simulation of the physical system. The distribution of conductivity using the Visual MODFLOW simulator is obtained by applying two well-established interpolation techniques, the inverse distance to a power and the Kriging Method. The PTC-Argus One simulator on the other hand uses the inverse distance and the nearest contour method.

Summarizing the analysis of the two simulators regarding the data processing it is obvious that the PTC-Argus One simulator allows the user to have more choices for data interpretation regarding elevation, while both simulators allow the user to test different methods of interpolation for spatial distributed parameter such as the hydraulic conductivity and porosity over the entire domain of interest. This characteristic is important because the statistical properties of the available data sets vary and each data set needs to be processed with an interpolation method that matches its statistical properties.

A major disadvantage of the Visual MODFLOW simulator concerning interpolation is the application of Kriging method. The simulator does not have the ability of calculating the required semi-variogram. For that reason, external software such as Surfer should be used (Theodosiou and Latinopoulos, 2006). Another weakness of the Visual MODFLOW simulator is that the estimated values of the interpolation are not cross-validated. Hill *et al.* (2000) wrote in their report regarding

MODFLOW simulator that before the implementation of any simulation its consistency with the original data must be verified. Therefore the required cross validation process can be performed using an external program (i.e. Matlab) or commercial software (Surfer, 2003). These results could show if the chosen interpolation method is acceptable or not. In this work, regarding the Visual MODFLOW simulator, the kriging method was chosen for the data interpolation after the application of the Surfer software that performed the semi-variogram calculation and the cross validation process.

The comparison of the operations for the two simulators used in this analysis is leading to some useful conclusions. There is a correspondence between the two simulators regarding the input of the model parameters such as hydraulic conductivity, topographic terrain, porosity, storage coefficient, the 1st type (constant head) and 2nd type (extraction-injection wells) boundary conditions (bc). A major difference between the two simulators is the way that each simulator treats the inland boundary of the area of study. PTC-ArgusOne treats this boundary as a 2nd type bc allowing a variable in and out flux along the domain depending upon the existent geological formations. Visual MODFLOW approximates the in and out flux along the boundary with a variable recharge rate at the cells located along the boundary.

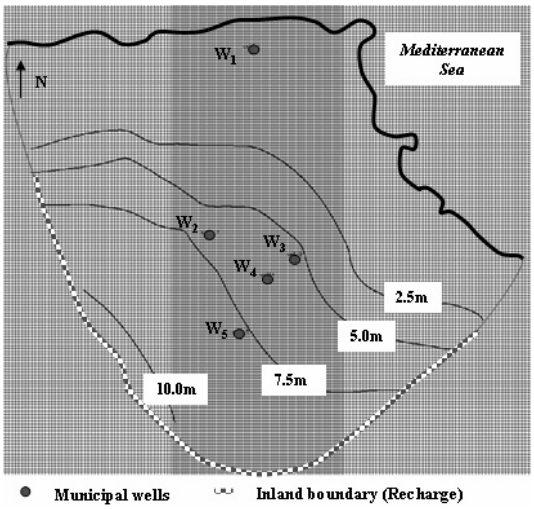


Figure 2. Hydraulic head distribution (Visual MODFLOW Simulator-Grid cells).

Due to the different numerical techniques that the two simulators used to solve the subsurface flow equation over the domain of interest the hydraulic head distribution obtained by the two simulators could be characterizes as qualitatively similar (Figures 2 and 3). PTC-ArgusOne simulator approximates the hydraulic head value over a triangular element whereas in Visual MODFLOW the hydraulic head value is approximate at the nodes of a grid cell.

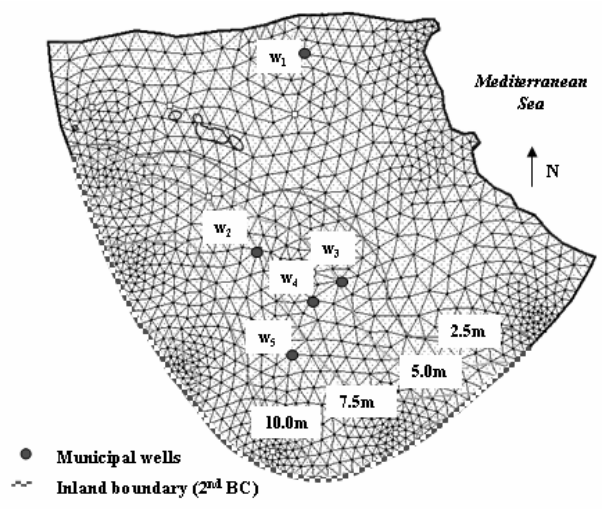


Figure 3. Hydraulic head distribution (PTC-ArgusOne Simulator-Triangular Mesh).

In reality as it is shown in Table 1, there is substantial deviation between the hydraulic head values that the two simulators obtained at the five municipal well locations. Especially at areas closed to the boundary (Well 1) where a loss of accuracy in predicting the hydraulic head distribution along and near the boundaries occurs due to the way that Visual MODFLOW simulator adjusts the geometry of the inland boundary (Andreson and Woessner, 1992). An estimation error up to 24% is also observed at the location between different hydraulic conductivity zones (Well 2). In the other observation locations where a similar discretization size is applied the estimation error did not exceed a value of 9% which is considered reasonable since two completely different numerical approaches were used. In their analysis Haijema *et al.* (2001) showed that the appropriate cell size should be chosen each time in order to represent more precisely the geometry of the boundary. This mismatch is also due to the variable recharge rate that MODFLOW applies at each boundary cell in order to simulate the in and out flux along the boundary.

Table 1. Calibrated hydraulic head values-Estimated error.

Well ID	PTC-ArgusOne Simulator	Visual MODFLOW Simulator	Hydraulic Head Difference	Error %
W1	0.01m	0.004m	0.006m	59%
W2	7.592m	5.75m	1.842m	24%
W3	4.856m	5.30m	0.444m	9%
W4	6.459m	6.29m	0.169m	3%
W5	8.211m	7.60m	0.611m	7%

6. CONCLUSIONS

In the present work, a comparison of two numerical simulators that use a finite element (PTC-ArgusOne Simulator) and a finite difference (Visual MODFLOW Simulator)

methodology, to simulate the subsurface flow of a coastal physical system is presented. The results showed that there is always a trade off between accuracy and computational burden. A major advantage of the finite element methodology is the ability to simulate irregular domains with great precision. Also the discussion concerning geostatistical methodologies showed that the assessment of different interpolation techniques in association with cross validation of estimated values with the original data, would allow a more accurate interpolation method of the available data set.

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Modeling a Coastal Aquifer in the Island of Thira using Three-Dimensional Variable Density Models

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Abstract: The advancement of seawater intrusion in a phreatic aquifer located in the central part of Thira island (Santorini) is examined using variable density flow and transport models. The fluid flow and mass transport processes are considered and the resulting mathematical models are solved numerically using the software program FEFLOW. A boundary condition based on a constraint is imposed on the seaside boundary regarding the chloride concentration. Several scenarios are examined for steady and time – dependent conditions. Two hypothetical cases of aquifer replenishment, with natural and artificial recharge are also simulated. The results demonstrate a significant dependence of the development of seawater intrusion on the initial and the boundary conditions on the seaside boundary of the aquifer. The transition zone predicted by the variable density flow and transport models is of considerable width.

Keywords: Coastal aquifers, coastal groundwater, seawater intrusion.

1. INTRODUCTION

Various methods for simulation of seawater intrusion in coastal aquifers have been developed. They are generally grouped into two broad categories: the sharp interface approximation (moving or steady), and the variable density models where a wide transition zone separates fresh from salt water. The present study belongs to the second category where two miscible fluids (two components of a single phase) are mixed as a result of hydrodynamic dispersion, while fluid density depends on concentration. A complex system of coupled nonlinear partial differential equations is obtained and solved numerically using the finite element method and software package FEFLOW. The system is solved iteratively and is CPU time intensive. Isothermal conditions are assumed, i.e. density depends only on concentration, while changes of viscosity are not considered.

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2. SALTWATER INTRUSION IN COASTAL AQUIFERS

For three – dimensional (3D) isothermal processes the representative model equations are as follows (Diersch, 1988; 1998), ($i, j = 1,2,3$):

$$S_h \frac{\partial h}{\partial t} + \frac{\partial q_i}{\partial x_i} = Q_\rho + Q_{EB}(C), \text{ (continuity equation)} \tag{1}$$

$$q_i = -K_{ij} f_\mu \left(\frac{\partial h}{\partial x_j} + \frac{\rho - \rho_0}{\rho_0} e_j \right), \text{ (Darcy equation)} \tag{2}$$

$$\frac{\partial}{\partial t}(nC) + \frac{\partial}{\partial x_i}\left(q_i C - D_{ij} \frac{\partial C}{\partial x_j} \right) - Q_C = 0, \text{ (transport equation)} \tag{3}$$

$$\rho = \rho_0 \left\{ 1 + \frac{\overline{\alpha}}{(C_s - C_0)}(C - C_0) \right\}, \text{ (fluid density)} \tag{4}$$

where,
h is the hydraulic head, q_i is the Darcy fluid velocity vector, C is concentration of salt, S_h is specific storage coefficient (compressibility), Q_ρ is source/sink function of fluid, Q_c is source/sink function of contaminant mass, Q_{EB} is term of extended Boussinesq approximation (Diersch, 1988; 1998), K_{ij} is the aquifer hydraulic conductivity tensor, f_u is a constitutive viscosity relation function, e_i is the gravitational unit vector, ρ, ρ_o are the fluid and reference fluid density respectively, n is the aquifer porosity, D_{ij} is the hydrodynamic dispersion tensor, and C_o, C_s are reference and maximum concentration, respectively. For each time step, the coupled system of flow and transport equations is solved numerically and the resultant concentration is used to calculate the density and Darcy velocity. Since changes of viscosity are not considered, here it is assumed that f_u=1.

Constant hydraulic head was assigned along the sea boundary: $h_f = (1+\alpha)h - \alpha z$ where $\alpha = (\rho - \rho_f)/\rho_f$ is the density difference ratio. Term h_f is the equivalent freshwater head (Guo and Langevin, 2002), h is the head, ρ is density of saline groundwater, ρ_f is density of freshwater and z is the elevation above the datum. For impermeable boundaries, flow normal to boundary is null (q=0). The following type boundary condition is used to represent the wells (Diersch,, 2002):

$$Q_\rho^w(x_i, t) = \sum_m Q_m^w \prod_i \{\delta(x_i - x_i^m)\}; \ \forall (x_i, x_i^m) \in \Omega \tag{5}$$

where,
Q_ρ^w is the well function, Q_m^w is the pumping/injecting rate of single well m, x_i^m is the coordinate of single well m.
For impermeable boundaries, the boundary condition is

$$q_C\left(x_i\ ,t\right)=qC-D_{ij}\ \frac{\partial C}{\partial x_j}n_j\ =0 \tag{6}$$

where,
q is the mass flux through the sea boundary. Eq. 6 means that neither convective nor dispersive mass flux occur through this boundary.

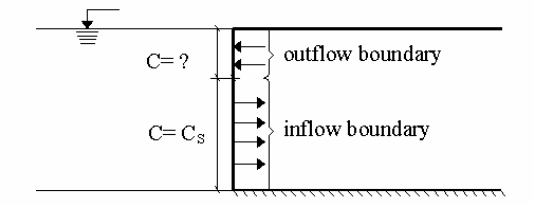


Figure 1. Description of the sea boundary.

$$C\left(x_i\ ,t\right)=C_s \tag{7}$$

Regarding mass transport boundaries, a constant mass concentration in the saltwater boundary s usually applied, where C_s is the concentration of seawater. Notice however, that Eq. 7 applies in the lower elevations of the boundary where saltwater water enters the aquifer (Figure 1). At the upper parts of the boundary where water flows out from the aquifer, concentration is unknown and must be computed by the model. This is accomplished by a dynamic BC (Diersch 1998, Smith 2004) as follows: (i) The problem is solved for the BC's determined at the end of the previous time step, (ii) The hydraulic gradient and therefore the convective mass flux are used to specify the new BC. A saltwater boundary is imposed in the lower parts of the boundary where the hydraulic gradient corresponds to inflow of groundwater. Along the remaining portion of the sea boundary a BC of specified concentration is no longer valid and a flux – type BC is assigned (Diersch, 1988), and (iii) The problem is then solved for the new BC's. This updated mass transport constraint result in a decrease of chloride concentration in the upper layers of the aquifer, since the dispersive mass flux is set to zero along the out flowing portion of the boundary.

3. CASE STUDY IN THE ISLAND OF THIRA

Thira is a small and very touristy island located in central Aegean Sea. The available water resources are limited, while the water demands are very high, especially during summer months, due to the increased tourist activities. An attempt was made to meet the freshwater demand through intensive pumping, which finally led to extensive seawater intrusion and deterioration of water quality.

The most important aquifer of the island is phreatic one and is located in the central part of the island. This aquifer was selected in the present study and is shown in Figure 2. The depth to the aquifer base is not exactly known and for the purposes of this study was assumed to be about 120 m below sea level. The aquifer is devided into 30 layers of equal height (4m). The model examined here is a 3-D one, which allows vertical water movement.

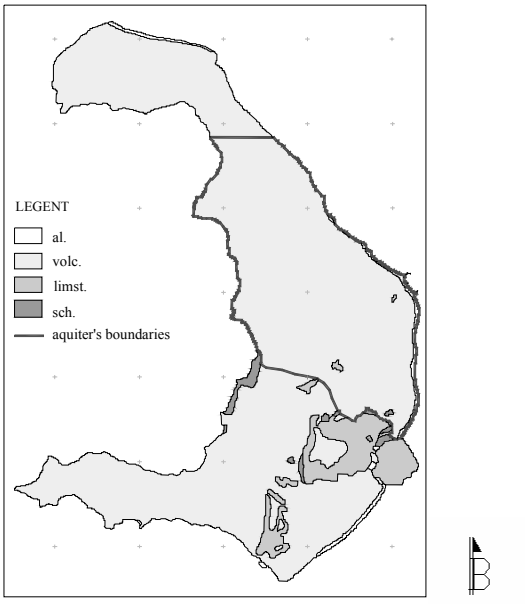


Figure 2. Hydro-lithologic map of Thira Island (Giannoulopoulos and Mantoglou 2003).

The hydraulic conductivity was based on measured piezometric data obtained during the spring of 2003 from a well network. An initial calibration was done by Giannoulopoulos and Mantoglou (2003, 2005) and Mantoglou and Giannoulopoulos (2004), using a simpler flow model. To simplify the complex seawater intrusion model, the aquifer is considered to be isotropic. The recharge estimated in previous studies (Giannoulopoulos and Mantoglou 2003) was applied.

Special mention should be made regarding initial conditions. Given the lack of sufficient data on chloride concentration, an effort to generate credible initial conditions has been made through simulation of groundwater flow for very long periods without pumping.

Regarding the boundary constraint mentioned in Section 2, several simulations with and without the constraint were performed. The final results indicated that the use of the complex dynamic boundary does not significantly change the size of the transition zone on a large scale. Nevertheless, the use of the constraint was finally chosen in the present study in order to test its applicability. The aquifer's response in several scenarios is examined below.

3.1. Constant Pumping and Recharge

In the first scenario the development of seawater intrusion, under constant pumping over a period of 100 years is considered. The total pumping rate from all wells is 722,309 m^3/year, which corresponds to 53.3% of the aquifer's recharge. The individual pumping rate from the wells depends on their location relative to the coast. Regarding recharge, an adjustment was made in order to consider lateral inflows from the aquifer's southeastern border. The density difference ratio value is $a = 0.0254$, while storativity and compressibility were taken as 0.07 and 10^{-4} / m respectively. Porosity and molecular diffusion were 0.3 and $10^{-9} m^2 / s$ respectively, and the longitudinal and transverse dispersivity were 5 m and 0.5 m respectively. Figures 3 and 4 depict a 3D and 2D distribution of mass concentration and hydraulic head respectively, at the end of simulation period.

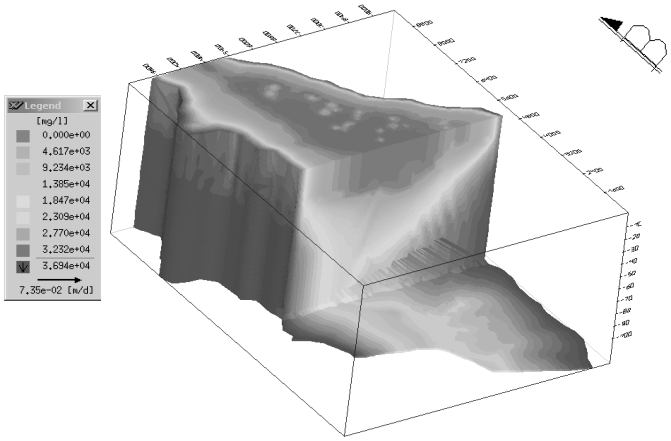


Figure 3. Distribution of chloride concentration (3D projection).



Figure 4. Distribution of hydraulic head.

The results indicate a significant spreading of seawater intrusion, in both horizontal and vertical directions, and a decrease of groundwater quality. At the end of the simulation period no steady state was achieved. The asymmetric advance of the interface zone is due to well distribution since the majority of the wells are located on the eastern region of the island. The transition zone has considerable width which cannot be predicted by sharp-interface approximation models based on the Ghyben-Herzberg relation (Mantoglou and Giannoulopoulos, 2004).

3.2. Time – Dependent Pumping and Recharge

Time dependent simulations should provide better approximations of the real system since they resemble better the underlying physical processes. On the other hand the CPU time required to complete the simulations increases significantly. Recharge was uniformly distributed over a four month time period (January – April) and the total pumping rate was applied based on the monthly water consumption by the local communities. It should be noted that periods of high water consumption coincide with those of low recharge.

The results indicate a fluctuation of the hydraulic head during the year, which rises up to 0.5 m. Regarding chloride concentration, an increase of the transition zone appears. The latter should be attributed to the fact that variable conditions, such as pumping and recharge, increase the thickness of transition zone, whereas steady flows decreases it (e.g. Bobba, 1993). Further simulations for longer periods would allow determining the exact relation between the variability of the parameters and the range of the transition zone.

3.3. Replenishment Scenarios

Two hypothetical cases of aquifer replenishment, with natural and artificial recharge are examined in the present paper. The required data concerning the available total amount of water as well as the available water resources came from previous studies (NAMA, 1998). In the first scenario the aquifer's response without pumping is investigated. The idea was to examine aquifer replenishment through natural recharge. It should be noted that this scenario is not practically feasible, since Thira's water demands cannot be met without the contribution of the island's groundwater. It was found that the hydraulic head is restored relatively quickly while restoration of water quality takes 400 years or more. In reality, the conditions are less favorable, since the pumping rate cannot be minimized to zero.

The 2nd scenario examines the case of artificial recharge. According to previous studies (NAMA, 1998) a total amount of approximately 260,000 m^3 is available for such use. The aquifer would be recharged with the certain amount of water through infiltration basins. A uniform distribution of this water on the entire aquifer surface was assumed, which was achieved by increasing the recharge rate. The total duration of simulation was 100 years. The main conclusion from the last scenario is that the

restoration of the water quality could not be achieved within the simulation time period. However, partial restoration was achieved significantly quicker, compared to the 1st scenario. It should be mentioned that a uniform distribution on the aquifer surface is not possible because of the morphological and geological conditions. Figures 5 and 6 plot the mass concentration in the 15th layer (-60m) for the two replenishment scenarios.

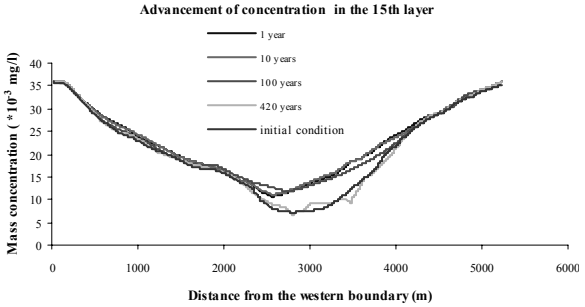


Figure 5. Advancement of chloride concentration along an island cross section (1st replenishment scenario).

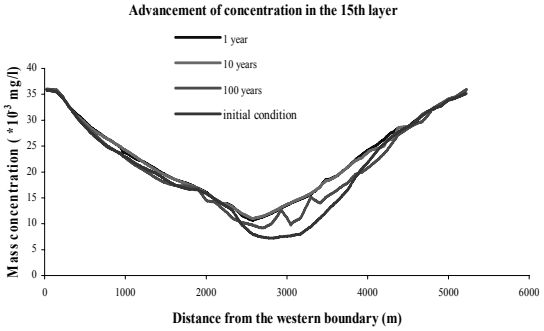


Figure 6. Advancement of chloride concentration along an island cross section (2nd replenishment scenario).

4. SUMMARY AND CONCLUSIONS

Density-dependent models of saltwater intrusion in coastal aquifers were developed and utilized in the case of an aquifer in Thira Island. An improved constraint regarding chloride concentration was imposed at the seawater boundary condition. Although this constraint affects the results in the upper parts of the aquifer, the size of the transition zone is not significantly affected in a large scale. Since the CPU time required to handle this constraint is significantly increased, this approach is not recommended in problems which require dense spatial discretization and time

dependent simulations. The simulation results indicated that the final solutions depend on the initial conditions, and it takes very long time to achieve steady-state conditions.

The main conclusion from this application is that there is significant spreading of seawater intrusion and deterioration of groundwater quality in the aquifer. This spreading cannot be predicted by sharp interface models. It should be noted that the scenarios examined in the present study are moderate, since the aquifer is not pumped to its limit. Finding the limits of aquifer requires a pumping management optimization module.

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Current Conditions of Saltwater Intrusion in the Coastal Rhodope Aquifer System, Northeastern Greece

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Abstract: This paper describes the geological, hydrogeological and hydrochemical regime of the coastal Rhodope aquifer system, North Eastern Greece. This system includes two aquifers within coarse grained alluvial sediments. Both vertical and lateral saltwater intrusion occurs, usually caused by overpumping. Water has been pumped from the aquifer system at an ever-increasing rate for many years. Water samples for chemical analyses were obtained from 41 carefully selected sampling points (36 productive wells and 5 research wells) to cover the entire study area. The EC and chloride concentration distribution illustrate the large extent of saltwater intrusion in the aquifer system of the study area. The concentrations of various ions in the groundwater at the sample sites are highly variable; however, many similarities also exist. The samples are grouped into two main categories according to their chemical composition and the ion domination. Significant nitrate concentrations were measured in the aquifer of the study area, illustrating the effects of the agricultural practices. The results of this study show that the development of a strategy for managing the aquifer system is vitally necessary.

Keywords: Coastal aquifers, salinisation, hydrochemistry.

1. INTRODUCTION

Saltwater intrusion in coastal aquifers is a wide-spread and important phenomenon that degrades water quality by raising salinity to levels exceeding acceptable drinking and irrigation water standards. Saltwater intrusion endangers future exploitation of coastal aquifers (Jones *et al.*, 1999). The problems associated with natural saline groundwater, saltwater intrusion, and upconing of saline water in pumping wells are of major concern in many coastal aquifers of Greece (Lambrakis *et al.*, 1995; 1997). Both vertical and lateral intrusion occurs, usually caused by overpumping.

In the coastal aquifer system of Rhodope, Northeastern Greece, East of Vistonis

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Lagoon, saltwater intrusion occurs because of flows induced by extensive freshwater withdrawals. Petalas (1997) and Petalas *et al.* (1999) listed several saline sources that affect water quality in the Rhodope coastal aquifer, which, however, are not directly related to seawater encroachment. These include displacement of old saline groundwater from the adjacent saline aquifer of Vistonis Lagoon. Water has been pumped from the Rhodope aquifer system at an ever-increasing rate for many years. During the last 25 years, withdrawal has exceeded the maximum quantity of water that can be continuously withdrawn from the Rhodope aquifer system without adverse effects (Petalas and Lambrakis, 2006). The problem is intense particularly in Glykoneri area adjacent to the inlet of Vistonis lagoon (Figure 1).

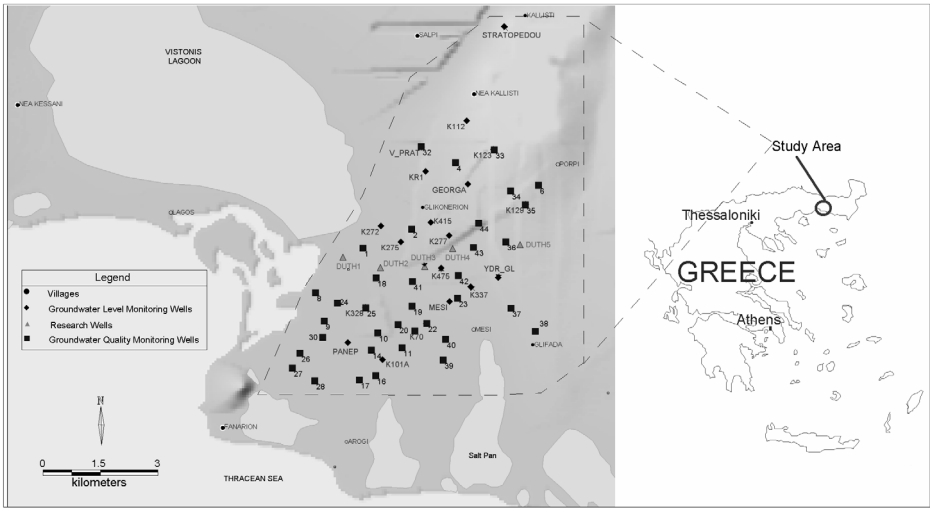


Figure 1. Location map of the study area, indicating groundwater level monitoring wells, groundwater quality monitoring wells and the five research wells.

In this paper the hydrogeological and hydrochemical characteristics of the coastal aquifer system of Rhodope are investigated. The study area has a low relief and mostly gentle slopes. The regional annual precipitation, one of the sources of freshwater recharge of the coastal aquifer, ranges from 329–844 mm. The average annual precipitation is 628 mm. Most precipitation is lost through runoff and evapotranspiration. The climate of the area is Mediterranean with hot and dry summers.

2. MATERIALS AND METHODS

2.1. Investigation of Geological and Hydrogeological Conditions

A detailed investigation was conducted during the last two years in the study area,

based on the existing geological, hydrochemical and surface geophysical data, and on the construction of five fully-cased exploratory wells (including geophysical well logging, spontaneous potential logs, gamma ray logs, SN and LN resistivity logs) open only at the lower confined aquifer, along a WE line adjacent to Vistonis Lagoon inlet. These wells were used to collect water samples and to measure groundwater level at discrete points in the aquifer. A uniformly distributed network of 19 productive wells was utilized along with the 5 newly constructed research wells as monitoring wells. Groundwater level measurements were made using a Solinst Model 101 water level meter. Prior studies of the aquifer system of the study area include the work of Petalas (1997), constituting a hydrogeological and hydrochemical study of a wider area, part of which constitutes the study area. Transmissivity, storage coefficient and horizontal hydraulic conductivity were estimated by means of 10 aquifer pumping tests (Petalas, 1997).

2.2. Groundwater Sampling and Analysis - Groundwater chemistry

In order to establish groundwater quality characteristics, a regional hydrochemical survey was performed in July 2006, in the middle of the pumping period, when considerable groundwater quantities have been extracted. Water samples for chemical analyses were obtained from 41 carefully selected sampling points (36 productive wells and 5 research wells) to cover the entire study area. The methods of sample collection and preservation used are specified in Title 40, Code of Federal Regulations, Part 136 by the U.S. Environmental Protection Agency (USEPA, 1986). Water was only taken from boreholes that were pumped for a significant amount of time. Conductivity (EC), temperature (T) and pH were measured in the field in order to acquire representative values of ambient aquifer conditions. Standard methods were used for chemical analyses (APHA, 1998).

3. RESULTS AND DISCUSSION

3.1. Hydrogeological Conditions and the Conceptual Model of the Aquifer System

The stratigraphic sequence in the study area includes late Miocene formations, composed mainly of interbedded marl, silt, clay, loose sandstone, and conglomerate (Figure 2). Major marine transgressions and regressions and the resulting shifts of facies in either the landward or seaward direction are the chief determinants of the location of regionally extensive high- and low-permeability sediments.

The main aquifer of the system consists of the lower confined unit, made up of silt, loose sandstone, and conglomerate intercalations, of 30 to 60 meters thick. This aquifer serves as the main source of freshwater in the area and has been affected by serious saltwater intrusion, due to overpumping.

An overlying semiconfined aquifer, separated from the lower one by thin clay layers, is developed in alluvial sediments (predominantly interbedded loose sandstone and siltstone). It mainly contains saltwater in a limited area along the coast. The vertical relations between those hydrogeologic units are shown on several generalized hydrogeologic sections produced by interpolating recently acquired data from the research wells (Figure 2).

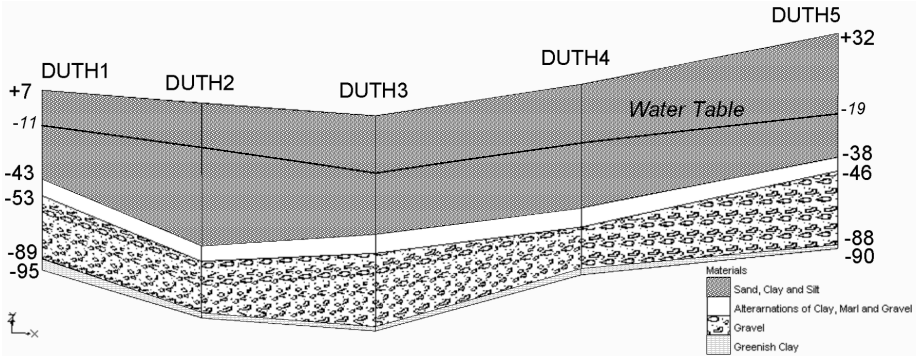


Figure 2. Aquifer layer profile based on well logging from the five research wells.

The above mentioned aquifers are hydraulically connected with the saline phreatic aquifer (Petalas, 1997) in the inlet of Vistonis Lagoon. This phreatic aquifer is directly recharged from saltwater of Vistonis Lagoon. Therefore, due to overpumping, saltwater from the phreatic aquifer intrudes both the semiconfined and confined aquifers, forming thus a sharp interface between freshwater and saltwater.

The hydraulic conductivity of the upper Miocene aquifer system in the coastal area ranges from $8.3x10^{-4}$ to $8.7x10^{-2}$ m/s whereas the storativity (S) ranges from $1.55x10^{-5}$ to $1.18x10^{-3}$ (Petalas, 1997). In the case of the semiconfined aquifer, the respective hydraulic conductivity ranges from $3.5x10^{-4}$ to $2.4x10^{-3}$ m/s and the storage coefficient from $1.25x10^{-5}$ to $3.4x10^{-3}$.

Long term measurements of the water level elevations in numerous wells (Petalas 1997) penetrating the upper Miocene aquifer system showed that water level declined over the last years at least 30 m due to intensive pumping.

Figure 3 reveals that the hydraulic head is seriously reduced in the study area due to overpumping of groundwater and a permanent regional cone of depression has been formed in the potentiometric surface of the aquifer system, western of Porpi. High hydraulic heads at the western boundary of the aquifer system show an increased recharge of the upper Miocene aquifer system. The aforementioned cone resulted in seawater intrusion into the aquifer system from the Southern boundary of the study area and in saltwater intrusion from Vistonis lagoon at the western boundary. Figure 4 illustrates the groundwater level distribution in the study area at 1/4/1993. Comparing with that of 30/3/2006, it is obvious that groundwater level is continuously decreasing leading to saltwater intrusion.

Average groundwater inflows in the aquifer of the wider area are about 33.4x10^6 m^3 (Petalas, 1997) and can be attributed to groundwater recharge to both aquifers, mainly occurring from groundwater inflow from the adjacent Kompsatos River fan aquifer and recharge from rainwater infiltration and irrigation returns. Average groundwater outflows are attributed to pumping for irrigation and are about 35x10^6 m^3. For this a pumping rate of 530 m^3/d could be proposed as the average pumping rate for each productive well during the irrigation period as groundwater safe yield in the study area.

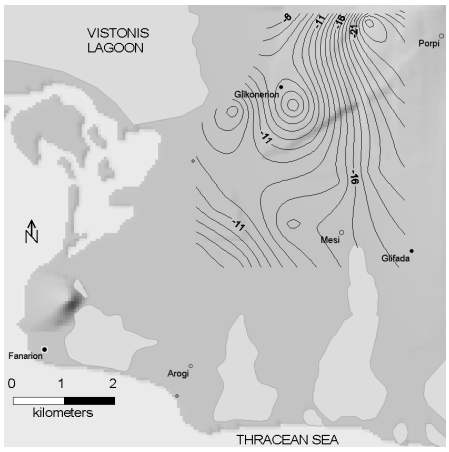


Figure 3. Groundwater level map at 30/3/2006.

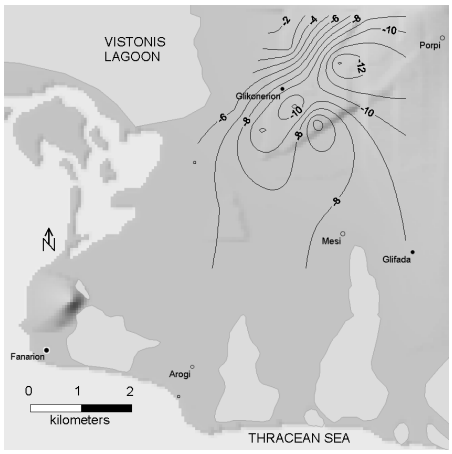


Figure 4. Groundwater level map at 1/4/1993.

3.2. Hydrochemistry Investigation Results

Table 1 summarizes a statistical overview of the chemical analyses of groundwater samples. An electrical-conductivity value greater than 1000 μS/cm was considered to be indicative of saltwater intrusion and was used for mapping contaminated areas (Petalas, 1997). The electrical conductivity of saline groundwater normally ranges from 1,010–14,300 μS/cm. The electrical conductivity in groundwater samples from the freshwater wellfields (confined aquifer) ranges from 554–940 mS/cm. pH of fresh groundwater in the study area normally ranges from 7.47–7.95 (mean value 7.69); pH of saline groundwater ranges from 6.77–8.12 (mean value 7.30). Groundwater temperature ranges from 18.4–24.6 ^{o}C, reflecting a higher than normal geothermal gradient (greater than 1 ^{o}C per 33 m of depth).

The EC (Figure 5) and chloride concentration (Figure 6) distribution illustrate obsviously the saltwater intrusion in the aquifer system of the study area. The assumption that the main point of entry of saltwater into the Rhodope aquifer system is the inlet of Vistonis lagoon is proved because of the fact that in the western part of the study area, EC values were up to 14300 μS/cm and chloride concentration were up to 5158.4 mg/L.

Table 1. Descriptive statistics from chemical analyses of 41 groundwater samples.

	Mean	Median	Standard Deviation	Range	Minimum	Maximum
EC (μS/cm)	3,789.3	3,210.0	2,969.1	13,746.0	554.0	14,300.0
pH	7.4	7.4	0.3	1.4	6.8	8.1
T (^{o}C)	20.9	21.0	1.5	6.3	18.3	24.6
HCO_3^- (mg/L)	219.6	218.2	50.6	355.8	0.0	355.8
Cl^- (mg/L)	1,176.8	894.7	1,087.1	5,084.4	74.0	5,158.4
SO_4^{2-} (mg/L)	67.1	52.6	49.5	236.9	17.9	254.7
NO_3^- (mg/L)	23.6	16.5	26.1	134.6	1.8	136.4
NO_2^-(μg/L)	9.3	0.0	27.1	143.4	0.0	143.4
NH_4^+ (μg/L)	6.6	6.9	3.6	17.7	2.3	20.0
PO_4^{3-} (μg/L)	23.4	19.1	21.8	105.5	1.8	107.3
Na^+ (mg/L)	215.5	182.5	168.8	781.6	31.5	813.1
K^+ (mg/L)	3.4	2.9	2.7	12.4	0.5	12.8
Ca^{2+} (mg/L)	454.4	384.9	356.0	1,648.4	66.4	1,714.8
Mg^{2+} (mg/L)	96.9	82.1	75.9	351.4	14.2	365.5

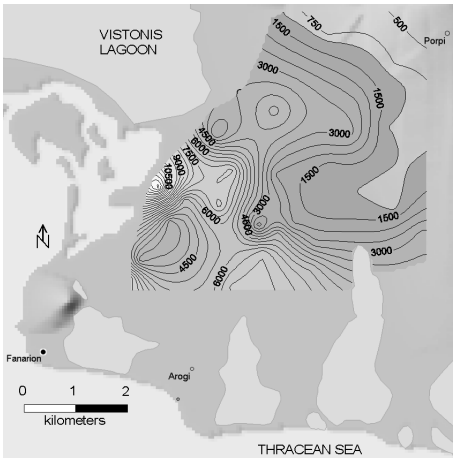


Figure 5. Distribution of EC (μS/cm) in the aquifer system of the study area

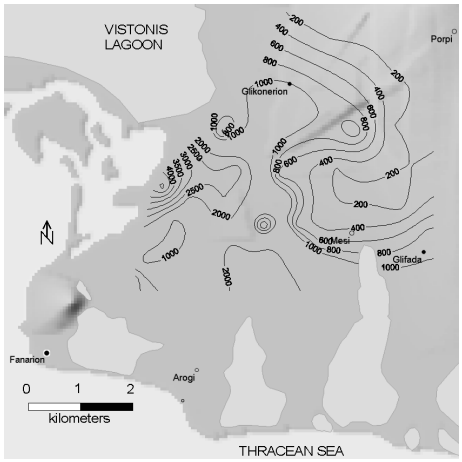


Figure 6. Distribution of Cl (mg/L) in the aquifer system of the study area.

The expanded Durov diagram has been used in order to separate the different groundwater types. According to this diagram, the samples can be classified in two major groundwater groups. The first group includes 7 groundwater samples of the following hydrochemical types Na^+-HCO_3^-, Ca^{2+}-HCO_3^-, whereas the second group including all the other samples has the hydrochemical types Na^+-Cl^-, and Ca^{2+}-Cl^-. Both groups show ion exchange (Lloyd and Heathcote, 1985). The presence of Ca^{2+}-Cl^- and Na^+- HCO_3^- hydrochemical types indicates salinization and remediation of the

groundwater quality phenomena respectively that may be attributed to ion exchange processes between the ions Ca^{2+}, Mg^{2+} and Na^{+} (Appelo and Postma, 1994; Petalas and Diamantis, 1999). The Durov diagram (Figure 7) clearly proofs that both aforementioned competitive processes take place simultaneously. Thus, the salinization process is clear in groundwater samples from wells, located in the southern and south-western coastal zone, while the inverse phenomenon takes place in the northern area because of the presence of recharging waters.

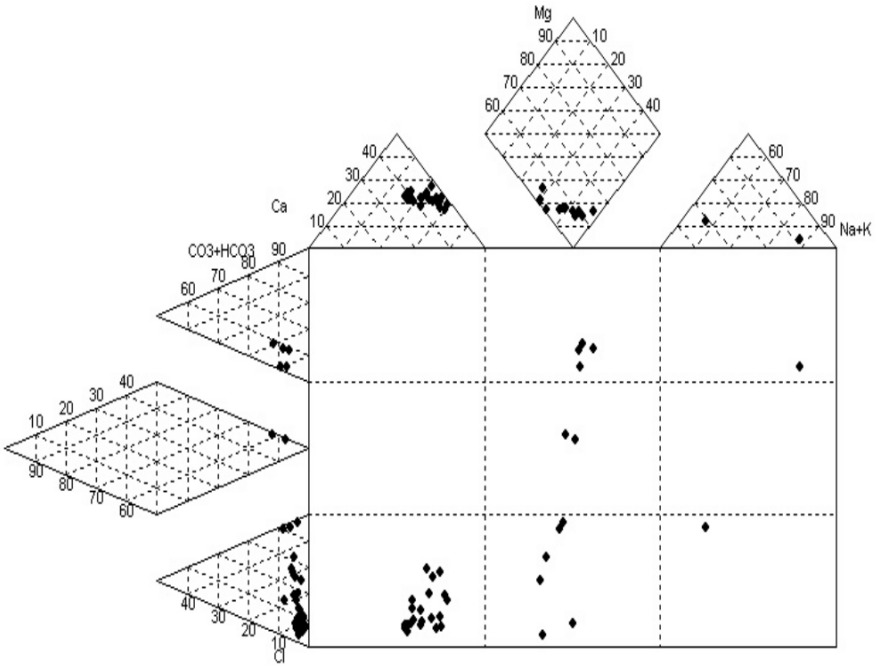


Figure 7. Expanded Durov diagram.

Significant nitrate concentrations were measured in the aquifer of the study area. Mean value is almost equal to the upper recommended concentration value of 25 mg/L proposed by WHO (1985). The nitrate concentration of 14 samples was higher than 25 mg/L and two of them exceeded the maximum allowed nitrate concentration of 50 mg/L. Agricultural practices including the use of fertilizers containing nitrogen, cattle feeding operations and sewage from septic tank systems are the important sources for this kind of contamination (Domenico and Schwartz, 1998).

4. CONCLUSIONS

Groundwater salinization is one of the most common environmental problems in Greece, and definitely the most serious groundwater quality degradation factor of

coastal aquifers. The overexploitation of Rhodope coastal aquifer system for the satisfaction of anthropogenic needs and especially for irrigation has resulted in dramatic decline of groundwater level and thus in saltwater intrusion mainly from the inlet of Vistonis lagoon.

Additional field information and well monitoring are needed to model the behaviour of the zone of saltwater intrusion more satisfactorily and to estimate the rate of movement of the fresh-water/salt-water interface. Use of data gained from the construction of the five research wells resulted in a well defined conceptual approach of the Rhodope aquifer system.

The absence of a management plan is obvious in the study area as water users, and particularly the farmers, install new productive wells without any prior environmental impact assessment, at regions close to the coastline where the saltwater enters the mainland. The optimum economic and environmentally effective development of the Rhodope coastal aquifer system requires an integrated approach in order to maintain sustainability. Firstly, the construction of new pumping wells should be controlled by the local authorities. The modelling of the aquifer system and an optimization scheme could also help in order to locate appropriate places for well construction and maintain the optimum pumping rate. Moreover, several groundwater saving practices could be applied to the aquifer of the study area, including: a) increase of drip irrigation systems; b) increase water use efficiency in irrigation networks by using closed pipelines for water distribution; and c) use of mulch and minimum tillage techniques as a water saving practice in farm level. Training of young farmers in water management techniques could also help in water saving and nutrient pollution reduction. Finally, an artificial aquifer recharge project could be developed and help in sustainabililty maintenance in the study area.

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Dynamic Attribution of Water Quality Indexes in a Multi-Reservoir Optimization Model

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Abstract: In the southern regions of Mediterranean Europe, the greatest part of water resources for supply systems comes from artificial reservoirs. A simplified approach to the need to include water quality aspects in the mathematical optimization model can be achieved by considering the Trophic State Index (TSI) (Carlson, 1977). This paper makes some improvements to the linear optimization model designed for large system management already presented by Sechi and Sulis (2006), addressing the possibility of a dynamic attribution of quality indexes in the LP model. The application of the optimization approach to different operating rules in a real multi-reservoir system in Southern Sardinia highlights the need to consider quality and quantity aspects in an effective water resource management scenario.

Keywords: Complex water systems, trophic status, WARGI-QUAL software.

1. INTRODUCTION

In dealing with water quality in a multi-reservoir system, eutrophication needs to be addressed since it is one of the most serious problems affecting the quality of water stored in reservoirs. The increase in nutrients leads to greater productivity of the water system which may lead to excessive increase in algal biomass or other primary producers such as macrophytes. Excessive algal biomass can seriously affect water quality, especially if it creates anaerobic conditions. Therefore, even when using a simplified approach, which is admissible in view of the mathematical optimization tools, there is a requirement to include water quality indexes associated with the trophic state of reservoirs.

This paper can be considered as a development of the optimization approach presented by Sechi and Sulis (2006). It has been produced for use as a modelling tool to assist decision-makers in identifying and evaluating management alternatives when considering simplified forms of water quality classification. It is well known that mathematical optimization procedures available for large water resources systems are

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still unable to deal with all the complexities of the real world even in cases where they can be easily incorporated in a simulation model. Nevertheless, optimization results obtained by recourse to a model suitably reflecting reality can be seen as a management reference-target for simulation since results can be considered as obtained by an ideal system manager. Many studies have been carried out on the development of optimization models for multiple reservoir systems. However few of them have taken water quality as an objective. Dandy and Crawley (1992) modified an existing linear programming model to identify policies minimizing average salinity in water supply. A network optimization model for water allocation to demands with different quality requirements was described by Mehrez *et al.,* (1992). Hayes *et al.,* (1998) integrated water quantity and quality modelling in an operational model for use in multi-reservoir hydropower systems.

An essential factor in the construction of an advanced optimization model for addressing real problems is linked to the definition of water quality in the water bodies and identification of constraints so as to set out correct system management criteria. In Sechi and Sulis (2006), a criterion for classifying reservoirs in complex water systems, was defined using Carlson's Trophic State Index (Carlson, 1977). Trophic State Index (TSI), which in recent years appears to have attained general acceptance in the limnological community, can be evaluated using Chlorophyll-*a*, Total Phosphorus and Secchi disk transparency measurements. The formulation of water quality constraints was addressed starting from the trophic state of reservoirs basically using Chlorophyll-*a* measurements by means of which it is possible to evaluate the TSI(Chl) trophic index. In this paper an improvement on the approach outlined in Sechi and Sulis (2006) is presented in order to consider the possibility of the dynamic introduction of quality indexes in the optimization model. Instead of static introduction of quality values, the modified model defines TSI(Chl) values according to changes in stored volumes in reservoirs. Two different operating rules are tested to highlight significant increases in system performances when considering quality as well as quantity in a common system management strategy.

2. THE SPLIT-STORAGE GRAPH FOR DYNAMIC QUALITY INDEX ATTRIBUTION

In the optimization model presented by Sechi and Sulis (2006) the physical system in a single period is represented by a direct network (basic graph) consisting of nodes and arcs. A dynamic multi-period network is generated by replicating the basic graph for each period t=*1,T* in the time-horizon *T* and then connecting the corresponding reservoir nodes for different consecutive periods. Objective function (OF) in the model incorporates operation, maintenance and repair (OMR) costs, user-defined costs to assure efficiency of the system and purification costs. Purification costs can be either fixed or dependent on water quality. As usual, optimization model

constraints are introduced in order to take into account relationships between system variables: continuity equations at the nodes, relations between the flow variables and project variables, bounds attributed to these two sets of variables and, finally, water quality constraints. In particular, for each reservoir, quality constraints require that reservoir sources, seen overall, guarantee higher quality (or at least the same quality) as that required for water issued to users. No stricter water quality constraint has been assumed, since the model considers only non-diffusive and conservative water quality processes. In Sechi and Sulis (2006) the major shortcoming of this relation is its linearity. Linearity implies that flux values can be a variable whereas quality indexes must be constrained by pre-assigned values. Even when the proposed model is implemented in a scenario analysis environment (Pallottino *et al.*, 2005) to address the uncertainty factor in water quality by taking into account a set of different scenarios corresponding to possible different evolution over time of quality indexes, the quality indexes were totally independent of volume changes in reservoirs.

In order to overcome this shortcoming, the LP model has been modified to consider the dependency of TSI(Chl) values on storage volume. As already stated in previous papers (Sechi and Sulis, 2006; USGS, 2004) TSI(Chl) and storage volume can be considered as stochastically independent. Nevertheless, intensive monitoring programs implemented in different countries indicate that measured TSI(Chl) values generate different statistical behavior according to different total storage values. According to these analyses, threshold values of total storage within the range of technical feasibility (minimum storage - maximum capacity) can be evaluated. Considering reservoir *j* at time intervals *t=(1, T)*, threshold values are set equal to $C_j^{(k)}$. The model could define *k=1,K* reservoir threshold values. Each storage band limited by threshold values is characterized by statistics of $TSI(Chl)^{(k)}$ (minimum, mean, and maximum values). This approach does not strictly represent the stratification behavior of reservoirs. At time *t* reservoir *j* could involve storage band *k* on the basis of total storage value. To represent system configuration, the basic graph needs to be modified into the split-storage graph shown in Figure 1.

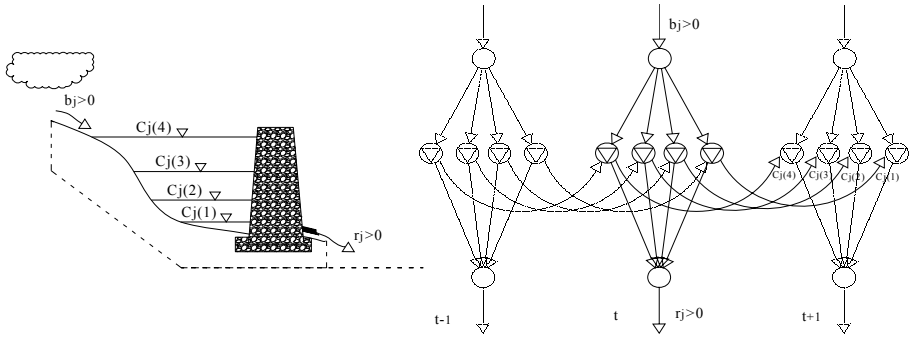


Figure 1. The multi-period split-storage graph for k=4.

Considering applications to real systems, storage bands should be characterized by

different TSI(Chl)$^{(k)}$ estimated using measured values once threshold values have been assigned. At each time step *t*, the TSI(Chl)$^{(k)}$ represents the quality status of the whole reservoir, based on current total storage values.

The OF for the split-storage graph model should take into consideration adequate flow costs configuration. Costs would be assigned also in order to assure correct reproduction of water treatment needs for each storage band.

3. CASE STUDY

Application of the optimization model considering the split-storage graph for reservoir quality modeling was carried out for the *Flumendosa-Campidano* water system. The system extends over South-Eastern Sardinia (Italy), reaching the centre of the Island. Hydrology is typically Mediterranean, with alternation of several drought years with years marked by intense rainfall. A detailed description of the main characteristics of the system is available in reports from European Union Project SEDEMED (2003). Synthetic data describing the system are as follows:

- ten reservoirs with a total capacity of $723 \cdot 10^6$ m^3;
- ten civil demand centers with a total request of $116 \cdot 10^6$ m^3/year;
- nine irrigation demands with a total request of $224 \cdot 10^6$ m^3/year;
- five pumping stations with a total potentiality of $441 \cdot 10^6$ m^3/year;
- two industrial demands with a total request of $19 \cdot 10^6$ m^3/year;
- nine water treatment plants with total treatment capacity of $116 \cdot 10^6$ m^3/year;

one waste-water treatment plant with total treatment capacity of $23 \cdot 10^6$ m^3/year.

Table 1. Flumendosa-Campidano inflow to reservoir and diversion sections.

	Min (Mm^3/yr)	Max (Mm^3/yr)	Mean (Mm^3/yr)	Standard Deviation (Mm^3/yr)
Historical Series (71 years)	134.80	1032.69	438.51	239.35

Regarding the monthly hydrological inflows, data on 19 selected river sections is available on a monthly basis from the official hydrologic database of the Sardinian Regional Authorities ranging over the period 1922-1992 (Table 1). Measurements of the main chemical and biological parameters in the most important reservoirs in the system were made by the Regional Board for water system management (*Ente Autonomo del Flumendosa, EAF*). Sampling for Chlorophyll-*a* was conducted at least once a month from January 1994 to December 2003 at different depth intervals. On the basis of TSI(Chl) evaluated for Flumendosa-Campidano main reservoirs, a single threshold value ($V_j^{k=1}$) of total storage at each reservoir was considered. The threshold value is significant for TSI(Chl) characterization even if the water bodies are not stratified. It defines two different possible statuses for the reservoir at time *t,* as the total storage at time *t* can be above or below the threshold value. Measurements of TSI(Chl) confirm that mean and minimum TSI(Chl) values depend on their being

above or below (C_j^k). As an example, the threshold value equal to 4 Mm^3 for the Is Barroccus reservoir is considered on the basis of values reported in Figure 2. Threshold values in the four main reservoirs of the system are summarized in Table 2.

To address water system optimization in the framework of different synthetic hydrologic and demand scenarios, quality index attribution to reservoirs must be made by extending the procedure examined in (Sechi and Sulis, 2006). Hydrologic and demand scenarios for the Flumendosa-Campidano system have been reconstructed using the official hydrologic database.

Attribution of TSI(Chl) values to the reservoirs in the absence of any kind of direct observation is, obviously, a difficult task and a high level of uncertainty is to be expected for these attributions needed to provide synthetically reconstructed quality data. Several types of relations based on regression analysis were considered with the aim of modeling the dependency of TSI on other data characterizing the reservoirs of the system. Indeed, in order to determine the regression equation, a simple linear least-squares curve was determined between hydrological parameters and TSI values at each reservoir. More complex approaches do not produce significantly better results, at least for the water bodies under consideration.

Table 2. Threshold values and maximum capacity in the main reservoirs .

	Flumendosa	Mulargia	Is Barroccus	Cixerri
Threshold storage [10^6 m^3]	100	150	5	10
Maximum Capacity [10^6 m^3]	316	347	14	32

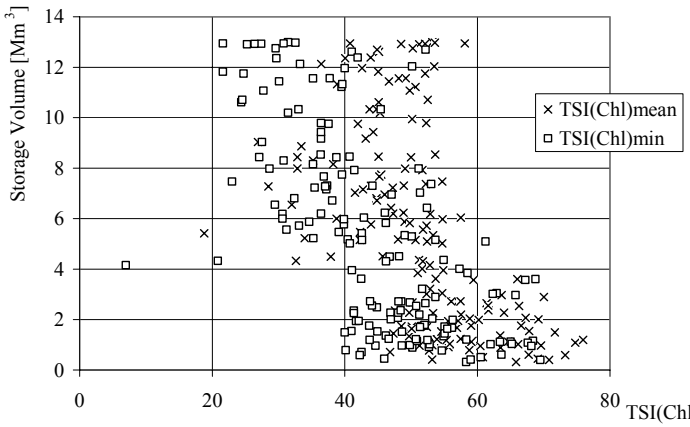


Figure 2. Relationship between Storage Volume and TSI(Chl) in the Is Barroccus reservoir.

The equations used to model the relationship between water quality parameters and hydrological input to reservoirs is a multiple linear regression incorporating month-explanatory TSI variation in the reservoir when under or above threshold storage values:

$$TSI_t^{(1)} = a_1 \cdot TSI_{(t-1)} + b_1 \cdot T + c_1 \cdot HI_t + d_1 \cdot \sum_{i=1}^{N} HI_{t-i}^{1/i} + h_1, \quad if \quad V \le C_{crit}$$
$$TSI_t^{(2)} = a_2 \cdot TSI_{(t-1)} + b_2 \cdot T + c_2 \cdot HI_t + h_2 \quad if \quad V > C_{crit} \tag{1}$$

where,
TSI_t: evaluated variable at current time t;
$TSI_{(t-1)}$: evaluated variable at the antecedent time period;
T: transformed time date;
$HI_{t-i,\ i=0,\ldots,N}$: hydrological inflow in the reservoir in the ith antecedent time period;
a,b,c,d,h: regression coefficients;
C_{crit}: threshold value in storage volume.

Equation (1) can be used to model minimum (TSI_{min}) and mean (TSI_{mean}) values. Coefficients have been evaluated by measures taken at each reservoir. TSIs attribution to storage bands has been made based on stored volume at measurement time. Analysis has shown that, to better relate hydrologic inflows to TSI_{min} and TSI_{mean} values, it is not possible to select 'a priori' a single time horizon *N* for all reservoirs. The best choice of time horizon probably depends on the ratio between storage capacity and mean inflow rate, as well as on the characterizing physical, chemical and biological processes influencing water quality status at each reservoir.

Seasonal fluctuation in processes was accounted for by inserting into the model a transformed time date. The behavior of both observed TSI values (TSI_{min} and TSI_{mean}) shows that two peaks in TSI exist during the year and that these two peaks are separated by about six months. The time function used is expressed as follows:

$$T = \sin\left[\frac{1}{2}(t - t_0) * \frac{2\pi}{P}\right] \tag{2}$$

where,
t: date of the observation;
t_0: time of selected origin;
P: time period cycle in TSI value (one year).

Regression coefficients of equations (1) were estimated using ordinary-least-squares multiple linear regression. The best equation at each reservoir was selected on the basis of the coefficient of determination, standard error of the regression and F-value. Coefficients of determination R^2, residual standard errors σ and parameters of each relation are listed in Table 3. Coefficient of determination ranges from 0.44 to 0.84 and, for this kind of analysis, it can be considered sufficiently accurate to perform subsequent determinations (Håkanson, 1999).

Pursuant to Italian legislation, in (Sechi and Sulis, 2006) the Authors described the procedure for associating a Quality Evaluation (QE) index in the form of a single synthetic quality index varying from 1 to 5. On the basis of TSI_{min} and TSI_{mean} values we attributed a minimum and a mean QE value to the water stored. Figure 3 shows the frequency distribution of QE_{min} and QE_{mean} in the 5 classes for the reservoirs monitored.

Table 3. Regression results for the Flumendosa, Mulargia, Is Barroccus and Cixerri reservoirs (V≤ Ccrit ; V>Ccrit).

		R^2	σ	a	b	c	d	h
Flumendosa	TSI_{mean}	0.84;0.51	0.51;4.89	0.01;0.26	0.12;-5.93	1.57;-1.09	0.08	-4.94; 28.71
	TSI_{min}	0.73;0.64	8.08;4.56	1.43;0.32	15.25;-1.47	2.58;0.84	-0.13	-44.85; -0.41
Mulargia	TSI_{mean}	0.66;0.65	3.39;5.02	0.49;0.57	-0.51;-7.61	-1.03;-0.21	-0.69	24.32; 20.61
	TSI_{min}	0.71;0.65	6.39;3.67	0.73;0.33	6.68;3.39	-1.04;0.14	1.63	3.09; 17.54
Is Barroccus	TSI_{mean}	0.47;0.57	3.19;3.51	0.33;0.23	-5.52;-7.52	0.84;0.24	-0.35	36.64; 50.51
	TSI_{min}	0.51;0.49	7.89;6.94	0.55;0.57	0.08;2.68	2.82;-3.00	-2.26	20.93; 26.57
Cixerri	TSI_{mean}	0.44;0.51	2.26;8.73	0.26;0.55	0.67;-0.77	0.14;-0.36	0.21	51.50; 29.63
	TSI_{min}	0.59;0.46	8.57;3.29	0.60;0.48	-0.17;0.48	-0.32;0.23	-0.38	24.41; 34.95

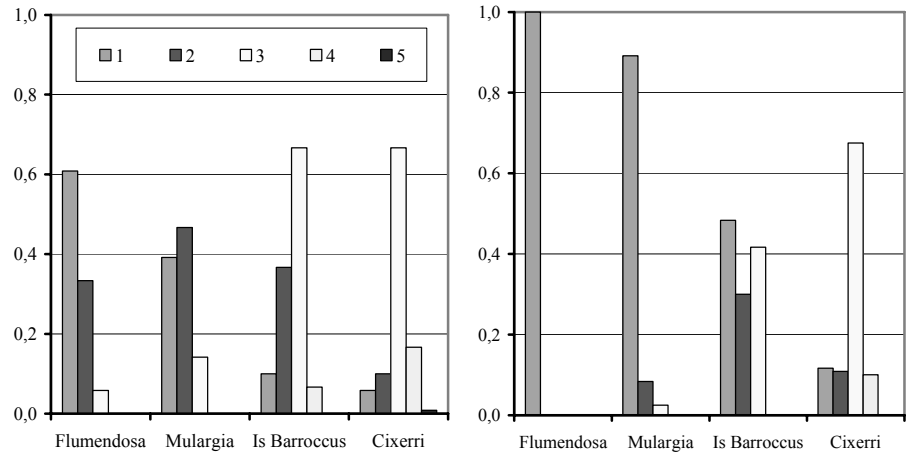


Figure 3. Frequency distribution of QE values on the basis of TSI(Chl)mean and TSI(Chl)min estimation.

Required QE values (QE_R) are the lowest QE values that must be guaranteed for demand centers. Modeling the Flumendosa-Campidano system, QE_R are assigned equaling 1 for urban use, 3 for industrial use and 4 for irrigation use.

4. RESULTS AND CONCLUSIONS

Water drawn from reservoirs can be located at different depths. This allows use of water to obtain better quality in the reservoir: so-called “selective withdrawal”. Being able to regulate both quantity and quality of releases, selective withdrawal is a typical reservoir-operating rule that integrates these aspects in the management of a reservoir system. When the selective withdrawal operating rule (OP(1)) is used, water released from the reservoir is characterized by a QE value equal to QE_{min}. On the other hand, a non-selective withdrawal operating rule (OP(2)) does not select the best layer and

considers only quantity issued. In this case, the QE value can be considered equal to QE_{mean}. These operating rules have been implemented in the model proposed by Sechi and Sulis (2006) by means of two different constraints.

The split-storage optimization model described above was implemented to assess to what extent system performances improve when considering quality and quality aspects in a common management strategy (OP(1)), instead of focusing attention on water quantity alone, using a non-selective withdrawal operating rule (OP(2)). Optimization analysis was carried out considering a 54-year time horizon and the two optimization procedures in order to highlight the consequences of examining quantity and quality issues jointly. Time reliability (expressed as percent value of the months in which deficit is equal or less than predefined thresholds) and volumetric reliability (expressed as deficit rate on the demand value for the entire period of analysis) are used to quantify system performances.

Table 4. Performance indexes using OP(2).

	Urban Demand	Industrial Demand	Irrigation Demand
Temporal reliability (% of months) Deficit = 0%	87.29	96.35	44.03
Temporal reliability (% of months) Deficit < 25%	88.02	97.40	45.55
Volumetric reliability (%)	88.72	97.77	58.43

Using the non-selective operating rule (OP(2)), Table 4 summarizes the degree of reliability of system management. Poorest volumetric reliability is obtained for irrigation (58%) since urban and industrial demands are marked by the higher deficit penalization cost in the OF. Temporal reliability values are estimated considering deficit values of 0% and 25%. As was to be expected, applying a selective withdrawal operating rule led to modification of optimal flux configuration. Operation of a reservoir under quality criteria can achieve the most restrictive downstream use, called designed use (Loucks and van Beek, 2005). System performances in Table 5 show that OP(1) guarantees designated uses better than OP(2). Industrial demand is met during the whole time horizon. Temporal and volumetric reliability in civil demand improve significantly. Persistence of a significant deficit in irrigation demand occurred. In this case, not even a selective withdrawal rule is able to ensure better performances.

Table 5. Performance indexes using OP(1).

	Urban Demand	Industrial Demand	Irrigation Demand
Temporal reliability (% of months) Deficit = 0%	92.92	100.00	45.15
Temporal reliability (% of months) Deficit < 25%	93.13	100.00	48.87
Volumetric reliability (%)	94.65	100.00	66.92

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Regional Scale Irrigation Scheduling using a Mathematical Model and GIS

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Abstract: A user – friendly software that contributes to the regional scale irrigation scheduling is presented. Its ability to produce coloured maps of irrigation water needs is derived from the solution of the Richards equation by the numerical model SWBACROS, which was integrated into the ArcGIS environment. The study region should be densely sampled in order to be divided into texture and crop homogeneous sub-regions. Spatial variability of the soil hydraulic parameters inside each sub-region is handled by applying the scaling method to the set of point measurements contained within each sub-region. This paper describes the main theoretical considerations behind this software and compares two irrigation scenarios.

Keywords: GIS, irrigation scheduling, regional scale, Richards equation, scaling method.

1. INTRODUCTION

The use of water for irrigation is by far the greatest consumer of fresh water globally. Although irrigated regions of the planet occupy approximately 17% of the cultivated regions, they consume more than 70% of the world's water resources (Wolff, 1999). Saving just a small amount of the water destined for irrigation and using it for other purposes instead, (mainly drinking use) could improve the living conditions of millions of people. For example, in Morocco by transferring just 5% of the currently used irrigation water to drinking use is enough to almost double the amounts of drinking water (Wolff, 1999).

The use of mathematical models for the simulation of soil water movement in the unsaturated zone is a precise approach to rational irrigation and the key to irrigation water saving. Most of the mathematical models are simple water balance applications in contrast to the detailed mathematical ones that solve the partial differential

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equation known as the "Richards equation". Additionally the majority of the mathematical models do not take into account the spatial variability of the soil hydraulic parameters.

The combination of a detailed mathematical model, which solves the Richards equation, with some method of handling the spatial variability is therefore highly desired. If this could be done by a user-friendly graphical interface from inside a GIS software, then a regional water resources framework would be born. Using such a software would help the irrigation scheduling of a large agricultural area. In addition to saving fresh water meant for irrigation, the software would greatly contribute to (i) the preservation of the environment by reducing the surface and subsurface water resources pollution with agrochemicals and (ii) minimizing the degradation of soil quality due to the use of bad quality irrigation water.

The software in question, due to its increased requirements in specialized data, is not aimed at the farmer but rather to an organization that could provide irrigation advice services to the farmers. Such services, although well provided abroad, (Smith and Muñoz, 2002) are still lacking from Greece.

In this paper such a new software is presented. It was built in the Hydraulics Laboratory of the School of Agriculture of AUTh and can be used to schedule the irrigation of large agricultural areas. Its core is the mathematical model S.W.BA.CRO.S. (**S**oil **W**ater **BA**lance of a **CRO**pped **S**oil) model which interacts with ESRI® ArcGIS using code partly created in Visual Basic (VB) and partly in Visual Basic for Applications (VBA). The ArcGIS software extends the S.W.BA.CRO.S. capabilities and takes advantage of its accuracy, thus making it a valuable environmental decision support system for any organization in charge of operating collective irrigation networks. The reason is because it provides them with a way to achieve the sustainable management of irrigation water.

2. THEORETICAL CONSIDERATIONS

2.1. Mathematical Simulation

Computer simulation is a very useful and low cost tool for irrigation scheduling in the hands of managers of collective irrigation networks. Most of them follow the simplified water balance approach. A small number of them solve the well known and data demanding Richards equation (Richards, 1931). This partial differential equation describes the movement of water through unsaturated porous media, subject to appropriate boundary and initial conditions, while it accounts for the water uptake from the plant roots. The solution of this equation provides the depth distribution of soil water in a cultivated soil at one point in time.

The mathematical model S.W.BA.CRO.S. which is used in this software, solves the one-dimensional form of Richards equation using the Douglas-Jones predictor-corrector finite difference scheme (Douglas and Jones, 1963; Babajimopoulos, 1991;

Babajimopoulos, 2000). This implicit scheme has been proven to be one of the most satisfactory in simulating the one dimensional movement of soil water in the unsaturated zone (Haverkamp *et al.*, 1977; Babajimopoulos, 2000).

2.2. Description of Hydraulic Functions Spatial Variability

A very useful and practical method of describing the spatial variability of h(θ) and K(θ) functions is the scaling method (Miller and Miller 1955a; 1955b). According to this, the variability of a hydraulic parameter in a region is described by a parameter called scaling factor α which (i) is characteristic for each position and (ii) relates the measured values of hydraulic parameters in the sampling positions to the most representative (or "mean") value of the region.

The scaling method has been widely applied to problems that deal with the spatial variability of soil hydraulic parameters. Among other distinguished papers based on it, are those of Klute and Wilkinson (1958); Philip (1967); Warrick *et al.* (1977); Vogel and Cislerova (1993); Kelleners *et al.,* (1999) and Tuli *et al.,* (2001). A very good collection of publications may also be found in Hillel and Elrick (1990). An experimental investigation in the Greek language can be found at Georgoussis and Babajimopoulos (2006).

The simple approach of scaling theory was the reason for using it in this paper. The method used was proposed by Peck *et al.* (1977). According to this, for each location r (out of a total of R locations) of a soil layer k in a region, measurements (h_i,S_i) and (K_i,S_i) exist where S is the degree of saturation and i=1…I each step of degree of saturation. If we consider scaling factors $\alpha_{r,k}$ according to:

$$\alpha_{mean,k} = \frac{1}{R}\sum_{r=1}^{R}\alpha_{r,k} = 1 \tag{1}$$

then it can be shown that (Peck *et al.*, 1977**;** Ahuja *et al.*, 1984):

$$h_{mean,k}(S_i) = \frac{R}{\sum_{r=1}^{R}\left(1/h_{r,k}(S_i)\right)} = \frac{R}{\frac{1}{h_{1,k}(S_i)} + \frac{1}{h_{2,k}(S_i)} + \ldots + \frac{1}{h_{R,k}(S_i)}} \tag{2}$$

and

$$K_{mean,k}(S_i) = \frac{1}{R^2}\left(\sum_{r=1}^{R}\sqrt{K_{r,k}(S_i)}\right)^2 = \frac{1}{R^2}\left(\sqrt{K_{1,k}(S_i)} + \sqrt{K_{2,k}(S_i)} + .. + \sqrt{K_{R,k}(S_i)}\right) \tag{3}$$

Afterwards the scaling factors can be estimated from:

$$\alpha_{r,k}^{h} = \frac{1}{I}\left[\sum_{i=1}^{I} h_{mean,k}(S_i)/h_{r,k}(S_i)\right] \tag{4}$$

and

$$\alpha_{r,k}^{K} = \sqrt{\frac{1}{I}\sum_{i=1}^{I} K_{r,k}(S_i)/K_{mean,k}(S_i)} \tag{5}$$

2.3. GIS Integration Implementation

The aim of the present study was the creation of a software that can assist the farmers of a large region with diverse cultivations in deciding when and how much to irrigate their fields, taking into account the spatial variability of the hydraulic parameters of the soil. Integrated into the ArcGIS environment it can produce maps of irrigation water needs through a user-friendly interface and provide the user with the possibility of investigating various irrigation scenarios under different climatic conditions.

The application of the software in a region consists of various steps; the first being the division of the study region into sub-regions of common crops and common soil texture of the 0–30 cm and 30–60 cm depths. For example, such a homogeneous sub-region could be the "Cotton – Medium – Heavy" (coded as COT_MH) which is (i) cultivated with cotton (ii) the 0–30 cm texture is classified as of medium composition in the United States Department of Agriculture (U.S.D.A.) taxonomy system while (iii) the 30–60 cm is classified as of heavy composition. In order to do this division, the initial region is densely sampled. Afterwards the sand, silt and clay percentages are kriged in the well known ArcInfo extension Geostatistical Analyst which produces two layers of soil texture, one for each of the two depths.

Overlapping a third layer (the cultivation layer) onto the previous two, produces the final sub-regions whose "mean" hydraulic parameters are then found using the scaling method described in paragraph 2.2.

The necessary for the simulation meteorological data are gathered from the nearest weather station and can either be considered the same for the whole region or can be different for every sub-region (in case the climatic conditions of the region are monitored by more than one weather stations).

Simulation is then executed and the result is the mean root zone water content. This is compared to the field capacity and a user defined maximum allowed depletion limit, different for each sub-region.

It must be noted that the use of such a software demands that the user be an experienced agricultural engineer who can coordinate the collection of various input data, ranging from soil to plant and weather data.

2.4. User Interface

Using parts of VB6 and VBA code, a new toolbar has been added in ESRI® ArcGIS. For compatibility reasons version 9.x of ArcGIS should be used. The new toolbar gives the ability to carry out a single or multiple sub-regions simulation. After following some simple steps, the user can view the results (root depth mean soil water content – Figure 1) for the whole of the simulation period using a calendar (single region case – Figure 2) or by just taking a look at the coloured map (multiple region case – Figure 3).

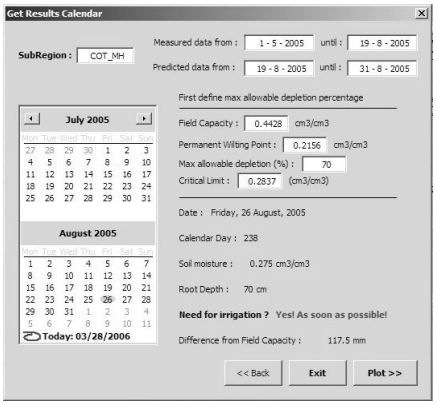


Figure 1. Viewing the simulation results of a specific sub-region.

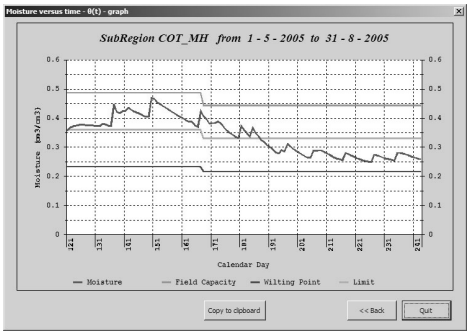


Figure 2. Root depth mean soil water content changes in a simulation period.



Figure 3. Visualization of the irrigation water needs of a specific day.

3. TEST APPLICATION AND SCENARIOS

The software was applied for testing and verification purposes to a 6302.1 hectare part of the Thessaloniki plain (Figure 4). This so called Halastra and Kalohori part is

irrigated by a collective gravity irrigation network, whose waters are managed by the local land reclamation organization that is called “TOEB of Halastra and Kalohori”.



Figure 4. Test region.

The test region has been densely sampled with a total of 625 sampling points (Figure 5) to determine sand, silt and clay percentages.



Figure 5. Dense sampling in test region.

Two scenarios were investigated for the test region: one using the real weather data of May to September 2004 and the second assuming a 10% increase in air temperature and 80% decrease in rainfall on the 2004 data. The aim was to investigate the effect of extreme weather conditions on irrigation scheduling of the existing cotton (COT), corn (COR), tomato (TOM) and alfalfa (ALF) fields.

The software ability to map the changes in soil water content can be demonstrated if irrigation is deliberately withheld for some days (Figure 6):

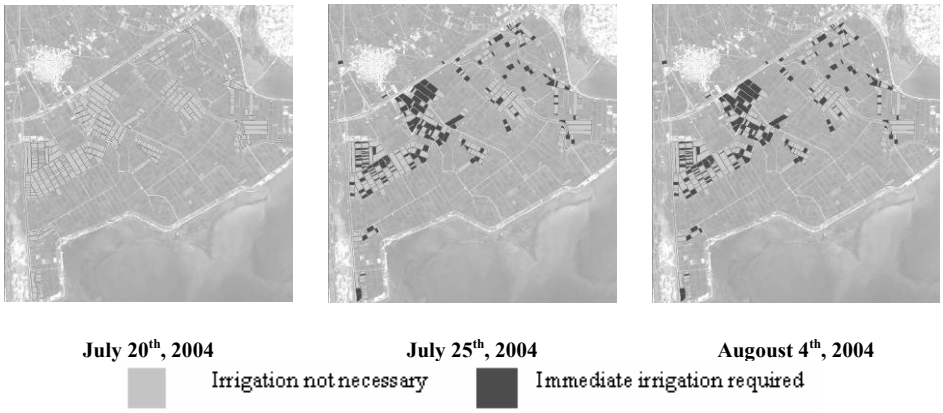


Figure 6. Soil water decrease with time.

Comparison of the irrigation scheduling of the two scenarios showed that the scenario weather data caused an increase in the amounts of applied water up to 58% more than the real 2004 data.

4. CONCLUSIONS

A GIS integrated software that contributes to regional irrigation scheduling has been presented herein. To achieve its goal, the initial region has been divided into homogeneous sub-regions. The mean hydraulic properties of each sub-region have been found using the scaling method. Soil water distribution – over time is presented via graphs or colored maps from inside ArcGIS 9.

Two scenarios were investigated after applying the software to part of the Thessaloniki plain using (a) the May – September 2004 weather and crop data and (b) data hypothetically showing an 80% decrease (-50.1mm) in rainfall with a concurrent 10% increase (+2.2°C) in the mean daily air temperature of the same period.

The rational irrigation schedule suggests an increase of 12% (for ALF_MH sub-region) to 77% (for COT_MM sub-region) in applied irrigation water, if the scenario weather data prevail in the region.

Improvements needed to the regional irrigation scheduling using this software are: (i) understanding the usefulness of such a kind of software and the necessity to disseminate the results of it to the farmers (ii) the increase in sampling points in order to improve the kriging estimates of sand, silt and clay percentages and consequently the delineation of sub-regions (iii) the increase in the number of points to determine soil hydraulic functions which consequently improves the “mean” functions estimates via scaling (iv) the integration of plant-water relations in order to fine tune the irrigation scheduling under insufficient irrigation conditions (v) programming of a variable in time maximum allowable depletion limit.

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An Application of SWOT Analysis and GIS for the Optimization of Water Resources Management in Korinthia Prefecture, Greece

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Abstract: The prefecture of Korinthia in NE Peloponnesus is characterized as an agrotouristic area, with non homogeneous distribution of rainfalls and water resources, and is mainly dependent on groundwater resources for its water supply. This study deals with the determination of parameters of water balance of all the hydrologic basins. Monthly rainfall and temperature values from seventeen stations were used, utilizing the technology of geographic information systems (GIS). The main aquifer systems are developed in carbonate rocks and alluvial deposits. In SW part of the studied area the rainfall is high and decreases towards eastern and northern parts. The unequal distribution of precipitation effects the occurrence of water surplus in the western part and water deficit in the eastern part of Korinthia prefecture. SWOT analysis was applied in order to evaluate the strengths, weaknesses, opportunities and threats of the groundwater resources. Based on the results of the above analysis a set of measures are proposed in order to achieve sustainable water resources management of Korinthia area.

Keywords: Korinthia prefecture, SWOT analysis, water resources management.

1. INTRODUCTION

The socio-economic development in the Korinthia prefecture will be determined mainly by the water, rather than any other natural resource, as this prefecture shows significant regional variations in water availability. Unfavourable climatic conditions combined with the continuous population growth, rapid urbanization, touristic and intensive agricultural development have increased the demand for water.

Water needs are covered mainly by groundwater abstracted by numerous (approximately 6000) wells and boreholes. The intense groundwater exploitation has led to head decline and quality deterioration. A decline of groundwater levels has been observed since 1980's, due to overexploitation by agriculture and a growing population in coastal areas combined with prolonged drought periods (Voudouris *et al.,* 2002). Seawater intrusion phenomena have been recorded in many coastal

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aquifers. The high nitrate concentrations should be attributed to nitrogen over-fertilization. Further deterioration can be associated with disperse pollution sources in the area, namely urban and industrial waste disposal and the use of septic tanks (Panagopoulos *et al.,* 2002).

The aim of this study was the optimization of water resources management using GIS technology and SWOT (Strengths, Weaknesses, Opportunities, Threats) analysis. SWOT analysis was applied as a technique for the optimization of the sustainable groundwater management. This work presents the results of a hydrogeological study, including development of aquifers, as well as the hydrological balance for each individual basin of the Korinthia prefecture, which was estimated using Thornthwaite method and GIS. The analysis was accomplished on the basis of a raster grid with spatial resolution of 20 m. Finally, a set of actions were proposed towards the direction of sustainable management of water resources, using SWOT analysis.

2. GENERAL FEATURES OF KORINTHIA PREFECTURE

The prefecture of Korinthia (Figure 1) covers a total area of 2290 km² and has a population of 153,940 (data 2001). The population has significantly increased during the last two decades, especially in coastal areas (8.5% between 1991-2001 and 25% between 1981-2001). The region has the further advantage of being very near to Athens and as a result attracts many permanent residents of Athens to spend weekends and summer holidays in the coastal areas of Korinthia prefecture.

A percentage of 18.3% of the total area belongs to lowlands, 22.5% belongs to semi-mountainous and 59.2% belongs to the mountainous area. The prefecture is surrounded by sea (Korinthiakos and Saronikos Gulf) and mountains in the southern part: Killini (2376 m), Chelmos (2109 m) and Oligirtos (1934 m). Currently 30.5% of land is occupied by forest, 26% by grassland and 31.6 km² (1.3%) is occupied by waters. More than 35% of the total area is used for agriculture. Irrigated land makes up 39% of all farmland (315 km²). The land is used mainly for the cultivation of citrus fruits, olives, apricots and vineyards. Chemical fertilizers are used for the cultivated crops in the prefecture. Commonly, $(NH_4)^2SO_4$, K and P-fertilizers are used. For the domestic and municipal usages the withdrawal is 15x106 m³/y. Irrigated agriculture is the largest water using category in Korinthia with withdrawal rates up to $165x10^6$ m³/yr (Ministry of Agriculture, Unpublished data). The Digital Elevation Model (DEM) was used in order to delineate watersheds. The study area consists of 23 drainage basins as shown in Figure 1. The greatest is the Asopos river basin, covering an area of 257 km². Flow in the torrents persists throughout the wet period. The annual streamflow depends on the amount of rainfall. In the study area there is the shallow lake Stimfalia, which is one of the Greece's mountainous lakes, covering an area which ranges from 350-770 hectares. The study area has a semiarid climate in lowlands and a typical Mediterranean climate in highlands. Generally 85% of total rainfall occurs mainly in the wet period, which normally extends from late October

through April. Extreme climatic events in the form of both droughts and floods have been occurred (Voudouris *et al.,* 2002).

The following geological formations can be identified in Korinthia prefecture (Figure 1): 1. Quaternary deposits consisting of alternations of coastal (sands) and fine, coarse or mixed facies formations, 2. Plio-pleistocene deposits consisting of fine, coarse or mixed facies sediments (marls, sandstones, conglomerates etc), which cover a high extent in the northern and central part of the study area. Base rock formations (flysch, limestones-dolomites, semi-metamorphic formations), mainly cover the SW part of the area. The study area is covered by integrated geological maps at a scale of 1:50,000 (Institute of Geological and Mineral Exploration, IGME).

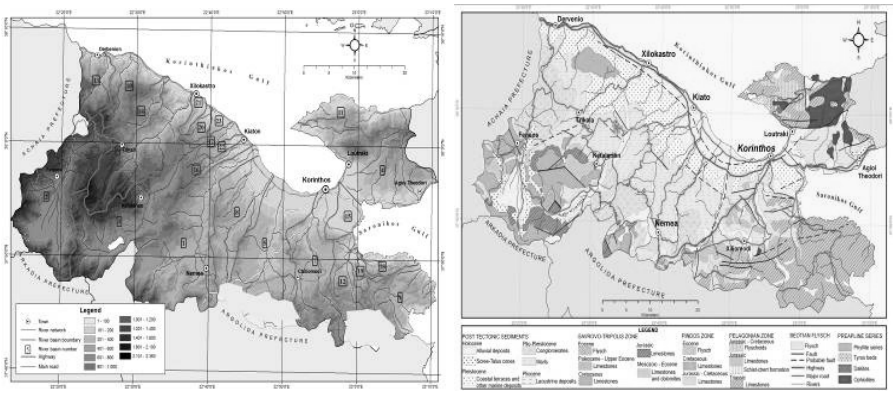


Figure 1. Topographic map of Korinthia prefecture, showing the hydrological basins (left). Geological map of Korinthia prefecture (Voudouris *et al.*, 2006).

3. WATER RESOURCES

Based on the relationship between the mean annual rainfall and geographical parameters of each rain gauge stations, the annual rainfall distribution map was illustrated using GIS technique (Voudouris *et al.,* 2006) (Figure 2). As illustrated in Figure 2, in the SW part the rainfall is high and decreases towards eastern and northern parts of the studied area. The unequal distribution of precipitation effect the occurrence of water surplus in the western part and water deficit in the eastern part of Korinthia prefecture. The mean annual rainfall was estimated to be 1.39×10^9 m³.

According to Thornthwaite method, the estimated actual evapotranspiration (Er) is 74%-81% of the annual rainfall in lowlands and 38-50.5% in mountainous areas (the long term mean annual is 0.72×10^9 m³). Using GIS, the hydrological balance for each basin as shown in Fig. 1, is listed in Table 1. The spatial distribution of the infiltration coefficients derived from the product of the raster of infiltration coefficients to the raster of annual rainfalls. Infiltration coefficients of similar formations in Greece were adopted for the geological formations. The analysis was accomplished on the basis of a raster grid with a cell size of 50 m x 50 m (Voudouris *et al.,* 2006).

The total infiltration (I) and streamflow (R) were estimated to be 0.29×10^9 m^3/yr or 21% and 0.38×10^9 m^3/yr or 27% of the annual rainfall, respectively. The mean water balance shows a water surplus for the period January-April, while in May-June the process of consumption of soil humidity takes place. Water deficit is observed from June to October, while the period November-December is the replenishment period. The major aquifers in the prefecture of Korinthia occur within the carbonate rocks (total area 587 km^2) of the area (karstic aquifers), due to their high infiltration capacity (47-52 of the annual rainfall). The most important amongst them is the aquifer developed within carbonate rocks of Kyllini mountain. In the SE part of the prefecture, the limestones are in contact with the sea. This fact, in combination with the faults of East-West direction that occur in this area, cause the seawater intrusion at a distance of 15 km from the coastline. A volume of water infiltrates in carbonate rocks and then a part of this volume is discharged through springs; submarine brackish water or inland freshwater springs (117.5×10^6 m^3/yr). The chloride concentration of submarine springs ranges between 5000 and 7000 mg/L, depending on the period of the hydrological year. Karstic aquifers are important water resources, but are highly vulnerable to contamination due to relatively fast transport and limited attenuation processes.

Table 1. Hydrologic balance in the basins of Korinthia (Voudouris *et al.*, 2006).

	Basin	Area (km^2)	P ($x10^6$ m^3)	Er ($x10^6$ m^3)	I ($x10^6$ m^3)	R ($x10^6$ m^3)
1	Rachianis	203.4	102.3	66.1	13.9	22.3
2	Zapandis	136.0	70.2	45.0	5.7	19.4
3	Elisson	33.7	19.6	11.0	2.0	6.6
4	Sithas	180.5	134.3	54.7	30.6	48.9
5	Fonissa	54.5	37.5	18.4	3.6	15.4
6	Stimfalia	218.7	175.7	73.0	45.6	57.0
7	Skoupeikos	71.6	45.0	23.8	9.4	11.8
8	Olvios	238.4	208.9	76.9	52.0	79.9
9	Dervenios	61.5	41.5	20.6	5.3	15.6
10	Asopos	279.5	175.0	98.8	34.4	41.8
11	Potamias	168.3	89.7	53.3	22.6	13.8
12	Examilia	37.5	15.2	10.7	2.2	2.3
13	Arachnaio	101.7	49.5	30.3	18.0	1.2
14	Sofiko	27.6	12.0	7.9	3.7	0.3
15	Fournias	64.8	34.8	20.2	11.7	2.8
16	Xerias	19.8	9.6	6.0	2.9	0.6
17	Kirillou	13.7	7.0	4.4	0.6	1.9
18	Seliandros	30.0	17.3	9.8	1.8	5.6
19	Melissi	10.0	4.5	3.1	0.5	0.8
20	Katharoneri	14.6	7.2	4.7	0.8	1.7
21	Agiorgitiko	17.6	9.4	5.8	0.8	2.8
22	Loutraki	212.8	95.9	59.0	17.2	19.6
23	Gerania	69.2	29.8	18.8	5.3	5.6
	Total	2265.4	1390	722.3	290.6	377.7

A large number of aquifers occur in the Plio-Pleistocene deposits as well as in the Quaternary deposits along the streams (Sithas, Asopos, Rachianis, Zapantis, *etc*). The water needs in the coastal zone of the study area are predominantly being covered by the exploitation of these aquifers, through a large number of wells. In recent years many borehole fields have been established in the coastal areas and many shallow irrigation water supply wells have run dry (Voudouris, 2006). Overexploitation causes a deficient groundwater balance in many coastal aquifers, triggering saline water intrusion, which has negative consequences in the socioeconomic development of the area. In recent years the seawater intrusion is more evident, due to high rates of urbanization and the increased agricultural activities (Koumantakis *et al.,* 1999).

The majority of municipal wells in coastal areas show nitrate levels in excess of the WHO drinking standard of 50 mg/L. The main sources of nitrate are fertilizers and domestic sewage effluents (Voudouris *et al.,* 2004).

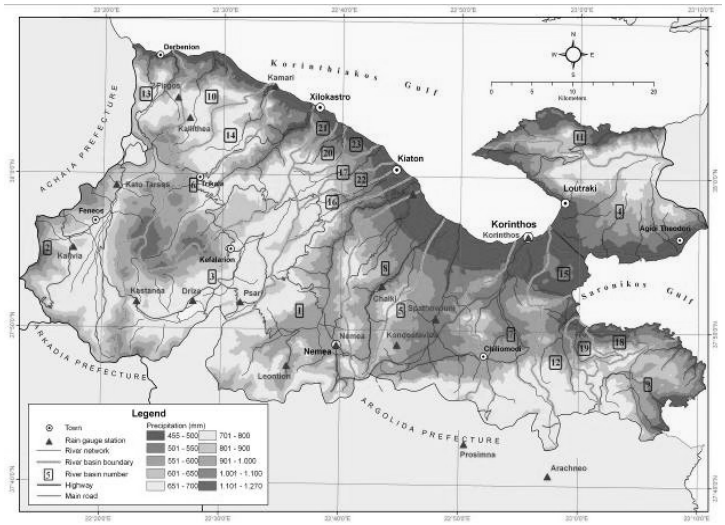


Figure 2. Distribution of annual rainfall.

4. SWOT ANALYSIS

As was mentioned above, the causes and origins of pressures on water resources of Korinthia prefecture, the so called driving forces, are human activities (urbanization, change in land use, intensive agriculture, touristic development, climate changes). Furthermore, the major socio-economic drivers that affect the prefecture are the Common Agricultural Policy of European Union, the Greek development strategy for the study area, the Water Framework Directive (2000/60/EC) and Nitrate Directive (91/676/EC). Agriculture is the first important economical sector in the prefecture and is a significant source of widespread contamination of groundwater with nitrate.

SWOT analysis is a useful tool for the planning development and decision making

and has widely been applied to environmental planning and water resources management (Danca, 2000; ELARD, 2004). Concerning the application of SWOT analysis, it is necessary to minimize or avoid both weaknesses and threats. Weaknesses should be converted them into strengths. Likewise, threats should be converted into opportunities. In addition, strengths and opportunities should be matched to optimize the water resources management. The steps of the application of SWOT analysis has been described in many studies (E.C., 1999; Diamantopoulou and Voudouris, 2007). According to the SWOT technique it is necessary to analyze the strengths and the weaknesses that influence the aquifer systems of the prefecture. Furthermore, the opportunities and the threats must be analysed in order to facilitate the sustainable development of the water resources.

Table 2 shows the framework of SWOT analysis in water resources, including the elements of the internal environment (strengths and weaknesses) and external environment (opportunities and threats), that are detected in the prefecture of Korinthia. Thus, the SWOT analysis is useful in order to find the best match between environmental trends and internal capabilities.

Furthermore, Analytic Hierarchy Process (AHP) was used in order to estimate the relative weights of parameters. The AHP has been used in a large number of applications and is a mathematical decision making technique that allows consideration of both qualitative and quantitative aspects of decisions (Saaty, 1990).

Table 2. SWOT analysis for groundwater resources management in Korinthia prefecture.

	Strengths	**Weaknesses**
Internal environment	- Availability of surface (torrents) and springs water during the wet period - Availability of treated wastewater for irrigation use - Availability of brackish water from submarine springs for desalination - Feasibility of exploitation of karstic aquifers in mountainous area - Existence of recent hydrogeological studies	- Increased water requirements for agricultural and tourism during the dry period (April-late October) when the water availability is low - Lack of adequate monitoring data (groundwater levels, quality data, discharge of springs and torrents, land uses changes) - Lack of municipal sewage-treatment systems in all cities and surface water protection measures - Insufficient knowledge of water-saving practices in agriculture - Inadequate water pricing and lack of economic incentives for efficient water use
	Opportunities	**Threats**
External environment	- EU Water Framework Directive (2000/60/EC) to ensure good chemical status for groundwater and Nitrate Directive (91/676/EC) which requires member states to take measures to reduce agricultural nitrate - Financial assistance from EU that can be used in water resources management	- General problems in national administration (centralization, lack of special regulations and laws, non coordination among institutes-universities-ministries and others end users, lack of dissemination of information etc) - Climatic changes (prolonged dry periods, sea level rise and enlargement of seawater intrusion) - Deforestation

According to the recording of the present situation, the main pressures on water resources, the analysis of the strengths and the weaknesses of the system, the analysis of the opportunities and the threats that come from the external environment, and finally the regional development plan, the following results can be drawn:

The strengths are the availability of the surface water during the wet period and treated wastewater that can be exploited for irrigation purposes.

The weaknesses are the increased water demands during summertime for agricultural and tourism when the water availability is low, the lack of systematic monitoring data (groundwater levels, quality data, discharge of springs, torrents etc), the lack of surface water protection measures, the insufficient knowledge of water-saving practices in agriculture, the inadequate water pricing and the lack of economic incentives for efficient water use.

The opportunities conclude situations e.g. EU financial assistance, EU Framework Directive (2000/60/EC), Nitrate Directive (91/676/EC) and experiences on integrated management from other areas. The threats are the situations that might cause problems e.g. climatic changes (sea level rise, enlargement of seawater intrusion), tourist competition pressures, deforestation and finally the lack of effective administration on national basis (non coordination among institutes-universities-ministries and others end users, centralization, lack of special regulations and laws, lack of dissemination of information).

Based on the AHP, high relative weight shows the availability of surface water (strengths), the increased water requirements for agricultural and tourism during the dry period (weaknesses), the EU financial assistance (opportunities) and the lack of effective administration (threats).

5. PROPOSED SOLUTIONS

The following suggestions are proposed in order to restore the apparent negative water balance in coastal aquifers and achieve sustainable water resources management on a regional scale (Voudouris, 2006):

- Construction of small interception dams in the main streams of the hilly region, aiming at retardation of wintertime torrential flows and the increasing groundwater recharge from streambed infiltration.
- The use of more runoff water would result in the elimination of the water deficit, especially in coastal areas.
- Utilization of the treated urban effluent from the plants (Korinthos, Kiato, Xilokastro) for irrigation purposes. In addition, nowadays the desalination of brackish spring water represents another potential source of water in order to meet water demands during the summertime. The technology has considerably improved and costs are now perfectly affordable for the tourist industry.
- Introduction of water saving irrigation methods and limitation of flood irrigation technique aiming at both reducing water needs and groundwater quality

deterioration.

- Application of a water pricing policy that would encourage water savings.
- Improvement in the water distribution system in order to reduce water losses.
- Exploitation of the karstic aquifer of the mountainous Killini region, which is high quality groundwater reserves and practically not exploited (Koumantakis, 2002).
- Application of groundwater recharge via boreholes from the rivers wintertime discharge in coastal aquifers, could augment groundwater flow budget and contribute to water quality improvement against seawater intrusion (Koumantakis *et al.,* 1999).
- Planning of surface water protection measures. The main pollution source of surface water results from uncontrolled direct disposal of olive mill waste effluent into the torrents and rivers.

6. CONCLUSIONS

The socioeconomic stability of the studied area is based on water resources availability, which ensures optimal agricultural production and tourism development. The prefecture of Korinthia is characterised by non homogeneous distribution of rainfalls and water resources. In the SW part, the rainfall is high and decreases towards eastern and northern parts. The coastal aquifer systems are characterized by deficient groundwater balance; the abstraction rate is more than the recharge rate. The long term mean annual rainfall is 1.39x109 m³. Based on the Thornthwaite method and GIS technology, infiltration and streamflow were estimated to be $0.29x10^9$ m³/yr and $0.38x10^9$ m³/yr, respectively. The analysis reveals that the highest surplus of water is recorded in the western part of the area and the lowest in the eastern part.

The mismanagement of water resources has caused negative effects including depletion of the aquifer storage, groundwater level decline, seawater intrusion in coastal areas and quality deterioration. The coastal aquifers are characterized by deficient groundwater balance. In order to optimize the water management, SWOT analysis was applied, which is a useful tool for local authorities and decision makers. Based on the results of the above analysis some recommendations are made to maximize existing opportunities for sustainable groundwater management in Korinthia prefecture. To correct the negative water balance in coastal areas, an integrated management scheme should be applied and based on conjunctive of surface, spring and ground-water, capture of diversion of more runoff water, construction of small dams, desalination of brackish spring water and the increased use of treated wastewater.

Furthermore, water resources management would benefit by improvement in hydrometeorological, groundwater table, land use, water needs for different uses and quality data monitoring.

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On the Prediction of Incipient Motion of Sediment Particles, with ANFIS

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Abstract: The use of a sophisticated soft-computing method (ANFIS) is proposed in this paper in order to model the dynamics of incipient motion of a sediment particle. A novel experimental setup is employed, which provides Laser Doppler Velocimetry (LDV) measurements synchronously with the exact time of occurrence of the displacement events of a grain out of its resting position. Both inputs in the time and frequency domain are utilized to train the models in order to investigate if any of the two representations has a potential benefit. The models are compared in terms of their efficiency and robustness.

Keywords: Fuzzy inference, prediction, turbulence, wavelets.

1. INTRODUCTION

Sedimentation or erosion of estuarine or fluvial ecosystems is a phenomenon of paramount importance for their evolution. In the long term the not only does it affect their geomorphology, but also its suitability to support aquatic life. The problem of incipient motion for sediment grains, uniquely defines when erosion is initiated. Thus determining the conditions under which incipient motion occurs is of utmost importance for river engineers or biologists who study stream habitats.

A lot of researchers have pursued the issue of incipient motion and tried to formulate methods, hoping to attain a better understanding of the underlying dynamics that characterize it. The research spans several decades devoted on the aforementioned problem, mostly with physically based approaches. For instance Shields introduced a deterministic averaging method resulting in the well known and widelly used from river engineers Shields shear stress (Shields, 1936). Papanicolaou *et al.* (2001) as well as Shvidchenko and Pender (2001) employed visualization techniques to investigate the effect of near bed turbulence in rough beds, and later a stochastic methodology was utilized (Papanicolaou *et al.,* 2002). Smcheeckle and

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Nelson (2003) performed numerical simulation of the bedload transport taking into account flow dynamics and local geomerty conditions.

The main advantage of the above methods is that they attempt to offer a better understanding, depicting directly the interaction of the key variables with the desired result. However, most of the times they lack in universality, since the formulas or simulations are derived based on experiments carried out for a limited range of changing parameters, and most of the times introduce great simplifications.
However this is not the case for soft-computing data driven methods which under certain conditions may equip us with useful generic models, which replicate the dynamics of the studied phenomenon.

Here we attempt to acquire a better understanding of the physical mechanisms that trigger the grain dislodgement in a turbulent near bed flow. Our basic assumption is that the forces acting on the particle are mainly due to gravity (submerged weight as a resisting force) and the pressure field which surrounds the particle. The only way to directly simulate the acting instantaneous forces on the particle, responsible for its dislodgement, is by measuring its instantaneous velocity.

Thus we utilize Laser Doppler Velocimetry (LDV) measurements to obtain the time history of the instantaneous local velocity components, right above the particle which we presuppose that suffices for the characterisation of the pressure or lift-drag force combinations applied on the particle, leading to its dislodgement.

2. METHODOLOGY

2.1. ANFIS

Artificial Neuro-Fuzzy Inference Systems (ANFIS), merge the advantages of both Artificial Neural Networks and Fuzzy Inference Systems; have inherent learning capabilities due to the neural network structure incorporated for the tuning of the premise parameters and they also provide If-Then rules which may shed light in the dynamics of the studied phenomenon. Thus ANFIS is in contrast to ANNs a transparent model which allows for the derivation of the parameters interconnection. A plethora of studies has proved the superiority of soft-computing techniques over traditional physically based methods which utilize heuristics about the interconnection of the salient variables, for systems which exhibit highly nonlinear characteristics.

The last few years soft-computing methods have been extensively utilized and are successfully employed for the prediction of sediment load (Nagy *et al.,* 2002; Yitian and Gu, 2003; Ariffin *et al.,* 2004). Valyrakis *el al.* (2006) constructed several ANFIS structures and compared them with ANN models for the prediction of dislodgement events of a particle. However the time series of the output was constructed by means of visual inspection of the recorded movie of dislodgement events of the particle, which introduced great uncertainties regarding the exact instant of occurrence of the dislodgement initiation. Thus we attempt to implement certain ANFIS structures in

order to obtain the one with the optimum performance regarding the physical model simulation. A trial and error approach is followed since the model's performance and the factors which determine it are not known a priori.

2.2. ANFIS Structure

We will construct and compare several different architectures for the ANFIS which correspond to a first order Takagi-Sugeno Fuzzy Inference System (FIS) with 3 to 4 inputs and 1 output (Figure 1 –for more details see Valyrakis *et al.,* 2006). For the input we considered three separate cases: α) lagged in time velocity vectors for different embedding dimensions and β) input vectors composed of wavelet decomposition coefficients such as Approximation 1 and Details 1 and 2 for decomposition level 2 (Figure 3) using Daubechies wavelet of third order as mother wavelet and γ) input vectors composed of continuous wavelet coefficients at various scales using Symlet of 4rth degree as mother wavelet (Figure 2). Wavelets are used instead of Short Time Fourier Transform since they have the ability to identify events in time and thus enable us to study transient processes.

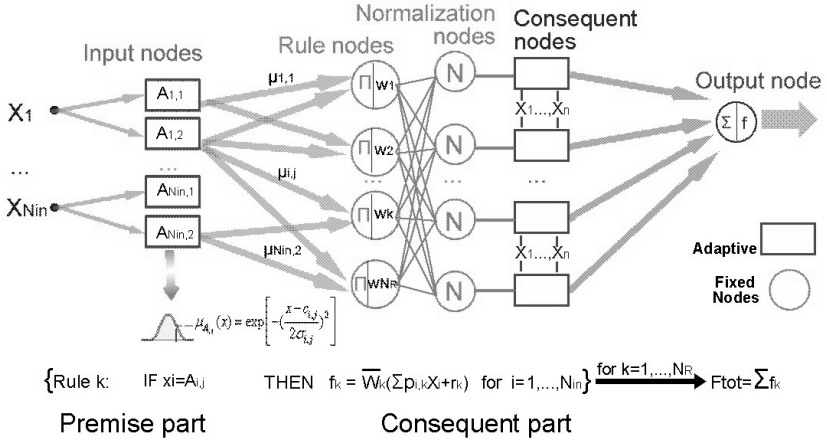


Figure 1. ANFIS architecture for $N_{i,j}$ number of inputs, Nx =2 linguistic labels ($A_{i,j}$) for each input and NR number of rules.

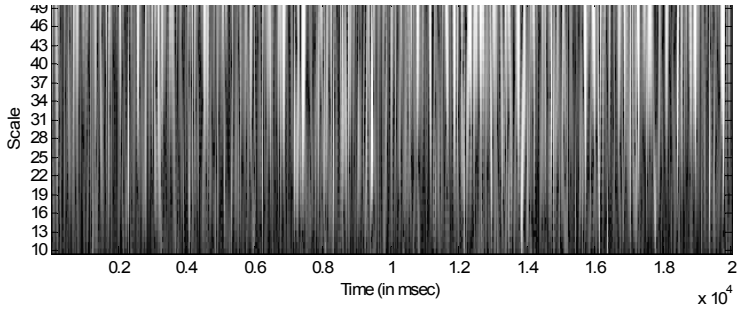


Figure 2. Continuous Wavelet Transform of time series U.

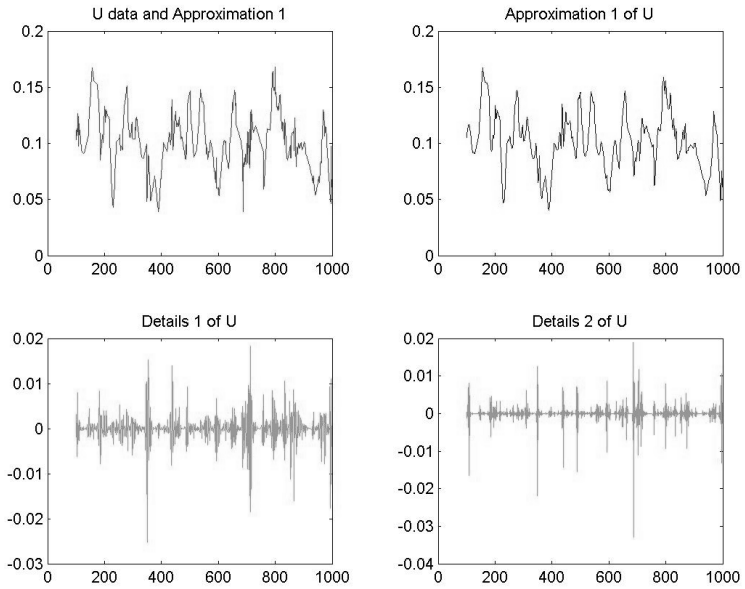


Figure 3. Waveforms of a 2 level decomposition of a portion of the U data (abscissas denote U in m/sec and ordinates illustrate time in msec).

3. EXPERIMENTAL SETUP

A series of experiments were performed at a research flume, located at the Baker Environmental Hydraulics Laboratory, at Virginia Tech. The flume dimensions are 20.5 m, 0.6 m and 0.3 m for its length, width and depth respectively. The bed of the flume is composed four layers of 8mm glass beads.

The instantaneous time series of the local velocity longtidutinal and tranverse components are obtained by means of two component Laser Doppler Velocimetry. The tiny measurement volume, formed by the intersection of blue and green laser beams, is located about 2mm above the mobile particle (Figure 4). The particle simulates an ideal sediment grain, and it is an 8mm Viton® spherical bead. The motion of the particle is constrained by a fixed pin, which consequently allows a very small dislodgement of the particle out of its pocket. After the grain has dislodged, it will fall back into its pocket because of the retaining pin, enabling uninterupted velocity measurements.

At the same time the dislodgement events of the sediment spherical particle are recorded utilizing a He-Ne laser (red laser beam –see Figure 4) along with a Photomultiplier Tube (PMT). The above is a novel setup, which enables the detection of tiny movements by recording the intensity of the He-Ne laser partially blocked by the protruding particle, which is inversely proportional to the particle's displacement; as the particle reaches the maximum dislodgement position, the beam targets the particle and the PMT measures the minimum intensity (while the maximum intensity is provided for the settled particle). Detrending of the obtained signal is required since

there exist a small loss in the He-Ne intensity over time. The detrended signal is transformed to the dislodgement events history, after normalization and filtering. This setup compared to previous configurations (e.g. Valyrakis *et al.,* 2006), has certain advantages since it provides both the streamwise and vertical components of local velocity and also enables locating in time the dislodgement events more accurately than when a video setup is utilized.

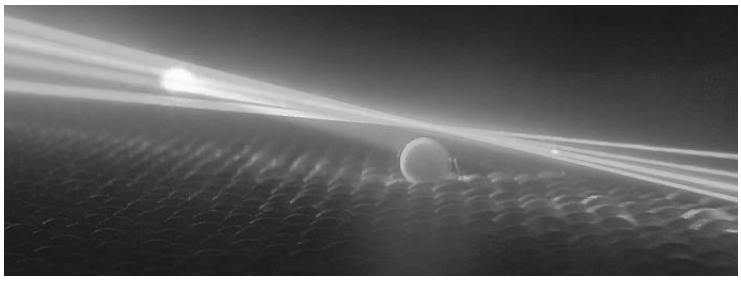


Figure 4. LDV and He-Ne laser beams crossing above and partially targeting the mobile sediment particle.

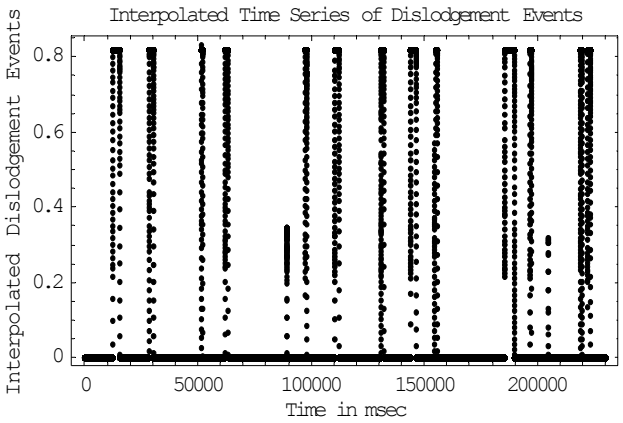


Figure 5. Time history of dislodgement events (obtained from He-Ne setup).

3.1. Data Preparation

Due to the fact that the LDV measurements occur randomly in time, the resulting time series has data points unevenly spaced in the time domain. The same occurs for the PMT data, as they are obtained synchronously with the LDV measurements. Thus the aqcuired time series have to be interpolated in time (Figure 5), which inavoidetly will result in loss of certain amount of information. The data were simultaneously obtained at a frequency of about 400 Hz, meaning that a good approximation of the original signal may be recovered employing linear interpolation in time with step of 1msec. The resulting time series have about 230000 data points (considering spacing in time of 1 msec), for a total of 230 seconds.

For the training and validation of the constracted ANFIS architectures, we split up the data sets into two equally sized subsets, after preprocessing them. The training and validation subsets are totally independent, so that the validation procedure is not biased. The training subset as its name implies, is used to train the ANFIS model, which is performed by tunning of its premise and consequent parameters (Figure 1).

4. DISCUSION

4.1. Evaluation Criteria

The performance of the different ANFIS architectures is evaluated using certain criteria such as the Normalized Mean Square Error (NMSE), Root mean square error (RMSE), Correlation coefficient (CC) and Mean average error (MAE) (see Valyrakis 2006). For instance, the Correlation Coefficient is defined as follows:

$$CC = \frac{Cov(o_i, t_i)}{\rho_{o_i} \rho_{t_{iN}}}$$
$$Cov(o_i, t_i) = \sum_{i=1} \left[\left(o_i - o_{mean} \right) \left(t_i - t_{mean} \right) \right] / N \quad (1)$$

where:

ρ_{oi}, ρ_{ti} are the standard deviations of the model's output and the target respectively, o_i,t_i are the computed and the observed (target) output the for each (i) of the N data points, while t_{mean} is the mean of the observed values.

The above error indicess are computed for both training and validation subsets in order to access the generalization ability as well as the accuracy of the model. Table 1, shows the best models trained with different input cofigurations.

Table 1. Evaluation of the best ANFIS models according to computational time and error indices.

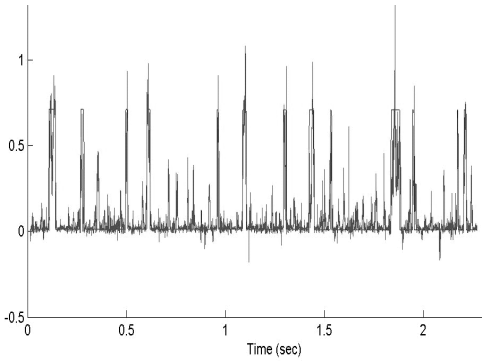
ANFIS	CPU	RMSE		NMSE		CC		MAE	
models	time (s)	Training	Test	Training	Test	Training	Test	Test	Training
WANFIS	2241.8	0.183	0.265	0.883	1.680	0.341	0.084	0.0335	0.0704
U3In	916.47	0.111	0.128	0.323	0.385	0.823	0.799	0.0123	0.0162
W3Com	685.44	0.188	0.208	0.772	0.858	0.477	0.378	0.0352	0.0432
W4Com	2391.3	0.134	0.203	0.394	0.820	0.778	0.576	0.0181	0.0415

WANFIS is the hybrid ANFIS, which is trained and validated using the details and approximations of the decomposition of the streamwise time series (U). Due to computational restrictions in memory and resourses (given the huge size of our data sets), we were able to utilize only 3 inputs, with 3 Membership Functions (MFs) per input, which are the Approximation 1 and Details 1 and 2 for the level 2 decomposition of the streamwise time series. The model U3In corresponds to an ANFIS structure of 3 lagged inputs of the streamwise component, with 2 MFs per input.

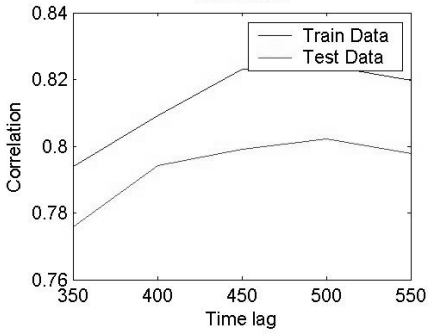
The model W3Com and W4Com, represents models for which the training and

validtion scheme is based on the use of wavelet coefficients of certain scales. The models that performed best were those for which we used as scales the values of 25, 500, 4500 and symlet wavelets of fourth order. W3Com has 3 inputs with 3, 2 and 2 MF's for each, while W4Com has 4 inputs with 3, 3, 2 and 2 MF's per input.

We studied many different architectures for the ANFIS model, with different size of shifted input vectors or number of membership functions per input variable, in order to assess the sensitivity of our model to that parameter (Table 1). We observe that some of the models either loose their generaliztion ability (e.g. WANFIS) or attain poor accuracy (e.g. W3Com). Also it is obvious that there always exists a trade off between accuracy and computational effectiveness, which may affect our choice for the optimum model.



Observed (blue) vs Predicted (green) normalized dislodgements, for the whole U time series



Correlation curves for training and test subsets

Figure 6. For the model with the best performance (U3In).

The predicted versus the observed values are given for the model with the best performance (Figure 6a). To evaluate generalization ability of this model, we plot its correlation curves (Figure 6b). As we observe from the Table 1, the model was able to generalise fairly well since the training error is about the same with the test error. The optimum time lag is the one which gives the maximum correlation for the validation set (about 500 msec -Figure 6b).

4.2. Results

We attempted to predict the incipient motion of a grain interacting with near bed turbulence in threshold flow conditions, employing ANFIS. We observed that the results improved in comparison to previous reported results (Valyrakis *et al.,* 2006) possibly due to the fact that the dislodgement events were accurately measured employing the He-Ne and PMT setup. The above also confirms that ANFIS as a data driven method is highly dependent on the quality of the provided data. The best developed system had the ability to generalize well despite the complexity of the physical system it simulates. Even though there is no need for heuristics or prior

knowledge for the underlying physics of the system, the structure which optimally represents it has to be obtained by a trial and error approach, tuning up a variety of different parameters. Moreover the model performance is restricted due to limited computational resources. The wavelet approach was not particularly fruitful possibly due to the use of a small number of MF's. However a choice of different mother wavelets might return better results. Finally the model which employed the second component of velocity did not improve their performance in respect to those which used only the longitudinal component, which means that possibly the second component does not furnish more information on the dynamics of the phenomenon of incipient motion.

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Chemical Status in a Cretan River Basin with Intensive Agriculture and Intermitent Flow

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Abstract: In the framework of the implementation of the WFD, the Crete Region assigned a project to the Institute of Inland Waters (IIW)/ Hellenic Centre for Marine Research to assess the ecological status of Anapodaris River. The aim of this article is to present the chemical quality of the river in relation to the pressures exerted in the basin. In light of the temporal character of the river and specific anthropogenic pressures, sediment chemical quality elements were considered, additionally to hydrochemical ones. Elevated concentrations of nitrates, nitrites and conductivity in river water as well as salts, heavy metals and nutrients in sediments were attributed to agricultural land uses, geochemical background and point sources of organic pollution. According to these results, a number of chemical and physicochemical quality elements, related to anthropogenic activities in the basin, were selected for the assessment of the chemical status of the river. Despite the extensive agricultural activities in the basin, the river still presents an overall satisfactory chemical status, attributed to in-stream self-purification processes.

Keywords: Anapodaris river, chemical quality, heavy metals, nutrients, WFD.

1. INTRODUCTION

Through the Water Framework Directive (WFD 2000/60 EC) the EU seeks to impose guidelines to preserve good ecological status and recover degraded water bodies. For assigning water bodies to high and good ecological status, complementary to the biological quality elements, the values of chemical-physicochemical (from now on termed as "chemical") quality elements should be taken into account. Water bodies with moderate ecological status should be considered within integrated river basin management plans in order to improve their quality at least to good status. To classify ecological status, the lower of the values of the biological and chemical monitoring results should be used. Hence, the role of chemical quality elements is crucial.

In Crete, land degradation related to agriculture occurred since the Minoan times.

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Nowadays, the island contributes 22% to the total olive orchard areas in Greece (Crete Region, 2000), while olive oil production almost tripled in the last 30 years (NAGREF, accessed 2007).

Anapodaris is a temporary river and such rivers play a pivotal role in both public and agricultural water supply and ecosystem preservation. They are extremely sensitive to hydrological pressures (e.g. surface water abstraction, groundwater overexploitation), impact of point and non-point pollution and changes in catchment land use. Hence, temporal rivers and streams need to be specially considered in WFD-compatible river basin management plans (tempQsim Consortium, 2006).

Crete Region, being a pioneer concerning the implementation of the WFD in Greece, assigned a study to the IIW to assess the ecological status of the temporay flowing river Anapodaris (Skoulikidis and Gritzalis, 2007). The aim of this article is to trace the origin of pollutants, found in river water and sediments, and assess the impact of intensive agricultural practices and point pollution sources on the chemical status of Anapodaris River.

2. MATERIALS AND METHODS

2.1. Description of the Study Area

Mesara, the largest plain of southern Crete is characterized by semi-arid climatic conditions and is drained by the two major rivers, Geropotamos R. and Anapodaris R. The later is situated at the eastern part of Mesara plain. The Anapodaris basin covers an area of 525 km^2. The main part of the basin (68%) is covered by recent (neogene-quaternary) deposits, where phreatic aquifers are developed, while limestones and flysch, which cover the mountainous margins of the basin are equally represented (~16%). Mafic magmatic and metamorphic rock outcrops (ophiolites and schists) are dispersed in restricted areas of the mountainous part of the basin. The basin has a well-developed hydrological network. The total runoff of the river is 29 Mi m^3 (data: Crete District, 1967/68-1996/97). The main tributaries of Anapodaris are Erganas and Inniotis at the eastern and Plakiotissa at the western part of the basin. The main part of the river system dries out in summer. Due to the smooth relief of the largest part of the basin, which is covered by recent deposits, the area is intensively cultivated, mainly with olive groves. Figure 1 shows the hydrological network, the main towns, the monitoring sites and their sub-basins.

2.2. Sampling and Analysis

Field measurements and sampling took place at the end of February beginning of March 2006. The period of sampling is considered to be satisfactory, since precipitation events and snow melting took place before the monitoring period and the majority of the river network was active. The river and its tributaries were supplied by

both surface and, primarily karstic, groundwater runoff from the mountainous limestone margins of the basin and the river system was hydrologically stabilized. Moreover, olive oil refineries were operating. A monitoring network (Figure 1), which was representative of the natural characteristics and human pressures in the basin was designed according to following prescriptions: existence of flow, homogenous distribution of stations in the active hydrological network and distribution of point pressures (towns and villages, oil presses and olive oil manufacturing waste disposal sites, hydrological alterations) and cultivated areas. In total, twenty monitoring stations were selected.

At each sampling site, coordinates were determined by a GPS device and physico-chemical parameters (current velocity, water temperature, conductivity, dissolved oxygen and pH) were measured in situ with the use of portable instruments. Water samples were placed in plastic vials and a few drops of saturated solution of $HgCl_2$ were added. A sub-sample was kept separately for the determination of chloride ions. Water samples were transferred in portable refrigerators in the laboratory where, after filtration through 0.45 μm membrane filters, hydrochemical variables (total hardness, Ca^{2+}, Mg^{2+}, Na^{+}, K^{+}, HCO_3^{-}, CO_3^{2-}, Cl^{-}, SO_4^{2-}, SiO_2) and conventional pollution parameters (nitrate, ammonia, nitrite, total nitrogen, orthophosphate and total phosphorous) were analysed. Calcium, magnesium, sodium, sulphate, chloride and potassium were determined with the use of Capillary Ion Analyser (US Patent 5,104,506/ European patent 0442315 A1). Hydrogen carbonate, carbonate and total hardness were determined by automatic titration (TIM 900 – Radiometer). Nutrients were analysed photochemically by a Merck Nova 400 analyser.

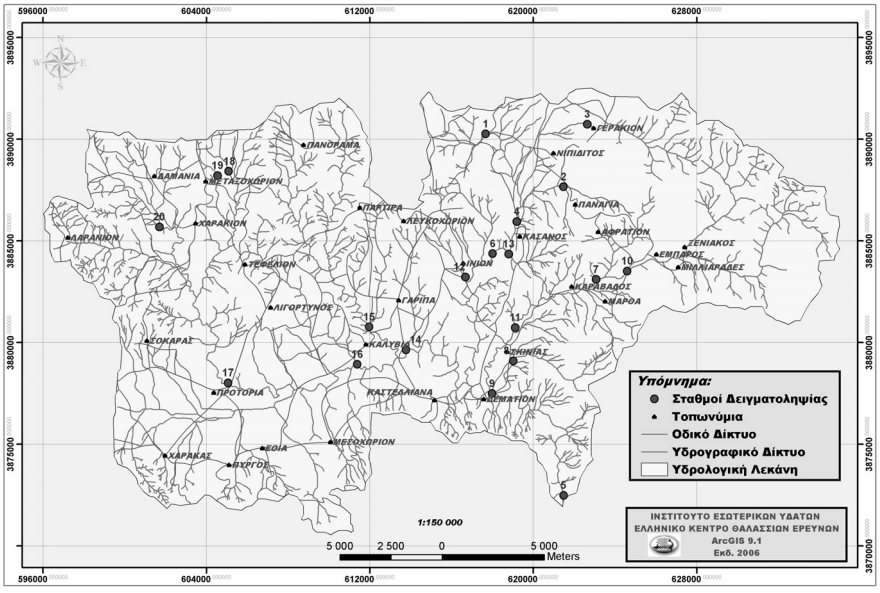


Figure 1. River network, sampling network (cycles) and subbasins in Anapodaris catchment.

Sediment samples were placed in plastic bags and transferred under freeze in the laboratory. Concentrations of heavy metals (V, Cr, Mn, Co, Ni, Cu, Zn, As, Rb, Sr, Mo, Ba, and Pb) and total sulphur were determined with the use of a PHILLIPS X-ray fluorescence (XRF) device. Total and organic carbon and total nitrogen were determined by a Fisons Instruments CHN EA-1108-type element analyzer, while TP and inorganic P analysis were achieved by using the Aspila *et al.* (1976) method.

Information about the site and its catchment, e.g. stream order, altitude, slope, catchment area, stream density, geology, land use was gathered by GIS applications. Additional data, such as stream morphology, hydrological aspects, floodplain characteristics, river cross sections, riparian vegetation, human pressures and pollution sources, were collected on the sampling site, at a reach scale, or were provided from Crete District (Directorate of Planning and Development, Department of Water Resources Management). The resulting data matrix consists of 1,280 observations (20 stations x 64 variables).

2.3. Classification of the Chemical Status

Regarding dissolved oxygen, the strictest criteria of existing classification systems (Cadorso *et al.*, 2001) were employed. For the classification of nutrient levels a recently developed Nutrient Classification System (NCS) (Skoulikidis *et al.*, 2006) was applied. For the classification of river water regarding its salt content, irrigarion water criteria (conductivity, SAR) were used. For the classification of heavy metals in river sediments, references of undisturbed sites and standards of undisturbed and polluted soils of agricultural and urban areas were applied. For the classification of the chemical status, the different chemical quality elements were ranked in five categories, ranging from high to bad status.

3. RESULTS AND DISCUSSION

3.1. Analysis of Pressures

The Mesara plain, a region of ‘high-intensity’ cultivations, is one of the most intensively cultivated areas in Greece (Caraveli, 2000). Cultivated areas in Anapodaris catchment cover 70% of the basin, leaving 30% to naturally vegetated areas, while the half of the basin is occupied by olive orchards (CORINE land cover 2000). Wastes from 59 registered oil presses are usually disposed in sinks, which occasionally overflow, or are even directly disposed within the hydrological network. Other inappropriate agricultural practices include overfertilization, application of river water to wash tanks used for spraying with agrochemicals and dumping of garbage, including agrochemical packages. 0.6% of the basin is covered by intermittent urban fabric. The basin hosts 34 towns and villages with a total population of approximate 18,000 inhabitants. In absence of sewer systems and WWTPs, household wastewaters

are usually disposed in septic tanks. Numerous drillings, five small reservoirs and the surface water network serve irrigation purposes. There are about 230 registered drillings in the neogene and quaternary aquifers of the basin. The reservoirs are located off-stream. Three of them are situated at Erganas tributary, one at Inniotis and two at Plakiotissa tributary. As a result of the combined action of high evapotranspiration losses, infiltration in the phreatic aquifers and water abstractions, only about 10% of precipitation (57 mm) is running towards the river network, which dries out during July – September. Only Erganas tributary retains some karstic flow in summer (data: Crete District, Directorate of Planning and Development, Department of Water Resources Management).

3.2. State and Impacts

3.2.1. River water

In general, pH and the concentration of dissolved oxygen varied within normal levels. However, sites 1 and 8 illustrated low oxygen concentration (3.9 and 3.1 mg/l respectively) due to low current velocity and organic matter accumulation originating from oil mill wastes and municipal wastewaters respectively.

Conductivity was exceptionally high. In fact, the mean conductivity (1241.5 μS/cm) was three times higher compared to the mean conductivity of small/medium (catchment area < 1000 km²) Greek rivers found in the hydrogeochemical-climatic zone 3 (Skoulikidis *et al.*, 2006), where Crete island belongs. The high mineralization of Anapodaris river system is attributed to the dry climatic conditions, high evapotranspiration and agricultural practices in the basin. In fact, the positive correlation of conductivity with olive groves indicates the impact of agriculture on the salt balance of the river (Figure 2). At stations 6, 12 and 13 conductivity surpassed the limit set by Hart *et al.* (1990) for macroinvertebrate communities (1492 μS/cm), while at stations 4, 8 and 11 conductivity was close to the limit. We thus decided to include salinization in the evaluation system of the chemical status of the river. Site 12, which belongs to a rare sodium-chloride type, revealed maximum conductivity (2110 μS/cm). This is attributed to both the high percentage of cultivations in its subbasin (95.6%) and to dissolution of halite and gypsum deposits, that were excavated during works carried out for the construction of the upstream Ini-reservoir (M. Kritsotakis, pers. com.).

The average nitrate and nitrite concentrations (3.65 and 0.040 mg/l N) were very high, as they exceeded the respective concentrations of small/medium Greek rivers (Skoulikidis *et al.*, 2006) and the European median (Stanner and Burdeau, 1995). While high nitrate concentrations can be directly attributed to fertilization, the high nitrite levels may be caused by denitrification processes occurring in shallow aquifers (Trudell *et al.*, 1986) interacting with surface waters. In contrast to nitrate, the average concentrations of ammonia (<0.01 mg/l N), phosphate and total phosphorous (<0.02 mg/l P) were very satisfactory. The low ammonia levels could be explained by a shift

of the nitrification process towards nitrite and nitrate during the time of sampling, while the low P levels may be attributed to adsorption on sediment particles.

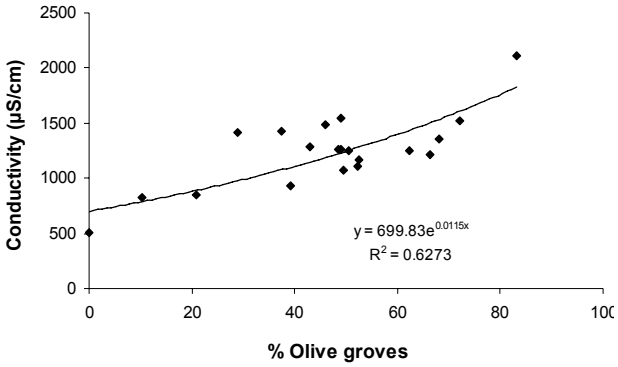


Figure 2. Correlation between conductivity in the different sites and olive grove percentage in the respective subbasins.

3.2.2. River sediments

The average concentrations of C, N, S and P compounds in river sediments were lying between those found in a minimally disturbed river (Krathis, N. Peloponnisos/Greece) and a heavily disturbed one (Soulou, N. Greece). Maximum organic matter concentration showed an apparently minimally disturbed site (3), which belongs to a very small and thus sensitive karstic basin (0.3 km²). Sediment nutrients were correlated with the percentage of agricultural areas, olive orchards and urban areas in the respective subbasins.

Heavy metal levels exhibited a negative correlation with the percentage of natural areas in the subbasins. High Ni and Cr concentrations found at the NW part of the basin were attributed to mafic mineral weathering. These metals where primarily correlated with the percentage of neogene-quaternary deposits, that contain mafic rock debris, and secondarily with cultivated areas. Cu, Zn, Mn, Pb, Co and As were primarily correlated with agricultural areas, while Pb additionally with urban fabric. Fig. 3 shows the relationship between heavy metals and agricultural areas.

3.3. Classification of the Chemical Status of Anapodaris River System

According to the requisite criteria used for reference site selection (STAR project, ENVK1-CT-2001-00089, www.eu-star.at), reference sites could not be found within Anapodaris basin. Consequently, it was not possible to develop a type-specific classification scheme for Anapodaris basin (e.g. for nutrients or heavy metals). Hence, for the classification of the chemical status, existing classification systems and standards were applied. Regarding dissolved oxygen, the majority of the sites revealed high quality and only two were termed as sites of poor quality. According to NCS, the majority of the sites were termed as bad for nitrate, between moderate and

poor for nitrite and high for phosphate, while concerning ammonia all sites were classified as high. As far as the salt content of river water is concerned, the high SAR quality balanced the low conductivity status of the waters resulting to a good quality for all sites, besides one site which illustrated a moderate quality. Concerning "anthropogenic" heavy metals (As, Zn, Pb, Cu) the majority of the sites showed high quality. Regarding heavy metals that derive from both mafic rock weathering and pollution (Cr, Ni, Co) the majority of the sites ranged between good and moderate quality.

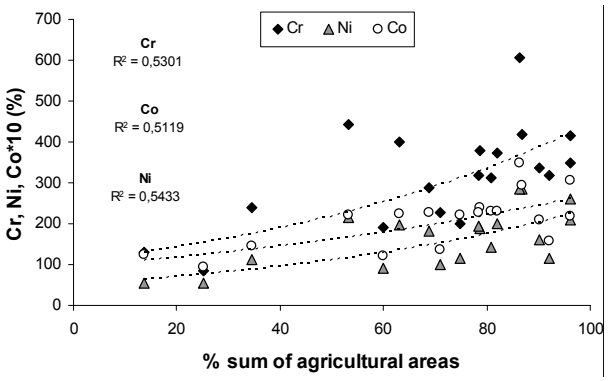


Figure 3. Correlation between Cr, Ni, Coin river sediments and the sum of agricultural areas in the respective subbasins.

4. CONCLUSIONS

When monitoring river pollution, water sampling during low water flow is essential. However, most temporary rivers rapidly dry out in summer as a result of intensive water abstractions and thus are difficult to be monitored. Moreover, since contaminants accumulate in sediments, the later are more appropriate for the detection of pollution, and especially past pollution incidents, compared to the water column. In Anapodaris River, sediment nutrients and heavy metals were related to anthropogenic pressures, suggesting that they comprise good indicators for the assessment of its chemical status.

Despite intensive agricultural practices (overuse of agrochemicals, water abstractions) and point sources of organic pollution (oil mill wastes, municipal wastewaters), the Anapodaris river system, as a whole, still shows a satisfactory chemical status. Intensification of agriculture degraded the high chemical status of the river system and created a, more or less, homogenous background of good status. In parts of the basin, local sources of organic pollution further deteriorated the status of a number of chemical quality elements from good to moderate. The overall satisfactory chemical status is attributed to self-purification processes occurring in the basin, e.g. dilution (high runoff during the time of sampling), nitrification resulting to ammonia reduction, or accumulation of phosphorus in river sediments. The construction of

waste water treatment plants and effective management of olive oil refinery wastes will improve the state of chemical quality elements and the entire basin will reach good chemical status. To retain the aquatic system at the good status, best agricultural practices should be adopted and applied.

ACKNOWLEDGEMENTS

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A Generalized Reach–Scale Biogeochemical Model for Temporary Rivers

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Abstract: The RS-tempQ Model (Reach–Scale Temporary Flow Biogeochemical Model) is a generalized model that can describe the hydrologic, sediment transport and biogeochemical processes of temporary rivers at the reach scale. The model takes under account the expansion-contraction of the inundated area of the river. The RS-tempQ Model is capable of simulating the processes during the wet and dry period and spatially where the river corridor is expanded and contracted. The RS-tempQ Model was used to assess the hydrologic and geochemical reactions occurring in a temporary river reach (Krathis River).

Keywords: Biogeochemical processes, biogeochemical processes, modeling, sediment transport, temporary river.

1. INTRODUCTION

In the semi-arid climate of the Mediterranean region, temporary rivers are the predominant surface water courses (Hooke *et al.,* 2002). Temporary rivers are natural water bodies that experience a recurrent dry phase of varying duration and spatial extent (Uys *et al.,* 1997). There is high spatial variability during the wet and dry periods especially at their estuaries. During the wet period, there are flooding events that expand the wet area of the river, remobilize nutrients and sediments and during the dry period there is slow accumulation of sediments, high variation of temperature and soil moisture and contraction of the river's wet area (Sheldona *et al.,* 2002). These rivers have not been studied sufficiently in the past. The European Union funded project tempQsim was focused on the evaluation of the in-stream biogeochemical processes of Mediterranean temporary rivers that influence the availability of nutrients and carbon. A conceptual site model for each temporary rivers was developed and an evaluation of the predominant in-stream processes of the

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nitrogen, phosphorous and carbon cycles were identified and studied in the laboratory. The geochemical information obtained through base flow and episodic event basis as well as aquatic and sediment sampling guided us to the formulation of the initial hypotheses on the important processes that affect the fate and transport of nutrients in temporary rivers. This analysis aided in the planning of the experiments and studies that are necessary for a comprehensive understanding of the in-stream N and P cycles. Existing water quality models such as QUAL2E, WASP, BASINS, HSPF, ETD don't take under consideration the expansion and contraction dynamic of the temporary rivers, the wet and dry period of the riverbed and the dependence of biogeochemical processes on soil moisture and temperature. In order to simulate the biogeochemical processes that dominate in temporary rivers, the creation of a new model that works at reach scale was necessary. The objective of this work was the development of a generalized reach-scale geochemical model for the simulation of the hydrology, sediment transport and water quality of temporary rivers.

2. SITE DESCRIPTION: KRATHIS RIVER

The study site (river reach) is located in the northern end of Krathis River (Figure 1) and has a length of 300 m (the reach was selected to have a length 5 times the width of the riparian corridor). No point sources or tributary streams enter the study reach. A gauging, meteorological and water quality stations were installed directly upstream of the reach for continuous measurements of water temperature, dissolved oxygen, pH, conductivity and turbidity (July 2003 to October 2004). Water quality monitoring took place on a monthly basis (April 2003 to December 2004). The water samples were analyzed for total nitrogen (TN), nitrate nitrogen (NO_3-N), nitrite nitrogen (NO_2-N), and ammonia nitrogen (NH_4-N), total phosphorous (TP), dissolved inorganic phosphorous (DIP), dissolved organic Carbon (DOC), particulate organic carbon (POC), particulate total carbon (PTC), particulate total nitrogen (PTN), particulate total phosphorous (PTP) and total suspended solids (TSS) (Analytical methods, Supporting Information). These data were used to determine the input loads to the reach. Three intensive field campaigns (June and November of 2003 and June of 2004) were conducted aimed at gaining better understanding of the hydrologic and water quality regime of the river. Detailed description of field techniques, instrumentation, sampling, analysis and results are described in Tzoraki *et al.* (2006).

3. MODEL DEVELOPMENT

The RS-tempQ Model (Reach–Scale tempQsim Biogeochemical Model) is a generalized model that can describe the hydrologic, sediment transport and biogeochemical processes of temporary rivers at the reach scale. The model takes under account the expansion-contraction of the inundated area of the river (Figure 1).

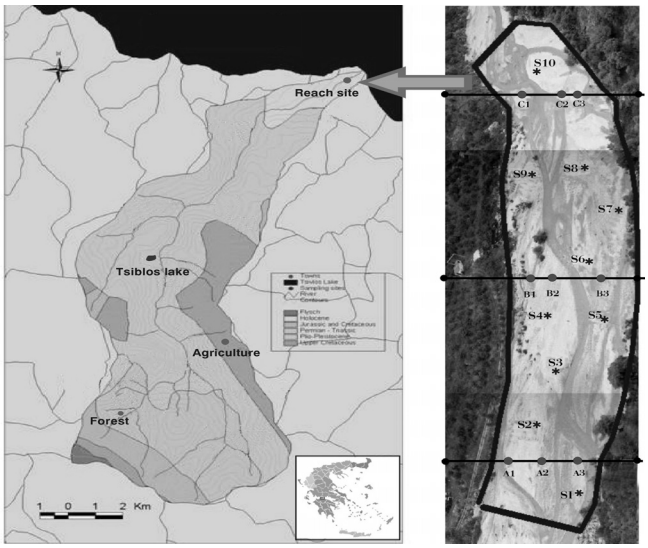


Figure 1. Geologic Map of Krathis river basin and the reach site (left) and Krathis river study reach (Divided in three transects A, B, C, where A1, A2, A3, B1, B2, B3, C1, C2, C3 are the multi-level wells and S1-S10 are the sediment samples) (right) (Tzoraki *et al.,* 2006).

3.1. Hydrologic Submodel

Figure 2 presents a conceptual schematic of the hydrologic fluxes in a reach of a temporary river. The upstream inflow is entering the reach covering only part of the floodplain. Within the reach there are transmission losses. The water has the opportunity to enter the sediment in the down-welling region and then resurface when the water table is high in the up-welling region of the reach. Finally, there are net transmission losses from the reach to groundwater.

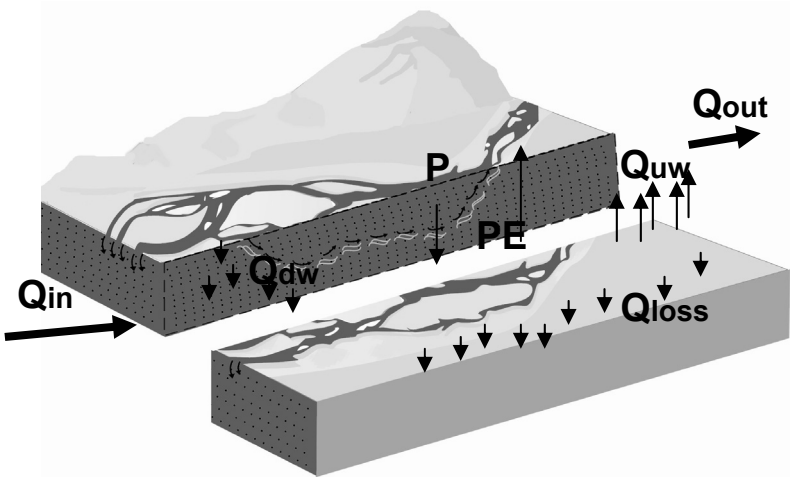


Figure 2. Conceptualization of Reach Scale Hydrology.

Aquatic Phase Hydrologic Balance: The river reach is conceptualized into 2 distinct regions with a total reach area AT: A1 is the wet area – flowing river and A2 is the dry sediment. From the aerial photos, one can obtain relationships between the wet river area as a function of flow. The dry sediment area can be found by difference, A2=AT-A1. Such a relationship is entered in the model and based on the upstream inflow, the percent inundation and the wet area of the floodplain are computed.
The hydrologic mass balance equation is written as follows:

$$\frac{dV_W}{dt} = PA_1 + Qin - PE + Quw - Qdw - Qout \quad (1)$$

where:
V_w is the volume of the water in the reach (m^3), P is the direct precipitation (m /hr), Qin is the upstream discharge (m^3/hr), Quw is the upwelling discharge (m^3/hr), Qdw is the downwelling discharge (m^3/hr), Qout is the outflow discharge (m^3/hr), PE is the Potential Evaporation (m^3/hr), and A_1 is the area of the reach covered by the river (m^2).

The components of the hydrologic mass balance can be estimated from data or be parameterized as follows:

- *Qin* is the upstream discharge that is measured by the gauging station or can be estimated by a hydrologic model.
- *PE* is calculated from meteorological data using typical evaporation equations.

The transmission losses *Qdw* are estimated as a function of infiltration rates in the river bed as follows:

$$Qdw = a \cdot f_c \cdot A_1 \quad (2)$$

where:
f_c is the infiltration capacity (m /hr) and *a* is a calibrated constant.

- Qout is estimated busing hydrologic routing or $Qout = k(V_w/A_1)^n$, where *k* and *n* are calibrated parameters.

Sediment Phase Hydrologic Balance: The hydrologic mass balance equation of the sediment phase is written as follows:

$$\frac{dV_b}{dt} = Qdw + P \cdot A_2 - E \cdot A_2 - Q_{loss} - Quw \quad (3)$$

where:
$P \cdot A_2$ is the direct precipitation on dry areas of the floodplain, *E* is evaporation from dry sediments (A_2 area) calculated from the potential evaporation estimate adjusted for soil moisture

$$E_D = PE\left(\frac{SM}{SM_s}\right) \quad (4)$$

where:
SM is the actual soil moisture and SM_s is the saturated soil moisture.

The model assumes that if the active sediment layer is a certain percentage full, then the excess water will upwell into the stream. Assuming the ε is porosity, V_T is the total sediment volume and n is the fraction that the sediment is full (a calibrated parameter), then

$$\frac{dV_b}{dt} = 0 \Rightarrow \; Qdw + P \cdot A_2 - E \cdot A_2 - Q_{loss} - Quw = 0 \tag{5}$$

$$Quw = Qdw + P - E - Q_{loss} \tag{6}$$

$$\text{when } V_b = n\varepsilon V_T \text{ and } n > 0.7 \tag{7}$$

the excess water will be the upwelling flow. Q_{loss} is net loss to groundwater and is assumed to be a fraction b of Qdw:

$$Q_{loss} = b \cdot Qdw \tag{8}$$

3.2. Sediment Transport Submodel

Sediment transport is driven by bed shear stresses. The shear stress (force per unit area) of the flow on the bed is what causes sand gain resuspension and enters them into the flow. If the bed shear stress exceeds a threshold value for a particular grain size and composition, that grain will be lifted from the bed and carried by the flow (Partheniades, 1965). If the bed shear stress drops below that threshold value, the entrained grain will drop back to the bed (Krone, 1962). So the bed shear stress controls whether erosion (entrainment, removal of grains) or deposition (settling of grains) will occur. In the river, the bottom shear stress is computed using the average tangential stress in the wall: $\tau = \rho g R_h S_f$ (Eq. 9), where τ is shear stress (Pa), ρ is water density (kg/m^3), g is gravity acceleration (ms^{-2}), R_h is the hydraulic radius, which is the quotient between the cross section vertical area and the wet perimeter, and S_f is friction slope.

Aquatic Phase Sediment Balance: The sediment mass balance equation for the aquatic phase is written as follows:

$$\frac{d(S)}{dt} = \frac{Qin}{V_w} \cdot Sin - \frac{Qout}{V_w} \cdot S + F_E \frac{A_b}{V_b} - F_D \frac{A_b}{V_w} \tag{10}$$

where:
S is the concentration of suspended sediments (kg/m³), S_{in} is the input sediment concentration in the water (kg/m³), S_{out} is the output sediment concentration [kg/m³], F_E is the erosion flux (kg/(m²s)), F_D is the deposition flux (kg/(m²s)), A_b is the bottom surface area (m^2), V_w is the volume of water in the aquatic phase (m³), and V_b is the volume of water in the sediment phase (m³).

Sediment Phase Sediment Balance: The sediment balance of the sediment phase is computed by:

$$\frac{d(S_b)}{dt} = -F_E \frac{A_b}{V_b} + F_D \frac{A_b}{V_w} \tag{11}$$

Where:
S_b is the concentration of deposited sediments (kg/m²), F_E is the erosion flux (kg/(m²s)), F_D is the deposition flux (kg/(m²s)), A_b is the bottom surface area (m²), V_w is the volume of water in the aquatic phase (m³), and V_b is the volume of water in the sediment phase (m³).

3.3. Biogeochemical Submodel

Phosphorous, nitrogen and carbon are transported to the river reach via dissolved and particulate phases. The processes that affect the fate and transport of nitrogen in the aquatic phase include nitrification, denitrification, mineralization, and ammonia sorption and volatilization. The processes that affect the fate and transport of phosphorous in the aquatic phase include mineralization, sorption and leaching of phosphorous. Mineralization is the most important process of carbon in temporary rivers, where dissolved organic carbon is transformed into dissolved inorganic carbon. These cycles operate both in the aquatic phase and the sediment phase. Two mass balance equations have to be written to account for the changes in the two compartments for each water quality constituent. In general there are 7 state variables in the water quality submodel; 3 for the nitrogen species (DON, ammonia, and nitrate) 2 for phosphorous (DOP and DIP) and 2 for carbon (DOC and DIC). The particulate forms of nitrogen, phosphorous and carbon are estimated by applying a simple partitioning coefficient using the concentration of suspended sediment.

Aquatic Phase Water Quality Balance: The generalized water quality constituent mass balance equation for the aquatic phase is written as follows:

$$\frac{d(V_w C_w)}{dt} = Q_{in} \cdot C_{in,w} + P \cdot A_1 \cdot C_{atm} - Q_{out} \cdot C_w - Q_{dw} \cdot C_w + Q_{uw} \cdot C_b - Q_{loss} \cdot A_1 \cdot C_w \pm reactions \tag{12}$$

where:
C_w is the concentration in the reach (mg/l), $C_{in,w}$ the initial concentration in the inflow (mg/l), C_{atm} is the concentration in the precipitation (mg/l), C_b is the concentration in the sediment (mg/l).
The reaction term is for each water quality state.

Sediment Phase Water Quality Balance: The generalized water quality constituent mass balance equation for the sediment phase is written as follows:

$$\frac{d(V_b C_b)}{dt} = Q_{dw} C_w + P C_{atm} A_2 - Q_{uw} C_b \cdot A_1 - Q_{loss} C_b \cdot A_1 \pm reactions \tag{13}$$

The reaction term is for each water quality state variable.

Experimental work conducted in previous studies suggest that temperature and soil moisture are the master variables in temporary environments (Tzoraki *et al.,* 2005). It is important that temperature and soil moisture corrections on the rate constants to be incorporated into the R-S tempQ Model. These corrections should be established for both the wet and dry part of the reach. The model estimates soil temperature using statistical relationships as function of air temperature in order to estimate Water Temperature (Tw) and Soil Temperature (Ts) time series. The wet area and semi-wet areas will be assigned saturated conditions while the dry area will have variable soil moisture depending on the number of days since it was wet. Once the soil moisture is established then the soil moisture correction equation that can be used is the following:

$$K_{SM} = K_{100}\left(\frac{SM}{SM_{sat}}\right)^{n} \tag{14}$$

where:
K_{SM} is reaction rate at soil moisture (SM), K_{100} is the reference reaction rate when the soil moisture is 100%, SM_{sat} is the soil moisture under saturated conditions and n is a constant (Bowie *et al.,* 1985).

4. MODEL SIMULATION RESULTS

Hydrologic simulation. The Krathis catchment was simulated for one year period 15/7/2003-14/7/2004 using the HSPF model (Tzoraki and Nikolaidis, 2006) and the output hydrology, sediments and water quality time-series from the last sub-catchment were used as input time series in the RS-tempQ model. In addition, the meteorological data from the nearest station were used. The annual precipitation for the simulation year 2003-4 was measured to be 848.1 mm. The annual average temperature was 16.1ºC with a minimum monthly average in January of 7.8ºC and a maximum monthly average in August of 27ºC. The annual potential evapotranspiration (Thornthwaite) was estimated at 848.8 mm. An estimate of the annual transmission losses was obtained by using aerial photographs of the reach at different time periods and various percentage inundations. The annual average flow was 225000 m^3/d and the annual average transmission losses from the reach were 8900 m^3/d, corresponding to 4 % of the total flow. These losses were over the whole area of the reach that was calculated to be 16314 m^2. The infiltration capacity was set equal to 0.11 m/h and changing in the model the infiltration coefficient, the % transmission losses were estimated. When the Infiltration Coefficient is 0.001, the transmission losses are estimated to be 12.2%. The transmission losses ($Q_{Downwelling}$) correspond to 12.2% of the inflow and are equal to the loss to the groundwater.

Sediment simulation. From the monthly aerial photographs of the study site and the projection in ArcGIS was estimated that the Net Deposition Rate was 1258 m^3/y in the reach area and the mean depth of fresh sediments was 0.077 m (Tzoraki *et al.,* 2006). Considering that the mean dry bulk density of the sediments is equal to 2500

kg/m³ then the mass of the sediment deposited annually is 3271 tn. Changing the value of the critical shear stress, the model estimate the sediment deposition mass. For a value of the critical shear stress deposition parameter of 0.0005, the mass of deposited sediments simulated by the model was 4620 tn which is the same order of magnitude with the field measurements. Concerning the sediment mass balance, 29295 tn of sediments are coming in the reach and 24675 are going out. The difference 4620 tn was the net deposition. This value is very close to the estimate obtained by topographic measurements in the reach.

Water Quality simulation. The fluxes of nutrients that are incoming, outgoing and transformed in the study reach were estimated for the simulation period. Approximately 5.1% of the input nitrate fluxes reacted within the study site. 4.6% of the input fluxes of ammonia and 12.5% of the incoming phosphate fluxes react in the study reach. All nutrient fluxes decreased with the exception of particulate P that slightly increased. Figure 4b presents the decrease of the concentration of Particulate Nitrogen and Phosphorous, after the rain events. The RS-tempQ Model was shown to be capable of simulating the processes temporally during the wet and dry period and spatially where the river corridor is expanded and contracted. The RS-tempQ Model can be used to assess the hydrologic and geochemical reactions occurring in temporary river reaches.

5. CONCLUSIONS

A new generalized reach-scale geochemical model was developed that simulates the hydrology, sediment transport and water quality of temporary rivers. It simulates the special morphological characteristics of temporary rivers, such as the expansion – contraction dynamic of the river bed, up-welling and down-welling zones, and the effect of temperature and sediment moisture on the processes. The RS-tempQ Model can be used to assess the hydrologic and geochemical reactions occurring in temporary river reaches and evaluate the impact climate change to such aquatic systems.

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Evaluation of Water and Nitrogen Balance in the Ponded Water of Rice Fields

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Abstract: A water and nitrogen balance model for the surface ponded water zone of rice fields was developed. The model estimates the daily ponded water depth and the daily losses and uses of NH_4-N and NO_3-N in their transformation processes. The model was applied with data obtained from a rice field during 2006 at Thessaloniki plain in Northern Greece. An amount of 30 % and 45 % of applied water were lost with surface runoff and deep percolation to groundwater, respectively. The gaseous losses of nitrogen (volatilization and denitrification) and nitrogen uptake by algae were the main processes of nitrogen reduction in the ponded water of rice fields. The study showed that the system of rice field is a natural system where an important amount of influent nitrogen applied by irrigation water can be reduced. These processes decrease the possibilities of water resources contamination.

Keywords: Nitrogen balance, ponded water, rice, water balance.

1. INTRODUCTION

Rice is one of the most important cultivated crops in the world. Globally, the total cultivated area in the world is 150.7 million ha, 90 % of which is in Asia. In Europe and especially in the Mediterranean countries as Greece, Italy, Spain, France and Portugal, the cultivated area reaches 400 thousand ha (MED-Rice, 2003).

Agricultural activities are the most significant anthropogenic source of groundwater nitrate contamination and the surface water eutrophication. Nitrogenous (NO_3-N) fertilizers are a major source of non-point pollution of inland waters and groundwater. Nitrate losses from applied fertilizers can be high in rice fields, due to the flooding conditions. Evaluation of nitrate losses from rice fields is complicated, because of the variable nature and different properties of soils and the complexity of N-transformation processes that occur in flooded rice fields. Rice, also, accounts for a significant share of water resources consumption among agricultural crops.

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Several mathematical models are available to describe the water balance, the nutrient and pesticides behaviour in flooded rice fields. Modelling of water balance and movement is generally based on deterministic approach e.g. one-dimensional SAWAH model (Wopereis *et al.,* 1994). Their main use is attributed to the water management and focus on percolation losses, water efficiency and groundwater recharge (Wopereis *et al.,* 1994; Bouman *et al.,* 1994; Singh *et al.,* 2001; Liu *et al.,* 2004). Most models of nitrogen fate in rice fields focus on the processes in the flooded water [e.g., PADDY, (Inao and Kitamura, 1999), PCPF-1 (Watanabe and Takagi, 2005), RICEWQ, (Williams *et al.,* 1999)]. Some others describe the mass transport in the flooded water and the soil beneath it. Chung *et al.* (2003) developed the GLEAMS-PADDY model to describe nutrients loading in surface water bodies and the groundwater. Chowdary *et al.* (2004) developed and applied a simple model for assessing concentration of nitrates in water percolating out of the flooded rice. The modeling of natural processes of water balance and nitrogen in rice fields is a challenge in the last years in relation of water saving, food production and environmental protection.

In this paper a field scale model that simulates the water balance and the temporal dynamics of N in the ponded water of rice fields under flooding conditions is developed. The model includes all the important N-transformation processes such as volatilisation, nitrification, denitrification, uptake and leaching. The main purpose of this paper is to develop a simple model and to apply it under the conditions of rice cultivation practice in North Greece environment.

2. MATERIAL AND METHODS

2.1. Model Development

In order to predict the concentration of nitrate in the water that percolates out of the root zone, it is necessary to determine a nitrogen and water balance in the flooded water and the soil root zone. Quantification of N losses from a rice field requires information of daily percolation rates and daily standing water depths in the fields. For this reason, an integrate model was developed which consists with the hydrology and N submodels. The first module is a daily soil water balance model that estimates ponding water depth and percolation rates. The second module is essentially a N-balance model that quantifies various N transport processes, along with N-uptake by plants and nitrate leaching out with the percolated water.

2.2. The Water Balance Model

The water balance in flooded rice fields is described by a general equation in which the sum of inflow is balanced with the sum of outflows and the change in water storage of flooded pond for a time period. The form of this equation is:

$$\frac{dV}{dt} = R_o + GWi - R_{ro} - R_{inf} - GWo + A(R-E) \quad (1)$$

where:
V volume (m³), A surface area (m²), R rainfall (m d^{-1}), E evaporation from the free water surface (m d^{-1}), R_o inflow from the irrigation network (m^3d^{-1}), R_{inf} infiltration (m^3d^{-1}), GWi and GWo inflow and outflow from groundwater, respectively, R_{ro} surface runoff (m^3d^{-1}). The term dV/dt describes the change in water storage in the pond.

The water balance applied at the system of the water volume on soil surface and the water in soil rooting depth. This consideration is due to the saturated conditions until soil surface that resulted from the high water table, in the lowland rice fields. During the non-cultivated period the water table is account to be approximately at 1 m below soil surface while during the growing period it reaches the soil surface.

The water supply from the irrigation network and the surface runoff from the field can be measured by establishing the appropriate inflow and outflow devices, respectively. The flooded water on the rice field is infiltrated in the soil. In fields with high water table, as in the case of lowland areas with the water table near or at the soil surface, regulated by the water surface in the drainage ditches, the infiltrated water in the soil is moved then horizontally to the ditches. The drainage discharge to the ditches is evaluated as a problem of water movement to drains (de Ridder and Zijlstra, 1994).

Crop reference evapotranspiration are estimated using Penman-Monteith method (Allen *et al.,* 1998). The Kc varies over growing period. The growing period can be divided into four distinct stages: initial, crop development, mid-season and late season. For Mediterranean regions, the duration of these stages for paddy rice are 30/30/60/30 days, respectively (Allen *et al.,* 1998).

2.3. Nitrogen Balance Model

The N available for plant uptake and leaching out of the root zone depends on the transport and transformations of different N species that undergo in the soil–water–plant–atmosphere system (Chowdary *et al.,* 2004). The various processes involved are mineralization, immobilization, nitrification, ammonia volatilization, denitrification, and ammonium adsorption, leaching and plant uptake.

The presence of oxidized and reduced soil processes makes the flooded soils a unique system where both aerobic and anaerobic N metabolism can occur in close proximity. In the zone of flooded water the dominant N-transformation processes are nitrification, ammonia volatilization, denitrification, loading from irrigation water, removing with runoff water, leaching with the infiltrated water and uptake by the presented algae.

The mass balance law describes the rate of total mass of a solute in a pond. Mathematically it is expressed by the equation:

$$\frac{d(VC)}{dt} = Q_K C_K + Q_R C_R - Q_E C - Q_G C - L_{vola} - L_{nitro} - L_{deni} - L_{uptake} \quad (2)$$

where:

C concentration of solute in pond (kg m^{-3}), L_{vola} volatilization of NH_4 (kg d^{-1}), L_{nitro} nitrification of NH_4 (kg d^{-1}), L_{deni} denitrification of NO_3 (kg d^{-1}) and L_{uptake} N uptake by plants (algae) (kg d^{-1}). The symbols K and R indicate the inflow from irrigation canal and rainfall, respectively. Q_E and Q_G are the surface runoff and infiltration/percolation, respectively.

The term of denitrification is not involved in equation for NH_4-N, while the term of volatilization is not involved in equation for NO_3-N. The N transformations that take place in aqueous and soil media are approximated by first-order kinetics. It may be noted that in a given system, most of these reactions may occur simultaneously, depending on the energy source, oxygen level, soil water content, temperature and other dependent variables. Volatilization, nitrification and denitrification (Rao *et al.*, 1984; Stockle *et al.*, 1994) are described by

$$L_{vola} = C_{NH4}(1-exp(-K_{vol}t)) \quad (3)$$

$$L_{nitro} = C_{NH4}(1-exp(-K_{nit}t)) \quad (4)$$

$$L_{deni} = C_{NO3}(1-exp(-K_{den}t)) \quad (5)$$

where:

L_{vola} ammonia losses during volatilization (kg N m^{-3}); L_{nitro} nitrate produced during nitrification (kg N m^{-3}); L_{deni} amount of nitrate denitrified (kg N ha^{-1}); C_{NH4} and C_{NO3} concentration of NH_4-N and NO_3-N in the standing water (kg N m^{-3}), respectively; and K_{vol}, K_{nit} and K_{den} volatilization, nitrification and denitrification rate constant (per day), respectively.

A part of inorganic N in the ponded water is uptaken by algae growing in the water. The N uptake is described by a kinetic reaction of zero order. The infiltrated water transports the inorganic forms of N into the soil. The infiltrated nitrogen load into the soil is evaluated by multiply the infiltration rate times the concentration of nitrogen the flooded water, while the leaching of N with the percolated water is evaluated by the same rate of infiltrated water times the concentration in the soil profile. Nitrogen leaching to the groundwater occurs when the water table is deep enough or to the drainage ditches when the water table is near soil surface.

2.4. Site Description and Measurements

The mathematical model was applied to simulate the water and nitrogen balance in an experimental field of 0.55 ha at the southeastern region of Thessaloniki plain in Northern Greece (40° 40' N, 22° 50' E) for the growing period of 2005. Rice is one of the main irrigated crops (approximately 15 to 24 % of the irrigated area). One week before sowing day (8 of May), the field received 200 kg/ha of N and 45 days later (2 of July), received 50 kg/ha.

The rice field began to receive irrigation water from a canal of open channels irrigation network at 12 of May and maintained at 5 to 20 cm depth in the field until

20th of September. Twenty-four days before harvest, irrigation stops in order to drain the field. For several periods of one week or more there were no inflow water in the field. The inflow discharge to the field was approximately 2.8 to 3 L/sec. Overflow at the end of the field was measured at some periods with discharges ranged from 0.75 to 1.1 L/sec. The water table was measured at 105 cm before the flooding of the field and it reached the soil surface after the flooding.

The soil layer near surface is characterized as silty clay (0-60 cm) texture. The bulk density and hydraulic conductivity of the soil of the rooting zone (0-30 cm) are ρ_b = 1.36 g cm^{-3} and 3.6 cm d^{-1}, respectively.

The climatological data for the simulation period obtained from a nearby meteorological station. The daily data is used to calculate the potential evapotranspiration by the Penman –Monteith method (Allen *et al.,* 1998). Also daily rainfall data is also used.

The reaction rate constants for volatilization, nitrification and denitrification were estimated from the literature and corrected during the calibration procedure (Johnsson *et al.,* 1987; Chowdary *et al.,* 2004). These process rate constants in literature ranged for K_{vol}: 0.03 to 0.8 per day, for K_{nit}: 0.02 to 0.2 per day and for K_{den}: 0.02 to 0.2 per day. The water, NH_4-N and NO_3-N mass balance and their transformations, in the zone of flooded water were evaluated for the rice growing period (1/5/2005 to 15/10/2005).

3. RESULTS AND DISCUSSION

3.1. Evaluation of Water Balance in Ponded Water

The measured amounts of rainfall, inflow and runoff to the pond of rice field and the estimated evapotranspiration were used in Eq. 1 to calculate the daily water balance and the cumulative water storage in the rice pond. The depth of water in the pond expresses the cumulative water storage each day.

The cumulative amounts of water balance components (inflow, runoff, evapotranspiration, infiltration) are given in Figure 1. In Figure 2 the variation of ponded water depth and the daily water balance is presented. The surface inflow from irrigation networks and runoff from the end of the field were 3647 mm and 1118 mm respectively. Rainfall contributes in 248.6 mm of water balance, while losses from evapotranspiration and infiltration were 930 and 1867 mm, respectively.

The major route of water losses in the ponded water was runoff and infiltration, which accounted for the 29.6 and 46.4% of the total applied water. The evapotransriration accounted for 24.0% of the total water applied.

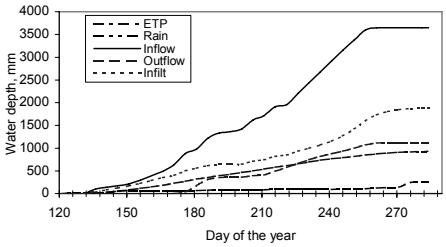


Figure 1. Cumulative amounts of water balance components (inflow, runoff, evapotranspiration, infiltration).

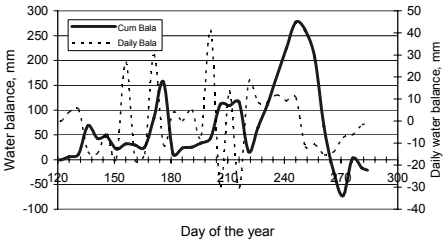


Figure 2. Variation of ponded water depth and daily water balance in rice field.

3.2. Evaluation of Nitrogen Balance in Ponded Water

The NH_4-N and NO_3-N mass balance is evaluated separately for daily basis in the ponded water. The calibrated values of transformations rate constants were K_{nit}= 0.2 d^{-1}, K_{vol}= 0.5 d^{-1}, K_{den}= 0.24 d^{-1}, K_{upta}= 150 kg d^{-1}. The evaluated NH_4-N daily runoff load in comparison with the measured values is given in Figure 4. It is seen that the evaluated values are in reasonable agreement with the measured values. Unfortunately only a small number of daily measurements are available for this comparison. The model evaluation with statistical indicators is not valid with a limit number of measurements and is not used in this study. Reasonable agreement is also resulted from the comparison between evaluated NO_3-N daily runoff load and the measurements (Figure 6).

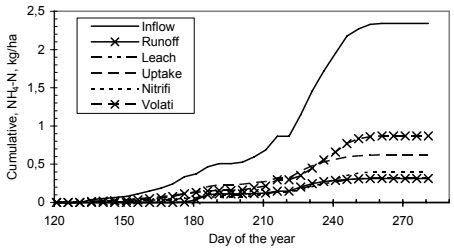


Figure 3. Cumulative amounts of NH_4-N mass balance components (loading with irrigation water, runoff losses, nitrification, volatilization, uptake, leaching infiltration).

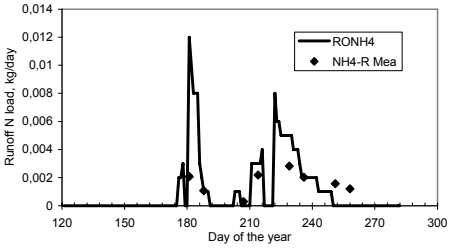


Figure 4. Comparison between estimated runoff load and measurements of NH_4-N.

The cumulative mass of balance components described as nitrified and volatilized NH_4-N, denitrified NO_3-N and uptake by algae in the pond are presented in Figure 3 and Figure 5. The total amounts of nitrogen transformation for the growing period are given in Table 1.

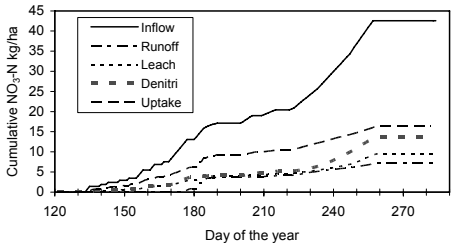


Figure 5. Cumulative amounts of NO_3-N mass balance components (loading with irrigation water, runoff losses, nitrification, denitrifycation, uptake, leaching with infiltration).

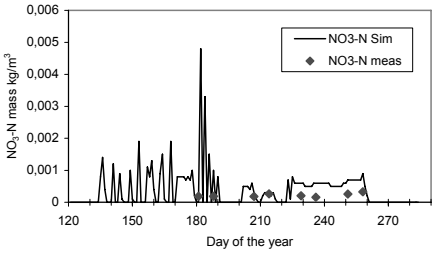


Figure 6. Comparison between estimated runoff load and measurements of NO_3-N.

Table 1. Total N mass balance in the pond of rice field at the end of growing period.

Processes of NH_4-N	Amounts (kg/ha)	Processes of NO_3-N	Amounts (kg/ha)
Inflow Irrigation load	2.340	Inflow irrigation load	42.548
Outflow		Outflow	
Runoff losses	0.314 (12.4 %)	Runoff losses	7.120 (15.2 %)
Nitrification	0.401 (15.8 %)	Nitrification	0.401
Volatilization	0.870 (34.4 %)	Denitrification	13.756 (29.3 %)
Leaching	0.324 (12.8 %)	Leaching	9.553 (20.4 %)
Uptake by algae	0.623 (24.6 %)	Uptake by algae	16.471 (35.1 %)

The main source of N loading in rice's ponded water was the N loads from the irrigation water. Significant difference between the loading with irrigation water and the runoff/leaching losses was observed. These NH_4-N losses accounted the 27.3 % of the total NH_4-N load to the ponded water of the rice field. The NO_3-N losses from the ponded water with runoff and infiltration were the 39.2 % of the total added load to the rice pond. The gaseous (volatilization and nitrification) losses of N were 34.4 % and 29.3 % for NH4-N volatilization and NO_3-N denitrification, respectively. An important part of N is lost with uptake by algae which grown in the ponded water, which accounts 24.6 % for NH_4-N and 35.1 % for NO_3-N respectively.

4. CONCLUSIONS

A water and nitrogen balance model for the flooded water zone of rice fields is presented in this article. The model predicts daily ponded water depth and daily losses and uses of NH_4-N and NO_3-N under different transformation processes. The water and nitrogen balance were studied in a rice field at Thessaloniki plain of northern Greece for the growing period of 2005 (May to October).

The irrigation practice in the area showed that high amounts of water were used. A 30 % of applied water lost with surface runoff and 45 % of this contributed to groundwater leaching. In the frame of best water management, a program of reduction of water use is needed. Significant part of the total water applied in the field at the last part of growing period. During this period no water is used for irrigation of other

main crops in the area, as maize and cotton and plenty water is available to the rice.

The nitrogen balance study showed that the system of ponded water of rice field could assimilate an important amount of applied nitrogen by irrigation water. This process usually happens in surface flow wetlands, and is often used to improving the water quality in constructed wetlands. The gaseous losses of nitrogen were 34.4 % NH_4-N volatilization and 29.3 % NO_3-N denitrification. Significant part of nitrogen loading (26.4 and 35.1 % for the NH_4-N and NO_3-N, respectively) was used by the algae that grow in the ponded water of rice fields. A validation of model under the conditions of nitrogen fertigation with embodying to the ponded water might be required. The proposed procedure for evaluation of water and nitrogen balance in the flooded system of rice fields can be used for the water and applied nitrogen management and for the protection and conservative of the water resources.

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Subsurface Drip Irrigation of Sorghum with Treated Municipal Wastewater

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Abstract: The aim of this study was to investigate the effects of subsurface drip irrigation with treated municipal wastewater on growth and yield of sweet sorghum (*Sorghum bicolor L.*), as energy plant and consequently to evaluate the use of wastewater in water saving terms compared to freshwater use. For this purpose a field experiment was conducted at the farm of the University of Thessaly during the growing seasons of 2005 and 2006. The experimental field was separated in two parts. The first part, constituted the first treatment, which was irrigated only with fresh water from the borehole of the farm. The second part, constituted the second treatment, and was irrigated periodically with wastewater, provided by the treatment plant of the city of Volos and with fresh water. Irrigation water depth was determined using a class-A evaporation pan. Plant growth measurements included plants' height and biomass production. The experimental results show that growth and final biomass production did not differ significantly between the treatments. As regards the water consumption, the use of wastewater resulted in significant saving of fresh water revealing the potential of sweet sorghum irrigated with wastewater as an alternative crop for biomass production.

Keywords: Energy crops, treated effluent, water conservation.

1. INTRODUCTION

As the demand for potable water resources increases, wastewater is receiving attention as an alternate irrigation source. Research studies in Greece have investigated the possibility of use of liquid wastes for irrigation of agricultural row crops, greenhouse tomato and pepper, ornamental plants and turf (Panoras *et al.*, 2000; Vakalis and Tsantilas, 2002; Sakellariou – Makrantonaki *et al.*, 2005). From these research has been evident that the irrigation with treated liquid wastes has given better or equal crop yields compared to fresh water irrigated crops.

Presently, effluent used in irrigation is normally delivered through surface or sprinkle irrigation systems; however, in recent years interest in microirrigation

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systems for this purpose has increased. Subsurface drip is an emerging alternative wastewater technology with a great deal of merit. The main advantages of the method are (Phene *et al.*, 1983): the use of low quality irrigation water (e.g. water with increased salinity), the reduction of water losses due to low evaporation, the precise application of fertilizers and pesticides, the decrease in weed growth (soil surface remains dry), and the absence of requirement of moving the network before seeding or harvest. Also, SDI is considered to be ideal for irrigation with wastewater because it constitutes a close system and does not expose farmers to any danger since they do not come in immediate contact with the recycled water. Disadvantages of subsurface drip irrigation include the high initial cost, the difficulty in inspection of underground network and the mechanical damages caused by cultivation machinery or by rodents, (Sakellariou – Makrantonaki *et al.*, 2002).

Following the increasing interest for renewable raw material included in a new course of EU agricultural policy (Agenda 2000), a particular attention was given to biomass crops in the last decade. Sorghum, a C_4 annual crop with high photosynthetic efficiency and biomass yields, has received considerable attention during the last years as an alternative source for fuel production. This crop promises high yield in biomass with low inputs of water and fertilization (Curt *et al.*, 1995; Dalianis 1996, Sakellariou – Makrantonaki *et al.*, 2005). Generally, sorghum is a plant that can tolerate drought, however, when irrigated its yield is considerably increased (Mastrorilli *et al.*, 1995).

The aim of present work was to investigate the effects of irrigation with treated municipal wastewater through subsurface drip method on growth and yield of sorghum and to evaluate the use of recycled water in water saving terms compared to freshwater use.

2. MATERIALS AND METHODS

The growth and the yield of the energy plant sorghum irrigated with treated municipal wastewater was studied at the farm of University of Thessaly at Velestino during the cultivation seasons of years 2005 and 2006.

The experiment was carried out in 2 treatments (irrigation with fresh water and irrigation with recycled water) replicated four times. The variety of sweet sorghum that used was Keller. The soil at the site is calcareous, with silty clay loam texture, well drained and belongs to the subgroup Typic Xerochrepts. The irrigation network was placed 45 cm under the soil surface using a special plow. The network included electric valves, connected with a controller for the automatic starting and ending of irrigation and an equal number of water gauges, for the control of any divergences from the desirable irrigation depth. Special flushing valves were fitted to the system in order to avoid any clogging problems.

The seeding took place on 30 May 2005 and 11 May 2006, respectively. No fertilisers were applied. The first year of the experiment the irrigations were applied

from 11 July to 18 October and the quantity of water applied covered the 85% of the estimated evapotranspiration, accounting the negligible evaporation (subsurface water application). The second year the irrigations applied from 21 June to 20 September and the quantities of water covered the 100% of evapotranspiration.

The first treatment was irrigated with fresh water. The second one was irrigated alternately with fresh water and wastewater due to the lightly increased salinity that existed in the wastewater and also because of its increased concentration of ions of chloride. Each irrigation with wastewater was followed by two irrigation applications with fresh water. The measurements that were taken included the height of plants, the number of leaves, the leaf area index, the soil moisture as well as the green and dry biomass. Also measurements of meteorological data (rainfall and air temperature) were taken from the automatic meteorological station of the farm.

The analyses of water from the treatment plant (Table 1) show that the electric conductivity (E.C.) was marginally suitable for irrigation of crops, and also, the concentration of ions of chloride was very high (ranged from 1584 mg/l to 1938 mg/l, with allowable limits 355 mg/l according to Bahri and Brissaud (2002). Chloride ions' concentration was high due to the chlorination of wastes in treatment plant for decontamination. For this reason alternate irrigations were applied with fresh water in "wastewater" treatment.

Table 1. Qualitative characteristics of irrigation water (averaged values).

	pH	Cl^- (mg/l)	SS (mg/l)	COD (mg/l)	$N\text{-}NH_3$ (mg/l)	$N\text{-}NO_3$ (mg/l)	P (mg/l)	EC (mS/cm)
Recycled water	7.97 (0.10)	1,858 (64.9)	17 (0.42)	55 (0.60)	3.75 (0.05)	6.6 (0.05)	1.3 (0.01)	6.73 (0.11)
Fresh water	7.65 (0.11)	22 (1.14)	-	-	<1	17.9 (0.20)	0.5 (0.02)	0.65 (0.01)

Numbers in brackets represent standard error values

3. RESULTS AND DISCUSSION

3.1. Climatic Data

The study area is characterized by a typical Mediterranean climate with a hot and dry summer and cool-humid winter. During 2005 growing season, rainfall was low during the summer months, compared to the previous years, while the temperatures were slightly lower to the averages of the last 25 years. During 2006, evapotranspiration values reached a maximum of 7.2 mm per day on 18/7/2006, 6/8/2006 and 14/8/2006, a period with very high temperatures and no or low rainfall (Figure 1). From 7/9/2006 to 18/9/2006 evapotranspiration was gradually decreased because of the low temperatures which prevailed this period. From 7/6/2006 to 10/10/2006 263.18 mm of rain were recorded.

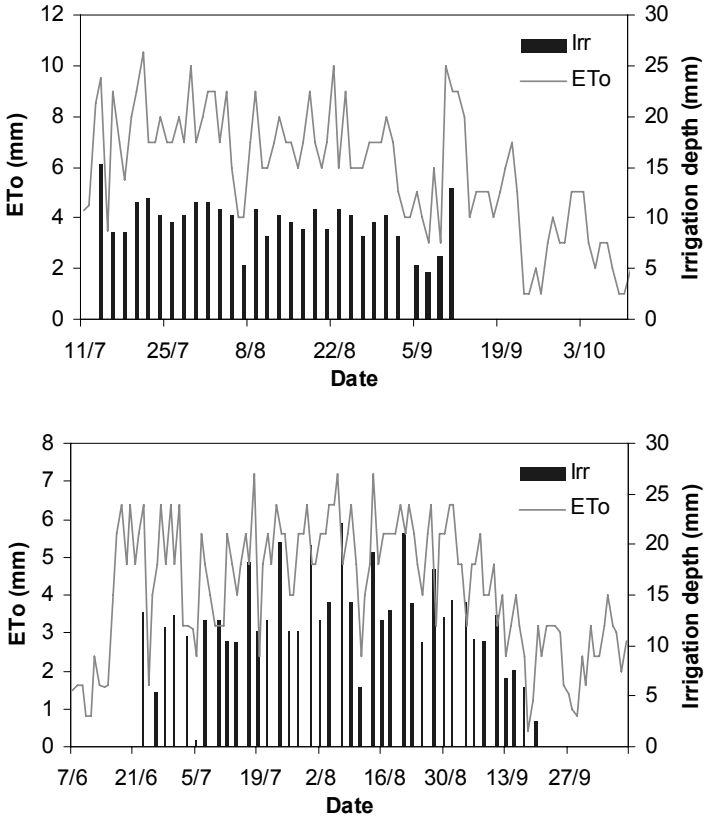


Figure 1. Reference evapotranspiration and applied irrigation depth for the growing season of 2005 (above) and 2006 (below).

3.2. Irrigation

During the farming period of 2005, 30 irrigations were applied. A total amount of approximately 425 mm of irrigation water was supplied to each one of the two treatments. In wastewater treatment the 112 mm were recycled water and thus a saving of 26.3% of fresh water was achieved. Moreover, if we also take into account the saving of irrigation water due to subsurface application (15%), then the globall saving of irrigation water reaches the 41.3% in the treatment irrigated with wastewater and 15% in the treatment irrigated with fresh water only. It must be reported that a gun sprinkler was employed for the germination irrigation which applied 125 mm of water. In the duration of the farming period 91 mm of rain were measured.

During 2006 period, 533 mm of water were totally given. From the 533 mm of water, the 483 mm were given with the subsurface drip irrigation system during the irrigation period from 21/6/2006 to 20/9/2006 while the rest 50 mm were applied with a gun sprinkler for germination. During the farming period of 2006, 40 irrigations

were applied. The quantity of the recycled water that was given to the treatment of "wastewater" was 143.7 mm resulting in a 27% saving of fresh water.

3.3. Crop Growth and Yield

3.3.1. Height of plants

The growth of Sorghum as reflected by plant height is illustrated in Figure 2. Crop reached a maximum height of 3.1 m. Statistically significant differencies were not observed between the 2 treatments (fresh water – recycled water), apart from a small divergence recorded on 08/08/2005 and 28/08/2005 where the wastewater treatment surpassed that of fresh water.

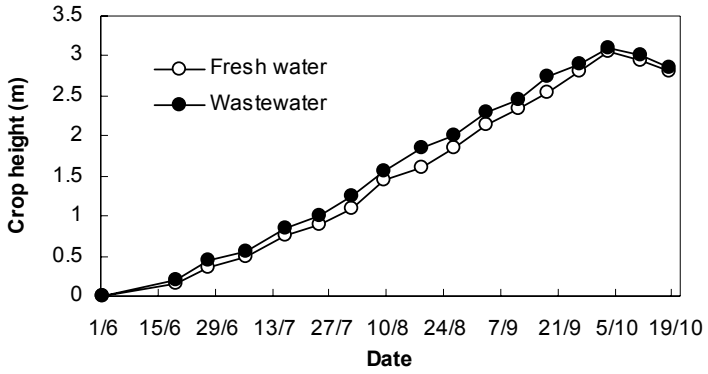


Figure 2. Height of Sorghum bicolor L. in both treatments during the growing season of 2005.

Figure 3 presents the average height of plants during 2006 growing season. The average height of plants at the treatment irrigated with fresh water reached 3.14 m while the average height of plants of wastewater treatment reached the value of 2.94 m. Statistically significant differencies were not observed between the 2 treatments, apart from a small divergence that recorded between 28/8/2006 and 21/9/2006, where the fresh water treatment excceded that of wastewater.

This could be attributed to the increased concentration of salts and chloride ions of wastewater, as, until 28/8/2006 when this difference in growth began, nine irrigations with wastewater were already applied. On 21/9/2006, when the last measurement of height of plants took place, a very small increase in height was observed in both treatments, in relation to the height's increasing rate up to this point, a fact that leads to the conclusion that the plant reached its final height.

3.3.2. Biomass production

During the growing season of 2005, as its presented in Figure 4, the dry biomass reached 25 T/ha, while there were no significant differencies between the two

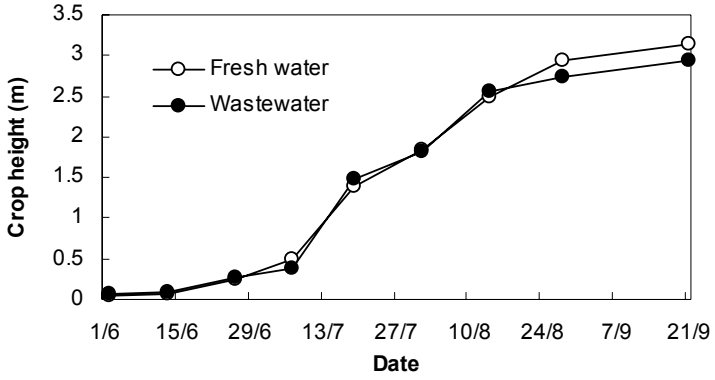


Figure 3. Height of Sorghum bicolor L. in both treatments during the growing season of 2006.

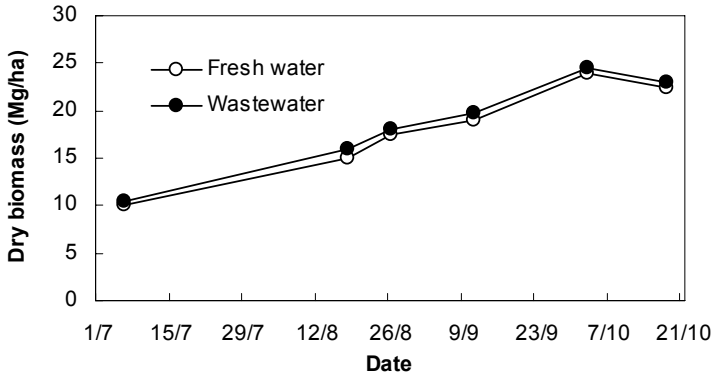


Figure 4. Dry biomass of Sorghum bicolor L. in both treatments during the growing season of 2005.

treatments. Generally, the dry biomass increased regularly during the first 280-day period, at the end of which it reached its maximum value, roughly 25 T/ha, and decreased afterwards. With the medium thermal value of biomass corresponded to 0.4 T of equivalent oil (TEO) per T of dry biomass (Dolcioti *et al.* 1996), the biggest production that was observed was 25 T/ha, corresponds to 1.0 TEO.

Figure 5 presents the dry biomass of crop during 2006. During the whole cultivation period, between the two treatments no statistically significant differencies were observed, except for the 3rd and 6th cutting. The higher yields were recorded at the cutting of 26/9/2006, with 31.9 T/ha for the treatment of wastewater and 32.6 T/ha for the treatment of fresh water. Although, no significant differencies exist between the two treatments, there is a slight superiority of the treatment that was irrigated only with fresh water.

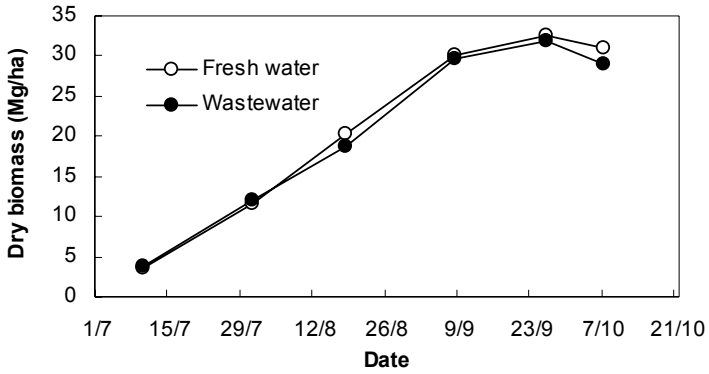


Figure 5. Dry biomass of Sorghum bicolor L. in both treatments during the growing season of 2006.

From the 1st (10/7/2006) to the 5th cutting (26/9/2006) included, the dry biomass increased, while at the 6th cutting (7/10/2006) a lower dry biomass was measured, due to the completion of the development stages of the plant and the aging of the crop. The highest production of dry biomass that was observed at the treatment of fresh water corresponds to 1.3 TEO while the highest biomass production that was observed at the treatment of wastewater corresponds to 1.276 TEO.

4. CONCLUSIONS

The possibility of irrigation of sorghum with treated wastewater was investigated. No statistically significant differences in growth and in dry biomass were observed between fresh water and wastewater irrigated crop.

The treated municipal wastewaters by their utilisation for irrigation of crops constitute an important mean for saving fresh water for other uses.

The performance of subsurface drip irrigation as a wastewater distribution system appeared to be very effectiveness. The applicability of this method in wastewater systems should increase, as future research succeeds in establishing proper system sizing criteria and continuing improvements in the reliability of system components.

Finally, the growth and yields of sorghum make obvious the potential of the crop as an alternative cultivation for biomass production.

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Guidelines for Drought Management

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Abstract: The Guidelines are the result of the research carried out within the framework of the MEDA-Water project MEDROPLAN that analyses drought and water scarcity management in Mediterranean countries promoting a risk based preparedness and mitigation approach. The Guidelines respond to the growing issue of drought preparedness planning, monitoring, and mitigation which has worldwide application. The purpose is to provide countries with a framework for effective and systematic approach to prevent and/or minimize the impacts of drought on people. The methodologies and lessons learned are focused on a specific, drought-prone region so the applications have more significance. The Mediterranean region exemplifies many other drought-prone regions with rapidly expanding populations that are placing increased pressure on already limited water supplies The experiences in the development and implementation of drought and water scarcity management plans highlight the success and challenges of coping with drought for societies with different vulnerabilities and emphasize risk-based drought management as a critical approach to mitigate the impacts associated to drought-induced water shortages. Based on these experiences and the current legislation, management, technology and methods for evaluating risk, the Guidelines synthesise drought management operational actions that link science and policy and that can be applied to other regions.

Keywords: Drought, management actions, water scarcity, water use, Mediterranean region.

1. THE PURPOSE OF THE GUIDELINES

The purpose of the MEDROPLAN Guidelines is to provide Mediterranean countries with a framework for effective and systematic approach to prevent and/or minimize the impacts of drought on people. The Guidelines are intended to complement the ongoing regional and country water basin planning efforts and the ongoing agricultural policy initiatives. The Guidelines outline both long term and short term measures that are to be used to prevent and mitigate the effects of drought.

The experiences in the development and implementation of drought and water scarcity management plans highlight the success and challenges of coping with

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drought for societies with different vulnerabilities and emphasize risk-based drought management as a critical approach to mitigate the impacts associated to drought-induced water shortages. Based on these experiences and the current legislation, management, technology and methods for evaluating risk, the Guidelines synthesise drought management operational actions that link science and policy and that can be applied to other regions. Finally, the Guidelines are produced in six languages to reach the largest possible number of stakeholders in the Mediterranean (Arabic, English, French, Greek, Italic, and Spanish).

2. COMPONENTS OF THE GUIDELINES

2.1. The Planning Framework

The planning framework defines the local, regional and national purpose for developing drought planning and highlights the dynamic process that responds to changing pressures in the environment and society. The planning framework guides the user of the Guidelines to define the planning purpose and process, establish a common language among stakeholders, and highlights the importance of using a common set of terms and concepts for developing a drought management plan that can be discussed among a full range of stakeholders.

2.2. The Organizational Component

The social and hydrological context of Mediterranean have resulted in the complex institutional framework, and highlighted the importance of stakeholder involvement and awareness building for successful drought management. Designing effective risk based strategies that mitigate the effects of drought in agriculture and water supply systems, ultimately depends on the role of organizations, institutions, and civil stakeholders involved in drought in each case. The organizational component assists the user to: (a) Compile and provide the most comprehensive information about how society responds to drought and establish the linkages among the stakeholders; (b) Coordinate with the various institutions to avoid conflict, duplication, and expedite the administrative and legal process; (c) Provide responsible and timely public information; and (d) “Declare drought”.

2.3. The Methodological Component

The methodological component presents the scientific approach to risk evaluation, including characterization of drought episodes, development of indicators of risk in hydrological and agricultural systems, and analysis of the role of economic instruments and groundwater for risk mitigation. This component also includes the description of an integrated method for evaluating social vulnerability based on indicators that include the capacity to anticipate, cope, and respond to drought.

Drought is characterisation and risk and vulnerability analysis are complex and there are a wide range of methods applied. Each method has its own merit and they are usually supportive of each other. A combination of methods is usually most rewarding.

The methodological component provides a framework to: (a) Compile and provide the most comprehensive technical and scientific approaches to drought characterization, development of indicators of risk in hydrological and agricultural systems; (b) Define the methods used for risk management in the context of Mediterranean climate and social characteristics: including economic instruments, application of technology, and groundwater use, etc; (c) Define the academic methods for evaluating social vulnerability based on indicators that include the capacity to anticipate, cope, and respond to drought; and (d) Encourage technical studies to strengthen the use of indicators and the declaration of drought.

2.4. The Operational Component

The operational component identifies both the long and short term activities and actions that can be implemented to prevent and mitigate drought impacts. The activities and actions are essential for the creation of specific drought planning and response efforts. The operational component includes five aspects: (a) Preparedness and early warning (permanent measures); (b) Establishing priorities to be respected during water scarcity situations; (c) Thresholds defined by drought indices and indicators (physical and social); (d) Evaluating the process to implement the actions; and (e) Defining the actions. Table 1 summarises the types of actions that may be adequate for preparedness before drought and three levels of drought risk.

2.5. The Public Review Component

The Guidelines are developed in the context of current drought vulnerability, legislation, management, and technologies. Drought management plans are always in progress and all components need to be considered dynamic. As technologies evolve, new programs are developed, and institutional responsibilities change, these plans have to be revised.

ACKNOWLEDGEMENTS AND FURTHER INFORMATION

The work reported in this paper is the effort of the participants in the MEDROPLAN project; we gratefully acknowledge all their valuable contributions.

The MEDROPLAN Guidelines for drought management. CIHEAM – IMAZ, Spain. http://www.iamz.ciheam.org/medroplan.

Table 1. Summary of the types of drought management actions.

Risk level	Monitoring	Objective	Actions
Preparedness	Indicators show a normal situation	To ensure that a preparedness and early warning plan is in place	Development of a management plan and strategy for revision Implementation of a monitoring and early warning systems
Pre-alert	Indicators show initial stage of danger; no observed impacts	To ensure acceptance of measures to be taken in worst situations	Low cost, indirect, voluntary Non structural directed to influence water demand and avoid worse situations Focus on communication
Alert	Drought impacts will occur if measures are not taken	To overcome the drought situation and to guarantee water supply	Low cost, direct, coercive Non structural Water restrictions for non-urban users Changes in management Revision of tariffs Rights Exchanging Centres
Emergency	Drought is persistent and impacts have occurred; water supply is not guaranteed	To minimize damage, the priority is drinking water	High cost, direct, restrictive Structural (infrastructure, water transfers) Non structural (permission for new groundwater abstraction) Water restrictions for all users

Leaching Behaviour of Stabilised/Solidified Mixed Character Waste

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Abstract: The purpose of this work is to evaluate different S/S formulations in an mixed character waste in order to improve the control of the mobility of the pollutants and the ecotoxicity of the samples in the landfill. Different mixtures of cement or lime as binders and additives (silicieous additives and sorbents) have been used in order to reduce the mobility of the chemical and ecotoxicological regulated parameters and to compare the results for commercial and residual additives. The best results have been obtained when sorbents or sodium silicate have been used. Finally, some linear expressions between the chemical parameters and the quantity of material used in the samples have been obtained.

Keywords: Ecotoxicity, foundry sludge, landfill, leaching, stabilisation/solidification.

1. INTRODUCTION

The foundry activities generate a great volume of industrial waste. One of the most important waste materials is the foundry sludge, which is obtained from gas wet cleaning treatment. It is characterised as hazardous waste by the European Waste Catalogue and ecotoxic waste due to organic and inorganic compounds (Coz *et al.*, 2004). Among the different treatment technologies, Stabilisation/ Solidification (S/S) processes are the most suitable ones to manage the foundry sludge prior to land disposal, because of its characteristics. The environmental availability of the S/S products can be studied through leaching tests (Andrés *et al.*, 2006). Among these leaching tests, the EN 12457-2 test for leaching of granular waste materials (EN) and the TCLP are typically compliance tests. In this work, new formulations with different additives (foundry sands, silica fume, sodium silicate, silicic acid, activated carbon and black carbon) have been used in order to fulfil the regulated limits and compare the behaviour of the commercial and residual additives in the process.

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2. METHODS AND MATERIALS

Foundry sludge (FS) was used as waste. Portland Cement Type I (CM) and Lime (LM) were used as binders. The siliceous additives incorporated to the mixtures were silica fume (SF), sodium silicate (SS), silicic acid (SA), foundry sand ashes (FA) and siliceous resin sand (SR). Activated carbon (AC) and black carbon (BC) have been studied as sorbents. The samples with cement as binder have been formulated according to the literature (Conner and Hoeffner, 1998). Waste samples were mixed in a CEMEX W-20, X-02-G laboratory-scale solid mixer prototype and cured for 7 days. The samples were characterised using the TCLP and EN 12457-2 leaching tests. The parameter EC_{50} was determined using the luminescence bioassay with *Vibrio fischeri* in a Microtox toxicity analyser. The Zn concentration on the leachate was measured using a Spectrometer of AE by ICP, Perkin Elmer 400. To determine the TOC the system Euroglas TOC 1200 was employed. Phenol index was determined according to DIN 38409-H16 norm, using a Perkin Elmer VIS spectrophotometer.

3. RESULTS AND DISCUSSION

Seven samples of waste FS have been characterised (Table 1). The foundry sludge is an ecotoxic waste due to organic (phenol index and TOC on the EN leaching test) and inorganic parameters (zinc on EN and TCLP leaching tests).

Table 1: Results of characterisation of foundry sludge.

Test		**EN leaching test**				**TCLP**		
Parameter	pH	Zn $mgkg^{-1}$	Phenol index mgL^{-1}	TOC mgL^{-1}	pH	Zn $mgkg^{-1}$	EC_{50} mgL^{-1}
Waste FS	9.07-9.65	18.4-29	1.23-45.3	9.95-254.2	5.79-6.59	1826-2133	769-1026
Non hazardous limit	<6	50		80			<3000

Figures 1 and 2 show the results of the value of pH and zinc concentration, respectively, in all the S/S products versus the quantity of the studied additives in the binder-additive mixture. In all cases, a 70% waste material has been used.

Only the product with 25% CM and 75% SS does not fulfil the limit for non-hazardous waste. To control the mobility of this amphoteric metal on the leaching tests, a value of pH on the leachate of 9-10 should be achieved (van der Sloot *et al.*, 2006). When a mixture of LM and SF is used, if an EN test is carried out with only SF, the pH value of the leachate is 8.15, but when mixed with LM it can be observed that lime neutralises the acid effects of the SF. The pH value on the TCLP leachate diminishes when increasing the quantity of additive in the mixture, without dependence of the binder used, according to a linear law whose regression coefficient (R^2) is 0.937. This is observed for all the studied additives, except for the additions of 10 and 20% of SR, where the acid character of this residual additive gives a high

mobility of zinc on the TCLP leachate. The results of pH value on the TCLP leachate with CM and SS follow also a linear law whose regression coefficient is 0.980.

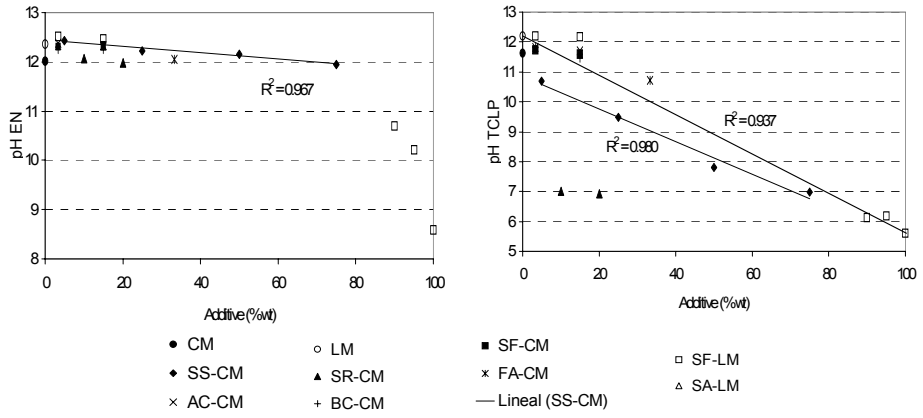


Figure 1. Results of pH value from the leaching tests.

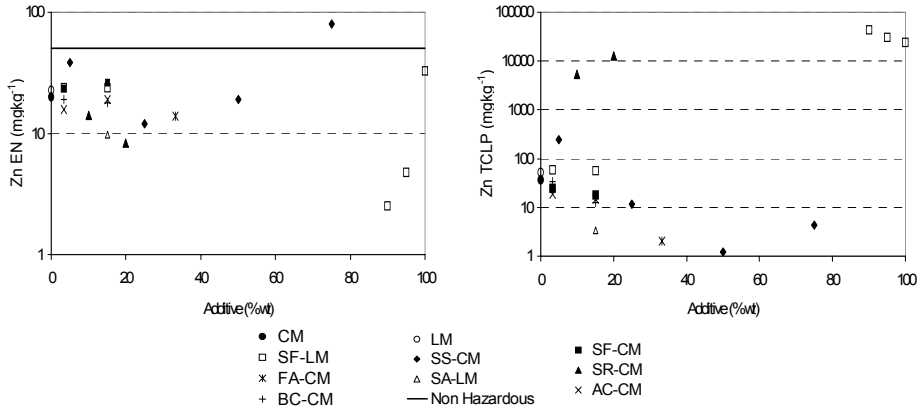


Figure 2. Results of zinc concentration from the leaching tests.

The results of organic parameters in the S/S products have been obtained as immobilised percentage due to their heterogeneous composition (Table 1). These results are given in Figure 3.

The best results are obtained in the samples with sorbent materials (AC and BC). Furthermore, the higher quantity of sorbent is, the higher organic immobilisation results are obtained. Other satisfactory results of organic parameters are obtained in the products with LM and low quantities of SF and the mixtures with 5-25% of SS and CM. The organic results in the samples with SS show a lineal law versus the quantity of the additive in the sample ($R^2 = 0.972$). The results of phenol index in the products with SR are not satisfactory due to some phenolic compounds in the sand.

The immobilisation results in that case are below 0% and have not been represented in the Figure.

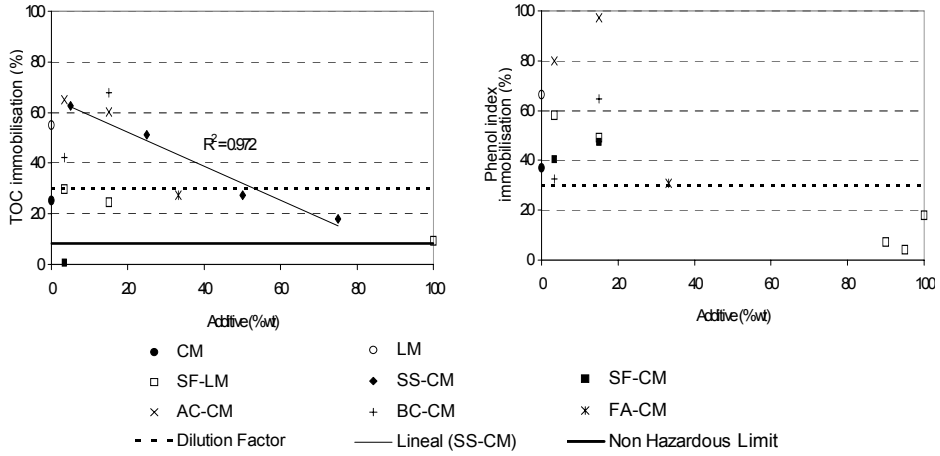


Figure 3. Results of organic parameters from the EN leaching test.

Ecotoxicity results are shown in Figure 4. If the value of the ecotoxicity is greater than 3000 mgL^{-1}, then the sample is not ecotoxic in relation to Spanish regulation. The best results are obtained with sorbent materials in the samples with CM.

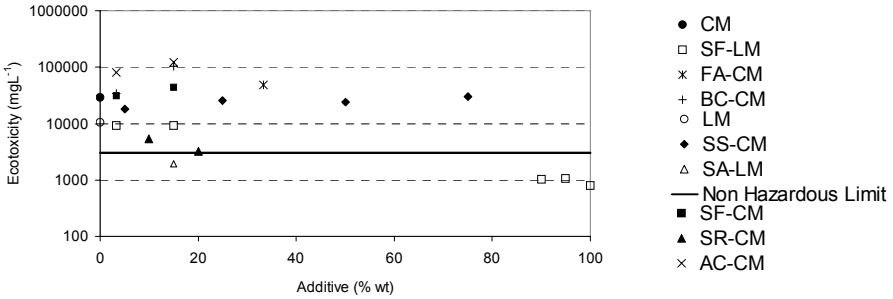


Figure 4. Results of ecotoxicity from the TCLP leaching test.

4. CONCLUSIONS

This work has studied the influence of siliceous and sorbent materials as additives on the S/S of a real waste of mixed character. The use of sorbents in all cases has originated the best results due to their organic retention and the best neutralisation capacities, giving non ecotoxic end-products. Among the siliceous additives, the best results have been with cement and 5-25% sodium silicate. Foundry sand ashes can be

used as replacement of cement due to their similar behaviour in relation to the cement product and their low cost because of their residual character. However, the results of siliceous resin sand as residual additive have not been satisfactory due to their acid character and its content of phenolic compounds. Silica fume as additive in the mixtures with lime gives high amounts of leached zinc. However, the results of silica fume in the mixtures with cement have been satisfactory. Finally, some lineal expressions have been obtained between the studied parameters and the quantity of additive used in the process.

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Temporal Analysis of Hydrometeorological Characteristics in Crete

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Abstract: Water is the most significant factor for the agriculture. Soil moisture deficits are especially valuable for estimating the consequences of climatic variability on crop growth and production. In this study, daily precipitation and pan evaporation measurements provided by the Institute of Land Reclamation of Crete for 11 sites, are utilized in a daily water balance model for estimating various hydrometeorological components. Trends in yearly, warm- and cold- season, precipitation, potential evapotranspiration, and soil moisture deficit are examined for the period 1974-2003. Averaged (for all sites) values for precipitation and pan evaporation are very slightly increasing or decreasing, respectively. Both trends are not significant (in the 90% level). The well documented decreasing of evaporation is apparent in only six of the pans (at 95% significant level). Precipitation and soil moisture deficit do not seem significantly increasing or decreasing in almost all stations.

Keywords: Pan evaporation, precipitation, soil moisture deficit, trends, water balance.

1. INTRODUCTION

The moisture balance at the terrestrial surface can be described as a balance between the atmospheric supply (precipitation) and atmospheric demand (potential evaporation) (Budyko, 1974). Pan evaporation data (taken to be proportional to potential evaporation) are often used together with precipitation measurements to obtain the surface moisture balance, and estimate its variability. Changes in terrestrial precipitation around the world are well documented, and the general trend has been reported as increasing, over most regions (with exceptions in parts of Africa), over both the last 50 and 100 years (Folland *et al.,* 2001). Several studies also have reported declines in pan evaporation rate throughout the Northern and Southern

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hemisphere for various periods since the 1950s (e.g. Peterson *et al.,* 1995; Roderick and Farquhar, 2005). At individual sites, both increases and decreases in pan evaporation have commonly been reported (e.g. Chattopadhyay and Hulme, 1997; Cohen *et al.,* 2002; Roderick and Farquhar, 2004). In this study, variations of daily precipitation and pan evaporation measurements in Crete are evaluated and the temporal analysis is also presented for other water balance components.

2. ANALYSIS AND RESULTS

Daily precipitation and pan evaporation data (covering the period 1974-2003) are provided by the Institute of Land Reclamation of Crete, for 11 hydrometeorological stations in important hydrological basins. Table 1 presents the characteristics of the stations used in the analysis. The daily precipitation (P) and evaporation estimates are utilized in a single-layer water balance model, which calculates actual evapotranspiration (AET) and soil moisture deficit (DEF). Penman-Monteith estimates are used for calibrating potential evapotranspiration (PET) from pan measurements.

Average yearly values of P, PET and DEF are also presented in Table 1. The slopes of the trends of the yearly, warm-, or cold-season estimates of the above parameters are shown in Table 2. The results show that the trends in pan evaporation varied from site to site. Decreasing evaporation is apparent in only six of the pans at 95% significant level, (noted with bold numbers). Five of the pans showed statistically insignificant increases. Precipitation is insignificantly decreasing in 6 sites and increasing in 4 sites. Only one site with significant increasing trend in P is observed. Soil moisture deficit do not seem significantly increasing or decreasing in almost all stations. Yearly increasing precipitation, decreasing potential (pan) evaporation, and decreasing soil moisture deficit, as a function of years is apparent in Figure 1 for Metaxohori.

When averaged over the 11 sites, downward and upward trends for the pan and the precipitation are observed but the corresponding slopes of their overall trends (1.490 or –1.499, respectively) are not significant even in the 90% level. Averaged (over the 11 sites) values for precipitation and pan evaporation are presented in Figure 2.

Table 1. Characteristcs of the stations and their corresponding average annual values of precipitation (P), potential evaporation (PET) and soil moisture deficit (DEF).

Station	Lat.	Long.	Alt. (m)	P (mm/year)	PET (mm/year)	DEF (mm/year)
Protoria	35° 02'	25° 09'	220	548.69	1289.18	932.29
Asimi	35° 03'	25° 06'	200	606.22	1087.61	743.38
Gergeri	35° 08'	24° 56'	450	904.34	1415.51	972.73
Ag. Barbara	35° 08'	25° 00'	570	969.53	1120.11	725.22
Metaxohori	35° 08'	25° 08'	430	753.75	1308.44	915.21
Armaha	35° 10'	25° 21'	500	812.69	1143.73	745.38
Kasteli	35° 13'	25° 20'	350	738.66	1070.93	692.12
Mithi	35°02'	25° 35'	200	546.25	1093.11	764.14
Pahia Ammos	35° 05'	24° 49'	50	587.13	1382.32	962.73
Kalo Horio	35° 07'	25° 44'	20	539.69	1211.47	846.16
Levkogia	35° 11'	24° 27'	90	839.91	1600.88	1147.15

Table 2. Slope parameters for the trends in yearly and warm- or cold- season precipitation (P), potential evaporation (PET) and soil moisture deficit (DEF).

Stations	Yearly			Warm season			Cold season		
	b P mm/yr^2	b PET mm/yr^2	b DEF mm/yr^2	b P mm/(0.5yr)^2	b PET mm/(0.5yr)^2	b DEF mm/(0.5yr)^2	b P mm/(0.5yr)^2	b PET mm/(0.5yr)^2	b DEF mm/(0.5yr)^2
Kalo Horio	1.129	0.144	0.518	-0.264	0.213	0.390	1.393	-0.069	0.128
Mithi	-1.966	-13.443	-10.577	-0.462	-10.068	-10.269	-1.504	-3.375	-0.308
Pah. Ammos	2.827	-11.001	-8.041	0.378	-3.871	-4.742	2.449	-7.130	-3.299
Gergeri	7.143	-6.999	-6.270	0.781	-5.631	-6.200	6.362	-1.368	-0.071
Armaha	-1.239	-6.078	-4.345	0.173	-3.147	-4.233	-1.412	-2.930	-0.112
Asimi	-3.831	1.222	3.770	-2.077	1.834	3.451	-1.754	-0.612	0.319
Kasteli	-0.856	-5.561	-2.734	-1.082	-3.342	-2.845	0.226	-2.219	0.111
Ag. Barbara	-2.467	6.361	7.406	-0.299	5.420	6.866	-2.168	0.941	0.540
Protoria	-2.305	4.065	5.906	-0.904	5.255	5.533	-1.402	-1.190	0.373
Levkogia	12.190	6.055	4.991	1.380	5.330	4.750	10.811	0.725	0.241
Metaxohori	4.437	-9.983	-6.579	0.026	-6.738	-6.849	4.411	-3.245	0.270

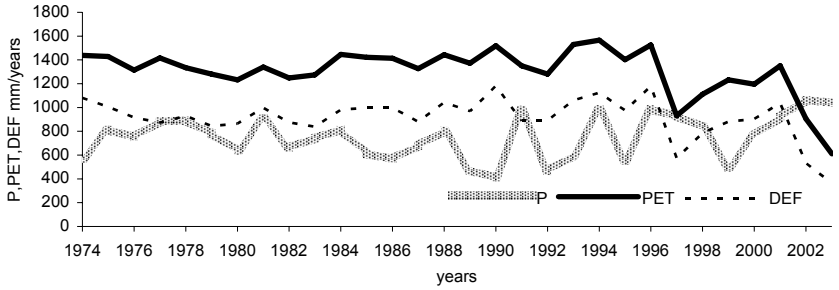


Figure 1. Yearly precipitation (P), potential evaportranspiration (PET) and soil moisture deficit (DEF), as a function of years.

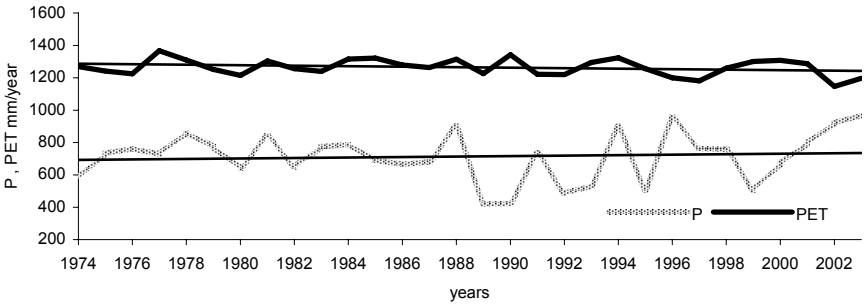


Figure 2. Averaged (over the 11 sites) values of precipitation and pan evaporation.

3. CONCLUSIONS

The trends in precipitation and pan evaporation are very similar to those previously reported for several countries (Roderick and Farquhar, 2005). In particular, there is variation in the trends from pan to pan, but overall, there are very few sites showing statistically significant changes in precipitation, while 6 sites show statistically significant declines in pan evaporation. There are sites showing statistically significant increases in pan evaporation. Precipitation and soil moisture deficit do not seem significantly increasing or decreasing in almost all stations.

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Drought Indices over Crete

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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. Quantitative drought definitions identifying the beginning, end, spatial extent and severity of a drought, formulated in terms of drought indices integrate various meteorological and hydrological parameters. One of the most widely used drought indices, the Standardized Precipitation Index (SPI), is based only on rainfall data and is accepted as a good measure of short- and long-term meteorological drought, for a monthly temporal resolution. In this study, SPI is calculated on 1-, 2-, or 3-months scale (SPI1, SPI2, SPI3, respectively) in 11 stations in Crete, for the period 1974-2001. These SPIs are compared with two other indices, the evapotranspiration deficit index (EDI) and the water balance drought index (WBI). Both indices based on daily evapotranspiration and rainfall data and a daily single layer water balance model, are selected as estimating droughts in a finer temporal (and spatial) resolution. EDI and WBI are only in reasonable agreement with the mentioned SPIs, since they represent, apart from precipitation, the additional effect of potential evapotranspiration in droughts.

Keywords: Drought analysis, evapotranspiration deficit index, standardized precipitation index, water balance drought index.

1. INTRODUCTION

Drought, one of the major natural hazards, is most often caused by a departure of precipitation from the normal amount. 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. The new distributed hydrologic models are used to simulate soil moisture and evapotranspiration from daily weather data at a high spatial resolution. An increased temporal and spatial

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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. Soil moisture and crop water stress estimates are especially valuable for estimating the consequences of climatic variability on crop growth and production

Many drought indices have been developed to monitor and define agricultural drought and the standardized precipitation indices SPI_1, SPI_2 and SPI_3 very easily handled, (as only precipitation data are required for their computation) have been successfully used for agricultural droughts (Tsakiris and Vangelis, 2004; Sonmez *et al.,* 2005; Livada and Assimakopoulos, 2007; Narasimhan *et al.,* 2005). The evapotranspiration deficit index (EDI) proposed by Narasimhan *et al.* (2005) and the water balance drought index (WBI) (McCaskil and Kariada, 1992; Papaioannou *et al.*, 2004) have been reported, as achieving finer resolutions of agricultural droughts, since they are based on daily and more physical parameters (rainfall, potential or actual evapotranspiration). In this study, daily rainfall and pan evaporation measurements are used for detecting agricultural droughts in Crete. The standardized precipitation indices SPI_1, SPI_2 and SPI_3 are calculated, for each station, (from monthly precipitation data) and are compared to EDI and WBI drought indices. Both last indices based on daily evapotranspiration and rainfall data and a daily single layer water balance model, are selected as estimating droughts in a finer temporal (and spatial) resolution. The agreement between EDI and WBI and the mentioned SPIs, is examined in order to evaluate their capability in detecting droughts.

2. ANALYSIS AND RESULTS

Daily rainfall and pan evaporation data, covering the period (1974-2001), obtained from the "Institute of Land Reclamation of Crete" for 11 sites in Crete (presented in Table 1) are utilized in a single-layer water balance model. This model calculates actual evapotranspiration (AET), available soil moisture (AW), the daily evapotranspiration deficit index (EDI) as the ratio (PET-AET)/PET and the water balance drought index (WBI) as the percentage of days in each month when AET/PET is less than 0.3. The value 0.3 is selected, according to new soil-moisture classifications (Kamara and Jackson, 1997). The SPI can track drought on multiple timescales and for the present study SPI is calculated for each month, at 1- month, 2-month and 3-month timescales, reflecting short-length drought events. The corresponding indices SPI_1, SPI_2 and SPI_3 are calculated according to Guttman (1999), to give a representation of abnormal dryness over Crete. Table 1 presents the average yearly values of the estimated indices for the period 1974-2001 and Figure 1 shows their monthly values, over a specific time period (1990-2001), for two selected stations. It is obvious that WBI and EDI indices are in great agreement and this is also observed between the SPIs. The regression analysis between the values of all indices for the period 1974-2001, indicates that WBI and EDI indices are highly correlated (correlation coefficients R are of the order of 0.98) for all stations. SPI_1 is in a good agreement with SPI_2 and SPI_3 and the best correlation is between SPI_2 and SPI_3 with R of the order of 0.79. Linear regressions between SPI_1 and SPI_2 or SPI_1 and SPI_3 present correlation coefficients of the order of 0.71 or 0.55, respectively. An

important correlation between WBI (or EDI) with any of the three SPI indices is not apparent in all stations. These findings are also evident from the curves in Figure 1.

Table 1. Average yearly values of precipitation (P), potential evapotranspiration (PET), water balance drought index (WBI), evapotranspiration deficit index (EDI) and SPI indices, for all stations.

Station	P (mm/year)	PET (mm/year)	WBI	EDI	SPI1	SPI2	SPI3
Protoria	548.69	1292.96	0.45	0.48	0.258	0.130	0.069
Asimi	606.22	**1091.38**	0.39	0.42	0.271	0.203	0.094
Gergeri	904.34	1423.55	0.41	0.44	0.236	0.172	0.085
Agia Barbara	969.53	1126.43	0.38	0.39	0.299	0.154	0.061
Metaxohori	753.75	1319.34	0.42	0.45	0.173	0.221	0.110
Armaha	812.69	1149.64	**0.38**	**0.41**	0.287	0.152	0.075
Kasteli	738.66	1076.88	0.39	0.42	0.256	0.116	0.046
Mithi	567.36	1095.51	0.41	0.45	0.256	0.205	0.103
Pahia Ammos	587.13	1387.29	**0.47**	**0.50**	0.339	0.181	0.094
Kalo Horio	**539.69**	1213.19	0.44	0.48	0.238	0.190	0.092
Levkogia	839.91	**1608.73**	0.45	0.47	0.158	0.227	0.103

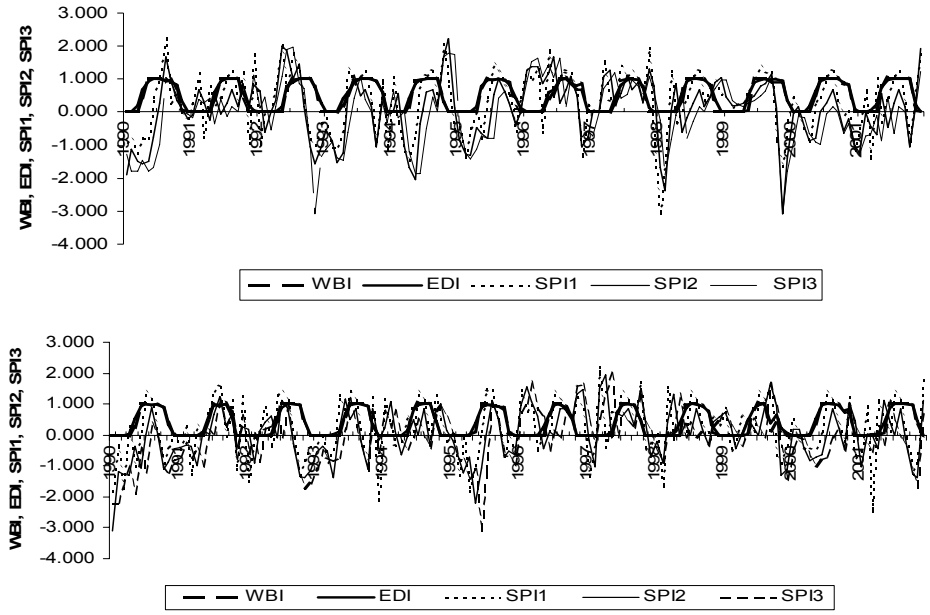


Figure 1. Time series of WBI, EDI, SPI_1, SPI_2,,SPI_3 indices over the period 1990-2001 for two selected stations: Pahia Ammos (up) and Agia Barbara (down).

3. CONCLUSIONS

This study shows that the evapotranspiration deficit index (EDI) and the water balance index (WBI) are only in reasonable agreement with SPI_1, SPI_2 and SPI_3. The

time-appearance of any drought period, as indicated by EDI and WBI is not coincident with the ones presented from the time evolution of any of SPIs. The shortage of rainfall is obvious rather in advance in SPIs, as compared with the results of water balance estimations presented in both EDI and WBI indices time series. The differences are mainly attributed to the additional effect of potential evapotranspiration, (apart from precipitation), and this is in accordance with the drought results reported by Tsakiris *et al.* (2006) from a similar standardised index.

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The authors wish to express their thanks, to the Institute of Land Reclamation of Crete, for providing the data. This work is part of the EU-funded project "DESURVEY: A surveillance system for assessing and monitoring of desertification".

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Modeling of Groundwater Flow and Transport of Nitrates in the Area of Malia, Crete (Greece)

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Abstract: The pollution of groundwater by nitrates constitutes a very crucial problem because the damage caused by the presence of high concentration nitrates in water can affect public health as well as the environment. The major cause for this pollution is primarily farming activities and especially the use of nitrogen based fertilizers. The paper presents a study of groundwater nitrate pollution of the greater area of the municipality of Malia located about 35 Km west to Heraklion City. At first a description of the area at study is given followed by the modeling of groundwater flow and nitrate transfer using the Princeton Transport Code (PTC) model in combination with the Argus ONE model. Based on the results of the model certain conclusions are reached and specific actions are proposed for diminishing the nitrate pollution in the area of study.

Keywords: Groundwater pollution, mass transport, nitrates.

1. INTRODUCTION

The main sources of groundwater pollution with N_2 are biological fixation, ammonization, nitrification and denitrification (Figure 1). Nitrogen and nitrates primarily originate from natural and human sources that are: fertilizers, stock-raising activities, rain water, industrial liquid wastes, urban liquid wastes, septic tanks and private sewage systems and decomposing plants.

The most harmful pollutants for the human health are the nitric ions that are easily transferred by the water via the unsaturated zone and the groundwater flow into the groundwater aquifers. Irrigation in combination with the application of inorganic nitrogen fertilizers contribute to the fast rising of nitrate concentration in many farming areas. This rise can also be seen in non-irrigated areas having organic soils. In this case nitrates are released during the decomposition of animal and plant

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residuals that are incorporated in the ground.

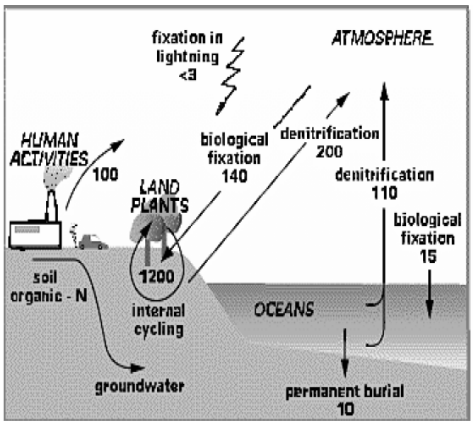


Figure 1. Routs of nitrogen.

Nitrates that originate from nitrogen fertilizers, and from other sources (humus, manure) are very agile inside the ground. They are easily soluble in water, as they are not restrained by the ground –as other nutrients- they move towards the deeper ground layers and end up into the groundwater where they accumulate. When the nitrate concentration exceeds certain limits water becomes not potable. On the other hand in the case that the ground has a low infiltration factor and some slope, nitrates are transferred to the surface water affecting their quality and causing eutrophication.

According to EC Directives potable water must not contain more than 50 mg/l or (50ppm) nitrates or else it is considered unsuitable. The quantity of nitric nitrogen that moves towards the ground or surface water depends on rainfall, irrigation, ground slope, mechanical composition, infiltration, plant coverage, as well as the cultivation management system (Ministry of Agriculture 2000).

2. HYDROLOGICAL BASIN OF MALIA

The ground water nitrate contamination has been studied for the expended area of the municipality of Malia, located 35 km east to Heraklion City (Figure 2). In the hydrological basin of Malia, the groundwater aquifers are characterized by neogenic stratifications; weathering deposits of proneogenic carbonate stratifications and proneogenic carbonate stratifications. In this area there are several pumping wells (Figure 2, pumping wells are indicated by the white circles) with an annual pumping activity that is estimated to be $10x10^6$ m³ of water, out of which $4x10^6$ m³ is transferred to Heraklion City for watering (Polihronaki and Kathinaki 1989).



Figure 2. Pumping well network.

3. NUMERICAL SIMULATOR AND THE "ARGUS ONE" MODEL

Argus ONE is a model-independent Geographical Information System (GIS) for numerical modeling. As with other GIS systems, the various types of geospatial information are stored and viewed in coverages or "layers" which the user may view and interact with on screen. Argus ONE provides an easy, graphical way to generate finite-element meshes or finite-difference grids. It supports a set of utilities that allows importing digitized maps, extracting domain outlines from the imported maps and automatically generating grids or meshes on the domains (Pinder 2002, Olivares 2001).

The Princeton Transport Code (PTC) is a three-dimensional groundwater flow and contaminant transport simulator which can use both finite-element and finite-difference discretization. It can create and process up to 2000 elements. PTC is written in FORTRAN 77, thus can be easily applied in combination with Argus ONE, in a user-friendly Windows environment (Babu *et al.,* 1997). PTC uses the following system of partial differential equations to represent groundwater flow described by hydraulic head h,

$$\frac{\partial}{\partial x}\left(K_{xx}\frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(K_{yy}\frac{\partial h}{\partial y}\right)+\frac{\partial}{\partial z}\left(K_{zz}\frac{\partial h}{\partial z}\right)-S\frac{\partial h}{\partial t}+Q=0 \qquad (1)$$

the groundwater velocity components,

$$V_x=-K_{xx}\frac{\partial h}{\partial x} \qquad V_y=-K_{yy}\frac{\partial h}{\partial y} \qquad V_z=-K_{zz}\frac{\partial h}{\partial z} \qquad (2)$$

and contaminant transport described by concentration C,

$$\begin{aligned}&\frac{\partial}{\partial x}\left[D_{xx}\frac{\partial C}{\partial x}+D_{xy}\frac{\partial C}{\partial y}+D_{xz}\frac{\partial C}{\partial z}\right]+\frac{\partial}{\partial y}\left[D_{yx}\frac{\partial C}{\partial x}+D_{yy}\frac{\partial C}{\partial y}+D_{yz}\frac{\partial C}{\partial z}\right]+\\&\frac{\partial}{\partial z}\left[D_{zx}\frac{\partial C}{\partial x}+D_{zy}\frac{\partial C}{\partial y}+D_{zz}\frac{\partial C}{\partial z}\right]-\left[V_x\frac{\partial C}{\partial x}+V_y\frac{\partial C}{\partial y}+V_z\frac{\partial C}{\partial z}\right]+\\&Q\left(C^{\omega}-C\right)-\theta[1+E(C)]\left(\frac{\partial C}{\partial t}\right)=0\end{aligned} \qquad (3)$$

where h is the hydraulic head [L], K_{xx}, K_{yy}, K_{zz} are the hydraulic conductivities in the x, y and z directions [LT^{-1}], S is the specific storage coefficient [L^{-1}], Q is the source/sink term [L^3T^{-1}], V_x, V_y, V_z, are Darcy's velocities in directions x, y and z [LT^1], C is the contaminant concentration [M/L^3], θ is the porosity of the aquifer, C^{ω} is the concentration of the pumped fluid, [M/L^3], $E(C)$ is the function representing the chemical absorption properties and D_{ij}, with $i, j = x, y, z$, are the dispersion terms in several directions (Babu and Pinder, 1984).

The imported data are the following: Molecular diffusion, Arial Maps, Geological Data, Drilling Supply (in m^3/day), Conductivity, Elevations, Porosity, Adsorption, Rain, Initial Heads, Initial concentrations (in ppm). Due to seasonal differences the time-depended data of the simulation were considered for two periods, which cover the summer and winter seasons.

4. RESULTS

Two different stress periods were applied in each year: summer and winter. High pumping rates and nitrate loads were applied for the first stress period, reflecting the high water consumption and intensive agricultural activities during the summer season, while lower pumping rates and nitrate loads were considered for the second period. Results for the hydraulic heads and nitrate concentrations are presented in Figures 3, 4, 5 and 6, respectively.

For the summer period the hydraulic heads varied from 63 to 67 m as shown in Figure 3 and the nitrate concentrations from 5 to 525 ppm as shown in Figure 4.

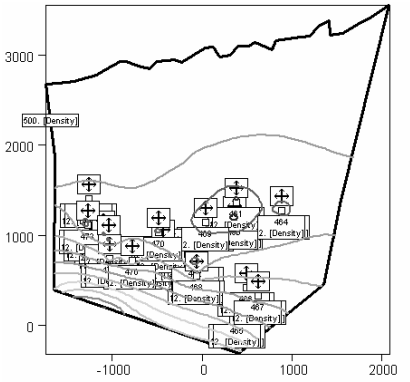


Figure 3. Summer heads for the summer period.

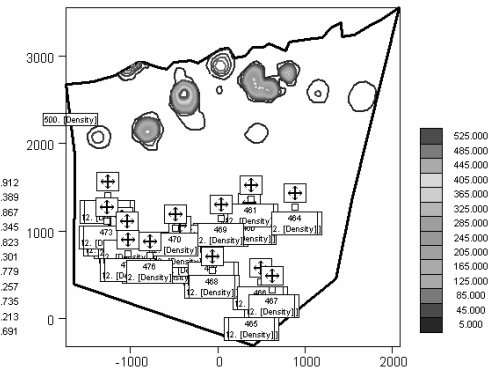


Figure 4. Nitrate summer concentrations.

For the winter, the hydraulic heads varied from 65 to 76 m as shown in Figure 5. The nitrate concentrations varied from 5 to 53 ppm as shown in Figure 6.

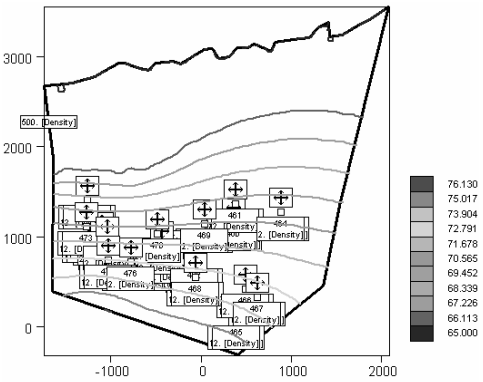


Figure 5. Winter heads for the winter period.

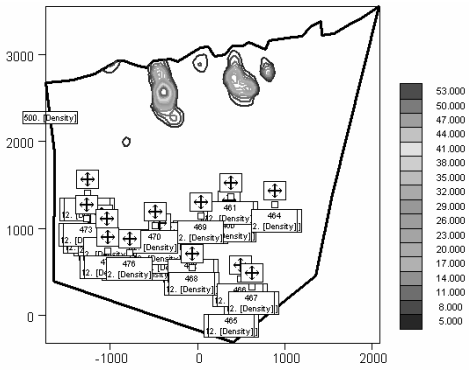


Figure 6. Nitrate winter concentrations.

5. CONCLUSIONS

By comparing the hydraulic heads during the summer and winter period respectively, the following are observed: During the summer period there is a sub-saline head, which extends up to numerous drillings located at the northern part of the area of study. The free surface of the aquifer is lower than the sea surface, which results to the seawater intrusion through the hydro-penetrable carbonate stratifications and penetrates into the underground aquifer as well as the impropriety of the drillings for irrigation and watering. It is observed that during the winter period (when different conditions exist, like the rainfall and the reduction of the drilling pump) there is no indication of a sub-saline phenomenon. Even though the area of study contains significant quantities of groundwater, the area is extremely vulnerable since there is a bad combination of seasonal increase in the population (which implies over-pumping of the aquifers), which coincide with a cultivating period (which implies the use of fertilizers). Comparison of the nitrate concentration during summer and winter period, the values of nitrate concentration during the winter period vary between 5 and 53 ppm, which means that they are set bellow the ultimate limit, suggested by the European Community for the drinking water standards. During the summer period the concentration values increase rapidly up to 525 ppm, which makes the groundwater to be dangerous for watering and irrigation.

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Investigating Spatial and Temporal Dependencies using Crete Rainfall Data

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Abstract: A spatial time-series framework is adopted for modelling the spatiotemporal distribution of daily rainfall data over the area of the island of Crete in Greece. Using various diagnostic tools, the existing spatial and temporal structures are identified within the island's regions. Inference is focused on a spatial periodic-trend model. For large data sets, the estimation of the main diagnostic tool such as the likelihood function, is a cumbersome task. Here, using the Innovation Algorithm, we manage to compute it for all spatio-temporal specifications. Additionally, using non-likelihood based criteria, like the autocorrelation and covariance functions, we select one among various model specifications. Finally, via kriging we regionalise the resulting parameter estimates of the spatial periodic-trend model in space at any un-monitored location.

Keywords: Gaussian maximum likelihood estimator, innovation algorithm, Kriging, rainfall data, spatio-temporal modelling.

1. INTRODUCTION

Rainfall data analysis has long been of great significance due to its effects on ecological, environmental and socio-economic policy issues. Because of rainfall level impact on many issues associated with water intake, it is of great importance to study the level of water accumulation in certain areas of interest. The data sets used in this paper consist of daily rainfall values from 10 different locations in the island of Crete, Greece, in the period ranging from 1.1.1991 to 31.12.1997. The locations used are the rainfall monitoring stations of Souda, Rethymnon, Iraklion, Sitia, Ierapetra, Timbaki, Anogia, Gortina, Kasteli, and Fourni.

Temporal dynamics and spatial correlations are modelled simultaneously using daily rainfall data series in the island of Crete. Our basic aim is to propose a

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spatio-temporal model that can best fit Cretan rainfall data in space and time. First, the existing spatial and temporal structure is investigated based on an empirical analysis (Sections 2 and 3). This can help recommending some models that can possibly fit our data. Second, a spatio-temporal model is introduced together with tools for statistical inference in a Gaussian framework. A spatial periodic-trend process is indtroduced. We focus on parametric estimation and model selection by calculating the likelihood function (Section 4). This is implemented adopting the computationally efficient "Innovation Algorithm" based on pre-whitening (Brockwell and Davis, 1991; Yao and Brockwell, 2001). Finally, Section 6 is devoted to Kriging, where the resulting parameter estimates of the best-fitted model are regionalised in space at any un-monitored location.

2. ANALYSIS OF TEMPORAL STRUCTURE

This section investigates the temporal structure of the data sets. This is done by presenting the data time series of each station, independent of spatial information. For this purpose we use the autocorrelation function (ACF) since it is a good measure of temporal correlation (Handcock and Wallis, 1994). Figure 1 presents selectively the empirical autocorrelation functions from the Souda and Rethymnon stations. It is well documented that certain locations are characterised by strong temporal dependence such as Souda, and Ierapetra, whereas some others like Rethymnon, Iraklion, and Sitia by a relatively lighter one. Studying the empirical autocorrelation functions derived from daily rainfall observations of the period 1990-1997, it is observed the high level of autocorrelation plus the cyclical type of behaviour as the correlation dies off at all stations.

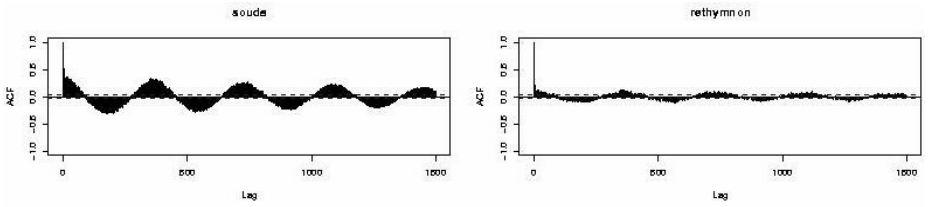


Figure 1. Empirical autocorrelation functions from Crete rainfall data in stations: Souda and Rethymnon.

3. ANALYSIS OF SPATIAL STRUCTURE

Our first attempt is to derive some measures of data spatial dependency for the whole time domain of any two spatial locations. One of these empirical measures of spatial dependencies is the covariance function. Suppose spatial processes $\{\boldsymbol{Y}(\mathbf{s})\}$ and $\{\boldsymbol{Y}(\mathbf{r})\}$ indexed by a two dimensional location $\mathbf{s},\mathbf{r} \in S \subset \Re^2$. Their empirical

covariance function can be expressed with (Cressie, 1993).

$$\mathrm{Cov}(Y(\mathbf{s}),Y(\mathbf{r})) = \frac{1}{T}\sum_{t=1}^{T}(Y_t(\mathbf{s}) - \bar{Y}(\mathbf{s}))(Y_t(\mathbf{r}) - \bar{Y}(\mathbf{r})), \quad \mathbf{s},\mathbf{r} \in \mathcal{S}. \tag{1}$$

where

$$\bar{Y}(\mathbf{s}) = 1/T\sum_{t=1}^{T} Y_t(\mathbf{s}), \bar{Y}(\mathbf{r}) = 1/T\sum_{t=1}^{T} Y_t(\mathbf{r})$$

represent the mean location values. This measure of spatial dependency is usually expressed by various theoretical covariance functions such as the Exponential, the Gaussian, the Wave or the Matérn class (Cressie, 1993). As we can see from Figure **2**, there is a systematic relationship between locations in the North using the spatial correlation function formula 2.

$$Corr(Y(\mathbf{s}),Y(\mathbf{r})) = \mathrm{Cov}(Y(\mathbf{s}),Y(\mathbf{r}))/[Var(Y(\mathbf{s}))]^{1/2}[Var(Y(\mathbf{r}))]^{1/2} \tag{2}$$

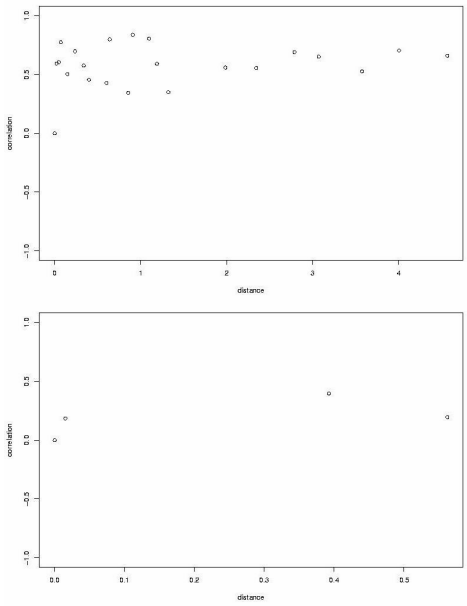


Figure 2. Empirical correlations for the North (up) and South (down) regions of Crete.

4. SPATIO-TEMPORAL PERIODIC PROCESS

This section, proposes a class of models that can be viewed as candidates for fitting our rainfall data. This is the periodic-trend which is selected due to the periodic nature of our data series (see Sections 2 and 3). Here we propose a class of spatio-temporal models to fit with spatial time-series data located at irregular space. More specifically, suppose that $\{Y_t(\mathbf{s}),\ t=0, \pm1, \pm2, \cdots\}$ represent a periodic-trend model at location

$\mathbf{s} \in S \subset \Re^2$ (Kyriakidis and Journel, 2001).

$$Y_t(\mathbf{s}) = a_{0,\mathbf{s}} + a_{1,\mathbf{s}} \cos(\frac{2\pi}{PER}t) + a_{2,\mathbf{s}} \sin(\frac{2\pi}{PER}t) + \sigma_{\mathbf{s}}\epsilon_t(\mathbf{s}) \quad (3)$$

Each fixed $\mathbf{s} \in S$ $\{\varepsilon_t(\mathbf{s}),\ t=0,\ \pm1,\ \pm2,\ \cdots\}$ is an i.i.d. sequence with mean 0 and variance 1. The $\{a_{i,s}\}_{i=0,1,2}$ coefficients represent the intercept and the seasonal coefficients and π stands for the period. For the coefficients' interpretation, we need to clarify that the $a_{1,s}$ and $a_{2,s}$ are linked to the amplitude a(s) and the phase $\phi(\mathbf{s})$ of the periodic component in our temporal specification (Kyriakidis and Journel, 2001) as:

$$\alpha(\mathbf{s}) = \sqrt{a_{1,\mathbf{s}}^2 + a_{2,\mathbf{s}}^2} \quad (4)$$

$$\phi(\mathbf{s}) = \tan^{-1}(\frac{a_{2,\mathbf{s}}}{a_{1,\mathbf{s}}}) \quad (5)$$

5. THE MODEL

Let $\{Y_t(\mathbf{s}_j),\ t = 1 - p,\ 2 - p,\ \cdots,\ T;\ j = 1,\ \cdots,\ N\}$ be observations from our process and $\{Y_t(s_j),\ t = 1-(p+P),\ 2-(p+P),\ \cdots,\ T;\ j = 1,\ \cdots,\ N\}$ from our process. We introduce some notation first. Let $\mathbf{X}(\mathbf{s})$ be the T x p matrix with $Y_{j\text{-}i}(\mathbf{s})$ as the (i, j)-th element, and $\mathbf{X}$=diag$\{\mathbf{X}(\mathbf{s}_1),\ \cdots,\ \mathbf{X}(\mathbf{s}_N)\}$ a (T x N) x (p x N) matrix for model 3. Here, the quantity p is replaced by 3. Let $\mathbf{R}(\boldsymbol{\theta})$ be an N x N matrix with $\rho(s_i,s_j;\boldsymbol{\theta})$ as the (i, j)-th element, $\boldsymbol{\Lambda}$=diag$(\sigma_{s1},\ \cdots,\ \sigma_{sN})$ and $\boldsymbol{\Sigma}=\boldsymbol{\Lambda}\mathbf{R}(\boldsymbol{\theta})\boldsymbol{\Lambda}$. The actual choice among these covariance functions takes part at the MLE stage. We define, $\mathbf{Y} = (\mathbf{Y}(\mathbf{s}_1)\ \cdots\ \mathbf{Y}(\mathbf{s}_N))'$, $\mathbf{Y}$=$(Y_1(\mathbf{s})\ \cdots\ Y_1(\mathbf{s}_N))'$, $\mathbf{b}$=$(\mathbf{b}(\mathbf{s}_1)\ \cdots\ \mathbf{b}(\mathbf{s}_N))'$ and $\mathbf{b}(\mathbf{s})$=$(b_{1,s}\ \cdots\ b_{p,s})'$. Now the conditional Gaussian likelihood function $\{Y_t(s_j),\ t\leq 0;\ j = 1,\ \cdots,\ N\}$ can be written as

$$\frac{|\mathrm{R}(\theta)|^{-\frac{T}{2}}}{\{\prod_{j=1}^{N}\sigma_{s_j}\}^T}\exp\{-\frac{1}{2}(\mathbf{Y}-\mathbf{X}\mathrm{b})^{\tau}(\boldsymbol{\Sigma}^{-1}\otimes\mathbf{I}_T)(\mathbf{Y}-\mathbf{X}\mathrm{b})\} \quad (6)$$

Where $\otimes$ denotes the matrix Kronecker product and $\mathbf{I}_T$ the T x T identity matrix. By maximising the above function, we obtain the Gaussian maximum likelihood estimators: $\hat{\mathrm{b}}$, $\hat{\theta}$ and $\hat{\sigma}_{sj}$, which should be refered to as quasi-maximum likelihood estimators. Here $\hat{\mathrm{b}}$ and $\hat{\sigma}_{sj}$ are estimated using LS-methods to location by location and $\hat{\theta}$ by minimising:

$$\ell(\hat{\theta},\hat{\mathrm{b}},\hat{\sigma}_{s_j};\mathbf{Y}) = \log\{\frac{1}{N}\sum_{j=1}^{N}(Y_j-\hat{Y}_j)^2/\nu_{j-1}\} + \frac{1}{N}\sum_{j=1}^{N}\log\nu_{j-1} \quad (7)$$

We refer to this as the reduced likelihood function, with Y_j representing the observed process. Here v_j=Var$(Y_{j+1}$-$\hat{Y}_{j+1})$, denotes the estimation error variance and $\hat{Y}_j$ the estimate in the Innovation Algorithm (Brockwell and Davis, 1991; Yao and Brockwell, 2001). Table 1 presents the results for the PER values between 7 and 370.

For PER=360 we get the minimum reduced likelihood value, i.e -115.2773. For values out of this range, we observe log -likelihood values much away from its minimum.

Table 1. Reduced log-likelihood estimators for various periodic model specifications using daily rainfall data.

	Periodic models				
Estimator log-likelihood	PER=7	PER=30	PER=90	PER=300	PER=310
	-47.2243	-48.2483	-47.5479	-48.3505	-51.9228
	PER=330	PER=340	PER=350	PER=360	PER=370
	-49.7133	-61.3627	-95.9798	-115.2773	-77.4724

6. KRIGING

This section is dedicated to spatial estimation at any un-monitored location in the island of Crete. This is performed using Ordinary Kriging (Kyriakidis and Journel, 2001). Kriging, is an interpolation technique that aims to estimate any unobserved spatial location using the observed location estimates. Figure 3 present the Kriging estimates of the Periodic model PER(360) for the cosine, and the sine coefficients respectively. What is well documented especially for the sine temporal coefficient is that the Timbaki, Kasteli and Gortina locations are characterised by similar patterns.

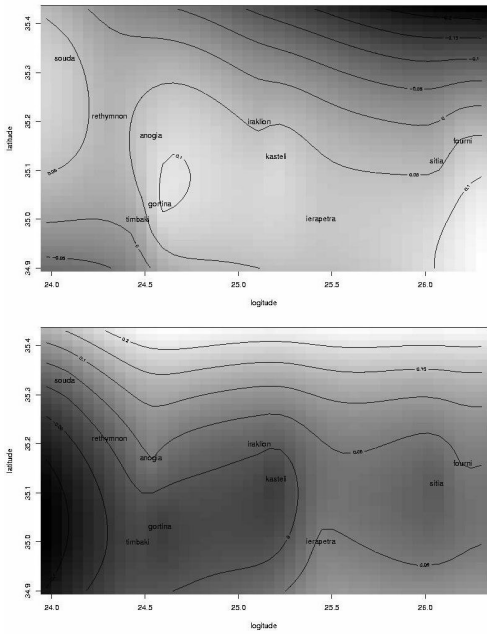


Figure 3. Krigin estimates of the periodic model PER360: cosine coefficient (up), sine coefficient (down).

7. CONCLUSIONS

With this work we have addressed the problem of rainfall modelling over the island of Crete in Greece. After investigating the temporal and spatial structure of the data series, we proposed three alternative spatial-temporal processes. Using LS and MLE, we defined the parametric estimates as well as those of the reduced likelihood function of the periodic spatial-temporal process specifications. From these inference results, we derived the fitted residuals spatial covariance functions and ACF. The results showed that most of the existing spatial and temporal dependency were best captured by the PER(360) model.

Also a Kriging estimation was performed using the PER(360), parametric estimates of a large map over Crete. The findings, either pointed or regional showed strong spatial and mostly temporal dependencies. The use of these models can further contribute to a better forecasting of the rainfall level and also to increase the accuracy of environmental studies in parts of the island not currently covered by monitoring stations.

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Technical Peculiarities and Laying Experiences with Polyethylene Corrugated Pipes for Sewer System

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Abstract: This paper deals with the technical peculiarities and laying experiences with polyethylene corrugated pipes for sewer systems. It is discussed the use and installation of the pipes with reference to piling up, bedding and trench backfill. Major advantages of the corrugated pipes are provided, compared to concrete and PVC pipes.

Keywords: EN 13476-1 standard, polyethylene corrugated pipes, thermoplastic piping system.

1. INTRODUCTION

The corrugated pipe was introduced on the global market (USA and North Europe) about 40 years ago. In 1992, Polieco was the first Italian company to produce high density polyethylene corrugated twin-wall pipes for the construction of conduit and drainage systems. Production started with a diameters range from OD.40 to OD.200 mm. In 1995, the know how developed in the course of time was extended to the twin-wall pipes for civil and industrial sewer systems. These pipes, sold with the trademark `Ecopal`, are available in a diameters range from OD.125 to OD.1200 mm and from ID.300 to ID.800. The present production is 1,6 million meters for the `Ecopal` pipes, for a total amount of 10,800 tons of processed HDPE per year. 35% of the pipes are sold abroad (Est and South-East Europe).

Nowadays, POLIECO produce `Ecopal` with three extrusion lines, able to produce up to 3,000 meters a day (depending on diameter), working 24-7 (whole day+whole week).

The production of polyethylene corrugated pipe is made according to European standard EN 13476–1 referred to pipe systems of plastic material (PE, PVC, PP) with structured wall.

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2. CHARACTERISTICS OF CORRUGATED PIPE

Success of corrugated pipes in HDPE comes from the technical and economical advantages assured by them. The main technical characteristics of a HDPE corrugated pipe are:

a. It is made with high density polyethylene, that is a polymer which offers a high impact resistance even at low temperatures as well as an excellent chemical resistance. Comparing to other materials used for pipes production, high density polyethylene presents a better abrasion resistance;
b. Its peculiar geometric structure guarantees a high deflection resistance and is produced in two ring stiffness classes (Class 4 kN/m² - Class 8 kN/m²). By this way the pipe can fulfil different applications even in deep underground;
c. Its outer black wall guarantees a high UV stability, so it can be stored outdoors for a long period without any significant changes in its physical and mechanical characteristics. `Ecopal` inner wall is white to ease the pipe video-inspection;
d. It is produced in 6 and 12 m bars; by this way the number of joints is reduced. The corrugated pipe is generally connected with coupling and elastomeric seals. The elastomeric seal is placed inside the corrugation, thus avoiding the risk of being displaced during the coupling operation. The elastomeric seal of a particular profile avoids not only leakage, but also the groundwater entrance into the pipe (Figure 1);

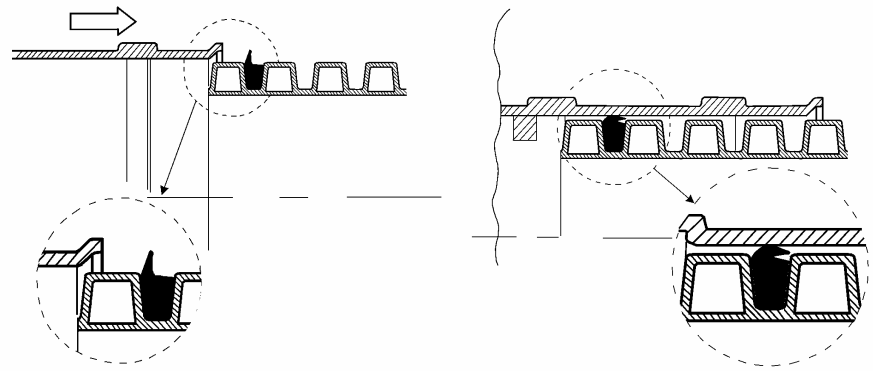


Figure 1. Connections of corrugated pipes.

e. It is extremely versatile, due to the possibility to have a wide assortment of fabricated special fittings and can be connected with any other type of existing pipes (Figure 2);
f. Thanks to its lightness, it is easy to handle, to store, to transport, to install and does not need any particular equipment to be moved. The lightness, the length of bars and the modality of the jointing of plastic piping, ease the handling and the laying on the building place of the corrugated piping. That means quicker works' execution; less danger for the worker and less troubles for people and economic activities.

g. It is partially flexible and can avoid obstacle during pipe-lining and digging inaccuracy.



Figure 2. View of corrugated pipes.

3. REFERENCE STANDARD

Reference standards for Ecopal® are contained in EN 13476–1 (Thermoplastic piping systems for non-pressure underground drainage and sewerage - Structured-wall piping systems of unplasticise (polyvinyl chloride, PVC-U), polypropylene (PP) and polyethylene (PE) - part 1: Specification and Requirements for pipes, fittings and the system). The EN 13476–1 standard provides for three groups of prescriptions for structured-wall pipes (Figure 3):

- Mechanical characteristics
- Physical characteristics
- Functional characteristics

Equivalent prescriptions are given for fittings and accessories.

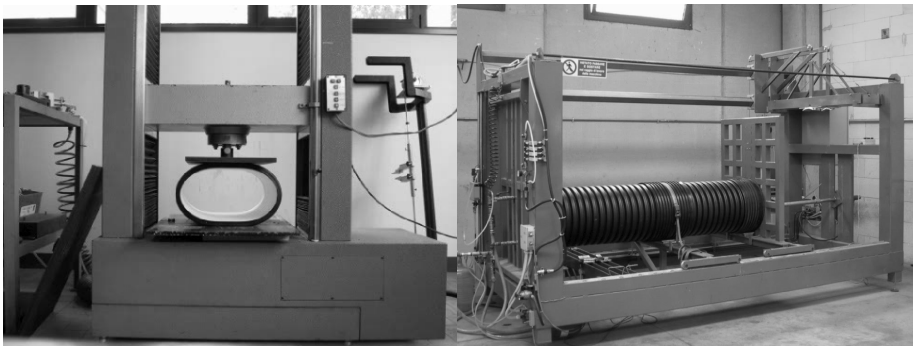


Figure 3. Testing corrugated pipes.

According to the EN 13476 standard the tightness of the jointing system (see photo at the right) certified at 0.5 bar under pressure and 0,3 bar under negative pressure at the temperature of 23°C for 15 minutes. These conditions are guaranteed

with an angular deflection (variable, according to diameter, from 2° to 3°) or a ring deflection (10% of the pipe and 5% of the coupler). Regarding surface characteristics of the pipes, the standard prescribes that:

- to a visual examination the internal and external surfaces must be smooth, clean and free from scratches, blisters or other surface irregularities;
- the material must not include visible impurities or pores;
- the pipe ends must be cleanly cut perpendicularly to the axis.

4. USE AND INSTALLATION

Ecopal® is transported under the same conditions as any other type of pipe. All pipes, joints and special fittings must be delivered with marks or labels indicating the manufacturer's name, the nominal diameter and their use. Loading, transportation, unloading and all connected operations must be carried out very carefully using suitable means according to the pipe type and diameter and using all the measures required to avoid failures, cracks, damages or else. Avoid all impacts or deflections, slipping, contacts with components which could damage the pipes.

4.1. Piling Up

Pipes must be piled up on a flat, stable and sheltered surface so as to avoid any risks of fire; they must be protected against the sun which could cause thermal variations. The base of the piles must rest on well spaced-out boards or a supporting surface. The stack height depends on the diameters so as to avoid deformations at the base of the pipes and allow an easy collection. The piled-up pipes must be fixed by wedges to avoid sudden rolling. In all cases, it is suggested adopting protection measures to avoid damages to the pipe ends.

4.2. Trenching

The best advice that can reasonably given is to prescribe a narrow trench in the specifications, maximum 2-3 times the diameter, at least up to 1 m above the top of the pipe. The walls must be vertical as much as possible, at least in that area, and eventually stabilised with props or piles to protect the persons working on the trench. In the case of an embankment or wide trench, it is advisable to foresee a contrast to the covering material to remain as near as possible to narrow trench conditions. Piles must be removed just after the partial backfilling and before compaction.

4.3. Bedding

To lift and lay the pipes in the trench or on supports, we suggest following the same procedures as for the previous operations, always making sure that the

pipe surfaces are not damaged and using means suitable for the given diameters. When laying the pipes, make sure that no debris, foreign bodies or any other material get into the pipes and damage the external surface. If a bedding is required or if the first backfilling requires the use of materials other than the ones coming from the trench, it is suggested to check the presence of materials which could damage the pipe during laying. Anyway, it is forbidden to adjust the pipe position inside the trench using stones or bricks or other unsteady supports. The pipe surface must guarantee a stable support and where ground settling is foreseen special measures must be adopted, such as the use of suitable joints or special treatments for the trench bottom.

4.4. Trench Backfill

Filling up the trench is the most delicate and important aspect when installing a sewer pipe (Figure 4). We have seen how a backfill without adequate compaction has a negative effect both on rigid and flexible pipes. We have also seen that compacting made without the necessary precautions can cause breakage; video-inspection after the installation of concrete, vitrified clay and even PVC collectors showed them heavily damaged even before being put to use. Independently of the type of pipe, the trench backfill must be made with the correct compaction of successive layers, in compliance with prEN 1295.



Figure 4. Trench backfilling.

Concepts that must be observed for correct and long-lasting installation are:

1. **Choice of correct backfill material:** the material must be dry with a low granulometry, with no sharp edges, stones or debris at least in the part in contact with the pipe and up to at least 30 cm above it.
2. **Accurate compaction**: compaction must be made in successive layers of approximately 30 cm, with suitable equipment until reaching one metre of cover over the top of the pipe. A good compaction should reach 90–92 % Proctor. The first layer of side fill must exceed the half-diameter of the pipe to prevent its being lifted, otherwise it must be temporarily blocked during compacting operations. According to German standards, after covering with

good material for at least 1 meter above the top, the excavated material can be used to complete the backfill.

3. **Regular compaction**: irregular compaction must be avoided to prevent misalignments and therefore stress on the joints or anomalous curves in the pipe body.
4. **Compacting equipment**: up to 1 meter above the pipe, compacting must be made with light equipment, then normal means can be used above this height. Pay attention not to exaggerate by compacting with road construction vehicles without calculating the effect of the dynamic load on the pipe below.

5. COMPARISON WITH OTHER MATERIAL

The advantages of corrugated pipes in comparison with the most important other pipe materials are described as follows.

5.1. HDPE Corrugated pipe vs. Concrete Pipe

- higher resistance to abrasion;
- higher resistance to chemical agents;
- lower roughness coefficient, that means higher discharge with an equal diameter;
- lower weight, that makes easier the stockage, the handling and the laying on the building site: i.e. a DN/ID 800 concrete pipe has a weight of 800 kg/m while a DN/ID 800 Ecopal pipe has a weight of 40 kg/m);
- longer bars: 6 m or 12 m instead of 2 m
- more versatility, that permits to realize a wide range of special pieces and to joint its with any other existing type of pipe

5.2. HDPE Corrugated Pipe vs. PVC Pipe

- higher resistance to chemical agents;
- higher resistance to abrasion;
- lower fragility at the low temperatures, that makes easier the laying during the winter period;
- lower degradation: Ecopal pipe is guaranteed for a 12 month stockage period from the production date;
- available not only in 6 m bars, but also in 12 m
- better behaviour during earthquake or landslide movement thanks to lower rigidity

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