Conceptual Model for the Safe Withdrawal of Freshwater from Coastal Aquifers

A. Mahesha[1]

Abstract: The effect of subsurface barrier on the motion of the saltwater—freshwater interface in coastal aquifers is analyzed for wide ranging freshwater pumping scenarios. A Galerkin finite-element model considering sharp interface approach is used for this purpose. A semi-pervious subsurface barrier extending up to impervious bottom of the aquifer is considered at certain distance inland, parallel to the seacoast. The effect of barrier is analyzed in checking the advancement of the saltwater-freshwater interface under different scenarios of freshwater withdrawals at seaward and landward locations of the barrier and compared with nonbarrier conditions. The results indicated that barrier is able to check the advancement of the intrusion significantly and in certain cases, the progress is completely stalled for withdrawals on the landward side. Also, marked variations in the interface profile are observed as compared to no barrier condition, especially, for the seaward freshwater developments. From the model, nearest possible locations from the seacoast have been worked out for the safe withdrawal of freshwater where their effects are negligible on the saltwater advancement.

DOI: 10.1061/(ASCE)EE.1943-7870.0000081

CE Database subject headings: Coastal environment; Aquifers; Numerical models; Salt water; Fresh water; Salt water intrusion.

Introduction

Under natural conditions, when a pervious formation outcrops into sea, the equilibrium between saltwater and freshwater is being established in the form of saltwater intrusion. There are probably a few places around the world where this is or this can be the case, and some degree of intrusion may be acceptable without significant loss of fresh groundwater resources. However, modernization and advanced technologies enable extraction of water from even deepest aquifers to meet the ever increasing demand for freshwater. As a result, in recent years overexploitation of coastal aquifers has aggravated the problem of saltwater intrusion in many countries. Owing to this, strategies to control saltwater intrusion are being taken up by the government/nongovernment agencies using different techniques. Sincere efforts were made to the control of saltwater intrusion since 1960s and significant field studies reported include status of saltwater intrusion (Atkinson et al. 1986; Barlow 2003), recharged well battery (Bruington and Seares 1965), injection-extraction system (Sheahan 1977), groundwater barrier and recharge (Williams 1977), and physical barriers (Sugio et al. 1987). Theoretical investigations to support the field studies were also carried out by several researchers (Bear et al. 1999; Hunt 1985; Kashef 1976; Mahesha 1996; Schroeder et al. 1989) along with management solutions (Cheng and Ouazar 2003; Nutbrown 1976; Reichard and Johnson 2005; Shamir et al. 1984; Willis and Finney 1988). Several management, institutional, and legal enforcements will add to the above measures in maintaining an efficient aquifer management strategy and hence preventing further damage to the freshwater aquifer and allowing optimal development. A comprehensive report on the historical development and research on saltwater intrusion can be found in Reilly and Goodman (1985) and Bear et al. (1999).

Out of the large body of literature available on saltwater intrusion the focus here is on the effectiveness of subsurface barriers in preventing saltwater intrusion. Subsurface barriers are considered to be one of the solutions for the control of saltwater intrusion (Todd 1974). Inadequate supply of freshwater for recharge, wastage of substantial amount of freshwater, and regular maintenance of wells required have been made to look into physical barriers. In this method, saline water inflow into freshwater basin is prevented by physical impervious or semipervious subsurface barriers extending over the entire depth of aquifer. The extent of control over intrusion by these barriers is primarily dependent on the location, depth, and permeability of the barriers. Suitable materials for the barrier (Todd 1974) include cement grout, sheet pile, puddled clay, bentonite, emulsified asphalt, montan wax, silica gel, calcium acrylate, or plastics singly or in combination. Todd (1980) lists three major obstacles for the implementation of subsurface barriers. They are construction cost, resistance to earthquakes, and chemical erosion. The cost of barrier depends on the width, depth, and hydraulic conductivity (type of material). The cost of soil-bentonite slurry barrier (one of the economical barrier) in soft to medium soil ranges from \$540 to \$750 (1991 currency value) per square meter excluding testing costs (U.S. Army Environmental Center 2002). The normal depth range for such barriers is 30 m and width of 0.1–1.2 m (Basri 2001; USAEC 2002). For depths greater than 30 m, cost per unit area increases by a factor of three which is the major limitation of subsurface barriers (Hanson and Nilsson 1986; Ragoshewski et al. 1983; Todd 1980; U.S. Environmental Protection Agency 1987). Osuga (1996) reports that if the width of the barrier can be minimized the construction cost may be reduced. Also, in general, less

[1]Professor, Dept. of Applied Mechanics and Hydraulics, National Institute of Technology, Karnataka Surathkal, Mangalore 575 025, India. E-mail: maheshamai@yahoo.com

Note. This manuscript was submitted on July 22, 2008; approved on March 18, 2009; published online on March 20, 2009. Discussion period open until March 1, 2010; separate discussions must be submitted for individual papers. This paper is part of the ***Journal of Environmental Engineering***, Vol. 135, No. 10, October 1, 2009. ©ASCE, ISSN 0733-9372/2009/10-980–988/\$25.00.

J. Environ. Eng., 2009, 135(10): 980-988

permeable barrier costs more due to the volume of the construction material required (Basri 2001).

Knox (1983) discusses the effectiveness of physical barriers including slurry walls, grout cutoffs, and sheet piles in preventing the movement of contaminated groundwater. The analytical and numerical solutions obtained by him for the movement of contaminated groundwater under or through subsurface barriers showed the dependence on permeability of the barrier, depth to impermeable formation, and the joint between the barrier and the underlying formation. Sugio and Nakada (1984) and Sugio et al. (1987) described a finite difference model with practical design and management strategy of a semipervious (about 3×10^{-4} times aquifer conductivity) subsurface barrier for unconfined coastal aquifers of Okinawa-Jima island of Japan. The barrier was found to be effective in preventing saltwater intrusion for about 2 months under extreme conditions of total drought and continuous pumping. Nagata et al. (1993) reported on 11 subsurface barriers in Japan either to increase groundwater storage or to control saltwater intrusion. Later, Nagata and Kawasaki (1997) reviewed the construction methodology for subsurface barriers. They reported about seven subsurface barriers with a depth range of 11–25 m, length of 60–1,835 m which have been constructed in Japan and four other barriers with a depth range of 36–81 m and length of 1088–2489 m under construction.

The determination of shape and position of the saltwater-freshwater interface in a coastal aquifer by solving the mass balance equation in three-dimensional space with the nonlinear interface boundary conditions may not always be feasible due to the nature and quantum of data available. In this regard, the sharp interface approach in conjunction with integration over the vertical can be reasonable and be applied to large physical systems (Bear 1977). Even though this approach does not give information concerning the nature of the transition zone, it reproduces the regional flow dynamics of the system and response of the saltwater-freshwater interface to applied stresses (Essaid 1990). However, vertically integrated sharp interface models do not account for the transient behavior of regimes exhibiting significant vertical flow and/or dispersion (Hill 1988; Pinder and Stothoff 1988).

In the present work, sharp interface approach is followed in carrying out parametric studies on the behavior of saltwater-freshwater interface subjected to freshwater withdrawals in the presence of semipervious subsurface barrier running parallel to the coast. The semipervious nature of the barrier was recommended by several researchers in the past with their experience in the field. The semipervious nature of the barrier is found to be useful in avoiding the impoundment of large quantum of freshwater on the landward side of the barrier and also to avoid the accumulation of agricultural fertilizers. This work compares the results of performance of the barriers with nonbarrier cases. The main objective of this work is to monitor the landward progress of saltwater under different levels of stresses on freshwater aquifers. Also, having analyzed this, to work out the "safe distance" from the seacoast beyond which freshwater withdrawals have no effect on the saltwater-freshwater interface.

Conceptual Model

Geometry of Flow Domain

The geometry of flow domain is given in Fig. 1. The figure shows a coastal phreatic aquifer with two distinct regions. One is the region where only the freshwater flows in the vertical cross section of the aquifer, termed as freshwater region extending up to the landward end, i.e., up to a distance of four times initial length of saltwater intrusion. The other is freshwater-saltwater region, where freshwater outflows into the sea above the saltwater separated by a distinct interface. The saltwater-freshwater interface is under equilibrium due to constant seaward flow of freshwater, Q_I, and no areal recharge. The extent of saltwater intrusion be L_0 from the sea coast. Under these conditions, a semipervious barrier of thickness B_w extending up to impermeable aquifer bottom is considered at a distance B_L $(=L_0)$ from the sea face. The dependent variables in the present problem are the freshwater and saltwater heads which are expressed in the coupled form as given below.

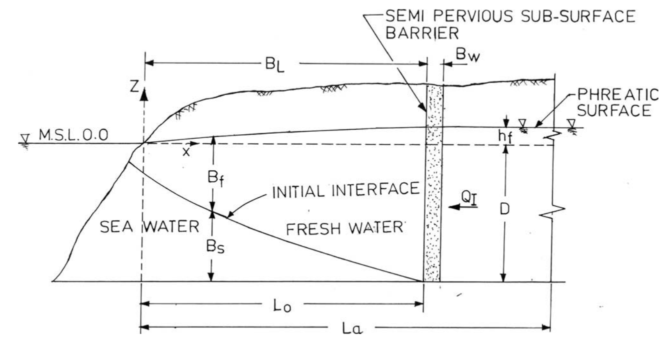


Fig. 1. Definition sketch of the problem

Governing Equations

The transient, one-dimensional, nonlinear, coupled, partial differential equations for freshwater and saltwater flow are (Bear 1979)

$$\frac{\partial}{\partial \mathrm{x}}\left(K_f B_f \frac{\partial h_f}{\partial x}\right)+\frac{n}{\alpha}(1+\alpha)\frac{\partial h_s}{\partial t}-\left[n\left(\theta+\frac{1}{\alpha}\right)+S_f\right]\frac{\partial h_f}{\partial t}$$
$$+Q_f\delta(x-x_0,y-y_0)=0 \tag{1a}$$

$$\frac{\partial}{\partial x}\left(K_s B_s \frac{\partial h_s}{\partial x}\right)-\left[\frac{n}{\alpha}(1+\alpha)+S_s\right]\frac{\partial h_s}{\partial t}+\frac{n}{\alpha}\frac{\partial h_f}{\partial t}=0 \tag{1b}$$

where K=hydraulic conductivity $(K_f=K_s)$(L/T); B=saturated thickness (L); h=vertically averaged piezometric head (L); n=porosity; α=excess density ratio$=(\rho_s-\rho_f)/\rho_f$; ρ=density; θ=a coefficient (=1 for unconfined aquifers and =0 for confined aquifers); Q_f=freshwater pumping rate (L^3/T); $\delta(x,y)$=Dirac delta function; t=time (T); x=coordinate axis (L); S=specific yield; and subscripts f and s refer to freshwater and saltwater, respectively. The above problem involves the following assumptions: (1) sharp interface between saltwater and freshwater; (2) validity of Darcy's law and Dupuit's approximations; (3) saturated flow in the region; (4) specific storage and horizontal permeability variations along the vertical are negligible; (5) vertical outflow face is assumed for saltwater and freshwater at seaward boundary during the simulation period; and (6) both freshwater and saltwater are homogeneous and isotropic fluids with constant density and viscosity. The additional assumptions for the present work are: (1) the drawdown resulting from pumping is from fully penetrating wells; (2) the simulations are conducted on two dimensional cross section of the aquifer; and (3) the initial position of saline wedge (L_0) is idealized in a nonstressed equilibrium aquifer, and subse-

J. Environ. Eng., 2009, 135(10): 980-988

quent pumping is sudden and instantaneous in relation to the starting position of the interface.

The freshwater interface storage term $[n(\theta+1/\alpha)+S_f]\partial h_f/\partial t$ in Eq. (1a) represents the amount of water that would be released from the storage if saltwater heads adjust to equilibrium instantaneously and the coupling term $n/\alpha(1+\alpha)\partial h_s/\partial t$ represents the impact of flow in the saltwater zone on head distributions in the freshwater zone (Essaid 1986). Similar explanation can be brought in for head distribution in the saltwater zone and hence the movement of the interface. The continuity of pressure across the interface leads to the following relationship between the interface elevation and the state vector which is used to compute the interface elevation from the nodal piezometric head values of saltwater and freshwater (Bear 1979)

$$z=\frac{1}{\alpha}[(1+\alpha)-h_f] \tag{2}$$

where z=interface elevation (L), positive upward from the mean sea level, i.e., 0.0.

Initial Conditions

The initial condition for the studies is assumed to be steady and is computed using the steady state analytical solutions (Rumer and Harleman 1963). The datum is at sea surface (zero level) and the saltwater head is fixed at this level at the two boundaries during the simulation. The initial conditions for various parameters are given by the following relationships:

1. Initial water table (freshwater) profile:

   The initial steady water table profile is assumed as per Rumer and Harleman (1963)

$$h_f(x,0)=\left[\frac{2\alpha Q_I x}{(1+\alpha)K}+0.52\left(\frac{Q_I}{K}\right)^2\right]^{1/2} \tag{3a}$$

   The above relationship requires data on seaward flow of freshwater which may be estimated from measurements or from the Darcy's law. However, in the absence of such data, equilibrium water level data along a line perpendicular to sea coast may be considered as initial water table profile.

2. Initial interface profile:

   The interface profile is computed as per Eq. (2). However, since nodal values of initial saltwater heads are 0, Eq. (2) reduces to

$$z(x,0)=-\frac{h_f(x,0)}{\alpha} \tag{3b}$$

3. Initial length of intrusion:

   Initial length of intrusion is a crucial parameter in the analysis and may be computed using the relation (Rumer and Harleman 1963) as

$$L_0(x)=(1+\alpha)\left[\frac{\alpha K D^2}{2Q_I}-0.26\frac{Q_I}{\alpha K}\right] \tag{3c}$$

   where L_0=initial length of intrusion from the sea (L); Q_I =initial seaward flow of freshwater per unit width (L^3/T) which is assumed to be constant with time for all the simulations; and D=depth of aquifer below mean sea level (L).

4. Initial saltwater head:

   Initial saltwater level is taken as equal to mean sea level i.e., 0 m over the entire domain

$$h_s(x,0)=h_s(L_a,0)=0.0 \tag{3d}$$

   where x=distance (L), positive toward right and L_a=length of aquifer perpendicular to the coast (L).

Boundary Conditions

A small seepage face (Dirichlet boundary) is assumed as per Eq. (4a) as freshwater boundary condition (Charmonman 1965) depending on the freshwater discharge into the sea. The major assumptions in deriving the above seepage face are (1) the aquifer is homogeneous; (2) Dupuit's assumption is valid in the region; (3) confining bed is absent in the aquifer except at the bottom; and (4) the seaward flow of freshwater exists in the region

$$h_f(o,t)=\frac{0.722Q_{ft}}{\alpha K} \tag{4a}$$

where Q_{ft}=seaward freshwater discharge at the wedge toe (L^3/T). Initially, $Q_{ft}=Q_I$, assuming initial steady state flow. For $t>0$, Q_{ft} is computed using Darcy's law considering the gradient around the toe of the interface, hydraulic conductivity, and the depth to impermeable layer. If landward flow of freshwater exists around the wedge toe, the above relationship is not applicable and zero hydraulic head is considered at the seaward end.

The landward freshwater discharge may be variable depending on the site specific conditions. Hence, a constant flux boundary (Neuman) is introduced for freshwater at the landward end [Eq. (4b)]. However, in this study, the landward boundary is fixed at the sufficiently large distance (x=4 L_0) so that, the boundary effect is minimal on the well field and the interface motion. A separate study was conducted to analyze the sensitivity of type of boundary condition on the well field and it was found that beyond certain distance, i.e., 4 L_0 from the seacoast (in the present case), the results do not get affected either due to free boundary or fixed boundary

$$\frac{\partial h_f(x,t)}{\partial x}=Q_I \tag{4b}$$

Dirichlet conditions are imposed for saltwater head at the seaward and landward boundary as follows:

$$h_s(0,t)=h_s(L_a,t)=0.0 \tag{4c}$$

Since the saltwater head is not fixed at intermittent nodal points, saltwater is under dynamic conditions.

Numerical Solution by Finite-Element Method

The Galerkin weighted residual method is used to obtain the element equations. The final form of governing equations can be expressed as

$$[S]\{\dot{H}\}+[C]\{H\}+\{F\}=0 \tag{5}$$

where $[S]$=storage matrix, which is made symmetric by scaling (6) by a factor $(1+\alpha)$; $[C]$=conductivity matrix; $\{F\}$=flux vector; and $\{H\}$=state vector. The matrices are symmetric and banded in

J. Environ. Eng., 2009, 135(10): 980-988

nature. The elements of the matrices are as follows:

$$s_{ij}=\begin{Bmatrix} \int_{\Omega}\left[n\left(\theta+\frac{1}{\alpha}\right)+S_f\right]N_iN_j d\Omega & -\int_{\Omega}\frac{n}{\alpha}(1+\alpha)N_iN_j d\Omega \\ -\int_{\Omega}\frac{n}{\alpha}(1+\alpha)N_iN_j d\Omega & (1+\alpha)\int_{\Omega}\left[\frac{n}{\alpha}(1+\alpha)+S_s\right]N_iN_j d\Omega \end{Bmatrix} \quad (6)$$

$$c_{ij}=\begin{bmatrix} \int_{\Omega}K_{fij}B_f\frac{\partial N_i}{\partial x_i}\frac{\partial N_j}{\partial x_j}d\Omega & 0 \\ 0 & \int_{\Omega}K_{sij}B_s\frac{\partial N_i}{\partial x_i}\frac{\partial N_j}{\partial x_j}d\Omega \end{bmatrix} \quad (7)$$

$$f_i=\begin{Bmatrix} -Q_{fi} \\ 0 \end{Bmatrix} \quad (8)$$

$$h_i=\begin{Bmatrix} h_{fi} \\ h_{si} \end{Bmatrix} \quad (9)$$

where N_i and N_j are the shape functions and $i,j=1,2$. The time derivative is discretized using the finite difference scheme as

$$\{\dot{H}\}=\frac{\{H\}_{t+\Delta t}-\{H\}_t}{\Delta t} \quad (10)$$

where Δt=time increment. A thin layer of saltwater (thickness $=B\times10^{-3}$) and nonzero hydraulic conductivity ($=K\times10^{-4}$) are assumed in the entirely freshwater zone to retain the positive definiteness of the conductivity matrix [Eq. (7)] without affecting the actual solution (Sa da Costa and Wilson 1979). The domain is discretized into 128 elements with finer mesh around the drawdown locations, subsurface barrier location, and near the outflow face where the piezometric surface and interface profile are highly nonlinear. Nodal values of hydraulic conductivity are given as input to represent barrier condition or homogeneous aquifer.

The final set of algebraic equations is solved iteratively using the Newton-Raphson method. Convergence of the numerical process is checked by comparing the values of the state variables between successive iterations

$$\frac{\sqrt{\sum_{j=1}^{\mathrm{NN}}h^{2(i)}}-\sqrt{\sum_{j=1}^{\mathrm{NN}}h^{2(i-1)}}}{\sqrt{\sum_{j=1}^{\mathrm{NN}}h^{2(1)}}}\le \text{tolerence}(=0.001) \quad (11)$$

where NN=number of nodes and i=iteration index. Usually the solution converges in one or two iterations, even though a maximum of 10 iterations are allowed. If the ratio is greater than the previous, the solution diverges and the execution ceases. If the solution does not converge within the above-specified limit, the solution is sought with smaller time scale or refined grid system.

The intersection of the interface with the bottom of the aquifer is referred to as the toe of the interface. It need not necessarily coincide with the element nodes. In such cases, a simple linear interpolation technique is adopted (Mahesha 1996) to track the toe of the interface. The distance between the outflow face and toe of the interface is the length of intrusion. Another convergence criterion is adopted to check the attainment of the steady state in terms of length of intrusion

Table 1. Aquifer Properties

Serial number	Property	Value
1	Hydraulic conductivity, K	30 m/day
2	Depth of aquifer below mean sea level, D	100 m
3	Porosity, n	0.4
4	Specific yield, $S_f=S_s=S$	0.004
5	Initial seaward flow of freshwater, Q_I	5 m^2/day
6	Density of freshwater, ρ_f	1 g/mL
7	Density of seawater, ρ_s	1.025 g/mL
8	Time step, Δt	0.5 day

$$\frac{|L_t-L_{t-1}|}{|L_0|}\times 100\le \text{tolerence}(=0.001) \quad (12)$$

where the subscript for L indicates the time level. The details of the model, solution procedure and the validation are outlined elsewhere (Mahesha 1995, 1996; Sa da Costa and Wilson 1979).

Earlier studies (Mohan Babu 1999) have indicated that barrier with conductivity $K'\ge 0.1$ K has no effect on the motion of the interface and behaves like homogenous aquifer. Also, large barrier widths ($\ge$0.5 m) may not be economical considering the depth up to which they have to be extended (aquifer bed). Hence in the present study, detailed analysis was carried out with barrier located at L_0 having a width of 0.3 m and conductivity of equal to 0.001 K. The barrier is considered to be continuous without any joints or gaps. In practice, usually these barriers extend up to several hundred meters and are reported to be without gaps or joints (Nagata et al. 1993; Nagata and Kawasaki 1997). The freshwater withdrawals are simulated at various locations (0.25 L_0, 0.5 L_0, 0.75 L_0, 1.25 L_0, 1.5 L_0, 1.75 L_0, 2.0 L_0, 2.25 L_0, and 2.5 L_0) with different rates, Q_f'=0.005 to 0.25 where the nondimensional pumping rate is defined as

$$Q_f'=\frac{Q_f}{\alpha KD^2} \quad (13a)$$

For the present aquifer considered, the above rates would be 37.5–1,875 m^3/day, respectively. Under the above scenarios of freshwater withdrawal at the specific locations, the saltwater-freshwater interface is monitored till a new equilibrium is attained, i.e., advancement of the interface <0.001 [as per Eq. (12)]. The nondimensional time factor is expressed as

$$t'=\frac{T_0 t}{nL_0^2} \quad (13b)$$

where T_0=average initial coefficient of aquifer transmissivity (L^2/T); n=porosity; L_0=initial saltwater intrusion (L); and t =time elapsed since start of pumping.

In all the above cases, performance of the barrier is compared with homogeneous (nonbarrier) condition. The coastal aquifer considered for the analysis had the properties as listed in Table 1. From the aquifer properties considered, as per Eq. (3c), the initial steady intrusion length L_0=767 m. The length of the aquifer perpendicular to the coast L_a is kept sufficiently large (=4 L_0) to minimize boundary effect on the well field. The effect of boundary at such a large distance is found to be negligible on the pumping and related interface motion.

J. Environ. Eng., 2009, 135(10): 980-988

Table 2. Freshwater Withdrawal on the Intruded Zone of Coastal Aquifer $B_L = L_0$, $B_w = 0.3$ m, and $K' = 0.001$ K

Withdrawal location (x/L_0)	Withdrawal rate Q_f'	With barrier: Water level $h' = (h/d) \times 100$	With barrier: Time to attain steady state (t') Tt/nL_0^2	Without barrier (homogenous): Water level $h' = (h/D) \times 100$	Without barrier (homogenous): Time to attain steady state (t') Tt/nL_0^2
0.25	0.005	0.38	0.025 (1.9 days)	−0.04	0.061 (4.6 days)
0.50	0.005	0.91	0.026 (2.0 days)	0.87	0.029 (2.2 days)
0.75	0.005	−3.54[a]	1.966 (149.4 days)	1.95	0.002 (0.2 days)
0.25	0.01	−0.46	0.024 (1.8 days)	−1.06	0.045 (3.4 days)
0.50	0.01	0.00	0.028 (2.1 days)	0.77	11.4 (866.4 days)
0.75	0.01	−9.52[a]	1.659 (126.1 days)	−6.70[a]	0.89 (67.6 days)

[a]Severe upconing of saltwater at withdrawal location.

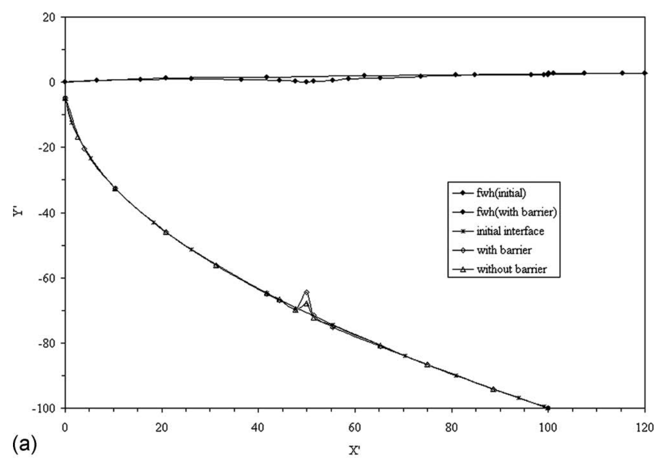
Results and Discussion

Withdrawal in the Two-Fluid Zone ($x \leq L_0$)

In this zone, the freshwater is overlying the saltwater as shown in Fig. 1 which may be termed as two fluid (saltwater intruded) zone. The scope for freshwater development in this zone is explored considering different freshwater withdrawal rates. Table 2 gives the details on freshwater withdrawals in the intruded zone. The withdrawal rate considered in this zone is relatively small i.e., $Q_f' = 0.005 - 0.01$ and the advancement of saltwater intrusion observed is negligible for the barrier and nonbarrier cases. Instead, the saline wedge shows upconing trend with increased magnitudes of pumping. Fig. 2(a) shows one such case for $Q_f' = 0.005$ in which the saltwater upconing is mild. The presence of barrier at L_0 was found to be disadvantageous since it prevents free flow of fresh water from landward side toward the withdrawal location as evident from the cones of depression observed in both the cases as shown in Fig. 2(b). The withdrawals at 0.75 L_0 require longer time to establish equilibrium due to the presence of barrier nearby and result in larger drawdown. In Fig. 3, severe saltwater upconing is observed for withdrawal rate of $Q_f' = 0.01$ at 0.75 L_0 and is not advisable to draw at this rate considering miscibility between freshwater and saltwater. From the various cases considered, it was found that withdrawal rate in this zone should be preferably less than $Q_f' = 0.005$.

Withdrawal in the One-Fluid (Freshwater) Zone ($x \geq L_0$)

The efficacy of the subsurface barrier is really tested during the groundwater exploration on the landward side of the barrier. The advancement of intrusion is being prevented/delayed by the barrier depending on the magnitude of withdrawal at any given location. Table 3 highlights the scenarios of reduced water levels and saltwater advancement due to various rates of freshwater draft on the landward side of the barrier. Fig. 4 shows the piezometric and interface profiles for withdrawal at 1.25 L_0 at the rate of $Q' = 0.05$. Due to the presence of barrier, free flow of freshwater from the seaward side of the withdrawal location is reduced resulting in a narrower cone of depression compared to no barrier case. However, the barrier arrests the advancement of the saline wedge from its original position. For increased rates of withdrawal, the cone of depression reaches the aquifer bottom before equilibrium is established and pumping need to be stopped at this stage. Such cases are indicated by table footnote "b" in Table 3. However, saltwater continues to advance for a while till the steady state is reached which is also indicated in the table. In the absence of barrier, the saltwater advances into the withdrawal location as shown in Fig. 4. Fig. 5 highlights the effectiveness of the barrier for a higher withdrawal rate of 0.1 at 1.50 L_0 by restricting the saline wedge at the original position itself. The interface and piezometric profiles for various withdrawal rates at 1.50 L_0 are presented in Figs. 6(a and b). The position of saline wedge is almost stationary without any advancement for the withdrawal rate up to $Q' = 0.15$. However, narrower cones of depression are observed due to restriction on the free flow of freshwater because of the presence of barrier at L_0. In the absence of barrier, saltwater progresses with the withdrawal rates [Fig. 7(a)] and



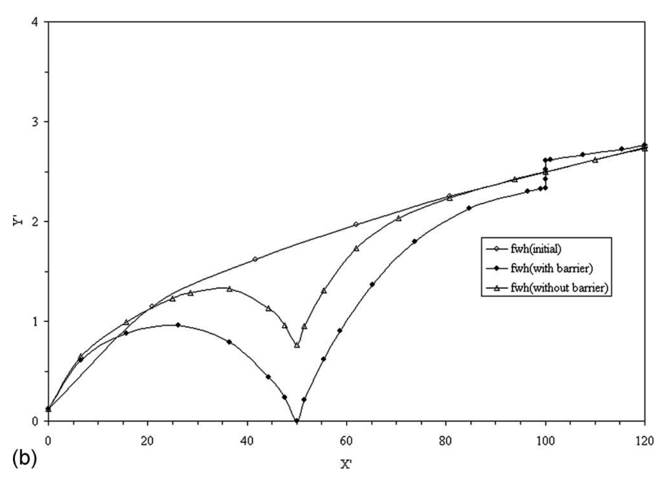


Fig. 2. (a) Steady state interface profiles for withdrawal rate $Q_f' = 0.01$ at 0.25 L_0; (b) cone of depression for withdrawal rate $Q_f' = 0.01$ at 0.25 L_0

J. Environ. Eng., 2009, 135(10): 980-988

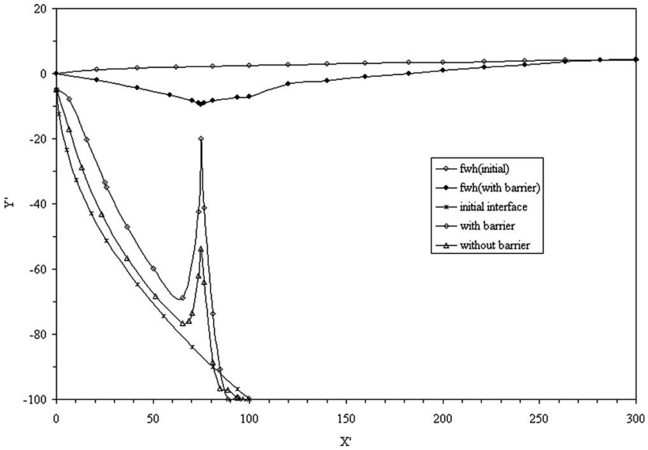


Fig. 3. Severe saltwater upconing due to withdrawal rate of $Q'_f = 0.01$ at 0.75 L_0

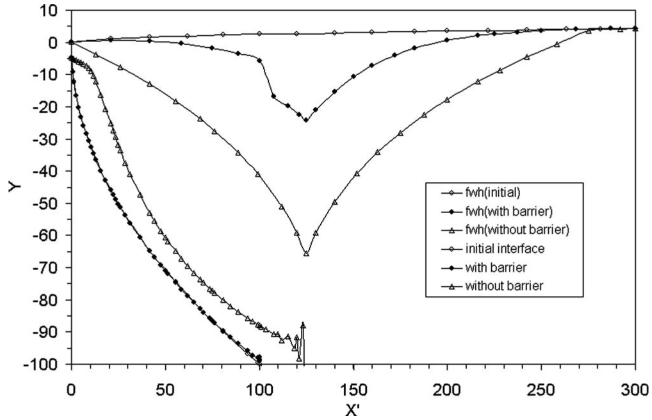


Fig. 4. Saltwater advancement due to withdrawal rate of $Q'_f = 0.05$ at 1.25 L_0

wider cones of depression are observed for the above rates of withdrawal [Fig. 7(b)]. Fig. 8 shows the interface and piezometric profiles for withdrawal rate of $Q' = 0.15$ at a farther location, i.e., 2.0 L_0. The interface is not getting affected at this withdrawal rate for both barrier and no barrier cases.

With increasing demand for groundwater resources due to rapid urbanization and industrialization in the coastal areas, freshwater withdrawals may often result in the piezometric levels dropping down significantly. Effect of such large-scale withdrawals was simulated by considering greater pumping rates ($Q'_f = 0.01 - 0.25$). It is of practical importance that safe distances from the seacoast need to be identified for these rates of withdrawals with no or negligible effect on the interface. These distances have been worked out for the numerical example considered in this work and are presented in Table 4. The distance at which drawdown has no effect on the interface may be termed as safe distance and it ranges from 1.2 L_0 for withdrawal rate from $Q' = 0.01$ to 2.0 L_0 for $Q' = 0.25$ in the barrier case. As a comparison, these distances have been worked out for nonbarrier cases also and are compared in Fig. 9. It is very much clear from the figure that, with barrier, greater freshwater developments may be possible much closer to the seacoast as compared to the nonbarrier situation. The results from the study may be useful to explore other cases of aquifer behavior under different stress scenarios with and without the presence of a semipervious barrier.

Conclusions

The problem of saltwater intrusion in unconfined coastal aquifers is analyzed for various practical ranges of withdrawal rates in the presence of a semipervious subsurface barrier. The results on the freshwater developments in the two fluid zones indicate that the saltwater upconing becomes severe for rates greater than $Q'_f = 0.005$ and is preferable to keep below this limit for avoiding water quality degradation. The presence of barrier may be disadvantageous for withdrawals in the two fluid zones since free flow of freshwater from the landward side is being prevented by the barrier. Studies on the freshwater withdrawals in the landward side of the barrier clearly indicate the effectiveness of the barrier in preventing/retarding the progress of saline wedge compared to no barrier cases. Specific rates of withdrawal are worked out for different locations which do not affect the saline wedge for barrier and no barrier cases. The nearest possible distances from the seacoast have been worked out for various freshwater withdrawal rates in the numerical example considered wherein their effect is negligible on the saltwater-freshwater interface. The results

Table 3. Effectiveness of the Barrier for Withdrawals on the Landward Side ($x > L_0$) $B_L = L_0$, $B_w = 0.3$ m, and $K' = 0.001$ K

		With barrier			Without barrier (homogenous)		
Withdrawal at (L_0)	Pumping rate Q'_f	% A $[(L-L_0)/L_0] \times 100$	Water level $h' = h/D \times 100$	Time t' Tt/nL_0^2	% A $[(L-L_0)/L_0] \times 100$	Water level $h' = h/D \times 100$	Time t' Tt/nL_0^2
1.25	0.05	0.61	−24.3	0.194 (14.7 days)	23.46[a]	−65.6	1.116 (84.8 days)
1.25	0.1	4.1412.62	−98.4[b]	0.309 (23.5 days)	13.4116.18	−98.3[b]	0.369 (28.0 days)
1.50	0.05	0.03	−14.0	0.099 (7.5 days)	33.5336.61	−98.1	1.774 (134.8 days)
1.50	0.10	0.03	−29.5	0.082 (6.2 days)	6.2515.42	−98.2[b]	0.374 (28.4 days)
1.50	0.15	0.010.58	−98.2[b]	0.165 (12.5 days)	2.3710.17	−98.9[b]	0.168 (12.8 days)
1.75	0.05	0.01	−5.0	0.025 (1.9 days)	2.5	−3.6	0.005 (0.4 days)
1.75	0.10	0.01	−13.7	0.025 (1.9 days)	0.00	−16.3	0.05 (3.8 days)
1.75	0.15	0.01	−25.2	0.029 (2.2 days)	0.827.94	−98.2[b]	0.171 (13.0 days)
2.00	0.15	0.01	−21.5	0.023 (1.8 days)	0.0	−7.7	0.005 (0.4 days)
2.25	0.15	0.01	−21.6	4.2 (319.2 days)	0.0	−7.5	0.005 (0.4 days)

[a]Salt water advances to withdrawal location, withdrawal rate is too high.

[b]Withdrawal rate too high, cone of depression reaches aquifer bottom; pumping to be stopped.

J. Environ. Eng., 2009, 135(10): 980-988

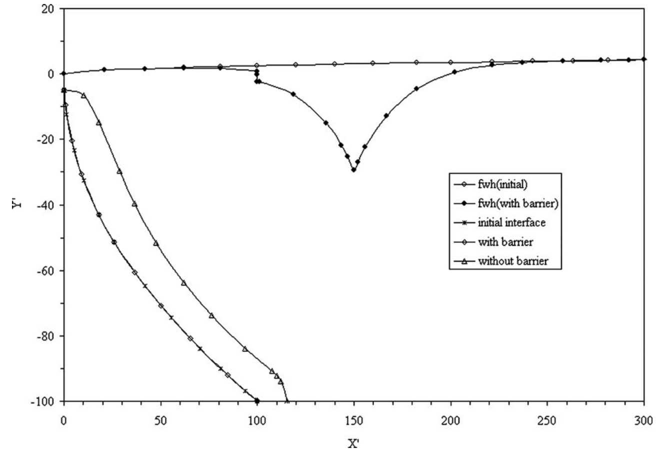


Fig. 5. Steady state interface profiles for withdrawal rate of 0.1 at 1.5 L_0

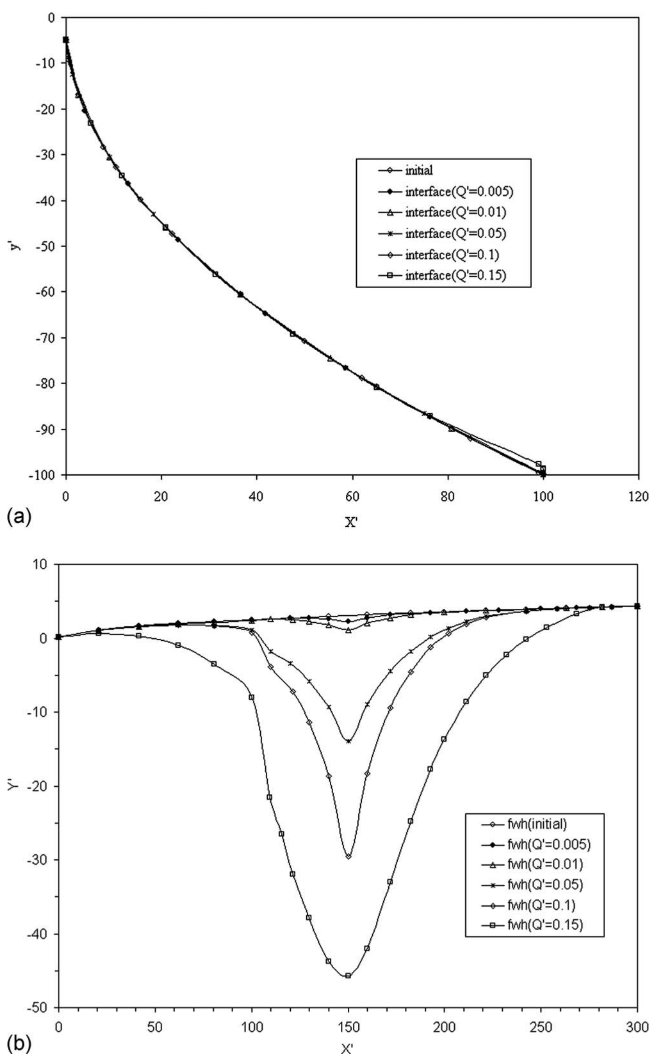


Fig. 6. (a) Steady state interface profiles for different withdrawal rates at 1.5 L_0 for barrier condition; (b) cones of depression for different withdrawal rates at 1.5 L_0 for barrier conditions

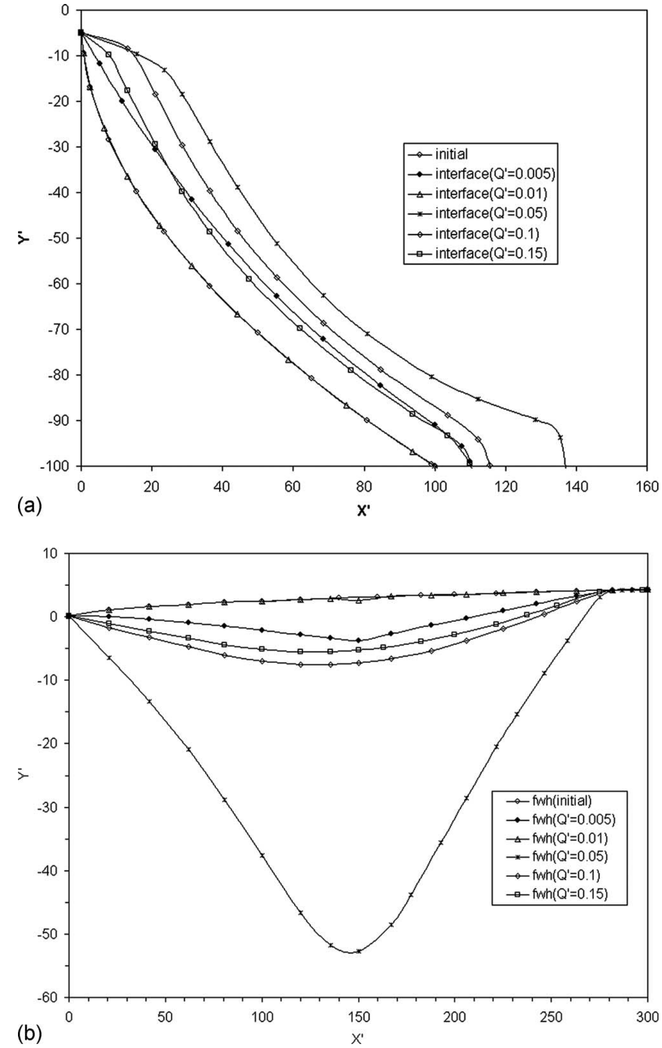


Fig. 7. (a) Steady state interface profiles for different withdrawal rates at 1.5 L_0 for nonbarrier conditions; (b) cones of depression for different withdrawal rates at 1.5 L_0 for nonbarrier conditions

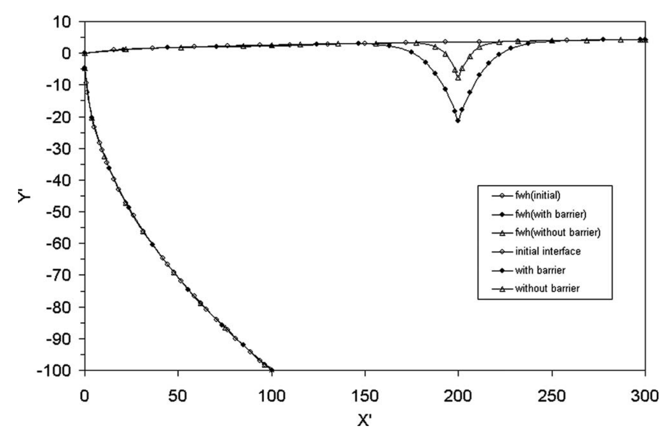


Fig. 8. Interface and piezometric profiles for withdrawal rate of $Q_f' = 0.15$ at 2.0 L_0

J. Environ. Eng., 2009, 135(10): 980-988

Table 4. Safe Distance from Seacoast for Groundwater Development

Magnitude of withdrawal (Q_f')	Safe distance (with barrier) (x/L_0)	Safe distance (without barrier) (x/L_0)
0.01	1.20	1.60
0.02	1.25	1.60
0.05	1.30	1.65
0.10	1.40	1.75
0.15	1.60	2.00
0.20	1.80	2.18
0.25	2.00	2.35

clearly indicate the advantage of having a semipervious barrier in view of increased stress scenarios on coastal aquifers and may be a suggested application after further validation in planning the coastal groundwater developments worldwide.

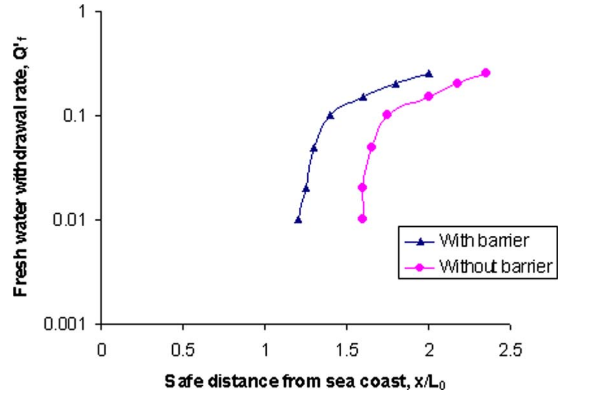


Fig. 9. Safe distance for coastal groundwater developments for barrier and nonbarrier conditions

Notations

The following symbols are used in this paper:

A = % advancement, $[(L-L_0)/L_0]\times 100$;
B_f, B_s = freshwater and saltwater saturated thickness, respectively (L);
B_L = perpendicular distance of the location of the barrier from the seacoast (L);
B_W = thickness (width) of barrier (L);
D = depth of aquifer below mean sea level (L);
h = vertically averaged piezometric heads (L);
h' = nondimensional water level=$(h/D)\times 100$;
h_f = initial freshwater piezometric head at the drawdown location (L);
h_{fL} = location of drawdown;
K' = barrier conductivity;
$K_f, K_s(=K)$ = hydraulic conductivity of freshwater and seawater, respectively (L/T);
L = final length of intrusion (L);
L_a = aquifer length (L);
L_0 = initial length of intrusion (L);
n = porosity;
Q_f = volumetric flow rate of freshwater (L^3/T);
Q_f' = nondimensional pumping rate=$Q_f/\alpha KD^2$;
Q_{ft} = seaward freshwater discharge at the wedge toe (L^3/T);
Q_I = initial seaward flow of freshwater per m width (L^2/T);
S = specific yield;
t = time (T);
t' = nondimensional time=$T_o t/nL_0^2$;
T_o = initial average transmissivity (L^2/T);
T = transmissivity (L^2/T);
X = coordinate axis (positive toward right) (L);
X' = nondimensional horizontal distance =$x/L_0\times 100$;
Y' = nondimensional vertical distance=$z/D\times 100$;
z = interface elevation (positive upward) (L);
α = excess density ratio=$(\rho_s-\rho_f)/\rho_f$;
θ = coefficient (=1 for unconfined aquifers; =0 for confined aquifers); and
ρ_f, ρ_s = freshwater and saltwater densities, respectively (M/L^3).

References

Atkinson, S. F., Miller, G. D., Curry, D. S., and Lee, S. B. (1986). *Salt water intrusion: Status and potential in the contiguous United States*, Lewis, Boca Raton, Fla., 312.

Barlow, P. M. (2003). "Groundwater in freshwater-saltwater environments of the Atlantic coast." *USGS Circular No. 1262*, USGS, Va., 159.

Basri, M. H. (2001). "Two new methods for optimal design of subsurface barrier to control seawater intrusion." Ph.D. thesis, Univ. of Manitoba, Canada.

Bear, J. (1977). "On the aquifer's integrated balance equations." *Adv. Water Resour.*, 1(1), 15–23.

Bear, J. (1979). *Hydraulics of groundwater*, McGraw-Hill, New York, 567.

Bear, J., Cheng, A. H.-D., Sorek, S., Ouazar, D., and Herrera, I. (1999). *Seawater intrusion in coastal aquifers—Concepts, methods and practices*, Kluwer Academic, Boston.

Bruington, A. E., and Seares, F. D. (1965). "Operating a sea water barrier project." *J. Irrig. and Drain. Div.*, 91(IR1), 117–140.

Charmonman, S. (1965). "Solution of pattern of freshwater flow in an unconfined coastal aquifer." *J. Geophys. Res.*, 12, 2813–2819.

Cheng, A. H.-D., and Ouazar, D. (2003). *Coastal aquifer management: Monitoring, modeling and case studies*, CRC, Boca Raton, Fla.

Essaid, H. I. (1986). "A comparison of the coupled freshwater-saltwater flow and the Ghyben-Herzberg sharp interface approaches to modeling of transient behavior in coastal aquifer systems." *J. Hydrol.*, 86(1–2), 169–193.

Essaid, H. I. (1990). "A multi-layered sharp interface model of coupled fresh water and saltwater in coastal systems: Model development and application." *Water Resour. Res.*, 26(7), 1431–1454.

Hanson, G., and Nilsson, A. (1986). "Groundwater dams for rural water supplies in developing countries." *Ground Water*, 24(4), 497–506.

Hill, M. C. (1988). "A comparison of coupled freshwater sharp interface and convective dispersive models of saltwater intrusion in a layered aquifer system." *Modeling surface and subsurface flow: Development in water science services*, Vol. 35, M. A. Celia, L. A. Ferrand, C. A. Brebbia, W. G. Gray, and G. F. Pinder, eds., Elsevier, Amsterdam, 211–216.

Hunt, B. (1985). "Some analytical solutions for sea water intrusion control with recharge wells." *J. Hydrol.*, 80(1–2), 9–18.

Kashef, A. A. I. (1976). "Control of salt water intrusion by recharge wells." *J. Irrig. and Drain. Div.*, 102(IR4), 445–457.

Knox, R. C. (1983). "Effectiveness of impermeable barrier for retardation of pollutant migration." Ph.D. thesis, Univ. of Oklahoma, Norman, Okla., 162.

Mahesha, A. (1995). "Parametric studies on the advancing interface in coastal aquifers due to linear variation of freshwater level." *Water Resour. Res.*, 31(10), 2437–2442.

J. Environ. Eng., 2009, 135(10): 980-988

Mahesha, A. (1996). “Transient effect of battery of injection wells on seawater intrusion.” *J. Hydraul. Eng.*, 122(5), 266–271.

Mohan Babu, M. (1999). “Effect of spatially varied hydraulic conductivity on saltwater intrusion.” MTech thesis, National Institute of Technology Karnataka Surathkal, India, 103.

Nagata, S., Enami, M., Nagata, J., Katoh, T., and Okamoto, M. (1993). “Design and construction of cut-off walls for subsurface dams on Amami and Ryukyu islands of southeastern part of Japan.” *International Association of Hydrogeologists*, 2, 45–63.

Nagata, S., and Kawasaki, S. (1997). “Hydrogeological development of subsurface dams.” *Rep.*, Ministry of Agriculture, Forestry and Fisheries of Japan, Tokyo.

Nutbrown, D. A. (1976). “Optimal pumping regimes in an unconfined coastal aquifer.” *J. Hydrol.*, 31, 271–280.

Osuga, K. (1996). “Miyakojima underground dam: New technology for water resources development.” *Rep.*, Japan Agricultural Land Development Agency, Tokyo.

Pinder, G., and Stothoff, S. (1988). “Can the sharp interface salt water model capture transient behavior?” *Proc., 7th Int. Conf. on Comp. Meth. Water Resour.*, Elsevier, New York, 217–222.

Reichard, E. G., and Johnson, T. A. (2005). “Assessment of regional management strategies for controlling seawater intrusion.” *J. Water Resour. Plann. Manage.*, 131(4), 280–291.

Reilly, T. E., and Goodman, A. S. (1985). “Quantitative analysis of saltwater-freshwater relationships in groundwater systems—A historical perspective.” *J. Hydrol.*, 80(1–2), 125–160.

Rogoshewski, P., Bryson, H., and Wagnor, K. (1983). “Remedial action technology for waste disposal sites.” *Pollution Technology Review No. 101*, Park Ridge, N.J.

Rumer, R. R., Jr., and Harleman, D. R. F. (1963). ““Intruded salt water wedge in porous media”. *J. Hydr. Div.*, 89(HY6), 193–220.

Sa da Costa, A. A. G., and Wilson, J. L. (1979). “A numerical model of seawater intrusion in aquifers.” *Rep. No. 247*, Ralph M. Parsons Laboratory for Water Resources and Hydrodynamics, Dept. of Civil Engineering, MIT, Cambridge, Mass.

Schroeder, D. J., Harley, B. M., and Mejia, P. E. C. (1989). “Sea water intrusion barrier evaluated with 3-D groundwater models.” *Water Eng. Manage.*, 136(2), 26–29.

Sheahan, N. T. (1977). “Injection/extraction well system—A unique sea water intrusion barrier.” *Ground Water*, 15(1), 32–50.

Sugio, S., and Nakada, K. (1984). “Protection of groundwater reservoir against saltwater intrusion.” *Protection of groundwater reservoir against saltwater intrusion*, A. Das Gupta and A. Arbhabhirama, eds., Vol 2, Secretariat of APD—IAMR, Asian Institute of Technology, Bangkok, Thailand, 1567–1578.

Sugio, S., Nakada, K., and Urish, D. W. (1987). “Subsurface seawater intrusion barrier analysis.” *J. Hydraul. Eng.*, 113(6), 767–779.

Todd, D. K. (1974). “Saltwater and its control.” *J. Am. Water Works Assoc.*, 66(3), 180–187.

Todd, D. K. (1980). *Groundwater hydrology*, Wiley, New York.

U.S. Army Environmental Center. (2002). *Remediation technologies screening matrix and reference guide version 4.0*, General Contacts of Federal Remediation Technologies Roundtable Agencies, Md.

U.S. Environmental Protection Agency (1987). “Ground water handbook.” *Groundwater Rep. No. EPA/625/6–87/016*, Office of Research and Development, Robert S. Kerr Environmental Research Laboratory, Ada, Okla.

Williams, D. (1977). “The Dashte-Naz groundwater barrier and recharge project.” *Ground Water*, 15(1), 23–31.

Willis, R., and Finney, B. A. (1988). “Planning model for optimal control of salt water intrusion.” *J. Water Resour. Plann. Manage.*, 114(2), 163–177.

J. Environ. Eng., 2009, 135(10): 980-988