

CHAPTER 287

Influence of Nearshore Berm on Beach Nourishment

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Abstract

The influence of a large ($3.0 \times 10^6 \text{ m}^3$) nearshore berm on a large ($4.1 \times 10^6 \text{ m}^3$) beach nourishment project at Perdido Key, FL, is addressed via a monochromatic, numerical wave transformation model. Two years of post-placement survey data indicate that the nearshore berm, placed at a depth of 6 m with a relief of 1-1/2 m, did not migrate during this time period. Wave model results suggest that the berm influences the beach nourishment project via refraction, but breaking and diffraction effects are not significant. The berm causes wave transformation and leads to zones of wave energy focusing (and corresponding de-focusing) that affect longshore sediment transport rates in the lee of the berm. For a typical wave condition, predicted high-energy regions correlate well with observed erosional areas. The findings indicate the relative importance of project-induced wave transformation that should be considered during the design process for nearshore berm projects.

Project Background

The beach nourishment project at Perdido Key, FL, adjacent to Pensacola Pass, was initiated in late 1989. Pensacola Pass serves as an access channel to a major naval port; the beach nourishment material was generated by dredging of the entrance channel to 19 m. The region is characterized by diurnal tides with a very small range (mean tide range = 34 cm; spring range = 58 cm) and a relatively mild wave climate. The mean value of the significant wave height measured over 4 years

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as a part of this study near the 6 m contour is 0.6 m. The mean value of the wave period at which the peak of the energy spectrum is found is 6 sec for the same time interval.

The beach nourishment aspect of the project involved widening the dry beach by 140 m over a length of 7 km and required a volume of 4.1 million m^3 . This was completed in September, 1990. Construction of an offshore berm then commenced. The offshore berm was completed in 1991. The placed berm was 4 km (longshore) by 300-800 m (cross-shore) and included 3.0 million m^3 of sand. The bathymetry in the region where the berm was placed was quite flat (slope of 1:1000). Water depth averaged 6 m, and the berm relief is nominally 1.5 m, leaving 4.5 m of water above the berm. Figure 1 indicates the site location, and Figure 2 shows cross-sections through the berm at different times.

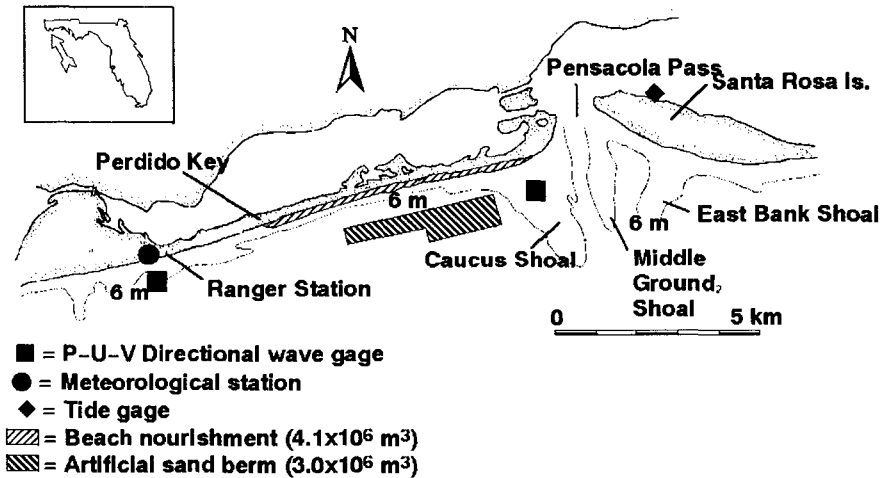


Figure 1. Site location and monitoring plan.

A monitoring program from 1989-1994 yielded a data set that is useful for assessment of hydrodynamics and sediment transport at Perdido Key, including annual bathymetric survey data and long-term wave and current data. Details of the monitoring program may be found in Work (1992), Otay (1994), and Work and Dean (1995).

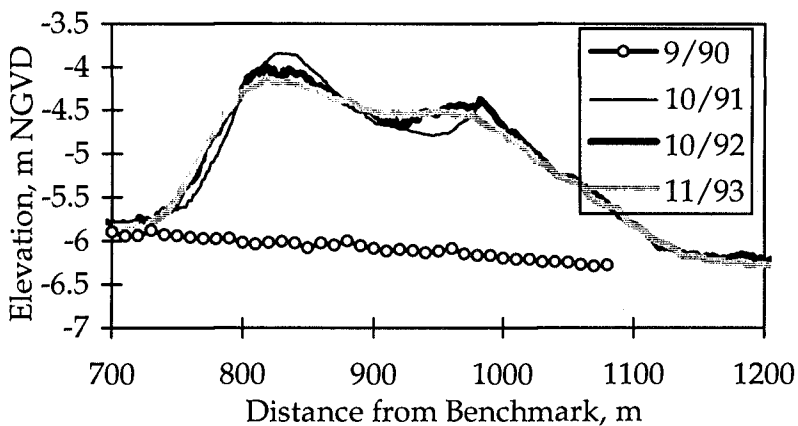


Figure 2. Cross-section through berm near midpoint.

Survey data indicate that the berm has not migrated since placement, although it has been smoothed slightly (Figure 2). But due to the 25% reduction in water depth that it introduces over a large area, it is expected that the berm will exert some influence on the waves that pass over it, and therefore modify nearshore sediment transport. The purpose of this paper is to investigate this effect and assess its significance.

A nearshore berm can provide at least two benefits, depending on its behavior:

- 1) Nourish the beach as sediment moves onshore.
- 2) Shelter the beach in the lee of the berm.

Berms are sometimes favored over beach nourishment because of the reduction in cost due to the relative ease of placement. But correct assessment of the benefits requires accurate prediction of both any migrational tendency and any effect on adjoining areas.

Bathymetry

A numerical grid of depths at the site was created for wave modeling. The grid represents a combination of measurements taken as a part of the monitoring study and digitized nautical charts. Depths at the offshore boundary are not uniform and the grid does not extend to deep water, posing some problems for wave modeling that are addressed below.

A second grid was created by artificially stripping out the berm from the first grid. A detail of the two grid files is shown in Figure 3. Some “scalping” of the nearshore contours is evident in the no-berm plot. This is due to limitations of the contouring routine when there are “stripes” of data (in this case due to surveying beach profiles at 300 or 600 m intervals along the beach), a common problem. This will lead to some overestimation of the effects of wave transformation.

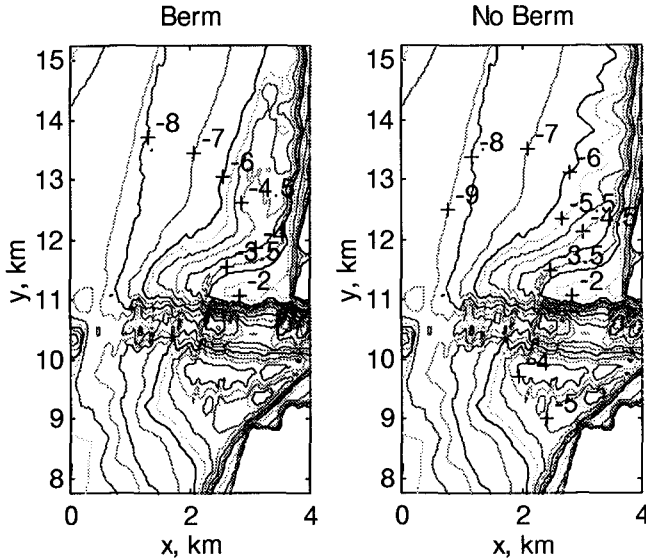


Figure 3. Bathymetry with and without offshore berm. Contour interval 1 m, with the addition of 3.5, 4.5, and 5.5 m depth contours. x -axis points onshore, y -axis in longshore direction. Pensacola Pass at right near $y = 10$ km.

Hydrodynamic Modeling

Measurements of wave energy spectra and mean currents are available at two locations at the site (Figure 1). A finite element hydrodynamic model (RMA-2, Norton, King, and Orlob, 1973, Thomas and McAnally, 1990) was employed to predict tidal currents. The friction factor in the model was adjusted until modeled currents agreed with published values for predicted tidal currents in the inlet throat (U.S. Dept. of Commerce, 1994). Results were then compared to measurements at the two wave gage locations for maximum ebb and maximum flood conditions. The model typically underpredicted currents slightly, not surprising due to the omission of wind- and wave-driven currents in the model. Figure 4 illustrates the results for maximum ebb tide.

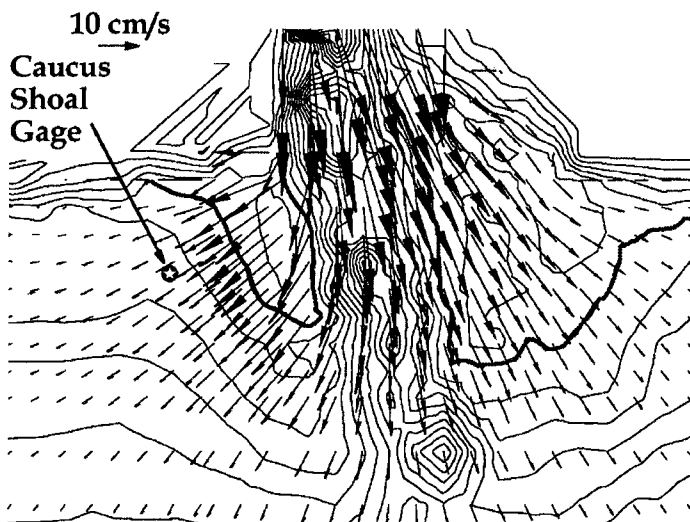


Figure 4. Predicted tidal currents (from RMA-2 model) for maximum ebb tide through Pensacola Pass. Note that flow is concentrated between shoals to either side of navigation channel.

Wave Modeling

A monochromatic, finite difference wave transformation model (REF/DIF 1, Kirby and Dalrymple, 1994) was used to investigate wave transformation at the site. The monochromatic model was selected for two reasons: 1) ease of use, compared to a spectral model, and 2) to assess the viability of a monochromatic model for such a situation. Ease and speed of use become major factors when long-term simulations (years) will be performed.

Since neither bathymetry grid extended to deep water for most of the wave conditions to be modeled, an iterative procedure was developed to estimate wave conditions at the offshore boundary, given wave conditions at the western wave gage, where the bathymetric contours are relatively straight and parallel (Work and Kaihatu, in press). This allows one to drive the model with data from one nearshore wave gage (a common situation) and predict wave conditions anywhere else in the domain.

A large number of model runs have been performed with the real (berm) bathymetry at the site, using data from the western wave gage to drive the model. The monochromatic model, on average, yields a reasonable prediction of wave height at the other gage, although there is a large amount of scatter (the average value of the ratio of model to measured wave heights is 0.99; standard deviation of this

ratio is 0.34). The model is not a good predictor of wave direction at the other gage, in part because of the fact that the contours run nearly perpendicular to the nominal shoreline orientation at the eastern wave gage, which sits near a shoal. Figure 5 shows results for one simulation near this shoal.

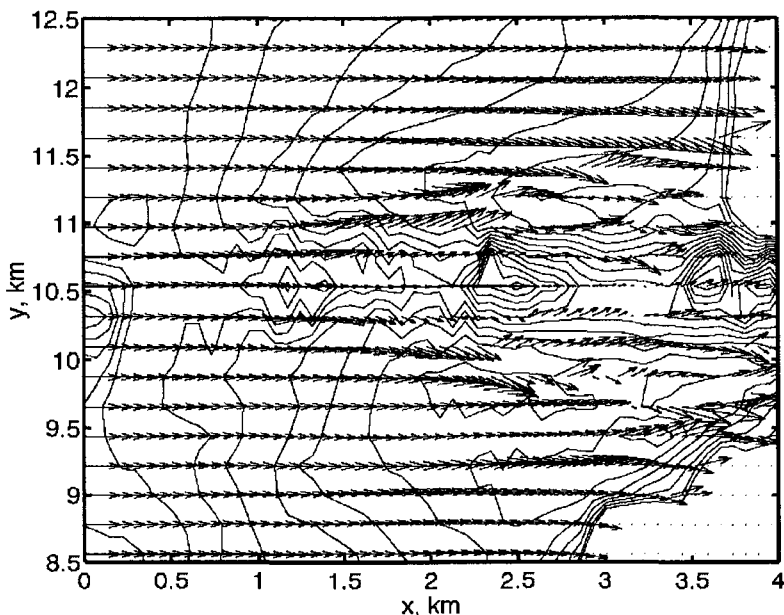


Figure 5. Wave vectors near Caucus Shoal, immediately west of Pensacola Pass. Conditions at western wave gage: $H_{mo} = 0.6$ m, $T_p = 6$ sec, shore-normal incidence.

Wave model results indicate that tidal currents at the site do not exert any significant influence on wave conditions outside of the inlet channel. Thus bathymetric gradients are the primary cause of wave transformation. This simplifies modeling efforts substantially.

Hypothetical cases were simulated to isolate the influence of wave period, tide stage, and incident wave height and direction, on wave conditions in the lee of the nearshore berm. Figure 6 compares the wave heights for the two bathymetric grids for the "typical" case ($T_p = 6$ sec, shore-normal waves with height $H_{mo} = 0.6$ m at western wave gage). Figure 7 shows the difference between the two results, normalized by the incident wave height. Some findings:

- Tide stage has minimal influence on the wave height differences (berm vs. no berm). This is largely due to the very small tide range at the site.

- Dependence on wave period is as expected: as period increases, refraction becomes more pronounced, and differences between the two results increase.
- Dependence on wave height is weak until the incident wave height is increased to the point where waves begin to break on the nearshore berm ($H_{mo} > 3.5$ m). The largest significant wave height measured during the 4-year wave monitoring program was 2.9 m, in August, 1992, while Hurricane Andrew was in the Gulf of Mexico. This event also yielded the largest measured wave period ($T_p = 13$ seconds). In summary, wave breaking on the berm has occurred rarely, if at all.
- Dependence on incident wave direction is relatively strong. Greatest differences between the berm and no-berm cases were observed for nearly shore-normal waves.

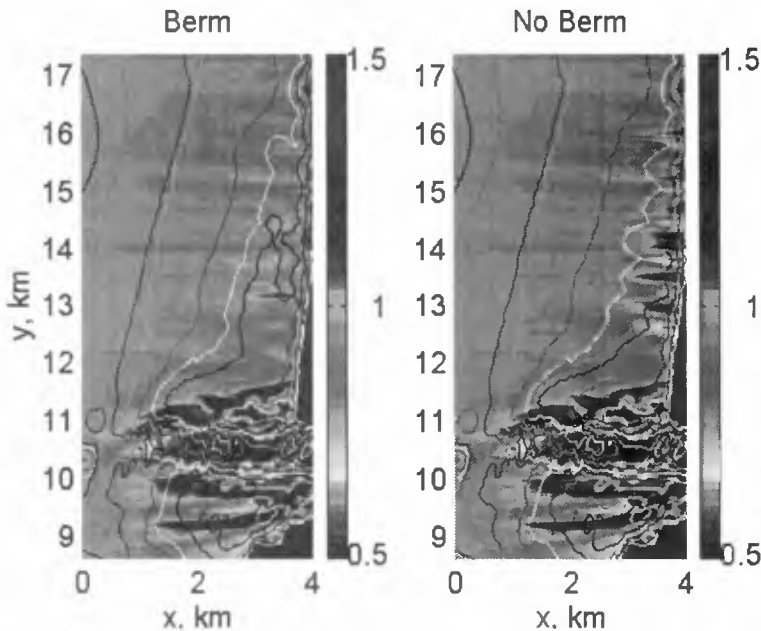


Figure 6. Wave model results with typical wave conditions ($H_{mo} = 0.6$ m, $T_p = 6$ sec, shore-normal incidence, at western wave gage), with and without berm. Contours indicate depth at 1 m intervals. Shade indicates wave height relative to wave height at western wave gage.

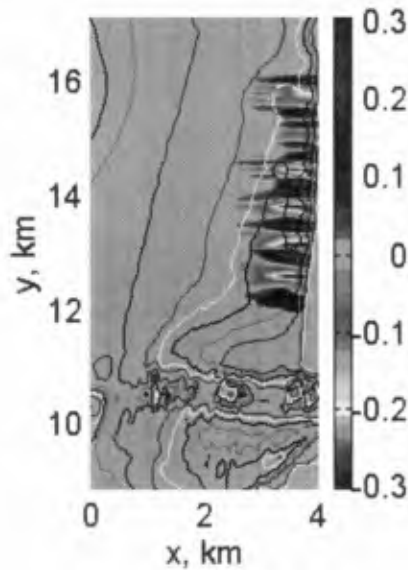


Figure 7. Difference between no-berm and berm wave model results shown in Figure 6. Normalized by no-berm results. Shade indicates percent difference in wave height. Positive values indicate percent increase in wave height in absence of berm, negative indicate percent reduction.

The maximum difference between the berm and no berm wave height fields, for the region behind the berm, is 30%. Because there is typically a minimal difference in energy dissipation between the berm and no berm cases, the total energy reaching the shoreline is the same whether or not the berm is present. The berm acts to redirect energy and leads to zones of focusing and de-focusing. If depths over the berm were less, breaking would occur on the berm and the shoreline behind the berm would be more sheltered. Figure 8 compares the wave height fields for large vs. small incident wave heights, both over the berm bathymetry, to illustrate this effect.

Limited tests were also performed with a spectral version of the same wave model (REF/DIF S, Ozkan and Kirby, 1993). This model essentially consists of superimposing many results from the monochromatic model for different frequencies. Results with the spectral model were similar to the monochromatic model results, but more muted. The presence of additional frequency components leads to less pronounced focusing of wave energy. One example is provided in Figure 9.

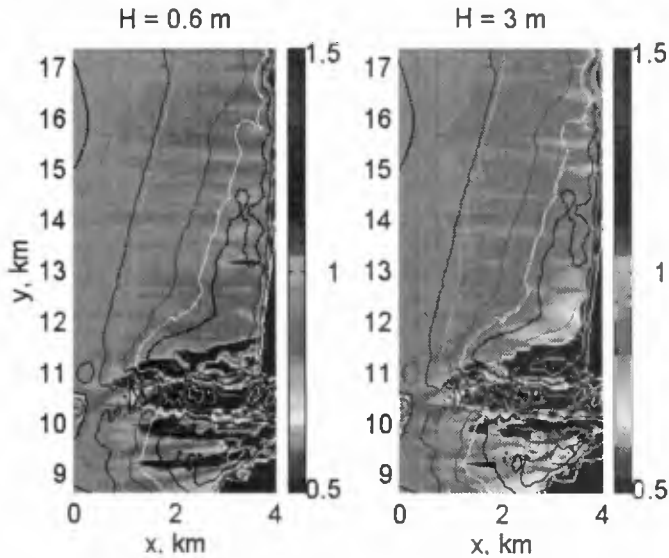


Figure 8. Small wave height ($H_{mo} = 0.6 \text{ m}$, $T_p = 6 \text{ sec}$, shore-normal at western wave gage) vs. large ($H_{mo} = 3.0 \text{ m}$) wave height results over berm bathymetry. Shade indicates wave amplification. Contours indicate depth at 1 m intervals.

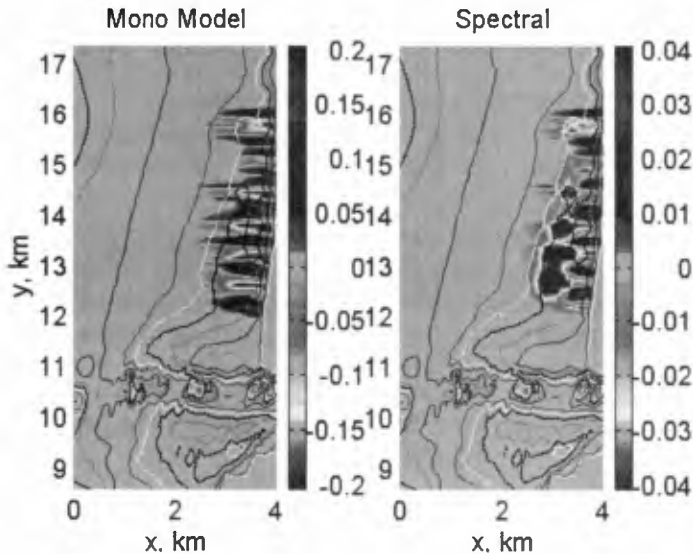


Figure 9. Monochromatic model (REF/DIF 1) vs. spectral model (REF/DIF S) results for conditions corresponding to Figure 6. Shade indicates percent difference in wave height. Positive values indicate percent increase in wave height in absence of berm, negative indicate percent reduction.

Shoreline Change

Redistribution or reduction of wave energy reaching the shoreline will affect sediment transport. Figure 9 shows the measured changes in shoreline planform, relative to the pre-nourishment condition. This is a classic signature for a beach nourishment project: the waterline moves back, rapidly at first, due to cross-shore sediment transport, since the as-built beach profile slope is much steeper than natural slopes. The “shoulders” (ends) of the project erode more rapidly due to the high rate of sediment diffusion in the longshore direction induced by strong planform gradients.

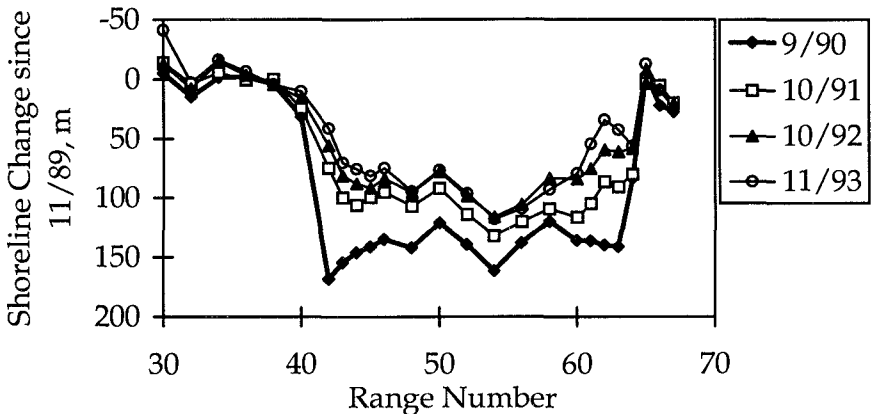


Figure 10. Measured change in shoreline position, relative to pre-nourishment condition. Nominal distance between survey ranges is 300 m, Pensacola Pass at right limit of figure.

Another means of investigating the evolution of a nourishment project is to calculate the volumetric changes per unit length of beach, i.e. the change in the cross-sectional area of the beach profile. Doing so allows one to integrate out the effects of cross-shore sediment transport: all observed changes are due to longshore redistribution of sediment arising from longshore gradients of longshore sediment transport. This requires assumption of a depth beyond which there is no cross-shore sediment transport. For the Perdido Key beach nourishment project, it is quite clear that there is negligible sediment transport beyond the 5.5 m contour (Work and Dean 1995).

Figure 11 shows the computed longshore gradient of longshore sediment transport, which is equal in magnitude (but opposite in sign) to the change in cross-sectional area of the beach profile per unit time. Positive values indicate erosion and

negative indicate accretion. The question is then whether these zones of erosion and accretion can be predicted by the numerical wave model. A definitive answer to this question would require that the wave model be run for every measured wave event to calculate a four-year time series of breaking wave height and direction. These time series could then be used to drive a shoreline change model.

Unfortunately it is not possible to model wave transformation for every measured wave event. There are many measured wave directions which are highly oblique (> 45 degrees) to the shoreline. The chosen wave model (and any other similar model which employs a parabolic approximation to the elliptic governing equation) has a limitation on the incident wave angle. If the incident wave angle exceeds a certain value (the value of which depends on details of the model formulation), the model will yield erroneous results or no results at all. In addition, the method developed to estimate wave conditions at the offshore boundary of the grid, given wave conditions at one nearshore wave gage, similarly will not work for highly oblique waves. In practice, it was possible to model only about one-half of the measured wave conditions.

A “typical” wave condition was therefore chosen for comparison of wave height fields and observed shoreline changes. Noteworthy regions are indicated by the arrows on Figure 11. Several features stand out:

- There is consistent focusing predicted (both with and without the berm, for nearly all wave conditions) at the eastern limit of the nourishment project. This is indicated by the arrow at the extreme right of Figure 11. A strong erosional trend is also observed there. This is erosion of the end of the beach nourishment project, enhanced by the focusing of wave energy.
- Placement of the berm reduced, for the “typical” wave condition, the wave heights near the midpoint of the beach nourishment project, where a switch from erosional to stable behavior is observed (middle arrow in Figure 11). A similar switch occurred near the western end of the nourishment project, where placement of the berm reduced wave heights, and a switch from a stable shoreline to accretion occurred (left arrow).

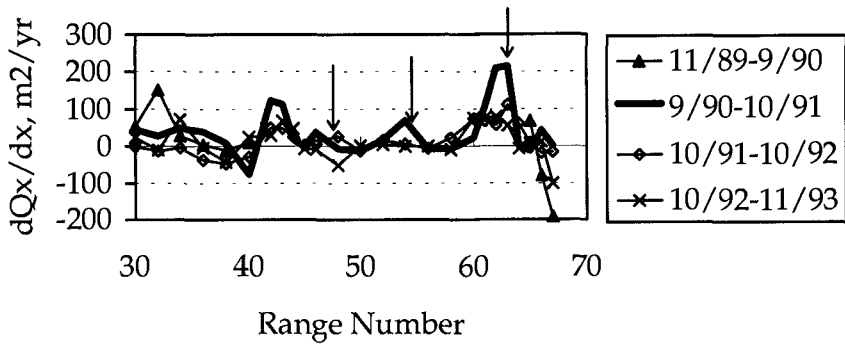


Figure 11. Longshore gradient of longshore sediment transport, calculated from surveyed beach profiles. Equivalent in magnitude to change in cross-sectional area of beach profile. Nominal spacing between ranges is 300 m, Pensacola Pass to right.

Project Performance

Volume of sand remaining within the nourished region was computed as a measure of project performance. Three years after placement of the beach nourishment material, 84% of the initial volume remains (Figure 12). If the entire monitored area is considered, the value is 82%. Some of the nourishment material has been deposited into the inlet, and some has moved westward due to the predominant wave conditions.

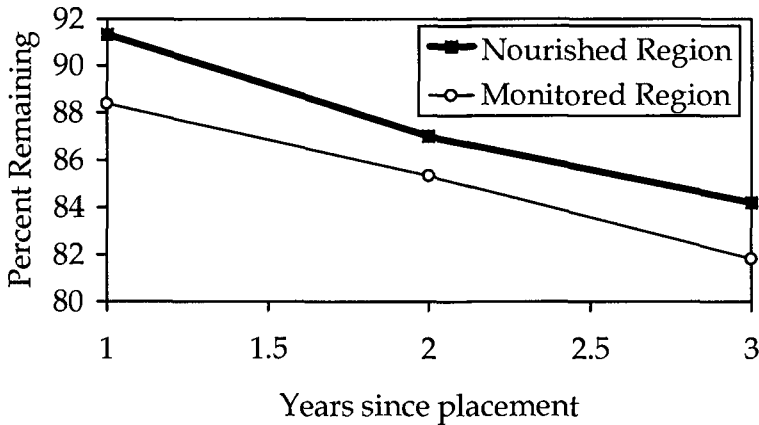


Figure 12. Volume changes for project. Nourished region is a subset of the monitored region.

Conclusions

Nearshore berms can and have been placed with the goal of gradual beach nourishment through onshore migration. Existing predictive capability with regard to rate and even direction of migration (onshore or offshore) is still limited. A berm can also exert a sheltering effect on the beach in its lee. The presence of the berm can either merely redistribute wave energy, or, if it induces additional dissipation through breaking over the crest of the berm, reduce wave energy reaching the shoreline. The effect of the berm on the shoreline behind it should be considered as part of the design process.

Data for two years after placement of a berm at Perdido Key, FL, indicate the berm did not migrate during this time. It has been slightly smoothed but its volume unchanged. Numerical wave model results indicate that the berm does exert some influence on the breaking wave climate by redistributing energy alongshore. This occurs even in the absence of waves breaking on the berm. Changes in nearshore wave heights which occur as a result of the berm are reflected in the measured shoreline signature, for a typical incident wave.

It was not possible to model all wave events with the chosen numerical wave transformation model. Waves which deviate significantly from shore-normal occur frequently at the site but could not be modeled. Improvements in wave modeling techniques will eventually remove this constraint.

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