NUMERICAL MODELING OF GROUNDWATER FLOW BEHAVIOR IN RESPONSE TO BEACH DEWATERING

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ABSTRACT

NUMERICAL MODELING OF GROUNDWATER FLOW BEHAVIOR IN RESPONSE TO BEACH DEWATERING

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In this study, The Beach Dewatering System, a relatively recent technology to combat beach erosion, which is proposed as a practical alternative to more traditional shoreline stabilization methods, is investigated and an informative overview on the genesis, development and recent use of this technique is provided. On the basis of the link existing between the elevation of beach groundwater and erosional or accretionary trends at the beach face, a numerical model that simulates groundwater flow in a coastal aquifer under beach drainage is presented. In this model, the seaward boundary of the domain is considered to be tidally fluctuating in a large scale to represent the occurrence of seepage face significantly. The unsteady groundwater flow equation is solved numerically using the method of finite differences. The results clearly showed that the water table being lowered caused the reduction of the seepage face which is the main aim of Beach Dewatering projects. The positional design parameters, i.e. horizontal and vertical location of the drain, are also investigated by utilizing an efficiency index. It is observed that the system efficiency decreased as the drain is shifted landward. The results also indicated that, the efficiency slightly increased with the vertical drain elevation.

Keywords: Unsteady Groundwater Flow, Beach Groundwater Table, Coastal Erosion, Seepage Face, Tide, Numerical Modeling.

ÖZ

YERALTI SUYU AKIMININ PLAJ DRENAJI ALTINDA DAVRANIŞININ SAYISAL MODELLENMESİ

Goler, Güneş

Yüksek Lisans, İnşaat Mühendisliği Bölümü

Tez Yöneticisi: Prof. Dr. Halil Önder

Ağustos 2004, 66 sayfa

Bu çalışmada, kıyı erozyonu ile mücadelede yeni geliştirilmiş bir teknoloji olan ve geleneksel kıyı stabilizasyonu metodlarına pratik bir alternatif olarak önerilen Plaj Drenajı Sistemi araştırılmış ve söz konusu tekniğin tanıtımı; kökenleri, gelişimi ve günümüzdeki kullanımı incelenerek yapılmıştır. Plaj yeraltı suyu tabakası ile plaj yüzeyindeki aşınma ya da birikim eğilimleri arasındaki bağlantı gözönüne alınarak, plaj drenajı altında bir kıyı akiferindeki yeraltı suyu akımını simüle eden bir sayısal model sunulmuştur. Bu modelde, sızıntı yüzeyi oluşumunu anlamlı olarak ifade edebilmek amacıyla, ilgilenilen alanın deniz tarafındaki sınırında geniş ölçekte bir gel-git dalgası düşünülmüştür. Zamana bağlı yeraltı suyu akımı denklemi, sonlu farklar metodu kullanılarak sayısal olarak

çözülmüştür. Elde edilen sonuçlarda yeraltı suyu tabakası seviyesinin düşürüldüğü ve buna bağlı olarak, plaj drenajı projelerinin temel amacı olan sızıntı yüzeyi indirgenmesinin gerçekleştiği görülmüştür. Ayrıca, drenajın yatay ve düşey yönlerdeki pozisyonlarını temsil eden tasarım parametreleri, bir verimlilik göstergesinden yararlanılarak incelenmiştir. Sistem verimliliğinin, drenaj yerleşimi kara tarafına kaydırıldığında azaldığı, yükseltildiğinde ise hafifçe arttığı saptanmıştır.

Anahtar Kelimeler: Zamana Bağlı Yeraltı Suyu Akımı, Plaj Yeraltı Suyu Tabakası, Kıyı Erozyonu, Sızıntı Yüzeyi, Gel-git Dalgası, Sayısal Modelleme.

To my family

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$\overline{Q}$	:	Mean drainage rate, [L^2T^{-1}]
$\overline{(R_{sp})^2}$	:	Square of seepage face reduction averaged over tidal cycle, [L^2]
A	:	Amplitude of the tide, [L]
g	:	Gravitational acceleration, , [LT^{-2}]
H	:	Elevation of the mean sea level, [L]
h(x,t)	:	The dependent variable water table elevation, [L]
H_1	:	Constant water elevation at the landward boundary, [L]
$H_2(t)$	:	Sea level varying with the diurnal tidal cycle, [L]
ID_e	:	Efficiency index, [-]
K	:	Hydraulic conductivity, [LT^{-1}]
L	:	Horizontal coordinate of the intersection point of the sea level and the beach face, [L]
L_1	:	Horizontal coordinate of the intersection point of the minimum sea level and the beach face, [L]
L_2	:	Horizontal coordinate of the intersection point of the maximum sea level and the beach face, [L]
L_a	:	Length scale of induced tidal effects, [L]
L_d	:	Horizontal coordinate of the drain, [L]
L_e	:	Horizontal coordinate of the intersection point of the water table exit and the beach face, [L]

L_m	:	Horizontal coordinate of the intersection point of the mean sea level and the beach face, [L]
n_e	:	Effective porosity of the porous medium, [-]
P_w	:	Pressure at the drain, [$ML^{-1}T^{-2}$]
Q	:	Drainage rate, [L^2T^{-1}]
S_y	:	Specific Yield, [-]
t	:	Time, [T]
x	:	Horizontal coordinate, [L]
z	:	Elevation head, [L]
z_d	:	Vertical coordinate of the drain, [L]
z_{Ed}	:	Elevation of exit point with drainage, [L]
z_{EO}	:	Elevation of exit point without drainage, [L]
ρ	:	Groundwater density, [ML^{-3}]
ω	:	Tidal frequency, [rad T^{-1}]

CHAPTER 1

INTRODUCTION

Beaches are temporary geological features composed of an accumulation of rock and shell fragments, ranging in size from fine sand to large boulders. Because the accumulation can be moved by changing wave action, the beach morphology is a dynamic one.

Many shorelines throughout the world are experiencing shoreline retreat due to damaging actions related to storm waves and their resulting generated currents (Vesterby, 2004). Sand conservation is a critical matter on many leisure and resort beaches on the earth, particularly the maintenance of sand inshore during high-energy conditions.

Different approaches have been utilized in attempt to solve or alleviate the problem of beach erosion since the loss of beaches threatens recreative areas and buildings, structures and has a direct impact on local and national economy. In combating the erosive forces from wave actions, soft engineering solutions are likely to be more effective and environmentally acceptable than hard, structural defenses.

An environmentally acceptable approach to the alleviation of beach erosion problems has been developed and comprehensively tested in practice – The Beach Dewatering System. This dynamically working Beach Face Dewatering System causes artificial interplay with nature's morphology through a localized slowdown of one natural process and speed-up of another, thereby tipping the balance of erosion. It involves the permanent installation of pipes and pumps, but once installed it is not a visible eyesore or physical obstruction as almost all components are buried underground (Vesterby, 1995).

1.1 Statement of the Problem

A link between elevation of coastal groundwater and erosion or accretion trends at the shoreline has been reported in the coastal literature for over sixty years. The origins of this work can be traced to parallel but initially unrelated strands of beach research in the 1940's that were simultaneously providing new insight into the role of swash infiltration in determining erosion or accretion at the beach face , and the dynamics of beach groundwater in controlling the saturation characteristics of the foreshore (Turner and Leatherman, 1997).

There are three major processes involved in this problem:

(1) Wave motion on the beach

(2) Coastal groundwater flow

(3) Cross-shore sediment transport in the swash zone (Fig. 1.1)

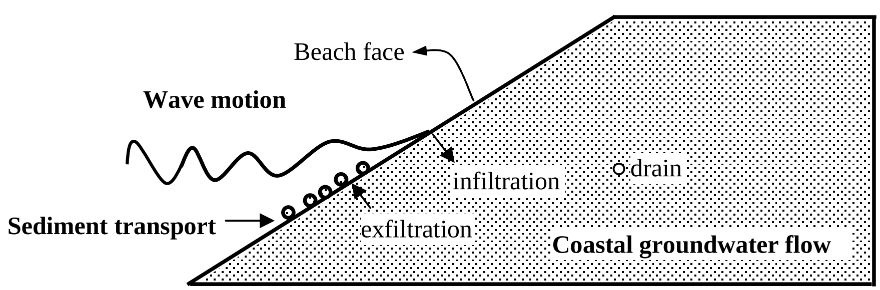


Figure 1.1: Beach Environment and Physical Processes

The waves themselves and the current generated by them are the primary reasons for the littoral transport. The transport of sediment material – both as suspended load and bed load – is associated with the dissipation of excess wave energy.

As waves run up and down a beach, providing the mechanism for sediment transport, they interact with the groundwater beneath the beach face. Especially in macrotidal beaches, during flood tide, the beach water table of the aquifer is low in comparison to the mean sea level. Water infiltrates into the aquifer as the waves run up (swash) above the exit point of the groundwater table. As a consequence, the fluid's sediment carrying capability decreases resulting in sediment deposition. Second the water volume and velocity of the run down (backwash) will be reduced due to infiltration and less sediment is transported offshore by the backwash. These effects enhance onshore sediment transport and hence beach accretion. Conversely, a relatively high water table exists during the ebb tide and water exhilarates from the aquifer into the sea. Under these conditions, opposite effects will occur, causing enhanced offshore sediment transport and beach erosion (Grant, 1948).

The correlation observed in the field between beach accretion/erosion and the relative position of the coastal groundwater table has led to beach dewatering projects. The aim of these projects is to promote onshore sediment transport and beach stabilization by artificially lowering the beach water table.

Also in microtidal beaches, lowering the groundwater table in the foreshore results the widening of the unsaturated zone on the foreshore. The unsaturated zone facilitates percolation of water from run up and backwash as well. With less water in the backwash and reduced run up height, less sand will be brought back to the sea than was brought up by the run up volume. The zone of lowered water table furthermore cuts off the local ground water flow towards the sea and the seepage through the beach face and dune toe. Thus stabilizing the slope and reducing the backwash quantity and velocity, the Beach Dewatering System decreases the erosive effect of backwash and seepage and leaves more sand on the beach face. (Fig. 1.2)

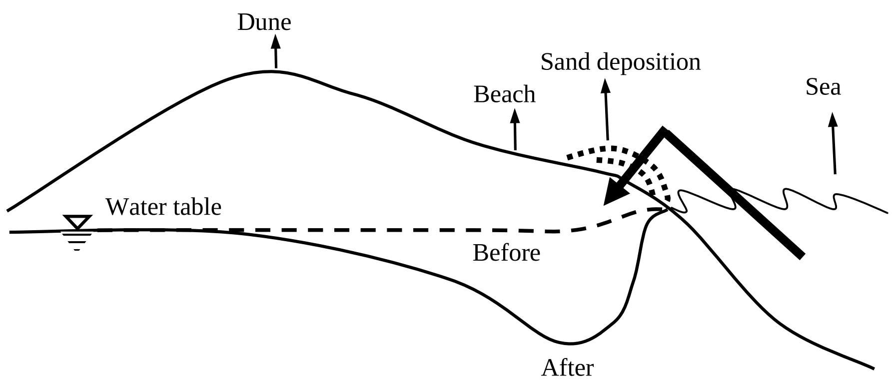


Figure 1.2: Sand Deposition due to Swash Infiltration into Coastal Aquifer

1.2 Scope of the Thesis

The objectives of this study are defined as follows:

1. Discussing the Beach Face Dewatering Concept – a recently available methodology regarding shoreline stabilization – as an appealing alternative to the more traditional techniques.
2. Investigating the coastal groundwater response to beach drainage comprehensively by a numerical model that also introduces some basic design parameters.

In the first chapter the statement of the problem and introductory comments are presented.

Chapter 2 provides a definitive and informative overview for the beach dewatering concept, containing a comparison with the traditional shoreline stabilization methods and reviewing its historical and current status.

Chapter 3 involves the theoretical background for the numerical model used and also covers information about the MODFLOW-2000 groundwater flow modeling code.

Chapter 4 covers the results of the numerical study.

Discussions and conclusions are presented in Chapter 5.

CHAPTER 2

THE BEACH DEWATERING CONCEPT: AN OVERVIEW

2.1 Introduction

When an incoming wave breaks or spills in the surf zone, it propagates to a level of the foreshore which is the sum of following components: tide, wave setup and wave run up. The level is determined by the wave energy, the slope of the foreshore, the roughness and the permeability of the beach deposits.

The run up carries sand as it swashes up the foreshore. During the up rush, the momentum decreases until a velocity of zero is reached whereby most sand is temporarily deposited on the beach face. As the prism of water accelerates down the foreshore, sand is picked up and moved back seawards. The net result under a specific set of conditions is often a quasi equilibrium, however often combined with long term erosion or accretion, depending on the overall littoral budget of the actual sediment cell.

The elevation of the groundwater table in the beach is partly determined by the prevailing wave and tide conditions, precipitation and landwards groundwater level and partly by the characteristics of the beach deposits, i.e. the grain size distribution and corresponding porosity, which determine the permeability and the storage coefficient.

The down rush volume and the velocity will be relatively high when the water table due to the beach deposits is high due to flux from the hinterland, due to the falling tide or due to percolating water from the run up. Under such circumstances the water will seep out of the seepage face which is located between the instant water level at the lower part of the foreshore and the exit point of the groundwater table further up on the foreshore.

In addition to the relatively high amount of sand picked up due to the high down rush velocity in this situation, the seepage forces, together with the pressure unloading at the surface, also loosen the sand and move the grains into the turbulent flow (Vesterby et al., 1999). The force required to move water through the soil is normally called the seepage force. It is present whenever and wherever water flows through the soil. The interfacial bed particles experience an upward force where the water flows out of the bed, and conversely a downward force wherever the fluid flows into the bed (Martin and Aral, 1971). This enhances the erosion and flattens out the fore shore. The mechanism is responsible for the generally very gentle slope of the foreshore on tidal dominated beaches, the so called tidal flats.

2.2 The Beach Dewatering Concept

"In concept artificial manipulation of beach groundwater is an appealing "soft" engineering solution to coastal erosion. Many coastal engineers and scientists are familiar with the basic idea; the water table within a sandy beach is lowered by buried drains with the objective to enhance sediment deposition at the beach face. The accelerated build-up of beach width accomplished during calmer wave conditions could provide a buffer to property or beach amenity that is otherwise threatened during episodic storm erosion. In contrast to repeated beach nourishment or the use of “hard” engineering structures such as groins and seawalls, the appeal of beach drains is that they could provide long-term coastal protection that has little or no impact on the aesthetic attraction of the protected beach (Turner and Leatherman, 1997).”

Without their beaches many of the world’s main tourist resorts would lose their appeal, and tourist revenues would fall. In severe cases of beach erosion the consequences can be even more dramatic as buildings and structures become undermined and crack or collapse. And sand removed from beaches can accelerate the filling-up of navigation channels thereby increasing maintenance costs.

Nature’s forces are awe-inspiring. However, nature’s own forces can also be used for eco-friendly – as well as invisible – coastal stabilization with no impact on human activity and the environment.

“The key – Beach face dewatering – produces gradients towards a drain, cuts off the natural groundwater seepage and creates an unsaturated zone of depression under the beach face allowing downwards percolation of water from the wave up-rush. This reduces the backwash on the beach face and limits the erosion process while depositing more sand on the beach resulting in a more stable profile (Vesterby, 1994).”

Beach drainage or beach face dewatering involves the localized lowering of the water table beneath and parallel to the beach face. This has been demonstrated to cause accretion of sand above the installed drainage system. Sand is in continual movement on a wet beach face due to wave and tidal action in the swash zone. Under specific conditions, beach drainage systems can halt beach erosion and promote sand accretion by adjusting the dynamic equilibrium that exists on sand beaches.

The accretion or erosion of a beach is influenced by a number of hydrodynamic forces in a beach surf zone. The effects and interaction of these sediment transport mechanisms have been studied since the 1940's. It is well understood that lowering the water table in granular soils improves their stability and eliminates the tendency for them to move (i.e., well-pointing). A number of theories have been proposed to explain the empirical evidence for sand deposition from beach drainage (i.e., backwash reduction, seepage reduction, liquefaction reduction). These and other theories continue to be studied around the world expanding our understanding of the complex interplay of forces that can be seen working on the beach face.

It is evident that lowering the water table under the beach eliminates buoyancy factors and reduces the lubricating effect between the grains, restoring the frictional characteristics of the sand. Percolation of 'swash water' into the beach means less backwash energy, which encourages suspended sand to settle out on the beach face. This is achieved by installing a drainage system in the beach that lowers the beach face water table, intercepting the flow of swash, tidal and inland ground water. Collection pipes are buried in the beach parallel to the coastline to create an unsaturated zone beneath the beach face. This unsaturated zone is achieved by draining the seawater away by gravity to a collector sump and pumping station. The sump and buried pumping station can be located at the back of the beach, where they are not readily visible. A typical pumping station might consist of two submersible electric pumps located in a buried concrete chamber. The only visible feature of the system may be the pump station control panel that regulates and monitors the pumps, sends data and receives control signals. (Fig. 2.1)

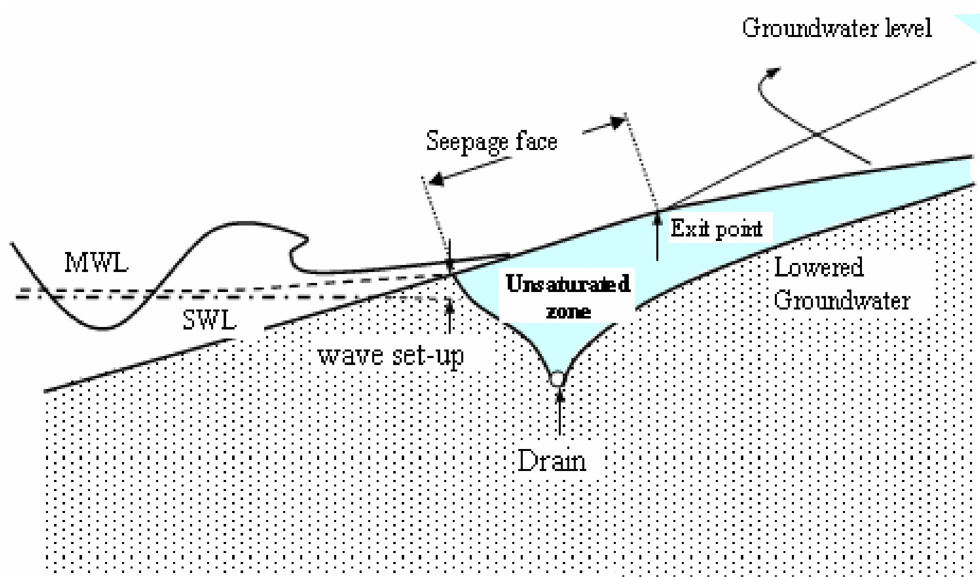


Figure 2.1: Water Table Lowering Resulting in Seepage Face Reduction and Widening of the Unsaturated Zone

2.3 Design of Beach Dewatering Systems

Presently, the design of beach drain systems relies heavily on experience, empirical models and scale model tests.

To design a beach drainage system, historical and current information is required on the following:

- permeability, uniformity and depth of beach sand layer(s)
- wave & wind climate
- tidal dynamics

- Materials budget in the littoral drift zone.

The viability of a proposed site must be confirmed before any detailed design is undertaken. This can be a staged process that may include pre-feasibility and feasibility studies.

The critical elements for the successful design of a beach drainage system are:

- the correct evaluation of the beach material with regard to the cone of depression for water table draw-down
- design, position and depth of the filter drains
- required flow calculations and subsequent pump and pipeline sizing.

All of the above elements are site specific. Common elements to all beach drainage systems are:

Filter drains: A number of designs and materials have been successfully employed. Existing system lengths ranging from 180 m to 600 m connected to a single pump station. Flow rates have ranged from 100 to 1400 m^3/hr. Pipe diameters have ranged from 50 mm to 450 mm at depths of 0.8 m to 2.5 m below MSL.

Pumping station: These can be very similar in design to common sewer pump stations. The required chamber depth will vary depending on total distance of gravity flow and ground elevation at the site of the pump station (commonly, 4 to 8 m deep). The pumping arrangement is normally gravity wet well with pressure discharge piping and submersible electric pumps. Piping materials are usually stainless steel. The control systems normally associated with the pump station are incorporated in a small steel cabinet.

Discharge pipeline: Design is dependent on discharge location (back to sea or to a sheltered location behind the beach) or whether water utilization is incorporated in the system.

The potential advantages of a beach drainage system include:

- The discrete nature and minimal environmental impact of the operating system compared with nourishment or existing hard engineering solutions
- Provision of a buffer zone from storm events and seasonal erosion and improved recovery time to pre-storm equilibrium following storm events
- Improved amenity value for recreational beaches by increasing the available high tide beach width/height and providing a dry beach surface between the tides
- Natural dune growth and rehabilitation will be encouraged adjacent to the installed system due to the increased availability of wind blown sand
- Protection of coastal fresh water environments from sea over-topping and seepage contamination
- Better natural character outcomes than hard engineering or nourishment
- A gentle form of sand replenishment – the risk of reef and beach habitat damage from nourishment activities is eliminated
- The cost per meter of coastal protection can be much lower than other solutions but this is dependent on system length and other cost sensitive factors
- The system may offer an easier route through local environmental consent and permitting procedures.

A useful side effect of the system is that the collected seawater is very pure because of the sand filtration effect. It may be discharged back to sea but can also be used to oxygenate stagnant inland lagoons/marinas or used as feed for heat pumps, desalination plants, land-based aquaculture, aquariums or seawater swimming pools. (Richards, 2002)

2.4 Comparison of Options

The following comparison table compares erosion options against six assessment factors (Table 2.1). This table has not been designed as an option selection tool. Scores would vary between beach erosion sites and expert opinion. The table is designed to highlight the differences and similarities between the various options, and with beach drainage technology (Richards, 2002).

The costs of installation and operation per meter of coastline treated by a beach face dewatering system will vary from project to project due to the following cost sensitive factors,

- system length (non-linear cost elements)
- pump flow rates (sand permeability, power costs)
- soil conditions (presence of rock or impermeable strata)
- discharge arrangement /filtered seawater utilization
- drainage design, materials selection & installation methods
- geographical considerations (location logistics)
- regional economic considerations (local capabilities/costs)
- Study requirements/consent process

The costs associated with a beach drainage system are generally considerably lower than hard engineered structures, particularly when long-term project economics are considered.

Table 2.1: Comparison of Coastline Erosion Protection Options (Richards, 2002)

		SOFT COASTLINE EROSION PROTECTION - Possible Options											Weighting Variations		
	1 = no benefit/high cost 2 = lower benefit/cost 3 = neutral/uncertain 4 = benefits/low cost 5 = max.benefit/no cost *ASSESSMENT FACTOR*	**revetment**	**sea wall**	**groyne**	**managed retreat**	**rock dumping**	**sand sausage**	**artificial reef**	**nourishment**	**do nothing**	**dune care**	**beach drainage**	***protection focused***	***cost focused***	***environment focused***
	shoreline protection	5	5	4	1	4	3	4	3	1	2	4	50	20	20
	cost (expenditure)	1	1	2	2	3	2	1	2	5	4	2	20	50	15
	natural character	1	1	1	4	1	3	3	3	4	5	5	15	15	50
	beach amenity value	1	1	3	3	1	4	5	5	2	4	5	5	5	5
	property value	4	4	3	1	3	4	5	5	1	4	5	5	5	5
	consent process	1	1	1	5	1	3	2	3	5	5	4	5	5	5
	raw score	13	13	14	16	13	19	20	21	18	24	25	100	100	100
WEIGHTED SCORES	***protection focused***	315	315	290	195	300	290	325	300	250	320	385			
	cost focused	195	195	230	225	270	260	235	270	370	380	325			
	environment focused	195	195	195	295	200	295	305	305	335	415	430			
	AVERAGE WEIGHTING	**235**	**235**	**238**	**238**	**257**	**282**	**288**	**292**	**318**	**372**	**380**			

2.5 Genesis of the Beach Dewatering Concept

The origins of the beach drain concept can be traced back sixty years to early work in two parallel fields of coastal research: the role of beach permeability in controlling erosion or accretion (e.g. Bagnold, 1940); and the tidal dynamics of beach groundwater (e.g. Grant, 1948). The installation within the last twenty five years of prototype beach dewatering systems in Europe (Vesterby, 1995) and the United States (Lenz, 1994) signified the transition of the beach dewatering concept from hypothetical to practical. The potential use of beach drain technology is beginning to be noted within the mainstream coastal engineering community and more frequent papers presented at coastal engineering conferences have served to raise the awareness of the beach dewatering concept.

The first researchers to propose that groundwater within beaches could be artificially manipulated to promote shoreline accretion were Machemehl et al. (1975); these USA coastal engineers undertook a laboratory study of the effect of a subsurface filter and pump system on the stability and accretion of the foreshore. The Australian researchers Chappell et al. (1979) were the first to experiment with pumping water out of natural beaches, and in 1983 the Danish Geotechnical Institute undertook the first prototype installation of a beach dewatering system at Hirtshals, on the northeast coast of Denmark (Ovesen and Schuldt, 1992).

In his classic paper describing laboratory experiments on beach formation due to waves, Bagnold (1940) concluded that beach face gradients are a function of the up rush energy dissipated above a given elevation relative to the total up rush energy passing that point. By inserting an impermeable barrier immediately beneath the sand surface (analogous to a high water table and saturated beach face), infiltration was inhibited and the energetic of the backrush enhanced. This simple experiment demonstrated perhaps the first time that infiltration in to an unsaturated beach face will result in enhanced onshore transport and steeper gradients, relative to a saturated ("impermeable") beach face. A field demonstration of the same phenomenon was reported by Longuet-Higgins and Parkin (1962), who inserted roofing, felt 10 cm. below the surface of a shingle beach. The shingle overlying the impermeable layer was observed to erode quickly, in contrast to little disturbance either side. Again, the reduction of swash infiltration – corresponding to a high water table – was observed to enhance offshore sediment transport relative to the permeable (unsaturated) regions either side. These two studies clearly demonstrated that the potential for swash infiltration is an important mechanism controlling observed erosional and accretionary trends at the beach face.

Coincident to Bagnold's seminal laboratory investigation, Emery and Foster (1948) undertook the first published study describing the dynamics of the water table in sandy beaches. Emery and Foster speculated that bed dilation due to groundwater seepage may result in beach face erosion in tidal ebb.

The first explicit link between the elevation of groundwater and erosional and accretionary trends on sandy coastlines was proposed by Grant (1946, 1948). From observations of the fluctuating width and slope of southern Californian beaches spanning several years, Grant recognized that the elevation of the beach water table had an important bearing on deposition and erosion across the foreshore. Grant concluded that a lower water table (unsaturated beach face) facilitates deposition by reducing flow velocities during backwash and prolonging laminar flow.

The combined insights of Grant (1948) and Emery and Foster (1948) were interpreted by Duncan (1964) to explain a cyclic pattern of beach face cut and fill monitored through a semidiurnal tidal cycle on Manhattan Beach, Santa Monica Bay, California. Duncan concluded that deposition is promoted by run up infiltration and hence a loss of swash volume as the swash zone extends to the unsaturated beach face.

Cyclic erosion and accretion of the beach face, as a function of relative elevations of the water table and swash zone, has since been substantiated by a number of researchers. Repeated profiling undertaken by Strahler (1966) on the New Jersey Atlantic open coast revealed a similar pattern of tidal cycle beach response, superimposed on a late-summer period of beach equilibrium. Nordstrom and Jackson (1990) describe comparable tidal cycle changes in beach morphology from a protected estuarine environment. Other studies reporting similar tidal cycle adjustment of the intertidal profile include Otvos (1965) and Schwartz (1967). Harrison (1969) undertook a multivariate analysis of foreshore changes through the tidal cycle. Measuring 15 environmental variables concurrently, he demonstrated a strong empirical relationship between water table elevation and foreshore erosion and slope.

Packwood (1983) described a numerical model to calculate the influence of a porous bed on the run up of a bore on a gently sloping sandy beach. Significant differences between impermeable and porous bed solutions are found in the backwash which might explain certain sand erosion and deposition phenomena. Eliot and Clarke (1988) applied a relatively sophisticated moving-axis technique to distinguish between beach face erosional and depositional states, and confirmed that maximum degradation occurred when the beach face was most saturated, and the unsaturated region above the water table outcrop established a zone of beach face deposition.

Baird and Horn (1996) made a review on previous work on groundwater behaviour in sandy beaches. It is noted that most work on beach groundwater processes has tended to be empirical and that if understanding of beach groundwater/swash zone sediment transport interactions is to be improved, better measurement and physical representation of the relevant processes are needed. Turner and Leatherman (1997) traced the origins and development of the dewatering concept, from early work on beach face permeability and beach groundwater dynamics, to recent field and laboratory studies that have explicitly examined the effect of artificial groundwater manipulation on beach face accretion and erosion. It is concluded that the effectiveness of the dewatering concept in maintaining beach stability and controlling coastal retreat is yet to be convincingly demonstrated at the prototype scale.

Li et al. (1996a), as an initial point of a chain of studies, presented a BEM (Boundary Element Method) model for simulating the tidal fluctuation of the beach groundwater table. The model solves the two-dimensional flow equation subject to free and moving boundary conditions, including the seepage dynamics at the beach face. The model replicated three distinct features: steep rising phase versus flat rising phase, amplitude attenuation and phase lagging. Li et al. (1996b) have also applied the boundary element method to solve the Laplace equation for the velocity potential in modeling coastal groundwater response to beach dewatering. Simulations were conducted on two different kinds of drainage system, i.e. artificial and gravity drainage. The simulation results showed the watertable being lowered and the seepage face reduced due to the drainage as observed in the field and laboratory experiments. Other information from the simulations in addition to the watertable elevation, the drainage rate , etc. can be used for designing a beach drainage system.

Turner and Nielsen (1997) noted the existence of a zone of saturated sand above the water table results in the observed and rather striking phenomenon of rapid and large magnitude fluctuations of the phreatic surface. Li et al. (1997) developed a modified kinematic boundary condition for the watertable, which takes into account those capillarity effects. The new kinematic boundary condition was incorporated into a BEM model. The model was then applied to simulate watertable response to high frequency sea level oscillations in a rectangular domain. The model was also used to simulate the watertable response to wave runup at a sloping beach.

Masselink and Li (2001) used a process-based numerical model to examine in detail the role of swash infiltration in determining the beachface gradient. It is found that swash infiltration increases the onshore asymmetry in the swash flow thereby enhancing onshore sediment transport and resulting in relatively steep beachface gradients.

Li et al. (2002a) derived 2-D analytical solutions to study the effects of rythmic coastlines on tidal watertable fluctuations. The computational results demonstrated that the alongshore variations of the coastline can affect the water table behaviour significantly, especially in areas near the centers of the headland and embayment. Li et al. (2002b) presented a process-based numerical model that simulates the interacting wave motion on the beach, coastal groundwater flow, swash sediment transport and beach profile changes. Results of model simulations demonstrated that the model replicates accretionary effects of a low beach water table on beach profile changes and has the potential to become a tool for assessing the effectiveness of beach dewatering systems. Using this 2-D numerical model - termed BeachWin – with a set of field data, Alaee and Moghaddam (2002) examined how dewatering system can be used against coastal erosion in the southern coastlines of the Caspian Sea.

2.6 Laboratory Studies

The first engineers to investigate the possibility of artificial groundwater manipulation to control coastal erosion were Machemehl et al. (1975) who undertook a laboratory study of the effect of a sub sand filter system on the stability and accretion of the foreshore. A two-dimensional wave flume was used, and four tests with varying monochromatic wave heights were undertaken. The removal of water from within the beach was observed to greatly accelerate accretion at the foreshore and proved effective in promoting the growth or replacement of a previously eroded berm. Kawata and Tsuchiya (1986) report similar results, utilizing both single and multiple wave experiments. Tests were performed under both “normal” (low wave steepness) and “stormy“(high wave steepness) conditions and the dewatering system were observed to have a positive effect in both cases.

Ogden and Weisman (1991) undertook 2-D tests using irregular waves ranging from erosive to accretive and concluded that for the range of conditions tested, the beach drain had no significant effect on the rate of erosion or accretion at the still waterline, but did promote berm development and hence resulted in beach face steepening. Weisman et al. (1995), examined the effectiveness of beach dewatering under the influence of the tides, and concluded that water table lowering maintains its effectiveness in promoting berm growth and beach face steepening for both tidal and non-tidal cases. Heaton (1992) undertook a series of single and multiple wave experiments, and quantified a general trend that increasing water table elevation resulted in an increasing volume of sediment eroded from the beach face. Oh and Dean (1992,1994) report a set of three experiments where the water table was alternatively elevated, lowered and equal to mean sea level, and concluded that an elevated water table resulted in the overall destabilization and erosion of previously marginally stable regions of the beach face. A simple seepage model (Oh and Dean, 1994) demonstrated that outflow across beach face may act to reduce the effective weight and hence stability of surficial sediment. A somewhat inconclusive laboratory study is reported by Sato et al (1994) in which it is apparent that the positive effect of a beach drain installed within a 3-D wave basin was dominated by an unrealistic shoreward flow induced in the basin due to the rapid rate of groundwater pumping .

Herrington (1993) reported details of seven tests under varying wave conditions in a large (approximately 100 m long x 3.5 m wide) wave flume subject to both regular and irregular waves. Adjacent dewatered and non-dewatered test sections were subjected to waves of varying steepness within the range of 0.007 (swell) to 0.04 (storm). The overall conclusion of the study was that the dewatered text section exhibited greater stability than the adjacent non-dewatered test section.

As being the first step of the long-term researches started in the İstanbul Technical University Hydraulics Laboratory, Günaydın et al. (2001) presented the results of the experimental studies which were performed using regular waves in a 22.5 m long and 1 m wide wave flume, indicating the effects of the changes in the groundwater level on the coastal stability.

2.7 Field Investigations and Commercial Installations

The application of beach dewatering technology in the field has taken several forms. Chappell et al. (1979) first made the transition from the laboratory to prototype scale, as a series of mechanical beach dewatering wells were installed on the southern coast of New South Wales, Australia. Chappell et al. report qualitative evidence that the accretion of beach material on the foreshore of the profile can be induced by lowering the near-coast groundwater elevation. Due to the highly dynamic shoreline, the investigators were unable to quantify the influence of the wells on the morphologic response of the beach.

In 1981, the Danish Geotechnical Institute (DGI) installed a water filtration system in the beach at Hirtshals in Thorsminde, on the northern coast of Denmark (Vesterby 1991; Ovesen and Schuldt 1992; Vesterby, 1994). The filtration system was designed to pump seawater from below the swash zone to provide water for heat pumps and aquaria located at the Danish North Sea Research Center (DNRC). The filtration system pumped approximately 400 m^3/h, and originally consisted of a 200 m section of 0.2 to 0.3 m perforated PVC pipe buried in a shore-parallel orientation 2.5 m below mean sea level (MSL) 5 m landward of the shoreline. Following 6 months of operation, the volume of water supplied to the DNRC by the filtration system discharge pumps was substantially reduced. A site inspection of the beach revealed that the shoreline in the vicinity of the drain pipe had prograded 20 to 30 m seaward, lengthening the filter path and subsequently decreasing discharge yield by 40 percent. To increase discharge, a second 220 m drain line was installed as an extension of the first. The result of the extension was that the shoreline composed of well-sorted, medium-grain sand, in the vicinity of the horizontal wells prograded seaward, even during the winter storm season.

In 1983, a second site (termed Hirtshals East) was chosen by DGI to field test the effect of beach dewatering on shoreline response and was located 1km from the DNRC site. A 20 m drain pipe was installed in a beach composed of mixed fine grained sand, silt, and clay. The Hirtshals East project was terminated after 8 months of operation. During the 8 month evaluation period, the system was unable to prevent severe storm-induced erosion of the beach. However, the system did function to accrete beach material even under less-than-ideal soil conditions. Shoreline response to mechanical beach dewatering at the Hirtshals sites was deemed encouraging and prompted the first full-scale demonstration of beach dewatering technology as a shoreline stabilization method.

DGI installed a 500 m long, 0.2 m diameter perforated drain pipe at Thorsminde on the west coast of Denmark in 1985 (Vesterby 1991, Ovesen and Schuldt 1992). The drain pipe was buried at an elevation between -2.0 m and -2.5 m in a shore-parallel orientation in a beach composed of gravel and well-sorted, medium-grained sand superimposed on a layer of fine-grained lagoonal deposits occurring below -3.5 to -5.0 m. The Thorsminde system operated until 1991, when the demonstration was intentionally terminated. Monitoring of the demonstration by DGI revealed that after 7 years of operation, the dewatered beach accreted approximately 30 m^3/m of beach material, while neighboring control beaches experienced approximately 25m^3/m of erosion. Monitoring of the system also revealed that the drain had an effective length 100 m to 200 m longer than the actual drain pipe, and that no negative environmental effects were observed.

In 1988, under patent license to DGI, CSI (Coastal Stabilization Inc.) installed a 580 ft long STABEACH system at Sailfish Point (Stuart) on the Atlantic coast of Florida in a shore-parallel orientation (Terchunian 1989, 1990; Ovesen and Schuldt 1992; Lenz 1994). The system was installed in a beach composed of medium-grained sand and initially yielded approximately 75 liters/second of discharge. The system was comprised of a 0.46 m diameter collector header with 1.2 m long horizontal well points attached on 3 m centers. Following 11 months of operation, Dean (1990) concluded in an independent evaluation that it was not possible to differentiate between system induced and naturally occurring morphologic changes, and that there were no adverse effects of the system on the beach. Subsequent monitoring of the Sailfish Point installation led Dean to conclude that the

STABEACH System had a positive effect on the shoreline. Dean(1990) observed that the system induced moderate accretion on the dewatered shoreline, while adjacent non-dewatered beaches experienced erosion, and the dewatered beach was considerably more stable than adjacent non-dewatered beaches. Again, Dean observed no adverse effects on beach dynamics within the system’s influence.

Figure 2.2: The Accretionary Effect of Beach Dewatering, Sailfish Point, USA

In 1993, CSI installed a second beach dewatering system at Englewood Beach located on the Gulf Coast of Florida (Lenz, 1994). The system consisted of a series of well points along a 600 ft reach of shoreline. Following a limited operational period, the system was rendered inoperable by a series of storm events, and not replaced.

In 1994, newly constructed European commercial beach dewatering installations included a 600 m long Beach Management System installed by DGI at Enoe Strand on the south coast of Denmark, and a 180 m long Beach Management System at Towan Bay, Cornwall, UK. The Towan Bay system was constructed by MMG Beach Management Systems Ltd.,, UK, under patent license to DGI. In an early evaluation, Burstow (1995) reported a general accretionary trend of beach material on the foreshore at Towan Bay.

Beach drainage systems have been installed in many locations around the world to halt and reverse erosion trends in sand beaches. 33 BD systems have been installed since 1981 in Denmark, USA, UK, Japan, Spain, Sweden, France, Italy and Malaysia with 4 more under construction or approved for installation in 2004.

The figure below shows the progression of Beach Dewatering system installations around the world since 1981 to 2004:

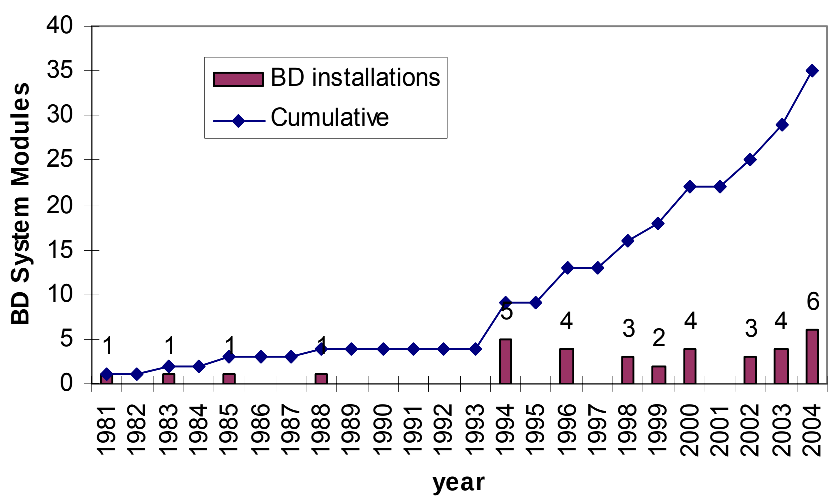


Figure 2.3: The Progression of Beach Dewatering system installations

The following table (Table 2.2) summarizes the Beach Drainage Installations since 1981 to 2004:

Table 2.2: Beach Dewatering Installations since 1981 to 2004

Beach Drain Module	Project	Year of Installation	Length of System (m)	Tidal Range (m)	Initial Beach Slope	Sand Grain Size (d_{50}/U)	Pump Capacity (m^3/h)	Drain Materials Installation Method	Comments
1	Hirtshals W, Denmark	1981	200	1.5	1:20	0.26/1.7	400	a-1	25, 000 m^3 sand harvested each year to renourish other beaches
2	Hirtshals E, Denmark	1983	200	1.0	1.25	0.2/1.3	100	a-1	Width maintained (back-ground erosion rate: 7 m/ year)
3	Thorsminde, Denmark	1985	500	1.5	1.25	0.35/1.7	700	a-1	Experimental system, width increased by 25 m
4	Sailfish Point, Florida, USA	1988	177	0.8	1.25	0.3/3	340	b-1	Width increased by 20-25 m during operation. Ceased operation following neighboring nourishment program.
5	Englewood Beach, Florida, USA	1993	200						Experimental well-point system. Damaged by storm events & not reinstated.
6	Enoe Strand, Denmark	1994	600	0.5-1.0	1:15	0.25/2.3	300	c-2	Width increased by 3 m August 1996. Maintained.
7	Towan Bay, U. K.	1994	180	7	1:45	0.2/1.7	200	d-1	Improved amenity (dry low tide). Exposed seawall footing safeguarded
8	Codfish Park Nantucket I, MA, USA	1994	357	1.0-1.5	1:45	1.5/4.2	700	e-3	Decreased in shoreline width due to storm events. Shoreline erosion rate in the treated areas has been reduced compared to untreated areas.
9	Lighthouse S Nantucket I, MA, USA	1994	309	1.0-1.5	1:06	0.8/3.2	1400	e-3	
10	Lighthouse N Nantucket I, MA, USA	1994	405	1.0-1.5	1:06	0.4/3.7	1400	e-3	
11	Holme Beach, Norfolk, U. K.	1996	200	3.5					Temporary trial system at remote nature reserve
12	Chigasaki- Naka Beach, Japan	1996	180	1.6	1:10	0.5/4	500	f-3	Temporary shut down due to typhone damage. Repaired and reactivated. Shoreline stabilized. Beach level increased.
13	Riumar I, Ebro Delta, Spain	1996	300	0.2-0.4	1:20	0.2/1.4	290	g-1	Width maintained after severe storm event in Oct 97.

Table 2.2: (continued)

Beach Drain Module	Project	Yearof Installation	Length of System (m)	Tidal Range (m)	Initial Beach Slope	Sand Grain Size (d_{50}/U)	Pump Capacity (m^3/h)	Drain Materials Installation Method	Comments
14	Hornbaek W, Denmark	1996	450	0.2-0.4	1:10	0.3/2	170	g-2	Width increased by 0-5 m, May 1997. Seasonal fluctuations reduced.
15	Hornbaek E, Denmark	1996	530	0.2-0.4	1:20	0.3/2	325	g-2	
16	Ystad, Sweden	1998	200	1.0	1:15	0.3/3	240	g-2	Beach width increased 10-15 m on the lea side of the 90 metres long groyne 1999
17	Branksome Chine, Dorset, UK	1998	100	2.0	1:18	0.25/1.6	65	g-4	Experimental system. Increased beach level
18-19	Hitotsumatsu Beach, Japan	1998	800	2.0	1:20	0.25/2	600	g-3	Accretionary trend. Beach level increased. 200 metres foreshore treated by 4 drain structures in parallel.
20	Les Sables d'Olonne, France	1999	300	6	1:70	0.25/3	250	g-3	Accretionary trend and substantial foreshore dry up in the drain zone.
21	Riumar II, Ebro Delta, Spain	1999	300	0.2-0.4	1:20	0.25/1.6	400	g-1	Beach width increased 6-8 m. 2000
22	Markgrafenheide, Germany	2000	300	0.3	1:30	0.7/2.6	300	g-3	Width increased by 8-10 m in October 2000.
23	Lido di Ostia I, Italy	2000	115	0.3	1:40	0.25/2	140	g-2	Beach width increased approximately 10 m. September 2001.
24	Lido di Ostia II, Italy	2000	90	0.3	1:40	0.25/2	140	g-2	Beach width increased approximately 10 m Febuary 2001. Drain pipe exposed April 2001 due to storm event. Reinstalled September 2001. Beach width increased to initial position.
25	Lido di Ostia III, Italy	2000	175	0.3	1:40	0.25/2	140	g-2	Beach width increased approximately 15 m. September 2001.
26	Kota Bharu I, Malaysia	2002	500	0.6	1:7	0.4/1.5	1000	h-1	Storm damage following pump commissioning delay.

Table 2.2: (continued)

Beach Drain Module	Project	Yearof Installation	Length of System (m)	Tidal Range (m)	Initial Beach Slope	Sand Grain Size (d_{50}/U)	Pump Capacity (m^3/h)	Drain Materials Installation Method	Comments
27	Kota Bharu II, Malaysia	2002	500	0.6	1.10	0.4/1.5	1000	h-1	Pump commissioning delay.
28	Les Sables d'Olonne (II), France	2002	600	6					Scientific monitoring by the University of Nantes.
29-32	Procida Island II, Italy	2003							Scientific monitoring by the Bari University.
33	Villers sur Mer, France	2003	300	7					Scientific monitoring by the University of Caen.
CONFIRMED PROJECTS (currently being designed or installed):									
34	Saint Raphaël, France	2004	600	0.4					Scientific monitoring by the University of Aix/ Marseille.
35	Port Dickson, Malaysia	2004	400	1.6	1:15	0.2/2	400	h-4	Installation in progress.
36	Morib, Malaysia	2004	200				200	h-1	Installation in progress.
37	Ravenna, Itally	2004							Status unknown

NOTES

U (Sand Grain Size) = Uniformity Coefficient, d_{60} /d_{10}

Drain Materials Installation Method

a Epoxy cemented filter sand around PVC perforated pipe

b Horizontal well points with epoxy cemented sand filter attached to PVD pipe

c Flexible perforated corrugate pipe with filter saled and geotextile cover (at bottom side)

d Perforated PVC pipe with gravel wrapped in geotextile

e Flexible PE perforated corrugated pipe with geotextile stocking

f Flexible perforated corrugated pipe with filter gravel 90 m and without filter gravel 90 m

g Flexible perforated corrugated pipe with geotextile stocking and filter gravel

h 'rib- lock' PVC with geotextile sock

1 Backhoe /well points

2 Plough

3 Trench machine

4 Backhoe

2.8 Concluding Remarks on Literature Review

The detailed review on the Beach Dewatering Concept provided in this chapter can be summarized as follows:

1. A link between the elevation of coastal groundwater and erosion or accretion trends at the shoreline has been reported in the coastal literature for over sixty years. The origins of this work can be traced to parallel but initially unrelated strands of beach research in the 1940's that were simultaneously providing new insight into the role of swash infiltration in determining erosion and accretion on the beach face, and the dynamics of beach groundwater in controlling the saturation characteristics of the foreshore.
2. In the mid 1970's the first laboratory investigations were reported that examined the artificial lowering of beach groundwater as a method to promote shoreline accretion and stability, and the results proved encouraging. By the late 1970's the results of the first field investigation of this approach were reported, but the results of this work were less conclusive.
3. Commercial interest in beach dewatering was initiated in the early 1980's on the Danish coast.
4. A full scale test of the dewatering concept on the open Atlantic coast of Denmark was undertaken during the period 1985 to 1991. Initial results proved encouraging , and for the first two and a half years of the system's operation published data suggest that, relative to the uncontrolled sites the dewatered beach stabilized and showed a positive trend of shoreline accretion .

5. In 1988 a second prototype dewatering installation was undertaken on a protected US Atlantic beach, and again early results proved promising. An independent assessment of the effectiveness of the system concluded after approximately two years of system operation that the treated section of beach had both stabilized and induced local moderate accretion.
6. 25 Beach Drainage systems have been installed since 1981 in many locations around the world to halt and reverse erosion trends in sand beaches with 4 more under construction or approved for installation in 2004.
7. Presently, the design of beach drainage systems is site specific and historical and current data on wave climate, sediment transport characteristics and groundwater table level variation of the proposed site are required. After performing the empirical and scale model tests, the common elements in the system such as drains and pumps with proper dimensions and locations can be installed and operated.
8. The advantages of the Beach Dewatering system relies mainly on its “soft engineering solution” character, i.e. the indirect impacts introduced to the nature’s morphology by the system. The costs associated with a beach drainage system vary from project to project, but they are generally considerably lower than the former solutions such as groin systems, particularly when long-term project economics are considered.

CHAPTER 3

THEORETICAL CONSIDERATIONS

3.1 General

The problem that will be investigated is the determination of timely variation of the groundwater table level in an unconfined coastal aquifer under beach drainage and subject to a permanent hydraulic connection with tidally fluctuating sea, for the purpose of observing the water table lowering and seepage face reduction. The aquifer is homogeneous and isotropic.

A modular groundwater model, called MODFLOW-2000, is used to simulate the flow below the water table in the beach. MODFLOW-2000 is a computer program that numerically solves the governing partial differential groundwater flow equation for a porous medium by using a finite-difference method. An informative knowledge on this program is provided in section 3.3.

3.2 Mathematical Model

To investigate a groundwater flow problem, its mathematical statement must be developed. A complete mathematical statement consists of five parts (Bear, 1979). Referring to Fig. 3.1a, Fig. 3.1b and Fig. 3.2, these are discussed as follows:

1. Flow region:

   $L \geq x \geq 0$ (where, $L_1 \geq L \geq L_2$) (3.1)

2. The dependent variable:

   h(x,t)

3. Governing partial differential equation:

$$\frac{\partial}{\partial x}\left[Kh\frac{\partial h}{\partial x}\right] = S_y\frac{\partial h}{\partial t} \tag{3.2}$$

4. Initial condition:

$$h(x,0) = \left[\frac{H - H_1}{L_m}\right]x + H_1 \tag{3.3}$$

5. Boundary conditions :

   $h(0,t) = H_1$ (prescribed head on $|AB|$) (3.4a)

   $h(x,t) = z$ (on $|CD|$, i.e. the seepage face, $L \geq x \geq L_e$) (3.4b)

   $h(x,t) = H_2(t)$ (on $|DE|$, $L_1 \geq x \geq L$) (3.4c)

   where, $H_2(t)$ changes with the tide, i.e.,

$$H_2(t) = H + A\cos(wt + \frac{\pi}{2}) \quad \text{(Li et al., 1996b)} \tag{3.4d}$$

   At the drain, the internal boundary condition is head prescribed:

   $h(L_d,t) = z + P_w/\rho g$ (3.4e)

   where,

   K is the hydraulic conductivity

   S_y is the specific yield for the porous medium

   P_w is the pressure at the drain

ρ is the groundwater density

g is the magnitude of gravitational acceleration

x is the horizontal coordinate

z is the elevation head

A is the amplitude of the tide

w is the tidal frequency

t is the time.

The governing partial differential equation in Eq. (3.2) is the one-dimensional non-linear groundwater equation and is used to describe unsteady flows. Once the boundary conditions are specified, Eq. (3.2) can be solved using numerical techniques, a finite-difference method is employed in this study by using MODFLOW-2000.

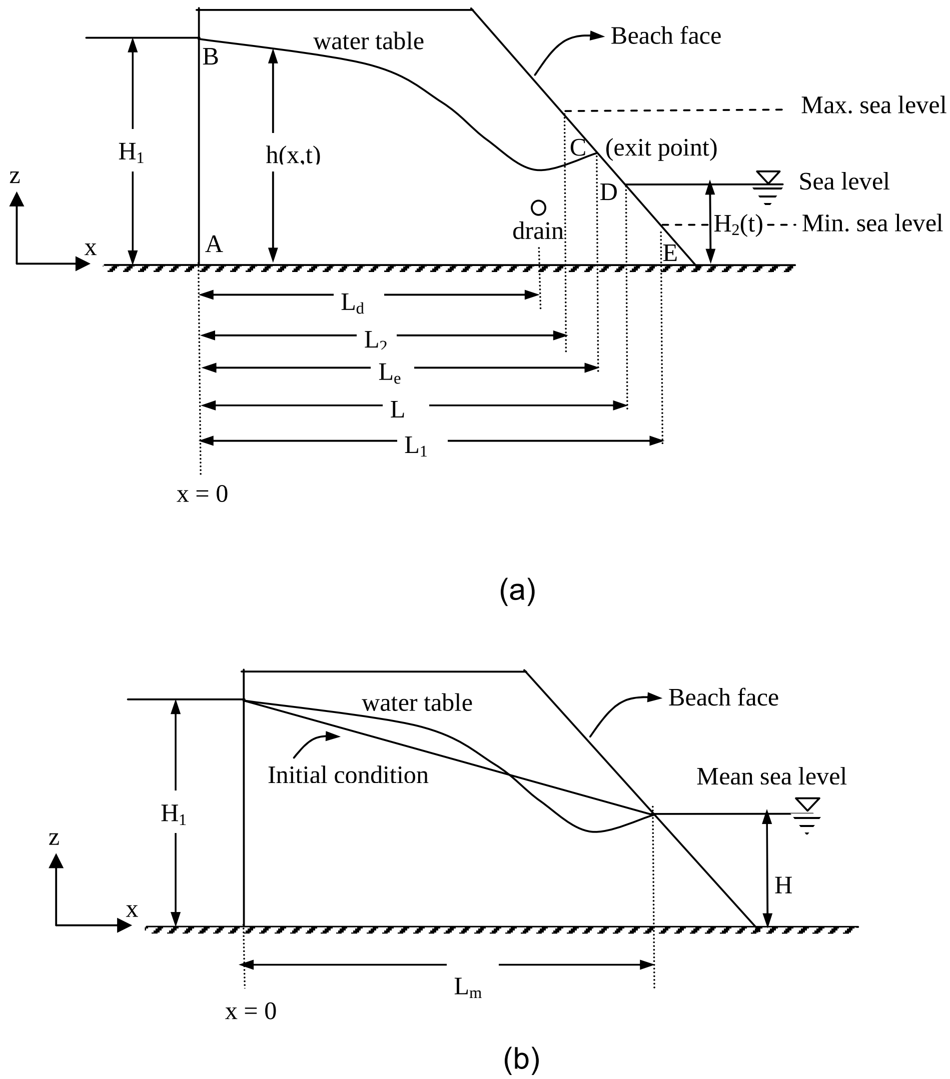


Figure 3.1: Boundary and Initial Conditions: (a) Boundary Conditions for the Aquifer Interacting with the Tide under Beach Drainage; (b) Sketch of the Approximate Initial Condition for the Groundwater Table

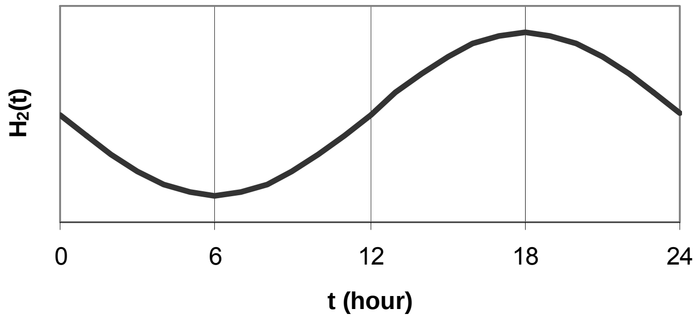


Figure 3.2: Variation of $H_2(t)$ with the Diurnal Tidal Cycle

3.3 Information on the Model Code

MODFLOW (McDonald and Harbaugh, 1988; Harbaugh and McDonald, 1996; Harbaugh et.al., 2000) is a three-dimensional finite difference groundwater model which has a modular structure that allows it to be easily modified to adapt the code for a particular application. MODFLOW-2000 is a recent version of the original model, which includes many new capabilities. MODFLOW-2000 is written in Fortran77.

MODFLOW-2000 simulates steady and unsteady flow in an irregularly shaped flow system in which aquifer layers can be confined, unconfined, or a combination of confined and unconfined. Flow from external stresses, such as flow to wells, areal recharge, evapotranspiration, flow to drains, and flow through river beds, can be simulated. Hydraulic conductivities or transmissivities for any layer may differ with space and direction (restricted to having the principal directions aligned with the grid axes), and the storage coefficient may be variable in space. Specified head and specified flux boundaries can be simulated as can a head dependent flux across the model's outer boundary that allows water to be supplied to a boundary block in the modeled area at a rate proportional to the current head difference between a "source" of water outside the modeled area and the boundary block. MODFLOW is currently the most used numerical model in the U.S. Geological Survey for groundwater flow problems.

3.4 Solution Procedure by the Finite Differences Algorithm

The groundwater flow equation is solved using the finite difference approximation. The flow region is subdivided into blocks in which the medium properties are assumed to be uniform. In plan view the blocks are made from a grid of mutually perpendicular lines that may be variably spaced. Model layers can have varying thickness. A flow equation is written for each block, called a cell. Several solvers are provided for solving the resulting matrix problem; the user can choose the best solver for the particular problem. Flow-rate and cumulative-volume balances from each type of inflow and outflow are computed for each time step.

In order to use MODFLOW, initial conditions, hydraulic properties, and stresses must be specified for every model cell in the finite-difference grid. For entering and editing input data, a pre-processor program called MFI2K is used. Primary output is head, which can be written to the listing file or into a separate file. Other output includes the complete listing of all input data, drawdown, and budget data. Budget data are printed as a summary in the listing file, and detailed budget data for all model cells can be written into a separate file.

The packages used in this study are tabulated in Table 3.1.

Table 3.1: List of Packages Used in the Present Application

Package Name	**Abbreviation**	**Package Description**
Basic	BAS	Handles those tasks that are part of the model as a whole. Among those tasks are specification of boundaries, determination of time-step length, establishment of initial conditions, and printing of results.
Block-Centered Flow	BCF	Calculates terms of finite difference equations which represent flow in porous medium; specifically, flow from cell to cell and flow into storage.
Drain	DRN	Adds terms representing flow to drains to finite difference equations.
Preconditioned Conjugate Gradient	PCG	Iteratively solves the system of finite difference equations using preconditioned conjugate gradient.

CHAPTER 4

PRESENTATION OF THE RESULTS

4.1 Introduction

Low frequency sea level-fluctuations such as the tide have important effects on sediment transport processes.

The coastal groundwater table changes with the tide. During the period of flood tide, the beach water table is lower than the sea level. Therefore, waves running up above the exit point of the groundwater table infiltrate into the aquifer, depositing the sediment carried from offshore on the beach face. Additionally, since the velocity and volume of the backwash (wave run down) reduce due to infiltration, less sediment is transported back to the sea by the backwash. These effects promote beach accretion. Conversely, during the ebb tide, the exit point of the groundwater table is higher than the sea level, and exfiltration from the aquifer into the sea occurs, opposite of the effects during the period of the flood tide are observed, and those opposite effects result in enhanced beach erosion.

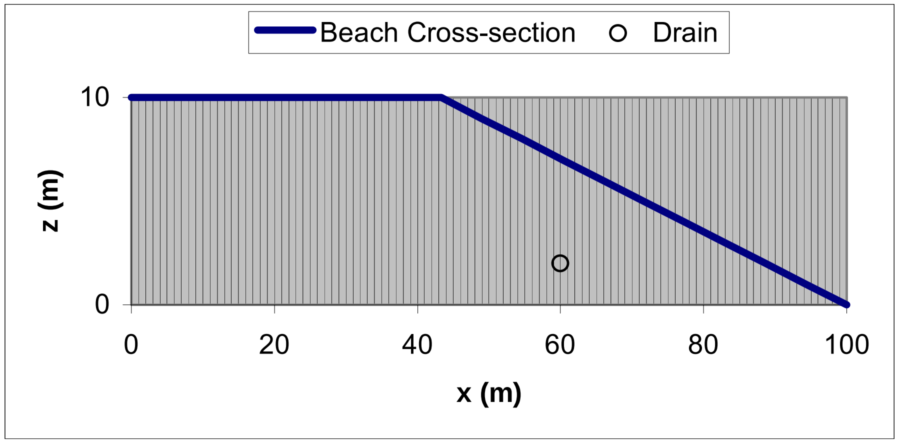
The correlation observed between beach erosion/accretion and the relative position of the coastal groundwater table has led to beach dewatering projects. In this chapter, a numerical modelling approach to the problem is adopted. To be able to observe the tidal effects significantly, a beach having a large scale of tidal range, i.e. a macrotidal beach, is selected. The fluctuation of the groundwater table under drainage will be the focus of this study, since the position of the coastal water table relative to the mean sea level is the most important parameter influencing water infiltration/exfiltration on the beach and, hence, the erosive or accretive trends on the beach face.

The intention here is not to provide a specific engineering design criteria for beach dewatering projects, rather, is to explore the behavior of groundwater flow in response to beach dewatering and its interaction with the sea through a numerical model that could be used in the design activities.

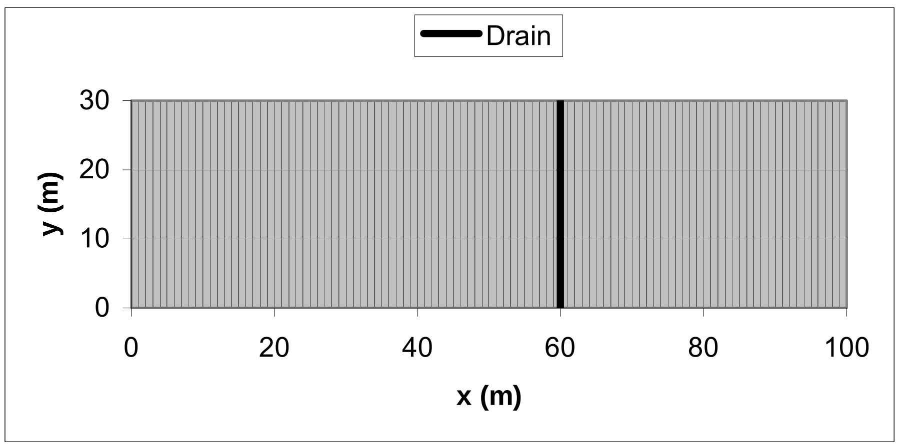
4.2 Configuration of the Simulation Domain

The model simulations were conducted in the domain shown in Fig. 4.1(a) and Fig. 4.1(b). Within this domain, 1 layer with a thickness of 10 m; 100 columns with a cell width of 1m; and 3 rows with a cell width of 10 m are defined in the MODFLOW data-input program.

The drain is located at x = 60 m and z = 2 m. The horizontal location of the drain corresponds to the horizontal position of the intersection between the highest tide sea level and the beach face (In this domain, the highest tide sea level is 7 m, corresponding to x = 60.3 m). From an engineering point of view, the drainage should be located landward of the intertidal zone for the purpose of construction and maintenance. The hydraulic conductance between the aquifer and the drain is 0.00486 m^2/s and the beach inclination angle is 10°.



(a)



(b)

Figure 4.1: The Simulation Domain: (a) Cross-sectional View;

(b) Plan View

The landward boundary is prescribed by a constant head of 10 m, i.e., H_1=10 m; while the seaward boundary conditions change according to a diurnal tide specified in Eq. (3.4d), i.e,

$$H_2(t) = H + A\cos(wt + \frac{\pi}{2})$$

Where, H = 4 m; A = 3 m; $\omega = 2\pi/24$ Rad/hr. (Fig. 4.2)

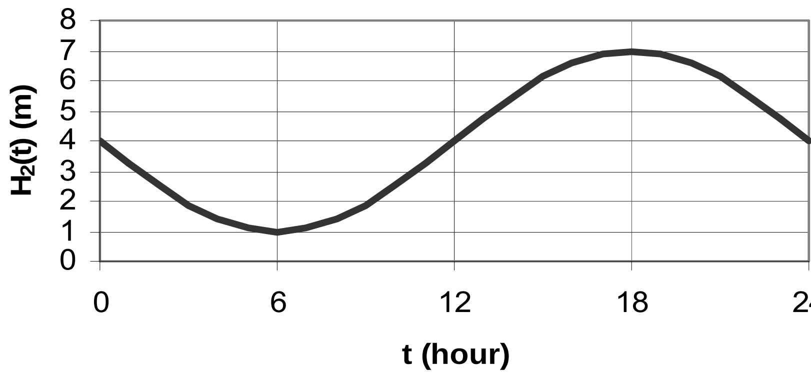


Figure 4.2: Specified Seaward Boundary Condition

The simulations were performed for a complete tidal cycle (24 hours) that includes 24 stress periods, thus, each of which has a length of 3600 seconds and composed of 60 time steps, where the time step is 60 seconds for all the simulations.

4.3 Water Table Lowering Due to Beach Dewatering

The variation of water table elevation under beach drainage over one complete tidal cycle is shown in Fig.4.3. With the aim of comparison, the simulations were also conducted using the same configuration with no drainage case and the computed water tables were plotted in Fig. 4.3.The results clearly demonstrate the significant lowering of the water table and the formation of the cone of depression above the drain.

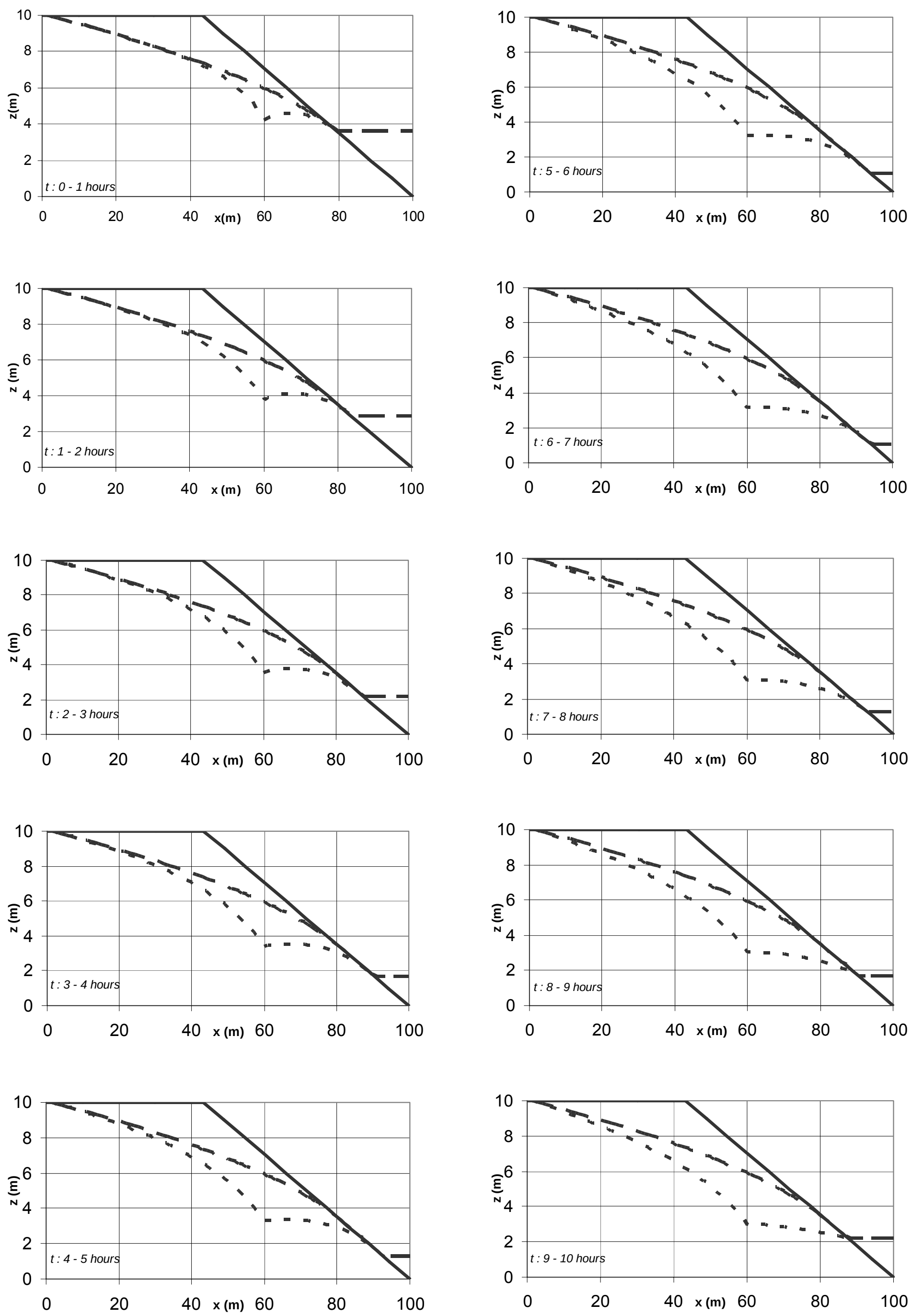


Figure 4.3: Hourly Variation of Groundwater Table under Drainage and without Drainage (Thick Solid Line (—) is the Beach Cross-section; Dots (···) are the Water Table under Drainage; Dashes (---) are the Water Table without Drainage)

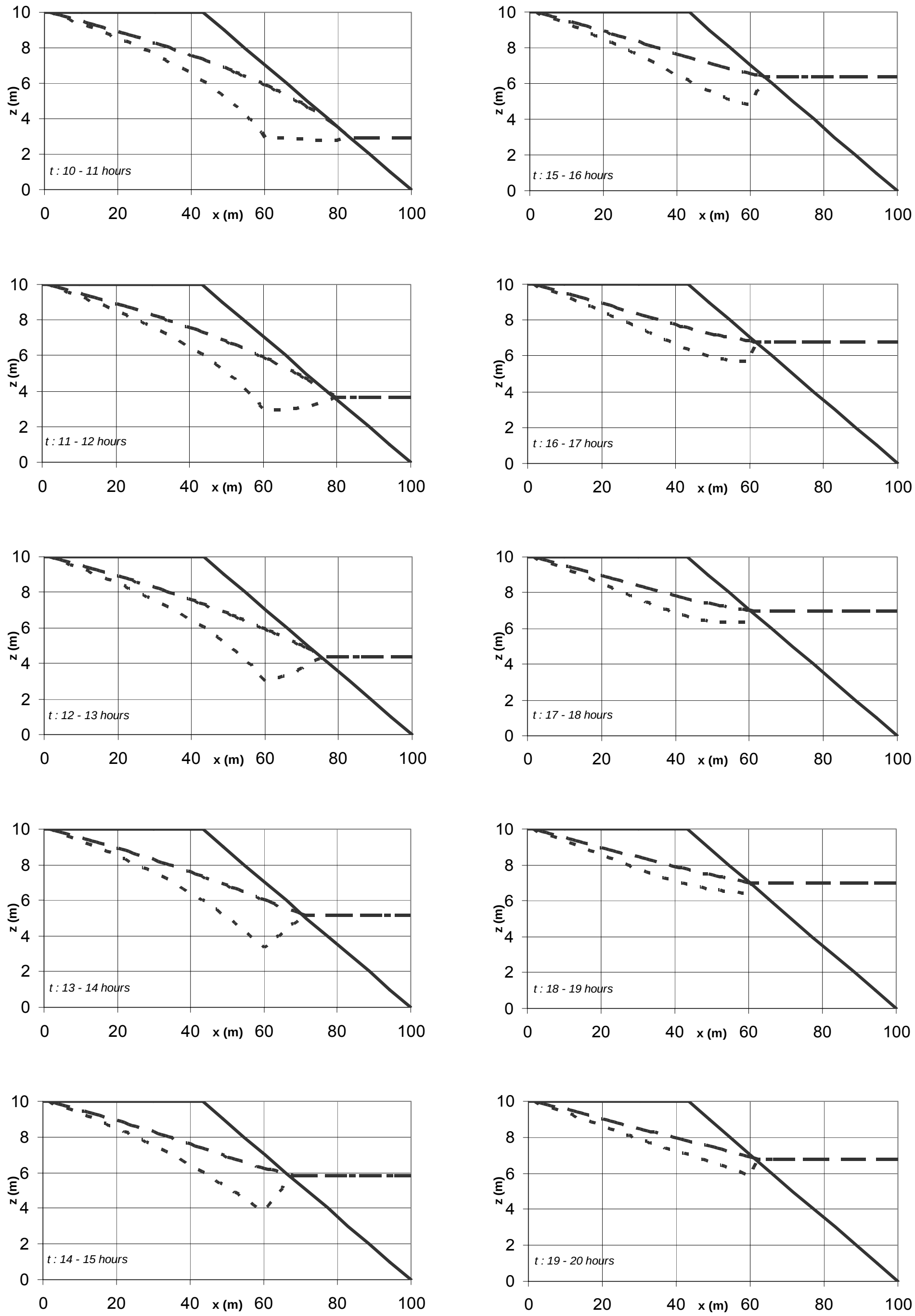


Figure 4.3: (continued)

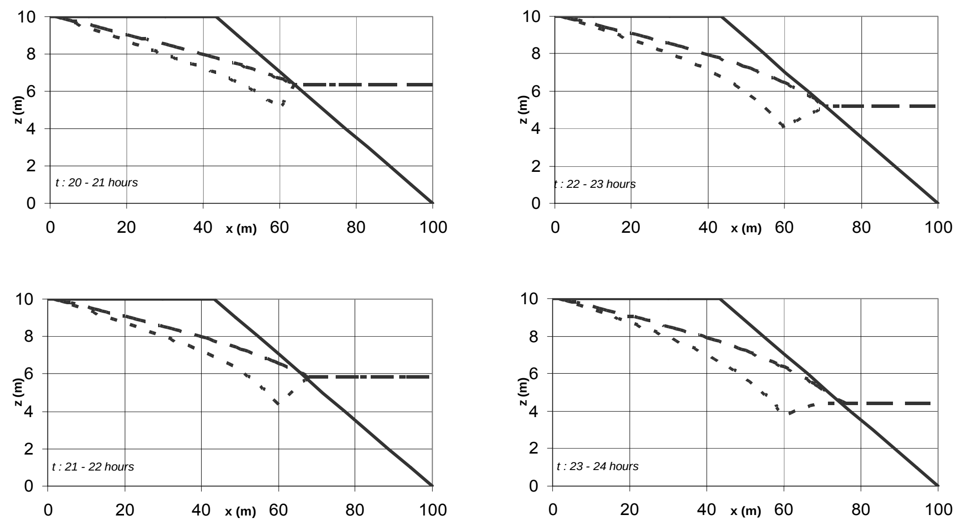


Figure 4.3: (continued)

4.4 Seepage Face Reduction

To examine the beach dewatering effects, seepage face changes are observed. The seepage face changes over the tidal cycle both under drainage and without drainage cases are shown in Fig.4.4. The results obtained show that the seepage face is reduced significantly due to drainage. Such reduction is the main aim of beach drainage projects to enhance water infiltration and sand accretion on the beach.

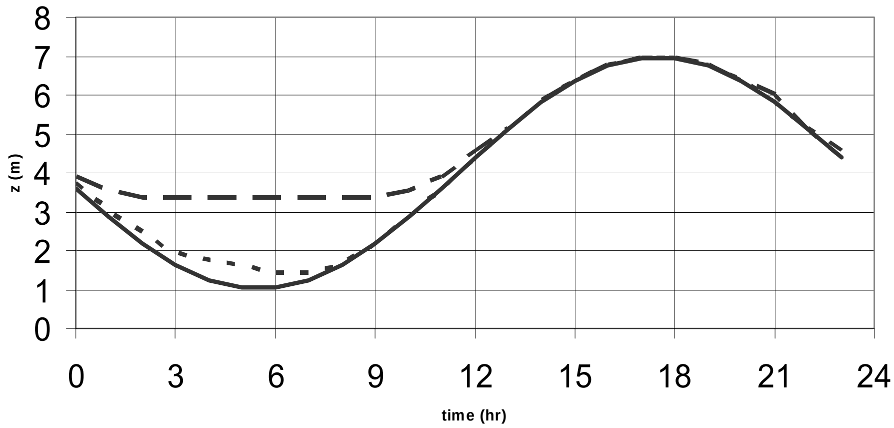


Figure 4.4: Seepage Face Reduction due to Drainage (Thick Solid Line (—) is the Elevation of Tidally Fluctuating Sea Level; Dots (···) are the Elevation of the Exit Point under Drainage; Dashes (---) are the Elevation of the Exit Point without Drainage)

4.5 Drainage Rate

Water drainage is due to the potential gradient caused by the lower pressure inside the drain compared with the surrounding pressure. As the water table shifts downward or upward by the tidal and drainage effects, the surrounding pressure changes and so does the potential gradient. Thus, the drainage rate will vary with time. (Li et al 1996b)

The drainage rates over the entire tidal cycle calculated within the numerical simulation are plotted in Fig. 4.5. This information regarding the time-varying drainage rate can be used for engineering design of beach drainage systems, i.e. in adjusting the pumping rate and estimating the operational cost since it is directly related to the energy input.

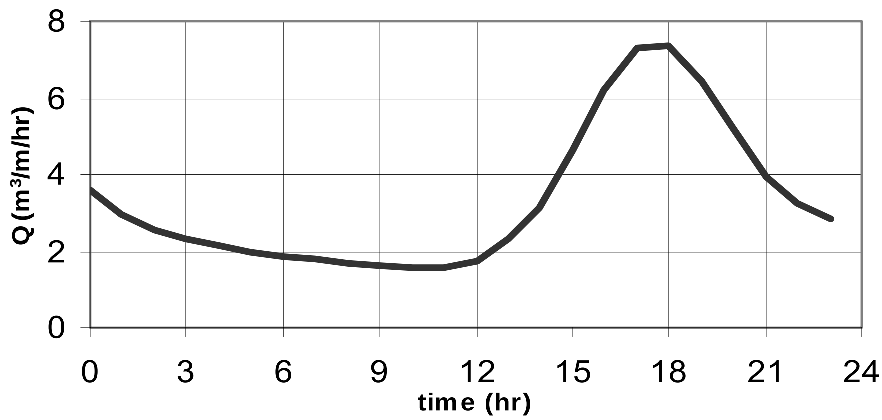


Figure 4.5: Drainage Rate Variation over the Tidal Cycle

4.6 Effects of Horizontal Location of Drain

It was noted previously that, for practical purposes, the drain should be located landward from the intersection point of the maximum sea level and beach face profile. The simulation results described previously were obtained with the drain located beneath the highest tide level, i.e. for L_d = 60 m. It is of interest to examine the effects of the horizontal drain location in lowering the beach water table. Therefore two more simulations were carried out with drains located at (55 m, 2 m) and (50 m, 2 m) respectively.

The results were compared with those obtained from the previous simulation [with the drain located at (60 m, 2 m)]. The groundwater table profiles at every 6 hours are sketched in Fig. 4.6. The seepage faces and the drainage rates plotted are shown in Fig.4.7 and Fig.4.8 respectively.

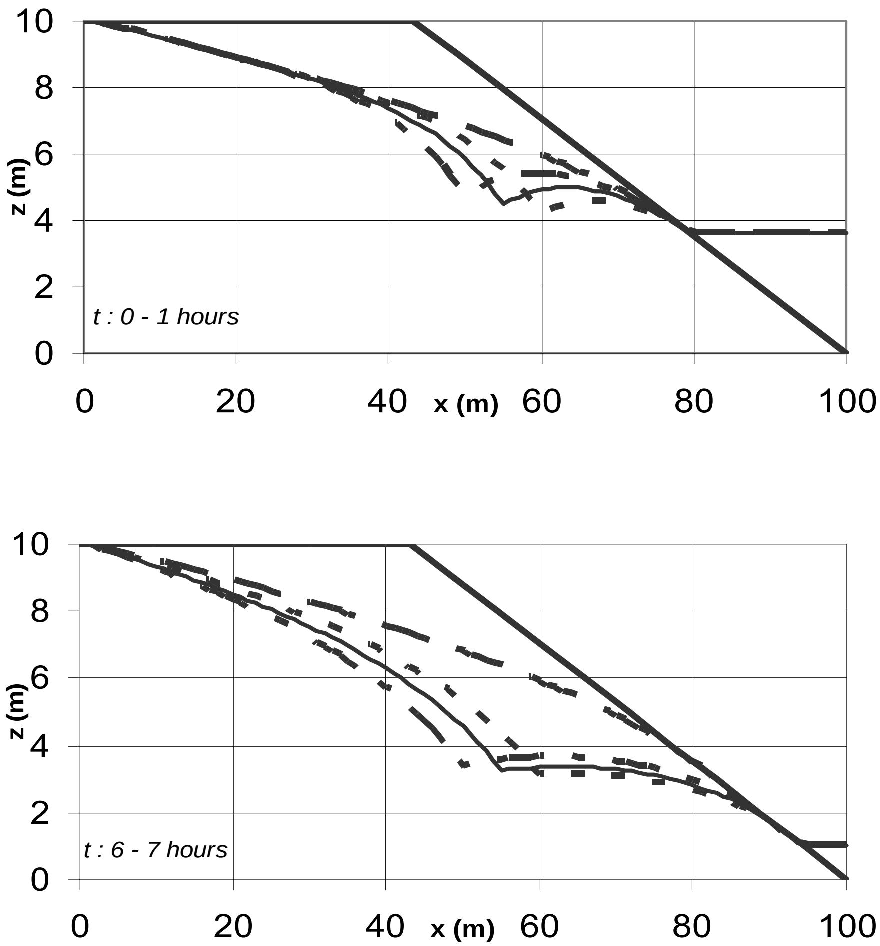
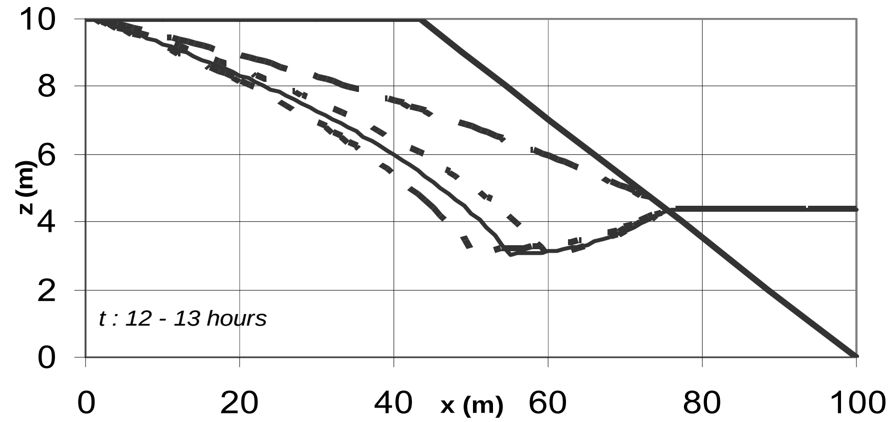


Figure 4.6: Variation of Groundwater Table under Drainage with Different Horizontal Locations of the Drain for Every Six Hours (Dots (···) are for L_d = 60 m; Solid Line (—) is for L_d = 55 m; Dash-Dots (−·−) are for L_d = 50 m; Dashes (---) are for the profile without drainage; Thick Solid Line (**—**) is the Beach Cross-Section)



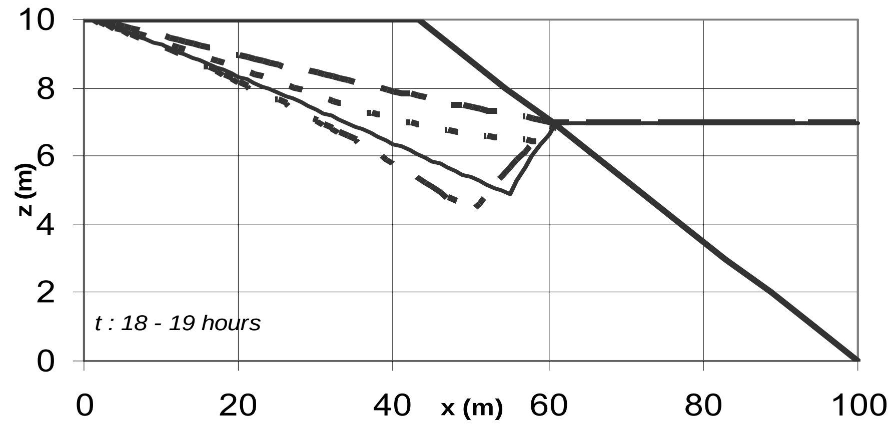


Figure 4.6: (continued)

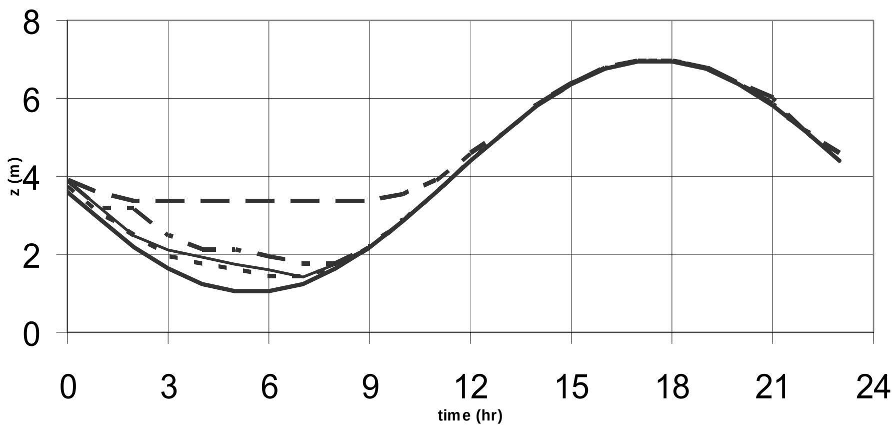


Figure 4.7: Seepage Face Reduction due to Drainage with Different Horizontal Locations of the Drain (Thick Solid Line (—) is the Elevation of Tidally Fluctuating Sea Level; Dashes (---) are the Elevation of the Exit Point without Drainage; Dots (⋯) are the Elevation of the Exit Point under Drainage for L_d = 60 m, Solid Line (—) is for L_d = 55 m, Dash-Dots (−·−) are for L_d = 50 m)

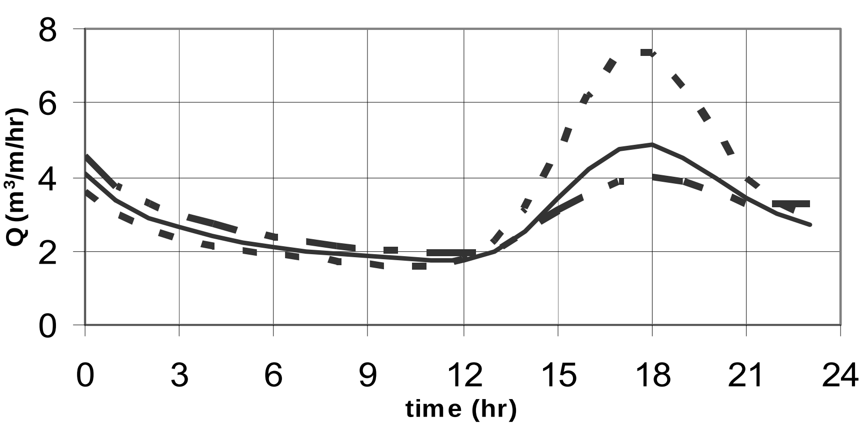


Figure 4.8: Drainage Rate Variation over the Tidal Cycle with Different Horizontal Locations of the Drain. (Dots (⋯) are for L_d = 60 m; Solid Line (—) is for L_d = 55 m; Dash-Dots (−·−) are for L_d = 50 m)

To compare the results in a quantitative way, a dimensionless parameter called the efficiency index, ID_e is introduced. ID_e is defined by:

$$ID_e = \frac{K\overline{(R_{sp})^2} / L_a}{2\overline{Q}} \qquad \text{(Li et al., 1996b)} \tag{4.1}$$

Where, $\overline{Q}$ is the mean drainage rate; and $\overline{(R_{sp})^2}$ is the mean square of seepage face reduction averaged over a tidal cycle, i.e. ,

$$\overline{(R_{sp})^2} = \overline{(z_{EO} - z_{Ed})^2} \tag{4.2}$$

Where, z_{EO} is the elevation of exit point without drainage, z_{Ed} is the elevation of exit point with drainage, and L_a is the length scale over which tidal effects are manifested inland (Nielsen, 1990), i.e.,

$$L_a = \sqrt{\frac{K(H_1 + H)}{n_e \omega}} \tag{4.3}$$

Where, n_e is the effective porosity of porous medium.

Since the optimal situation of the beach dewatering system occurs when the seepage face reduction is maximized by a minimum drainage, by replacing these terms to the nominator and the denominator respectively as stated in this dimensionless parameter, ID_e can be used as an efficiency index. After calculating the ID_e for each case, the results have shown that the efficiency index increased with L_d. (Fig. 4.9)

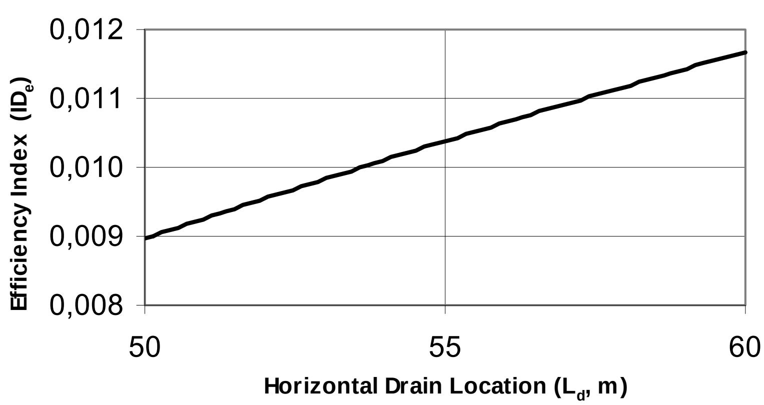


Figure 4.9: Efficiency Index as a Function of L_d

4.7 Effects of Vertical Location of Drain

Two more simulations were conducted with the drains located at (60m, 1m) and (60m, 3m) to investigate the effects of vertical location of the drain on the groundwater table behavior under Beach Dewatering. The groundwater profiles at every 6 hours are shown in Fig.4.10.

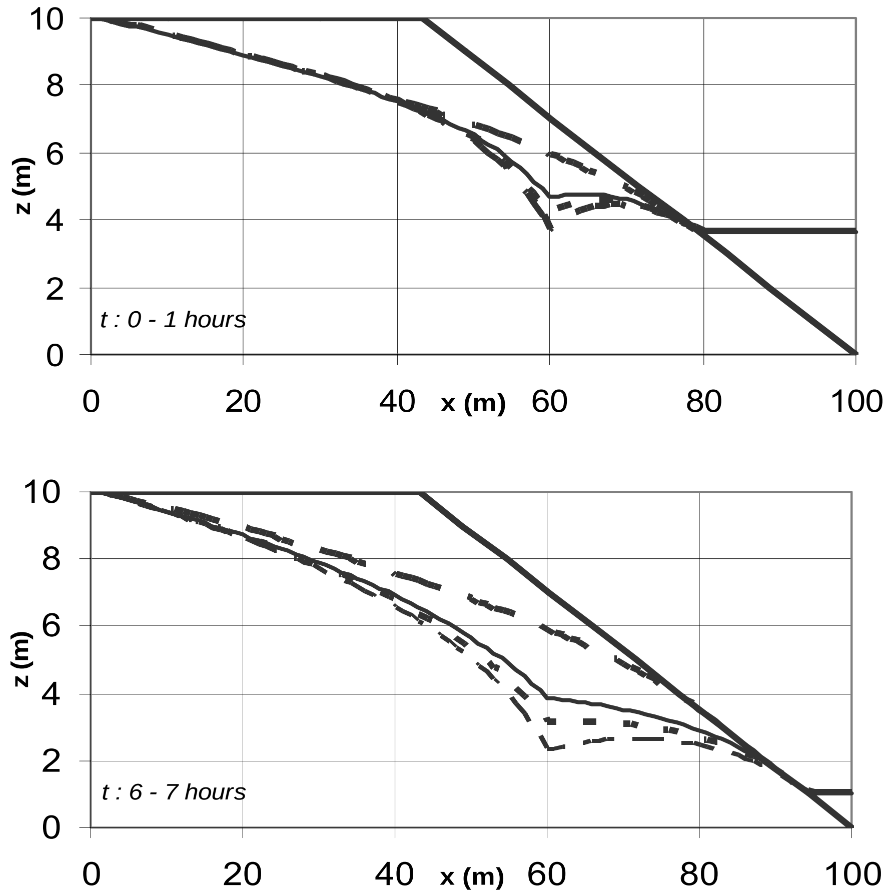
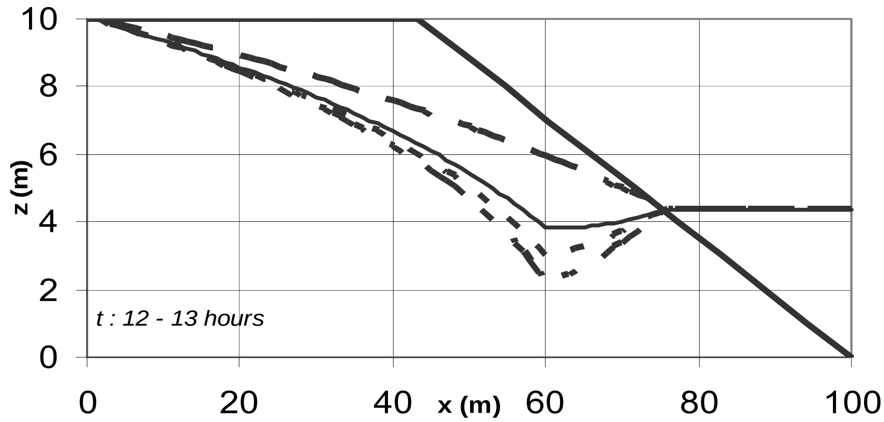


Figure 4.10: Variation of Groundwater Table under Drainage with Different Vertical Locations of the Drain for Every Six Hours (Solid Line (—) is for z_d = 3 m; Dots (···) are for z_d = 2 m; Dash-Dots (−·−) are for z_d = 1 m; Dashes (---) are for the profile without drainage; Thick Solid Line (—) is the Beach Cross-Section)



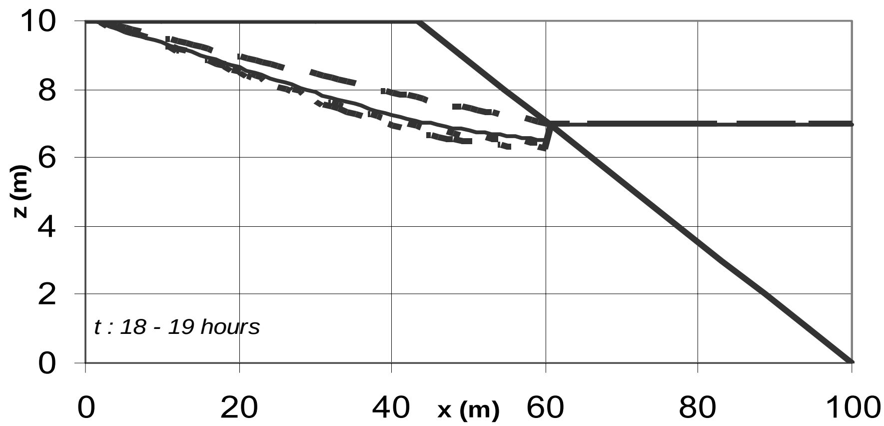


Figure 4.10: (continued)

The predictions of seepage face reduction and drainage rates are shown in Fig. 4.11 and Fig. 4.12 respectively. Although the seepage face reduction increases by the decreasing z_d, it is noted that the efficiency index increased slightly with the vertical elevation of the drain. (Fig. 4.13)

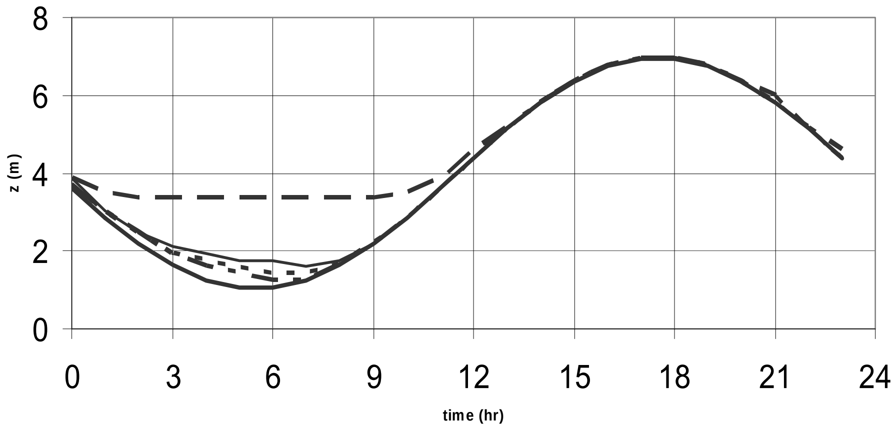


Figure 4.11: Seepage Face Reduction due to Drainage with Different Vertical Locations of the Drain. (Thick Solid Line (—) is the Elevation of Tidally Fluctuating Sea Level; Dashes (---) are the Elevation of the Exit Point without Drainage; Solid Line (—) is the Elevation of the Exit Point under Drainage for z_d = 3 m; Dots (···) are for z_d = 2 m; Dash-Dots (−·−) are for z_d = 1 m)

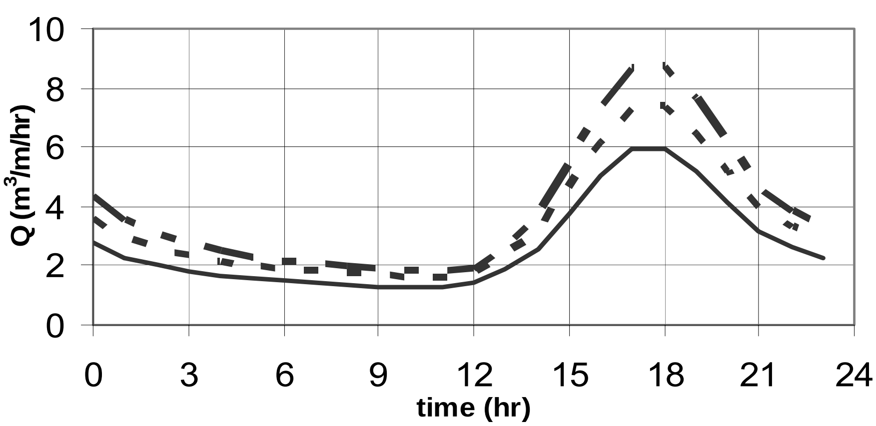


Figure 4.12: Drainage Rate Variation over the Tidal Cycle with Different Vertical Locations of the Drain. (Solid Line (—) is for z_d = 3 m; Dots (···) are for z_d = 2 m; Dash-Dots (−·−) are for z_d = 1 m)

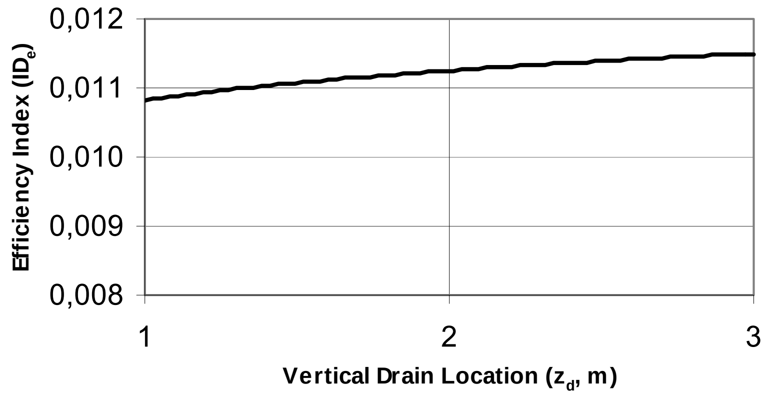


Figure 4.13: Efficiency Index as a Function of z_d

4.8 Drain Operation Period

During the flood tide, when the sea level is higher than its mean value, the water table exit point couples with the sea, therefore no seepage face exists at the beach. This phenomenon can be observed in Fig. 4.3 and in Fig. 4.4 after the 12th hour. Since the reduction of the seepage face existing on the beach is the main aim of beach dewatering projects in macro-tidal coastlines, operating the system during the flood period becomes unnecessary. Hence, the drainage can be stopped within that period to lower the operational cost of the system, which directly increases the system efficiency.

The efficiency indexes are calculated for the first and the second parts of the tidal cycle and are compared with the diurnal index when the drain is at x = 60 m and z = 2 m. The results shown in Table 4.1 clearly states that the system performs much more effectively in the first twelve hours of the day (i.e., when the water table exit point decouples with the sea level during the ebb tide) and the unnecessary drainage during the second part of the day reflects as a large amount of decrease in the total diurnal efficiency of the beach drainage system.

Table 4.1: Efficiency Indexes calculated for different parts of the tidal cycle

	Efficiency Index
For 0 – 12 hours	0.03501
For 12 – 24 hours	0.00009
For 0 – 24 hours	0.01129

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

Sand conservation is a critical matter on many leisure and resort beaches in the world, particularly the maintenance of sand inshore during high-energy conditions. The task of protecting beaches has motivated engineering research on various beach protection techniques. Beach Drainage is a relatively recent development.

The aim of the Beach Drainage system is to stabilise the beach by a reduction of the sediment transport during the wave run down.

With the beach drain system, the water table in the wave runup zone is lowered. This causes an increased infiltration through the foreshore during wave run up, and results in beach sand deposition. Furthermore, in macro-tidal environments, the exit point during ebb tide is lowered (i.e. the seepage face is reduced) which will cause lower flux through the beach face, resulting in lower transport rates and stabilization of the slope.

The position of the groundwater table is an important factor in cross-shore sediment transport on a beach. From theoretical, empirical and field studies that have been discussed briefly in Chapter 2, it is demonstrated that a high groundwater table relative to the mean sea level tends to enhance offshore sediment transport and hence beach erosion by the intensified water exfiltration from the aquifer into the sea which increases the strength of the backwash, while a relatively low ground water table promotes onshore sediment transport and beach accretion by water infiltration into the aquifer which reduces the backwash and results in increased sand deposition on the beach.

The beach groundwater table varies with the tide and therefore, such variations affect significantly the beach stability.

In this study, a macro-tidal beach is selected as an environment to observe the tidal fluctuations in a large scale. Since the position of the beach water table relative to the mean sea level is the most important parameter influencing the beach stability, the variation of the water table elevation under drainage has been the focus of this study.

In the present work, with the aim of comparison, the beach water table both under drainage and with no drainage cases are simulated by the numerical model and the simulation results clearly showed that the water table being lowered, therefore the seepage face reduced due to drainage which is the main aim of beach drainage projects.

As additional information to the variation of water table elevation, the drainage rate varying with time, which can be used for design purposes, is calculated.

The numerical model is also applied to investigate the effects of some design factors, e.g., the horizontal and vertical drain location. To be able to make a numerical comparison between the alternatives, an efficiency index is introduced which takes into account mainly the seepage face reduction and the drainage rate. The efficiency index is constituted with the target of maximum reduction of the seepage face by minimum drainage. The results obtained showed that the efficiency of the system decreased with the horizontal distance of the drain from the beach. The system efficiency is observed to be increased with the vertical elevation of the drain.

The final discussion is about “when to operate the drain”. The profiles showed that during the flood tide, the exit point of the ground water table couples with the sea and no significant seepage face occurs. Therefore, it is concluded that the drain does not need to be operated continuously for 24 hours, instead, performing the drainage during the low tide and stopping it when the sea level is higher than its mean value, can increase the effectiveness of the system, since the drainage is directly related to the energy input which is the most important parameter in the economical considerations.

From the results of this work, a set of recommendations may be identified for future studies.

The results obtained from the numerical model used in this study implicates the positive impacts of beach drainage on the stability of a beach in a macrotidal environment, where the low-frequency waves, i.e. tide, dominate the seaward boundary of the model. To examine the effects of beach drainage on a microtidal beach, the infiltration across the beach face is more important than the seepage face reduction, since the sea level oscillations are not significant and no decoupling (no seepage face) occurs between the exit point of the water table and the sea. In that case, high-frequency waves such as swell waves and their interaction with the aquifer should be investigated.

In other words, the main aim of the Beach Dewatering projects on a macrotidal coastline is to stabilize the beach by removing the erosive effects of seepage resulting from the decoupling of the low sea level and the exit point of the ground water table; where, on a microtidal beach, the main aim of the system is to provide stabilization with the accretion created by the water infiltrating into the aquifer and leaving the material transported from offshore on the beach face.

In both cases, for the purpose of predicting the final beach profile change after performing the Beach Drainage for a specific period, this study can be expanded by investigating the sediment transport mechanisms under the influence of infiltration/exfiltration.

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