Available online at www.sciencedirect.com
SCIENCE DIRECT®

ELSEVIER

Coastal Engineering 49 (2003) 125–153

Coastal Engineering
An International Journal for Coastal, Harbour and Offshore Engineers

www.elsevier.com/locate/coastaleng

Wave field modification by bathymetric anomalies and resulting shoreline changes: a review with recent results

Christopher J. Bender*, Robert G. Dean

Taylor Engineering, 9000 Cypress Green Drive, Jacksonville, FL 32256, USA

Received 4 July 2002; received in revised form 21 May 2003; accepted 20 June 2003

Abstract

This paper provides a review of available studies on wave transformation by bathymetric changes and the resulting shoreline impacts. Three case studies of beach nourishment projects with significant nearshore borrow areas are examined: Grand Isle, Louisiana constructed in 1984, Anna Maria Key, Florida in 1993, and Martin County, Florida in 1996. A review is presented of field and laboratory scale studies that have examined the impact of offshore pits on the local wave field and sediment dynamics. Solutions for wave transformation by changes in bathymetry are outlined primarily in chronological order following the development from analytical solutions for long waves in one horizontal dimension (1-D) through numerical models for arbitrary bathymetry that include many wave-related nearshore processes. Modeling of shoreline responses due to wave field modification owing to changes in offshore bathymetry is examined with models that include both wave field and shoreline changes and by coupling models that evaluate these processes independently. The wave transformation processes included in nearshore models are important factors in the capability to predict a salient leeward of a pit; the shoreline responses observed in the limited laboratory experiments and at Grand Isle, LA.

Keywords: Wave transformation; Borrow pit; Beach nourishment; Bathymetric anomaly; Dredging

1. Introduction

Modifications of offshore bathymetry by removal of large quantities of sediment alter the local wave field, which in turn modifies the equilibrium planform of the leeward beach. These effects as well as the impact on sediment dynamics near the sediment removal area have become of concern as the extraction of offshore sediment for beach nourishment, construction materials, and other purposes has increased. Unexpected shoreline planform changes in and adjacent to completed beach nourishment projects have been attributed to offshore borrow pits. Thus, a better understanding of the effects of bathymetric changes on the wave field and the resulting impacts on shorelines would be beneficial to more appropriate utilization of offshore sand resources.

Several studies encompassing field and laboratory scales have been conducted to investigate this issue. These studies have examined wave transformation over a bathymetric anomaly with the shoreline changes caused by the altered wave field. Initially, dating back to the early 1900s, the focus was on the modification of a wave train encountering a change in bathymetry.

* Corresponding author. Fax: +1-904-731-9847.
E-mail address: cbender@taylorengineering.com (C.J. Bender).

0378-3839/03/$ - see front matter © 2003 Elsevier B.V. All rights reserved.
doi:10.1016/S0378-3839(03)00061-9

This early research included development of analytical solutions for bathymetric changes in the form of a step, or a pit, first of infinite width (in one horizontal dimension; 1-D models), and, more recently, of finite dimensions (in two horizontal dimensions; 2-D models). The complexity of the 2-D models has advanced from a pit/shoal with vertical sidewalls and uniform depth surrounded by water of uniform depth to domains with arbitrary bathymetry. Many wave transformation models lack the capability to represent wave reflection and damping, which are both apparent significant processes in related shoreline modifications. Some models combine the calculation of wave transformation and resulting shoreline change, whereas others perform wave calculations separately and rely on a different program for shoreline evolution.

This report presents a review of studies relating to wave transformation by bathymetric anomalies and the resulting shoreline changes. Although many important and relevant papers from the fields of coastal and ocean engineering are discussed, this paper is not intended to present an exhaustive review of related work from either discipline.

2. Motivation

Changes in offshore bathymetry modify the local wave field, thus causing an equilibrium planform that may be altered significantly from the previous, relatively straight shoreline. In addition to wave transformation effects, a bathymetric change may alter the sediment transport dynamics by drawing sediment into it from the nearshore, or by intercepting the onshore movement of sediment. Knowledge of wave field modifications and the resulting effects on sediment transport and shoreline evolution is essential in the design of beach nourishment projects and other engineering activities that modify offshore bathymetry.

Beach nourishment has become the preferred technique to address shoreline erosion in some areas. In most beach nourishment projects, the fill placed on the eroded beach is obtained from borrow areas located offshore of the nourishment site. The removal of large quantities of fill needed for most projects can result in substantial changes to the offshore bathymetry through the creation of borrow pits, or by modifying existing shoals. The effect of the modified bathymetry in the borrow area on the wave field and the influence of the modified wave field on the shoreline can depend on the incident wave conditions, the native and nourishment sediment characteristics, and some features of the borrow area including the location, size, shape, and orientation.

The large quantities of sediment used in beach nourishment projects combined with the increase in the number of projects constructed, and an increased industrial need for quality sediment have, in many areas, led to a shortage of quality offshore material located relatively near the shore. In the United States, this shortage has increased interest in the mining of sediment deposits located in Federal waters, located a significant distance offshore and which fall under the jurisdiction of the Minerals Management Service (MMS). Questions have been raised by the MMS regarding the potential effects on the shoreline of removing large quantities of sediment from borrow pits lying in Federal waters (Minerals Management Service, 2002).

A better understanding of the effects of altering the offshore bathymetry is currently needed. There are four wave transformation processes that can occur as a wave train encounters a change in the bathymetry: refraction, diffraction, reflection, and dissipation. The first three of these processes are referred to collectively as “scattering”. These transformation processes modify the wave field in a complex manner depending on the local conditions. A more complete understanding and predictive capability of the effect of bathymetric changes to the wave field and the resulting shoreline modification leading to less impactive design of dredge pit geometries should be the goal of current research.

3. Case studies and experiments

Several methods have been employed to quantify the impact on the shoreline caused by changes in the offshore bathymetry including case studies, field experiments, analytical developments, numerical models, and laboratory studies. The intriguing behavior of the shoreline following beach nourishment projects at Grand Isle, Louisiana, Anna Maria Key, Florida, and Martin County, Florida has led to questions and investigations regarding the impact of the substantial offshore borrow areas present in each case. Field studies have been conducted to investigate the impact of offshore dredging in relatively deeper water to attempt to define a depth at which bathymetric changes will not induce significant wave transformation. Laboratory experiments have documented wave transformations caused by bathymetric changes and the resulting effects on the shoreline in controlled settings possible only in the laboratory.

3.1. Case studies

3.1.1. Grand Isle, Louisiana (1984)

The beach nourishment project at Grand Isle, LA provides one of the most interesting and well publicized examples of an irregular planform resulting from the effects of a large borrow area lying directly offshore. One year after the nourishment project was completed, two large salients, flanked by areas of increased erosion, developed immediately shoreward of the offshore borrow area. Combe and Soileau (1987) provide a detailed account of the shoreline maintenance history at Grand Isle, specifications of the beach nourishment project that was completed in 1984, and details of the shoreline evolution in the 2 years following completion.

The project required 2.1×10^6 m^3 of sediment with approximately twice this amount dredged from a borrow area lying 800 m from shore (Combe and Soileau, 1987) in 4.6 m of water (Gravens and Rosati, 1994). According to Combe and Soileau (1987), the dredging resulted in a borrow pit that was "dumbbell" shaped in planform with two outer lobes dredged to a depth of 6.1 m below the bed connected by a channel of approximately 1370 m in length dredged to 3.1 m below the bed. A plan view of the bathymetry prior to 1992, after significant infilling of the pits, could not be obtained.

The salients seen in Fig. 1 started to form during storm events that occurred during the winter and spring of 1984–1985. By August 1985, the salients and associated areas of increased erosion were prominent features on the shoreline. An aerial survey of the area conducted by the New Orleans District of the Army Corps of Engineers and the Coastal Engineering Research Center concluded that the size and the location of the borrow area were such that its presence could affect the local wave climate. Oblique aerial photography identified the diffraction of the wave field as a result of the borrow area. The area of increased erosion near the salients was found to "affect 25% of the project length and amounted to about 8% of the net project volume" (Combe and Soileau, 1987).

Three major hurricanes impacted the project area in the hurricane season following the project completion; an unprecedented number for the Louisiana coastline in the same season (Combe and Soileau, 1987). While

Fig. 1. Aerial photograph showing salients shoreward of borrow area looking East to West along Grand Isle, LA in August, 1985 (Combe and Soileau, 1987).

Fig. 2. Aerial photograph showing salients shoreward of borrow area along Grand Isle, LA in 1998 (modified from Louisiana Oil Spill Coordinator's Office (LOSCO), 1999).

these storms did considerable damage to the newly formed berm and caused large sediment losses, the location and the size of the salients were relatively unaffected. The salients have remained on the Grand Isle shoreline as shown by an aerial photograph from 1998 (Fig. 2). It appears that the eastern salient has decreased in size while the western salient has remained the same size or even become larger. A series of detached offshore breakwaters was constructed along the eastern part of Grand Isle in the 1990s, which terminate at the eastern salient and may have affected its shape.

Bathymetric surveys taken through the borrow area in February 1985 and August 1986 revealed that the outer lobes had filled to about half their original depth and the channel connecting the lobes had reached the sea bed elevation (Combe and Soileau, 1987). Currently, the borrow area is reported to be completely filled by very fine (silt and clay) material (Combe, 2002) which would have required the same approximate volume of sediment that was dredged for the initial placement. Although the origin of the sediment that has refilled the borrow pit is unknown, it is reported to be finer than the sediment dredged for the nourishment project, indicating that the material did not originate from the project. While no longer a bathymetric anomaly, the borrow areas are reported to continue to modify the wave field as local shrimpers use the waters shoreward of the pit as a harbor to weather storms (Combe, 2002). The reason for the sheltering effect of the filled pit may be due to wave damping and energy dissipation caused by fluid–mud interaction with the extremely fine material that has filled the pit.

3.1.2. Anna Maria Key, Florida (1993)

The 1993 beach nourishment project at Anna Maria Key, Florida is another example of a project with a large borrow area lying offshore in relatively shallow water. The project comprised the placement of 1.6×10^6 m^3 of sediment along a 6.8-km segment (DNR Monuments R-12 to R-35[1]) of the 11.6 km long barrier island (Dean et al., 1999). The borrow area for the project was approximately 3050 m long and ranged from 490 to 790 m offshore in approximately 6 m of water. A planview of the bathymetry near the

[1] The "DNR Monuments" are permanent markers spaced at approximately 300 m along the Florida sandy beaches for surveying purposes.

project including the borrow area, which highlights the longshore extent of the pit and its proximity to shore, is shown in Fig. 3. A transect through the borrow area, indicated in the previous figure at Monument R-26, is shown in Fig. 4 and shows dredging to a depth of 3.1 m below the local seabed. This figure shows one pre-project transect, a transect immediately following completion, and two post-nourishment transects. The post-nourishment transects indicate minimal infilling of the borrow pit. The closure depth for this location is 4.1 m (Dean, 2002), which is shoreward of the borrow pit. The gulfward movement of the nourished profile beyond the closure depth after project completion is interpreted as a result of the reconfiguration of the beach nourishment material due to gravity acting downslope, resulting in sand transport to locations greater than the closure depth.

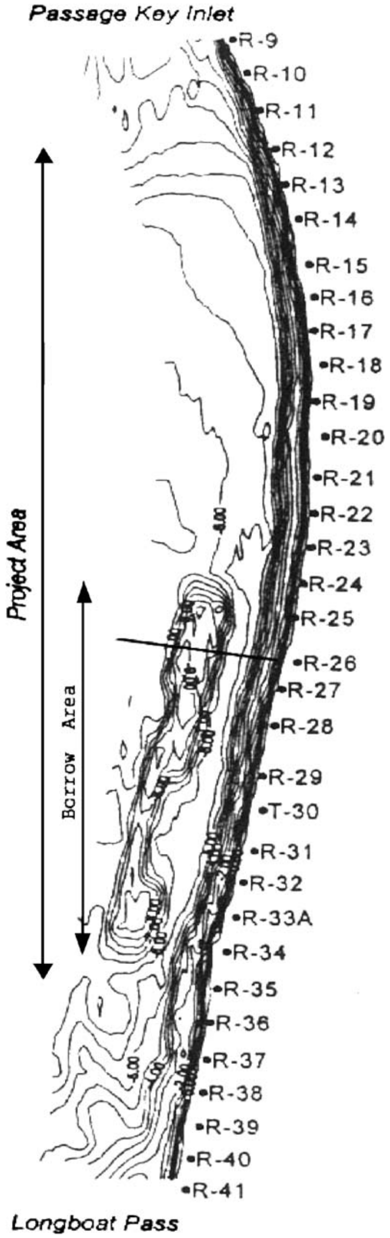

Fig. 3. Bathymetry off Anna Maria Key, FL showing location of borrow pit following beach nourishment project (modified from Dean et al., 1999).

The shoreline planform was found to experience the greatest recession shoreward of the borrow area. Fig. 5 shows the shoreline position relative to the August, 1993 data for seven different periods. A large area of negative shoreline change indicating erosion is found from DNR Monument numbers 25–34 for the July, 1997 and February, 1998 data. This area lies directly shoreward of the borrow area shown in Fig. 3. The behavior of the shoreline leeward of the borrow area is seen to be the opposite of the Grand Isle, LA response where shoreline advancement occurred.

Volume changes determined from profiles in the project area did not show large negative values near the southern end of the project. The difference between the shoreline and volume changes at the southern end of the project implies that the constructed profiles may have been steeper near the southern end of the project as compared to those near the northern end (Wang and Dean, 2001).

The proximity of the borrow area to the shoreline is one possible contribution to the local erosion. Although the reason for the increased shoreline recession in this area is not clear, it is interesting that the anomalous shoreline recession did not occur until the passage of Hurricanes Erin and Opal in August and October 1995, respectively. Hurricane Opal was a category 4 hurricane with sustained winds of 67 m/s when it passed 600 km west of Anna Maria Key (Liotta, 1999). A reported storm surge of 0.3–1.0 m combined with the increased wind and wave action resulted in overtopping of the beach berm, flooding of the back area of the project, and transport of sediment to the back beach or offshore. The average shoreline retreat for the project area was approximately 9.1–15.2 m based on observations (Liotta, 1999).

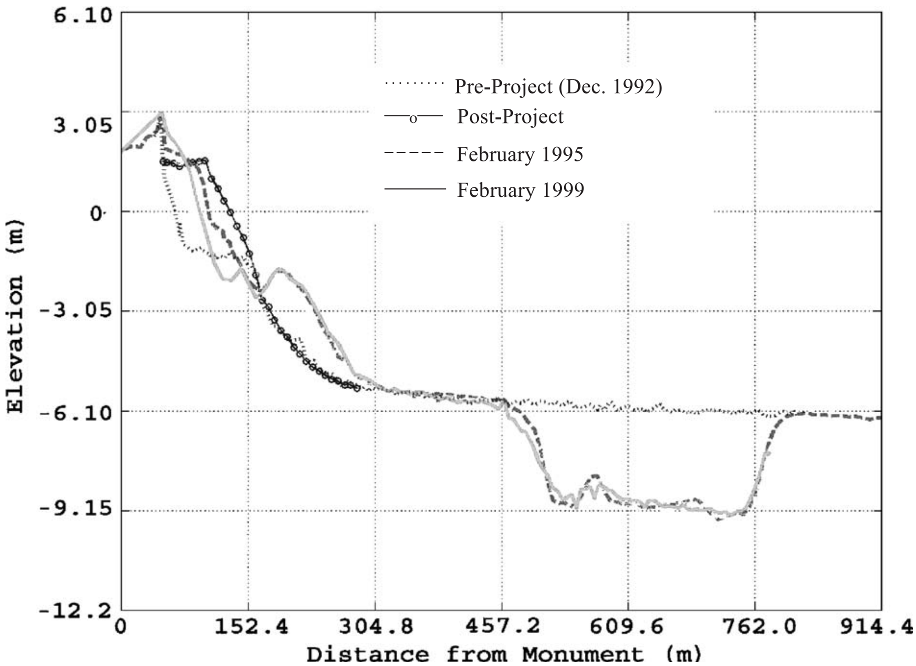

Fig. 4. Beach profile through borrow area at Monument R-26 in Anna Maria Key, FL; closure depth is 4.1 m (modified from Wang and Dean, 2001).

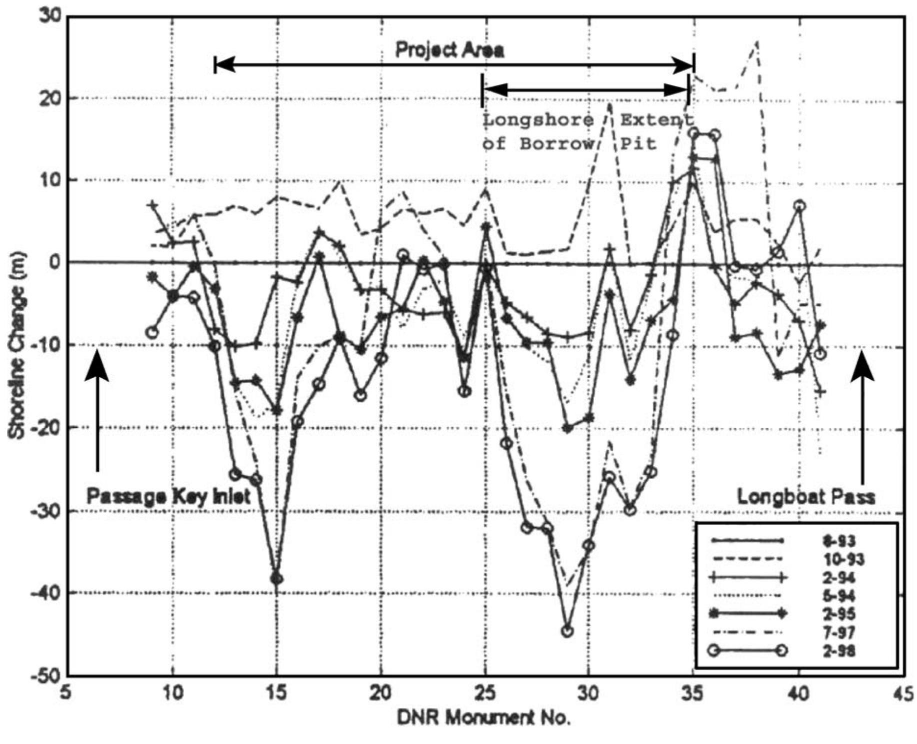

Fig. 5. Shoreline position for Anna Maria Key Project for different periods relative to August, 1993 (modified from Dean et al., 1999).

3.1.3. Martin County, Florida (1996)

The Hutchinson Island beach nourishment project in Martin County, Florida was constructed in 1996 with the placement of approximately 1.1×10^6 m^3 of sediment along 6.4 km of shoreline, between DNR Monuments R-1 and R-25 (Sumerell, 2000). The borrow area for this project was a shoal rising 4.9 m above the adjacent bed and lying 910 m offshore in 12.8 m of water. Fig. 6 shows the borrow area location offshore of the southern end of the project area. An average sediment thickness of 3 m was dredged from the central portion of the shoal.

The 3- and 4-year post-nourishment shoreline surveys show reasonable agreement with modeling conducted for the project, except at the southern end, near the borrow area (Sumerell, 2000). Fig. 7 shows the predicted shoreline and the survey data for the 4-year shoreline change. This case differs from the previous two as the borrow area did not create a pit, but reduced the height of an offshore shoal. By

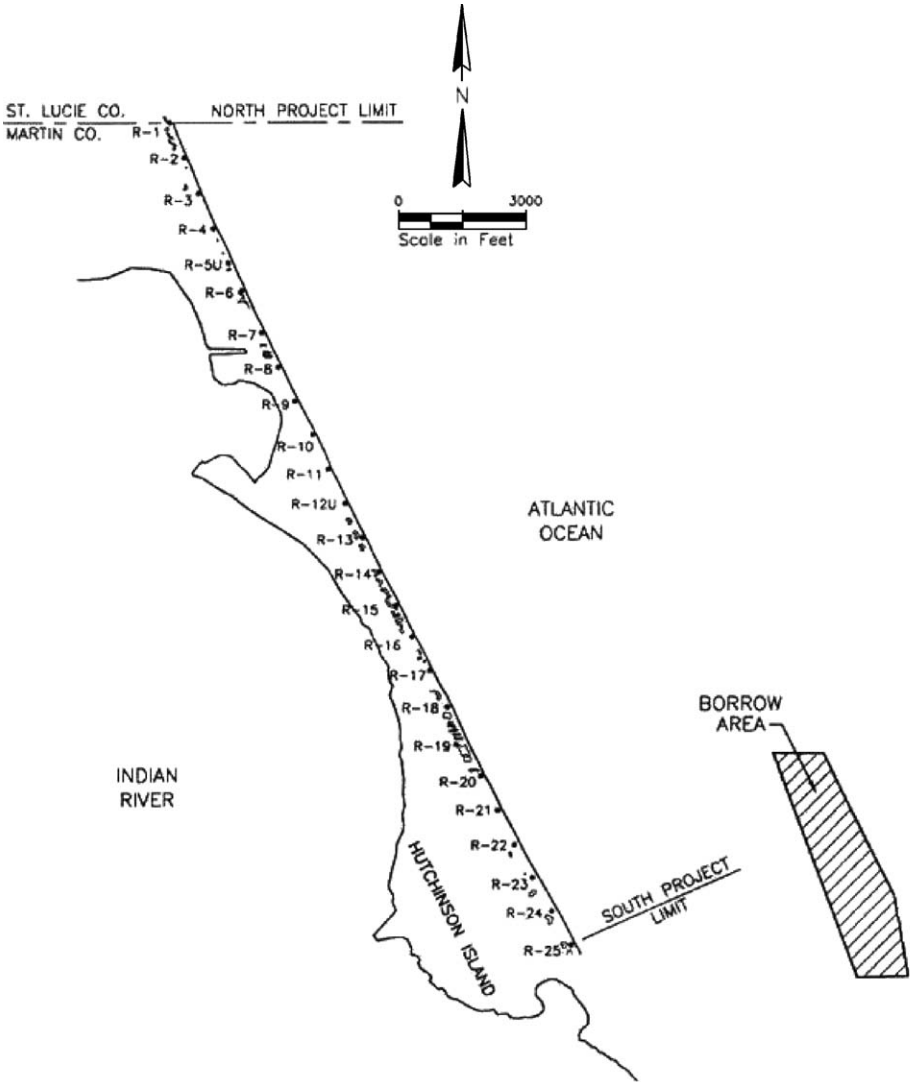

Fig. 6. Project area for Martin County beach nourishment project (Applied Technology and Management, 1998).

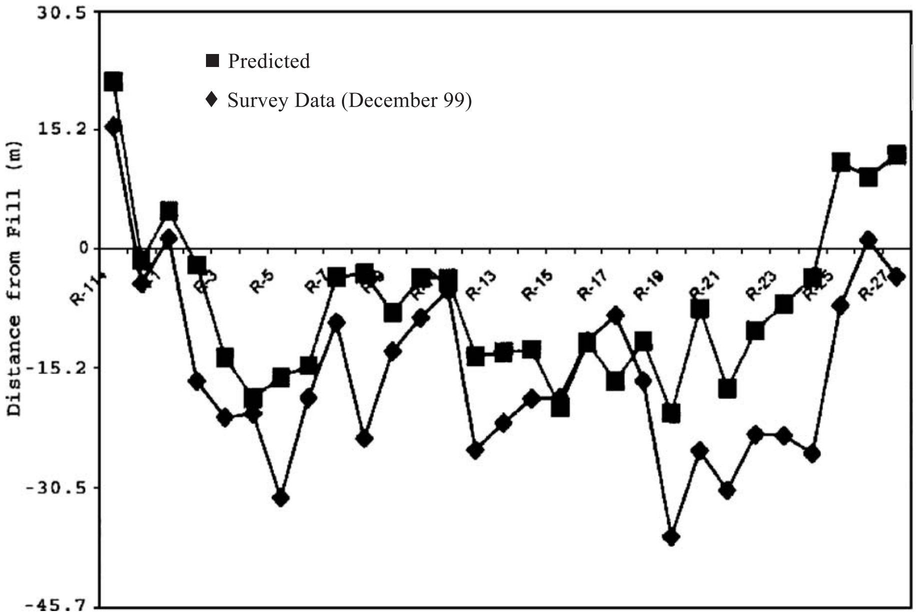

Fig. 7. Four-year shoreline change for Martin County beach nourishment project: predicted versus survey data (modified from Sumerell, 2000).

lowering the height of the shoal, the shoreline leeward of the borrow area was exposed to more wave action, which is the opposite of the sheltering (through reflection and possible damping) effect of an offshore pit. The borrow area, with its large extent and proximity to the project, is a possible reason for the higher than expected erosion at the southern end of the project.

3.1.4. Case studies summary comments

The three cases studies presented indicate the complex and multi-scale processes involved with large storm systems impacting the modified nearshore zone and driving the local sediment transport. While the Grand Isle, LA and Anna Maria Key, FL beach nourishment projects created large offshore borrow pits and were impacted by hurricanes soon after project completion, the shoreline response was contradictory with accretion occurring shoreward of the borrow areas at Grand Isle, and erosion occurring at Anna Maria Key. There are many possible reasons for the opposing behavior at the two sites including the smaller native sediment size, and therefore, more gradual slope at Grand Isle and differences in project construction between the two sites with a steeper slope, relative to other project areas, near the section of shoreline recession at Anna Maria Key. The beach nourishment project at Martin County, FL introduces another mechanism as dredging a shoal, while changing the local wave transformation will cause more wave energy to impact the shoreline. Even though the dynamics are too complex for general statements concerning shoreline response near bathymetric anomalies, the cases presented serve to illustrate possible shoreline behavior.

3.2. Field experiments

Field studies have been conducted to examine the effects of offshore dredging on the coastal environment. The purposes of these studies have varied and include the tendency of a dredged pit to induce sediment flows into it from the nearshore, the interception of sediment transport, and wave transformation effects of a newly dredged pit on the shoreline.

3.2.1. Price et al. (1978)

Price et al. (1978) investigated the effects of offshore dredging on the coastline of England and the tendency of a dredge pit to cause a drawdown of sediment and to prevent the onshore movement of sediment. The 3-year study by Inman and Rusnak

(1956) on the onshore–offshore interchange of sand off La Jolla, California was cited, which found vertical bed elevation changes of only ±0.03 m at depths greater than 9 m. Based on the consideration that the wave conditions off the southern coast of England would be less energetic than off La Jolla, California, Price et al. (1978) concluded that beach drawdown at depths greater than 10 m would not occur.

A radioactive tracer experiment off Worthing on the south coast of England was performed to investigate the mobility of sediment at depths of 9, 12, 15, and 18 m. This 20-month study found that at the 9 and 12 m contours there was a slight onshore movement of sediment and it was concluded that the movement of sediment beyond a depth contour of 18 m off the south coast of England would be negligible. Therefore, at these locations and in instances when the onshore movement of sediment seaward of the dredge area is a concern, dredging in water beyond 18 m of depth below low water level was considered acceptable (Price et al., 1978).

A numerical model of shoreline change due to wave refraction over dredged holes was also employed in the study, the details of which will be examined later in Section 5.2.1. The model found that minimal wave refraction occurred for pits in depths greater than 14 m for wave conditions typical off the coast of England.

3.2.2. Kojima et al. (1986)

The impact of dredging on the coastline of Japan was studied by Kojima et al. (1986). The wave climates as well as human activities (dredging, construction of structures, etc.) for areas with significant beach erosion and/or accretion were studied in an attempt to determine a link between offshore dredging and beach erosion. The study area was located offshore of the northern part of Kyushu Island. The wave climate study correlated yearly fluctuations in beach recession with the occurrence of both storm winds and severe waves and found that years with high frequencies of storm winds were likely to have high recession rates. A second study component compared annual variations in offshore dredging with annual beach recession rates and found strong correlation at some locations between erosion and the initiation of dredging; however, no consistent correlation was identified.

Hydrographic surveys documented profile changes of dredged holes over a 4-year period. At depths less than 30 m, significant infilling of the holes was found, mainly from the shoreward side, indicating a possible interruption in the longshore and offshore sediment transport. This active zone extended to a much larger depth than found by Price et al. (1978) and by Inman and Rusnak (1956). The explanation by Kojima et al. is that although the active onshore/offshore region does not extend to 30 m, sediment from the ambient bed will fill the pit causing a change in the supply to the upper portion of the beach and an increase in the beach slope. Changes in the beach profiles at depths of 35 and 40 m were small, and the holes were not filled significantly.

Another component of the study involved tracers and seabed level measurements to determine the depths at which sediment movement ceases. Underwater photographs and seabed elevation changes at fixed rods were taken at 5-m depth intervals over a period of 3 months during the winter season for two sites. The results demonstrated that sediment movement at depths up to 35 m could be significant. This depth was found to be slightly less than the average depth (maximum 49 m, minimum 20 m) for five proposed depth of closure equations using wave inputs with the highest energy (H=4.58 m, T=9.20 s) for the 3-month study period.

3.3. Laboratory experiments

3.3.1. Horikawa et al. (1977)

Laboratory studies have been carried out to quantify wave field and nearshore modifications due to the presence of offshore pits. Horikawa et al. (1977) performed wave basin tests with a model of fixed offshore bathymetry and uniform depth except for a rectangular pit of uniform depth and a nearshore region composed of moveable lightweight sediments. The experimental arrangement is shown in Fig. 8. The incident wave period and height were 0.41 s and 1.3 cm, respectively. With the pit covered, waves were run for 5.5 h to obtain an equilibrium planform followed by wave exposure for 3 h with the pit present. Shoreline measurements were conducted at 1-h intervals to determine the pit-induced changes. The results of the experiment are presented in Fig. 9. Almost all of the shoreline changes with the pit

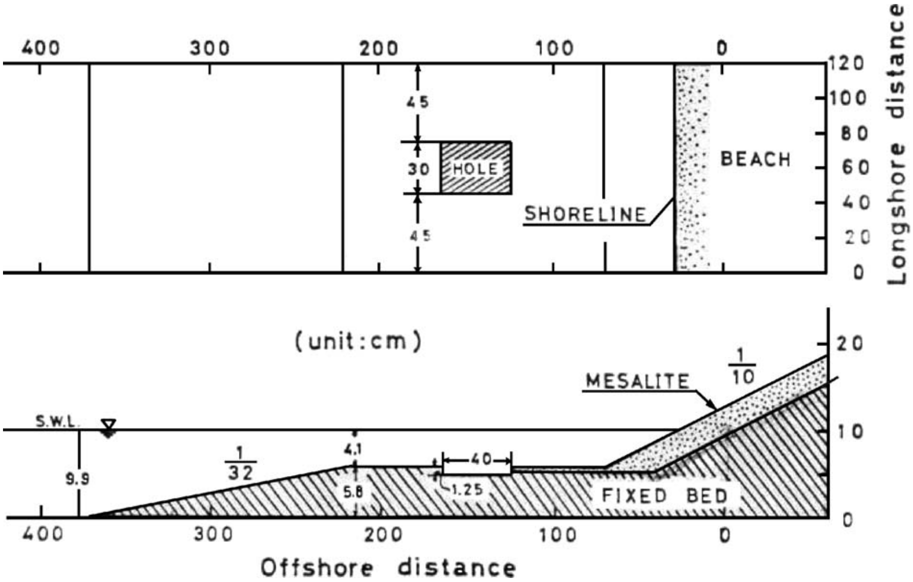

Fig. 8. Setup for laboratory experiment (Horikawa et al., 1977).

present occurred in the first 2 h. At the still water level, a salient formed shoreward of the pit, flanked by two areas of erosion that mostly extended to the sidewalls of the experiment; however, the depth contour at a water depth, $h = 0.85$ cm, also shown in Fig. 9, shows only a slightly seaward displacement at the pit centerline.

Wave height measurements were also conducted during the experiment. The wave heights measured along three shore-parallel transects were compared with results from a mathematical model developed for the study, which is discussed in Section 5.2.2. The observed wave heights 30 cm seaward of the pit and 30 cm leeward of the pit show fair agreement with the predicted wave heights. At the shoreline, the predicted and observed wave heights diverge with the superposition of incident and resonant standing waves generated between the basin walls offered as a possible reason for the variation by Horikawa et al. (1977). It would be desirable to conduct future experiments with

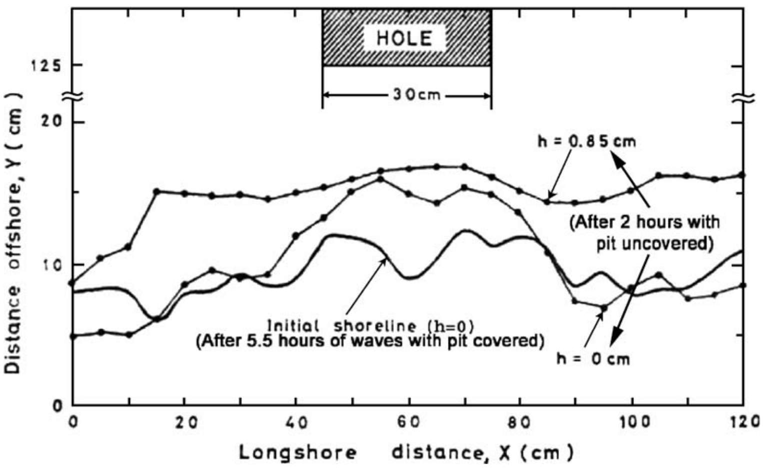

Fig. 9. Results from laboratory experiment showing plan shape after 2 h (modified from Horikawa et al., 1977).

a wider basin to reduce the potential impact of the sidewalls on the wave measurements and shoreline change results.

3.3.2. Williams (2002)

Williams (2002) performed wave basin experiments similar to those of Horikawa et al. (1977). The experimental setup of a fixed bed model containing a pit with a moveable sand shoreline was constructed for similar trials by Bender (2001) and was a larger scale version of the Horikawa et al. (1977) arrangement with a basin width of 3 m. The pit was 80 cm wide in the cross-shore direction, 60 cm in the longshore direction, and 12 cm deep relative to the adjacent bottom. The Williams experimental procedure consisted of shoreline, bathymetric, and profile measurements after specified time intervals that comprised a complete experiment. For analysis, the shoreline and volume measurements were presented relative to the final measurements of the previous 6-h phase. The conditions for the experiments were waves of 6-cm height with 1.35-s period and a depth of 15 cm in the constant depth region surrounding the pit.

Shoreline and volume change results were obtained for three experiments. The volume change results showed that the model beach landward of the pit lost volume at almost every survey location during the period with the pit covered and experienced a gain in volume with the pit uncovered. Similar volume change per unit length results were found in all three experiments indicating a positive volumetric relationship between the presence of the pit and the landward beach. In addition to the local volume changes, which were attributed to the presence of the pit, total volume changes were documented. In some tests, these were substantial and due, in part, to sediment deposition in the pit.

The shoreline change results showed shoreline retreat, relative to Time 0.0, in the lee of the borrow pit during the control phase (pit covered) for all three experiments with the greatest retreat at or near the centerline of the borrow pit. All three experiments showed shoreline advancement in the lee of the borrow pit during the test phase (pit uncovered). With the magnitude of the largest advancement being almost equal to the largest retreat in each experiment, it was concluded that, under the conditions tested, the presence of the borrow pit resulted in shoreline advancement for the area shoreward of the borrow pit (Williams, 2002).

An even–odd analysis was applied to the shoreline and volume change results in an attempt to isolate the effect of the borrow pit. The even function was assumed to represent changes due solely to the pres-

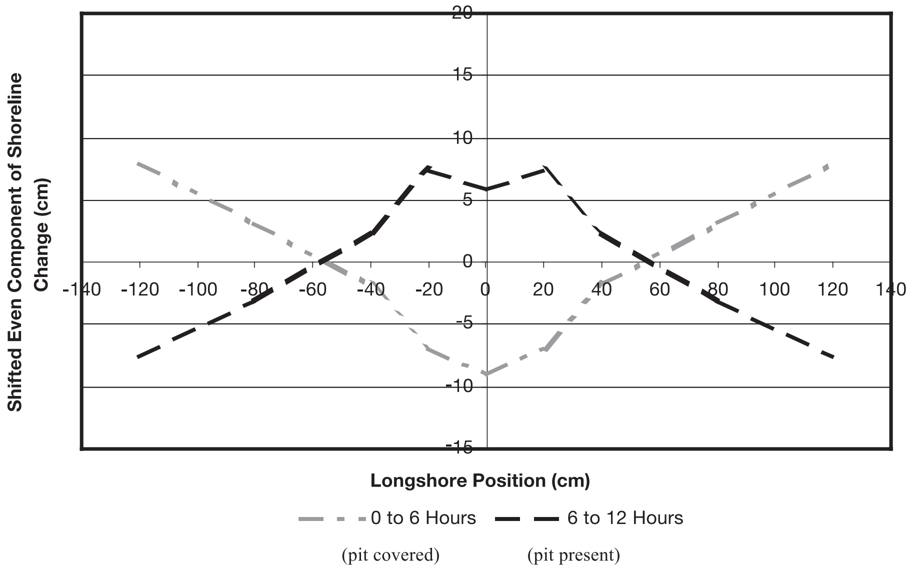

Fig. 10. Shifted even component of shoreline change for first experiment (Williams, 2002).

ence of the borrow pit. The even components were adjusted to obtain equal positive and negative areas, which were not obtained using the laboratory data. For each experiment, the shifted even results shoreward of the pit showed positive values during the test phase for both the shoreline and volume changes with negative values during the control phase. The shifted even component of shoreline change for the first experiment is shown in Fig. 10 with the results indicating the recovering from the prior wave conditions with the pit either uncovered or covered. These results further verify the earlier findings concerning the effect of the pit. Although an even basin mode in the longshore of twice the incident wave length is slightly greater than the basin width (3.10 versus 3 m), this mode is not believed to be significant in the shoreline change results.

4. Wave transformation

4.1. Analytic methods

There is a long history of the application of analytic methods to determine wave field modifications by bathymetric changes. Early research centered on the effect on normally incident long waves of an infinite step, trench, or shoal of uniform depth in an otherwise uniform depth domain. More complex models were later developed to remove the long wave restriction, add oblique incident waves, and allow for the presence of a current. More recently, many different techniques have been developed to obtain solutions for domains containing pits or shoals of finite extent. Some of these models focused solely on wave field modifications, while others of varying complexity examined both the wave field modifications and the resulting shoreline impact.

4.1.1. 1-D methods

By matching surface displacement and mass flux normal to the change in bathymetry, Lamb (1932) was one of the first to develop a long wave approximation for the reflection and transmission of a normally incident wave at a finite depth step. Bartholomeusz (1958) performed a more thorough analysis of the finite depth step problem and found that the Lamb solution provided correct results for the reflection and transmission coefficients for lowest order (kh), where k is the incident wave number and h is the water depth upwave of the step. Sretenskii (1950) investigated oblique waves over a step between finite and infinite water depths assuming the wave length to be large compared to the finite depth. An extensive survey of early theoretical work on surface waves including obstacle problems is found in Wehausen and Laitone (1960).

Jolas (1960) studied the reflection and transmission of water waves of arbitrary relative depth over a wide submerged rectangular parallelepiped (sill) and performed an experiment to document wave transformation. To solve the case of normal wave incidence and arbitrary relative depth over a sill or a fixed obstacle at the surface, Takano (1960) employed an eigenfunction expansion of the velocity potentials in each constant depth region and matched them at the region boundaries. The set of linear integral equations was solved for a truncated series. A laboratory experiment was also conducted in this study.

Dean (1964) investigated long wave modifications by linear transitions, which included both horizontal and vertical changes. The formulation allowed for many configurations including a step, either up or down, and converging or diverging linear transitions with a sloped wall. A solution was defined with plane waves of unknown amplitude and phase for the incident and reflected waves with the transmitted wave specified. Wave forms, both transmitted and reflected, were represented by Bessel functions in the region of linear variation in depth and/or width. The unknown coefficients were obtained through matching the values and gradients of the water surfaces at the ends of the transitions. Analytic expressions were found for the reflection and transmission coefficients. The results indicate that the reflection and transmission coefficients depend on the relative depth and/or width and a dimensionless parameter containing the transition slope, the wave length, and the depth or width. The solutions were shown to converge to those of Lamb (1932) for the case of an abrupt transition.

Newman (1965a) studied wave transformation due to normally incident waves on a single step between regions of finite and infinite water depth with an integral equation approach. This problem was also examined by Miles (1967) who developed a plane-wave solution for unrestricted kh values using a variational approach (Schwinger and Saxon 1968), which for this case essentially solves a single equation instead of a series of equations (up to 80 in Newman's solution) as in the integral equation approach. The difference between the results for the two solution methods was within 5% for all kh values (Miles, 1967).

Newman (1965b) examined the propagation of water waves past wide obstacles. The problem was solved by constructing a domain with two steps placed "back to back" and applying the solutions of Newman (1965a). Complete transmission was found for certain water depth and pit width combinations; a result proved by Kreisel (1949) for an obstacle (trench or shoal) with arbitrary cross-section with the upwave and downwave depths uniform.

The variational approach was applied by Mei and Black (1969) to investigate the scattering of surface waves by rectangular obstacles. For a submerged obstacle, complete transmission was found for certain kd values where d is the depth over the obstacle. A comparison of the results of Mei and Black (1969) and those of Newman (1965b) is shown in Fig. 11, which presents the reflection coefficient versus kd for an elevated sill. Data from the Jolas (1960) experiment are also included on the plot and compared to the results of Mei and Black (1969) for a specific ℓ/d, where ℓ is the half-width of the obstacle. In this figure, and some that follow, the notation has been changed to provide consistency within the present paper.

Black and Mei (1970) applied the variational approach to examine the radiation caused by oscillating bodies and the disturbance caused by an object in a wave field. Two domains were used for both submerged and semi-immersed (surface) bodies: the first domain was in Cartesian coordinates, with one vertical and one horizontal dimension for horizontal cylinders of rectangular cross-section; and the second domain was in cylindrical coordinates, for vertical cylinders of circular section. The second domain allowed for objects with two horizontal dimensions to be studied (see Section 4.1.2). Black et al. (1971) applied the variational formulation to study radiation due to the oscillation of small bodies and the scattering induced by fixed bodies and demonstrated the scattering caused by a fixed object in a single figure; see Black and Mei (1970) for further results.

Lassiter (1972) used complementary variational integrals to solve the problem of normally incident waves on an infinite trench where the depth on the two sides of the trench may be different (the asymmetric case). The symmetric infinite trench problem was studied by Lee and Ayer (1981), who employed a transform method. The fluid domain was divided into two regions, one an infinite uniform depth domain and the other a rectangular region representing the trench below the uniform seabed level.

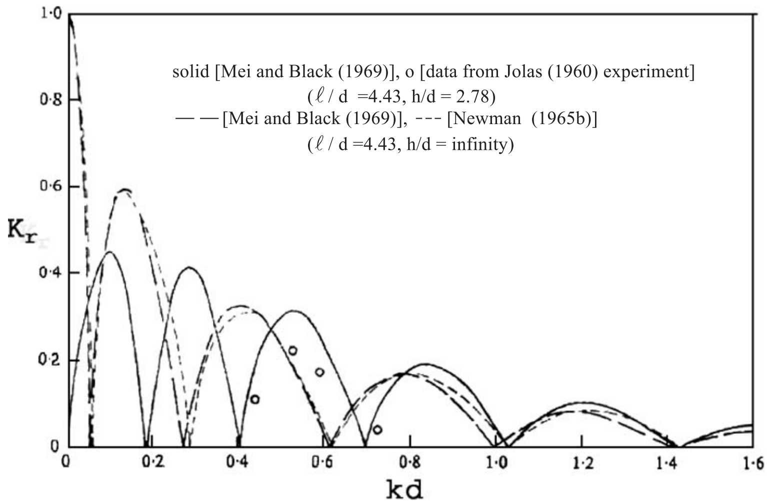

Fig. 11. Reflection coefficient for a submerged elevated sill (modified from Mei and Black, 1969).

Lee et al. (1981) proposed a boundary integral method for the propagation of waves over a prismatic trench of arbitrary shape, which was used for comparison to selected results in Lee and Ayer (1981). The solution was found by matching the unknown normal derivative of the potential at the boundary of the two regions. A comparison to previous results for trenches with vertical sidewalls was conducted with good agreement. A case with "irregular" bathymetry was demonstrated in a plot of the transmission coefficient for a trapezoidal trench where λ is the wave length (Fig. 12).

Miles (1982) solved for the diffraction by an infinite trench for obliquely incident long waves. The solution method for normally incident waves used a procedure developed by Kreisel (1949) that conformally mapped a domain containing certain obstacles of finite dimensions into a rectangular strip. Kreisel (1949) presented this method without derivation and with no consideration of the phase. To add the capability of solving for obliquely incident waves, Miles used the variational formulation of Mei and Black (1969).

The problem of obliquely incident waves over an asymmetric trench was solved by Kirby and Dalrymple (1983a) using a modified form of Takano's (1960) method. Fig. 13 compares the reflection coefficient for the numerical solution for normally incident waves and the results of Lassiter (1972), along with results from a boundary integral method used to provide verification. In this figure, h_1 is the water depth upwave of the trench and k_0 is the deep-water wave number. Differences in the results of Kirby and Dalrymple and those of Lassiter are evident. Lee and Ayer (1981, see their Fig. 2) also demonstrated differences in their results and those of Lassiter (1972). The effect of oblique incidence wave investigated by Kirby and Dalrymple and results are shown in Fig. 14, where the transmission coefficients for two angles of incidence are plotted. This study also investigated the plane-wave approximation and the long-wave limit, which allowed for comparison to Miles (1982).

An extension of this study is found in Kirby et al. (1987), where the effects of currents flowing along the trench are included. The presence of an ambient current was found to significantly alter the reflection and transmission coefficients for waves over a trench compared to the no current case. Adverse currents and following currents resulted in less and more reflective conditions, respectively (Kirby et al., 1987).

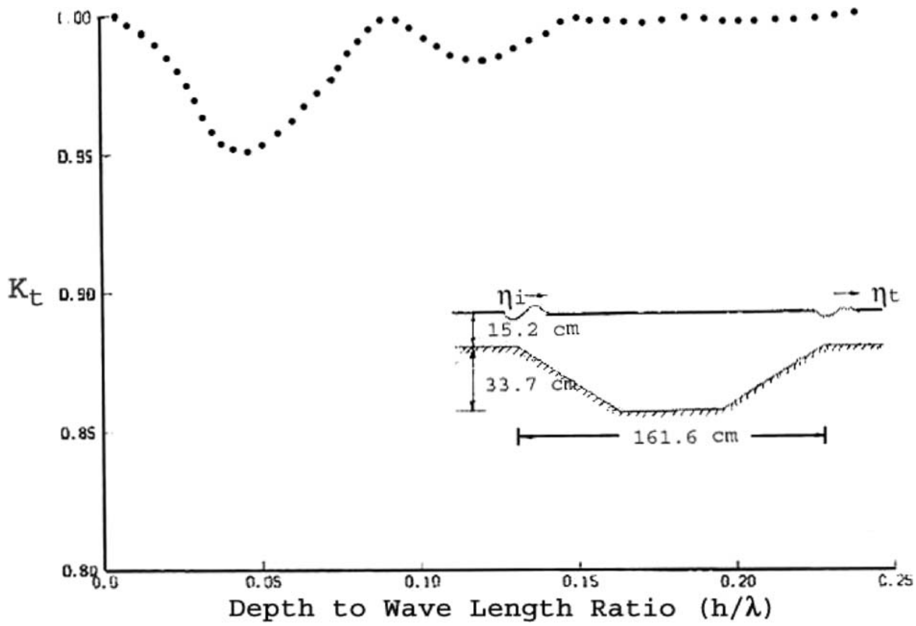

Fig. 12. Transmission coefficient as a function of relative wave length for trapezoidal trench; setup shown with inset diagram (modified from Lee et al., 1981).

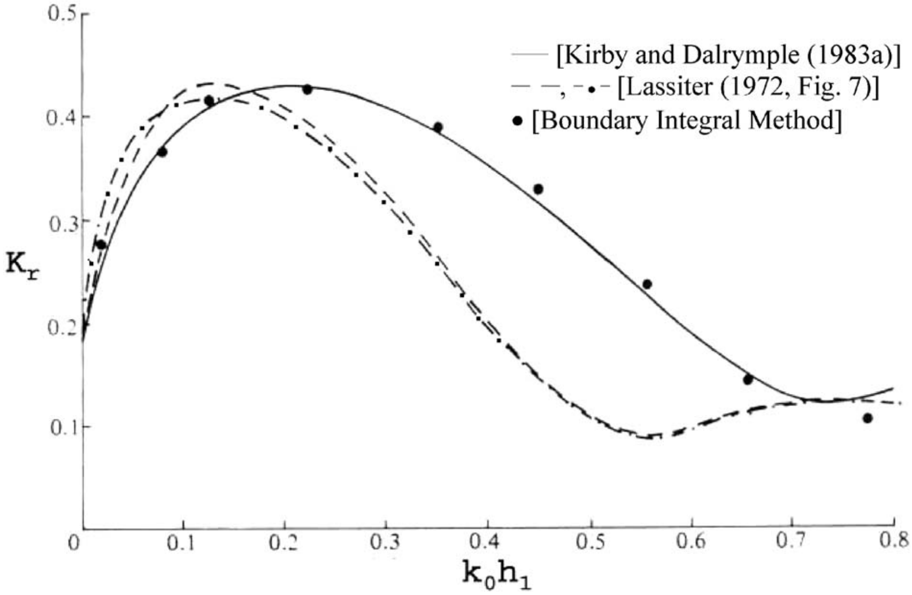

Fig. 13. Reflection coefficient for asymmetric trench and normally incident waves as a function of k_0h_1: $d/h_1=2$, $h_3/h_1=0.5$, $L/h_1=5$ (modified from Kirby and Dalrymple, 1983a).

4.1.2. 2-D methods

Extending the infinite trench and step solutions (one horizontal dimension) to a two-dimensional domain is a natural progression allowing for the more realistic case of wave transformation by a finite object or depth anomaly to be studied. Changes in bathymetry can cause changes in wave height and direction through the four wave transformation processes noted earlier. Some of the two-dimensional models consider only wave transformation, while others use the modified wave field to determine the impact of a pit or shoal on the shoreline. Several models use only a few equations or matching conditions on the boundary of the pit or shoal to determine the wave field, and in some cases, the impact on the shoreline in a simple domain containing a pit or shoal. Other much more complex and complete models and program packages have been developed to solve numerically for the wave field over a complex bathymetry, which may contain pits and/or shoals. Both types of models can provide insight into the effect of a pit or shoal on the local wave field and the resulting impact on the shoreline.

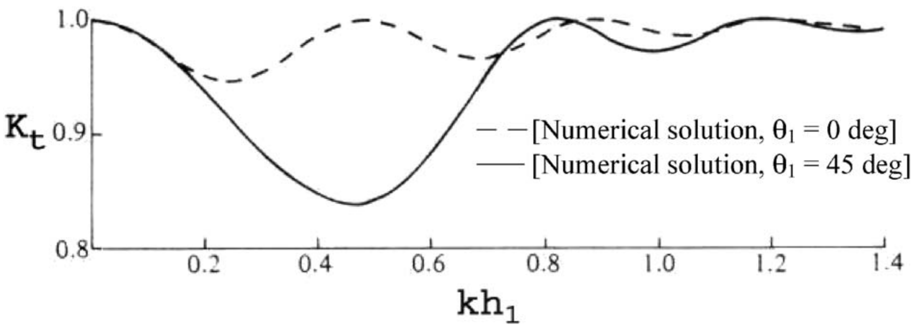

Fig. 14. Transmission coefficient for symmetric trench, two angles of incidence: $L/h_1=10$, $d/h_1=2$ (modified from Kirby and Dalrymple, 1983a).

Black and Mei (1970) studied wave transformation in a two-dimensional domain by solving for the radially symmetric case of a submerged or semi-immersed fixed circular cylinder in cylindrical coordinates. A series of Bessel functions was used for the incident and reflected waves, as well as for the solution over the shoal with modified Bessel functions representing the evanescent modes. As mentioned previously in Section 4.1.1, a variational approach was used and both the radiation by oscillating bodies and the disturbance caused by a fixed body were studied. The focus of the fixed body component of the study was the total scattering cross-section, Q, which is equal to the width between two wave rays within which the normally incident wave energy flux would be equal to that scattered by the obstacle and the differential scattering cross-section, representing the angular distribution of the scattered energy (Black and Mei, 1970). Fig. 15 shows the total scattering cross-section for a circular cylinder at the seabed for three ratios of cylinder radius (r) to depth over the cylinder (d).

Williams (1990) developed a numerical solution for the modification of long waves by a rectangular pit using Green's second identity and appropriate Green's functions in each region that comprises the domain. This formulation accounts for the diffraction, refraction, and reflection caused by the pit. The domain for this method consists of a uniform depth region containing a rectangular pit of uniform depth with vertical sides. The solution requires discretizing the pit boundary into a finite number of points at which the velocity potential and the derivative of the velocity potential normal to the boundary must be determined. Applying matching conditions for the pressure and mass flux across the boundary results in a system of equations amenable to matrix solution techniques. Knowledge of the potential and derivative of the potential at each point on the pit boundary allows determination of the velocity potential solution anywhere in the fluid domain. A partial standing wave pattern of increased and decreased relative amplitude was found to develop seaward of the pit with a shadow zone of decreased wave amplitude landward of the pit flanked by two areas of increased relative amplitude.

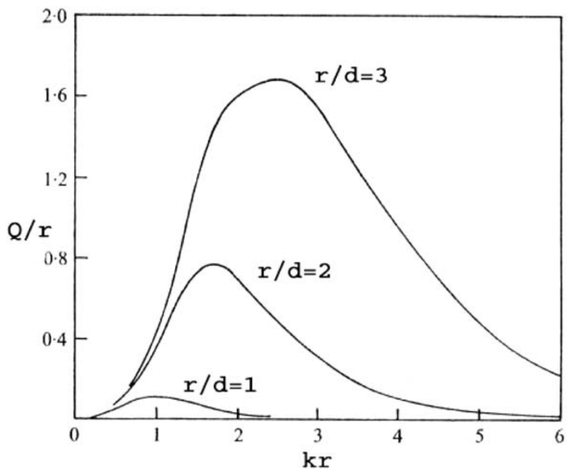

Fig. 15. Total scattering cross-section of a bottom-mounted vertical circular cylinder (modified from Black et al., 1971).

McDougal et al. (1996) applied the method of Williams (1990) to the case of a domain with multiple pits. The first part of the study reinvestigated the influence of a single pit on the wave field for various pit geometries. A comparison of the wave field in the presence of a pit versus a surface piercing structure is presented in Figs. 16 and 17, which present contour plots of the transformation coefficient, K (equal to relative amplitude), with the characteristics discussed in the previous paragraph. In these figures, a and b are the longshore and cross-shore pit dimensions, respectively. Although a similar minimum transformation coefficient was found for the two cases, a greater sheltering effect was found landward of the pit than for the case of the full depth breakwater.

For the case of multiple pits, it was found that placement of one pit in the shadow zone of a more seaward pit was most effective in reducing the wave height leeward; however, adding a third pit did not produce significant wave height reduction as compared to the two-pit results.

Williams and Vazquez (1991) removed the long wave restriction of Williams (1990) and applied the Green's function solution method outside of the pit. This solution was matched to a Fourier expansion solution inside the pit with matching conditions at the pit boundary. Once again the pit boundary must be discretized into a finite number of points and a matrix solution for the resulting series of equations was used. Removing the shallow water restriction allowed for many new cases to be studied and as the wave conditions approach deep water, the influence of the

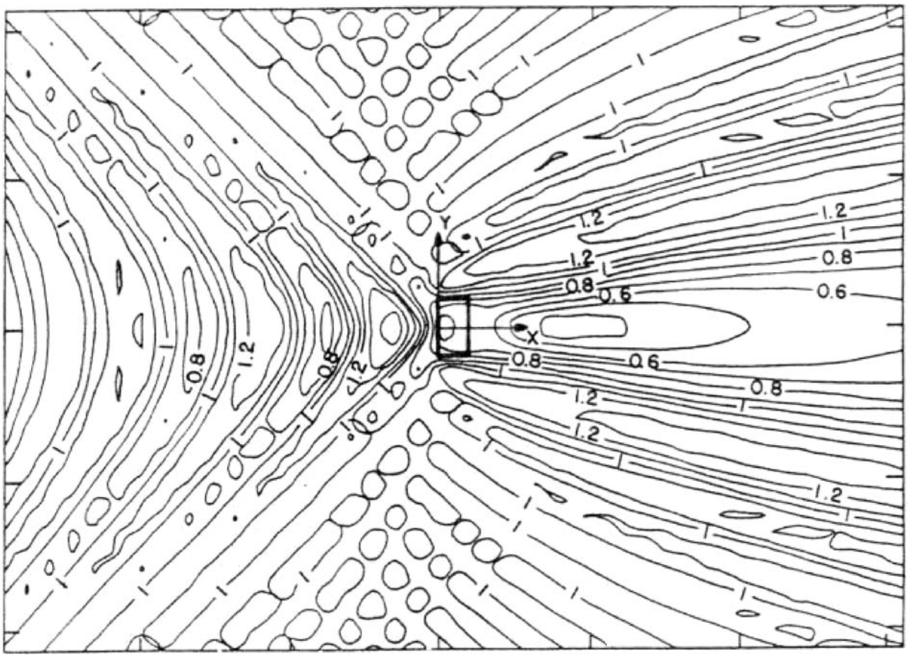

Fig. 16. Contour plot of transformation coefficient in and around pit for normal incidence; $a/\lambda = 1$, $b/\lambda = 0.5$, $d/h = 3$, $kh = 0.167$ (McDougal et al., 1996).

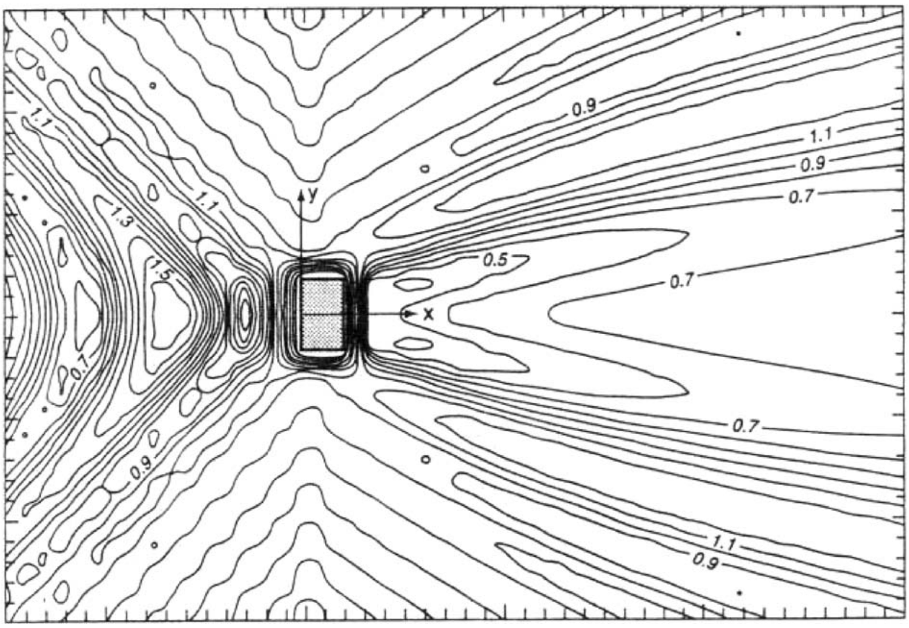

Fig. 17. Contour plot of transformation coefficient around surface-piercing breakwater for normal incidence; $a/\lambda = 1$, $b/\lambda = 0.5$, $kh = 0.167$ (McDougal et al., 1996).

pit diminishes. A plot of the global minimum and maximum relative amplitude versus the dimensionless pit width (the wave number outside of the pit times the cross-shore pit dimension, kb) is shown in Fig. 18. The maximum and minimum relative amplitudes are seen to occur near $kb = 2\pi$ or when $\lambda = b$ and then approach unity as the dimensionless pit width increases. The reason that the extreme values do not occur exactly at $kb = 2\pi$ is explained by Williams and Vazquez (1991) as due to diffraction effects near the pit modifying the wave characteristics.

4.2. Numerical methods

The previous two-dimensional solutions, while accounting for some of the wave transformation processes caused by a pit, are limited in their representation of the bathymetry and none account for wave energy dissipation or nonlinear effects. Numerical approaches allow much greater flexibilities. Berkhoff (1972) developed a formulation for the three-dimensional propagation of waves over an arbitrary bottom in a vertically integrated form that reduced the problem to two dimensions. This solution is known as the mild slope equation, and different forms of the solution have been developed into parabolic (Radder, 1979), hyperbolic, and elliptic (Berkhoff et al., 1982) models of wave propagation, which vary in their approximations and solution techniques. Numerical methods allow solution for wave propagation over an arbitrary bathymetry. Some examples of the parabolic and elliptic models are RCPWAVE (Ebersole et al., 1986), REF/DIF-1 (Kirby and Dalrymple, 1994), and the MIKE 21 EMS Module (Danish Hydraulic Institute, 1998). Other models such as SWAN (Holthuijsen et al., 2000) and STWAVE (Resio, 1988b; Smith et al., 2001) model wave transformation in the nearshore zone using the wave-action balance equation. These models provide the capability to model wave transformation over complicated bathymetries and may include processes such as bottom friction, non-linear interaction, breaking, wave–current interaction, wind–wave growth, and white capping to better simulate the nearshore zone. An extensive review of any of the models is beyond the scope of this paper; however, a brief summary of the capabilities of some of the models is presented in Table 1.

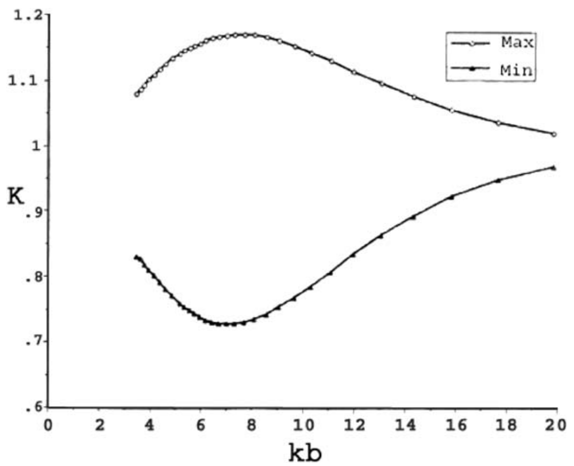

Fig. 18. Maximum and minimum transmission coefficient versus kb for normal incidence, $b/a = 6$, $b/d = \pi$, and $d/h = 2$ (modified from Williams and Vazquez, 1991).

Maa et al. (2000) provide a comparison of six numerical models. Two parabolic models are examined: RCPWAVE and REF/DIF-1. RCPWAVE employs a parabolic approximation of the elliptic mild slope equation and assumes irrotationality of the wave phase gradient. REF/DIF-1 extends the mild slope equation by including nonlinearity and wave–current interaction (Kirby and Dalrymple, 1983b; Kirby, 1986). Of the four other models included, two are defined by Maa et al. (2000) as based on the transient mild slope equation (Copeland, 1985; Madsen and Larsen, 1987) and two are classified as elliptic mild slope equation (Berkhoff et al., 1982) models. The transient mild slope equation models presented are the Mike 21 EMS Module and the PMH Model (Hsu and Wen, 2001). The elliptic mild slope equation models use different solution techniques with the RDE Model (Maa and Hwung, 1997; Maa et al. 1998a) applying a special Gaussian elimination method and the PBCG Model (Preconditioned Bi-conjugate Gradient) (Maa et al., 1998b).

A table in Maa et al. (2000) provides a comparison of the capabilities of the six models. A second table summarizes the computation time, memory required, and where relevant, the number of iterations for a test case of monochromatic waves over a shoal on an incline; the Berkhoff et al. (1982) shoal. The parabolic approximation solutions of REF/DIF and RCPWAVE required significantly less memory (up to 10 times less) and computation time (up to 70 times less) than

Table 1
Capabilities of selected nearshore wave models

	RCPWAVE	REF/DIF-1	Mike 21 (EMS)	STWAVE	SWAN 3rd generation
Solution method	Parabolic mild slope equation	Parabolic mild slope equation	Elliptic mild slope equation (Berkhoff et al. (1982)	Conservation of wave action	Conservation of wave action
Phase	Averaged	Resolved	Resolved	Averaged	Averaged
Spectral	No	No (Use REF/DIF-S)	No (Use NSW unit)	Yes	Yes
Shoaling	Yes	Yes	Yes	Yes	Yes
Refraction	Yes	Yes	Yes	Yes	Yes
Diffraction	Yes (Small-angle)	Yes (Wide-angle)	Yes (Total)	No (Smoothing)	No
Reflection	No	Yes (Forward only)	Yes (Total)	No	Yes (Specular)
Breaking	Stable energy flux: Dally et al. (1985)	Stable energy flux: Dally et al. (1985)	Bore model: Battjes and Janssen (1978)	Depth limited: Miche (1951) criterion	Bore model: Battjes and Janssen (1978)
White capping	No	No	No	Resio (1987)	Komen et al. (1984), Janssen (1991), Komen et al. (1994)
Bottom Friction	No	Dalrymple et al. (1984) both laminar and turbulent BBL	Quadratic friction law, Dingemans (1983)	No	Hasselmann et al. (1973), Collins (1972), Madsen et al. (1988)
Currents	No	Yes	No	Yes	Yes
Wind	No	No	No	Resio (1988a)	Cavaleri and Malanotte-Rizzoli (1981), Snyder et al. (1981), Janssen (1989, 1991)
Availability	Commercial	Free	Commercial	Free	Free

the elliptic models, which is expected due to the solution techniques and approximations contained in the parabolic models. The required computation times and memory requirements for the transient mild slope equation models were found to be intermediate to the other two methods.

Wave height and direction were calculated in the test case domain for each model. The models based on the transient mild slope equation and the elliptic mild slope equation were found to produce almost equivalent values of the wave height and direction. The parabolic approximation models were found to predict different values, with RCPWAVE showing different wave heights and directions behind the shoal and REF/DIF showing good wave height agreement with the other methods, but no change in the wave direction behind the shoal. A plot of the computed normalized wave heights for the six models and experimental data along a transect taken parallel to and downwave of the major shoal axis is shown in Fig. 19. Only four results are plotted because the RDE model, the PMH model, and PBCG model produced almost identical results.

The wave directions found with REF/DIF-1 in Maa et al. (2000) were found to be in error by Grassa and Flores (2001), who demonstrated that a second order parabolic model, equivalent to REF/DIF-1, was able to reproduce the wave direction field behind a shoal such as in the Berkhoff et al. (1982) experiment.

Application of numerical models to the problem of potential impact on the shoreline caused by changes to the offshore bathymetry was conducted by Maa and Hobbs (1998) and Maa et al. (2001). In Maa and Hobbs (1998), the impact on the coast due to dredging of an offshore shoal near Sandbridge, Virginia was investigated using RCPWAVE. National Data Buoy Center (NDBC) data from an offshore station and bathymetric data for the area were used to examine several cases with different wave events and directions. The resulting wave heights, directions, and sediment transport at the shoreline were

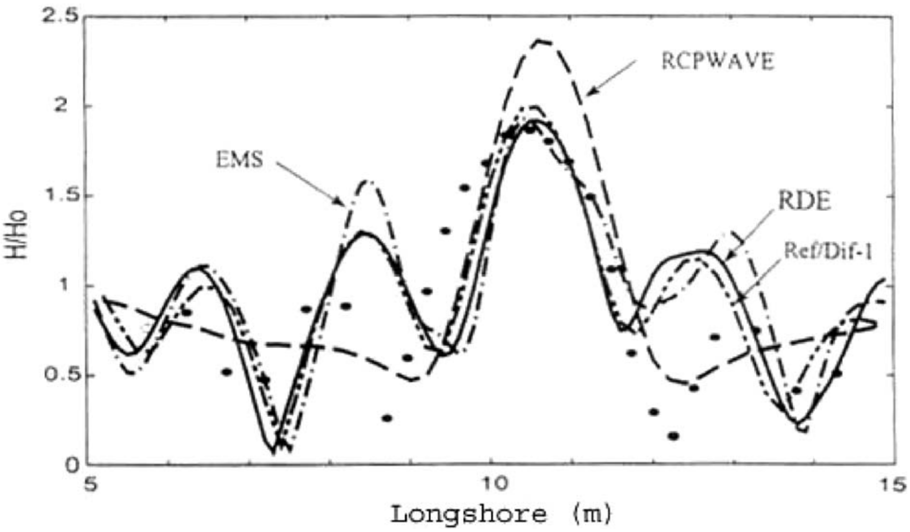

Fig. 19. Comparison of wave height profiles for selected models along a transect parallel to the major shoal axis located 9 m shoreward of shoal apex [● = experimental data] (modified from Maa et al., 2000).

compared. The sediment transport was calculated using the formulation of Gourlay (1982), which contains two terms, one driven by the breaking wave angle and one driven by the gradient in the breaking wave height in the longshore direction. Section 5.1 provides a more detailed examination of the longshore transport equation with two terms. The study found that the proposed dredging would have little impact on the shoreline for the cases investigated.

Later, Maa et al. (2001) revisited the problem of dredging at the Sandbridge Shoal by examining the impact on the shoreline caused by three different dredging configurations. RCPWAVE was used to model wave transformation over the shoal and in the nearshore zone. The focus was on the breaking wave height; wave direction at breaking was not considered. The changes in the breaking wave height modulation (BHM) along the shore after three dredging phases were compared to the results found for the original bathymetry and favorable or unfavorable assessments were provided for ensuing impact on the shoreline. The study concluded that there could be significant differences in wave conditions revealed by variations in the BHM along the shoreline depending on the location and extent of offshore dredging.

5. Shoreline response

5.1. Longshore transport considerations

The previous discussion on one- and two-dimensional models focused first on simple and complex methods of determining wave transformation caused by changes in offshore bathymetry and then applications that determined changes to the wave height, direction, and longshore transport at the shoreline. The focus of this section is shoreline change modeling using analytic and numerical methods. With wave heights and directions specified along the shoreline, sediment transport can be calculated and, based on the gradients in longshore transport, the changes in shoreline position can be quantified.

Longshore transport can be driven by two terms as was discussed previously in the review of Maa and Hobbs (1998). In most situations where offshore bathymetry is somewhat uniform, the magnitude and direction of the longshore transport will depend mostly on the wave height and angle at breaking as the longshore gradient in the breaking wave height will be small. In areas with irregular bathymetry or in the presence of structures, the transformation of the wave field can lead to areas of wave amplification and reduction resulting in considerable longshore gradients in wave height. Longshore transport equations containing a transport term driven by the breaking wave angle and another driven by the longshore gradient in the wave height can be found in Ozasa and Brampton (1979), who cite the formulation of Bakker (1971) for the longshore current, Gourlay (1982), Kraus and Harikai (1983), and Kraus (1983). While the value of the coefficient for the transport term driven by the gradient in wave height is not well established, the potential contribution of this term is significant in some cases. It is shown later that under steady conditions, the diffusive nature of the angle-driven transport term is required to modify the wave height gradient transport term in order to generate an equilibrium planform when both terms are active.

5.2. Refraction models

5.2.1. Motyka and Willis (1974)

Motyka and Willis (1974) were among the first to apply a numerical model to predict shoreline changes due to altered offshore bathymetry. The model only included the effect of refraction caused by offshore pits for idealized sand beaches representative of those found on the English Channel or North Sea coast of England. A simplified version of the Abernethy and Gilbert (1975) wave refraction model was used to determine wave transformation of uniform deep water waves over the nearshore bathymetry. The breaking wave height and direction was calculated and used to determine the sediment transport which was combined with the continuity equation to predict shoreline change. The longshore transport was calculated using the Scripps (or CERC) Equation as modified by Komar (1969):

$$Q = \frac{0.045}{\gamma_s} \rho g H_b^2 C_g \sin(2\alpha_b) \tag{1}$$

where Q is the volume rate of longshore transport, γ_s is the submerged unit weight of the beach material, ρ is the density of the fluid, g is gravity, H_b is the breaking wave height, C_g is the group velocity at breaking, and α_b is the breaking wave angle relative to the shoreline. This form of the Scripps Equation combines the transport and porosity coefficients into one term; the value used for either parameter was not stated and would only affect the time scale of evolution. This process was repeated to account for shoreline evolution with time.

The model determined that erosion occurs shoreward of a pit, bordered by areas of accretion. For the wave conditions used, shoreline planform stability was found after an equivalent period of 2 years. During the runs, 'storm' waves (short period and large wave height) were found to cause larger shoreline changes than the 'normal' waves with longer periods and smaller wave heights, which actually reduced the erosion caused by the storm waves. Fig. 20 shows a comparison of the predicted shorelines for the equivalent of 2 years of waves over 1 and 4 m deep pits with a longshore extent of 880 m and a cross-shore extent of 305 m. The detailed pit geometries were not specified. The erosion directly shoreward of the pits is shown in Fig. 20 with more erosion occurring for the deeper pit.

5.2.2. Horikawa et al. (1977)

Horikawa et al. (1977) developed a numerical model for shoreline changes due to offshore pits. The model applies the Scripps Equation for the longshore sediment transport; however, to match the Scripps Equation and for a dimensionally correct expression, the g term in the numerator should be removed from the equation presented. A model by Sasaki (1975) for diffraction behind breakwaters was modified to account for refraction only. The model computes successive points along the wave ray paths. Interpolation for the depth and slope is conducted along the ray path with an iteration procedure to calculate each successive point. The wave conditions

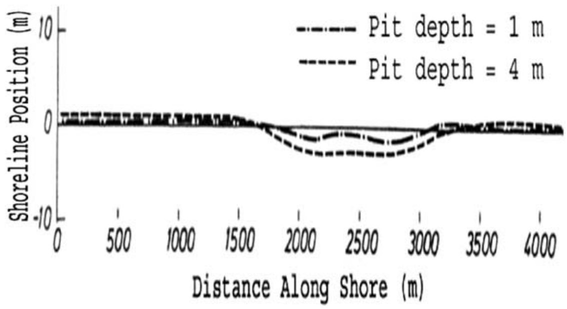

Fig. 20. Calculated beach planform due to refraction after 2 years of prototype waves for two pit depths (modified from Motyka and Willis, 1974).

were selected to be typical of the Eastern Japan coast facing the Pacific Ocean. Several pit dimensions and locations were used with the longshore dimension of the pit from 2 to 4 km, a cross-shore width of 2 km, pit depth of 3 m, and water depths at the pit from 20 to 50 m.

For the configurations modeled, accretion was found directly shoreward of the pit, flanked by areas of erosion. The magnitude of the accretion behind the pit and the erosion in the adjacent areas were found to increase with increasing longshore pit length and for pits located closer to shore. The shoreline planform for a model after the equivalent of 2 years of waves is shown in Fig. 21 with a salient directly shoreward of the pit.

Although Horikawa et al. state that good qualitative agreement was found with Motyka and Willis (1974), the results were opposite with Horikawa et al. and Motyka and Willis predicting accretion and erosion shoreward of the pit, respectively. The proposed reason for the accretion given in Horikawa et al. was that sand accumulates behind the pit due to the "quiet" water caused by the decrease in wave action behind the pit. However, a model that considers only refraction caused by a pit and only includes a transport term dependent on the breaking wave angle would have wave rays that diverge over the pit and cause sand to be transported away from the area behind the pit, resulting in erosion. Regardless of the differing results from Motyka and Willis, the mathematical model results of Horikawa et al. follow the trend of the laboratory results contained in that study showing accretion behind a pit (Fig. 22); however, the aforementioned anomalous prediction of accretion considering only wave refraction remains.

5.3. Refraction and diffraction models

5.3.1. Gravens and Rosati (1994)

Gravens and Rosati (1994) performed a numerical study of the salients and a set of offshore breakwaters at Grand Isle, Louisiana (Figs. 1 and 2). Of particular interest is the analysis and interpretation of the impact on the wave field and the resulting influence on the shoreline of the "dumbbell" shaped planform borrow area located close to shore. The report employs two numerical models to determine the change in the shoreline caused by the presence of the offshore pits: a wave transformation numerical model (RCPWAVE) and a shoreline change model (GENESIS (Hanson,

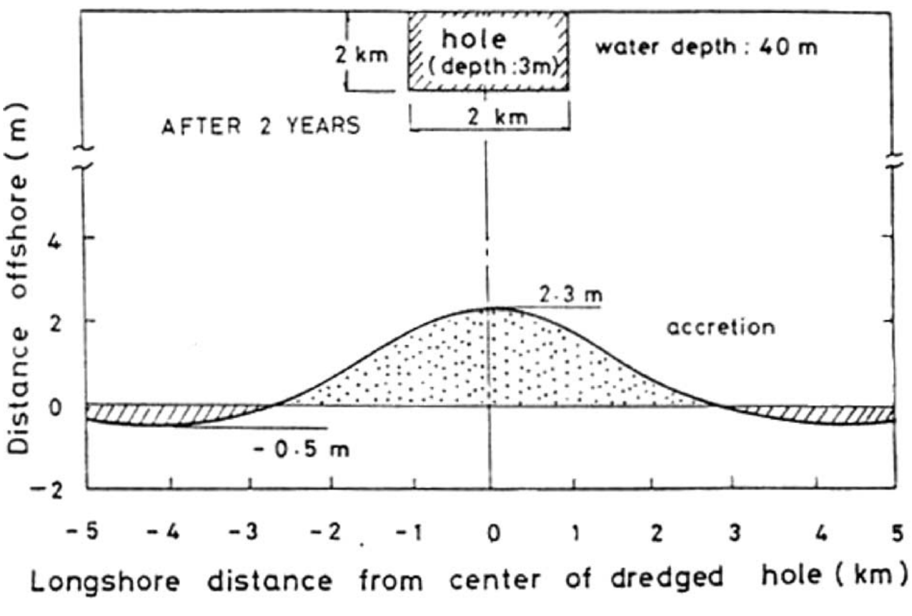

Fig. 21. Calculated beach planform due to refraction over dredged hole after 2 years of prototype waves (Horikawa et al., 1977).

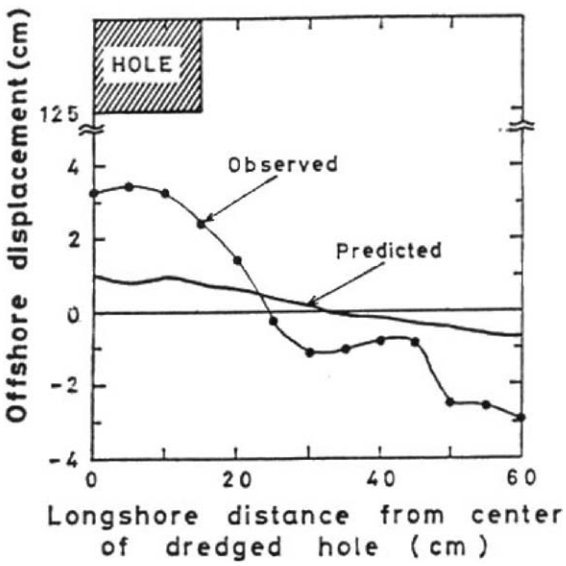

Fig. 22. Comparison of changes in beach plan shape for laboratory experiment and numerical model after 2 years of prototype waves (Horikawa et al., 1977).

1987, 1989)) using the wave heights from the wave transformation model. RCPWAVE was used to calculate the wave heights and directions from the nominal 12.8-m contour to the nominal 4.3-m contour along the entire length of the island for three different input conditions. Figs. 23 and 24 present the wave height transformation coefficients (K) and wave angles (θ) near the pit (centered about alongshore coordinate 130). Significant changes in wave height and direction are found near the offshore borrow area. The shadow zone centered at Cell 130 suggests the presence of one large offshore pit as compared to the "dumbbell" shaped borrow pit for the project described in Combe and Soileau (1987).

The shoreline changes were calculated using a longshore transport equation with two terms; one driven by the breaking wave angle and one driven by the longshore gradient in the breaking wave height. Each of these terms includes a dimensionless transport coefficient. In order for GENESIS to produce a salient leeward of the borrow pit, an unrealistically large value of the transport coefficient associated with the gradient

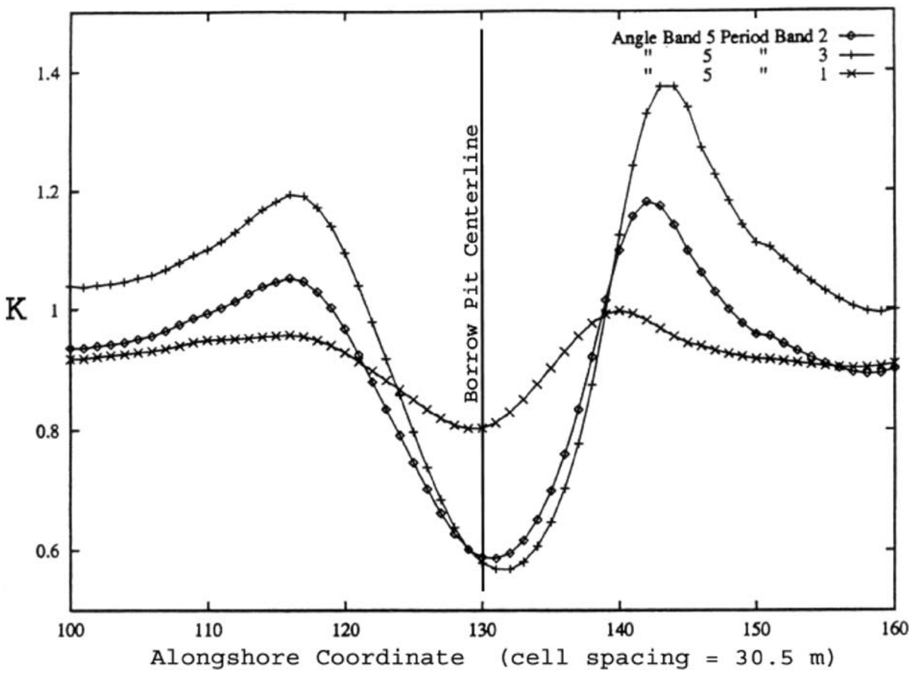

Fig. 23. Nearshore wave height transformation coefficients near borrow pit from RCPWAVE study (modified from Gravens and Rosati, 1994).

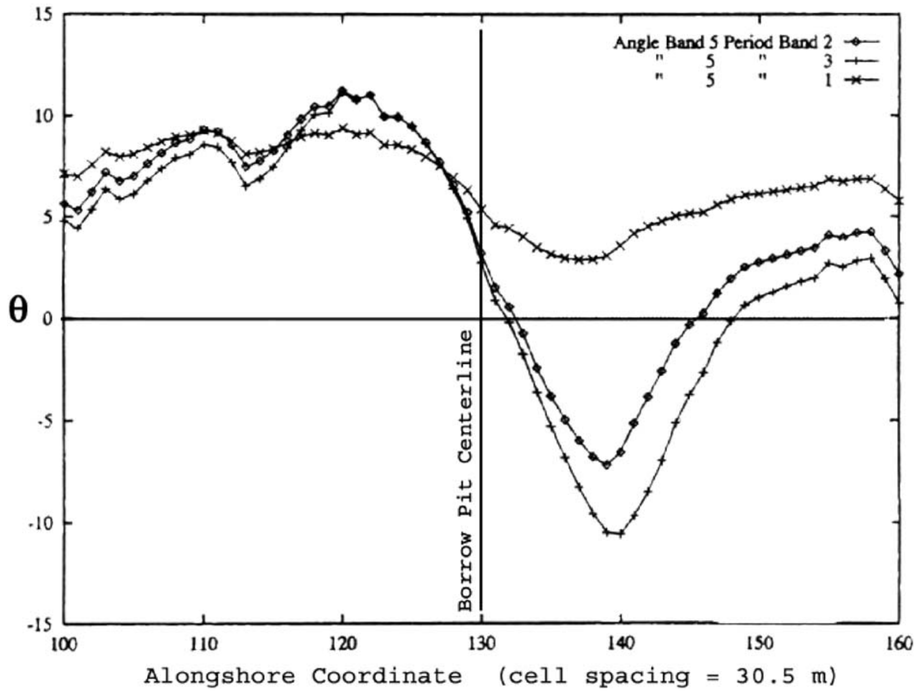

Fig. 24. Nearshore wave angles near borrow pit from RCPWAVE study; wave angles are relative to shore normal and are positive for westerly transport (modified from Gravens and Rosati, 1994).

in the breaking wave height ($K_2 = 2.4$) was needed, whereas 0.77 is the normal upper limit. While a single salient was modeled after applying the large K_2 value, the development of two salients leeward of the borrow pit, as shown in Figs. 1 and 2, did not occur. The nearshore bathymetry data used in the modeling were based on surveys taken in 1990 and 1992. Significant infilling of the borrow pit occurred prior to the surveys in 1990 and 1992; however, details of how the pit filled over this time period are not known nor was the character of the infilling material documented.

The authors proposed that the salient was formed by the refractive divergence of the wave field created by the borrow pit that resulted in a region of low energy directly shoreward of the borrow area and regions of increased energy bordering the area. The gradient in wave energy will result in a circulation pattern where sediment suspended in the high-energy zone is carried into the low energy zone. For GENESIS to recreate this circulation pattern, $K2$ must be large enough to allow the second transport term to dominate over the first transport term.

5.3.2. *Tang (2002)*

Tang (2002) employed RCPWAVE and a shoreline modeling program to evaluate the shoreline evolution leeward of offshore pits including the Grand Isle, LA pit geometry and a range of idealized pit geometries. The modeling was only able to generate embayments in the lee of the offshore pits using accepted values for the transport coefficients. This indicates that wave reflection and/or dissipation is important wave transformation processes that must be included when modeling shoreline evolution in areas with bathymetric anomalies.

5.4. *Refraction, diffraction, and reflection models*

5.4.1. *(Bender, 2001)*

A study by (Bender, 2001) extended the numerical solution of Williams (1990) for the transformation of long waves by a pit to determine the energy reflection and shoreline changes caused by offshore pits and shoals. An analytic solution was also developed for the radially symmetric case of a pit following the method of Black and Mei (1970). The processes of wave refraction, wave diffraction, and wave reflection are included in the model formulations; however, wave dissipation is not. Both the numerical and analytic solutions provide values of the complex velocity potential at any point, which allows determination of quantities such as velocity and pressure at any location in the field of interest.

The amount of reflected energy was calculated by comparing the energy flux through a transect perpendicular to the incident wave field extending to the pit center to the energy flux through the same transect with no pit present (see Fig. 25 for reflection coefficients). The amount of energy reflected was found to be significant and dependent on the dimensionless pit diameter and other parameters. Subsequently, a new method has been developed which allows the reflected energy to be calculated using a far-field approximation with good agreement between the two methods.

The shoreline changes caused by the pit were calculated using a simple model that considers continuity principles and the two-term longshore transport equation with values of the wave height and direction determined along a transect representing the shoreline. A nearshore slope and no nearshore refraction were assumed. The impact on the shoreline was modeled by determining the wave heights and directions along an initially straight shoreline, then calculating the transport and resulting shoreline changes. After updating the shoreline positions, the transport, resulting shoreline changes, and updated shoreline positions were recalculated for a set number of iterations after which the wave transformation was recalculated with the new bathymetry and values of the wave height and direction were updated at the modified breaker line.

The impact on the shoreline was found to be highly dependent on the transport coefficients. Considering transport driven only by the breaking wave angle and wave height, erosion was found to occur directly leeward of the pit flanked by two areas of accretion as in Motyka and Willis (1974). Following an initial advancement directly shoreward of the pit, erosion occurs and an equilibrium shape was reached. Examining only the effect of the second transport term (driven by the longshore gradient in the wave height), accretion was found directly shoreward of the pit, with no equilibrium planform achieved, i.e., the shoreline continued evolving without limit. Including both

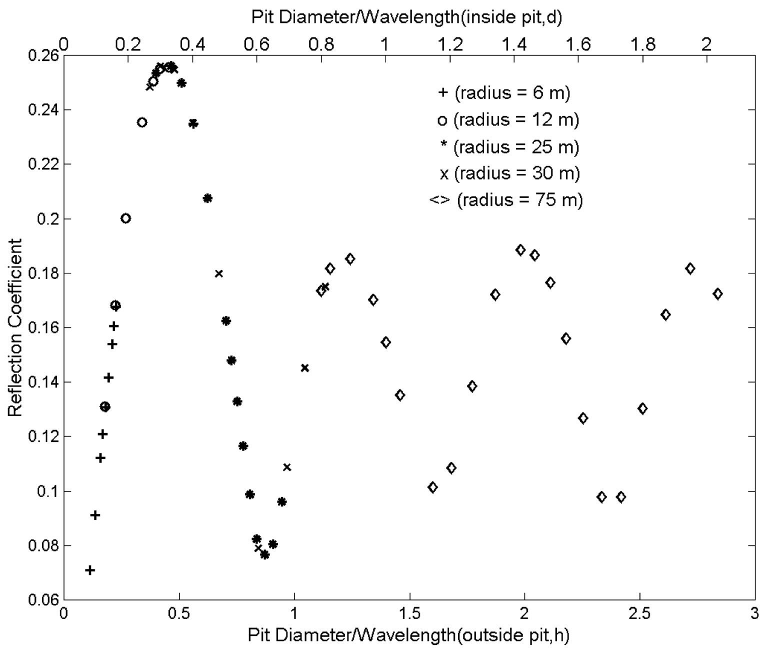

Fig. 25. Reflection coefficients versus dimensionless pit diameter divided by wave length inside and outside the pit; $h = 2$ m, $d = 4$ m (Bender, 2001).

transport terms with the same transport coefficients resulted in a shoreline with accretion directly shoreward of the pit that was able to reach an equilibrium state. These results are contained in Bender and Dean (2001).

The following two-term transport equation was used to determine the shoreline evolution:

$$Q = \frac{K_1 H_b^{2.5}\sqrt{g/\kappa}\sin(\alpha_b)\cos(\alpha_b)}{8(s-1)(1-p)} - \frac{K_2 H_b^{2.5}\sqrt{g/\kappa}\cos(\alpha_b)}{8(s-1)(1-p)\tan(m)}\frac{\mathrm{d}H_b}{\mathrm{d}y} \tag{2}$$

where κ is the breaking index, m is the beach slope, s and p are the specific gravity and porosity of the sediment, respectively, and K_1 and K_2 are sediment transport coefficients, which were set equal to 0.77 for the results presented here. The variables H_b and α_b were defined previously as conditions at breaking; however, for the model of Bender and Dean (2001), near breaking conditions were employed. The first transport term is driven by the waves approaching the shore at an angle and is equivalent to the CERC transport equation (Shore Protection Manual, 1984), while the second term is based on the form of Ozasa and Brampton (1979).

In these calculations, the water depth and pit depth are 2 and 4 m, respectively, the period was 10 s, the incident wave height is 1 m, and averaging over five wave directions was used to smooth the longshore variation in the wave height at large distances from the pit. The time step was 120 s and 10 iterations of shoreline change were calculated between wave height and direction updates for a total modeling time of 48 h. The diffusive nature of the angle-driven transport term is seen to modify the much larger wave height gradient transport term in order to generate an equilibrium planform when the two terms are used together. Comparison of these results with those described earlier establishes the significance of wave reflection and the second transport term.

6. Summary and conclusions

Recent interest in extracting large volumes of nearshore sediment for beach nourishment and construction purposes has increased the need for reliable predictions of wave transformation and associated shoreline changes caused by such modifications. This predictive capacity would assist the designer of such projects in minimizing undesirable shoreline changes.

The available laboratory and field data suggest that the effect of wave transformation by an offshore pit is complex and can result in substantial shoreward salients. Of the four wave transformation processes, a significant number of wave models include effects of wave refraction and diffraction; however, fewer incorporate wave reflection and/or dissipation over a soft medium in the pit. Computational results incorporating only refraction and diffraction and accepted values of sediment transport coefficients appear incapable of predicting the observed salients landward of borrow pits. Therefore, improved capabilities to predict wave transformation and shoreline response to constructed borrow pits will require improvements in both: (1) wave modeling, particularly in representing wave reflection and dissipation, and (2) longshore sediment transport by the wave angle and wave height gradient terms. Finally, additional laboratory tests with careful near field wave measurements and monitoring of field projects are necessary for future understanding of this significant problem.

Acknowledgements

An Alumni Fellowship granted by the University of Florida sponsored this study with partial support from the Bureau of Beaches and Wetland Resources of the State of Florida.

References

Abernethy, C.L., Gilbert, G., 1975. Refraction of a wave spectrum. Report No. INT 117, Hydraulics Research Station, Wallingford, England.

Applied Technology and Management, 1998. Martin County beach nourishment project: Project performance report. Gainesville, Florida.

Bakker, W.T., 1971. The influence of longshore variation of the wave height on the littoral current. Study Report WWK71-19. Ministry of Public Works (Rijkswaterstaat), The Hague, The Netherlands.

Bartholomeusz, E.F., 1958. The reflexion of long waves at a step. Proceedings of the Cambridge Philological Society 54, 106–118.

Battjes, J.A., Janssen, J.P.F.M., 1978. Energy loss and set-up due to breaking of random waves. Proc. 16th Int. Conf. Coastal Engineering. ASCE, pp. 569–587.

Bender, C.J., 2001. Wave field modifications and shoreline response due to offshore borrow areas. Masters Thesis, Department of Civil and Coastal Engineering, University of Florida.

Bender, C.J., Dean, R.G., 2001. Erosional hot spot prediction through wave analysis. Proc. 4th Int. Symposium on Ocean Wave Measurement and Analysis (WAVES '01), ASCE, II, pp. 1306–1315.

Berkhoff, J.C.W., 1972. Computation of combined refraction–diffraction. Proc. 13th Int. Conf. Coastal Engineering. ASCE, pp. 471–490.

Berkhoff, J.C.W., Booij, N., Radder, A.C., 1982. Verification of numerical wave propagation models for simple harmonic linear water waves. Coastal Engineering 6, 255–279.

Black, J.L., Mei, C.C., 1970. Scattering and radiation of water waves. Technical Report No. 121, Water Resources and Hydrodynamics Laboratory, Massachusetts Institute of Technology.

Black, J.L., Mei, C.C., Bray, M.C.G., 1971. Radiation and scattering of water waves by rigid bodies. Journal of Fluid Mechanics 46, 151–164.

Cavaleri, L., Malanotte-Rizzoli, P., 1981. Wind wave prediction in shallow water: theory and applications. Journal of Geophysical Research 86 (C11), 10961–10973.

Collins, J.I., 1972. Prediction of shallow water spectra. Journal of Geophysical Research 77 (15), 2693–2707.

Combe, A.J., 2002. Personal Communication, April, 2002.

Combe, A.J., Soileau, C.W., 1987. Behavior of man-made beach and dune: Grand Isle, Louisiana. Coastal Sediments '87. ASCE, pp. 1232–1242.

Copeland, G.J.M., 1985. A practical alternative to the "mild-slope" wave equation. Coastal Engineering 9, 125–149.

Dally, W.R., Dean, R.G., Dalrymple, R.A., 1985. Wave height variations across beaches of arbitrary profile. Journal of Geophysical Research 90, 11917–11927.

Dalrymple, R.A., Kirby, J.T., Hwang, P.A., 1984. Wave diffraction due to areas of energy dissipation. Journal of Waterway, Port, Coastal and Ocean Engineering, ASCE 110, 67–79.

Danish Hydraulic Institute (DHI), 1998. MIKE 21: Elliptic mild slope (EMS) wave module, Release 2.7, User Guide and Reference Manual.

Dean, R.G., 1964. Long wave modification by linear transitions. Journal of the Waterways and Harbors Division, ASCE 90 (1), 1–29.

Dean, R.G., 2002. Beach Nourishment: Theory and Practice. World Scientific, Singapore. 399 pp.

Dean, R.G., Liotta, R., Simon, G., 1999. Erosional hot spots. UFL/COEL-99/021, Department of Coastal and Oceanographic Engineering, University of Florida.

Dingemans, M.W., 1983. Verification of numerical wave equation models with field measurements. CREDIZ verification Haringvliet, Report W488, Delft Hydraulics Lab, Delft, p. 137.

Ebersole, B.A., Cialone, M.A., Prater, M.D., 1986. Regional coastal processes numerical modeling system, Report 1, A linear wave propagation model for engineering use. Technical Report CERC-86-4, CERC, Department of the Army, WES, Corps of Engineers, Vicksburg, Mississippi.

Gourlay, M.R., 1982. Nonuniform alongshore currents and sediment transport—A one-dimensional approach. Civil Engineering Research Report No. CD31, Department of Civil Engineering, University of Queensland, St. Lucia, Queensland.

Grassa, J.M., Flores, J., 2001. Discussion of: Maa, J.P.-Y., Hsu, T.-W., Tsai, C.-H., Juang, W.J., 2000. Comparison of wave refraction and diffraction models. Journal of Coastal Research, 16 (4): 1073–1082. Journal of Coastal Research, 17 (3): 762–764

Gravens, M.B., Rosati, J.D., 1994. Numerical study of breakwaters at Grand Isle, Louisiana, Miscellaneous Paper CERC-94-16, U.S. Army Corps of Engineers, Vicksburg, MS.

Hanson, H., 1987. GENESIS—a generalized shoreline change model for engineering use. Report 1007. Department of Water Resources Engineering, University of Lund, Lund, Sweden.

Hanson, H., 1989. Genesis—a generalized shoreline change numerical model. Journal of Coastal Research 5 (1), 1–27.

Hasselmann, K., Barnett, T.P., Bouws, E., Carlson, H., Cartwright, D.E., Enke, K., Ewing, J.A., Gienapp, H., Hasselmann, D.E., Kruseman, P., Meerburg, A., Müller, P., Olbers, D.J., Richter, K., Sell, W., Walden, H., 1973. Measurements of wind–wave growth and swell decay during the Joint North Sea Wave Project (JONSWAP). Deutsche Hydrographische Zeitschrift. Supplement 12, A8.

Holthuijsen, L.H., Booij, N., Ris, R.C., Haagsma, IJ.G., Kieftenburg, A.T.M.M., Kriezi, E.E., 2000. SWAN User Manual (not the short version), SWAN Cycle III, Version 40.11. Department of Civil Engineering, Delft University of Technology, GA Delft, The Netherlands.

Horikawa, K., Sasaki, T., Sakuramoto, H., 1977. Mathematical and laboratory models of shoreline change due to dredged holes. Journal of the Faculty of Engineering XXXIV (1), 49–57 (The University of Tokyo).

Hsu, T.W., Wen, C.C., 2001. A parabolic equation extended to account for rapidly varying topography. Ocean Engineering 28 (11), 1479–1498.

Inman, D.L., Rusnak, G.S., 1956. Changes in sand level on the beach and shelf at La Jolla, California. Technical Memorandum No 82, U.S. Army Corps of Engineers, Beach Erosion Board.

Janssen, P.A.E.M., 1989. Wave induced stress and the drag of air flow over sea waves. Journal of Physical Oceanography 19, 745–754.

Janssen, P.A.E.M., 1991. Quasi-linear theory of wind–wave generation applied to wave forecasting. Journal of Physical Oceanography 21, 1631–1642.

Jolas, P., 1960. Passage de la houle sur un seuil. Houle Blanche 15, 148.

Kirby, J.T., 1986. Higher-order approximations in the parabolic equation method for water waves. Journal of Geophysical Research 91, 933–952.

Kirby, J.T., Dalrymple, R.A., 1983a. Propagation of obliquely incident waves over a trench. Journal of Fluid Mechanics 133, 47–63.

Kirby, J.T., Dalrymple, R.A., 1983b. The propagation of weakly nonlinear waves in the presence of varying depth and currents.

Proceedings 20th Congress International Association for Hydraulic Research (IAHR), Moscow.

Kirby, J.T., Dalrymple, R.A., 1994. User's manual, combined refraction/diffraction model, REF/DIF-1, Version 2.5. Center for Applied Coastal Research, Department of Civil Engineering, University of Delaware.

Kirby, J.T., Dalrymple, R.A., Seo, S.N., 1987. Propagation of obliquely incident waves over a trench. Part 2. Current flowing along the trench. Journal of Fluid Mechanics 176, 95–116.

Kojima, H., Ijima, T., Nakamuta, T., 1986. Impact of offshore dredging on beaches along the Genkai Sea, Japan. Proc. 20th Int. Conf. Coastal Engineering. ASCE, pp. 1281–1295.

Komar, P.D., 1969. The longshore transport of sand on beaches. PhD thesis, University of California, San Diego.

Komen, G.J., Hasselmann, S., Hasselmann, K., 1984. On the existence of a fully developed wind–sea spectrum. Journal of Physical Oceanography 14, 1271–1285.

Komen, G.J., Cavaleri, L., Donelan, M., Hasselmann, K., Hasselmann, S., Janssen, P.A.E.M., 1994. Dynamics and Modeling of Ocean Waves. Cambridge Univ. Press. 532 pp.

Kraus, N.C., 1983. Application of a shoreline prediction model. Proc. of Coastal Structures '83. ASCE, pp. 632–645.

Kraus, N.C., Harikai, S., 1983. Numerical model of the shoreline change at Oarai Beach. Coastal Engineering 7, 1–28.

Kreisel, G., 1949. Surface Waves. Quarterly of Applied Mathemathics 7, 21–44.

Lamb, H., 1932. Hydrodynamics. Dover, Section 176.

Lassiter, J.B., 1972. The propagation of water waves over sediment pockets. PhD Thesis, Massachusetts Institute of Technology.

Lee, J.J., Ayer, R.M., 1981. Wave propagation over a rectangular trench. Journal of Fluid Mechanics 110, 335–347.

Lee, J.J., Ayer, R.M., Chiang, W.-L., 1981. Interactions of waves with submarine trenches. Proc. 17th Int. Conf. Coastal Engineering. ASCE, pp. 812–822.

Liotta, R., 1999. Erosional hot spots: Causes and case studies at Dade and Manatee Counties. Masters Thesis, Department of Coastal and Oceanographic Engineering, University of Florida.

Louisiana Oil Spill Coordinator's Office (LOSCO), 1999. Color Infrared Orthophoto, Date of Source Image Aerial Photography, 24-Jan-1998.

Maa, J.P.-Y., Hobbs III, C.H., 1998. Physical impacts of waves on adjacent coasts resulting from dredging at Sandbridge Shoal, Virginia. Journal of Coastal Research 14 (2), 525–536.

Maa, J.P.-Y., Hwung, H.-H., 1997. A wave transformation model for harbor planning. Proc. 3rd Int. Symposium on Ocean Wave Measurement and Analysis (WAVES '97), ASCE, I, pp. 256–270.

Maa, J.P.-Y., Hsu, T.-W., Hwung, H.-H., 1998a. RDE Model: a program for simulating water wave transformation for harbor planning. Special Scientific Report No. 136, Virginia Institute of Marine Science, Gloucester Point, Virginia.

Maa, J.P.-Y., Hwung, H.-H., Hsu, T.-W., 1998b. A simple wave transformation model, PBCG, for harbor planning. Proc. 3rd Int. Conf. on Hydrodynamics, vol. I, pp. 407–412.

Maa, J.P.-Y., Hsu, T.-W., Tsai, C.H., Juang, W.J., 2000. Comparison of wave refraction and diffraction models. Journal of Coastal Research 16 (4), 1073–1082.

Maa, J.P.-Y., Hobbs III, C.H., Hardaway Jr., C.S., 2001. A criterion for determining the impact on shorelines caused by altering wave transformation. Journal of Coastal Research 17 (1), 107–113.

Madsen, P.A., Larsen, J., 1987. An efficient finite-difference approach to the mild-slope equation. Coastal Engineering 11, 329–351.

Madsen, O.S., Poon, Y.-K., Graber, H.C., 1988. Spectral wave attenuation by bottom friction: Theory. Proc. 21st Int. Conf. Coastal Engineering. ASCE, pp. 492–504.

McDougal, W.G., Williams, A.N., Furukawa, K., 1996. Multiple pit breakwaters. Journal of Waterway, Port, Coastal, and Ocean Engineering, ASCE 122, 27–33.

Mei, C.C., Black, J.L., 1969. Scattering of surface waves by rectangular obstacles in water of finite depth. Journal of Fluid Mechanics 38 (3), 499–511.

Miche, M., 1951. Le pouvoir reflechissant des ouvrages maritimes exposes a l' action de la houle. Annals des Ponts et Chaussess. 121e Annee, pp. 285–319 (translated by Lincoln and Chevron, University of California, Berkeley, Wave Research Laboratory, Series 3, Issue 363, June 1954).

Miles, J.W., 1967. Surface-wave scattering matrix for a shelf. Journal of Fluid Mechanics 28, 755–767.

Miles, J.W., 1982. On surface-wave diffraction by a trench. Journal of Fluid Mechanics 115, 315–325.

Minerals Management Service, 2002. [On-line]. Environmental Surveys of Potential Borrow Areas on the East Florida Shelf. Available: http://www.mms.gov/intermar/florida.html.

Motyka, J.M., Willis, D.H., 1974. The effect of wave refraction over dredged holes. Proc. 14th Int. Conf. Coastal Engineering. ASCE, pp. 615–625.

Newman, J.N., 1965a. Propagation of water waves over an infinite step. Journal of Fluid Mechanics 23, 23–29.

Newman, J.N., 1965b. Propagation of water waves past long two-dimensional objects. Journal of Fluid Mechanics 23, 399–415.

Ozasa, H., Brampton, A.H., 1979. Models for predicting the shoreline evolution of beaches backed by seawalls. Report No. IT 191, Hydraulics Research Station: Wallingford, England.

Price, W.A., Motyka, J.M., Jaffrey, L.J., 1978. The effect of offshore dredging on coastlines. Proc. 16th Int. Conf. Coastal Engineering, ASCE, 2, pp. 1347–1358.

Radder, A.C., 1979. On the parabolic equation method for water-wave propagation. Journal of Fluid Mechanics 95, 159–176.

Resio, D.T., 1987. Shallow-water waves. I: Theory. Journal of Waterway, Port, Coastal, and Ocean Engineering 113 (3), 264–281.

Resio, D.T., 1988a. Shallow-water waves. II: Data comparisons. Journal of Waterway, Port, Coastal, and Ocean Engineering 114 (1), 50–65.

Resio, D.T., 1988b. A steady-state wave model for coastal applications. Proc. 21st Int. Conf. Coastal Engineering. ASCE, pp. 929–940.

Sasaki, T., 1975. Simulation on shoreline and nearshore current. Proc. Specialty Conf. on Civil Engineering in the Oceans, vol. III, pp. 179–196.

Schwinger, J., Saxon, D.S., 1968. Discontinuities in Waveguides. Gordon and Breach.

Shore Protection Manual, 1984. U.S. Army Corps of Engineers, CERC, U.S. Government Printing Office, Washington D.C.

Smith, J.M., Sherlock, A.R., Resio, D.T., 2001. STWAVE: Steady-state spectral wave model, User's manual for STWAVE, Version 3.0. Final Report ERDC/CHL SR-01-1, U.S. Army Engineer Research and Development Center, Vicksburg, Mississippi.

Snyder, R.L., Dobson, F.W., Elliot, J.A., Long, R.B., 1981. Array measurements of atmospheric pressure fluctuations above surface gravity waves. Journal of Fluid Mechanics 102, 1–59.

Sretenskii, L.N., 1950. Refraction and reflexion of plane waves in liquids at a transition from one depth to another (in Russian). Izvestiâ Akademii Nauk SSSR. Otdelenie Tehničeskih Nauk, 1601.

Sumerell, H.R., 2000. Martin County, Florida beach nourishment project: Evaluation of project performance and prediction capabilities. Masters Thesis, Department Civil and Coastal Engineering, University of Florida.

Takano, K., 1960. Effets d'un obstacle parallélélépipédique sur la propagation de la houle. Houille Blanche 15, 247.

Tang, Y., 2002. The effect of offshore dredge pits on adjacent shorelines. Masters Thesis, Department of Civil and Coastal Engineering, University of Florida.

Wang, Z., Dean, R.G., 2001. Manatee County beach nourishment project performance and erosional hot spot analyses. UFL/COEL-2001/003, University of Florida.

Wehausen, J.V., Laitone, E.V., 1960. Handbuch der Physik, vol. 9. Springer, Berlin.

Williams, A.N., 1990. Diffraction of long waves by a rectangular pit. Journal of Waterway, Port, Coastal, and Ocean Engineering 116, 459–469.

Williams, A.N., Vazquez, J.H., 1991. Wave interaction with a rectangular pit. Journal of Offshore Mechanics and Arctic Engineering 113, 193–198.

Williams, B.P., 2002. Physical modeling of nearshore response to offshore borrow pits. Masters Thesis, Department of Civil and Coastal Engineering, University of Florida.