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Effect of strip recharge on sea water intrusion into aquifers

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Abstract Parametric studies were carried out to analyse the effect on sea water intrusion of freshwater recharge through a finite width strip parallel to the coast. A vertically integrated one-dimensional finite element model was used for this purpose. The studies included the analysis of the effect of location, width, intensity and the period of recharge on sea water–freshwater interface motion. Relationships were established between the interface motion and the recharge parameters applicable to wide ranging practical cases. From the studies, the ideal location for recharge was identified to achieve the maximum repulsion of intrusion. The width of recharge also affects the interface motion and the widths greater than 2% of the initial intrusion length were effective in controlling the intrusion. The results indicated that the reduction of intrusion up to 30% could be achieved through strip recharge.

Key words aquifer; sea water intrusion; recharge; groundwater vulnerability

Effet d'une bande de recharge sur l'intrusion d'eau de mer dans les aquifères

Résumé Des études paramétriques ont été effectuées pour analyser l'effet sur l'intrusion marine de la recharge en eau douce à travers une bande de largeur finie et parallèle au littoral. Un modèle mono-dimensionnel vertical à éléments finis a été utilisé dans ce but. Les études comprennent l'analyse de l'influence de la localisation, de la largeur de la bande, de l'intensité et de la période de recharge sur le déplacement de l'interface entre l'eau marine et l'eau douce. Les relations entre le déplacement de cette interface et les paramètres de la recharge ont été établies pour une large gamme de situations. A partir des études, le lieu de recharge permettant de repousser l'intrusion au maximum a été identifié. La largeur de la bande de recharge a également un effet sur le déplacement de l'interface et des largeurs supérieures à 2% de la longueur initiale de l'intrusion se sont révélé efficace pour contrôler l'intrusion. Les résultats ont montré qu'une réduction de l'intrusion de 30% peut être obtenue grâce à une recharge en bande.

Mots clefs aquifère; intrusion de l'eau de mer; récharge; vulnérabilité de l'eau souterraine

INTRODUCTION

The analysis of the problem of sea water intrusion has received much attention during the last few decades. Numerical modelling has become a widely used tool for simulating the hydrodynamic behaviour of the aquifer systems. Many researchers have made quantitative analysis of coastal aquifer systems mathematically to describe the mechanics of intrusion (Cooper *et al.*, 1964; Bear & Dagan, 1964; Pinder & Page, 1977). The sharp interface models were found to be sufficiently accurate and cost effective in simulating the coastal hydrodynamics as compared to the dispersive approach (Bear, 1977). Several numerical models have been developed using this approach for the transient conditions in regional aquifer systems (e.g. Shamir & Dagan, 1971; Sa da Costa & Wilson, 1979; Mercer *et al.*, 1980; Inouchi *et al.*, 1985;

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Ledoux *et al.*, 1990; Singh & Gupta, 1999). However, in the context of extensive intrusion experienced worldwide, the present-day need is an effective method to combat it. Studies on the effect of uniform natural recharge on sea water intrusion were reported by a few researchers (Van der Veer, 1977a,b; Isaacs, 1985; Mahesha & Nagaraja, 1996) and it was found to be effective in coastal areas receiving heavy to very heavy rainfall. However, works on the finite width strip recharging of the aquifer were limited to the approximate analytical solution for the interface depths in thick coastal aquifers receiving uniform recharge from an infinite strip parallel to the coast (Hantush, 1968). Karandikar (1978) used a finite difference formulation and also conducted the Hele-Shaw experiments to analyse the effect of strip recharge on sea water intrusion considering laboratory cases. He broadly concluded that the recharge location and the seaward flow of freshwater are the predominant factors in controlling the intrusion. In the present work, detailed parametric studies are carried out to understand the behaviour of the interface subjected to different recharge scenarios applicable to wide ranging practical cases.

GROUNDWATER FLOW EQUATIONS

The definition sketch of the problem is given in Fig. 1. The initial equilibrium between the sea water and the freshwater was established by the seaward freshwater flow of Q_I. From an initial position of L_0, the sea water/freshwater interface was made to repel to a new position, L_t, by the freshwater recharge of uniform intensity, w, through a recharge strip located at a distance R_L from the coast. The recharge strip has a constant width b running parallel to the coast. The above problem is governed by two coupled nonlinear partial differential equations in freshwater and sea water piezometric heads (Bear, 1979):

$$\nabla \cdot (B_f\, K_f\, \nabla h_f) + \frac{n}{\alpha}(1+\alpha)\frac{\partial h_s}{\partial t} - \left[\frac{n}{\alpha}(1+\alpha) + S_f\right]\frac{\partial h_f}{\partial t} + w = 0 \tag{1a}$$

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

in which B is the saturated thickness [L]; K is the hydraulic conductivity [L T^{-1}]; h is the vertically averaged piezometric head [L]; n is the porosity; α is the excess density ratio = $(\gamma_s - \gamma_f)/\gamma_f$; γ is the specific weight [M L^{-2} T^{-2}]; S is the elastic storativity; w is the recharge intensity [L T^{-1}]; t is time [T]; and the subscripts f, s refer to the freshwater and sea water, respectively. The major assumptions made in deriving the above equations are:

(a) freshwater and sea water are separated by a sharp interface;
(b) Dupuit's approximation is valid and the flow is essentially horizontal;
(c) vertical flow is considered only in the case of recharge;
(d) Darcy's law is valid in the region;
(e) specific storage and horizontal permeability variations in the vertical direction are negligible; and
(f) vertical outflow face approximation holds good at the outflow to the sea.

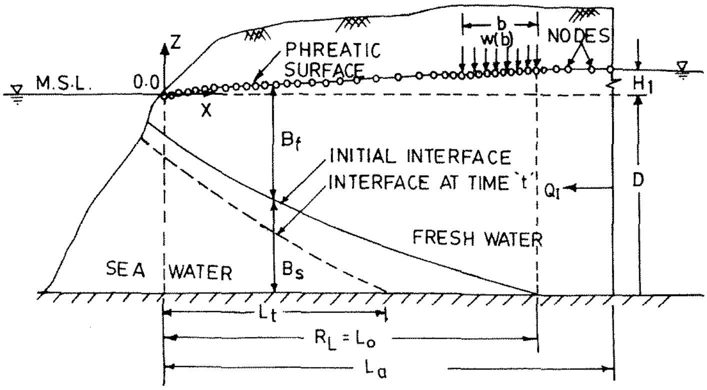

Fig. 1 Definition sketch of the problem.

The sharp interface approach adopted for the model is useful for broader delineation of sea-water/freshwater on a regional basis. However, the analysis may lead to erroneous results whenever horizontal distances of interest are small compared to the thickness (Bear, 1977). Also, the vertically averaged models do not account for the transient behaviour of regimes exhibiting significant vertical flow and/or dispersion (Pinder & Stothoff, 1988). Further assumptions made in the analysis were: (i) $K_f = K_s$; and (ii) since the effect of variation of S on the interface motion was found to be insignificant, it may be neglected in equation (1).

DESIGN OF THE FINITE ELEMENT MODEL

In the present study, the Galerkin finite element method was used to develop the element equations. There are two independent variables at each node, freshwater and sea water piezometric heads. The system of equations could be expressed as:

$$[\boldsymbol{S}]\{\dot{\boldsymbol{h}}\}+[\boldsymbol{C}]\{\boldsymbol{h}\}+\{\boldsymbol{F}\}=0 \tag{2}$$

in which $[\boldsymbol{S}]$ is the storage matrix, $[\boldsymbol{C}]$ is the conductivity matrix, $\{\boldsymbol{h}\}$ is the state vector, and $\{\boldsymbol{F}\}$ is the flux vector.

The state variable, h is a function of time and a backward scheme in time was used (Huyakorn & Pinder, 1983) to ensure a stable solution with minimum oscillation of the interface profile.

The initial values of the piezometric head and the interface elevation were computed as follows (Vappicha & Nagaraja, 1975):

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

$$h_s = 0.0 \tag{3b}$$

$$z = -\frac{h_f}{\alpha} \tag{3c}$$

where Q_I is the initial steady freshwater discharge per unit width [L^2 T^{-1}]; x is the coordinate axis [L], positive to the right; and z is the interface elevation (positive upwards). The analytical solution equation (3a) is derived for semi-infinite depth aquifers. For finite depth aquifers, the solution obtained may slightly underestimate the phreatic surface beyond L_0. Hence it is suggested to attain an initial steady state from the analytical solution which may be used for transient analysis. The following were the boundary conditions:

$$h_f\ (0,\mathrm{t}) = \frac{0.722 Q_{ft}}{K} \qquad \text{(Charmonman, 1965)} \tag{4a}$$

$$h_f\ (L_a,\ t) = H_1 \tag{4b}$$

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

where Q_{ft} is the freshwater discharge at the wedge toe per unit width [L^2 T^{-1}] = $KB_f\ \dfrac{\partial h_f}{\partial x}$; H_1 is the initial freshwater head above the mean sea level [L] at the landward end; and L_a is the length of the aquifer perpendicular to the coast [L]. A sufficiently greater length ($L_a = 1.75L_0$) was considered for the domain to minimize the effect of constant head condition at the landward end. The elemental equations were assembled to form the global matrix. The matrix is symmetric and banded in nature. After the application of the necessary boundary conditions, the system of equations was solved iteratively by Newton-Raphson method. The step-by-step solution procedure for the above process is given in Huyakorn & Pinder (1983) and Sa da Costa & Wilson (1979). From the converged solution, the interface elevation was computed using the relationship:

$$z = \frac{1}{\alpha}[(1+\alpha)\,h_s - h_f] \tag{5}$$

The intersection of the interface with the aquifer bed is known as the toe of the interface. The coordinate of the toe indicates the length of intrusion.

This length was expressed as a percentage of initial length of intrusion by:

$$P = \frac{L_0 - L_t}{L_0} \times 100 \tag{6}$$

where P is the reduction of sea water intrusion; L_0 is the initial length of intrusion [L]; and L_t is the length of intrusion at any time, t [L].

In order to test the validity of the numerical model, experimental results of the Hele-Shaw model (Karandikar, 1978) were considered. Karandikar conducted the Hele-Shaw experiments to study the effect of uniform strip recharge on the interface motion in unconfined aquifers by considering different recharge rates and seaward flow of freshwater. The interface profiles were photographed at regular time intervals. Out of them, one of the cases was compared with the numerical solution. The necessary data for this trial are shown in Fig. 2(a). The motion of the interface in terms of the length of intrusion $(\alpha KL_t)/Q_I$ *vs* time $(Q_I t)/nD^2$ is compared with the experimental results in Fig. 2(a). In another test case, the non-dimensional interface profiles $(\bar{z}\ vs\ \bar{x})$ are compared at different time intervals (Fig. 2(b)). The profile starts

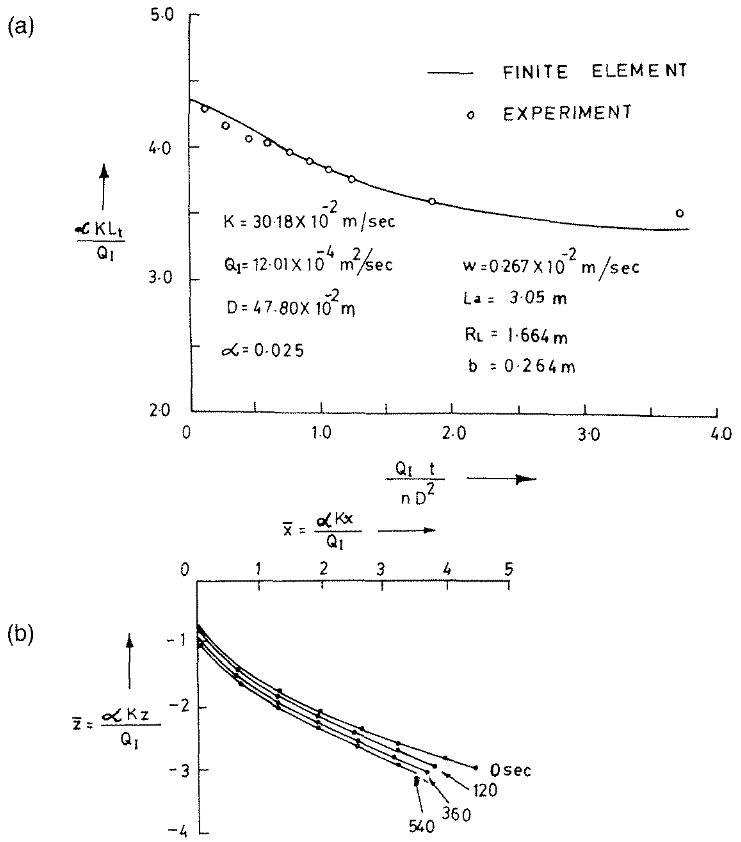

Fig. 2 (a) Motion of the interface due to strip recharge; and (b) interface profiles at different time intervals.

retreating with time due to the application of strip recharge. In both cases, fairly good agreement was observed between the numerical and the experimental results for the intrusion length and the interface profiles.

MODEL APPLICATION

The numerical model was then used to carry out the parametric studies considering ten hypothetical cases applicable to the possible cases in the range $L_0/D = 5$–500; in which L_0 is the initial length of intrusion [L]. The rate of recharge and time were expressed in non-dimensional forms as:

$$H_r = \frac{KDH_1}{nL_0^2 w} \qquad (7)$$

$$\delta_t = \frac{Tt}{nL_0^2} \qquad (8)$$

in which H_r is the non-dimensional recharge rate; δ_t is the non-dimensional time; T is the transmissivity $[L^2 T^{-1}] = K{\cdot}B$; $B = (h_l + 2H_l)/3 + D$; and h_l is the initial freshwater head above the mean sea level at the seaward end [L]. The domain was divided into smaller elements near the coast and around the area of recharge to simulate the nonlinearities of the phreatic surface effectively. In the present study, the domain was divided into 40–60 elements depending on the recharge width. Figure 1 shows the discretization pattern adopted in the present analysis. The recharge was applied with the intensities $H_r = 0.0001$–0.1 over the widths: $b = L_0/20$, $L_0/35$, $L_0/50$, $L_0/75$, $L_0/100$, $L_0/120$ and $L_0/140$, at the locations: $R_L = 0.5L_0$, $0.6L_0$, $0.7L_0$, $0.8L_0$, $0.9L_0$, $1.0L_0$, and $1.1L_0$, in which b is the recharge width [L]; and R_L is the perpendicular distance to the recharge location from the coast [L]. The interface motion was also monitored after the cessation of recharge. In all, more than 220 cases were solved.

The reduction of sea water intrusion due to the strip recharge is presented in Fig. 3 for the widths, $b = L_0/35$ and $L_0/100$. The motion of the interface was rigorous at the initial stages and slowed down as it moved away from the recharge location. The relationship established is useful in estimating the repulsion of intrusion at any time

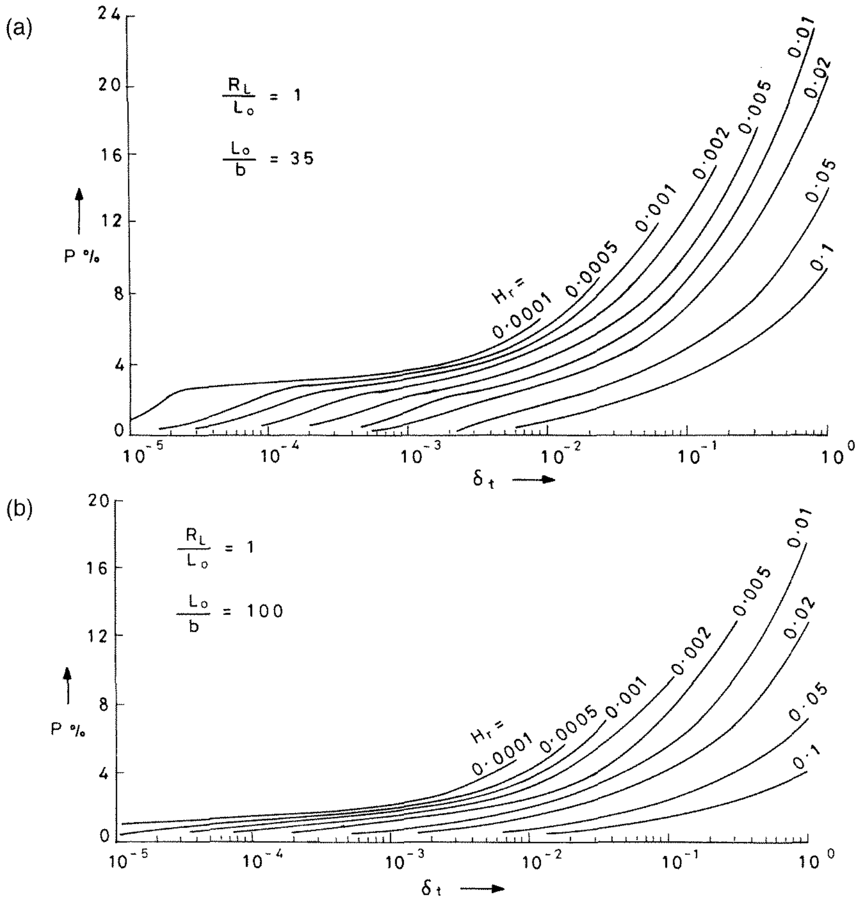

Fig. 3 (a) Reduction of sea water intrusion due to strip recharge: (a) $b = L_0/35$; and (b) $b = L_0/100$.

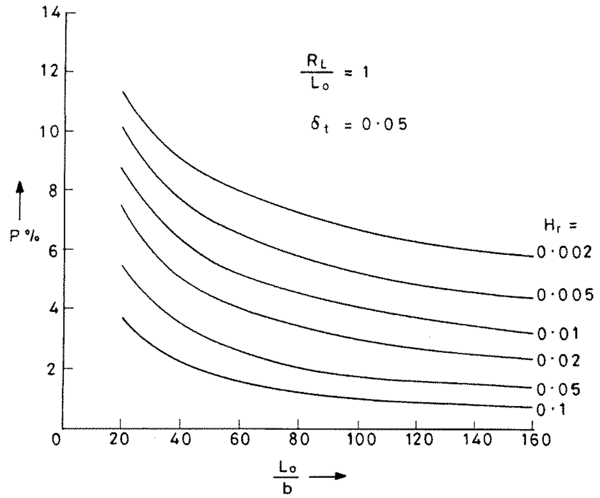

Fig. 4 Variation of the reduction of sea water intrusion with the recharge width.

due to a recharge of uniform intensity. The performance of the recharge strip for other widths was also analysed and the variation of the reduction of intrusion is presented in Fig. 4. Encouraging results were obtained for widths $b > L_0/60$. The efficiency decreased with the decreasing width and the performance was poor for widths $b < L_0/100$. The reduction of sea water intrusion owing to the recharge applied over different locations is presented in Fig. 5. Better results were obtained for the locations $R_L < L_0$. However, as the recharge location was shifted towards the coast, the

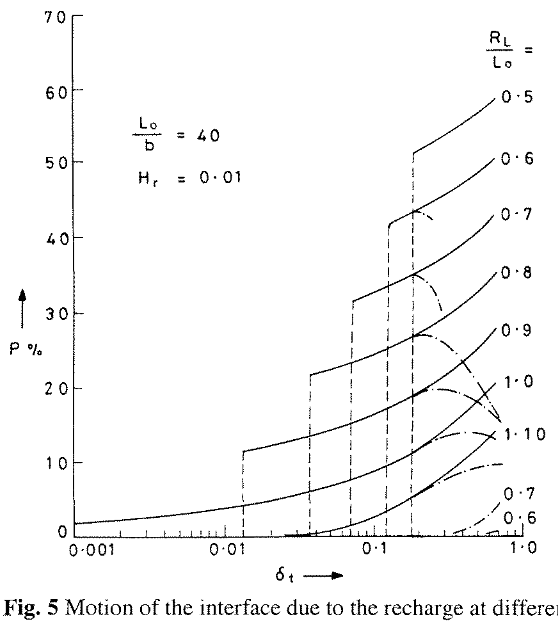

Fig. 5 Motion of the interface due to the recharge at different locations.

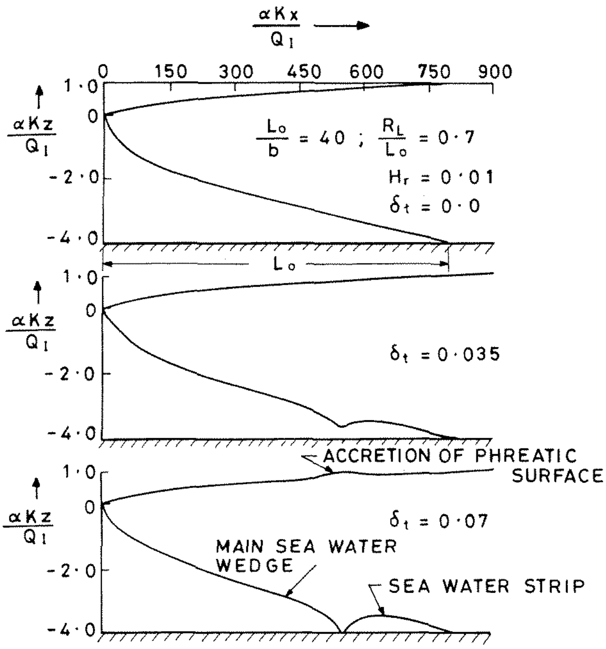

Fig. 6 Sketch illustrating the formation of the isolated saline wedge inland.

probability of the formation of isolated saline wedges inland increased. In this process, a portion of the saline wedge gets separated from the main wedge due to the freshwater ridge created by the recharge strip and it remains inland or sometimes moves further inland. Figure 6 illustrates the formation of the isolated saline wedge for the case $R_L = 0.7L_0$. The size of this wedge depends on the location and intensity of recharge. The formation of the isolated saline wedges inland is also indicated in Fig. 5 by the sudden increase in the reduction of intrusion for the locations $R_L < L_0$. In these cases, the freshwater ridge was unable to repel the entire saline wedge and instead divided the saline wedge into two portions. The length of saline wedge thus reduced by the breakaway portion resulted in an increase in the reduction of intrusion. The length of the breakaway portion of the saline wedge was proportional to the initial reduction of intrusion (starting point) indicated by the curves in Fig. 5. The performance of the recharge system may be termed unsatisfactory if the size of the isolated saline wedge (breakaway portion) is large enough to contaminate the inland aquifers.

In another part of the study, the interface motion was monitored after the cessation of recharge. In Fig. 5, the recharge was stopped at $\delta_t = 0.2$ and the motion of the interface thereafter is indicated through the dotted lines. The interface continued to retreat even after the cessation of recharge for the locations $R_L > L_0$. However, the retreating trend of the interface slowed down with time. In some cases ($R_L < 0.9L_0$), the retreating trend arrested after some time and the interface started advancing. This is indicated by the downward trend of the dotted curves. For the locations $R_L \leq 0.7L_0$, the main saline wedge advanced enough to rejoin the breakaway portion bringing down the reduction of intrusion to zero. However, the freshwater ridge built up, repelled the united wedge by 2–3%, as indicated by the dotted curves ($R_L/L_0 = 0.6, 0.7$) in Fig. 5.

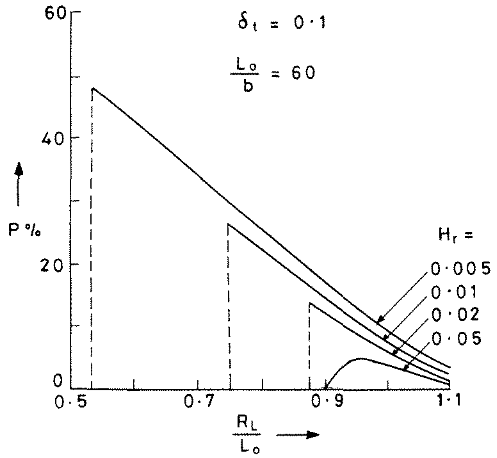

Fig. 7 Variation of the reduction of intrusion with the recharge location.

In practice, the recharge process would be discontinued inbetween for cleaning operations or due to lack of adequate supply of freshwater. The advancement of the interface during this period is highly undesirable. The variation in the performance of recharge strip with the location is presented in Fig. 7 for different widths and recharge rates. The recharge strip performed better for the locations nearer to the coast. However, as the recharge location is shifted towards the coast, the risk of formation of the isolated saline wedges inland also increased. The location $R_L < 0.9L_0$ showed greater reduction of intrusion, but resulted in the formation of isolated saline wedges inland depending on their location. For the extreme locations $R_L << L_0$, there was no effect of recharge on the saline wedge. The locations yielding the maximum reduction

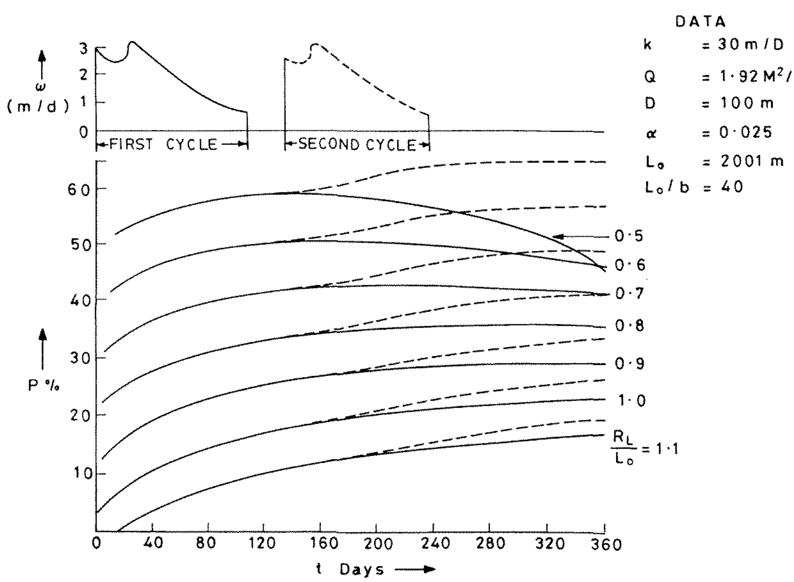

Fig. 8 Reduction of sea water intrusion due to an exponentially varying recharge rate.

of intrusion with the least isolated saline wedges inland were found to be in the region $R_L = 0.8\text{–}1.0L_0$. The locations beyond L_0 landwards contributed significantly in building up the inland groundwater resources.

The above studies assume a constant rate of recharge. As an illustrative example, the motion of the saline wedge due to an exponentially varying recharge rate (Bear, 1979) was considered for a hypothetical case. The necessary data are indicated in Fig. 8. The recharge was applied for a period of 104 days and the resulting motion of the interface was monitored (Fig. 8). The reduction of sea water intrusion up to 10–30% was achieved for the locations $R_L \geq 0.8L_0$. However, the locations $R_L < L_0$, resulted in the formation of isolated saline wedges inland and the size of these wedges was >20% of L_0 for the locations $R_L < 0.8L_0$. The motion of the interface was also monitored after the cessation of recharge and the locations $R_L > 0.8L_0$ showed a steady trend in the reduction of intrusion. For other locations, the interface showed an advancing trend after the cessation of recharge. This is indicated by the downward trend of the curves for $R_L = 0.7L_0$, $0.6L_0$ and $0.5L_0$. In another case, the recharge was applied in two cycles with an interval of 27 days for maintenance. The motion of the interface during the interval and thereafter is indicated by the discontinuous lines in Fig. 8. The ideal recharge location for the above example to achieve maximum reduction of intrusion without formation of trapped saline wedge inland would be in the region 1600–2000 m from the coast with a recharge width, b preferably larger than 33.4 m ($L_0/60$).

Based on the above studies (220 cases solved), the recharge locations causing maximum reduction of intrusion were identified and are plotted. Figure 9 shows the relationship between the recharge location, width and the rate at time $\delta_t = 0.1$. To illustrate the usefulness of this relationship, consider a specific case with $K = 30$ m day^{-1}; D = 100 m; Q_I = 0.92 m^2 day^{-1}; n = 0.4; α = 0.025; L_0 = 4178 m; H_l = 3.307 m (at $1.75L_0$); h_l = 0.0221 m; and B = 102.21 m. For recharge width

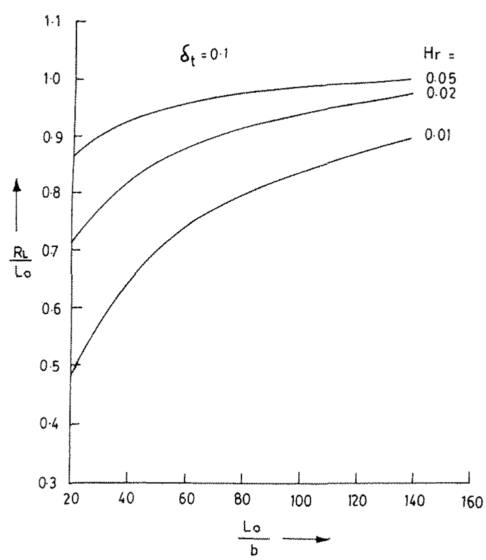

Fig. 9 Recharge locations yielding the maximum reduction of sea water intrusion.

$b = L_0/60$ (69.6 m), recharge intensity, $H_r = 0.02$ (0.071 m day^{-1}), and the recharge period $\delta_t = 0.1$ (227.7 days), the recharge location causing maximum reduction of intrusion (Fig. 9) is, $R_L = 0.87L_0$ (3635 m from the coast). The amount of reduction of intrusion, P in this case (Fig. 7) is 15%.

CONCLUSION

The key factors affecting the interface motion are the location, width, rate and the period of recharge. Increased reduction of intrusion was observed for the recharge locations, $R_L < L_0$. However, as the recharge location was shifted towards the coast, the probability of formation of the isolated saline wedges inland increased. For all purposes, the ideal location for recharge would be in the region $0.8L_0$–L_0. The locations beyond L_0 landwards usually help in building up the inland groundwater resources. The recharge widths, $b > L_0/60$ produced significant repulsion of intrusion. The efficiency decreased with recharge width and the performance was poor for widths, $b < L_0/100$. The motion of the interface was also monitored after the cessation of recharge, when the interface continued to retreat with a diminishing rate. In some extreme cases, e.g. $R_L \leq 0.7L_0$, this trend was reversed and the interface started advancing. Performance of the recharge strip at these locations may be unsatisfactory. In all, the study indicated that a reduction of intrusion of up to 30% could be achieved through strip recharge.

The results obtained from the present study are useful in understanding the behaviour of the dynamics of the saline wedge when a coastal aquifer is stressed by different recharge scenarios.

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