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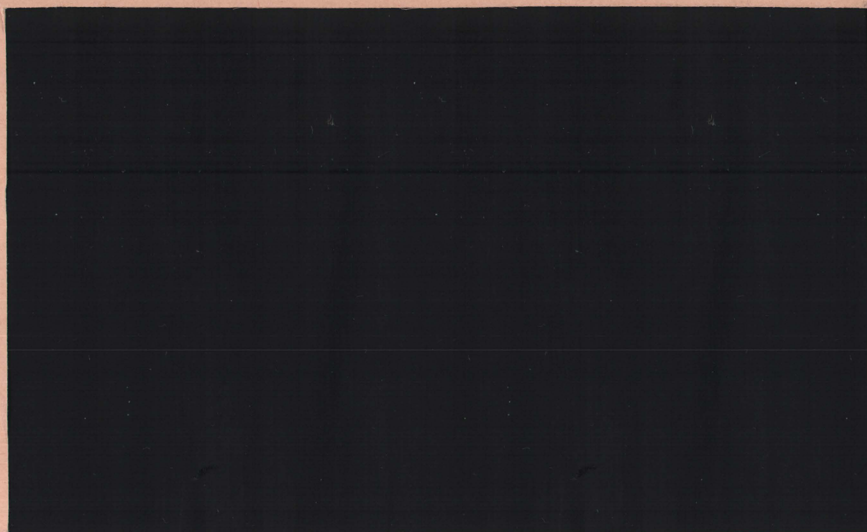

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SAND TRANSPORT NEAR THE GROINS AT GEORGICA POND, EAST HAMPTON, NY

Executive Summary

To determine the influence of the two Federal groins on shoreline changes at Georgica Pond, East Hampton, the rate of shoreline change was modeled both for existing conditions and for a hypothetical situation in which the Federal groins were assumed to be shortened to the length of the existing state groins (Bruno, this report). The state-of-the-art shoreline evolution model, that has been successfully applied elsewhere (Bruno, 1990), relied upon available wave (Corson et al., 1982) and longshore sediment transport data (Bokuniewicz, this report). The groin field was found to produce major shoreline fluctuations with the Federal groins greatly accentuating both downdrift erosion and updrift accretion under existing conditions. Because the eastward and westward littoral drift of sand is almost equal in this area, erosion sometimes occurs on the west of the groin and sometimes on the east. Over the long term, however, a westward drift is expected. The shoreline perturbations would be reduced by shortening the two Federal groins. Reduction in the rates of shoreline change would be greatest during periods when the long-term drift of sand is to the southwest, and slight reductions in the rates of change should be expected during individual storms.

Georgica Pond has played only a very minor role in the long-term sand budget here. Aerial photographs and ground surveys were used to calculate the total amount of beach sand (389,000 yd^3) that has infilled the Pond. Since the Federal groins were installed 26 years ago, the annualized rate of sedimentation in Georgica Pond would be 14,961 yd^3. This amount is equal to only about 10 percent of the average net longshore sediment transport of beach sand and only a few percent of the gross littoral drift. A 20-year computer model, including the net loss of sand at Georgica Pond, revealed that the loss of sand at the pond has only a very localized effect, concentrated within 500 ft east and west of the inlet; the rate of shoreline change due to sand loss at the pond differs only by a maximum of 7% from that experienced assuming no sand loss.

A final question of interest involves how the anticipated shoreline response to groin shortening affects inlet management at Georgica Pond. Reducing the length of the Federal groins should allow for a wider beach in front of Georgica Pond over the long term. Under conditions of southwestward drift the probability of accidental breaching would be reduced substantially and it would only be marginally increased during adverse conditions. Infilling of the Pond by overwash would be reduced, therefore, and Pond letting could be more surely controlled to occur during the most favorable conditions.

Computer Modeling of Shoreline Changes at East Hampton, New York

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Introduction

As a supplement to the analysis of historical records of shoreline position at East Hampton, and with the aim of systematically assessing the impact of the coastal structures in the area, a computer modeling effort was carried out. This effort utilized a state-of-the-art wave transformation-shoreline evolution model developed by one of the authors (Bruno, 1990). A description of the theoretical basis and capabilities of the model is included here in Appendix A. In essence, the model uses information regarding the bottom depth contours, bottom sediments, and deepwater wave characteristics to simulate the nearshore wave and sand transport processes and predict changes in the shoreline position. The influence of coastal structures such as groins and jetties on the wave characteristics and sand transport is included. It should be mentioned that the model has been used successfully in the analysis of other coastal regions, including the beach at Harvey Cedars, New Jersey (Bruno, 1990) and the areas to the north of Manasquan Inlet, New Jersey (Bruno, 1988).

In the following, we detail the procedures used in the application of the computer model to the beach and groin system at East Hampton.

Preparation of Model Grid

A primary requirement of the model is the detailed specification of water depth at points, or "nodes", throughout the offshore region to be modeled. The choice of the extent of this modeling region and the number of nodes defined within the region, is a function of the scale and accuracy desired. Figure 3 illustrates the model grid prepared for the East Hampton area. The grid has been superimposed on NOAA charts and the grid spacing, or separation between nodes, has been chosen as 100 meters, resulting in a total of 10,735 solution nodes in the modeled area. This is considered very fine resolution for coastal modeling and affords a precise simulation of the coastal processes and shoreline changes. The alongshore extent of the model grid has been chosen so as to include the widest possible zone of influence of the four groins being examined. The offshore extent of the grid represents the deepwater location of the incident wave data to be used as input to the model. For reference purposes we have numbered the location of each of the four groins, beginning with the State groin to the northeast of Georgica Pond. Groins 2 and 3 are the Federal groins.

The bathymetric data obtained from the NOAA charts was supplemented by detailed bottom surveys conducted in 1979 by SUNY Stony Brook along six shore-normal transects. The combination of these two data sources permitted the preparation of a highly detailed model grid.

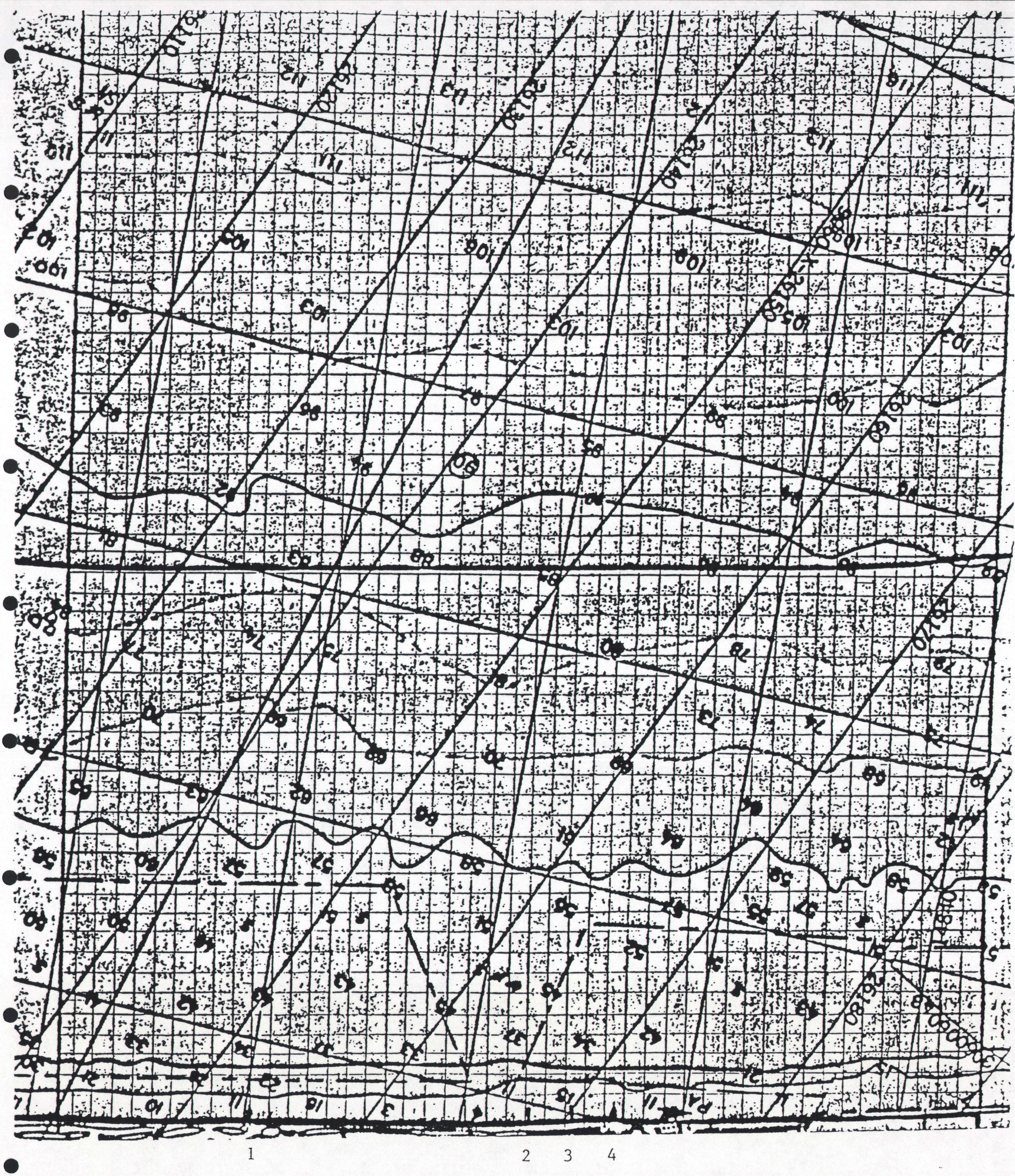

Figure 3 : Model Grid

Offshore Wave Information

In order to simulate the nearshore processes and shoreline changes over any given time period, the model requires as input the offshore (deepwater) wave characteristics observed or predicted for that period. Specifically, these characteristics include wave height, angle of approach, and wave period.

Because of our interest in the long-term response of the East Hampton shoreline to man-made alterations (e.g., groins), we have chosen to make use of a 20 year record of offshore wave characteristics, covering the period 1956-75, developed by the U.S. Army Engineer Waterways Experiment Station (Corson, et al., 1982). This hindcasted wave record includes wave heights, directions and periods at three-hour intervals at 73 locations along the east coast of the United States, for the entire 20 year period. For the present investigation, we have chosen the offshore location nearest to the study area. The 20 year record of deepwater wave characteristics was employed in a statistical format, that is, wave height, direction, period and percent of occurrence over the 20 year period for each wave "type". Nearly 500 different combinations of wave characteristics, each with a different percent occurrence, were employed in the preparation of the 20 year deepwater wave record for the present study.

It should be pointed out that in a statistical sense, this 20 year record can be viewed as representative of the long-term average wave climate in the region. As such, the record can be effectively used to simulate existing conditions and predict future changes due to natural or man-made alterations to the coastline.

Model Application

In order to investigate the impact of the four stone groins, particularly the two long Federal groins, on the patterns of erosion and accretion in the area, we followed a procedure designed to isolate each groin's influence under a variety of scenarios.

The most effective way of assessing whether or not the Federal groins are creating or exacerbating erosion conditions along adjacent beaches is to compare the model-generated shoreline changes under existing conditions, with the shoreline changes obtained assuming that the length of the two groins is reduced from 480 ft to 275 ft (the length of the two State groins). The reasons for this procedure are twofold: 1) the comparisons should indicate clearly the extent to which the Federal groins influence the adjacent beaches; and 2) it is felt that if the groins are indeed having a negative impact on the adjacent beaches, shortening the length to that of the State groins may ameliorate some adverse effects.

In the first model application, the entire, unedited 20 year wave record was employed. As alluded to earlier, the results included a net drift toward the northeast at a rate of 62,773 yd^3/year. The sand transport was directed toward the northeast 67% of the time and toward the southwest 33% of the time. Field observations suggest, however, that the long term net transport at East Hampton is toward the southwest at approximately 180,000 yd^3/year. There have also been prolonged periods of time when the net transport was toward the northeast at rates as high as 290,000 yd^3/year.

In an effort to make the most effective use of the wave transformation shoreline change model to examine the impacts of the groin structure, we have chosen to modify the 20 year deepwater wave record so as to produce the observed long term net drift of 180,000 yd^3/year toward the southwest. This modification was done in an iterative fashion by removing extremely large waves ("spikes") approaching from the south until the modeled transport rate approached the desired rate. It should be mentioned that 94% of the original wave record was retained during this procedure. This data set was treated as the long term average wave climate for use in the long term (20 year) modeling of shoreline changes.

The second primary wave data set was again obtained from the original wave record, but this time with the aim of producing a net drift of 290,000 yd^3/year toward the northeast. It was felt that the inclusion of these dramatic, yet observed, reversals in transport is essential to accurately assessing the effects of the groins.

Finally, we employed the model using two simple single wave conditions, one representing storm waves approaching from the south and the other representing storm waves approaching from the east. The comparisons between existing and shortened groin conditions for these two simple wave fields should provide additional, straightforward evidence of any groin related shoreline changes.

The loss of sand at the ephemeral inlet to Georgica Pond was assumed to have a negligible impact on the shoreline (Leatherman, this report). In order to verify this conclusion, a 20 year computer simulation was performed that included a net loss of sand at Georgica Pond in the amount of 19,450 yd^3/year. This simulation revealed that the loss of sand at the pond has only a very localized effect, concentrated within 500 ft east and west of the inlet. Furthermore, it was found that the rate of shoreline change due to sand loss at the pond differs only by a maximum of 7% from that experienced assuming no sand loss.

The wave transformation shoreline evolution model was applied to the model grid using the 20 year long term average wave climate described earlier. We should point out the enormous number of calculations involved in this simulation, including the determination of wave height, length and direction at 10,735 points, the specification of breaking wave characteristics at each point along the shoreline, calculation of wave alterations due to diffraction near the four groins, and determination of the longshore transport rate at each nearshore location; all of these calculations repeated for each of the nearly 500 different wave fields representing the 20 year record. Finally, the model uses the time and spatial history of the longshore transport rate to determine the resulting changes in the shoreline position at each alongshore location.

Long-Term Shoreline Changes with a Net Drift to the Southwest

The model simulation indicated a 20 year net transport rate of 173,000 yd^3/year toward the southwest for existing conditions and a net transport of 186,000 yd^3/year toward the southwest for the modified groin condition.

In an effort to determine the influence of the two Federal groins on shoreline erosion and accretion patterns, we illustrate in Figure 4 the rate of shoreline change for both the existing and modified conditions. The groin locations, numbered as in Figure 4, are indicated in the figure. Position values of shoreline change indicate accretion and negative values indicate erosion. For the net southwestward transport system modeled here, one would expect sand accretion to the northeast (the right in Figure 4) and sand erosion to the southwest of each groin. Figure 4 illustrates this pattern, with both the existing and modified conditions resulting in alternating erosion and accretion in the vicinity of the structures.

The accretion and erosion rates found at the two Federal groins under existing conditions are significantly higher than those found at the shorter State groins. Furthermore, by shortening the two groins, the accretion and erosion rates are reduced and the shoreline would readjust to achieve relatively small rates of change.

Shoreline Changes During Eastward-Dominated Drift

The model was applied using the modified wave record giving a net transport toward the northwest. At before, the model was implemented for both existing and modified (shortened) Federal groins. The resulting net transport rates were 282,000 yd^3/year toward the northeast for existing conditions, and 304,000 yd^3/year toward the northeast for modified conditions.

Figure 5 illustrates the resulting patterns of shoreline

SHORELINE RATE OF CHANGE
EAST HAMPTON BEACH

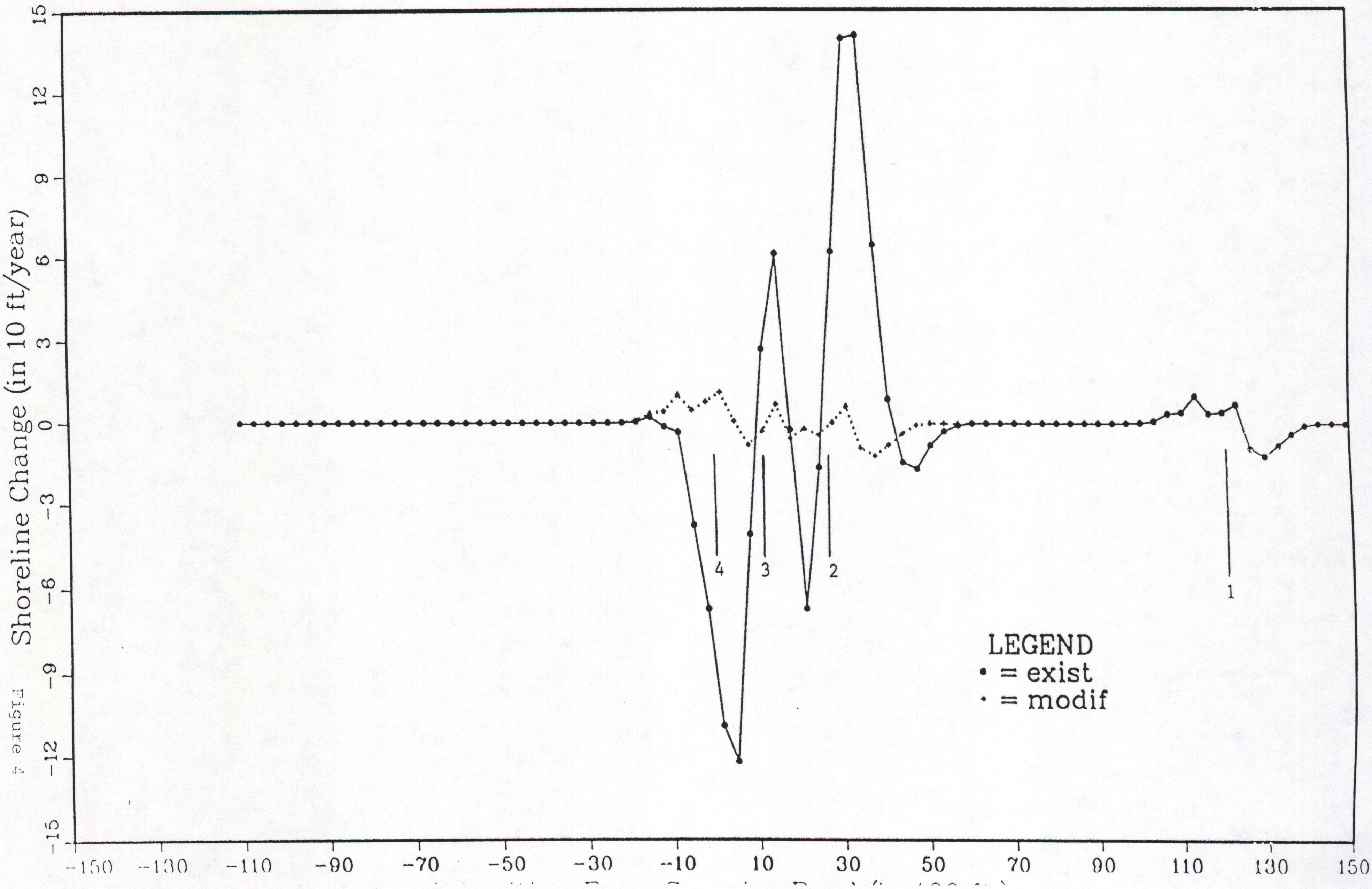

Figure 4

SHORELINE RATE OF CHANGE

EAST HAMPTON BEACH–Eastward Drift

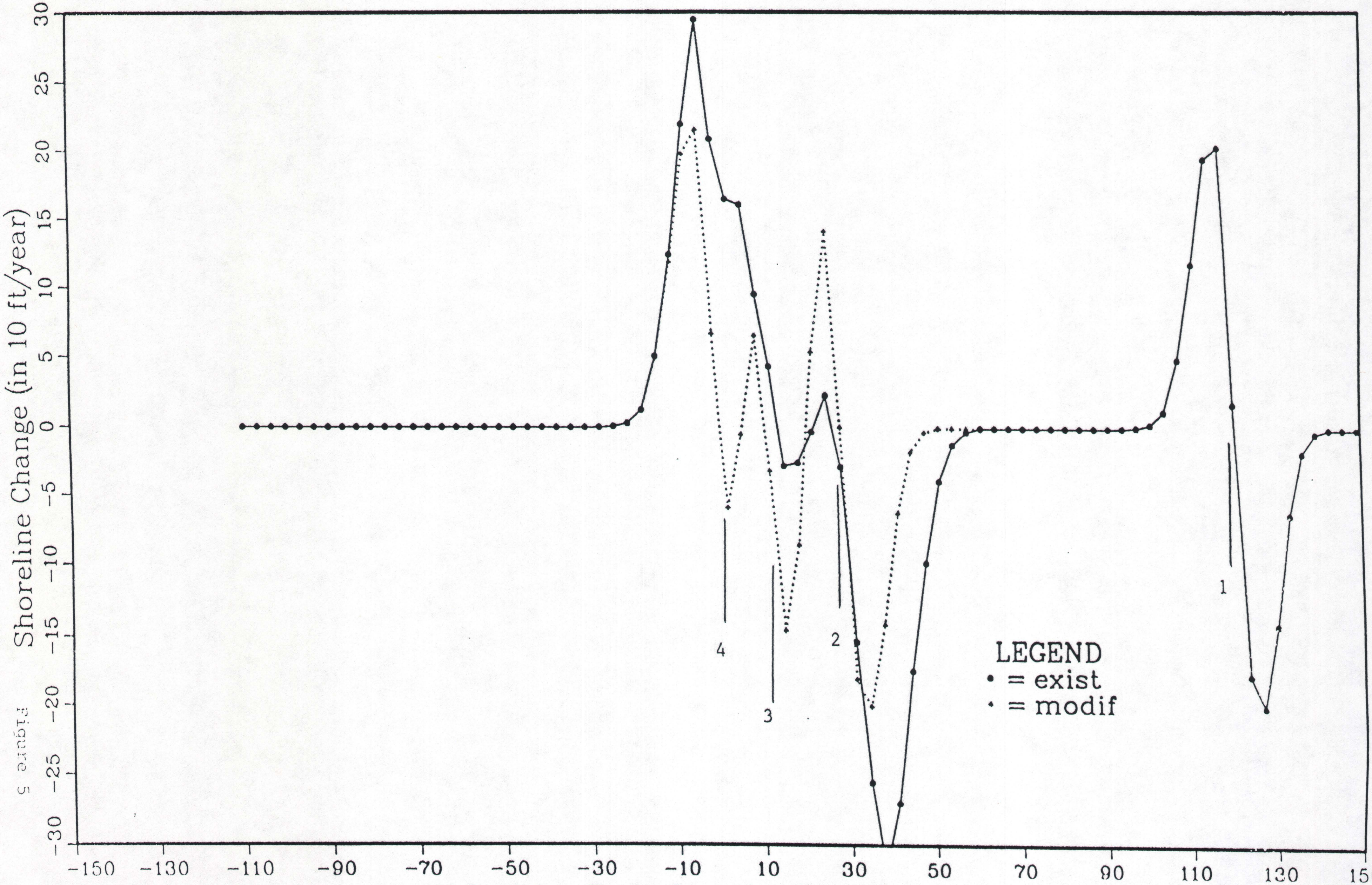

Figure 5

change rates. We note two major points: the reversed pattern of accretion/erosion compared to the southwest-dominated case discussed previously; and the much increased rates of shoreline change compared to the earlier result. For this application, however, we are more interested in the differences incurred by shortening the groin rather than the absolute magnitude of the event. The size of the changes, however, does suggest that the beach near the groins is slightly more sensitive to the eastward drift conditions. This is likely to be related to the distribution of large transporting events but such sensitivity is not fully explained by the model. Again from this figure the extent of shoreline fluctuations is reduced by shortening the two Federal groins.

Short Term Shoreline Changes During Storms

The shoreline change as a result of storm wave attack is another straightforward test of the hypothesis that the long Federal groins cause increased erosion of the adjacent (downdrift) beaches.

We employed two storm wave fields in this model application. Both storms were assumed to have a duration of two days. Both were characterized with an average deepwater wave height of 6 ft and an average wave period of 8 sec. However, the first storm was assumed to produce waves approaching from the east, at an angle of 50^{o} counterclockwise to the shore-normal; the second storm was assumed to produce waves approaching from the south, at an angle of 50^{o} clockwise to the shore-normal.

Rather than present the alongshore variations in rate of shoreline change, it was decided to here present the post-storm shoreline position relative to a pre-storm straight coastline condition. As before, the model runs were repeated for both existing and modified Federal groin conditions.

Figure 6 illustrates the post-storm shoreline positions for the storm waves approaching from the east. As expected, both the modified and existing conditions exhibit patterns of accretion (positive shoreline position) to the east and erosion (negative values) to the west of each groin structure. The extent of erosion is slightly higher at the Federal groins for the existing lengths than it is for the modified, shortened lengths, maximum erosion being 3.9 ft and 3.3 ft, respectively.

Figure 7 illustrates the post-storm shorelines for the storm waves approaching from the south. Note that, as expected, the patterns of erosion and accretion have reversed, with erosion now occurring to the east of each groin. As before, the shoreline change is approximately 15% to 20% greater for the existing, long Federal groins condition.

Conclusion

The extensive computer modeling effort has led to the following conclusions regarding the beach and groin structures at East Hampton:

1) The longshore sediment transport is sensitive to the occurrence of short-term extreme events i.e., storms). This makes any prediction of future long-term net transport very difficult, a conclusion that in itself

SHORELINE POSITION

EAST HAMPTON BEACH–Waves From East

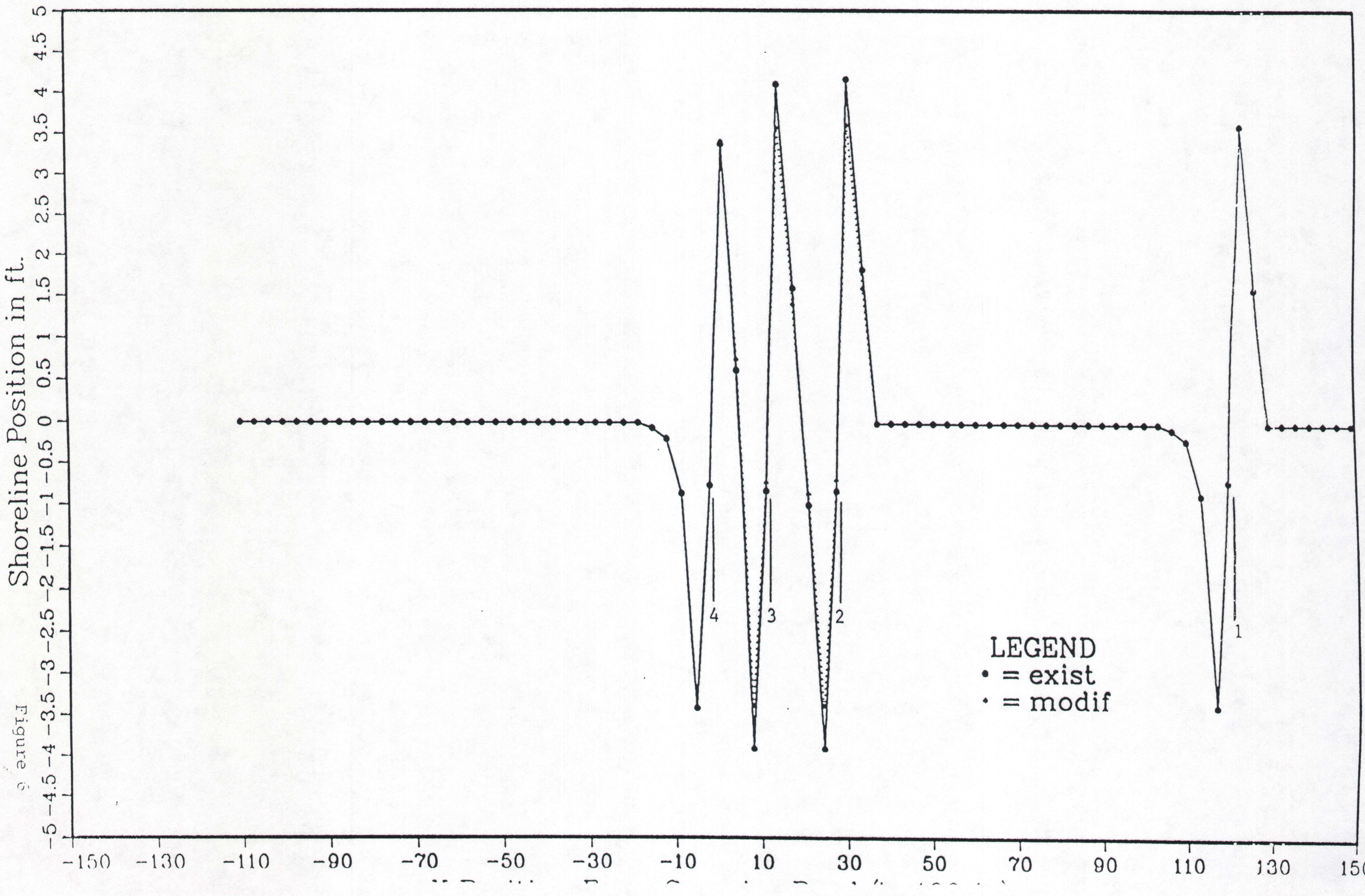

Figure 6

SHORELINE POSITION

EAST HAMPTON BEACH–Waves From South

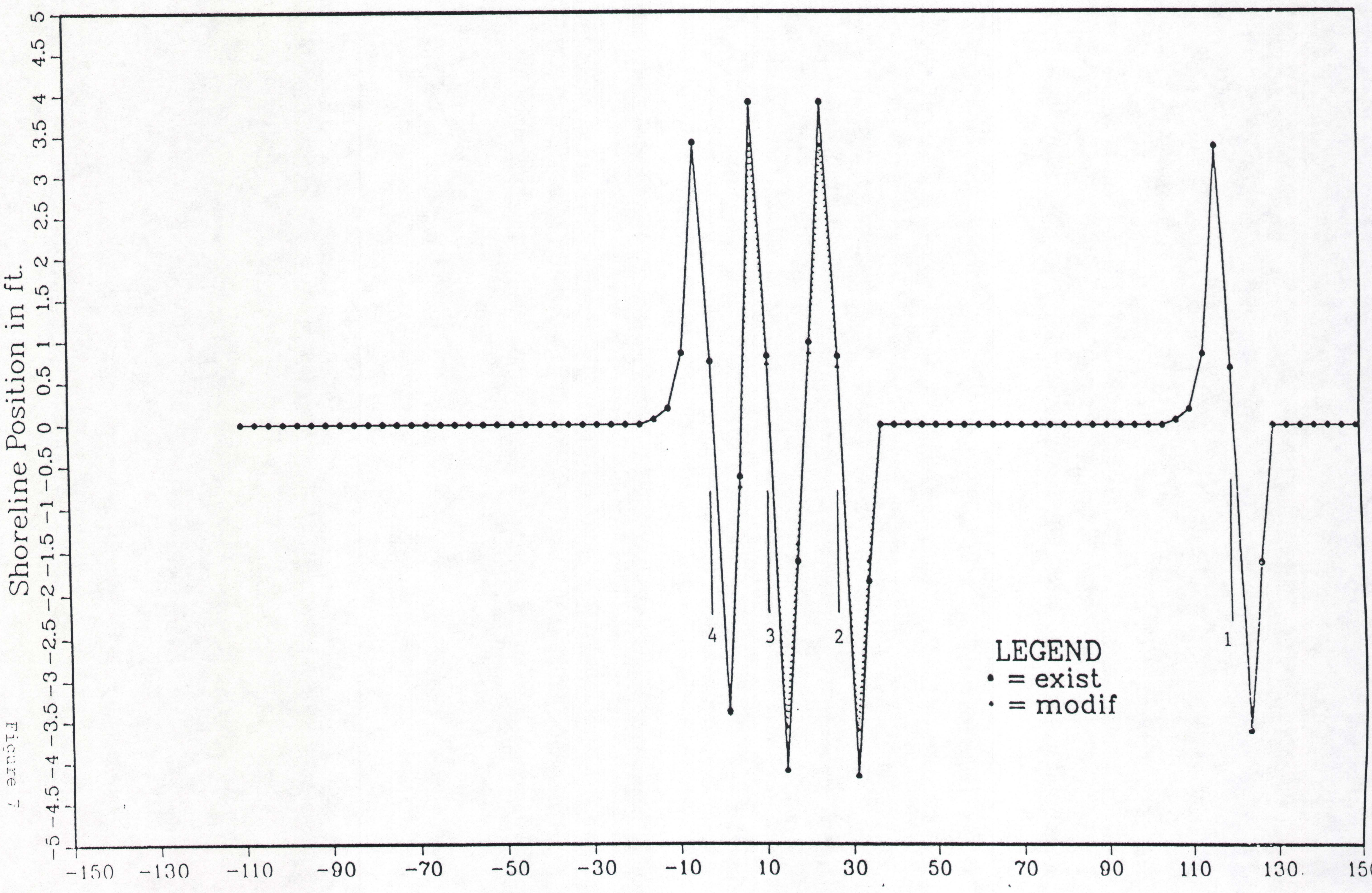

Figure 7

warns of the dangers of constructing a long groin barrier to sand transport.

2) For both east and west-moving transport, the long Federal groins produce rates of shoreline fluctuations considerably higher than those found in the vicinity of the shorter State groins.

3) Reducing the length of the two Federal groins lessens the impact on adjoining beaches, particularly over the long term. It is therefore suggested that the two groins either be: a) shortened to 275 ft long; or b) notched at the outer 205 ft by removing the upper layers of stone, so that waves (and sediment) can pass across the seaward end of each.

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point of wave breaking. Recently, field and laboratory experiments have shown a dependence of the breaking point on the wave period and the slope of the ocean bottom in the direction of wave travel. We shall here employ a methodology (Weggel, 1972) which includes the effect of bottom slope variations on the the breaking point.

In order to numerically solve equations (1) through (3), the equations are cast into finite-difference form and solved on a rectangular grid system which describes the ocean region in question. In the nearshore region, the effects of diffraction around coastal structures, and the possibility of wave breaking are also determined. Model input includes the initial (deepwater) wave characteristics, the water depth at each gridpoint, the bottom friction factor (a function of the bottom sediments), and the location and size of any coastal structures. The final product of the wave transformation model is the nearshore (or, alternatively, breaking) wave characteristics at each alongshore location within the model grid. This information can then be employed in the estimation of the longshore sediment transport rate.

Estimation of Longshore Transport Rate

Waves approaching the shoreline at an angle to the coast drive a mean current parallel to the shore. This current, commonly referred to as the longshore current, has the potential to transport bottom sediments in the alongshore direction. Although the physics governing the initiation of motion and subsequent transport of bottom sediments are fairly wellunderstood, the estimation of a total volumetric sediment transport rate remains an active area of research. For this reason, the longshore transport rate is commonly estimated with the use of experimentally-derived equations which relate the total sediment flux to the nearshore wave characteristics. In the present model formulation, we employ the well-known Army Corps of Engineers formula:

$$Q = \frac{K H_b^2 C_g \sin(2\theta)}{16(s-1)(1-a)} \tag{4}$$

where H_b is the wave height at breaking, s is the ratio of sand density to water density (usually, s = 2.65), a is the sand porosity (usually on the order of 0.4), K is a constant, and all other parameters are as described earlier.

The constant, K used in Equation (4) typically has a magnitude on the order of 0.5, with the Army Corps Shore Protection Manual (1984) suggesting K = 0.39 when using significant wave height and K = 0.77 when using root mean square wave height. In a study of the longshore transport rate in the vicinity of Sea Bright, New Jersey, Kraus et. al. (1988) estimated K = 0.41. A similar analysis by Bruno (1988) in the vicinity of Manasquan, New Jersey, found K in the range 0.28 to 0.45. A tracer study in the vicinity of Harvey Cedars, NJ, resulted in the calculation of an average value of K of 0.92.

Tracer Study of Longshore Transport

On June 14, 1988, a tracer experiment was conducted to assess the longshore transport characteristics along Harvey

Cedars. The procedure used in the experiment is commonly referred to as the Spatial Integration Method. This consists of releasing a known quantity of tracer material at an initial time, t = 0, and across the surf zone at an initial alongshore location, y = 0. At specified time intervals, and at known locations across the surf zone (x direction) and along the shoreline (y direction), core samples of the bottom sediments are then retrieved. Through an analysis of the tracer concentration at each sampling point, the alongshore position of the center of mass of the tracer "cloud" can be determined at each time, and the mean velocity of the sediment motion can be estimated. By inserting the resulting estimate of the sediment transport rate, Q, into Equation (4) along with measurements of the sediment and breaking wave characteristics, we obtain an estimate of the constant, K.

Success of the experiment depends in large part on insuring that the entire zone of sediment motion is represented in the choice of sampling stations. In addition, the tracer material must be chosen so that it behaves similar to the native sediments being studied. In the experiment described here, the tracer material was obtained by coating local sediments with a fluorescent dye. The resulting coated sand particles were then crushed and separated until the grain size distribution closely resembled that of the native material. This tracer material can be safely assumed to behave in the same manner as the bottom sediments at the study area. The fluorescent pigment makes the determination of tracer concentration at each sampling point a relatively simple task.

We obtained an average velocity, $V = 0.84$ ft./min. over the study period. Using visual observations of the sample cores, we estimated an approximate depth of moving sediment as $z = 0.2$ ft. Our samples indicated tracer movement at a distance of up to 400 ft. from the shoreline. We therefore estimate $Q = 67.2\ ft^3$ /min. $= 1.12\ ft^3$ /sec.

Measurements of the breaking wave characteristics were taken throughout the experiment. These measurements indicated an average breaking wave height, $H_b = 1.6$ ft., an average wave period, $T = 3.8$ sec., and an average wave approach angle, $\theta = 16.33^o$. Inserting our estimated value of the longshore transport rate and the measured wave characteristics into Equation (4), we obtain an average value for the constant, $K = 0.92$. We shall employ this value of K in the estimation of the longshore transport rate during our computer modeling of shoreline evolution in the study area.

Shoreline Change Model

We have already described the theoretical basis for the wave transformation and sediment transport model to be employed, as well as the empirical estimation of the constant, K, used in the longshore transport formula. The final stage of our modeling effort will be the estimation of the variation in shoreline position brought on by temporal and spatial fluctuations in the longshore transport rate. In order to model these changes, we shall make use of the "one-line" model of shoreline evolution. In this model, the rate of change of the shoreline position is a function of the alongshore variation of the longshore transport rate, Q:

$$\frac{dr}{dt} = -\frac{1}{d_0}\frac{dQ}{dy} \tag{5}$$

where r is the position of the shoreline and d_o is the depth of zero sediment motion. The magnitude of d_o is estimated from an empirically derived relation (Hallermeier, 1979):

$$d_0 = 2.28\ H_0 - 10.9\ (H_0^2/L_0) \tag{6}$$

where H_o and L_o represent the deepwater wave height and wavelength, respectively.

In the case of a coastal structure (e.g., a groin) extending into the surf zone, the total volume of sediment passing is reduced so that we must alter the magnitude of the longshore transport rate determined from Equation (4). This is accomplished by determining a "bypassing factor", B:

$$B = (1 - d_s/d_0) \le 1 \tag{7}$$

where d_s is the water depth at the end of the structure. The longshore transport rate at the structure is then determined as: $Q_s = B\ Q$, where Q is the longshore transport rate determined assuming no structure.

The final numerical model to be used in our examination of shoreline evolution trends in Harvey Cedars therefore consists of a wave transformation model, a longshore transport model, and a shoreline change model. The complete model, cast into finite-difference form, requires as input a model grid which details the bottom bathymetry of the study area. The grid was constructed from NOAA chart 12324. The grid boundaries, superimposed on the NOAA chart, are illustrated in Figure 1. The water depths were digitized from this chart onto a rectangular grid having a resolution of 100 ft. in the offshore direction and 300 ft. in the alongshore direction.

Estimation of 20 Year Wave Climate

When examining nearshore wave processes with the aim of predicting shoreline change patterns, it is desirable to use as input a deepwater wave climate that is representative of recent conditions and (as best as possible) resembling the wave climate to be expected during future periods. This task is best accomplished by assembling a data set detailing daily, seasonal and long-term trends in wave characteristics in the study area. For this reason, and because of the sparsity of wave data along much of the coastline of the United States, the U.S. Army Corps of Engineers has conducted a hindcasting study for the entire U.S. coastline, for the period 1956 to 1975 (Jensen, 1983). This study, termed the Wave Information Study (WIS), provides deepwater wave information at 3 hour intervals over the entire 20 year period. Figure 2 illustrates the hindcasting positions in the mid-Atlantic region, with three points (24, 27 and 28) located off of the New Jersey shoreline. We shall make use of the WIS data set at hindcast point 27, this point being essentially east of the study area.

Model Applications

As stated in our introduction, the aim of the present study is to assess the longshore transport patterns along the beach at Harvey Cedars, and to examine the effectiveness of structural solutions to erosion trends in the area. A model run was performed assuming no interception, or trapping of alongshore-moving sediment. In this fashion, the "natural" longshore transport rate for the study area can be examined. The input wave field consisted of the entire, 20 year WIS hindcast. The results indicate a longshore transport rate, Q, which varies substantially in magnitude and direction along the shoreline. Such variation is indicative of a "nodal zone", where the longshore transport diverges, being southward on one side and northward on the other. If a nodal zone truly exists within this region of the coastline, it is most likely induced by alongshore variations in the nearshore bathymetry of northern Long Beach Island. When examined in a reach-averaged sense, that is, averaging over all of the nearshore gridpoints, we obtain a net longshore transport rate over the 20 year period of Q = 481,000 cubic yards per year, directed to the south.

We shall examine the influence of the Bergen Avenue groin in Harvey Cedars. This groin is extremely long, with a design length of 250 ft., and so has the potential for behaving as a barrier to the longshore movement of sediment. The complete, wave transformation - shoreline evolution model was run, this time including the Bergen Avenue structure. We ran the model for two input wave conditions, both with deepwater wave height 3 ft. and wave period 8 sec. (typical local storm-generated waves), but one with an approach angle of 60° to the shore-normal (from the northeast) and the other with an approach angle of -60° to the shore-normal (from the southeast). The model was also run for the same deepwater wave conditions, but without the groin structure. Figures 3 and 4 illustrate the resulting pattern of shoreline evolution for each deepwater wave condition, with and without the Bergen Avenue groin. Note that the updrift side of the groin experiences accretion and the downdrift side experiences erosion in each case relative to the no-groin situation. If this trend were to continue, the updrift beach would reach its limiting width (the length of the groin) while the downdrift beach would continue to erode. In the case of an exceedingly long groin such as this, the outer limit of the structure lies in deep water relative to the depth of zero motion for most wave climates, so that little if any sediment "bypasses" the groin to reach the downdrift beach. This situation is clearly detrimental to the preservation of the beachfront, and warrants notching, or cutting down some of the outer length of the groin.

In our final model application, we ran the full model using the entire 20 year data set as input. Two cases were examined, the first assuming no coastal structures, and the second including the Bergen Avenue groin. Figure 5 illustrates the comparison of the shoreline evolution pattern for the two cases. In the vicinity of the structure, we again see erosion for the situation including the groin, relative to that predicted without a structure. The patterns of erosion and accretion, however, are here much more complicated than those observed for the simple, single wave climate employed earlier (Figures 3 and 4). Periods of southerly and northerly wave approach angles have produced alternating accretion and erosion along the shoreline. The magnitude of these fluctuations is enhanced by the presence of the groin, because of its behavior (for the majority of the wave events) as a nearly complete sediment transport barrier, as well as diffraction effects in the lee of the structure.

CONCLUSIONS

We have performed a wave transformation and shoreline evolution modeling study for the beach at Harvey Cedars, New Jersey. A net, southerly sediment transport of approximately 481,000 cubic yards per year has been estimated using a 20 year wave data set. It was found that the transport rate fluctuates appreciably along the shoreline, with some reversals in net direction. These reversals may be indicative of the presence of a nodal zone in the region. The existence of a nodal zone would suggest that extreme caution should be used in the siting and design of any structural beach stabilization device, especially a groin field. This caution is necessary because of the unpredictability of the net longshore sediment transport in the area, making the long term consequences of a coastal structure difficult to assess.

The erosion influence of a long groin such as that which presently exists near Bergen Avenue has been illustrated in model applications using both single wave and the full 20 year wave climates. Our results indicate that the use of a groin which extends to a water depth greater than the most commonly observed depth of zero sediment motion will have deleterious effects on adjoining beach areas, both in the short term (storm wave attack) and in the long term. Our model applications indicate that the depth of zero motion for the majority of wave events in the 20 year data set is less than 5.0 ft. As an approximate guideline, therefore, any groin extending to depths of approximately 5.0 ft (mean water) or greater should be cut down or notched at the outer end in order to allow for the movement of sediment around or over the groin tip and to the downdrift beaches.

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Georgica Pond:

Natural Processes and Human Modifications

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Introduction

Georgica Pond in East Hampton is actually a slow-moving river with its mouth often blocked by a barrier beach. Georgica is one of seven such rivers (e.g., Mecox Bay in Southampton being another with similar dynamics) along the South Shore of Long Island, New York. In addition to direct precipitation, most of the water in Georgica Pond flows into it through the porous glacial sands of the adjacent areas. In particular, Georgica serves to expedite the flow of water from the interior pine barrens to the Atlantic Ocean.

The Georgica Pond barrier is naturally breached during hurricanes and large winter northeasters. Legend holds that the Native Americans "let" or dug open an inlet through the barrier during the spring and fall to improve the fishing. This practice has been continued over the centuries, and now is the common practice of the Trustees of the Freeholders and Commonalty, proprietors of Georgica Pond.

It is widely believed that the seaward portion of Georgica Pond has shoaled considerably during the last several decades (Mr. Don Petrie and Mr. John Guldi, personal communication, 1991). Shoaling of the Pond along its ocean boundary is a natural process. The 1938 hurricane breached the barrier beach and resulted in a tremendous amount of sand deposition in Georgica Pond as clearly shown on post-storm aerial photography (Leatherman, 1989). As the overall position of the south shore of Long Island beach retreats landward over time, the barrier beach is forced to rollover by inlet and overwash processes. Wind-blown sand from the beach and adjacent, devegetated dunes, also contribute some sand to the Pond. The flood tidal delta (shoal area in the Pond) is the platform of the barrier beach and is paramount in the rollover process. However, these processes can be exacerbated by poor management practices, including the problem of timely closure of the artificial breach.

Pond Shoaling

Recent aerial photographs (Figure 1) show the extent of the shoaling in Georgica Pond, resulting primarily from inlet breaching and overwash (overtopping) processes. As the sand delta builds, the freshwater flow from the Pond through the barrier to the ocean is more impeded, which tends to increase the water levels in Georgica. In recent years, the Pond levels have reach new heights, flooding the basements of local residents and prompting a feasibility study for a Pond dredging project. Mr. John R. Guldi, Suffolk County Engineer, conducted the only known survey of water depths in Georgica Pond. These soundings, undertaken in 1988, indicate the degree of siltation (actually shoaling as little silt-sized material is present in the delta) in Georgica Pond (Figure 2). It was proposed that 150,000 yd^3 of sediment be dredged to lower the Pond levels. Calculations by Leatherman (1989 letter report to the Group for the South Fork), however, indicated that the proposed dredging would not greatly increase the flow-through rates of the groundwater nor significantly lower the water levels in Georgica Pond in a timely manner. Therefore, the barrier beach was artificially breached in August 1989 to drain off the excess water (6.6 feet above mean sea level on July 21, 1989 as measured by Mr. Don Petrie).

To estimate the amount of shoaling, December 1986 photography was used to determine the overall size of the shoaled area (Figure 1). Comparison to earlier photographs of Georgica Pond clearly indicated that the shoals have become more "solid" land through continued sedimentation (Leatherman, 1989). Scaling from this photograph, the sand shoal extends approximately 1,000 feet into the Pond and is about 1,500 feet wide. The dredging survey (Figure 2) indicates a maximum of 3.5 feet of sand deposition above mean sea level (MSL). It is also known that the general depth of Georgica Pond is less than 3.5 feet below MSL. Therefore, the shoaled area contains a maximum of 389,000 yd^3 of sand assuming the maximum dimensions of the flood tidal delta. This amount of sand is about half of the total gross longshore sediment transport or about twice the net annual littoral drift along the East Hampton beach.

East Hampton Groins

The East Hampton groins introduce a perturbation in the system, causing marked downdrift erosion along the Georgica Pond beachfront (Leatherman, 1989). If storm activity occurs during the time of a narrowed barrier (because of the westward shadow zone, especially evident in the winter months), then increased overwash activity can result. However, there is clearly a limit to the Pond shoaling, and a point of diminishing sand accumulation has probably been reached as the barrier beach effectively widens (Figure 1).

While there are no known historical surveys of Pond depths, local citizens maintain that Georgica has shoaled considerably in the last 25 or so years since groin construction (Mr. Don Petrie, personal communication, 1991). Previously the water was reportedly deep enough to dive into from the backside of the barrier; now this area is large tidal flats, exposed except during times of high Pond water. Enlargement of the sand delta through accelerated overwash processes is in response to barrier narrowing by groin-induced sand starvation. While annual rainfall ultimately controls Pond level, the greater amount of sand impedes the water flow-through, resulting in new heights in Pond levels as observed in recent years. While it is not possible to assess completely the role of the groin field, the overall effect is undoubtedly negative due to the constraints placed on a dynamic system by a static structure.

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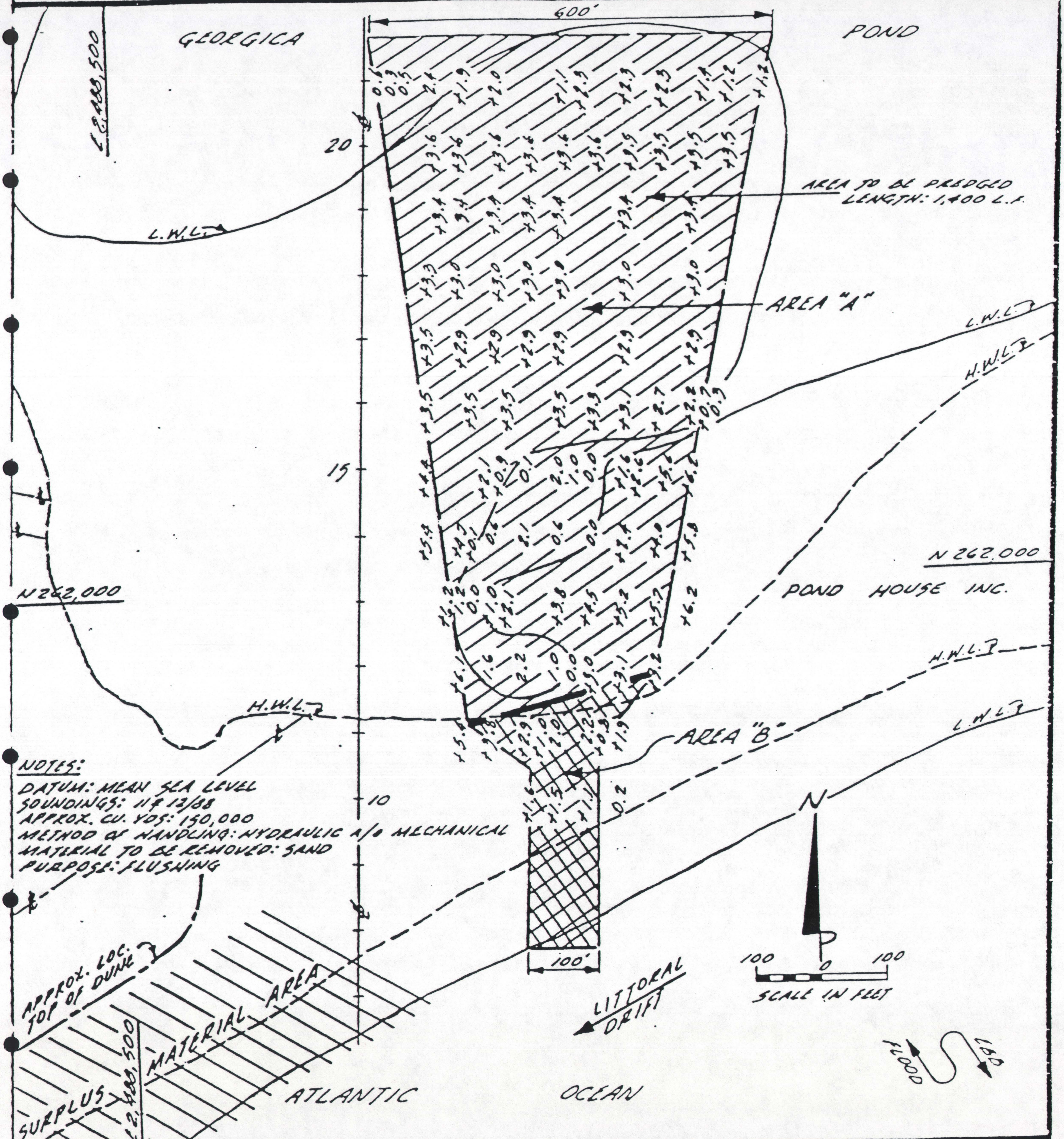
Figure 1



DATE: 2/89

GEORGICA POND - TOWN OF EAST HAMPTON
SUFFOLK COUNTY, NEW YORK
APPLICANT: SUFFOLK COUNTY DEPARTMENT OF PUBLIC WORKS

SHEET: 2 of 4

Figure 2

Longshore transport rates at East Hampton, New York

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Introduction

The rate and direction at which sand is transported along the shore at East Hampton is of practical importance because of the presence of several large groins. Mathematical models may be used to evaluate the impact of these structures, or to assess the shoreline's response to changes in these structures. The results of these models are based on calculations of the rate of longshore transport using the wave characteristics at the shoreline. There are no measurements of the wave characteristics at East Hampton, however, so they must in turn be estimated in other ways. The most sophisticated estimates available are predictions, called hindcasts, made from the meteorological data over a 20-year period (Jensen, 1983). Hindcasts are available off the eastern shore of Long Island.

At another location the hindcasts have been shown to accurately reproduce the wave heights (Miller and Jensen, 1990). The predictions of the wave period were less accurate; the forecasted wave periods tended to be about 2 seconds too low. The measured wave directions were not as uniformly distributed as the hindcasted directions. Nevertheless at this location the measured net longshore drift was fairly well represented by the prediction even though the gross drifts up and down the coasts were off by a factor of two (Miller and Jensen, 1990). This situation is one of concern for the prediction of longshore transport since the rates are sensitive to the directions. To decide if the concerns are justified for a particular situation, like at the beach at East Hampton, the model predictions should be matched to other, independently made, estimates of the site-specific longshore transport rates. It is the purpose of this article to discuss the characteristics of the longshore transport at East Hampton.

Regional Estimates

The asymmetric geometry of tidal inlets along the south shore of Long Island indicates a net longshore drift of sand from the east to the west. Johnson (1957) cites a value of 200,000 yd^3/yr to the west for the south shore of Suffolk County as determined from the accretion rates near inlets. By a similar technique, L. McCormack (1979, personal communication) has estimated a value of 150,000 yd^3/yr to the west in the vicinity of East Hampton. It is generally agreed that the rate of transport increases to the west and estimates of 300,000 yd^3/yr to the west are calculated from the migration rate of Moriches Inlet (U.S. Army Corps of Engineers, 1957; Taney, 1961). Panuzio (1968) gives an estimate of 300,000 yd^3/yr to the west.

The long-term, net transport rate along the shore, however, does not adequately capture the important features of the sand motion at East Hampton. Here both the eastward and the westward sand transport rates are much larger than the net transport. Examination of the single groin at Hook Pond shows that sand has accumulated on both sides of the groin approximately equally. In the presence of a strong, persistent net drift to the west, we would expect to see an accumulation of sand on the eastern side with erosion on the west. Similarly, the beach immediately to the east of the large groins at Georgica Pond suffered severe erosion between 1976 and 1979 presumably due to a persistent eastward net drift during that period. This location then accumulated sand for the next 10 years. Since 1988, however, an erosional trend has again been established, and a station immediately to the east of the easternmost groin has been losing sand at a rate of about 33 yd^3/foot of shoreline per year.

Observations at East Hampton

During a study of the behavior of East Hampton's beaches, visual wave estimates were made and used to calculate the longshore transport rates (Bokuniewicz, et al., 1980). Observations were made nearly every day and sometimes twice a day from October 1979 through May 1989; from January 1981 through August 1981; and from January 1982 through August 1982.

Observations of wave height, period and angle of attack were made from shore using a technique similar to that used in other areas of the county by the U.S. Army Corps of Engineers, Coastal Engineering Research Center, under the Littoral Environmental Observation (LEO) Program (DeWall, 1977; Bruno and Hiipakka, 1973; Schneider, 1971). Since there were no other independent, simultaneous measurements of the wave conditions made, the quality of the visual observations is virtually impossible to assess. Since a great number of observations are available, however, it is reasonable to expect that errors would have been random and tend to average out.

The wave period was easiest to measure. It was done twice during each observation. The wave height was probably the next most accurate measurement. Fairly broad divisions were used for classifying the wave heights because the largest changes are the most important. Wave heights were classified as "less than one foot", "between one and three feet", "between three and five feet", and "over five feet or about ____ feet". The angle of wave attack was the most difficult measurement to make. Two methods proved practical and both were used during each observation. One method involved plotting the angle of attack on a protractor that was used printed on each data sheet. This method was used by the LEO program. In the other method, the observer would chose a figure that best represented the wave conditions on that day from a series of seven on the data sheet. The angles measured by the first method were consistently larger than angles measured by the second method, sometimes by as much as 10°. After volunteers were supervised completing the data sheets, it seemed that the second method (choosing one of the figures) was more likely to be consistently done correctly so that this method might give the better results. Measurements made on 2 May 1980, however, proved otherwise. On that day, a set of aerial photographs was taken while the beach survey was being done. The wave crests were easily seen on the photographs and appeared to have been approaching the shore at an angle of about 15°. The values measured by the observer on that day were 15° and 28.5° by the first method and 5° by the second. In this case, the use of the protractor gave the better results. Since one method was not clearly superior to the other, the separate results were treated as independent observations; the analyses were done separately with both measured angles and the difference treated as an uncertainty in the results.

The wave energy flux was calculated in two ways. One uses the breaker height and the angle of attack (equation 4-35 in the "Shore Protection Manual", U.S. Army Corps of Engineers, 1977). The other uses the wave breaker height, the period, and the angle of attack (equation 4-28, ibid). The average total energy flux was then converted to a rate of longshore sand transport as recommended in the "Shore Protection Manual" (equation 4-40). Each period of observation was less than a full year but the results were adjusted to represent an annual transport rate assuming that the observed conditions would be similar to those occurring when observations were not made.

The average conditions calculated for each period were:

Period	Eastward Drift Frequency %	Eastward Drift Amount yd^3/yr	Westward Drift Frequency %	Westward Drift Amount yd^3/yr	Net yd^3/yr
22/10/79 - 31/5/80 (221 days)	45	295,973	38	436,007	180,032 westwardly
1/1/81 - 31/8/81 (243 days)	52	765,510	28	497,008	268,502 eastwardly
1/1/82 - 27/8/82 (239 days)	59	686,475	39	375,614	310,861 eastwardly

The uncertainties in these values, as estimated by the multiple calculation described above, were about ±50%. Despite these uncertainties, three important characteristics of the longshore drift at East Hampton were described by these observations. These are:

1. Eastward transport of sand was observed more frequently than the westward transport. Over the entire observation period, conditions causing eastward transport were seen 52% of the time while conditions causing westward transport were seen 35% of the time. (During the remaining 13% of the time, wave angles could not be distinguished from zero.)

2. The gross drift (i.e., the amount of sand moved in both directions) was about four times larger than the net drift.

3. The net westward drift of 180,032 yd^3/yr observed in 1979-80 was consistent with the long-term regional estimate but the eastward drifts averaging 289,682 yd^3/yr calculated for 1981 and 1982 is in accord with observed conditions at East Hampton.

The application of the hindcasted wave characteristics to a model for East Hampton shoreline resulted in an annual eastward drift 67% of the time amounting to 341,534 yd^3yr, an annual westward drift 33% of the time amounting to 278,348 yd^3/yr, and a net drift to the east of 62,773 yd^3/yr (Bruno, 1991). These results are reasonable. To better account for the range of conditions at East Hampton, however, it was decided that the model results be modified to produce (a) a net westward drift of about 180,000 yd^3yr in one case and (b) a net eastward drift of 290,000 yd^3/yr in another case in order to examine the observed pattern of transport.

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