CHAPTER 358

Watertable Dynamics in Coastal Areas

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Abstract

Fresh water lens measured under the coastal barrier is much thinner than that predicted by the classical Ghyben-Herzberg theory . Landward downsloping watertable which is usually seen at coastal areas results in a landward flow of salty water or waste water released into aquifer. Comparison between the laboratory experiments with regular waves and no tides and the field data has revealed significant difference in magnitudes of the infiltration velocity. It is obvious that the tidal phase is very important. Infiltration velocities are large when the shoreline moves landward on a partially saturated beach on a rising tide. General magnitudes of U_I are $0.08K$ for steady laboratory conditions to $0.45K$ for field conditions during the rising tides. A mathematical model of the watertable which includes the effects of runup infiltration is obtained in a steady state. A finite-difference numerical model of the watertable is also presented with the glassy/dry boundary as a boundary condition.

Introduction

Watertable dynamics in coastal areas are of obvious interest in relation to problems such as salt water intrusion to the aquifer and wastewater disposal from coastal developments, but research into these processes has been scarce. In addition, the modeling of swash zone sediment transport requires a better knowledge of the beach groundwater dynamics.

Extensive field studies along the east coast of Australia have revealed that the overheight in the coastal watertable due to wave runup and tidal action on the ocean side are sufficient to create a steady drift of salty ground water under narrow coastal islands

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and barriers. Figure 1 details a typical situation showing the landward downsloping watertable and the very thin fresh water lens monitored at the narrow northern end of Bribie Island near Brisbane, Australia.

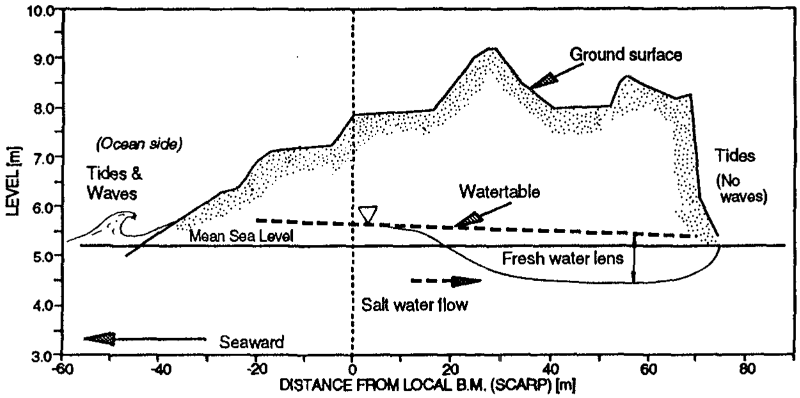

Figure 1. Field data showing the landward downsloping watertable and the thin fresh water lens (Bribie Island North, 14 July 1994).

In this case, the difference in groundwater level between the ocean side which is exposed to waves and the landward side which is relatively sheltered from waves is 0.4 meters. The magnitude of difference varies depending on the wave conditions, the slopes on the two sides and the landmass width. These features are important for understanding ecosystems in these areas and for pollution control. The difference in groundwater levels shown in Figure 1 is due to the combined effect of tides and waves.

The fresh water lens observed at the barrier island is seen to be much thinner than that predicted by the classical Ghyben-Herzberg (see Domenico and Schwartz, 1990) theory. The Ghyben-Herberg result was obtained from simple hydrostatics with no consideration of the details of the wave runup infiltration near the high water mark. The measured fresh water lens under the barrier also shows an asymmetry, i.e. it opens up gradually on the ocean side, but closes abruptly on the landward side.

This paper outlines the recently developed theory for wave effects significantly affecting coastal watertable dynamics and presents comparison with field and laboratory measurements. Watertable modeling is also given mathematically and numerically.

Wave Induced Watertable Overheights

Ocean waves raise the coastal watertable through their lifting of the free mean water surface seaward of the shoreline, i.e. wave setup, and due to the infiltration from wave runup landward of the shoreline. See Figure 2.

Wave Setup

The phenomenon of wave setup has been treated extensively in the water wave

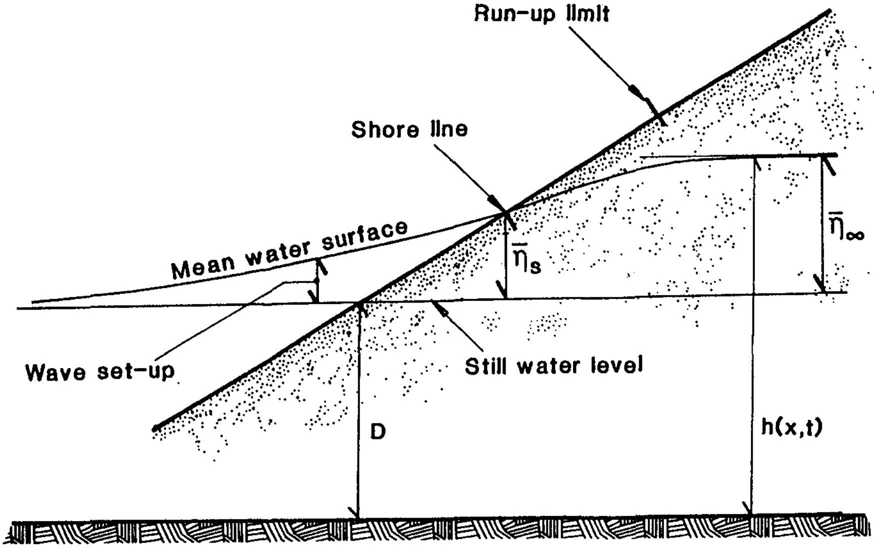

Figure 2. Definition sketch

literature. Extensive field data have been reported by Nielsen [1988] and Hanslow and Nielsen [1993]. They indicate that the shoreline setup $\overline{\eta_s}$

$$\overline{\eta}_S = 0.38 H_{orms} \tag{1}$$

or

$$\overline{\eta}_S = 0.048 \sqrt{H_{orms} L_o} \tag{2}$$

where H_{orms} is the root mean square deep water wave height and L_o the deep water wave length ($L_o = gT^2/2\pi$, where T is the wave period).

Regular Waves, No Tide

Experimental studies were undertaken in a wave flume with regular waves and no tide to gain insight into process of infiltration from wave runup and the resulting watertable overheight as reported by Kang et al. (1994). It was found from the flume experiments that the asymptotic inland overheight $\overline{\eta_\infty}$ depends on the beach profile, the wave height H_i and the wave period T, but not directly on the hydraulic conductivity K of the beach material. Kang et al. [1994] recommended the following formula in relation to Hunt's [1959] formula for the runup of regular waves:

$$\bar{\eta}_\infty = 0.62(Z_R - SWL) = 0.62\tan\beta_F\sqrt{H_i L_O} \tag{3}$$

where Z_R is the level of maximum runup, SWL the still water level and $\tan\beta$ the beachface slope. This indicates that the asymptotic inland overheight amounts to be 62% of the runup height irrespective of sand size. However, separate regression lines for coarse and fine sands, as seen in the relation between the inland overheight scaled on the wave height (h_∞-SWL/H_i) and the surf similarity parameter ξ (=$\tan\beta_F(L_o/H_i)^{0.5}$) (see Figure 3), show a weak effect/correlation of sand size, indicating that the watertable overheight is larger for larger sands, in the range of wave conditions of 1.2sec≤ T ≤2.8sec and 58mm ≤ H_i ≤ 200mm with aquifer depths ranging from 372mm to 436mm (regression coefficients: 0.64 with r^2=0.73 for coarse sands and 0.48 with r^2=0.86 for fine sands). This trend is consistent with Gourlay's [1985] data. The fact that the watertable overheight is larger for coarser sands may not be caused directly by different sand sizes, but by different beach profile shapes associated with different breaker types and different beach materials (see Rector, 1954; Gourlay, 1980; 1985). In general, finer sands form characteristically flatter foreshore beach profiles (with an outer bar formed at the breaking point of the waves), and more energy is dissipated through the surf zone than for beaches with coarse sands for the same wave conditions. The former results in less runup on the beach face than the latter. Smaller wave runup heights hence produce less watertable overheight.

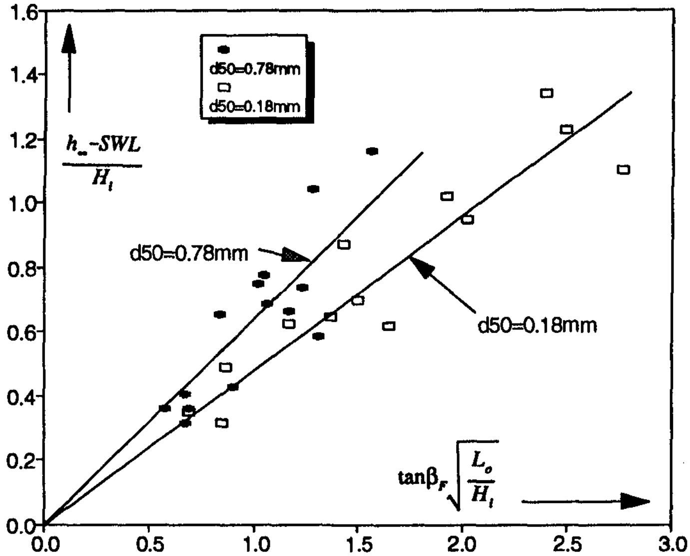

Figure 3. The normalized inland overheight as a function of the surf similarity parameter ξ(=$\tan\beta_F(L_o/H_i)^{0.5}$).

On the other hand, the present result is not directly consistent with Gourlay's [1992] example for a flat wave (H/L_o=0.007) which shows that the elevation of the watertable increased as the hydraulic conductivity of the beach material decreased. Equation (3) may apply when the breaking waves are not surging (for H/L_o>0.01: see Figure 4 of Gourlay [1985]) and the beach material is sand, but it may not hold for very flat waves (H/L_o<0.01). The wave steepness is an important factor governing onshore or offshore movement of beach material which determines the beach profile and breaking point/characteristic. It thus influences the beach watertable elevation/profile.

Wave Runup Infiltration

Recently, the wave runup infiltration contributing to significant lifting of the coastal watertable was stressed by several researchers (e.g. Nielsen et al., 1988; Hegge and Masselink, 1991 and Kang et al., 1994). Figure 4 outlines the processes of wave runup/infiltration and the coastal watertable.

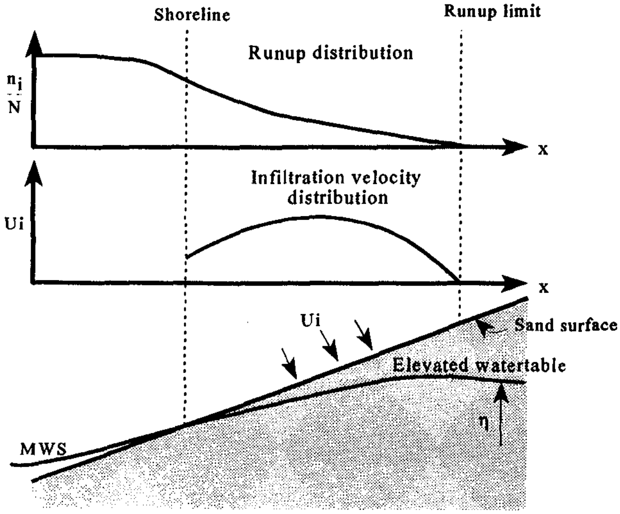

Figure 4. Relationships between wave runup, infiltration and the coastal watertable.

The top part of the figure shows the runup distribution (transgression statistics) of irregular waves, i.e. the fraction of the waves which transgressed certain points during the recording interval. The middle part shows the infiltration velocity distribution from wave runup, and the bottom part shows the corresponding elevated coastal watertable. With no sinks or sources landward of the runup limit the watertable will be horizontal landward of the runup limit.

Runup infiltration velocities indirectly determined from the watertable measurements and the parameters n and K of the porous medium, using the Finite

Difference approach, have maximum values roughly midway between the shoreline and the runup limit from both the field and laboratory data (see Kang et al., 1994 and Kang, 1996). General magnitudes of the infiltration velocity U_I are $0.08K$ for the steady laboratory conditions to $0.45K$ for field conditions during the rising tides. This difference is probably due to the fact that the field data were taken during rising tides where the beach is less saturated than in the steady state of the laboratory experiments. These magnitudes may vary according to characteristics of the porous medium.

Mathematical Modeling of the Coastal Watertable

Consider homogeneous sand body, with the specific yield n and the hydraulic conductivity K overlying a horizontal impermeable stratum at depth D. Under the Dupuit-Forchhermer assumption, the local height h of the watertable above the impermeable layer obeys the Boussinesq equation:

$$n\frac{\partial h}{\partial t} = K\frac{\partial}{\partial x}\left(h\frac{\partial h}{\partial x}\right) \tag{4}$$

In order to model the watertable in the zone of runup infiltration, the modified Boussinesq equation or the corresponding linearized equation (5) which includes the runup infiltration effect should be used as reported by Nielsen et al. (1988) and Kang et al. (1994):

$$n\frac{\partial h}{\partial t} = KD\frac{\partial^2 h}{\partial x^2} + U_I(x,t) \tag{5}$$

where $U_I\ (x,t)$ is the infiltration velocity averaged over numerous uprush-backwash cycles or the infiltration flow rate per unit area.

For steady flume condition (pure wave forcing, no tide), equation (2) leads to:

$$\frac{\partial^2 h}{\partial x^2} = -\frac{1}{KD}U_I(x) \tag{6}$$

The nature of $U_I(x)$ may reasonably be expected to be:

$$U_I(x) = \begin{cases} C_I K f(x) & \text{for} \quad x_s \le x \le x_R \\ 0 & \text{for} \quad x > x_R \end{cases} \tag{7}$$

where C is a dimensionless infiltration coefficient and $f(x)$ is a dimensionless function of x. x_s and x_R are the horizontal shoreline and the runup limit coordinates, respectively.

In order to define the solution for equation (6) under the steady laboratory conditions, consider a simple situation where the infiltration velocity $U_I(x)$ is distributed evenly between the shoreline x_S and the runup limit x_R. That is

$$U_I(x) \equiv U_I \tag{8}$$

Then equation (8) is written as:

$$\frac{\partial^2 h}{\partial x^2} = -\frac{U_I}{KD} \tag{9}$$

The boundary condition at the runup limit ($x=x_R$) may be written as:

$$\frac{dh}{dx}\Big|_{x=x_R} = 0 \tag{10}$$

and the boundary condition at the shoreline can be stated as:

$$h_{x=x_S} = D + \overline{\eta}_S \tag{11}$$

The solution to equation (9) with the boundary conditions (10) and (11) is :

$$h(x) = D + \overline{\eta}_S + \frac{U_I(x_R-x_S)^2}{KD}\left[\frac{x-x_S}{x_R-x_S} - \frac{1}{2}\left(\frac{x-x_S}{x_R-x_S}\right)^2\right] \quad , \qquad x_S \le x \le x_R \tag{12}$$

and for $x=x_R$, the asymptotic inland overheight is obtained as:

$$h_\infty = h_{x=x_R} = D + \overline{\eta}_S + \frac{1}{2}\frac{U_I(x_R-x_S)^2}{KD} \tag{13}$$

Hence we see that the asymptotic inland overheight is proportional to $(x_R-x_S)^2/D$. If $U_I(x)$ is always distributed in the same way between x_S and x_R, i.e.

$$U_I = KF\left(\frac{x-x_S}{x_R-x_S}\right) \tag{14}$$

where F is a universal function, equation (13) can be written as:

$$h_\infty = D + \overline{\eta}_S + C\frac{(x_R - x_S)^2}{D} \tag{15}$$

where C is a constant. A universal function F would give a universal constant C.

Numerical Modeling of the Watertable

Model Formulation

The numerical technique used here is the finite-difference approach. Consider the typical four-node grid with node spacings of δ_x and δ_t in each coordinate direction. The dependent variable $h(x, t+\delta_t)$ will be obtained from the following equation, which is a Taylor series expansion about node $h(x,t)$:

$$h(x,t+\delta_t) = h(x,t) + \delta_t\left(\frac{\partial h}{\partial t}\right) \tag{16}$$

By substituting the watertable equation (4) into (16), we get the governing equation:

$$h(x,t) = h(x,t) + \delta_t\frac{K}{n}\frac{\partial}{\partial x}\left(h\frac{\partial h}{\partial x}\right) \tag{17}$$

for the groundwater surface level h. In this way one equation is obtained for each node, which can be used to find $h(x, t+\delta_t)$.

In order to get the solution for the most seaward and landward wells, we need two boundary conditions. The first of these can be obtained using the glassy/dry boundary (G/D) which corresponds to the watertable exit point, as the seaward watertable boundary condition (see Figure 5). Hence, since the G/D is moving horizontally as well as vertically, the spacing δ_x from the G/D to the first well is not constant, i.e.

$$\delta_{x1} = \delta_{x2}\ , \qquad \textit{for the first well landward of MSS} \tag{18}$$

where δ_{x1} is the distance between the glassy/dry boundary and the first well landward of MSS, δ_{x2} is the distance between the first and second well landward of MSS and MSS defines the intersection between the mean sea level (MSL) and the beachface (cf. Figure 5).

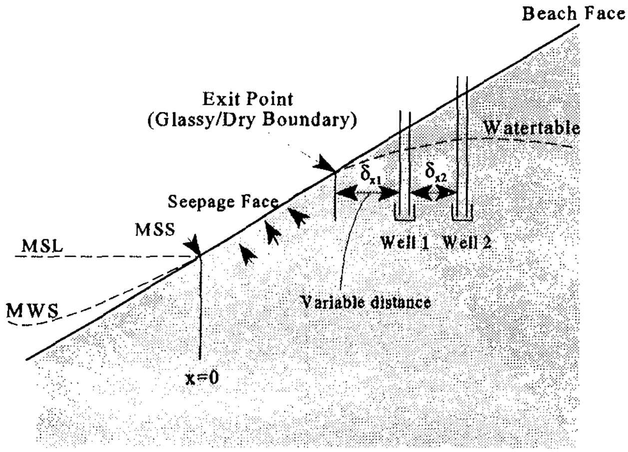

Figure 5. The moving glassy/dry boundary as the seaward watertable boundary condition

Secondly, at the landward boundary, a reflecting, no flux boundary condition is imposed. This is reasonably realistic since the watertable waves decay exponentially with distance from the shore. That is

$$h(x-\delta_x,t) = h(x+\delta_x,t) , \qquad for \quad x \to \infty \tag{19}$$

Numerical solution of equation (17) subject to the periodic boundary condition at the ocean and to (19) is then readily obtained by iteration. That is, in order to overcome the influence of the initial conditions, the model is run for several tidal cycles.

Comparison with Experiments

As an example, the watertable data measured for 25 hours from Kings Beach, Queensland are used because tides were approximately sinusoidal at the time of measurement.

Figures 6 and 7 show the time variations of simulated and measured watertables at 15m, 20m and 30m landward from the mean sea level shoreline with the variation of glassy/dry boundaries during two semidiurnal tidal cycles (25 hours period), respectively. The amplitudes of the watertable variation from the model give quite good agreement with those of the measured watertable. It is however seen that the simulated watertable levels always sit a bit higher than the measured watertable levels. This difference may be reduced by using the shoreline as a boundary condition because the shoreline always sits lower than the glassy/dry boundary level. Some improvement may also be obtained by varying hydraulic conductivity K and aquifer depth D at each well. The compactness of the beach sand body may vary in the shorenormal direction and hence hydraulic conductivity may vary shorenormally.

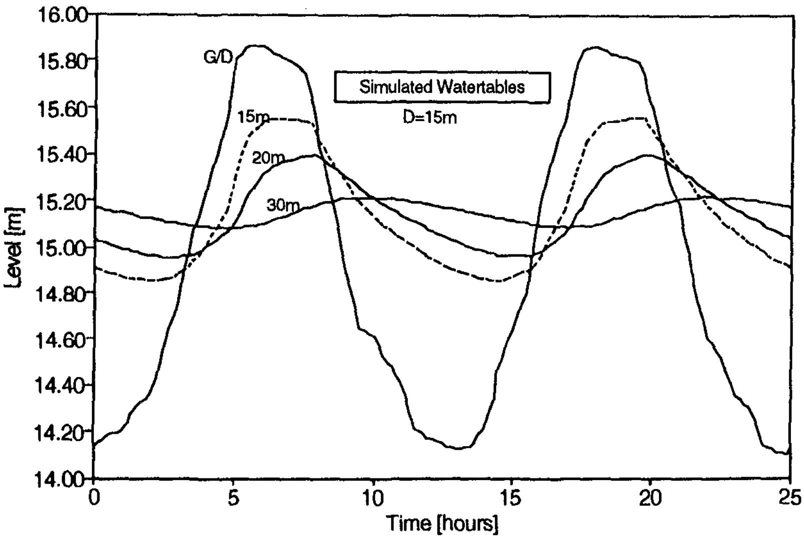

Figure 6. Simulated watertable variations during two semidiurnal tidal cycles at wells landward of the mean sea level shoreline. The numbers on the curves indicate distances landward from MSS.

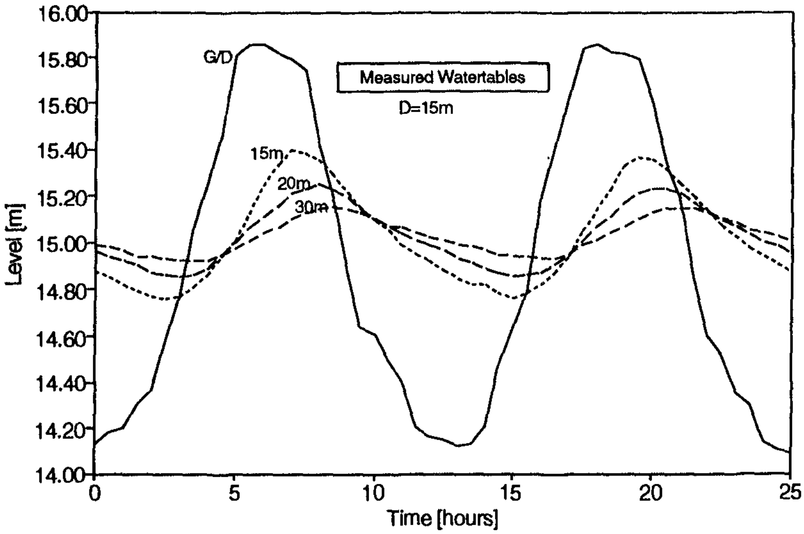

Figure 7. Measured watertable variations at the wells landward of the mean sea level shoreline from Kings Beach (Nov. 24-25, 1991).

Conclusions

Field measurements of groundwater dynamics in coastal barriers show landward downsloping watertable which results in a landward flow of salt water or waste water released into aquifer. Fresh water lens under the barrier tends to be much thinner than that predicted by the classical Ghyben-Herzberg principle.

Comparison between the laboratory experiments with regular waves and no tides and the field data (see Kang, 1996) has revealed significant difference in magnitudes of the infiltration velocity. It is obvious that the tidal phase is very important. Infiltration velocities are large when the shoreline moves landward on a partially saturated beach on a rising tide. General magnitudes of U_I are $0.08K$ for the steady laboratory conditions and $0.45K$ for field conditions during the rising tides.

A mathematical model of the watertable which includes the effects of wave runup infiltration was obtained in a steady state. The development of the numerical watertable model was also attempted with the glassy/dry boundary as a boundary condition. However, the numerical model of the watertable in this study is based on tidal fluctuations with the use of the wave-influenced G/D boundary condition.

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