Herzberg approximation of Appendix A.

Chapter 5

Optimal design of measures to correct seawater intrusion*

Many coastal aquifers are nowadays affected by seawater intrusion. They require corrective measures to restore groundwater quality or, at least, to reverse current negative trends. These measures can be grouped into actions over the demand (i.e., reduce pumping), actions over the recharge (i.e., artificial recharge and territorial planning), relocation of pumping wells and additional engineering solutions (i.e., seawater intrusion barriers). The decision of the selected management policy is subject to a number of constraints including: strategic nature of the aquifer; existing infrastructures that may affect the relocation of pumping wells; and historical rights that require the abstraction rates and water quality to be as close as possible to the present rates. A balance between pumping demand and quality requirements is necessary. This balance is hard to maintain when the final goal is to reverse the qualitative status of the already contaminated aquifer. Precautionary corrective measures are not applicable once a portion of the aquifer has already been affected by seawater intrusion. The recovery of groundwater quality is usually a very slow process as seawater intrusion

*This chapter is based on the paper: Abarca, E., Vázquez-Suné, E., Carrera, J., Capino, B., Gámez, D. and Batllé, F., submitted to *Water Resour. Res.*, Optimal design of measures to correct seawater intrusion is the result of a long term negative mass balance in the aquifer. Some measures are needed to boost the recovery of the aquifer while helping to maintain pumping. In our case study, an artificial increase in recharge and specific measures such as a seawater intrusion barrier are proposed to restore groundwater quality in a coastal aquifer suffering from severe seawater intrusion.

Optimization is a classical tool to evaluate groundwater management alternatives. Optimization of groundwater resources has been subject of much research. Reviews of the topic have been covered by *Gorelick* (1983), *Wagner* (1995) and *Ahlfeld and Heidari* (1994). The last study provides an overview of optimization methods applied to real cases.

Optimization problems are often aimed at maximizing the overall pumping rate subject to some hydraulic or/and economic constraints, which leads to relatively easy formulations when heads depend linearly on flow rates. Unfortunately, this approach is not valid for coastal aquifers that are well connected to the sea, where water quality is the critical factor. Application of quality constraints results in non-linear optimization problems since concentration is non-linearly related to the decision variables (groundwater abstraction). Nevertheless, in the last decades a broad range of optimization formulations have been used in coastal aquifers. Differential equations of groundwater flow and solute transport have been solved using either analytical solutions (*Cheng et al.*, 2000; *Mantoglou*, 2003) or numerical simulation to represent the aquifer response. Numerical simulations of exclusively groundwater flow have been used (*Hallaji and Yazicigil*, 1996; *Zhou et al.*, 2003) using equivalent freshwater heads as constraints to prevent seawater intrusion. In other cases, seawater intrusion has been explicitly simulated using a sharp interface model (*Emch and Yeh*, 1998; *Rao et al.*, 2003; *Mantoglou*, 2003; *Mantoglou et al.*, 2004; *Park and Aral*, 2004) or chloride transport models (*Gordon et al.*, 2000). Some of the latter models used density dependent flow codes (*Das and Datta*, 1999a,b). However, computational cost restricted their application to relatively simple hypothetical cases. In fact, not only simulation cost but also optimization complexity increases when density variations are taken into account, which may lead to highly non-linear and non-convex problems (*Finney et al.*, 1992).

The choice of optimization method is ideally determined by the nature of decision variables (discrete or continuous), the linearity of the problem (degree of nonlinear dependence of objective function and constraints on decision variables), the continuity of the objective function, the availability of information about derivatives and by the potential existence of local minima (*Cheng et al.*, 2000). Therefore, it is not surprising that many different objective functions and sets of constraints have been applied to coastal aquifers. Some authors aim at maximizing the total pumping rate (*Shamir et al.*, 1984; *Hallaji and Yazicigil*, 1996; *Cheng et al.*, 2000; *Mantoglou*, 2003) whereas others aim at minimizing the salinity of pumped water (*Das and Datta*, 1999a,b) or the volume of saltwater into the aquifer (*Finney et al.*, 1992; *Emch and Yeh*, 1998). *Emch and Yeh* (1998) and *Gordon et al.* (2000) also include pumping costs into the objective function. It is also possible to consider a multiobjective optimization problem (*Shamir et al.*, 1984; *Emch and Yeh*, 1998; *Das and Datta*, 1999a; *Park and Aral*, 2004). Constraints applied in those studies include maximum and minimum pumping rates (*Hallaji and Yazicigil*, 1996; *Emch and Yeh*, 1998; *Das and Datta*, 1999a; *Cheng et al.*, 2000; *Zhou et al.*, 2003), toe location (*Cheng et al.*, 2000; *Mantoglou*, 2003; *Mantoglou et al.*, 2004; *Rao et al.*, 2004; *Park and Aral*, 2004) , groundwater heads (*Hallaji and Yazicigil*, 1996; *Emch and Yeh*, 1998; *Zhou et al.*, 2003), flow potential (*Mantoglou*, 2003; *Mantoglou et al.*, 2004) or salt concentration of the pumped water (*Gordon et al.*, 2000; *Das and Datta*, 1999a,b).

The aforementioned papers provide interesting methodological developments and often yield insights into the best methods of managing coastal aquifers. Application to real aquifers is less abundant, but can be found in recent literature (*Shamir et al.*, 1984; *Willis and Finney*, 1988; *Finney et al.*, 1992; *Reichard*, 1995; *Hallaji and Yazicigil*, 1996; *Nishikawa*, 1998; *Zhou et al.*, 2003; *Mantoglou et al.*, 2004; *Reichard and Johnson*, 2005). All of these applications aim at maximizing pumping while preventing seawater intrusion. However, optimization methods have not been tested to design corrective measures to restore water quality in initially salinized aquifers, such as the main aquifer of the Llobregat delta.

The aim of our work was to select and design corrective measures so as to ensure a water quality trend reversal in the Llobregat delta main aquifer. Corrective measures include (1) reductions in pumping rates, (2) inland artificial recharge and (3) a coastal freshwater injection barrier, coupled to inland pumping to clean up trapped salinized water. We used two optimization methods to test the efficiency of these measures. The first one consists of maximizing pumping rates while constraining coastal heads. This approach is similar to the one used by *Reichard* (1995); *Hallaji and Yazicigil* (1996); *Nishikawa* (1998); *Zhou et al.* (2003); *Reichard and Johnson* (2005). The second method uses concentration constraints to guarantee a satisfactory qualitative status of the aquifer. This approach has been used by *Gordon et al.* (2000) in some synthetic cases.

The aim of this paper is to compare the two optimization approaches and to discuss the efficiency of corrective measures.

5.1 Methodology

Two different approaches are applied to assess the feasibility and to quantify the effectiveness of the proposed measures. A groundwater flow and chloride transport model is combined with two different optimization methods. In the first one head constraints are used to ensure a gradient reversal along the coast line, which results in a linear optimization problem. The second method consists in maximizing the amount of water pumped subject to quality constraints, which leads to a non-linear optimization problem. Results of both methodologies are compared and the drawbacks and advantages of each method are discussed. The basic steps to follow are:

1. Groundwater flow and transport model construction and calibration
2. Definition of the reference scenario
3. Optimization process
    (a) Problem formulation

(b) Solution of the optimization problem

The first two steps follow standard modelling approaches. Here, we focus on the description of the two optimization procedures. We defined the objective functions for both procedures in terms of pumping rates. Economic expressions could have been used. In fact, our objective functions could have been expressed in terms of the minimization of costs of meeting demand and satisfying hydraulic or chemical constraints. For the sake of simplicity, we used pumping rates as optimization variables.

5.1.1 Linear optimization problem

Our aim is to maximize the amount of water pumped subject to head constraints. The objective function is expressed as:

$$Q_{opt} = \sum Q_i \qquad i = 1, ..., M \tag{5.1}$$

where Q_i the pumping rates in each of the M pumping areas.

Constraints can be applied at a certain number (N) of control points at any time in the simulation period. These control points can be divided into two groups: inland and coastal control points. Constraints applied at inland points control drawdowns produced by pumping, either to ensure that heads remain above a minimum (i.e., to prevent drying a wetland) or below a maximum (i.e., to prevent flooding a landfill). Constraints at coastal points aim at reversing the hydraulic gradient along the coast. Heads are required to stay above the equivalent freshwater head of seawater at that particular location. The aim of these coastal control points is to try to reverse seawater intrusion only by constraining heads (note that this concept is only valid for relatively thin aquifers, where vertical variations of salinity can be neglected). In our case, head constraints are applied at the end of the simulation period as

$$h_{j_{min}} \leq h_{j0} - \Sigma Q_i \mathbf{A}_{ij} \leq h_{j_{max}} \tag{5.2}$$

where h_{j0} is the groundwater head for the unstressed aquifer at the j^{th} control point, $h_{j_{max}}$ and $h_{j_{min}}$ are, respectively, the maximum and minimum allowed head at the j^{th} control point and $\mathbf{A}_{ij}$ is the response matrix, defined as the calculated drawdown at point j in response to a unit pumping in well i.

Constraints in the maximum allowed pumping rate may also be applied:

$$Q_i \leq Q_{maxi} \tag{5.3}$$

where Q_{maxi} is the maximum allowed pumping rate at well i

The same type of constraints can be applied to any corrective measure implemented into the optimization process. To determine where freshwater injection is most efficient, the corrective measure (i.e., a barrier) can be divided into segments and the constraint applied is the total amount of water available for injection through the whole corrective measure by applying:

$$Q_{design_i} = \sum Q_{seg_i} \tag{5.4}$$

where Q_{design_i} is the injection rate in corrective measure i and Q_{seg_i} the injection rate in each of the segments in which the corrective measure has been divided.

Maximizing Equation 5.1 subject to constraints 5.2, 5.3 and 5.4 results in a linear problem. We used *IMSL MATH/LIBRARY* (1997) routines to solve the linear problem.

One of the outputs of linear programming codes is the shadow price of each constraint. The shadow price is the Lagrange multiplier of the constraint. It represents the sensitivity of the objective function to the value imposed to the constraint. It is termed price because it expresses the marginal cost (in terms of objective function units) of a unit increase in the constraint. Thus, inactive constraints yield zero shadow price because the objective function is not sensitive to them. For management purposes, shadow prices are very useful. They enable us to express, in terms of the objective function, the cost of all active constraints. We will use them to evaluate the hydraulic efficiency of corrective measures (i.e., the ratio between the increase in pumping rate per unit increase in recharge rate). As such, they will allow us to answer the question of whether or not a recharge barrier implies a net gain of resources.

5.1.2 Non-linear optimization problem

Groundwater abstraction is linked to groundwater quality in coastal aquifers. Therefore, it may be necessary to ascertain whether the projected alternatives warrant a minimum groundwater quality. Given that one of the goals is to protect existing water rights, our aim is to obtain (1) a pumping rate regime as close as possible to the current one and (2) the corrective measures needed to achieve a satisfactory groundwater qualitative status. To this end, constraints in concentration are imposed. Since concentration is non linearly related to groundwater abstraction, the resulting optimization problem is non-linear. The problem is somewhat simplified by assuming that maximum pumping rates lead to generalized intrusion (in fact, this is the current situation along the Llobregat delta shores). Therefore, if concentration is constrained to be below a certain threshold at a point where intrusion is known to occur, such a constraint will be active. Knowledge of the active constraints allows us to substitute the constraint by a penalty function in the objective function. We have used the penalty function of Courant (*Fletcher*, 1985):

$$c_i \leq {c_i}^* \quad active \Rightarrow c_i = {c_i}^* \sim min \frac{(c_i - {c_i}^*)^2}{{\sigma_i}^2} \tag{5.5}$$

where c_i is the computed concentration at the i-th control point, ${c_i}^*$ is the desired (maximum) concentration at such point and σ_i is the penalty's tolerance, which allows one to impose variable weights on each constraint (penalty). In fact, c_i will tend to be slightly larger than ${c_i}^*$. However, c_i can be as close as desired by reducing σ_i. Using Equation 5.5 together with the desired pumping and recharge rates allows us to replace the general optimization problem by the minimization of the multiobjective function:

$$
\begin{aligned}
F = {} & \lambda_c \sum_{i=1}^{N} \frac{(c_i^* - c_i)^2}{\sigma_{c_i}^2} + \lambda_Q \sum_{j=1}^{M} \frac{(Q_j^* - Q_j)^2}{\sigma_{Q_j}^2} \\
& + \lambda_R \sum_{k=1}^{L} \frac{(R_k^* - R_k)^2}{\sigma_{R_k}^2}
\end{aligned}
\tag{5.6}
$$

where N, M and L are the number of control points, pumping zones and additional corrective measures, respectively; Q_j^* the desirable pumping rate for the j-th flow zone; Q_j the calculated flow for the j-th flow zone; R_k^* the desirable injection rate in the k-th corrective measure; R_k the calculated injection for the k-th corrective measure; σ_{Q_j} and σ_{R_k} are tolerance parameters (analogous to σ_{c_i}) for the j-th flow zone and the k-th corrective measure; λ_c, λ_Q and λ_R are the weights of the objective function of concentration, pumping rates and corrective measures, respectively. The tolerance parameters allow us to vary the relative contribution to each objective. Note that for the concentration term, λ_c is equivalent to the σ parameter of *Fletcher* (1985). The method calls for sequential optimization runs with increasing λ_c while doing away with the control points whose concentrations do not satisfy Equation 5.5 (i.e., those for which c_i is strictly smaller than c_i^*). However, in practice, we performed some preliminary runs to find appropriate λ's and obtained satisfactory results.

With this formulation, the resulting problem is similar to that of parameter estimation. We used the method of *Medina and Carrera* (1996) to minimize Equation 5.6.

5.2 CASE STUDY: Application to the Llobregat Delta Main aquifer

5.2.1 Background and problem statement

The Llobregat delta is located to the south of Barcelona (Spain) (Figure 5.1). It is a quaternary formation and is considered to be a classic example of a Mediterranean Delta controlled by fluvial and coastal processes (i.e., storms and long-shore drift). Geological studies (*Marqués*, 1984; *Simó et al.*, 2005) consider the delta to be formed by an Upper Complex (Q4 in Figure 5.2) and a Lower Complex (Q3, Q2 and Q1 in Figure 5.2). The Upper Complex is a typical stratigraphic delta sequence. The Lower Complex is formed by three fluvial systems associated with three currently submerged paleodeltas.

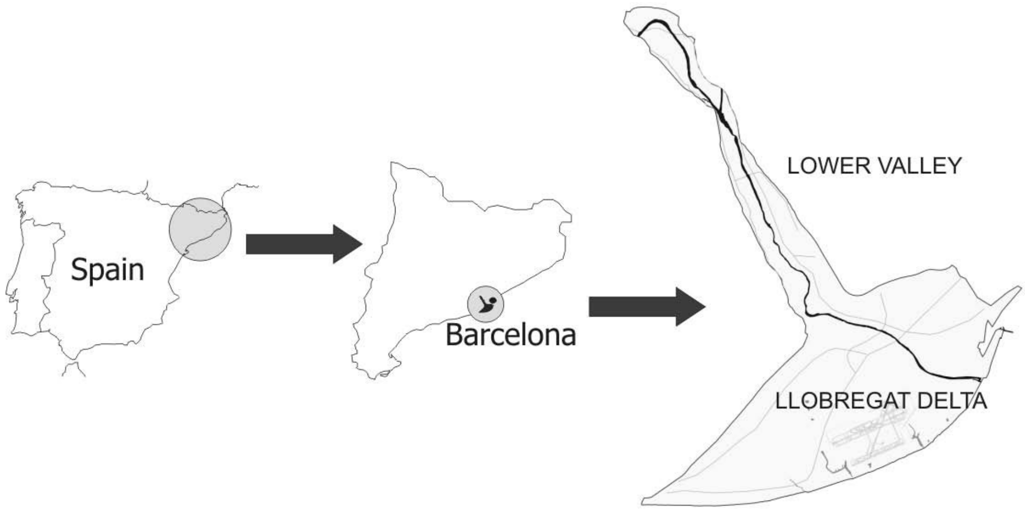


Figure 5.1: Location of the Llobregat delta and lower valley

Hydraulically, the prodelta silts of Q4 act as a confining unit separating the permeable upper units (upper, shallow aquifer) from a very thin and basal very permeable layer of reworked gravels and beach sands. This thin layer together with the upper gravels of Q3 constitute the Main aquifer of the Llobregat delta. It is an essentially horizontal aquifer (around 100 km^2) of 15-20 m thickness. High transmissivity zones are associated with the paleochannel systems of Q3 (Figure 5.3). The presence of this higher permeability channels are of extreme importance for the optimal management of this aquifer.

The Llobregat delta is well-known since numerous studies have been carried out in this area since the 60s, including among others, the hydrogeological synthesis works by *MOP* (1966); *PHPO* (1985) and more recently *Iribar and Custodio* (1992); *Iribar* (1992). In the late 70s, when salinization problems became a concern, hydrochemistry research improved the knowledge of

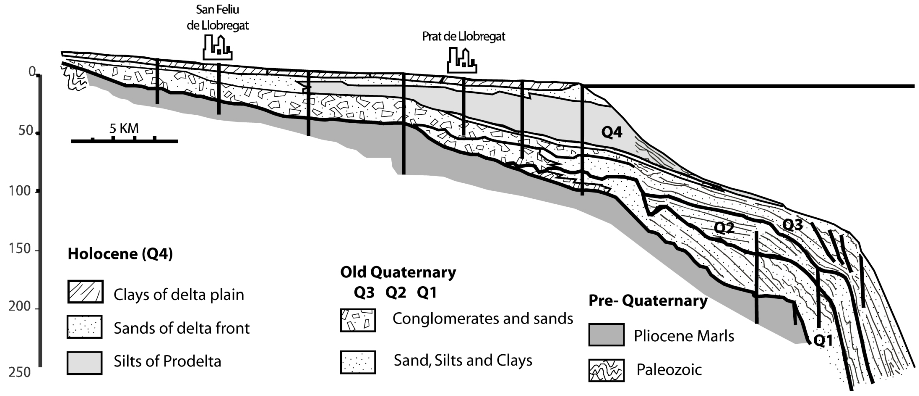


Figure 5.2: Modified from *Simó et al.* (2005). Geological cross section perpendicular to the coast of the emerged and submerged Llobregat delta. The very thin layer below the silts off Q4 and the upper gravels of Q3 form the Main aquifer studied in this work. Note that they are well separated from the shallow aquifer by the prodelta silts with the result that the main aquifer is effectively confined and relatively thin.

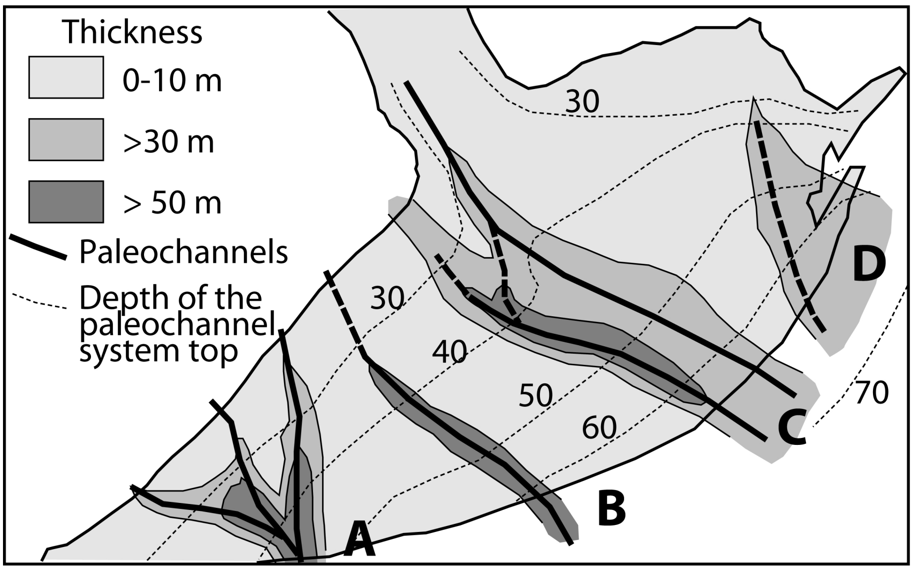


Figure 5.3: Modified from *Simó et al.* (2005). Paleochannel systems (A, B, C) associated with the main aquifer of the Llobregat delta. These paleochannels correspond to high permeability zones that may act as preferential seawater intrusion pathways and that make transmissivity distribution highly heterogeneous.

the aquifer systems and the mechanisms that cause seawater intrusion in the main aquifer of the Llobregat delta (*Custodio et al.*, 1976; *Custodio*, 1981; *Manzano et al.*, 1992; *Bayó et al.*, 1977; *Doménech et al.*, 1983), etc. Some groundwater flow models have also been developed for the Main Aquifer, termed Lower Aquifer in earlier works, (*Cuena and Custodio*, 1971; *PHPO*, 1985; *Custodio et al.*, 1989; *Iribar et al.*, 1997; *Vázquez-Suñé et al.*, 2005c). Chloride distribution (Figure 5.5) shows how seawater intrusion affects large areas of the delta.

We seek to assess sustainable management alternatives in this aquifer as well as to correct the current salinization problem. Our goal is to reverse the negative trends and recover the natural quality existing 35 years ago. Abstraction rates should remain as close as possible to current values. To achieve this goal, the following corrective measures are considered: (1) a reduction in current pumping rates, (2) a coastal barrier and (3) upland artificial recharge ponds. The last two measures should be chosen in accordance with efficiency and the optimized recharge/injection rates.

5.2.2 Groundwater flow and solute transport model construction and calibration

A two-layer areal model was built to simulate groundwater flow and chloride transport in the Llobregat delta aquifers. A two-dimensional horizontal model is used given that the concentration is well mixed in the vertical direction and that the aquifer thickness is small compared with the horizontal extent. Two dimensional models can take into account the density differences by working with freshwater equivalent heads at points with high salinity such as the seaside boundary (*Iribar et al.*, 1997). Therefore, our simulation model is linear. Note that this does not imply that the optimization problem needs to be linear. Linearity of the latter depends on whether constraints and objective function are a linear function of design variables (pumping rates). If the simulation problem is linear, then the optimization problem may or may not be linear. In fact, these two options are discussed in this paper. However, if the simulation problem had been non-linear, optimization would also have been non-linear.

The flow and transport problem uses a finite element grid of 4354 nodes and 9698 elements, divided into two layers. The upper layer depicts the upper aquifer (upper Q4 in Figure 5.2) and the lower layer represents the Main aquifer. These two layers are separated by 1D elements representing the silty wedge. The submerged portion of the lower aquifer is complex (Figure 5.2), but this complexity is deemed irrelevant for the management alternatives considered here. This submerged portion is modelled by extending the model domain (lower layer) 4 km seawards and the hydraulic parameters are estimated by calibration. Although both aquifers are considered in the simulation model, all pumping and corrective measures as well as constraints are applied to the Main aquifer.

Temporal variability of most sink and source terms make it necessary to evaluate management alternatives with a transient simulation model. The model was calibrated for the period from 1965 to 2001. Model parameters are divided into zones representing the aquifer heterogeneity. The value of the parameters are considered constant in each zone. The main aquifer inputs are the recharge from rain infiltration along with the recharge from the river during flood events (*Vázquez-Suñé et al.*, 2005a,b) while the main output is groundwater pumping. Inputs from surrounding aquifers are considered as well as interaction with surface flow (rivers, drainage systems, lakes, etc.)

Parameter values were updated during calibration. Automatic adjustments were carried out with the code TRANSIN-IV (*Medina and Carrera*, 2003), starting from the prior estimates available for every parameter. The code minimizes an objective function that considers the differences between measured and calculated head and/or concentrations, as well as the likelihood of calculated parameters (*Medina and Carrera*, 1996). Therefore, the code needs measured heads and concentrations at the defined observation points. The observation points provide a good coverage of the model domain. Head data at many observation points extend over almost the whole calibration period (1965-2001). However, the concentration data are more disperse in time. Parameter values obtained from calibration are consistent with prior information. The obtained fit both in terms of heads (Figure 5.4) and chloride concentrations (Figure 5.5) are considered satisfactory. The calibrated model was validated with additional data (2002-2004) showing a reasonable fit between measured and calculated data (Figures 5.4 and 5.5)

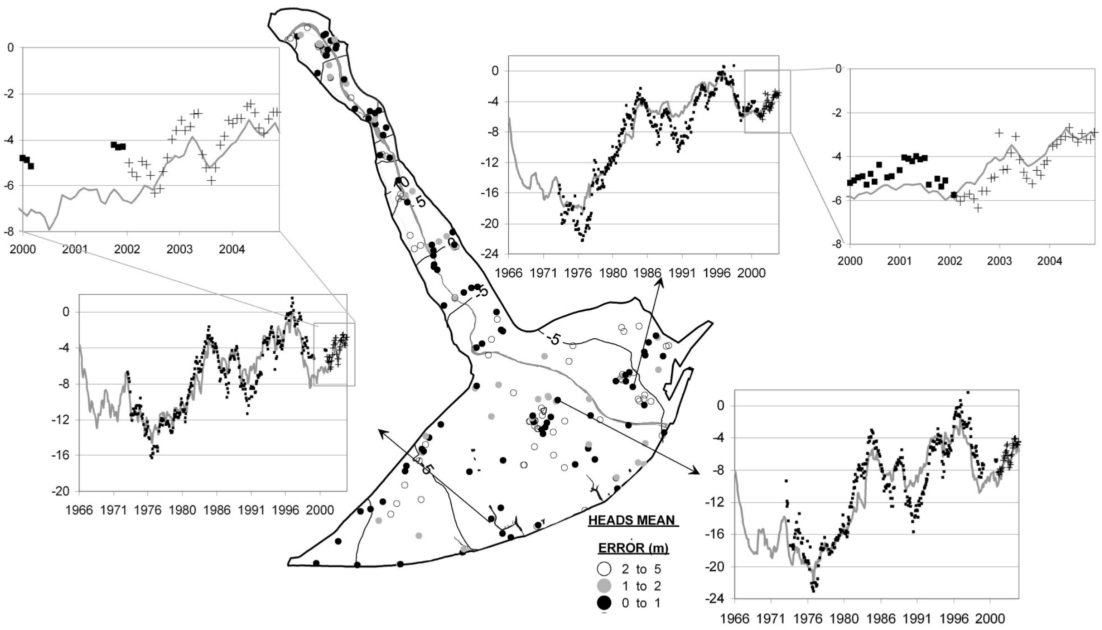


Figure 5.4: Piezometric map calculated for December 2001 together with the heads mean error and the calculated versus measured heads at four observation points. Calculated heads are plotted in a gray line, measured data used for calibration in black dots and data used for model validation in crosses.

5.2.3 Definition of a reference scenario

The model needs to be updated to be used as a management tool. As it will be applied to optimized future management strategies, some assumptions about the future behavior of the main terms of the water balance are necessary. These assumptions are uncertain. Some scenarios related to pumping rates, city growth and infrastructures were considered to evaluate this uncertainty. All these scenarios showed that current pumping is unsustainable given that seawater intrusion

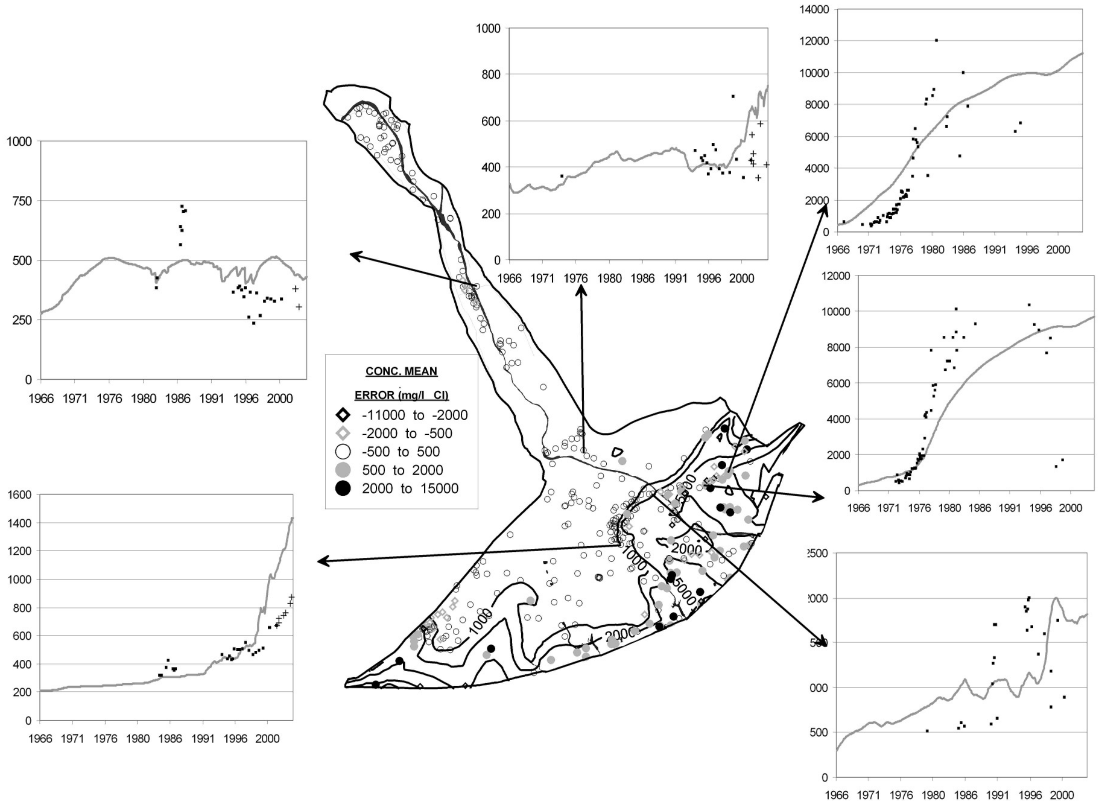


Figure 5.5: Chloride concentration map calculated for December 2001 together with the concentration mean error and the calculated versus measured concentrations at six observation points. Calculated concentrations are plotted at a gray line, measured data used for calibration in black dots and data used for model validation in crosses.

progressed further inland.

One of these scenarios was chosen as the reference with which to compare the optimization results. It considers the following assumptions:

- For the simulation/optimization period (2002-2036) meteorology was considered to be similar to that of 1965-2001. This would include both long dry periods and long wet periods.
- Pumping history assumed for future years is the average of the last five years (1997-2001) for all wells except for the main pumping area (AGBAR) whose temporal variability of the last five years was repeated along the simulation period.

- Location of the current pumping wells is maintained. The distribution of wells and the reference pumping rates are shown in Figure 5.6.

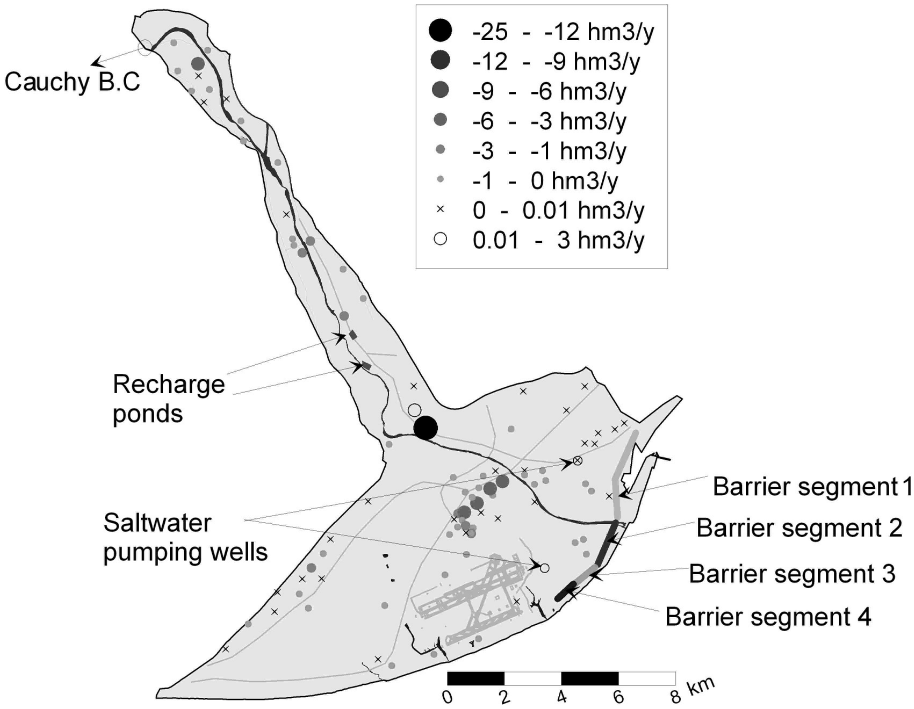


Figure 5.6: Location of the pumping wells and their average pumping rate in the period from 1997 to 2001 and location of the proposed corrective measures that are included in the optimization process

5.2.4 Management model

Optimization was applied to groups of existing wells defined as management polygons. Six polygons or management areas were defined in the main aquifer of the Llobregat Lower Valley. Five of them were defined according to geological, hydrological and management criteria. The sixth area, located along the coast, was defined for linear optimization requirements, as explained below. An equity principle was applied to the pumping wells located in the same management area with the result that pumping rates were reduced in the same proportion at all pumping wells. Figure 5.7a shows the six management zones defined in this aquifer.

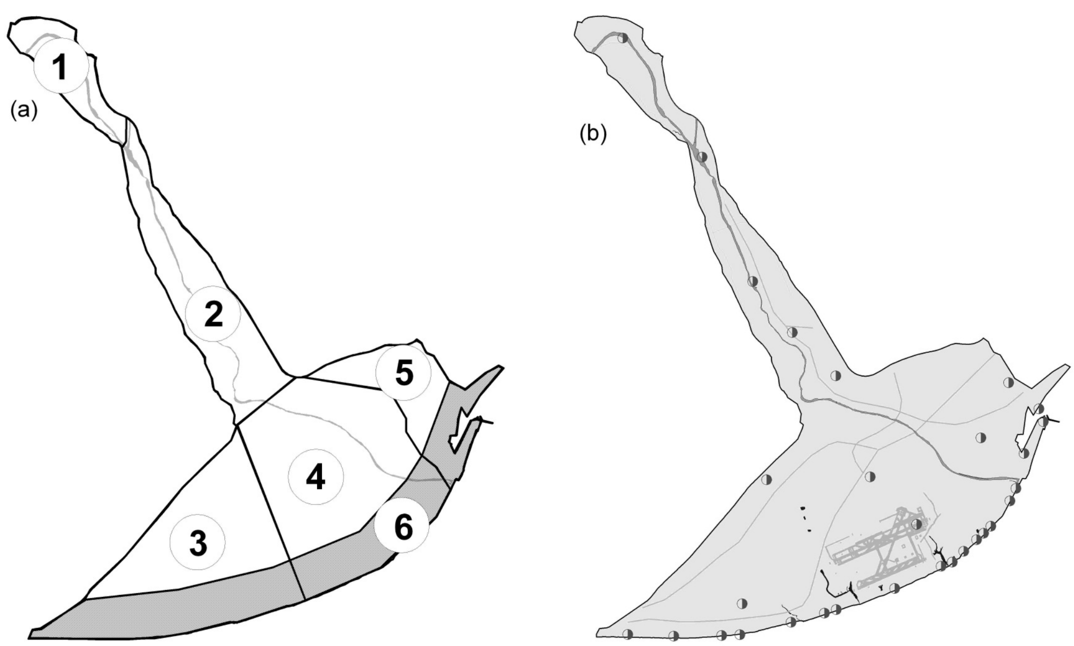


Figure 5.7: (a) Management areas defined in the aquifer according to geological, hydrological and management criteria and (b) location of the control points used for the optimization process

Two actions are considered, first to exclusively reduce the pumping rates and second, to increase the available resource by designing some additional corrective measures that help to maintain the current pumping rates. The considered additional measures are:

1. Recharge ponds to artificially increase the recharge. Two recharge ponds are proposed in the alluvial part of the main aquifer. The total area proposed is about 11 ha with a projected infiltration rate of 0.25 m/d, resulting in a total projected recharge of 11 hm^3/y. The location of the proposed ponds is indicated in Figure 5.6

2. A seawater intrusion barrier (hereinafter, referred to as barrier). A freshwater injection barrier, divided into four sections according to the different transmissivity zones defined in the numerical model, is proposed in the eastern part of the coast (Figure 5.6). The four segments are located in management Zone 6, however three of them are located in front of management Zone 4 and one in management Zone 5. Segment 1 is located over a preferential flowpath caused by the paleochannel C structure in Figure 5.3. The optimization problem

allows us to optimally distribute the total amount of water injected into the four segments and, in the case of the non-linear problem, to quantify the total amount of water needed to inject. This total projected injection rate is 3.65 hm^3/y.

5.2.5 Linear optimization problem

Our aim is to maximize the total amount of water pumped so that, by the end of the 35 year period, heads fall between a prescribed minimum and maximum head constraints. The constraints were applied to 30 control points spread all over the aquifer, 19 of them located along the coast (Figure 5.7b). Applied head constraints depend on the point location into the aquifer:

Minimum head constraints were applied at 25 control points. Minimum historic heads were applied at 6 inland control points located in the confined aquifer. h_{min} equalled the equivalent freshwater head of seawater at the other 19 points located along the coast, thereby imposing a seawards hydraulic gradient.

Maximum and minimum head constraints were applied at 5 control points located in the unconfined portion of the aquifer. The maximum head was also constrained because of the risk of affecting underground structures in the urban area and of washing out some dangerous landfills buried along the fluvial valley. The minimum head constraint at these points was 1/3 of the saturated thickness.

Initially, no restrictions were applied to pumping rates in the management areas. The total injection into the recharge ponds and the barrier equalled the projected values. In the case of the barrier, the total injected water is specified and the model optimizes the distribution of the total injection (11 hm^3/y for the ponds and 3.65 hm^3/y for the barrier) into the four defined segments.

Linear optimization results

Application of linear programming could lead to absurd results whenever the constraints are inappropriate. Therefore, care must be exercised to identify unrealistic solutions. Specifically, attention should be paid to the Dirichlet and Cauchy boundary conditions. While these boundary conditions may be necessary for modelling, they can act as unrealistic source terms for management problems (e.g., pumping near a Dirichlet boundary may lead to huge pumping rates with small computed drawdowns). Some of the constraints discussed above were used to minimize these problems. In fact, we started the optimization process with five management zones. Results showed that the first division was not suitable for linear programming. Initially, zones 3, 4 and 5 reached the coast. Since these zones contained pumping wells located close to the coast line where strict minimum head constraints were imposed, it was impossible to pump water from any of the three zones. Therefore, a long and thin additional management zone, the sixth in Figure 5.7a, was delineated parallel to the coast. This zone was defined as a dead zone in terms of water extration since the constraints prevented pumping.

Several optimization runs were required to obtain reasonable results, which were achieved by adding lateral constraints (Equation 5.3) to the pumping rates per management zone.

Results in terms of total pumping and its distribution per management zones are presented in Figure 5.8 together with current values (REF. Q in Figure 5.8) in the absence of corrective measures. Several optimization runs were required to obtain reasonable results, which were achieved by adding lateral constraints (Equation 5.3) to the pumping rates per management zone. In fact, when no lateral constraints are added (OPT1 in Figure 5.8), the total amount of water pumped turned out to be slightly above current values. However, the result is not acceptable in terms of the pumping distribution because most water must be extracted from the management zone, which is farther from the coast. This is caused by (1) the distance from the coast, which causes drawdown to be uniformly distributed and (2) by the presence of a Cauchy boundary condition (Figure 5.6),

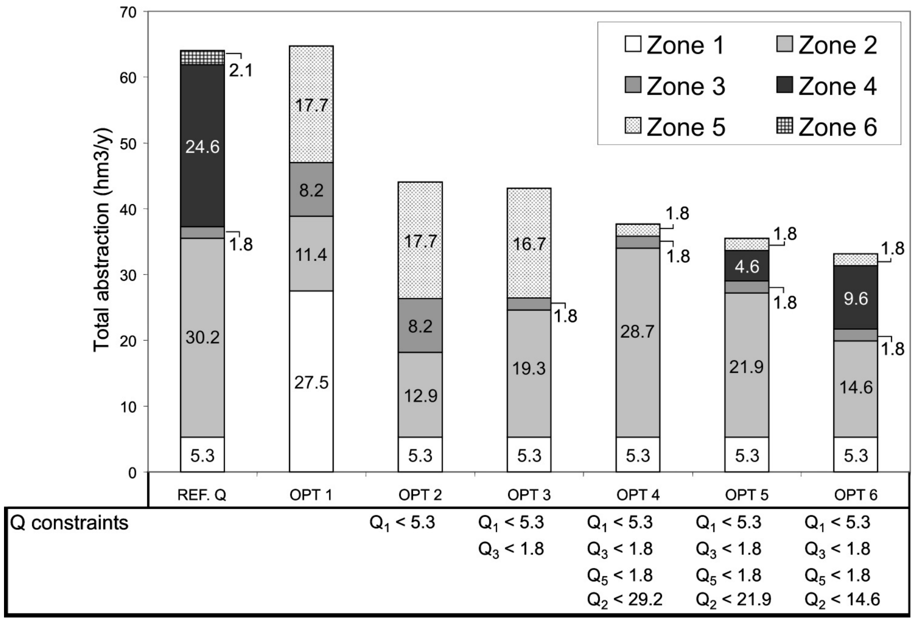
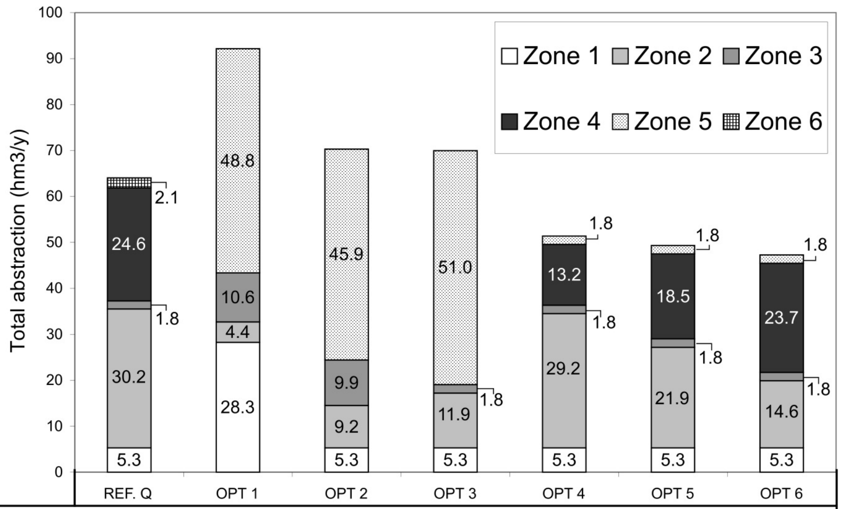


Figure 5.8: Results of the optimization of pumping rates for every management area and the total value of the objective function for different flow constraints applied. The reference values of the pumping rates are shown in the first bar.

which acts as an unrealistic source of freshwater. Therefore, this result cannot be considered to be reliable. Moreover, insufficient water can be pumped from the critical management areas (Zone 2 and 4, actually, zero from zone 4), where the main drinking supply companies are located. This solution is socially unacceptable with the result that *ad hoc* lateral constraints to pumping rates (i.e. maximum values) had to be imposed. When zone 1 was constrained (OPT 2 in Figure 5.8), a high pumping rate was assigned to zone 3. Nevertheless, pumping rates of zones 2 and 4 were below the required values, which gave rise to the subsequent runs. The addition of these constraints causes a reduction in the total objective function (OPT2-OPT6 in Figure 5.8). Only in the last two optimization problems (OPT5 and OPT6) can water be pumped from Zone 4. OPT6 is the most similar to the existing pumping distribution (REF. Q in Figure 5.8). A trade-off between Zone 2 and Zone 4 is clearly observed in OPT4-6. Constraining pumping in Zone 2 causes a pumping increase in Zone 4 together with a slight reduction in the total abstraction. Both zones are located over a high transmissivity zone (paleochannel C in Figure 5.3) and this trade-off shows how these zones compete for the same resource.



	REF. Q	OPT 1	OPT 2	OPT 3	OPT 4	OPT 5	OPT 6
Q constraints			$Q_1 < 5.3$	$Q_1 < 5.3$ $Q_3 < 1.8$	$Q_1 < 5.3$ $Q_3 < 1.8$ $Q_5 < 1.8$ $Q_2 < 29.2$	$Q_1 < 5.3$ $Q_3 < 1.8$ $Q_5 < 1.8$ $Q_2 < 21.9$	$Q_1 < 5.3$ $Q_3 < 1.8$ $Q_5 < 1.8$ $Q_2 < 14.6$
Shadow prices							
Ponds		0.85	0.86	0.95	0.69	0.69	0.69
Barrier		3.83	2.83	4.47	1.43	1.46	1.46

Figure 5.9: Results of the optimization with additional corrective measures of pumping rates for every management area and the total value of the objective function for different flow constraints applied. The reference values of the pumping rates are shown in the first bar.

The evolution of the total abstraction, the distribution of pumping and the shadow prices when additional corrective measures are implemented are presented in Figure 5.9. Total abstraction is smaller in Figure 5.8 because water is injected into the recharge ponds (10.96 hm^3/y) and into the seawater intrusion barrier (3.65 hm^3/y). The first three optimization scenarios yield total pumping above the reference values. However, in this case also, the distribution of pumping presents problems. According to optimization results, no water can be pumped from Zone 4 and most water must be pumped from Zone 5, where little water is currently pumped. This zone was added to the optimization process for possible use in future management scenarios. However, an underground structure acts as drainage in this zone. This drainage is implemented as a Cauchy boundary condition, which could act as an unrealistic source of water if the piezometric head should fall below the infrastructure base. In order to avoid this, pumping in zone 5 is constrained to 1.8 hm^3/y, which allows pumping from Zones 2 and 4, although total abstraction is significantly reduced. In

OPT4-6, the trade-off between Zones 2 and 4 is also observed. The most appropriate solution is OPT5 since it is the most similar to the existing distribution of abstraction. This result leads to a reduction of only a 25% in pumping rates of Zones 2 and 4.

The shadow prices obtained for the two additional corrective measures proposed are also shown in Figure 5.9. These shadow prices quantify the hydraulic efficiency of each corrective measure for each optimization scenario. The hydraulic efficiency of the recharge ponds is in all cases lower than 1. This means that some of the injected resource is lost before it gets to the active wells. This resource probably reaches the river (where it drains the aquifer) and the drainage underground structures located in management Zone 5. This is evidenced by the higher efficiency calculated when the maximum abstraction in Zone 5 is not constrained. The hydraulic efficiency calculated by the last three optimization cases (OPT4-6) is 0.69, i.e., almost 70% of the water recharged through the ponds can be pumped from the active wells. However, the calculated efficiency for the seawater intrusion barrier is higher than 1 in all cases. This is a striking result as efficiency higher than one means that the barrier allows pumping to increase by more than what is injected. Freshwater injection barriers are often criticized as corrective measures because part of the injected water flows seawards, causing part of the new resource to be lost. Although part of the injected water is indeed lost, our results show that the protective effect of the barrier is highly efficient. This barrier prevents a large amount of freshwater from being contaminated, resulting in an increase in available resources. The calculated hydraulic efficiency depends on lateral constraints. As in the case of the recharge ponds, the efficiency is higher when the maximum pumping rate in Zone 5 is not constrained (OPT1-3). In fact, it is linearly related to the pumping rate in Zone 5. It should be noted that the barrier was divided into 4 segments depending on the different transmissivity zones defined in the numerical model. The four segments are located in the management Zone 6 (Figure 5.7). However three of them are located in front of management Zone 4 and one in front of management zone 3 over paleochannel C in Figure 5.3. The optimization problem allows us to optimally distribute the total amount of water injected into the four segments. In scenarios OPT1-3, water is injected into the three segments located in front of Zone 5. However, when the maximum pumping rate in zone 5 is constrained, an abrupt decrease in the hydraulic efficiency is observed together with the reduction in the total abstraction. Furthermore, this is accompanied by a change in the segments where water is injected into the barrier. In cases OPT4-6, the optimal injection takes place exclusively in the segment located in front of management Zone 4, over a hydraulic preferential path (paleochannel C in Figure 5.3, which communicates the barrier with the main pumping areas in Zones 2 and 4.

Given the importance of maintaining a pumping distribution similar to the current one, the best solutions are obtained for OPT6 when no additional measures are considered and for OPT5 when the recharge pond and the injection barrier are added. In the former case, a reduction of more than the 50% of the pumping rates should be applied to Zones 2 and 4. In the latter, the reduction is limited to 25% in the same zones.

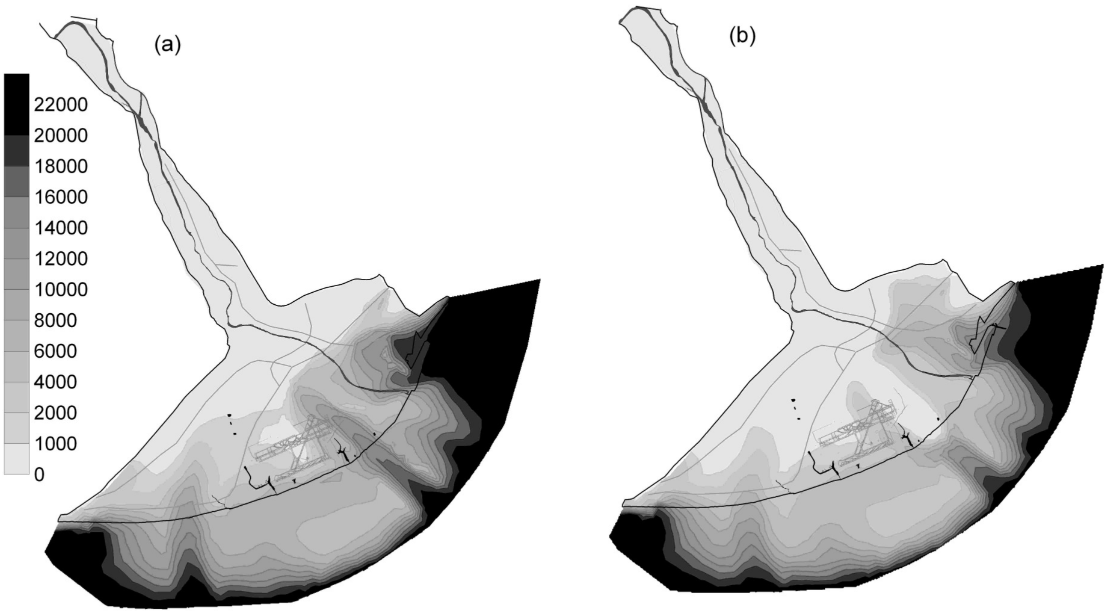


Figure 5.10: Chloride concentration map in the main aquifer of the Llobregat Lower Valley obtained (a) form the linearly optimized pumping rates and the implementation of corrective measures and (b) for the reference scenario

These optimal pumping rates guarantee that head objectives imposed as constraints are achieved after 35 years. As for seawater intrusion, head constraints at the coastal points ensure a reversal of the hydraulic gradient. However, this reversal may be not be sufficient to clean the aquifer areas that are currently contaminated. In order to confirm this, a transport simulation beginning with present chloride distribution was carried out with the optimal pumping rates calculated in OPT5 together with additional corrective measures. The concentration distribution obtained is shown in Figure 5.10b and compared with the chloride distribution calculated after 35 years of pumping according to the reference values (Figure 5.10a). An improvement in the quality is observed, but the existing salt continues to move towards the pumping or drainage areas. Although the pumping rates have been reduced and corrective measures imposed, the good qualitative status cannot be reached by simply optimizing hydraulic variables.

5.2.6 Non-linear optimization problem

The above results suggest that maximum salinity should be used as a constraint to guarantee the recovery of water quality. The resulting non-linear problem is presented below.

Pumping and injection rates are optimized so that a desired chloride concentration is reached at control points after 35 years. To ensure good water quality throughout the domain, control points were located along the coastline at the same 19 locations used in the linear optimization problem. Desired concentrations at the end of the 35 year management period, c^*, and tolerance, σ_{c_i}, (see equation 5.5) were set to 1000 mg/l of chloride. Therefore, final concentrations at the control points should not exceed 2000 mg/l. The desired recharge rate (R^* in Equation 5.6 was initially prescribed as the projected value. However, the penalty function was relaxed (large σ_R compared to σ_Q) to allow this variable to increase or reduce its value if necessary. In addition to the recharge ponds and injection barrier, two wells (Figure 5.6) were established for pumping 2500 m^3/day each during the first ten years to clean saltwater trapped by the barrier. Optimization parameters used are presented in Table 5.1.

Table 5.1: Statistical parameters used in the simulations

Parameter	Value	Units
λ_c	1.	
λ_Q	100.	
σ_{c_i}	1000.	mg/l
$\sigma_{Q_{well}}$	$0.1 \cdot Q_i$	m^3/day
$\sigma_{Q_{pond}}$	0.25	m/day
$\sigma_{Q_{barrier}}$	10000.	m^3/day

Management areas are those of the linear problem, except for Zone 6, which is no longer needed and was distributed among Zones 3, 4 and 5, as originally planned. Minimizing the objective function as presented in Equation 5.6, yields the values of Q_{well} for each pumping area and Q_R for the recharge measures.

Non-linear optimization results

Results are compared with reference pumping rates and projected recharge values in Figure 5.11. It should be noted that pumping can be maintained at current rates if corrective measures are applied with higher recharge rates than originally planned. Recharge should increase by 50% at the ponds and by 200% at the seawater intrusion barrier.

These results were obtained with the values of λ presented in Table 5.1, being λ_Q a hundred times greater than λ_c. When increasing the weight of the pumping rate function, pumping rates approach the desired values while relaxing concentration objectives. Concentration results in our case are twice the desired values at some of the control points remaining within the range of tolerance. A higher value of λ_c would result in lower concentrations to the detriment of the pumping rate objectives.

The concentration distribution of the transport simulation-optimization model after 35 years

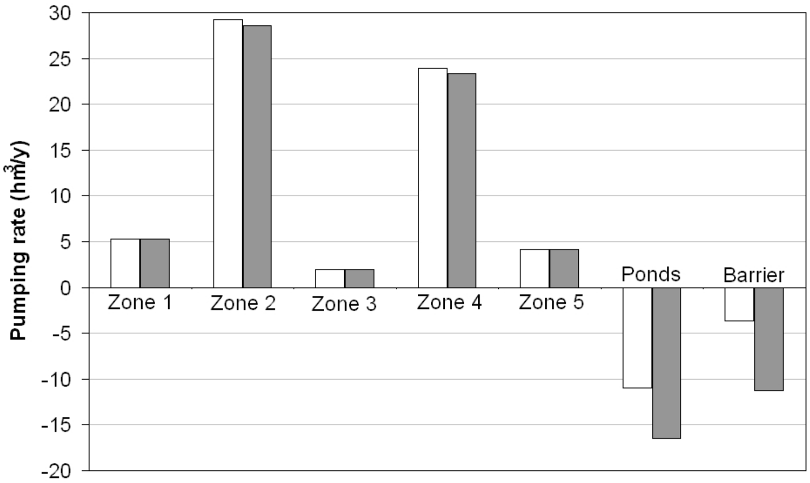


Figure 5.11: Results of the non-linear optimization problem. Obtained pumping rates per management zones and measures (gray bars) are compared to current values or projected values in the case of corrective measures (white bars).

is shown in Figure 5.12b to assess the effectiveness of the methodology. This result is compared with the chloride distribution calculated with the reference scenario, i.e., the current pumping rates (Figure 5.12a). The areas already contaminated located in the left margin of the delta are almost recovered with the optimized pumping and recharge rates. Some salty areas continue to be present, probably due to a poor choice of the location of the two pumping wells to extract salt trapped by the barrier or because the pumping period (ten years) was not long enough. A more detailed work would be necessary to design this pumping location and duration.

Another result that differs from the linear problem is the distribution of the injection water into the barrier segments. In the best linear result (OPT5 in Figure 5.9) optimal recharge at the barrier was concentrated in segment 4 (Figure 5.6), which is hydraulically more efficient since it coincides with a high permeability channel connected with inland pumping. However, in the non-linear problem, injection is equally distributed in order to meet quality criteria along the whole coastline.

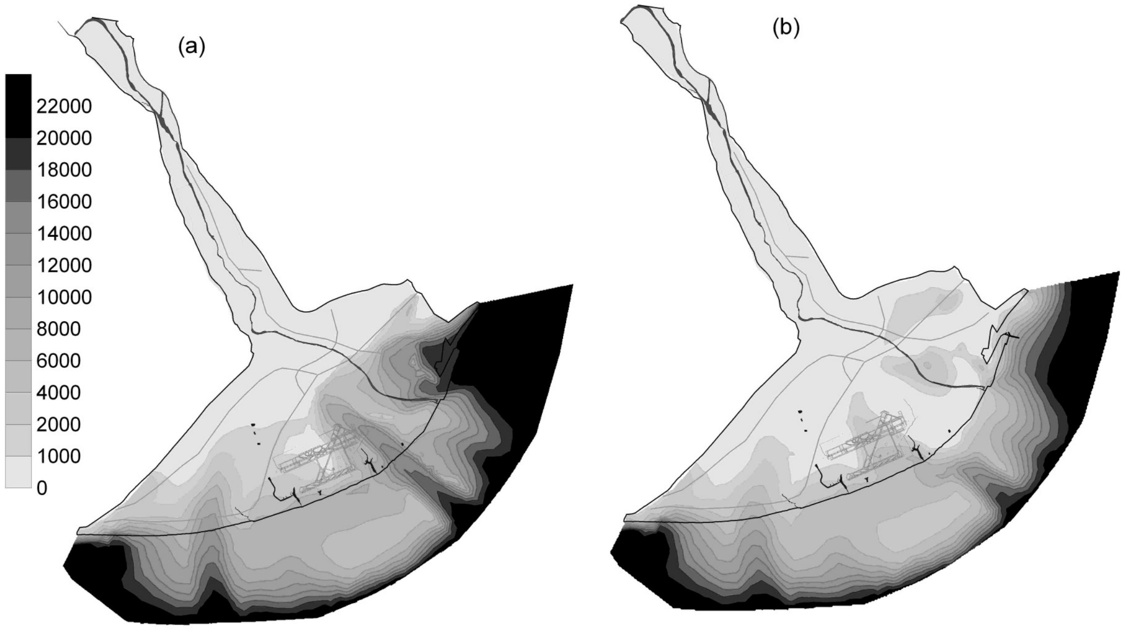


Figure 5.12: Chloride concentration map in the main aquifer of the Llobregat Lower Valley obtained (a) form the non-linearly optimized pumping rates and the implementation of corrective measures and (b) for the reference scenario

5.2.7 Comparison of results

The solutions obtained using both methods are presented in Table 5.2. Two solutions are offered for the linear optimization method: with (LOP+) and without (LOP) additional measures. A reduction of more than 50% of the pumping rates must be applied to Zones 2 and 4 when no additional measures are considered. If recharge ponds and the injection barrier are added, this reduction is limited to 25% in the same zones. These values would be suitable for managing an initially uncontaminated or unstressed aquifer. However, as the aquifer already suffers from considerable seawater contamination, these pumping reductions are not able to recover the quality of the aquifer. Nevertheless, this simple methodology yields information on the hydraulic efficiency of each corrective measure. In this case, the hydraulic efficiency for the ponds is always lower than 1 whereas for the seawater intrusion barrier it is about 1.5, resulting in a much more efficient measure.

Table 5.2: Comparison of the results obtained with the linear (LOP) and non-linear optimization process (NLOP) for the more realistic optimization scenarios (values in hm^3/y)

Management Zones	Average 1997-2001	LOP	LOP+	NLOP
Zone 1	5.29	5.28	5.28	5.29
Zone 2	30.23	21.92	21.92	28.62
Zone 3	1.77	1.83	1.83	1.98
Zone 4	24.59	4.64	18.49	23.38
Zone 5	0.00	1.83	1.83	4.16
Zone 6	2.14	0.00	0.0	–
Recharge ponds			-10.96	-16.45
Injection Barrier 1			-3.65	-2.68
Injection Barrier 2			0.00	-2.84
Injection Barrier 3			0.00	-1.27
Injection Barrier 4			0.00	-4.50
Total	64.02	35.48	49.35	63.41

The non-linear problem (NLOP) quantifies the amount of water to be artificially recharged to maintain the total current abstraction as well as its current distribution, while guaranteeing a groundwater quality recovery. Therefore, this method provides a satisfactory estimation of the injection rate that would be suitable for designing efficient corrective measures. Note that recharge rate values exceed the projected value both in the ponds and barrier, as shown in the results (Table 5.2).

The two methods lead to varying results (Table 5.2) since the application of alternative formulations leads to slightly different approaches and therefore to different solutions. However, both provide helpful and complementary information.

Table 5.3 contains a summary of the characteristics and main outcomes of each method as derived from their application to the main aquifer of the Llobregat delta. The most obvious difference is the definition of the objective function. The linear problem (LOP) aims at the maximization of the total pumping while the non-linear problem (NLOP) minimizes the differences with respect to

Table 5.3: Comparison of the two methods considered in this paper

	Opt. problem 1	Opt. problem 2
Objective	$\sum Q_i$ max	$F = \lambda_c \sum \frac{(c_i^* - c_i)^2}{\sigma^2_{c_i}} + \lambda_Q \sum \frac{(Q_j^* - Q_j)^2}{\sigma^2_{Q_j}} + \lambda_R \sum \frac{(R_k^* - R_k)^2}{\sigma^2_{R_k}}$
Constraints	$h_{j_{min}} \leq \sum_i \mathbf{A}_{ij}\, Q_j \leq h_{j_{max}}$ $Q_R \leq Q_{design}$ $Q_i \leq Q_{max\,i}$	$c_i = c_i\,(Q_i, Q_R)$ [Flow and transport equations]
Resulting problem	Linear	Non-linear
Results	• Optimal pumping regime to *prevent* seawater intrusion • Hydraulic efficiency of corrective measures ($\frac{\partial Q_{opt}}{\partial Q_R}$) • Trade-offs between management zones	• Optimal injection rate to *restore* a good qualitative status • Pumping rates close to current regime
Advantages and disadvantages	• Only applies to linear simulation models • *Ad hoc* changes needed (management zones redefinition and additional lateral constraints) • Does not ensure restoration	• Any simulation model • Previous sensitivity analysis to λs needed • Flexibility to obtain realistic results • Allows optimizing Q_R

current pumping values.

LOP effectively ensures that heads remain above sea level, thus ensuring that seawater will not penetrate. As such, it is suitable for evaluating sustainable pumping rates that prevent seawater intrusion. However, results cannot guarantee the recovery of an initially salinized aquifer.

The LOP was solved with a linear programming approach, a general optimization method. It is both simple and rapid. After some preliminary runs of the simulation model and once the response matrix is calculated, optimization runs are extremely rapid. This capacity makes it a very agile tool to test different optimization scenarios. Testing different possibilities is advisable since this method often yields unrealistic results. *Ad hoc* changes are often needed to adjust the optimization formulation. In our case, these changes consisted in the redefinition of management zones and in the addition of lateral constraints to the pumping rates of some management zones. However, the addition of constraints helps us to understand the trade-offs between management zones by revealing the areas that compete for the same resources, thereby providing guidelines for management. This is a valuable tool that can shed light on the behavior of the system and detect inconsistences in the simulation model that could lead to false optimization results.

As regards corrective measures, a number of points should be raised about the LOP methodology. First, in our methodology, recharge rates need to be constrained. Otherwise, the resulting optimization problem would be ill posed leading to an unlimited increase in both total pumping and recharge rates. Our implementation of LOP cannot quantify the optimal recharge rate in additional corrective measures such as recharge ponds and freshwater injection barriers. The resolution of this problem demands an economic analysis and the formulation of the problem in terms of maximizing the benefits of pumping minus the cost of recharge. The resulting optimization problem can be formulated in a linear programming framework provided that the objective function is convex. This option was not attempted here because we ignored the benefit functions of pumping and because our aim was to limit the changes to the present situation. This apparent disadvantage turned out to be of little importance because constraining corrective measures allowed us to identify their efficiency by means of shadow prices. Therefore, one could still assess the profitability of corrective measures, if necessary.

One of the main drawbacks of LOP is the lack of flexibility for the optimization problem due to the large amount of constraints often needed to obtain satisfactory results. Another important restriction is that this method is only applicable to linear simulation models.

The non-linear optimization method (NLOP) provides a reliable assessment of the required corrective measures needed to achieve a negligible deviation from the current pumping regime. Results are more complete than those of LOP since they guarantee the fulfilment of the quality objectives. Therefore, NLOP is useful to design remediation measures for aquifers undergoing seawater intrusion.

One advantage of NLOP is its applicability to any simulation model, either linear or non-linear. Given that the objective function is multiobjective, its main advantage is its flexibility. On the one hand, a simple change in the weights (λ_c, λ_Q and/or λ_R) redirects the objective of the optimization problem. On the other hand, varying the tolerance parameters (σ_c, σ_Q and/or σ_R) allows us to prioritize the accomplishment of the objectives in a particular management zone (reducing σ_{Q_j}), or, on the contrary, to relax the concentration constraint at specific locations (increasing σ_{c_i}). Performing sequential optimizations with varying λs and σs, the tradeoffs between objectives (pumping versus quality, one zone versus another, pumping versus corrective action, etc) can be analyzed. Flexibility in the formulation of NLOP also results in increased management flexibility. For example, one could ideally take advantage of the relatively large volume of aquifer below the sea for regulation purposes. It is possible to promote the displacement of the seawater front seawards in the confined portion of the aquifer during wet years. This leads to extended freshwater reserves that can be pumped during dry years. This possibility, which would have required the modification of the objective function, was not explored in our study since it demands a careful characterization of heterogeneities in the submarine portion of the aquifer. This kind of management scheme could not be achieved with LOP.

However, the simulation-optimization runs of NLOP are considerably longer than those of LOP (about 6 hours compared with a few seconds). Moreover, preliminary runs are needed to test the values of the weights of the objective function and of the tolerance parameters to obtain satisfactory results and to activate the constraints. Furthermore, NLOP may encounter a number of difficulties in converging when it starts from inappropriate values. This formulation provides a high degree of flexibility and control despite being fairly specific.

5.3 Conclusions

Two optimization methods, linear and non-linear, were successfully used to design management alternatives in the Llobregat delta Main aquifer. The application of these approaches to a real aquifer initially affected by seawater intrusion, and more specifically to a socially and hydraulically complex aquifer was a challenge in itself. These methods allowed us to:

- Evaluate the efficiency of corrective actions, as measured by means of the shadow prices derived from linear programming. Shadow prices represent a measure of the hydraulic efficiency of recharge corrective measures (i.e. the net increase in total pumping per unit increase in recharge). The hydraulic efficiency calculated for the recharge ponds is around 0.7. However, it is greater than 1 for the hydraulic barrier. This contradicts the widely held belief that coastal injection is inefficient because it loses water to the sea. While this is true, the net gain obtained from protection of inland pumping more than compensates for this loss.
- Quantify the optimal injection rate to be implemented in the ponds and in the barrier. This injection guarantees the recovery of aquifer quality without modifying the current pumping regime. This result was obtained through the non-linear optimization problem.
- Quantify the sustainable pumping rates to be applied once the aquifer quality has been recovered. Linear optimization of pumping and injection rates can be applicable to pristine aquifers in order to prevent seawater intrusion from re-occurring.

The combined use of these optimization approaches (linear and non-linear) to a real case allowed us to:

- Test the suitability of optimization methodologies for the design of corrective measures in aquifers suffering from problems of seawater intrusion
- Systematically compare both approaches identifying their main outcomes, advantages and applications.
- Improve our understanding of the system. The combination of methodologies provides insights into the system behavior and internal trade-offs. It is these insights rather than the

optimal results themselves that increase the confidence of decision makers and prompt them to act.

In short, linear and non-linear optimizations were tested as methodologies to design remediation measures for aquifers contaminated by seawater intrusion. We believe that they should be jointly used since they provide a comprehensive view of the problem and address a wide range of management questions.

Chapter 6

Conclusions

This chapter is a summary of the main contributions provided by this thesis. The main outcome responds to the general objective of advancing in the knowledge of the hydrodynamic processes that take place in coastal aquifers.

Seawater intrusion under natural conditions obeys the well known scheme of freshwater flowing seawards over a saltwater wedge. Saltwater in this wedge is not static but forms a convective cell. This scheme is better reproduced by the proposed dispersive problem than by the traditional Henry problem. This phenomenon is characterized not only by the interface penetration length, but also by the width of the mixing zone and by the saltwater flux entering the aquifer. The width of the mixing zone is an interesting parameter since it is easily measurable in vertical salinity profiles. The saltwater flux is important for characterizing reactive transport processes in the mixing zone. However, the quantification of this flux has been disregarded in most seawater intrusion studies until recent years (*Smith*, 2004; *Prieto*, 2005). Therefore emphasis has been placed on identifying the factors controlling these characteristic variables:

- The interface penetration can be evaluated as the theoretical penetration if there is no mixing (Ghyben-Herzberg approximation) minus a term that mainly depends on the geometric mean of the dispersion coefficients and the horizontal permeability (in anisotropic media).

- The width of the mixing zone is linearly related to the geometric mean of the dispersion coefficients. This is a relevant result with practical implications, the width of the mixing zone measured in vertical salinity profiles can be used to estimate the geometric mean of the dispersion coefficients. Longitudinal and transverse dispersion control different parts of the mixing zone but display similar overall effects. The longitudinal dispersivity increases the width of the mixing zone only at the bottom of the aquifer whereas the transverse dispersivity causes an increase in the interface slope accompanied by an overall widening of the mixing zone.

- The saltwater flux that enters the aquifer is mainly determined by the transverse dispersion coefficient and the permeability of the aquifer (the geometric mean in anisotropic aquifers). It must be remarked that it does not depend on the outflowing freshwater flux.

It is known that heterogeneity produces dispersion. Therefore, since seawater intrusion depends on dispersion, heterogeneity should be expected to affect the saltwater wedge. The effect of heterogeneity was studied by incorporating heterogeneity in the hydraulic permeability into the dispersive Henry problem. Different degrees of heterogeneity were considered. Independently of the considered heterogeneity, results show that heterogeneity causes the toe of the interface to recede while increases both the width and slope of the mixing zone. These displacements result in the rotation of the interface. This rotation could be explained by an increase in the dispersion coefficients, particularly in the transverse dispersion.

In general, the shape of the interface and the saltwater flux depend on the distribution of the permeability in each realization. The latter is highly dependent on the permeability distribution near the seaside boundary. The interface slope is low in high permeability zones and high in low permeability zones. Freshwater channeling takes place in the high permeability zones resulting in an accommodation of the interface under high permeability zones. However, high permeability zones well connected to the seawater boundary result in preferential paths for incoming seawater. Small convection cells are formed if these preferential paths are not well connected to other high permeability zones, but the overall aspect of the interface is not affected.

The toe penetration and the width of the mixing zone do not show large fluctuations. In fact, in cases of moderate heterogeneity ($\sigma^2_{lnK} \leq 1$), of either small or medium scale, satisfactory results can be obtained with homogeneous media with the equivalent values of the permeability and the local values of the dispersion/diffusion coefficients. When ergodic heterogeneous media with $\sigma^2_{lnK} \leq 1$ are considered, the effective dispersion coefficients calculated by means of perturbation theory can be applied to improve the results. However, they fail to reproduce the interface rotation, which implies that heterogeneity affects transverse dispersion more than what would be expected from the perturbation theory expressions for effective dispersivity.

Analyzing the separated effect of the correlation distance and the variance, results show that the critical factor is the variance whereas the mean value of toe penetration and the width of the mixing zone are not affected by changes in the correlation length. In cases of a high degree of heterogeneity (i.e., of large variance), heterogeneous structures need to be characterized by geological studies and must be explicitly represented in the models.

The previous conclusions were obtained modelling 2D vertical cross sections of the aquifer. The use of these models assumes that freshwater flows perpendicularly to the coast. This assumption may not be suitable to study seawater intrusion in horizontally large, confined aquifers with complex geometries. Four aquifer geometries were studied including seaward and lateral slopes. Seaward slopes do not affect the seawater intrusion behavior resulting in a saltwater wedge similar to the one developed by horizontal aquifers. However, lateral slope turned out to be a critical factor to understand seawater intrusion. Lateral slopes of more than 3% in the seaside boundary cause the development of convection cells whose main component is horizontal. These horizontal cells increase complexity in seawater intrusion behavior creating preferential zones for seawater to enter the aquifer (the deepest zones). The incoming seawater prevents freshwater to discharge in the deepest areas and diverts freshwater flow toward upward preferential discharging zones. Therefore, models in 2D vertical cross sections cannot be used to reproduce seawater intrusion in these type of aquifers. The dimensionless number, N_{by}, has been defined to estimate the relative importance of this effect. An expression equivalent to that of Ghyben-Herzberg has been obtained to get a first estimation of the interface position in this type of aquifers.

These insights have been integrated into the study of seawater intrusion in a real case, the Main aquifer of the Llobregat delta (Barcelona, Spain).

First, a deterministic numerical model of groundwater flow and chloride transport was developed integrating a thorough characterization of the geological structures. This model was calibrated and validated against both head and chloride data showing a satisfactory fit between calculated and measured data. Fingers observed in the salinity evolution were reproduced. This model was modified to become a management model to optimally design corrective measures to restore the water quality of the aquifer while minimizing changes in the actual pumping regime. The application of two different optimization methodologies, a linear and a non-linear optimization method, provided answers to a wide range of management questions. Shadow prices obtained from linear programming are a valuable tool to quantify the hydraulic efficiency of potential corrective measures to restore water quality in the aquifer. The resulting efficiency of the freshwater injection barrier was consistently larger than 1. This result is important by itself since it proves that the hydraulic barrier acts not only by increasing resources but also by protecting the existing ones. This result contradicts the widely extended belief that injecting freshwater to prevent seawater intrusion is a waste of resources.

The non-linear problem provides the optimal injection regimes into two recharge ponds and a freshwater injection barrier to recover the aquifer quality and maintain the current pumping regimen. The optimal injection rate in the barrier is about 11 hm^3/y, three times the initially projected value. The optimal recharge rate in the ponds is about 16 hm^3/y, also higher than the projected values.

The linear problem yields the sustainable pumping regimen to be applied once the aquifer quality has been recovered. The sustainable regime implies a reduction of more than 50% of the pumping rates in the main pumping areas if no additional measures were considered. If recharge ponds and the injection barrier are implemented, this reduction is limited to 25% in the same zones.

Appendix A

Tilted Ghyben-Herzberg Approximation

An approximate position of the saltwater wedge in coastal aquifers is usually calculated based on the Ghyben-Herzberg approximation in a cross section perpendicular to the coast. In aquifers with lateral slope this equation cannot be applied. Three-dimensionality of the flow field implies that freshwater flow is not perpendicular to the coast. However, a new approximation for thin laterally sloping aquifers can be used to obtain the tilted interface position. Two cases are considered here: (1) V-shaped aquifers and (2) Warped aquifers. The general domain is shown in Figure A.1.

Adopting the Dupuit-Forchheimer assumption, flow across the A-A' section, is given by Darcy's Law:

$$Q_x = TW(x)\frac{\partial h}{\partial x} \tag{A.1}$$

where Q_x is the freshwater flowing through the AA' section; T the aquifer transmissivity; $W(x) = (y_L - y)$ the aquifer width in which freshwater flows; y the interface position in y direction.

Ghyben-Herzberg assumptions of immobile saltwater and sharp interface (there is no mixing

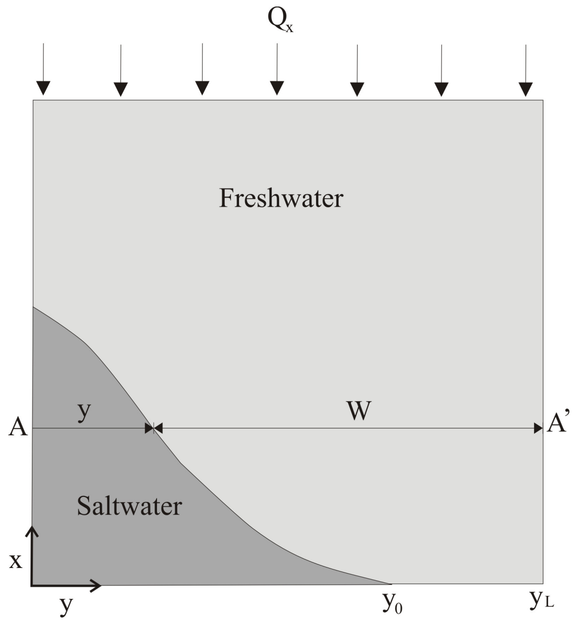


Figure A.1: Plan view of the aquifer domain and some of the variables that appear in the calculations

zone),imply:

$$h(x, y) = -\alpha z \tag{A.2}$$

where $z(x, y)$ is the elevation of the interface.

Expressing W in terms of the equation of the aquifer surface, imposing that the interface must satisfy A.2 and integrating in x yields the equation for the interface as shown below.

A.1 V–shaped Aquifers

In aquifers described as V-shaped and shown as Case 3 in Figure 4.3, the geometry of the boundary planes have the following expression:

$$z - z_L = m(y - y_L) = mW \tag{A.3}$$

where m is the plane's slope.

Solving (A.3) for W, substituting z by means of (A.2) and integrating along the flow direction(x) leads to:

$$m \int_0^x Q_x dx' = T \int_{h_0}^{h} (\frac{h'(x)}{\alpha} + z_L) dh' \tag{A.4}$$

$$m Q_x x = T \left[\frac{h^2 - h_0^2}{2\alpha} + (h - h_0) z_L \right] \tag{A.5}$$

where h_0 is the freshwater head ($z_0\alpha$) at the point where the interface intersects the seaside boundary. Q_x is assumed to be constant (i.e. there is no areal recharge). Again using A.2 for h and A.3 for z yields the areal equation of the interface

$$x = \frac{Tm\alpha}{2Q} \left[(y - y_L)^2 - (y_0 - y_L)^2 \right] \tag{A.6}$$

where y_0 is the y value at the interface intersection with the coast line. A priori, one does no know y_0 and several approximations need to be made:

1. If the interface intersects the coast line in y_L, then

$$x = \frac{Tm\alpha}{2Q} (y - y_L)^2 \tag{A.7}$$

2. If the interface intersects the coast line somewhere between $y = 0$ and y_L, A.6 must be applied.

A.2 Warped Aquifers

In Warped aquifers shown as Case 4 in Figure 4.3, the geometry of the boundary surfaces has the following expression:

$$z - z_L = -\frac{m}{L} (y_L - y)(x_L - x) \tag{A.8}$$

where m is the slope in x and y direction and L the model domain length.

Following the same procedure as above, the final expression for the interface equation for warped aquifers is:

$$x = x_L \left(1 - \sqrt{\frac{LQ + T\alpha m (y_L - y_0)^2}{LQ + T\alpha m (y_L - y)^2}} \right) \tag{A.9}$$

As above, two different approximations for y_0 can be adopted. First, assume that the interface intersects the coast line in y_L, in which case the interface equation simplifies to

$$x = x_L \left(1 - \sqrt{\frac{LQ}{LQ + T\alpha m (y_L - y)^2}} \right) \tag{A.10}$$

Second, assume that the interface intersects the coast line somewhere between $y = 0$ and y_L, in which case A.9 must be used.

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