Optimization of management of polluted fractured aquifers using genetic algorithms

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Abstract: This paper deals with optimization of management of polluted fractured aquifers. A fictitious case has been investigated, where two circular pollution plumes may affect the water quality of two production wells, while two fractures, one almost perpendicular and one almost parallel to the flow lines, may accelerate pollutant transport. The optimization task is to find the location and the flow rate of two additional pumping wells that avert, retard or pump the plumes, achieving minimization of the sum of main cost items, namely pumping, pipe network and pumped polluted water remediation costs. Genetic algorithms (GA) are used as the optimization tool. To alleviate the computational load, a simplified two dimensional groundwater flow model is implemented and a moving point code is used to simulate advective mass transport. Moreover, we assume that fractures affect pollutant transport only. Such simplifications may raise questions regarding the accuracy of the overall results. For this reason we have adopted certain safety counter-measures, such as virtual lengthening of the fractures and instant spread of pollutants along them. The quality of the results is also influenced by the GA parameters, such as crossover and mutation probability. For this reason we have undertaken an extensive set of tests, to identify the best combination for our problem. Our results did not verify the practical rule of setting mutation probability equal to the inverse of the chromosome length, since the best results were achieved with substantially larger mutation probability values. We have also investigated the range of the coefficients of the penalty function, which has been used in order to guarantee observance of the problem constraints. Finally, we have stressed the importance of an a-posteriori check of the results of the optimization process, which might allow elimination of solutions with unpractical features.

Keywords: aquifer pollution; fractured aquifer; moving points; genetic algorithms; groundwater management

1. INTRODUCTION

Optimal management of groundwater resources is a challenging task, especially in areas of increasing demand (due to population and per capita water consumption growth) or of decreasing water availability, due to unfavorable climatic change or to human-induced pollution. A method to deal with the latter is hydrodynamic control of pollution plumes that threaten production wells. It can be achieved using additional pumping wells that may retard, avert or even pump polluted water.

Pump-and-treat technologies to achieve remediation of a contaminated aquifer may be not effective or efficient for every contaminant-aquifer combination. Provided that the contaminant is highly water-soluble or miscible and not strongly sorbed to soil surfaces, pumping the maximum amount of contaminated ground water from the zone of contamination in the aquifer can be considered a viable remediation option. There is significant research towards optimization of pumping schedules of existing wells for aquifer remediation (e.g. Liu, 2000), but also towards optimal remediation with pump-and-treat optimization with well locations and pumping rates selected as continuous decision variables, as in Guan and Aral (1999) and Huang and Mayer (1997). Existence of fractures in flow fields presents additional difficulties, since fractures may facilitate pollutants’ spread, aggravating the magnitude of pollution of wells that supply fresh water, or even causing their pollution otherwise avoided.

This paper deals with optimal development of polluted aquifers, bearing few fractures of known geometrical characteristics. The aim is to maintain water quality and flow rate of the existing production wells, using additional wells for hydrodynamic control of pollutant plumes. The
objective function of the respective optimization problem includes the main cost items, namely pumping cost, pipe network amortization cost and pumped polluted water treatment cost. Following Kontos et al (2010) each fracture is simulated as an one-dimensional high speed runway for water and pollutant particles, that does not affect hydraulic head distribution. Cases of low and high pollutant treatment cost are studied. Genetic algorithms (GA) are used as optimization tool. To alleviate the computational load, a two dimensional (2-D) groundwater flow is implemented combined with a moving point code for advective mass transport simulation. The achieved best solution may be checked by a more detailed simulation model to alleviate accuracy concerns.

The computational tools and the aquifer studied are outlined in the following paragraphs.

2. FLOW FIELD AND SIMULATION MODELS

A horizontal, steady state flow in an infinite, confined, homogeneous, isotropic aquifer is studied. As shown in Figure 1a, two pollutant plumes may affect production wells 1 and 2, which pump a total flow rate of 250 l/s. Additional wells, pumping up to 120 l/s each, may be used for hydrodynamic control of the plumes. The aquifer bears two fractures, one almost perpendicular to the expected flow lines between plume 1 and well 1 and one almost parallel to the flow lines between plume 2 and well 2. The main aquifer features are: hydraulic conductivity $K = 10^{-4}$ m/s, thickness $a = 50$ m and porosity $n = 0.2$. The features of wells, fractures and plumes are presented in Table 1.

![Figure 1. Theoretical flow field with checkpoints’ trajectories of best solution of the simplified problem, aiming at minimization of pumping cost only. Magnified view of each plume on the right.](image)

<table>
<thead>
<tr>
<th>Object</th>
<th>X (m)</th>
<th>Y (m)</th>
<th>Radius (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Well 1</td>
<td>753.275</td>
<td>1373.927</td>
<td>0.25</td>
</tr>
<tr>
<td>Well 2</td>
<td>1451.369</td>
<td>1237.570</td>
<td>0.25</td>
</tr>
<tr>
<td>Plume 1</td>
<td>510.893</td>
<td>1878.212</td>
<td>50</td>
</tr>
<tr>
<td>Plume 2</td>
<td>1673.430</td>
<td>683.058</td>
<td>60</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Object</th>
<th>x-coord (m)</th>
<th>y-coord (m)</th>
<th>Length (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fracture 1 Start</td>
<td>645.561</td>
<td>1605.924</td>
<td>70</td>
</tr>
<tr>
<td>Fracture 1 End</td>
<td>688.038</td>
<td>1662.313</td>
<td></td>
</tr>
<tr>
<td>Fracture 2 Start</td>
<td>1508.571</td>
<td>971.469</td>
<td></td>
</tr>
<tr>
<td>Fracture 2 End</td>
<td>1525.572</td>
<td>924.448</td>
<td></td>
</tr>
</tbody>
</table>

Table 1. Characteristics of wells, fractures and plumes.
Since early investigations (e.g. Snow, 1965; Long et al, 1982) generic numerical (2D or 3D) Discrete Fractures Networks (DFN) have been used to simplify flow calculations in fractured aquifers. Their study, though, presents a lot of difficulties still (e.g. Papadopoulou et al, 2010). Other modeling approaches for groundwater flow in fractured rock are the Stochastic Continuum and the Channel Network approaches. A comparative analysis of the three methods can be found in Selroos et al (2002).

The basic assumption made in this paper is that fractures do not affect the hydraulic head distribution (Kontos et al, 2010). This allows use of analytical formulas for hydraulic head and velocity calculations outside the fractures (e.g. Bear, 1979). Advective pollutant transport is simulated using a simple particle tracking method, which has been used in optimization of groundwater resources management problems (e.g. Katsifarakis et al, 2009; Bauser et al, 2012). In our case, 16 ‘checkpoints’ are symmetrically placed on each circular plume’s perimeter, to serve as starting-points for polluted particles of infinitesimal mass. Their trajectories simulate the course of the respective plume. The study period, which is equal to the pollutant deactivation period (1000 days), is divided into 100 time steps of 10 days each. Local velocity components at any point of the flow field are used to calculate particle displacement during each time step.

In order to model pollution of a well W, we use the following criterion: if the line segment that simulates the displacement of a pollutant particle P during time step $\Delta t$, intersects the hypothetical circular security zone of W, then we consider that P has arrived at W (Katsifarakis & Latinopoulos, 1995). The radius of the ‘security zone’ of each well is considered proportional to $R_w$, namely to the radius of the aquifer’s cylindrical volume, which contains the water pumped by well W during one time step:

$$R_w = \sqrt{Q_w \cdot 10^{-3} \cdot \Delta t / (\pi \cdot n \cdot \alpha)}$$  \hspace{1cm} (1)

where $Q_w$ is well’s flow rate, $n$ aquifer’s porosity and $\alpha$ aquifer’s thickness.

3. GENETIC ALGORITHMS – OBJECTIVE FUNCTION

Genetic algorithms are based on the Darwinian theory of the survival of the fittest and imitate mathematically the biological process of evolution of species. Initially introduced by Holland (1975), the method is already a well established mathematical tool, very efficient in optimization problems when objective functions exhibit many local optima or discontinuous derivatives (e.g. Goldberg, 1989). We have adopted the classical binary representation of the chromosomes (namely of potential problem solutions). In our case each chromosome represents the coordinates and flow rates of the additional wells and the flow rate of one of the existing wells. It consists of 70 digits (namely the string length equals 70) and its structure is shown in Figure 2.

The genetic operators used are selection (the tournament procedure, including an elitist approach), crossover and mutation/antimetathesis (Katsifarakis and Karpouzos, 1998). The following GA parameter values are used: population size PS=60, number of generations NG=1500, selection constant KK=3, crossover probability CRP between 0.40 and 0.60, mutation probability MP between 0.01 (=1/SL, where SL is the string length) and 0.028.

![Figure 2. Typical 70-digit binary chromosome representing random proposed solution.](image-url)
Optimization of the pumping scheme, in our case, implies minimization of the following “cost” function $F_V$:

$$F_V = VB_1 + VB_2 + VB_3$$  \hspace{1cm} (2)$$

where $VB_1$=pumping, $VB_2$=pipe network amortization and $VB_3$=pumped polluted water treatment costs. Moreover, a penalty is incorporated in term $VB_1$, if the pumping scheme results in pollution of the existing wells.

### 3.1 Pumping Cost and Penalty function

Pumping is probably the main cost item in groundwater management problems (e.g. Sidiropoulos and Tolikas, 2008; Kalwij and Peralta, 2008). In our case it is calculated as:

$$VB_1 = A \cdot \sum_{i=1}^{TNW} Q_i \cdot S_i$$  \hspace{1cm} (3)$$

where $A$ is a pumping cost coefficient, that depends on the density of pumped fluid, the electricity cost per kWh, pump efficiency and the pumping duration (here $A=6.48$), $TNW$ the total number of wells, $Q_i$ the flow rate of well $i$ (in l/s) and $S_i$ is the hydraulic head level drawdown at well $i$ (in m).

To this cost a penalty ($PEN$) should be added, whenever the pumping scheme results in pollution of an existing well. We have opted for a $PEN$ depending both on the number of violated constraints (number of pollution particles arriving at existing wells) and on the magnitude of the violation (time step of pollutant arrival at existing wells):

$$PEN = \sum_{i=1}^{NP_1} [P_C + P_V \cdot (TP - t_i)]$$  \hspace{1cm} (4)$$

where $NP_1$ is the number of checkpoints that pollute an existing well, $P_C$ the constant part of the Penalty function (for each polluting checkpoint), $P_V$ the coefficient of the variable part of the Penalty function, $TP$ the number of time steps (here $TP=100$) and $t_i$ is the time step during which checkpoint $i$ arrives at the well. The complexity of the evaluation process lies with the calculation of $PEN$.

We have tested many sets of $P_C$ and $P_V$ values, in order to find the minimum Penalty Function that will deliver a penalty-free best solution, at least in the last generation. Large $P_C$-$P_V$ values will deliver a penalty-free best solution, but they may obscure the actual optimization process. The $P_C/P_V$ ratio is set to 10/1 in all scenarios, trying to blend the impact of straight forward, blind degradation of a solution just for violating the constraints with the more sophisticated variable part of the penalty that is proportional to the magnitude of the violation. Obviously, in cases where $PEN>0$, its value can range from $P_C$ to $NP_1[P_C + P_V(TP-1)]$.

### 3.2 Pipe Network Cost ($VB_2$)

$VB_2$ actually represents the pipe network amortization cost (e.g. Katsifarakis et al, 2006), which is directly proportional to the initial network cost. The latter depends on the total length and diameter of the pipes that carry pumped water from the additional wells to a central pumping station, located, in our case, close to the boundaries of the flow field, as shown in Figure 3 ($X_{ST}=1400$ m $Y_{ST}=2500$ m). Network construction cost is taken as 45€/m and 60€/m for small and large pipe diameters, respectively. The threshold is set at $Q=50l/s$, since pipe diameter is selected according to the flow rate. For a theoretical amortization period of 10 years and an interest rate of
5\%, the amortization cost is $A_{a1}=6 \, \text{€/m}$ and $A_{a2}=8 \, \text{€/m}$ for small and large pipe diameters, respectively. Thus, $VB2$ is calculated as:

$$VB2 = \sum_{i=1}^{\text{NADW}} A_{ai} \cdot L_i$$

(5)

where NADW is the number of additional wells, $A_{ai}$ is either $A_{a1}$ or $A_{a2}$ (if $Q_i$ is smaller or larger than 50 l/s, respectively) and $L_i$ is length of the pipe that carries water away from well $i$.

Figure 3. a) Best solution for combined problem ($VB1+VB2=\text{MIN}$), b) magnified view of Plume 2 checkpoints being pumped by additional Well 3 after having polluted Fracture 2.

Given the well coordinates, the task is to produce the shortest tree type pipe network, connecting the wells to the central station and to calculate the flow rate $QL_i$ through each pipe $i$, in order to select the proper $A_{ai}$ value. The wells are labeled according to their distance from the station. Label of the most distant well is set equal to 1. Thus, to find the shortest $L_i$ from well $i$, only wells with larger label values are checked. Moreover, $QL_i$ calculations start from the most distant well and proceed following increasing label order.

### 3.3 Pumped Polluted Water Treatment Cost ($VB3$)

$VB3$ is calculated similarly to the Penalty function, except for the fact that the variable part depends on the value of the respective additional well’s flow rate, too:

$$VB3 = \sum_{i=1}^{\text{NP2}} [V_C + V_Y \cdot Q_i (TP - t_i)]$$

(6)
where NP2 is the number of checkpoints that pollute an additional well, \( V_C \) the constant part of VB3 (for each polluting checkpoint), \( V_V \) the coefficient of the variable part of VB3 and \( Q_i \) the flow rate of the additional well that pumps checkpoint \( i \).

The values of \( V_C \) and \( V_V \) can vary, depending on the pollutant. High values imply a pollutant demanding high water treatment cost, while low values imply low treatment cost. The \( V_C/V_V \) ratio is set to 10/1, similarly to the \( P_C/P_V \) ratio, trying to scale the constant part of VB3 that represents the standard (installation) cost, to the variable part, which represents the operational cost. If \( V_V \) exceeds a certain value (given the \( V_C/V_V \) ratio) the minimization process (\( VB1+VB2+VB3 \)) leads to solutions that avoid pollution of the additional wells (\( VB3=0 \)). In this case the additional wells are only used for hydrodynamic control of the plumes, either by delaying or diverting the checkpoints’ trajectories. Obviously, if \( VB3>0 \), its value can range from \( V_C \) to \( NP2\cdot[V_C+V_V\cdot Q_{MAX}\cdot(TP-1)] \), where \( Q_{MAX} \) is the maximum allowed flow rate of an additional well (here 120 l/s).

4. SIMULATIONS – RESULTS

4.1 Minimization of pumping cost only (\( VB1=MIN \)), CRP-MP investigation (Sim. 1)

An extended set of test runs, investigating CRP and MP values for the simplified problem, leads to the proposed pumping scheme of Figure 1 and to interesting conclusions, partially deviating from established views regarding MP values. We have performed 10 runs for every CRP-MP combination, in order to produce a satisfactory statistical sample of solutions. In the tests CRP ranges from 0 to 0.6, with step 0.02 and MP from 0.01 to 0.028, with step 0.002. This means 10 runs \( \times \) 31 CRP values \( \times \) 10 MP values = 3100 total runs. The computational volume required for the whole set of test runs was enormous. The evaluation function has been calculated 279 million times and the total computational time needed was approximately 156 days (Intel Core i5 CPU 660 at 3.33 GHz, 4GB of RAM, OS Windows 7, Visual Basic 6.0). Some of this simulation’s key characteristics are presented in Table 2. As far as the Penalty is concerned, values \( P_C=100 \) and \( P_V=10 \) manage to achieve minimum pumping cost.

Table 2. Key characteristics of all simulations. Pollutant ‘toxicity’ actually refers to the relative cost of treatment.

<table>
<thead>
<tr>
<th>Simulation</th>
<th>VB1 (€)</th>
<th>VB2 (€)</th>
<th>VB3 (€)</th>
<th>Pollutant ‘Toxicity’</th>
<th>( P_C )</th>
<th>( P_V )</th>
<th>( V_V )</th>
<th>( V_C )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>✓</td>
<td></td>
<td></td>
<td>-</td>
<td>100</td>
<td>10</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>✓</td>
<td>✓</td>
<td></td>
<td>-</td>
<td>1500</td>
<td>150</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>3.1</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>LOW</td>
<td>4000</td>
<td>400</td>
<td>10</td>
<td>1</td>
</tr>
<tr>
<td>3.2</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>HIGH</td>
<td>45000</td>
<td>4500</td>
<td>200</td>
<td>20</td>
</tr>
</tbody>
</table>

The algebraic results of the best solution (MIN FV) of all test runs (Figure 1) are presented in Table 3. The solution concept seems to be simple: two additional wells are needed, each one located between a plume and a fracture, and pumping thoroughly that plume. In this simulation and of course due to the quite ‘symmetric’ layout of the theoretical field, the total potable water flow rate is equally distributed to the existing wells as shown in Table 3 (\( Q_1/Q_2=1.01 \)).

Table 3. Simulations’ results, including Fitness Value, pumping, pipe network and remediation costs, flow rates and coordinates of additional wells.

<table>
<thead>
<tr>
<th>Sim.</th>
<th>FV (€)</th>
<th>VB1 (€)</th>
<th>VB2 (€)</th>
<th>VB3 (€)</th>
<th>( Q_1 ) (l/s)</th>
<th>( Q_2 ) (l/s)</th>
<th>( Q_3 ) (l/s)</th>
<th>( Q_4 ) (l/s)</th>
<th>( Q_1/Q_2 )</th>
<th>( X_1 ) (m)</th>
<th>( Y_1 ) (m)</th>
<th>( X_3 ) (m)</th>
<th>( Y_3 ) (m)</th>
<th>( X_4 ) (m)</th>
<th>( Y_4 ) (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>72160</td>
<td>72160</td>
<td></td>
<td></td>
<td>125.490</td>
<td>124.510</td>
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<td>572</td>
<td>1789</td>
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<td>2</td>
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<td>72943</td>
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<td>125.490</td>
<td>124.510</td>
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<td>1.01</td>
<td>1491</td>
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<td>647</td>
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<td>3.1</td>
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<td>487</td>
<td>2117</td>
<td>253</td>
<td>1953</td>
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<td></td>
</tr>
</tbody>
</table>
In Figure 4 one can see the relation of Min or Mean FV and CRP (Figure 4a) or MP (Figure 4b). Min FV refers to the best solution out of all 100 runs for a specific CRP value in Figure 4a, and to the best solution out of all 310 runs for a specific MP value in Figure 4b. Mean FV refers to the average value of FV out of all the respective runs. While the variation of CRP seems not to have a drastic impact on Min or Mean FV, there is a correlation between FV and MP. There is an obvious trend, as far as Mean FV is concerned, to exhibit lower values as MP increases. Hence, we can assume MP values a lot higher than the empirically proposed $1/SL=0.014$, where $SL =$ String Length, are more likely to reach the ideal global optimum.

Figure 4. Minimum Fitness Values a) in relation to Crossover Probability values and b) in relation to Mutation Probability values, for Simulation 1 (minimization of pumping cost only).

Figure 5 presents the lower 5% percentile of FV for all CRP-MP tests. Here, it is even more obvious that there is a higher statistical probability for the algorithm to produce lower FV solutions when CRP and MP values derive from the upper right area of the diagram.

Figure 5. FV5% perc. in relation to CRP and MP values for Simulation 1. Each value’s symbol and color opacity denote the quarter it belongs to (FV5% perc. is the lower 5% percentile of FV MIN).
The overall optimal solution has been achieved for CRP=0.06 and MP=0.01. Overall, the CRP value demonstrating higher probability of achieving the lowest FV values is CRP=0.42 and the respective MP value is MP=0.024. The two, contradicting at first, observations confirm the stochastic nature of genetic algorithms, but lead us to the crude conclusion that values CRP>0.40 and MP>0.020 are more likely to achieve optimal performance of the algorithm.

4.2 Combined minimization of pumping and pipe network costs (VB1+VB2=MIN, Sim. 2)

Simulation 2 (VB1+VB2=MIN) includes test runs with a smaller range of CRP-MP values (broken-line rectangle of Figure 4). The evaluation function has been calculated 81 million times. The absolute best solution (MIN FV) of all test runs, shown in Figure 3, was achieved for CRP=0.50 and MP=0.022. In this problem, the Penalty applied to solutions violating constraints had to be 15 times higher than that of the previous simulation, namely Pc=1500 and Pv=150 (Table 2). This set’s best solution presents, as expected, a higher value of FV (Table 3) as the overall cost is now burdened with the pipe network cost as well, while the pumping cost alone is slightly higher than the previous set’s lowest FV.

The dominant concept here is quite similar to the previous simulation: two additional wells are needed, each one located between a plume and the existing wells, having the task to thoroughly pump a plume. The only difference is that in this simulation, the optimal solution locates the additional well that pumps plume 2 between fracture 2 and the existing wells, allowing pollution to reach the fracture before being pumped. In this simulation also, the total potable water flow rate is equally distributed to the existing wells as shown in Table 3 (Q1/Q2=1.01).

In this case the genetic algorithm manages to balance antagonizing cost items in order to minimize their sum. Given the pumping station’s location, it is easy to calculate the hypothetical pipe network cost of simulation 1 proposed pumping scheme. The shortest pipe route is: well 3 – well 4 – pumping station, so the total network length is 2540.427 m. Since flow rates of both additional wells are under 50 l/s the respective VB2 value would be: VB2=6 €/m · 2540.427 m= 15242.560 €, hence the total cost of that solution would be FV= 72160+15242= 87403 €. In simulation 2, well 4, dealing with plume 1, preserves its flow rate and roughly its location. Well 3, though, that deals with plume 2, is shifted towards well 4, past fracture 2. As a result, its flow rate has to be 11% larger (8.504 rather than 7.559 l/s) in order to pump all the pollution-simulating checkpoints originating from both plume 2 and the inevitably polluted fracture 2, but the total network length is now 2189.505 m. The increase of the pumping cost by 1% (72943–72160=783 €), however, is outbalanced by the decrease of the pipe network length and consequently pipe network cost by 14% (2540.427-2189.505= 350.922 m and 15242.560-13137.032= 2105.528 €). The final result is that the total cost (FV) is 1323 € or 1.5% smaller (87403-86080= 1323 €).

4.3 Combined minimization of all costs (VB1+VB2+VB3=MIN, Sim. 3)

Simulation 3 includes test runs with a smaller but efficient range of CRP-MP values (solid-line rectangle of Figure 5), saving computational time. The evaluation function has been calculated 10.8 million times. There are two sets of test runs, one for a low-cost treatment of polluted water (scenario 1) and another for a high-cost one (scenario 2). In the first scenario, MIN FV (Figure 6) is achieved for CRP=0.40 and MP=0.024. In the second scenario, MIN FV (Figure 7) is achieved for CRP=0.42 and MP=0.022. The respective Pc, Pv, Vv and Vc values are presented in Table 2. The series of test runs produce different kinds of solutions, not only algebraically, but in terms of different structures, too. Different solutions suggest different types of flow profiles. All profiles are studied and grouped by means of the following criteria: a) general placement of additional wells, b) pollution or no pollution of fractures and of additional wells and c) type of trajectories generated. For both scenarios, and out of 120 runs for each, we have identified 14 distinct profiles.
In scenario 1, the trend is to use two additional wells of low flow rates (resulting in low VB1) with at least one, pumping part of the pollution (resulting in VB3>0). The dominant concept seems to be: pump one plume and divert the course of the other, bypassing the intermediate fracture that would spread the pollution. The first is succeeded by direct pumping of one plume by at least one additional well and the second by the appropriate distribution of total flow rate to the existing wells and sometimes by the influence of the second additional well, too. Unlike the 2 previous simulations, the total potable water flow rate is not equally distributed among the existing wells (Table 3). The flow rate ratio is Q1/Q2=3.90, which means that well 1 pumps about 4 times more water than well 2, playing a crucial part in the diversion of the course of plume 2. As expected, solutions where plume 1 is the one pumped, exhibit lower FV, since Fracture 1 is more difficult to be bypassed, due to its orientation and its smaller distance from the respective plume.

On the other hand, in scenario 2, shown in Figure 7, the algorithm tends to use two additional wells of considerable flow rates (higher VB1), that do not pump polluted water (VB3=0). The concept here is to use both additional wells in order to slow down spreading of one plume, by counterbalancing the influence of the existing wells upon it. The other plume is diverted away from its nearest fracture and existing well, mainly by appropriate distribution of total flow rate to the existing wells. Plume 2 is actually diverted away from existing well 2, by pumping almost 80% of the required total flow rate from well 1 (Q1/Q2=3.72). For reasons similar to those of the previous scenario, lower FV values entail placing additional wells close to plume 1 (slowing down its spreading), rather than close to plume 2.

This simulation’s best solutions present, as expected, higher values of FV than those of simulation 2 (Table 3). In scenario 1, the overall cost is 12% higher than the best solution of the previous simulation, while the pipe network cost is half the previous one and the extra treatment cost is just about 2% of the total cost. The pumping cost alone is larger than the total cost of the previous simulation, despite the fact that the sum of additional wells flow rates is decreased by 44% (8.504 rather than 15.118 l/s). In scenario 2, the high cost of pumped pollution treatment entails the hydrodynamic control of plume 1, instead of its pumping. This leads to a significant increase of the pumping cost, a pipe network cost similar to that of simulation 2 and null remediation cost. The pumping cost alone is again larger than the total cost of every previous scenario or simulation.
Figure 7. Best solution for simulation 3 (VB1+VB2+VB3=MIN), high cost pollutant treatment.

Table 4 presents key results and Figure 8 qualitative sketches of some of the flow profiles. Both refer to the proposed best solutions for low cost treatment cases (a to d) and to high cost ones (e to h).

Table 4. Key results of best solutions of Simulation 3 (a, b, c for low and e, f, g for high cost treatment pollutants) and most frequent one (d for low and h for high cost treatment pollutants).

<table>
<thead>
<tr>
<th>Profile</th>
<th>Scenario</th>
<th>Min FV</th>
<th>Rank-Min FV</th>
<th>Mean FV</th>
<th>Rank-Mean FV</th>
<th>Frequency %</th>
<th>Rank-Frequency</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>1</td>
<td>97996.276</td>
<td>1</td>
<td>95816.309</td>
<td>1</td>
<td>19.17</td>
<td>2</td>
</tr>
<tr>
<td>b</td>
<td>1</td>
<td>98202.998</td>
<td>2</td>
<td>100161.710</td>
<td>2</td>
<td>3.33</td>
<td>7</td>
</tr>
<tr>
<td>c</td>
<td>1</td>
<td>102793.071</td>
<td>3</td>
<td>104496.652</td>
<td>4</td>
<td>13.33</td>
<td>4</td>
</tr>
<tr>
<td>d</td>
<td>1</td>
<td>104655.656</td>
<td>5</td>
<td>108855.631</td>
<td>9</td>
<td>33.33</td>
<td>1</td>
</tr>
<tr>
<td>e</td>
<td>2</td>
<td>130169.572</td>
<td>1</td>
<td>133063.641</td>
<td>1</td>
<td>26.67</td>
<td>2</td>
</tr>
<tr>
<td>f</td>
<td>2</td>
<td>130819.268</td>
<td>2</td>
<td>192873.766</td>
<td>11</td>
<td>6.67</td>
<td>5</td>
</tr>
<tr>
<td>g</td>
<td>2</td>
<td>147003.939</td>
<td>3</td>
<td>147003.939</td>
<td>2</td>
<td>0.83</td>
<td>13</td>
</tr>
<tr>
<td>h</td>
<td>2</td>
<td>162363.343</td>
<td>6</td>
<td>174831.015</td>
<td>8</td>
<td>0.83</td>
<td>1</td>
</tr>
</tbody>
</table>

In the first case, “a”, “b” and “c” are the 3 profiles linked to the lowest FV values respectively (Figure 6 solution’s profile is an “a”). But their ranking, concerning their frequency of appearance, is 2, 7 and 4 out of 14. The most frequent profile appearing in the algorithm output is “d” (Figure 8d), that has a ranking, concerning lowest FV value, equal to 5 out of 14. In the second case, “e”, “f” and “g” are the 3 profiles linked to the lowest FV values respectively (Figure 7 solution’s profile is an “e”). Their ranking, concerning their frequency of appearance, is 2, 5 and 12 out of 14. The most frequent profile appearing in the algorithm output is “h” (Figure 8h), that has a ranking, concerning lowest FV value, equal to 6 out of 14. These different profiles show that there is a substantial number of different good solutions, exhibiting similar fitness values. Their identification can be considered as an asset of the method of genetic algorithms.
Evaluating the first scenario best solution flow profile (Figure 6 and Figure 8a), it is obvious that the solution is rather unrealistic, since it entails construction of 2 wells of low flow rates so close together (especially when the one most distant from the plume exhibits Q < 1 l/s). Figure 6c demonstrates a partial failure of the moving point technique, namely unrealistic linear displacements of the plume’s checkpoints close to the additional wells, which is due to their small flow rate that entails small security zone rw. This problem can be avoided, if a larger value is introduced for rw. The overall accuracy can be also increased by a finer discretization (e.g. 1000 time steps of 1 day), which will of course increase computational time.

What is realistic and necessary is simulating the pumping scheme of the proposed best solution with this finer discretization. In this better approximation of the flow, all checkpoints were pumped by the well closest to the plume (well 4). In an even more daring test, the more distant well was removed. The results were exactly the same: plume 1 was entirely pumped by the unique additional well. Thus, we can accept that the concept of the proposed solution is correct and actually can work with only one well, which means even less pumping, pipe network and treatment costs.

5. DISCUSSION AND CONCLUSIONS

Efficient use of GAs presupposes adequate population size and number of generations. In order to keep total computational volume under control, some simplification of the evaluation process (in our case flow and mass transport models) is required. Such simplifications may raise questions regarding the accuracy of the overall results. For this reason we have adopted certain safety counter-measures, such as the virtual lengthening of the fractures and the instant spread of pollutants along them. Increase of pollutant’s deactivation period could be also adopted, since pollutant dispersion has not been taken into account.

The quality of the results is also influenced by the GA parameters, such as crossover (CRP) and mutation probability (MP), and the coefficients of the penalty function, too (Pc, Pv). For this reason we have undertaken an extensive set of tests, to identify the best combination. Our results did not verify the practical rule of setting MP equal to 1/SL. The best fitness values (FV) were achieved with MP>0.02, while 1/SL=0.014, namely with at least 30% higher values of MP. Finally, the importance of an a-posteriori check of the results of the optimization process has been stressed, which might eliminate unpractical features of the respective solutions. A finer discretization of the flow inside the optimization algorithm is prohibitive, due to the significant increase of computational load. Hence, one of those checks must be the simple test-simulation of the proposed
best solutions with this finer discretization, that can lead to an improved optimal solution with an even lower fitness value or even a different, more efficient flow profile. 

Future research includes implementation of an extra cost item, namely construction cost, in the evaluation function, to avoid profiles such as Figure 6c and direct GAs to the realistic equivalent solution instead. The optional use of additional recharge wells instead of, or together with, abstraction wells can also be implemented, to investigate whether the total cost can be further decreased and to point out which cases are most likely to benefit from it. Moreover, a brief additional well’s pumping sudden failure incident can be studied, in order to stress test the existing solution profiles and conclude on their fail-safe attributes. A standard failure scenario can then be introduced to the evaluation function to raise the safety standards of the proposed solutions.

REFERENCES


