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Physical Impact of Waves on Adjacent Coasts Resulting from Dredging at Sandbridge Shoal, Virginia

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ABSTRACT

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The possible changes of breaking wave heights, breaking wave angles, and associated longshore sediment transport along a resort strip from Virginia Beach to Sandbridge, Virginia, caused by a possible dredging at a shoal about 5 km offshore, were studied. This preliminary study revealed that the possible physical impact of waves on adjacent coasts resulting from the modeled dredging at Sandbridge Shoal, Virginia, is insignificant. The purpose of this dredging is to provide about 1.5×10^6 m^3 of beach-quality sand for beach nourishment. Because the minimum water depth at the shoal is about 9 m and the ambient water depth varies from 12 to 15 m, only large waves with long periods would be affected. In this study, three wave conditions (1. wave height, H = 1.9 m and wave period, T = 11.8 sec; 2. H = 3 m and T = 14 sec; and 3. H = 6.2 m, T = 20 sec) with six possible directions were studied to check the change of breaking wave conditions as well as the longshore-sediment-transport rates. The calculated changes of breaking wave height along this coast vary from 2 to 7% which is about the same as the accuracy of the wave measurement system, 5%. Accordingly, the possible changes of longshore sediment transport rates would be altered slightly. This study also demonstrates a significant wave energy convergence near Sandbridge for long period waves coming from NE. This natural phenomenon may be responsible for the severe beach erosion in the vicinity of Sandbridge.

ADDITIONAL INDEX WORDS: *ocean dredging, environmental impact, wave energy, control erosion, beach erosion, sand transport.*

INTRODUCTION

The recent resurgence of interest in beach nourishment (PILKEY, 1990; LEONARD *et al.* 1990; FINKL, 1993, 1996; HOUSTON, 1995; NATIONAL RESEARCH COUNCIL, 1995) and the particular needs of the City of Virginia Beach, Virginia (Figure 1) have paralleled on ongoing series of studies along the southeastern coast of Virginia (HARDAWAY *et al.,* 1995). Both the city's "Resort Strip" and the ocean-side, semi-private community of Sandbridge (Figure 1) potentially are targeted for beach nourishment and hurricane protection projects. The U.S. Navy is also planning a major nourishment project at its facility immediately north of Sandbridge.

Ongoing maintenance nourishment at the Resort Strip has relied upon sand from upland borrow pits and a large stockpile of dredged material. The relatively nearby borrow pits, however, have been exhausted or have closed leaving only borrow pits that are too far away, approximately 22 road km, for economically feasible trucking. The stockpile of dredged material is irregularly replenished and is of insufficient volume to meet existing and proposed demands. All the above statements point to the need for finding new sources of beach-quality sand.

Sandbridge Shoal, which is less than 5 km (3 n mi) offshore, provides a relatively easily accessible sand resource. The shoal might contain as much as 30×10^6 m^3 (40×10^6 yd^3) of beach-quality sand (KIMBALL and DAME, 1989; DAME, 1990; KIMBALL *et al.,* 1991; HOBBS, 1996) with very little overburden of unusable material. Although recent work by the Corps of Engineers (SWEAN, personal communication) indicates that the total quantity of sediment is somewhat less, Sandbridge Shoal is still an appealing source of beach-quality sand.

There is understandable concern that if a substantial quantity of material were removed from Sandbridge Shoal that the bathymetric alterations caused by that dredging would alter the wave transformation pattern across the shoal and possibly result in detrimental consequences on the nearby, developed shore. We undertook this study to address those concerns and to further the general understanding of the region's physical environment.

To understand the possible changes along the shoreline due to dredging at the shoal requires cognizance of the regional wave climate, the wave-transformation process, and the associated responses of the shoreline. Earlier studies of wave refraction and diffraction (*e.g.* BERKHOFF *et al.,* 1982) showed that a shoal may function as a convex lens that converges waves energy and may cause more beach erosion immediately inshore of the shoal. The degree of convergence depends on the size, shape, and location of the shoal as well as on the local wave conditions.

The following sections present the studies of wave climate off the Virginia coast, the transformation of waves when propagating toward Sandbridge, including variations caused

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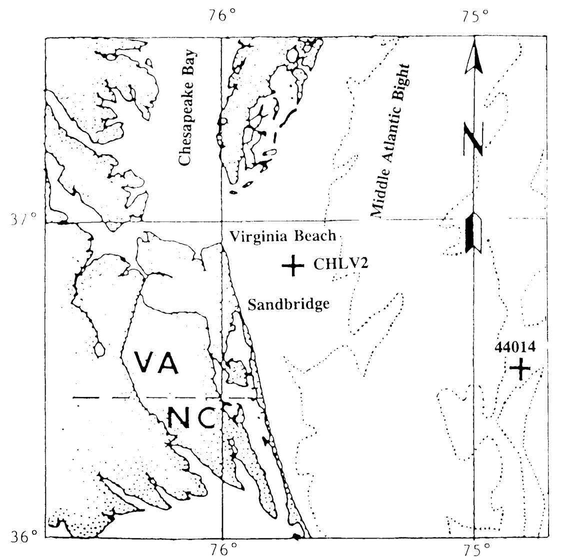

Figure 1. Location map of the study area and wave stations.

by the modeled dredging, and the change of wave-driven long-shore sediment transport in that section of the Virginia coast.

WAVE STATISTICS

The National Data Buoy Center (NDBC) has two wave stations off the Virginia coast (Figure 1) (WANG and METTLACH, 1992): A moored buoy station, 44014, located near the continental shelf break (36°34′59″ N, 74°50′01″ W, water depth = 48 m), and a Coastal-Marine Automated Network (C-MAN) station, CHLV2, located on a shoal (36°54′18″ N, 75°42′48″ W, water depth = 12 m) approximately 25 km east of the Chesapeake Bay mouth. Around the shoal, the ambient water depth is about 20 m.

The wave-measurement system at station 44014 used an accelerometer to record the buoy's heave, pitch, and roll. A NDBC on-board Wave Data Analyzer computed the wave spectral information from the time series of buoy motion and transmitted the results to the Stennis Space Center in Mississippi for further analysis and quality assurance. This station, which started operation in 1991, also provided wave directional information by using the approach proposed by LONGUET-HIGGINS *et al.* (1963).

Wave measurements at station CHLV2 were carried out with an Infrared Laser Wave Height Sensor which only measured the water surface displacement. Wave directional information is not available but the station has provided wave heights since 1985.

The overall accuracy of all systems for significant wave height, wave period, and wave direction is 0.2 m (or 5%), 1.0 sec, and ±5°, respectively (MEINDL and HAMILTON, 1992). Details of the NDBC wave measurement system and data processing technique can be found in STEELE *et al.* (1990). All processed data are archived in National Oceanic Data Center (NODC) in Washington, D.C. using a special ASCII format and were distributed in CD-ROM. Computer software was developed at VIMS to read these data and store the necessary parts separated for later uses.

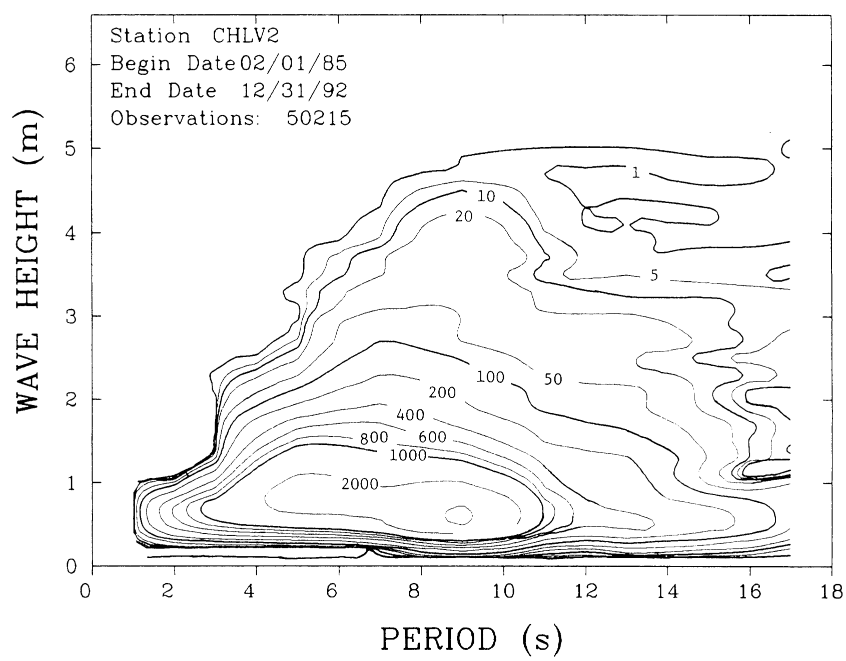

Figure 2. Joint distribution of significant wave height and peak energy wave period at station CHLV2. Contours are in number of occurrence.

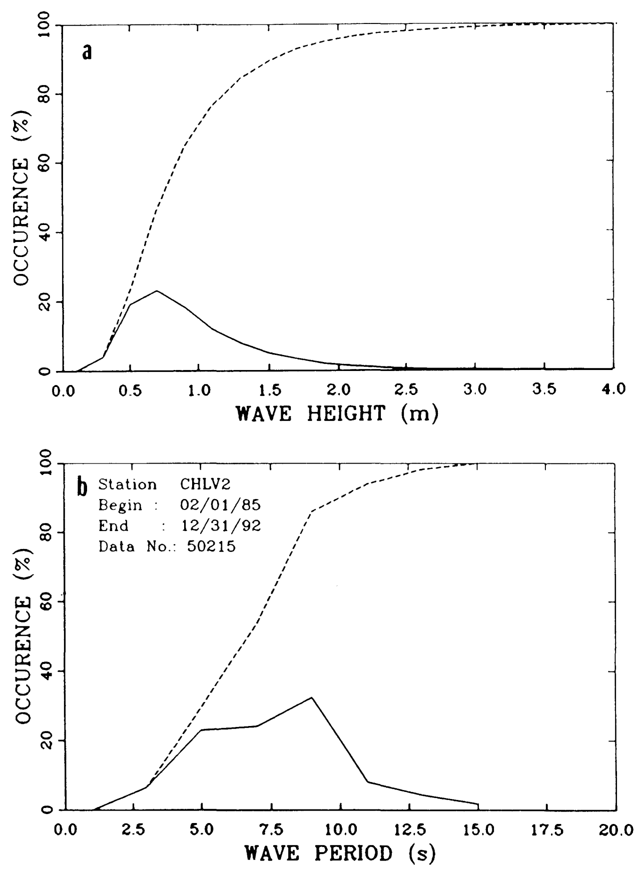

Figure 3. Percentage of wave occurrence at Station CHLV2. (a) Wave period of peak energy; (b) Significant wave height. Solid lines are the frequency distributions and dashed lines are cumulative occurrence.

Joint Distribution of Wave Height and Period

Although we have two wave stations in the coastal area of Virginia, only the data from station CHLV2 were used for studying the joint distribution of significant wave height and peak energy period. This is because station 44014 has a much shorter record and is a greater distance from the coast. Only the wave direction information from station 44014 was used. Figure 2 shows the joint distribution at station CHLV2 for the period 1985–1992. The most frequently occurring wave has a height of 0.7 meter and period of 9 seconds. Notice that there are many swells (small wave heights with long wave period). To show the percentage distribution of recorded significant wave height and peak energy period, the data from Figure 2 were reorganized to show the relative abundance of each wave height and period (Figure 3). This diagram and other information, discussed below, were used to determine the design wave conditions.

Table 1. *Maximum significant wave height and wave peak period at station CHLV2.*

Date	Time	H_Significant (m)	T_Peak (sec)
9/27/85	10:00	6.2	20
12/02/86	21:00	4.2	10
3/10/87	15:00	4.5	10
2/19/88	20:00	3.3	5.6
2/24/89	22:00	4.9	12.5
10/26/90	17:00	4.0	10
11/10/91	03:00	4.6	10
1/04/92	11:00	4.9	14.3

Table 2. *Average duration for H_s > 2 m, and > 3 m.*

Year	Hours (percentage) that H_s ≥2 m	≥3 m
85	350 (4.0%)	34 (0.4%)
86	229 (2.6%)	55 (0.6%)
87	341 (3.9%)	73 (0.8%)
88	129 (1.5%)	4 (0.0%)
89	568 (6.5%)	166 (1.9%)
90	216 (2.5%)	20 (0.2%)
91	510 (5.8%)	118 (1.3%)
92	542 (6.2%)	150 (1.7%)
Average	361 (4.1%)	77 (0.9%)

Table 3. *Average duration for peak wave period >12 sec and 14 sec.*

Year	Hours (percentage) that T_peak ≥12 sec	≥14 sec
85	483 (5.5%)	262 (3.0%)
86	406 (4.6%)	173 (2.0%)
87	254 (2.9%)	14 (0.2%)
88	254 (2.9%)	89 (1.0%)
89	331 (3.8%)	114 (1.3%)
90	581 (6.6%)	184 (2.1%)
91	1,069 (12.2%)	116 (1.3%)
92	506 (5.8%)	195 (2.2%)
Average	485 (5.5%)	143 (1.5%)

Model Waves

The record length (7 years) at station CHLV2 is not long enough for an accurate estimation of the most severe hurricane wave condition. The maximum significant wave heights that occurred during each of the 7 years are given in Table 1. We selected the recorded maximum significant wave height (6.2 m with a peak wave period of 20 seconds, occurring on September 27, 1985) as the most severe sea.

Based on the measurements at station CHLV2, Table 2 shows the total hours and percentage in each year that the measured wave height exceeds 2 and 3 m. On average the wave height exceeds 2 m about 5% of the time in a year. Only

Table 4. *Selected model wave conditions.*

Wave Height (m)	Wave Period (sec)	Remark
6.2	20	Most severe sea
3.0	14	Severe sea
1.9	12	Northeaster
0.72	6.7	Fair weather wave

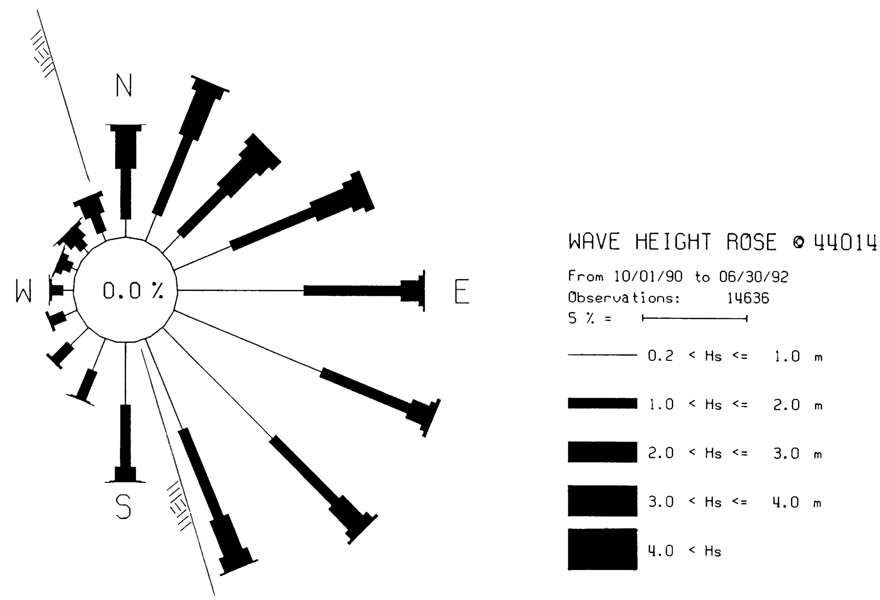

Figure 4. Wave height rose from station 44014. The Orientation of Coast line at Sandbridge is also plotted.

about 1% of the time in each year is the wave height more than 3 m. Table 3 indicates about 5% of the time in each year the peak wave period exceeds 12 sec and only about 1% of the time does it exceed 14 sec.

As indicated in Figure 3a, a significant wave height of 1.9 m exceeds 95% of the observations. This wave height was selected as the "northeaster storm wave." From Figure 3b, the corresponding wave period which also exceeds 95% of the observed wave periods is 11.8 sec. Similarly, a wave height and period of 3 m and 14 sec, which exceed 1% of the observations, was selected to represent the "severe sea." These values are very close to that given in Table 2 and 3.

These three wave conditions (the most severe sea, the severe sea, and the northeaster wave) were selected to check the possible impacts resulting from their transformation when passing across Sandbridge Shoal as modified by the modeled dredging. The selection is somewhat subjective, but it does include all possible large waves with longer wave periods.

Wave Direction

Although the recording period for directional information at station 44014 is not long, it is the best information we have. The wave-height rose (Figure 4) indicates that the directional distribution of wave height is relatively uniform from NNE to SSE, and the most common wave direction is ESE. Large waves, however, mainly come from NNE to ENE, likely caused by northeasters. The wave period rose (Figure 5) shows that long period waves are primarily coming from ENE and E because of the long fetch. Most of the waves from NNE and NE are less than 8 sec. Thus, waves coming from the ENE are most important because of the possible large wave heights and long wave periods. Waves coming from the SSE to ESE have varied wave heights, but their wave periods are rather short. Considering the water depth at Sandbridge Shoal is about 10 m, short period waves ($T < 11$ sec) will not be affected, and thus, they were ignored in this study.

Notice that the wave heights and periods at station 44014 are mainly from ESE with a large spread from N and S. Waves from other directions are negligible because of the limited fetch. When closer to the Virginia coast at station CHLV2, it can be expected that the majority of waves come from NNE to ESE. As indicated before, waves coming from ESE are mainly short period waves which cannot be affected by the dredging at Sandbridge Shoal. For this reason, we selected ENE as the main direction of threatening waves. The next two important directions are E and NE. At Sandbridge, the shore normal direction is 73 degrees clockwise from true north (N73°E). This direction is only 5.5 degrees from ENE. Considering the accuracy of wave direction measurements is ±5 degrees, there is almost no difference between ENE and N73°E. For this reason, ENE is considered as the shore normal direction as well as the main direction of threatening waves. The next two important wave directions are NE and E.

APPROACH

Wave rays tend to concentrate at the lee side of a shoal because of the wave refraction and diffraction processes. A typical example of these two processes can be found in BERKHOFF *et al.*'s (1982) laboratory experiment. The concentration of wave rays means wave energy is higher and may cause severe beach erosion if the shoal is close to the beach. Because of the size, shape, and location of Sandbridge Shoal, the response of wave transformation may not be as clear as that shown in BERKHOFF *et al.*'s (1982) experiment. We may assume, however, the dredging would reduce the wave con-

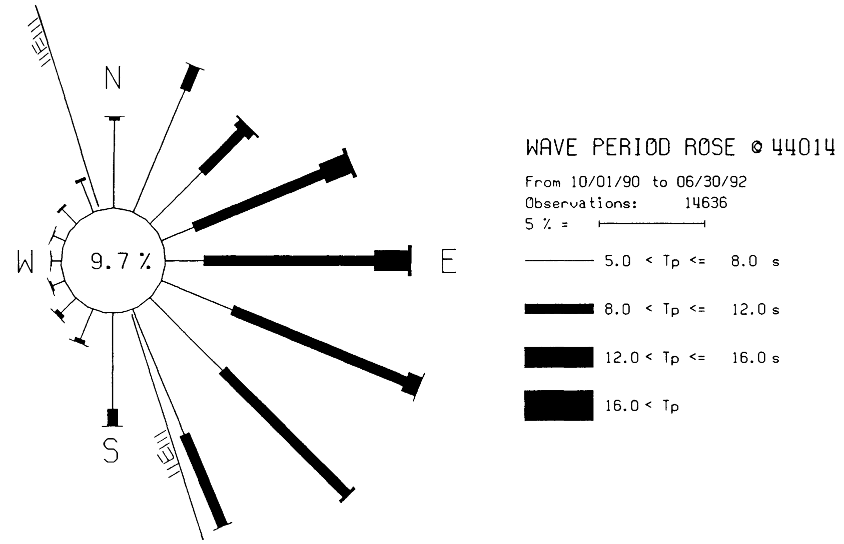

Figure 5. Wave period rose from station 44014. The Orientation of Coast line at Sandbridge is also plotted.

vergence because the shoal would be flattened. The actual responses, however, need to be studied carefully.

We began by obtaining the best available bathymetric data which then were used to generate the grid necessary for studying the wave-transformation processes. Second, we studied wave transformation processes (refraction, diffraction, shoaling, and energy dissipation caused by bottom friction) to generate the longshore profile of breaking wave height and wave breaking angle. Third, we examined longshore sediment transport using the calculated breaking wave conditions with the existing bathymetry. The above three steps depict the original beach responses at selected wave conditions. Then we assumed the dredging was completed, and the same exercises on wave transformation and sediment transport were repeated with the new bathymetry to determine the differences, if any. Notice that examining all possible wave conditions is not necessary because only a reasonably severe sea with a long wave period could be affected by the dredging. For this reason, only the northeaster waves, the severe sea, and the most severe sea were examined.

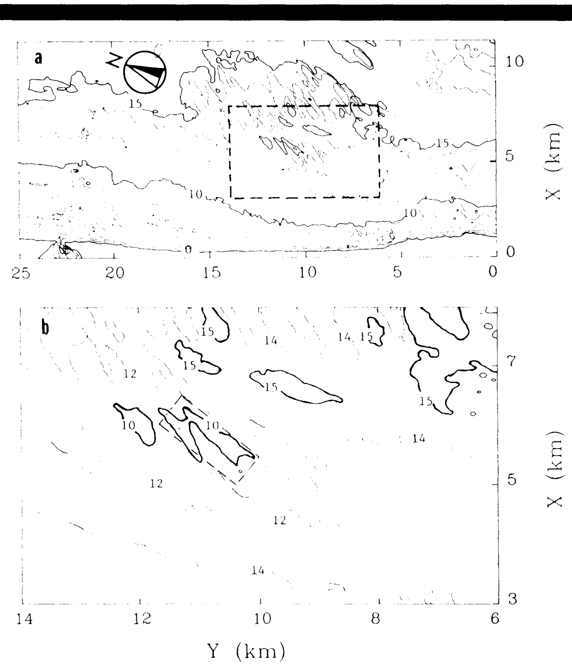

Figure 6. Bathymetry of the study area, water depth contours are in meters. The 0 contour is the shoreline. (a) The entire domain with the dashed rectangular area enlarged in Figure 6b; (b) the detailed bathymetry near the shoal. The dashed rectangular area is the modeled dredging area.

Bathymetric Data

Raw digital bathymetric data were obtained for this area from the NOAA Geophysical Data Center (Boulder, Colorado). After examining the data, we found several small areas for which digital data were not available. Fortunately, the original survey charts were available for digitizing. The new data were converted to the standard NOAA data format for further processing.

After collecting sufficient digital data to cover the study area (Figure 6), we developed a computer program to convert these randomly spaced data into regularly spaced data suitable for a wave refraction and diffraction model. The size of each cell for the grid is 30 m in the x (shore normal) direction and 60 m in the y (shore parallel) direction. The modeled dredging area (Figure 6b) is shown by a dashed rectangle, 500 m wide and 1500 m long. In general, the water depths within this rectangle are shallower than 11 m, with some places as shallow as 9 m. The modeled dredging is a uniform two meters in this rectangle which would yield 1.5×10^6 m^3 of sand for nourishment.

Wave Refraction and Diffraction Models

There are two popular numerical models (RCPWAVE and REFDIF-1) available for simulating wave refraction and diffraction. The first one was developed by EBERSOLE (1985) and the second one was developed by KIRBY and DALRYMPLE (1991). Both models solve a simplified version of the mild slope equation given by BERKHOFF (1972). We chose RCPWAVE model for the following two reasons: (1) An algorithm to count wave-energy loss caused by bottom friction was already implemented in this model (MAA and KIM, 1992); (2) the post-processing computer software to analyze the output files generated from the RCPWAVE for studying longshore sediment transport are already available.

Wave Pattern for the Original Bathymetry

For the three wave conditions (the most severe sea, the sever sea, and the northeaster wave), six possible wave directions (from NE to E with a roughly 10 degrees difference between each direction) were processed using the RCPWAVE model to estimate the wave transformation from offshore to the coast.

Figure 7 shows the calculated wave rays for the most severe sea coming from NE, ENE, and E, respectively. Only the section from y = 5 to 20 km is presented here; other wave ray plots are omitted because the point can be made clearly using this figure. In general, waves converge near Sandbridge for all the wave directions. The convergence is especially significant for the most severe sea with waves coming from NE. A breaking wave height of about 6 m can occur near Sandbridge (Figure 8c). This trend holds for the other two wave periods (12 and 14 sec), but the rate of convergence decreases as the wave period decreases. This trend can be seen more clearly in Figure 8. The large breaking wave height caused by wave energy convergence at Sandbridge may explain the severe beach erosion there.

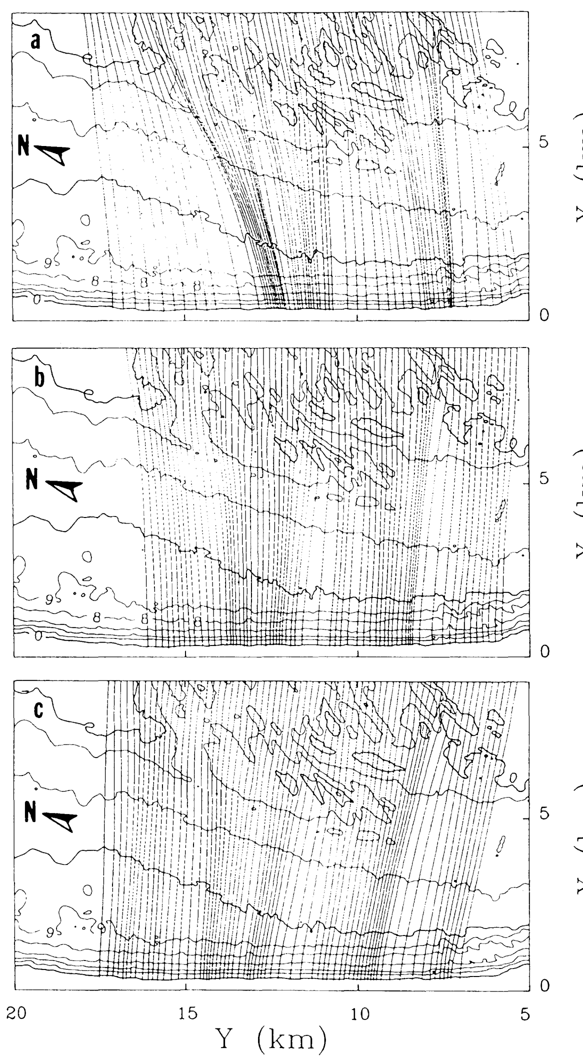

Figure 7. Wave rays for the most severe sea (H = 6.2 m, T = 20 sec) coming from (a) NE; (b) ENE; and (c) E.

Wave Pattern After Dredging

As has been discussed, wave convergence inshore of a shoal is expected because of wave refraction and diffraction. The offshore shoal studied here roughly covers a 5 km × 10 km area. The effect of wave refraction caused by the targeted dredge area (0.5 km × 1.5 km), however, is not significant. This is indicated as the wave rays do not have a significant convergence or divergence after excavating the targeted area (see Figure 8).

Assuming the modeled dredging has been completed, wave rays for the above stated sea conditions were also plotted. Because of the small differences between pre- and post-dredging, they are not presented but may be found in MAA (1995). To further demonstrate the changes in breaking wave height, H_b, and breaking wave angle, α_b, before and after the modeled dredging, the calculated H_b's and α_b's along a selected section of the beach were plotted together in Figure 8. The modeled dredge area is located approximately between y = 10 to 12 km.

Figure 8a (the northeaster waves), clearly shows that the maximum breaking wave height does not change with the modeled dredging. For the severe sea (Figure 8b) waves coming from the NE seem not to be affected significantly by the modeled dredging. Actually, H_b decreases a little at $y \approx 17$ km, but the maximum breaking wave height, which occurred at $y \approx 12$ km, does not. The breaking wave angles also have a minor change along this section.

For the most severe sea from the NE (Figure 8c), the change of maximum breaking wave height is a maximum, about 7%. For all other directions, the changes are much smaller, about 2%.

It is worth mentioning that the overall accuracy of NOAA's wave height measurement is 5%. Our calculations indicate that the possible change caused by the modeled dredging at Sandbridge Shoal is within the accuracy of the wave measurement system. This is an indication that the effect of the modeled dredging on wave transformation is insignificant. We may also expect that the longshore sediment transport

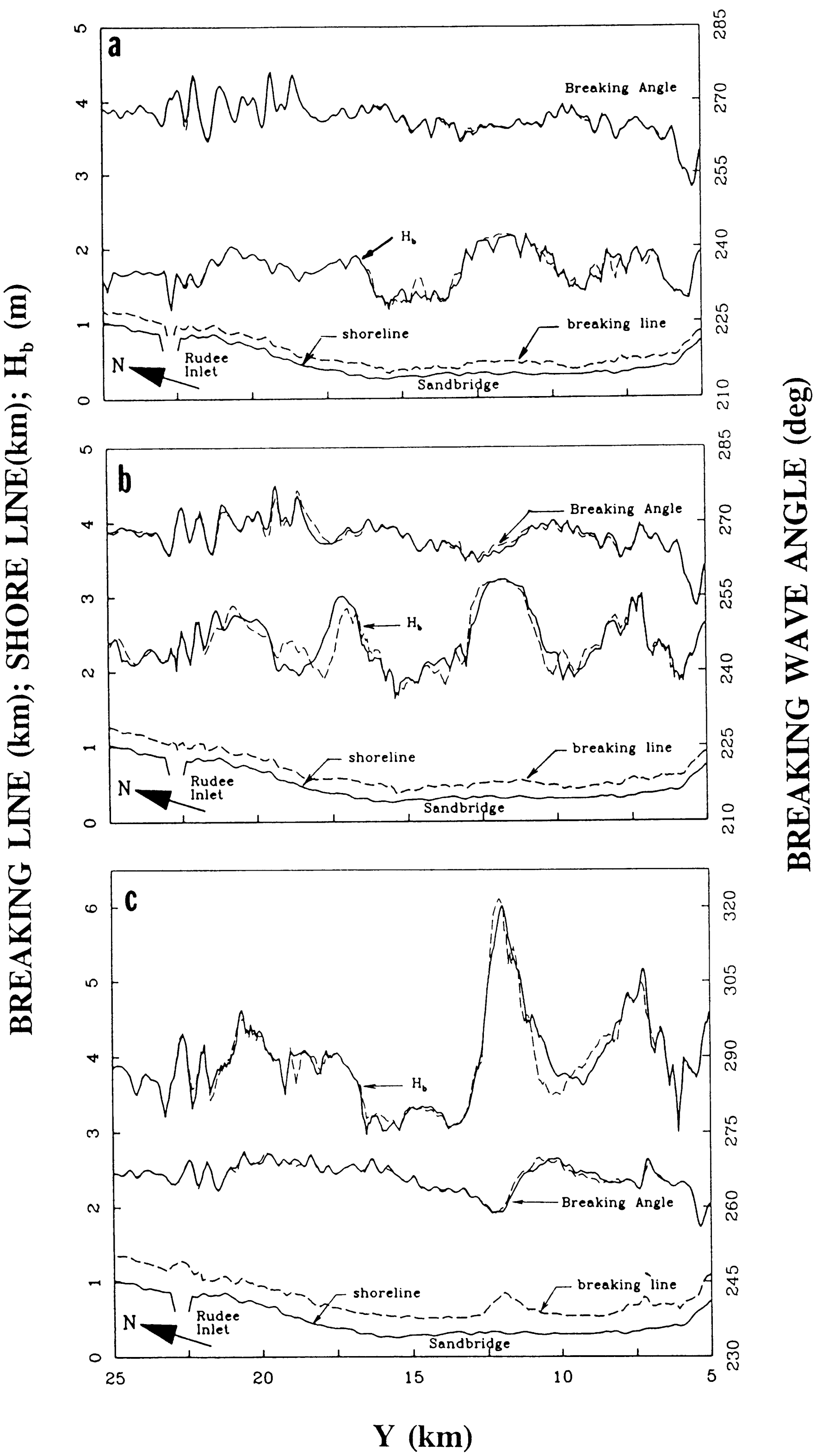

Figure 8. Comparison of breaking wave heights and angles before (solid lines) and after (dashed lines) the modeled dredging for three waves coming from NE. (a) Northeaster waves, H = 1.9 m, T = 11.8 sec; (b) The Severe Sea, H = 3 m, T = 14 sec; (c) The Most Severe Sea, H = 6.2 m, T = 20 sec.

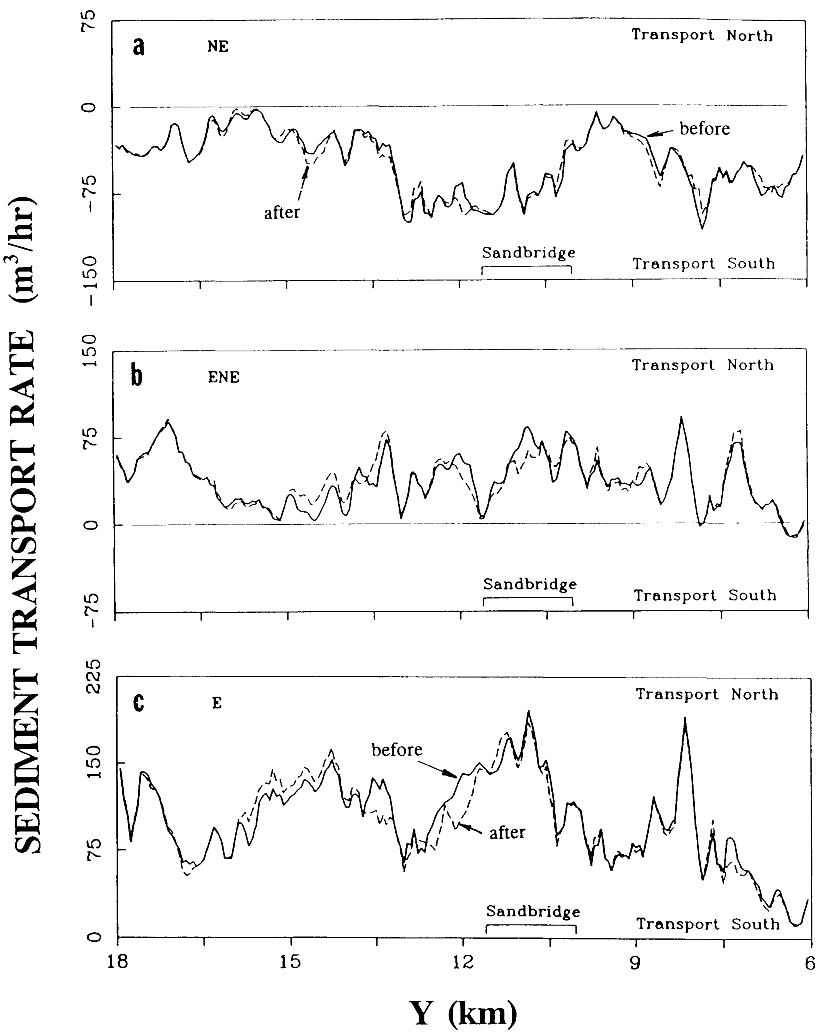

Figure 9. Comparison of longshore sediment transport rates before (solid line) and after (dashed line) the modeled dredging at Sandbridge Shoal for northeaster waves (H = 1.9 m, T = 11.8 sec) coming from (a) NE; (b) ENE; and (c) E.

process will not be affected significantly either. The following are a further verification of this hypothesis.

SEDIMENT TRANSPORT

There are many models for estimating longshore sediment transport rate. Although some models gave a similar trend of this rate, their absolute value can be quite different (WRIGHT *et al.*, 1987). It is necessary to point out that the absolute volume of sediment transport is somewhat irrelevant because only the difference in sediment transport rate before and after the modeled dredging at the shoal is of interest. For this study we selected an advanced longshore sediment transport model presented by GOURLAY (1982) to examine the possible impact of the dredging.

Longshore Sediment Transport Model

The most straight-forward approach to estimate the total shore-parallel sediment transport rate (either in mass, J, or in volume, Q) in the surf zone was simply related to the longshore breaking wave-energy flux, I, as follows:

$$I = K(EC_g)_b \sin \alpha_b \cos \alpha_b \tag{1}$$

where $(EC_g)_b$ is the wave-energy flux at the breaking point, C_g is wave group velocity, $E = (1/8)\,\rho g H^2$ is wave energy, $\rho = 1020$ kg/m³ is water density, $g = 9.8$ m/sec² is the gravitational acceleration, H is wave height, the subscript b stands for breaking wave condition, α_b is the breaking wave angle between x direction (also the shore normal direction) and the incoming breaking wave ray, and K is an empirically determined constant. The volume transport rate and mass transport rate are related as

$$I = g(\rho_s - \rho)(1 - p)Q \tag{2}$$

where $\rho_s = 2650$ kg/m³, is the sediment density, and $p = 0.4$, is the void ratio.

Although Eq. 1 has been widely used in the past two de-

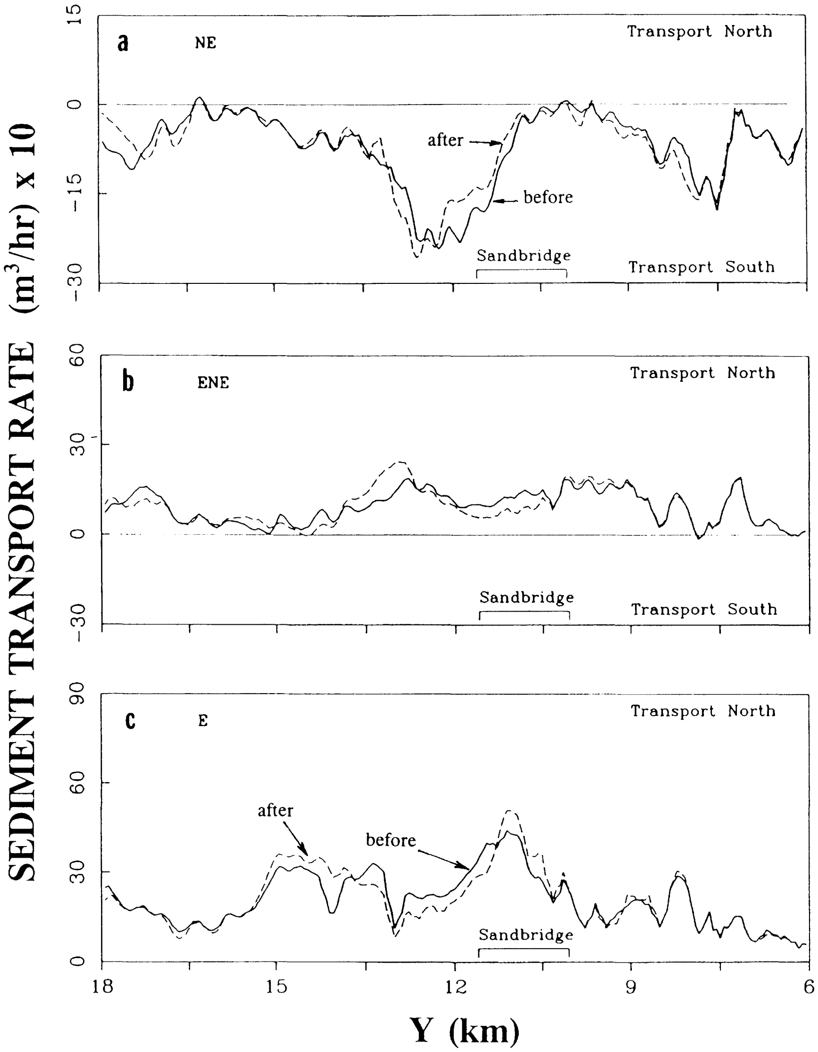

Figure 10. Comparison of longshore sediment transport rates before (solid line) and after (dashed line) the modeled dredging at Sandbridge Shoal for the severe sea ($H = 3$ m, $T = 14$ sec) coming from (a) NE; (b) ENE; and (c) E.

cades (WATTS, 1953; SAVAGE, 1962; BAGNOLD, 1963; KOMAR and INMAN, 1970; KOMAR, 1983) and was selected in the Shore Protection Manual (CERC, 1984), this formulation assumes breaking wave energy is totally dissipated in the surf zone, and the gradient of radiation stress (LONGUET-HIGGINS and STEWARD, 1962), $\partial S_{xy}/\partial x$, is the only force that drives the longshore current. Thus, Eq. 1 is good for an ideal coast with straight shoreline and parallel depth contours from the coastline to far offshore.

In reality, however, wave breaking condition (H_b and α_b) always varies along a coast because of the irregular bathymetry, *e.g.,* see Figure 8. For this reason, wave set-up induced by another component of the radiation stress, $\partial S_{xx}/\partial x$, at the coast will not be the same. This varying wave set-up (*i.e.,* water surface elevation) along a coast can induce longshore current even for a normally incident wave (*i.e.,* $S_{xy} = 0$). This second component of longshore current can either enhance or diminish the first component. Therefore both the longshore energy flux (caused by oblique waves) and the gradient of wave set-up (caused by changing breaking wave height along the coast) should be considered in the longshore current, *i.e.,* longshore sediment transport.

Based on the above principal, KOMAR and INMAN (1970) modified Eq. 1 to include the influence of nonuniform breaking wave condition along a coast. Later, GOURLAY (1982) modified Komar and Inman's model and proposed the following equation:

$$I = K^* (EC_g)_b \cos \alpha_b \left[\sin 2\alpha_b - \frac{K_{\Delta H}}{\mathrm{Tan}\ \beta} \frac{\partial H_b}{\partial x} \right] \qquad (3)$$

where Tan β is the average beach slope between the breaking point and the shoreline, $K_{\Delta H} = 23.7$, $K^* \approx 0.385 K_b$, and K_b depends on the Irribaren number, ξ, given as

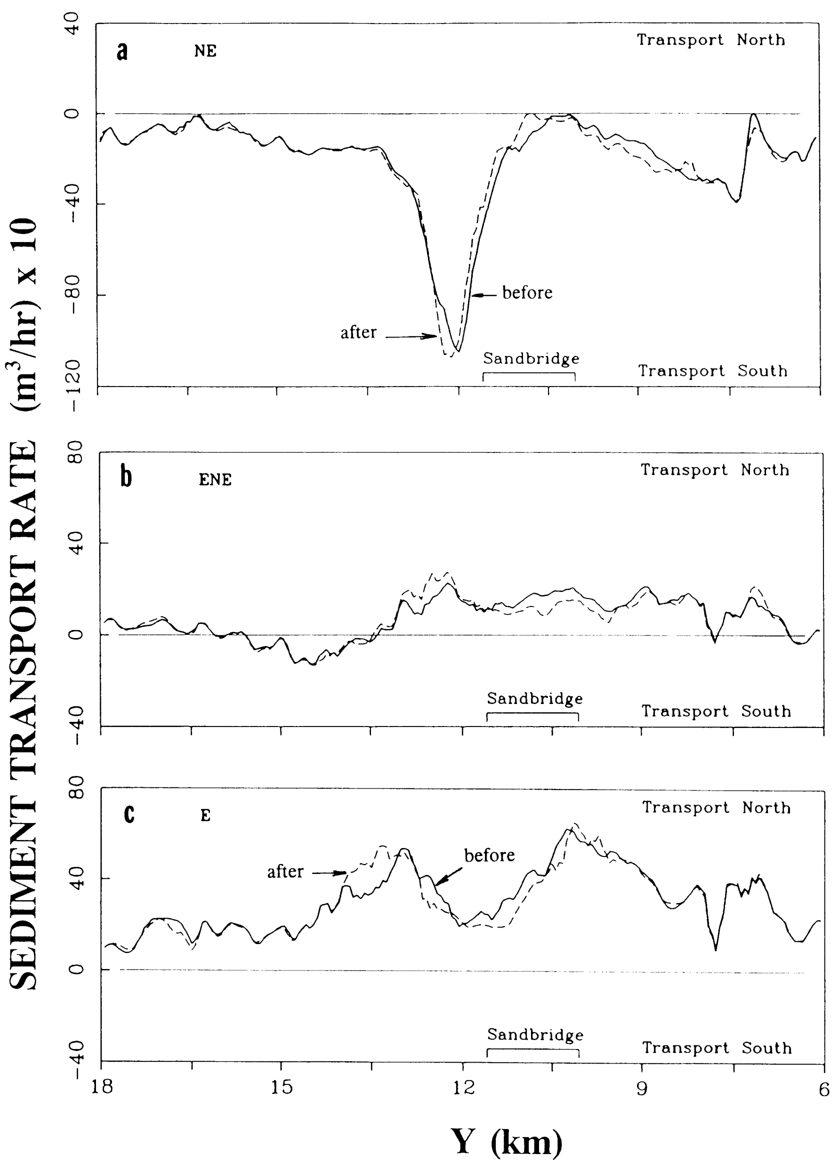

Figure 11. Comparison of longshore sediment transport rates before (solid line) and after (dashed line) the modeled dredging at Sandbridge Shoal for the most severe sea (H = 6.2 m, T = 20 sec) coming from (a) NE; (b) ENE; and (c) E.

$$\xi = \frac{2\pi \operatorname{Tan} \beta}{H_b g T^2} \quad (4)$$

When $\xi \geq 1.7$, $K_b = 1$, but when $\xi < 1.7$ then $K_b = 0.45\xi/K^*$.

Model Results

Using Gourlay's model (GOURLAY, 1982), we calculated the rate of longshore sediment transport based on the calculated breaking wave heights and breaking angles before and after the modeled dredging. An example of the results are given in Figure 9. This figure indicates that the modeled dredging does not have a significant effect on the longshore sediment transport for the northeaster waves. The calculated change increases as the wave direction shafts from NE to E. The maximum change, about 20% reduction, occurred locally between y = 11 and 14 km, for the wave coming from the E. The amount of north-going sediment transport, caused by waves coming from E, increases a little after dredging between y = 14 to 16 km. In general, the change of longshore sediment transport rate caused by the northeaster waves is not significant, and the possibly affected area is located to the north of Sandbridge. On the south side, the influence is negligible.

For the severe sea, Figure 10 reveals that there are small changes in sediment transport rate on the north side of Sandbridge. The amount of change for each wave direction is about the same.

The results for the most severe sea (Figure 11) are similar to those for the severe sea, with only small difference near Sandbridge and no significant difference before and after dredging. The large south-going transport of sediment at the immediate north side of Sandbridge (Figure 11a at y = 12 km) caused by the huge breaking waves is the worst condition for maintaining a beach in that area. The modeled dredging does not improve or worsen the condition at all. It just moves the location of maximum transport a little further north.

Considering the fact that in a year the waves may reach the level of the northeaster waves about 5% of the time, and the level of the severe sea about 1% of the time, the calculated possible small change of longshore sediment transport may be considered insignificant. If affected, the area would be mainly on the north side of Sandbridge.

CONCLUSIONS

The study of available wave records from station CHLV2 and 44014 indicates that long period waves (period > 12 sec) mainly come from NE to E. Because short period waves cannot be affected by the presence of Sandbridge Shoal, they were ignored in this study. Three categories of wave conditions were examined, and the possible influence by the modeled dredging are summarized as follows.

The responses of longshore sediment transport to the modeled dredging at Sandbridge Shoal for the three selected wave conditions are insignificant for all the six directions, from NE to E. The change of breaking wave conditions, as well as the longshore sediment transport rate, are within the accuracy of wave measurements. Small local changes do occur, but no significant alternation of the pattern of longshore sediment transport. Among those small changes, they occurred mainly to the north of Sandbridge.

The effect of modeled dredging on the nearby coast, mainly Sandbridge, is based on an assumption that the dredged area is about 500 m × 1500 m with a uniform 2 m dredging. An immediate concern about how large the dredged area can be allowed at this area without significantly changing the wave transformation process, or more aggressively, where to dredge in order to reduce the large breaking wave height caused by wave energy convergence like that shown in Figures 7a and 8c remains unanswered. This may be an important question as Sandbridge Shoal may be the most important reasonable source for beach-quality sand in the future. This critical issue needs to be addressed in future studies.

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