Shoreline Assessment and Recommendations for a Long Range Erosion Plan for the Sand Beach Conservancy District, Ottawa County, Ohio.

Prepared for:

Sand Beach Conservancy District
Oak Harbor, Ohio

Prepared by:

Matrix Engineering, Inc.
16817 Madison Avenue
Lakewood, Ohio 44107

August 1994

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I. INTRODUCTION

This study was prepared by Matrix Engineering, Inc as a generalized shoreline assessment analysis and long range erosion protection plan for Sand Beach, Ottawa County, Ohio. Technical analyses and designs were developed by Christopher Andrassy in consultation with Matrix Engineering, Inc. The study objective was to define the controlling processes and mechanisms of shoreline retreat and profile erosion and recommend coastal engineering alternatives for beach creation, stabilization, and modest storm protection.

Sand Beach is located in Ottawa County on the shoreline bulge that is known as Locust Point. It is located between the mouths of the Toussaint River to the east and Turtle Creek to the west, approximately 10 air miles northwest of Port Clinton. Sand Beach consists of approximately 7300 ft beginning at the rock jetty at the west end and extending east. Figure 1 is a vicinity map of the Sand Beach area.

Since the early 1970's, and especially following the severe November 1972 storm, Sand Beach has suffered from a general lack of beach above lake level. Since that time, individual homeowners have constructed bulkheads and/or rock revetments to provide protection from storm waves and prevent further erosion of their property. The current armored portions of the shoreline are considered stable but there is no expectation of a beach naturally establishing itself. This study will document the lake processes and conditions that created the eroded condition. Based on a 30-year design lifetime and general cost-restricted limitations, alternatives will be provided to create a recreational beach and to provide modest storm-wave protection along the present nonbeach areas.

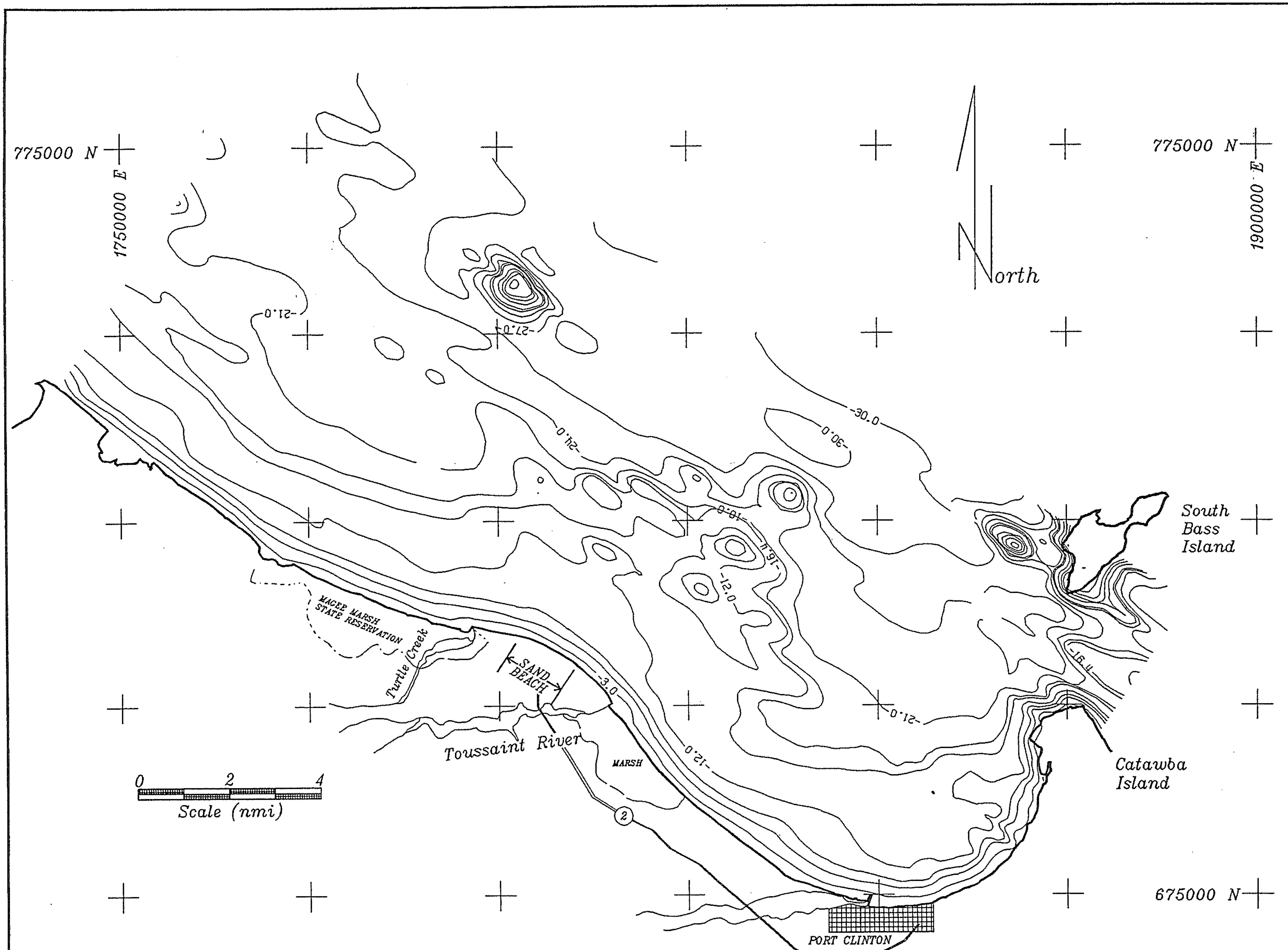


Figure 1. Sand Beach vicinity map.

II. PREVIOUS REPORTS AND STUDIES

There were no previous reports or studies conducted by the Ohio Department of Natural Resources (ODNR), US Army Corps of Engineers (USACOE), or other government agency or private engineering group which specifically dealt with Sand Beach. However, there are a number of studies and data sources developed for the western portion of Lake Erie which were useful in the present study. These are referred to throughout the text and listed in the *References* section.

III. LAKE PROCESSES

III.A. Water Levels

The Lake Erie water level is an important variable because it controls the landward limit and maximum breaking height of damaging storm waves. The lake level at any given time is a function of the total water volume in the upper Great Lakes system, diversion practices, winds, and regional barometric pressures.

The greatest control on lake level is exerted by the water volume in the system which is controlled by rainfall volumes over the lake itself and over the watersheds which drain into the lake. National Oceanic and Atmospheric Administration (NOAA) data reported in Carter (1973) shows the correlation of rainfall volumes and lake level. Seasonal effects result in the highest levels during the summer when runoff is greatest. The lowest level of the open water period is in early Spring, and the lowest annual level usually occurs in February.

NOAA measures the lake level at Cleveland and Toledo and the ODNR measures the lake level at Sandusky Bay in Sandusky. The mean annual lake level at Cleveland is shown in Figure 2 for the period 1860 to 1991. These data were digitized from a graph supplied by ODNR. The graph shows the extent of mean variation that has occurred historically. The average lake level for the entire period of record of Figure 2 is 570.6 ft International Great Lakes Datum of 1955 (IGLD), while the average for the period since 1950 is 571.0 ft IGLD.

The lake level during a storm can deviate from the pre-storm level by up to 4 feet (Pore et al., 1975). This deviation is referred to as the storm surge and is the result of winds blowing over the lake for prolonged periods. Depending on the wind direction, duration, and intensity the water level at the terminal end of the lake will increase with a compensating decrease at the upwind end. Besides the storm surge, large breaking waves can also result in an increase in water level within the surf zone. This component of water level increase is the wave setup.

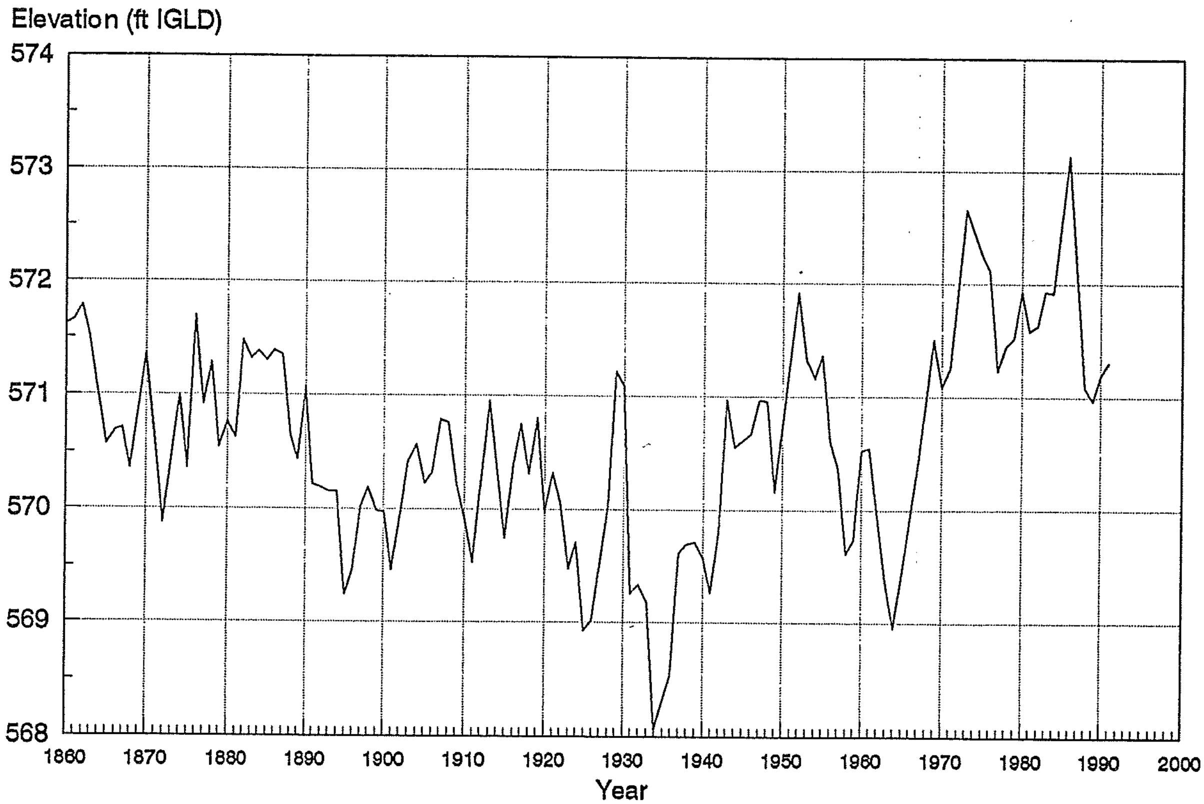


Figure 2. Mean annual historic lake level recorded at Cleveland by NOAA.

The Buffalo District of the USACOE has conducted a statistical study of maximum total water level to estimate the frequency of occurrence of extreme water levels along Lake Erie. The study was based on total water levels for the period 1961 to 1986. This information is presented in the form of return period total water levels. As an example, a water level with a return period of 10 years has a 1 in 10 chance of occurring in any given year. Table 1 lists various return period total lake levels at Sand Beach. Interpolating between the reported 10 and 50 year values for the Locust Point area yields a 30 year total water level of 575.8 ft IGLD.

T_R (yr)	level (ft IGLD)
10	575.2
50	576.4
100	577.0

Table 1. Total lake level and return periods.

III.B. Offshore Wave Conditions

III.B.1. Average Annual Conditions

The most extensive wave database for the western Lake Erie region has been prepared by the Coastal Engineering Research Center of the US Army Corps of Engineers (USACOE-CERC) (Driver et al., 1991). Using historic atmospheric pressure records and a computer model that generates wave conditions as a function of the over water wind fields, the wave conditions at various stations are determined. Wave characteristics generated in this way are referred to as hindcast waves, to distinguish them from wave characteristics which are measured directly. The nearest hindcast station to Sand Beach is Station 2, located at 41.73° N latitude, 83.08° W longitude, approximately 9 nmi north of Sand Beach.

Wave characteristics are reported every three hours for the entire 1956-1987 period of record. At each time step the significant wave height (H_s) peak period (T_p), and peak mean direction ($\propto$) values are reported. H_s is calculated according to $H_s=4\sqrt{E}$, where E is the spectral energy. T_p is the inverse of the frequency at which the peak energy occurs. The peak mean direction is an energy weighted mean of the directions associated with each of the discrete frequency bands in the spectrum.

The entire wave dataset was summarized and averaged to determine average annual and seasonal characteristics. The waves were first sorted according to direction to distinguish those travelling in an onshore direction from those travelling offshore which do not affect Sand Beach. Sand Beach is open to waves arriving from 303°-123° Azimuth (Az). Annually, 38.3% of waves at Station 2 are travelling onshore.

The onshore travelling waves were then processed to determine the discrete frequencies of occurrence for joint combinations of H_s, T_p, and direction. The annual and seasonal frequency tables are presented in the Appendix. Figure 3 is a graphic representation of the annual distribution of wave characteristics at Station 2.

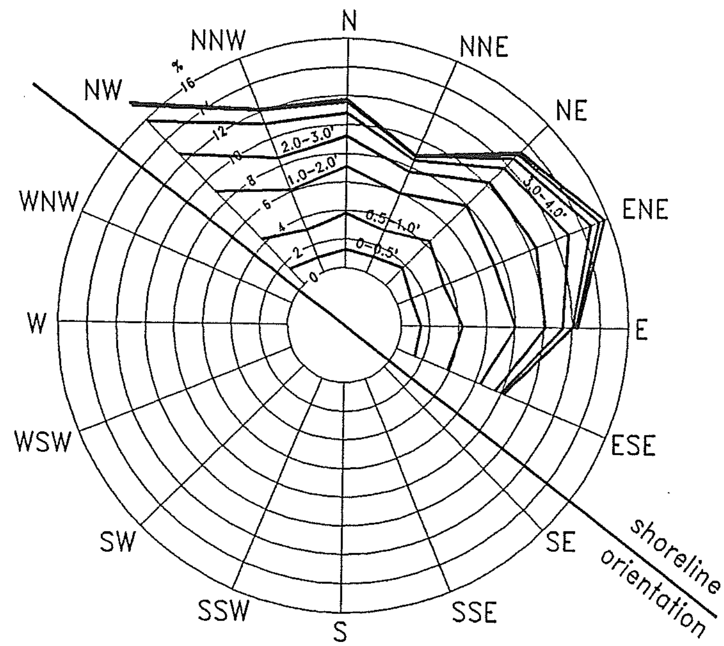


Figure 3. Annual distribution of onshore waves at Station 2.

III.B.2 Extreme Wave Conditions

The hindcast wave data of Station 2 were processed to examine the frequency of high waves, their period and direction, and frequency of occurrence. A computer program was written to read the 1956-1987 database and determine the annual maximum H_s values at Station 2, considering only onshore travelling waves. The results are presented in Table 2.

The maxima data of Table 2 are adequate for classifying the maximum wave heights and periods, considering the small range of values, but are too few for an adequate understanding of the predominant directions of storm wave approach. To provide a larger dataset of storm wave directions, the Station 2 dataset was searched for all H_s values ≥ 5 ft. The percentage of occurrence of these waves was calculated for direction bands of 11.25°. The frequency distribution of these storm wave directions is presented in Figure 4.

Figure 4 shows that the two dominant mid-band directions of storm wave approach at Station 2 are 56.25° and 67.50°. The weighted average predominant direction is 60.2° Az.

Date (yrmodyhr)	H(s) (ft)	T(p) (sec)	Dir (deg)
56031615	5.6	5.3	53
57040412	8.5	7.7	90
58022718	6.9	6.2	94
59032721	7.2	6.2	28
60032212	7.2	6.2	308
61030815	6.9	6.7	79
62030618	7.2	9.1	52
63040418	6.2	5.9	311
64011221	7.5	5.9	57
65011609	6.9	5.9	55
66042718	7.9	7.1	82
67040321	6.6	5.9	315
68031300	7.2	8.3	47
69041903	7.2	9.1	53
70032918	6.6	6.7	58
71022218	6.6	6.7	86
72040718	7.5	6.2	52
73042721	6.6	5.6	8
74040818	7.2	6.2	51
75101806	8.2	6.7	58
76042518	6.9	6.2	57
77120518	7.2	6.7	58
78050418	7.9	7.1	72
79022603	7.2	6.2	58
80031306	7.5	7.1	93
81041418	5.9	5.6	323
82040609	8.9	6.7	28
83040218	7.2	7.7	72
84022800	7.9	6.7	54
85030406	7.5	6.7	67
86020703	7.9	6.7	64
87040421	7.9	6.2	356
Average	7.2	6.7	
Minimum	5.6	5.3	
Maximum	8.9	9.1	

Table 2. Annual H_s maximums at hindcast Station 2.

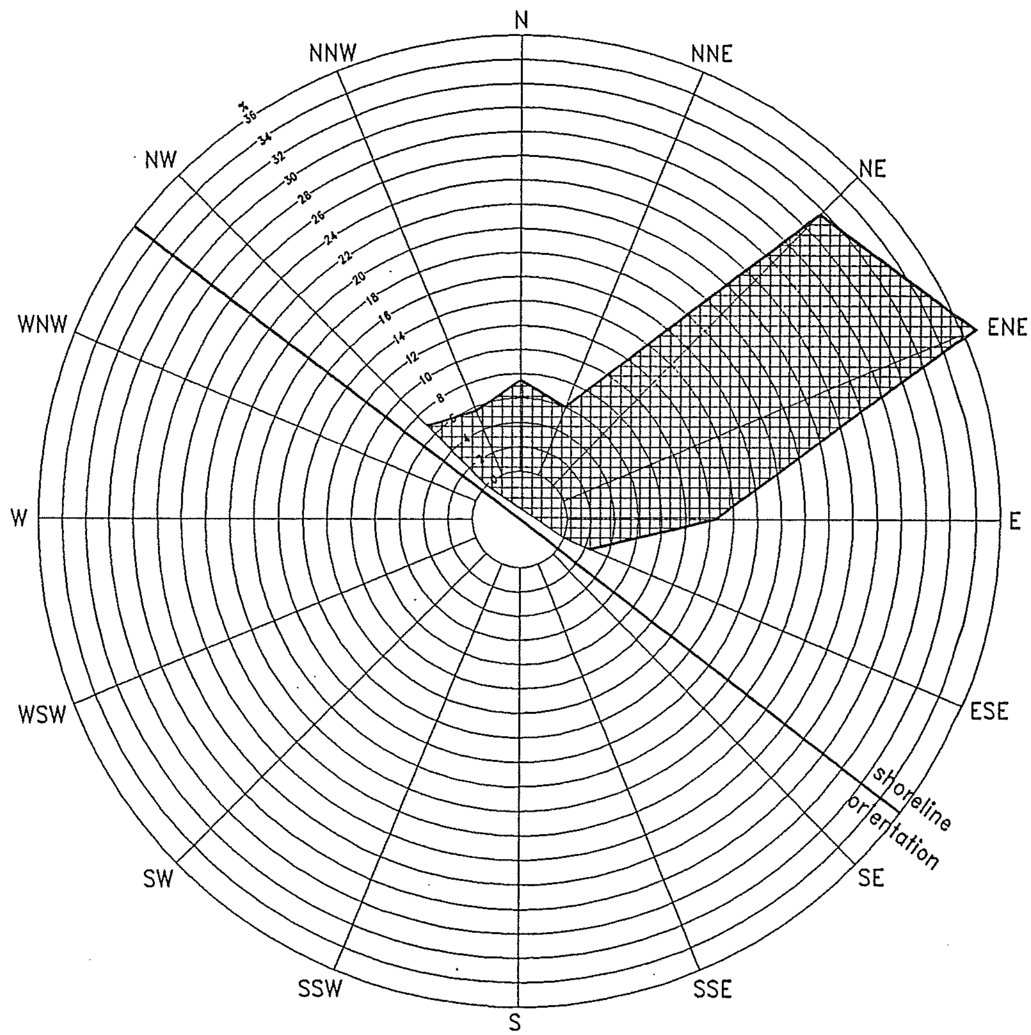


Figure 4. Distribution of storm wave directions at Station 2.

III.C. Nearshore Wave Characteristics

III.C.1. Wave Transformation Model - RCPWAVE

To develop estimates of potential longshore sediment transport rates, nearshore wave characteristics are required. The finite difference model, Regional Coastal Processes Wave Transformation Model (RCPWAVE), developed by USACOE-CERC (Ebersole et al., 1986) was used for this purpose. The model uses the wave conditions at the offshore site (Station 2) and calculates the wave height and direction for nearshore locations. Model requirements are a set of offshore wave characteristics and the regional bathymetry. Details concerning model development and internal algorithms will not be discussed.

Based on the distribution of wave approach directions, location of Station 2, regional bathymetry and shoreline orientation, model grid limits were established. At Sand Beach, where the average directional distribution is fairly uniform (Figure 3) the grid was oriented so the offshore axis was shore-perpendicular. A 50 x 50 rectangular element grid was established as shown in Figure 5. Grid cell dimension are 2814 ft x 1224 ft in the alongshore and cross-shore directions, respectively.

In Figure 5, the numbering along the grid axes is consistent with the numbering scheme used by RCPWAVE. The bathymetry shown in the figure was generated with an interpolation program using digitized sounding data from NOAA-National Ocean Service (NOS) chart number 14830. The chart datum is Low Water Datum (LWD), defined as 568.6 ft IGLD. The same interpolation program was used to generate an input file of water depths at each of the grid cell centers.

RCPWAVE uses one set of H, T, and α_o values for each transformation. It is unrealistic and unnecessary to run RCPWAVE for every set of H_s, T_p, and α_o contained in the offshore time series. Rather, a finite set of condition combinations can be used which will adequately describe the range of offshore conditions for the purpose of estimating potential sediment transport rates alongshore. The mid-band direction and period combinations run in

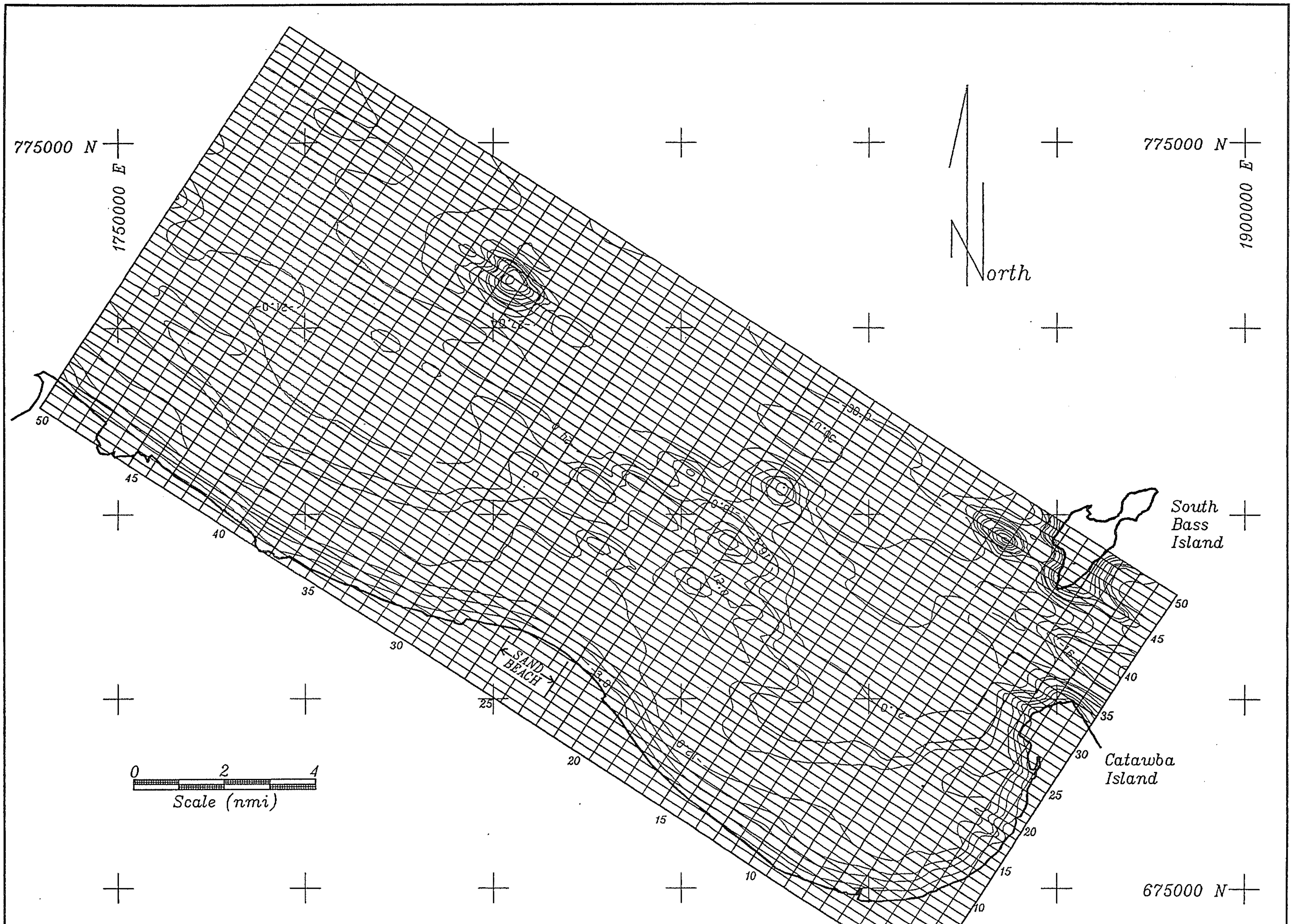


Figure 5. Regional bathymetry and RCPWAVE model grid centered on Locust Point.

RCPWAVE are presented in Table 3.

Direction (deg Az)	Period (sec)
309.0	2.5,3.5,4.5,5.5,6.5
337.5	2.5,3.5,4.5,5.5,6.5
360.0	2.5,3.5,4.5,5.5,6.5,7.5
22.5	2.5,3.5,4.5,5.5,6.5,7.5
45.0	2.5,3.5,4.5,5.5,6.5,7.5,8.5
67.5	2.5,3.5,4.5,5.5,6.5,7.5
90.0	2.5,3.5,4.5,5.5,6.5,7.5
112.0	2.5,3.5,4.5,5.5,6.5

Table 3. RCPWAVE input conditions.

RCPWAVE calculates output for each cell in the grid. Besides wave height, water depth, wave direction, breaker index, and wave phase function are reported at each cell. The breaker index indicates whether the wave has broken. Wave phase function is related to wave length and is described in greater detail in Ebersole et al. (1986).

RCPWAVE output for cells 23-27 (alongshore), cell 10 cross-shore, having water depths of 5-8 feet LWD were used to represent the nearshore conditions at Sand Beach. The results from each of the 46 input conditions were saved to calculate potential sediment transport rates.

A program was prepared to display the results graphically. It displays the wave direction and height at each cell using arrows. The arrow direction represents the wave direction and the length of the arrow is proportional to the wave height. Figure 6 shows a representative example for α_o=67.5° Az and T_p=5.5 sec. The cross-shore cells in Figure 6 are 1-40 and the alongshore cells are 5-44, corresponding to the cell numbers of Figure 5.

RCPWAVE operates most efficiently when the input wave direction is shore-perpendicular. For highly oblique cases the computations can become unstable. For these cases, the nearshore results from the closest (T_p,α_o) input combination are substituted.

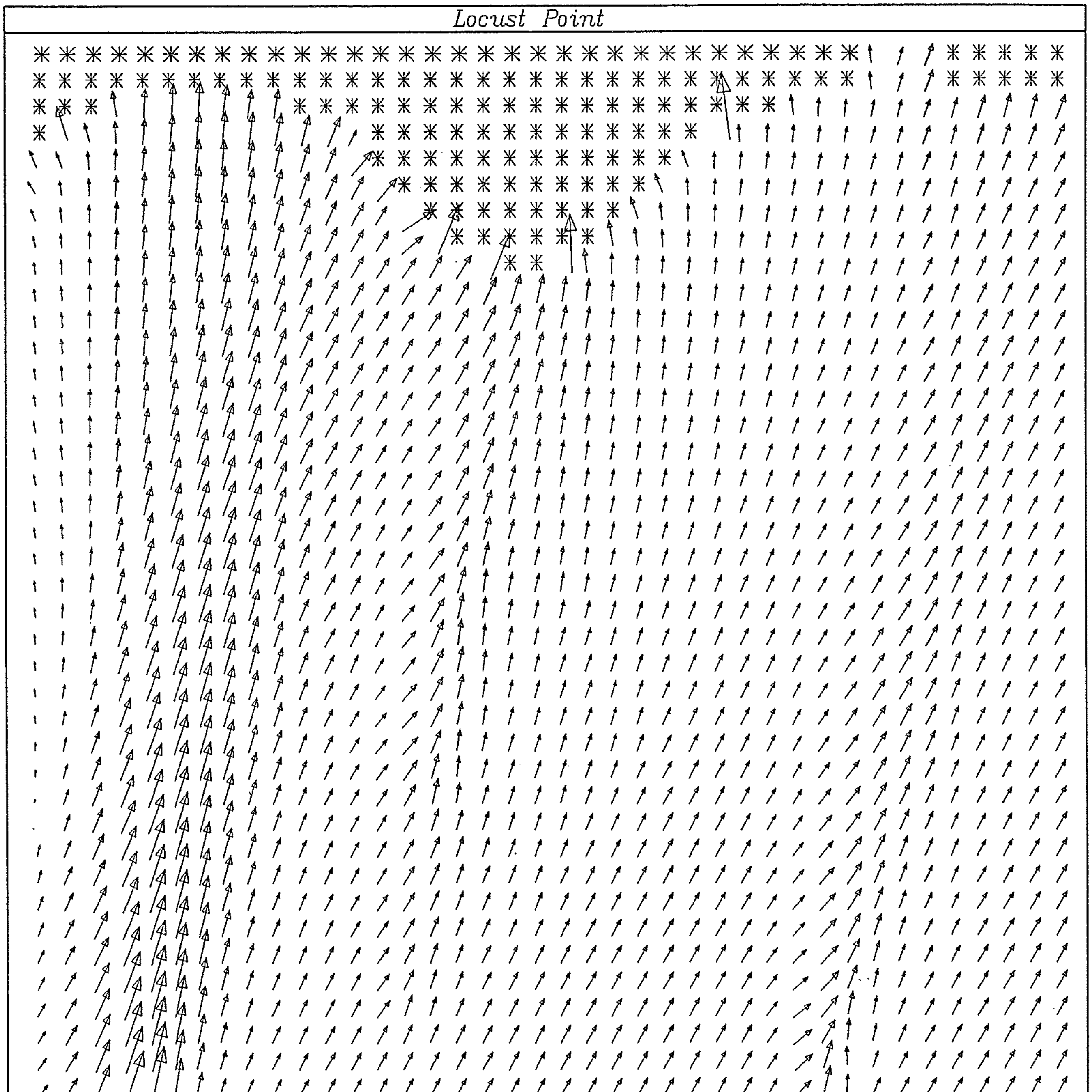


Figure 6. RCPWAVE output for T= 5.5 sec, °Dir= 67.5 Az.

III.D. Potential Longshore Sediment Transport Estimates

III.D.1. Importance of Longshore Transport Rate

Longshore sediment transport is defined as that sediment which moves along the coast under the influence of waves and currents. The longshore sediment transport rate (Q_1) is an expression of the volume of material passing a given shore point or area per time interval. Estimates of the gross and net potential transport rates are one the most important variables in a coastal processes analysis. Appropriate estimates of transport rates and alongshore rate variations are important to the proper design of a coastal engineering project. The calculation of the *potential* Q_1 was the basis for the detailed wave transformation analysis discussed in the previous section.

The gross transport rate is the sum of both upcoast and downcoast rates for the shoreline of interest. The net transport rate is the difference between the upcoast and downcoast rates. For net transport values, one direction is assigned as positive and the other negative. In this way the sign of the rate indicates its direction. At Sand Beach, transport to the east was chosen as positive.

An important consideration in the calculation of Q_1 is the difference between potential transport rate and actual transport rate. The potential rate is that which is calculated based on recognized empirical relationships developed by comparing measured transport rates with a measure of longshore wave energy for areas where there is <u>no deficit of sand supply</u>. In this case, actual and potential rates will be equal. However, when the sediment supply is deficient due to a lack of sand in the surf zone and/or from upland sources, or there is some sort of sediment sink intercepting the available transport upcoast of the study site, actual transport rates will be less than potential rates. Along most of Sand Beach, an analysis of historic beach profiles, shorelines, sediment samples in the nearshore, and previous studies indicate that there is little sand available to be transported alongshore.

Based on the sediment supply deficit at Sand Beach, only the net potential Q_1 values were calculated.

III.D.2. Calculation of Potential Transport Rates

Field studies conducted by coastal engineers and scientists have simultaneously measured longshore transport rates and nearshore wave conditions. These data have been used to develop empirical relationships between measures of the longshore wave energy and transport rates.

The empirical equation used in this study relates the wave-energy flux factor (P_{1s}) for incipient breaking conditions to the transport rate by multiplying P_{1s} by 7500 to obtain Q_1 in cy/yr (SPM, 1984). The transport rate was not calculated for Sand Beach due to its inherent uncertainty and the lack of sediment supply at Sand Beach. Instead, since P_{1s} is directly related to Q_1, it was used to examine the variation in potential Q_1 alongshore.

The basis for the relationship is described in the Shore Protection Manual (SPM) (USACOE, 1984). The empirical equation used to determine P_{1s} is:

$$P_{1s} = (\gamma/16)(H_s^2C_g)_b\sin(2\alpha_b) \qquad (1)$$

where: P_{1s} = longshore energy flux factor (lbs/sec)

γ = unit weight of water (64 lbs/cf)

C_g = wave group celerity (fps)

α_b = angle between wave crest and shoreline

The "b" subscript refers to breaking conditions.

A spreadsheet was prepared to first calculate P_{1s} as a function of the (T_p, α_o) combinations reported in Table 3. Then, the annual percentages of occurrence of offshore wave heights and the coefficients relating the offshore and nearshore heights were applied to determine the P_{1s} values for each (H_s, T_p, α_o) band. Finally, the P_{1s} values were summed and adjusted by the onshore travelling percentage of waves to yield the annual net P_{1s} value per nearshore cell.

The net transport rate results are shown in Figure 7. The near-zero P_{1s} value for cell 24 indicates that on average there is no annual tendency for net transport out of this cell. The adjacent

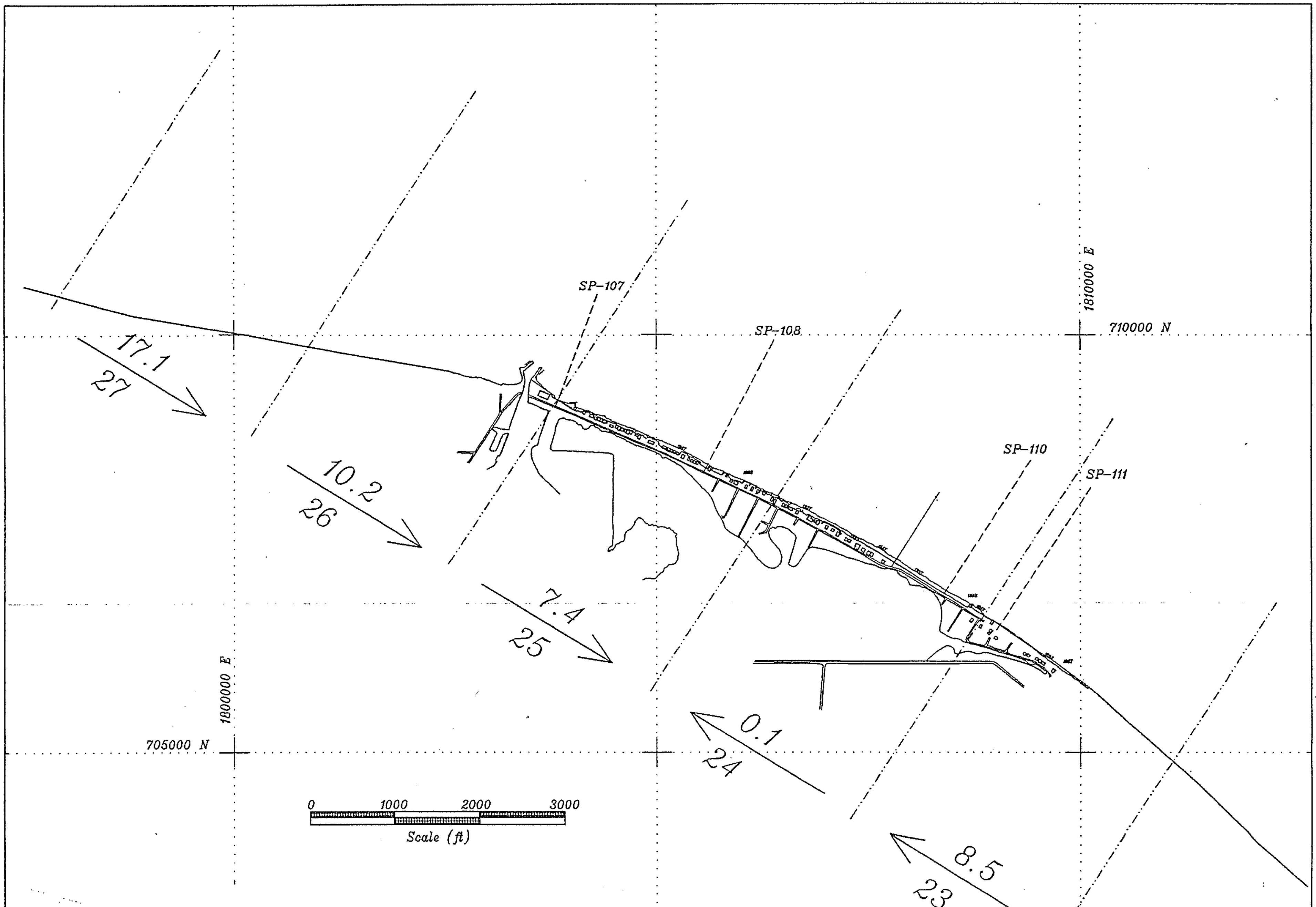


Figure 7. Variation of met annual longshore transport potential at Sand Beach. Values above arrows represent P(ls) and numbers below arrows are RCPWAVE cell numbers.

cell results predict transport directed toward cell 24 from both sides. This is the result of waves focusing on Locust Point. These results are discussed further in relation to the historic shoreline and beach profile change trends presented in the next section.

IV. SEDIMENT CHARACTERISTICS AND SUPPLY

The western region of the Lake Erie shoreline, including Sand Beach, is characterized by its lack of suitable beach material (USACOE, 1961). The available material comes from eroded glacial till, which in most cases occurs in thin layers in the nearshore overlying clay. ODNR staff (July 1994 personal communication) added that there are no major sand-carrying tributaries in the area. The major source of sand in the littoral zone had been upper profile erosion. The armoring of the shoreline at Sand Beach over the last two decades has effectively removed this source.

A suspended sediment study conducted in the early 1950's (USACOE, 1961) on the two major area tributaries, the Maumee and Portage Rivers, indicated that the suspended load contained only <1% and 3% sand, respectively. The remainder of the material was about 65% clay and 32-34% silt. The report stated that those distributions appeared to "apply equally well" to streams including the Toussaint River, located about 2.4 mi east of Sand Beach. In summarizing the area's shore erosion problem the report stated that "there appear to be no extensive deposits of sand which act as sources of supply for the littoral transport that takes place along the shore at various points." This analysis also applies to the 1994 condition.

The ODNR collected sediment samples at SP-108 in 1974 on the beach and at 100 feet from the shoreline. The grain size distributions of the samples were determined by sieve analysis. Table 4 presents typical statistical measures of the distributions and Figure 8 plots the discrete and cumulative size distributions. The general trend of fining in the lakeward direction is consistent with samples examined by USACOE (1961).

Statistical Measure	SP-108 (beach)	SP-108 (100 ft)
d_{50}	0.39 mm	0.15 mm
mean	0.43 mm	0.16 mm
stnd dev	0.98	0.48
skew	-0.37	-0.20

Table 4. GSD statistics for SP-108 samples.

During the July 1994 survey operations, sediment samples were collected from the upper beach and various offshore locations along the surveyed SP

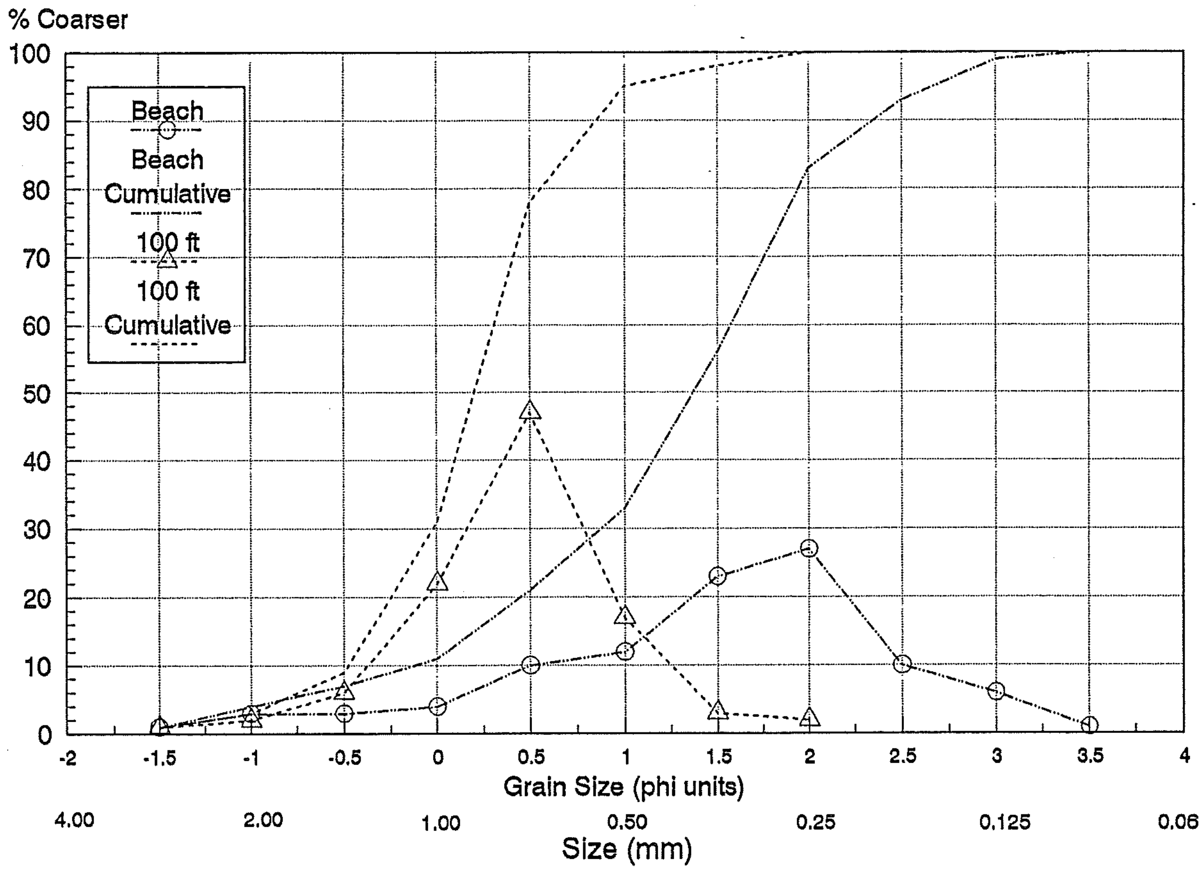


Figure 8. Grain size distributions of beach and 100 ft from shore samples taken by ODNR in 1974 at transect SP-108.

transects. Generally, the upper beach materials consisted of medium to coarse sand with some shell. These sand deposits occurred in thin layers overlying a coarse shell hash layer. The offshore samples were collected at 700 feet from shore at SP-107 and at 200 ft and 450 ft at SP-110. Repeated attempts to obtain a sample 700 ft from shore at SP-110 were unsuccessful due to the presence of stiff clay. At 700 ft from shore at SP-107 the bottom material consisted entirely of fluidized mud. At 450 ft from shore at SP-110 the bottom sample was approximately the same as the 700 ft sample at SP-107. At 200 from shore at SP-110, the bottom sample consisted of fine sand and silt.

V. HISTORIC SHORELINE AND BEACH PROFILE CHANGES

Historic shoreline positions and beach profile data are used to establish long-term change trends. The analysis was limited to the previous 50 years or so.

V.A. Historic Shoreline Position Changes

Aerial photographs taken in 1957, 1973, 1986, and 1993 were obtained from the ODNR for analysis. Photographic scale was 1:4800. The photographs were used to digitize the shoreline positions using computer drawing/drafting software. By superimposing the shoreline positions from these years on top of each other, a partial picture of recent decades' shoreline change history was obtained.

The photographs had no control points or coordinate grids superimposed on them to horizontally locate the shoreline or other physical features. Road intersections were chosen as arbitrary control points with a starting coordinate position assumed. This allowed an adjacent photographs' shoreline to be directly connected to the previous until the entire Sand Beach shoreline was digitized for each year.

The shoreline positions so determined are shown in Figures 9 to 11. The shorelines are unadjusted for water level. The average annual water levels for 1957, 1973, 1986, and 1993 were 570.4, 572.7, 573.1, and 572.6 ft IGLD, respectively. Since the 1973 and 1993 average levels were approximately equal, their shoreline positions can be compared directly. Using this level as a base, the 1957 digitized shoreline should be moved landward a distance equal to (2.2 ft)/(profile slope). However, an examination of beach profile slopes surveyed in 1956 indicates that the 2.2 ft elevation difference corresponds to a cross-shore distance of about 10 ft. Considering that the shorelines were digitized from photographs having a 1 in=400 ft scale, this difference is insignificant. The same argument applies to the 1986 shoreline, so that all the digitized shorelines may be compared directly.

Use of Figures 9 to 11 should be limited to a qualitative examination of shoreline change. The aerial photographs provided by

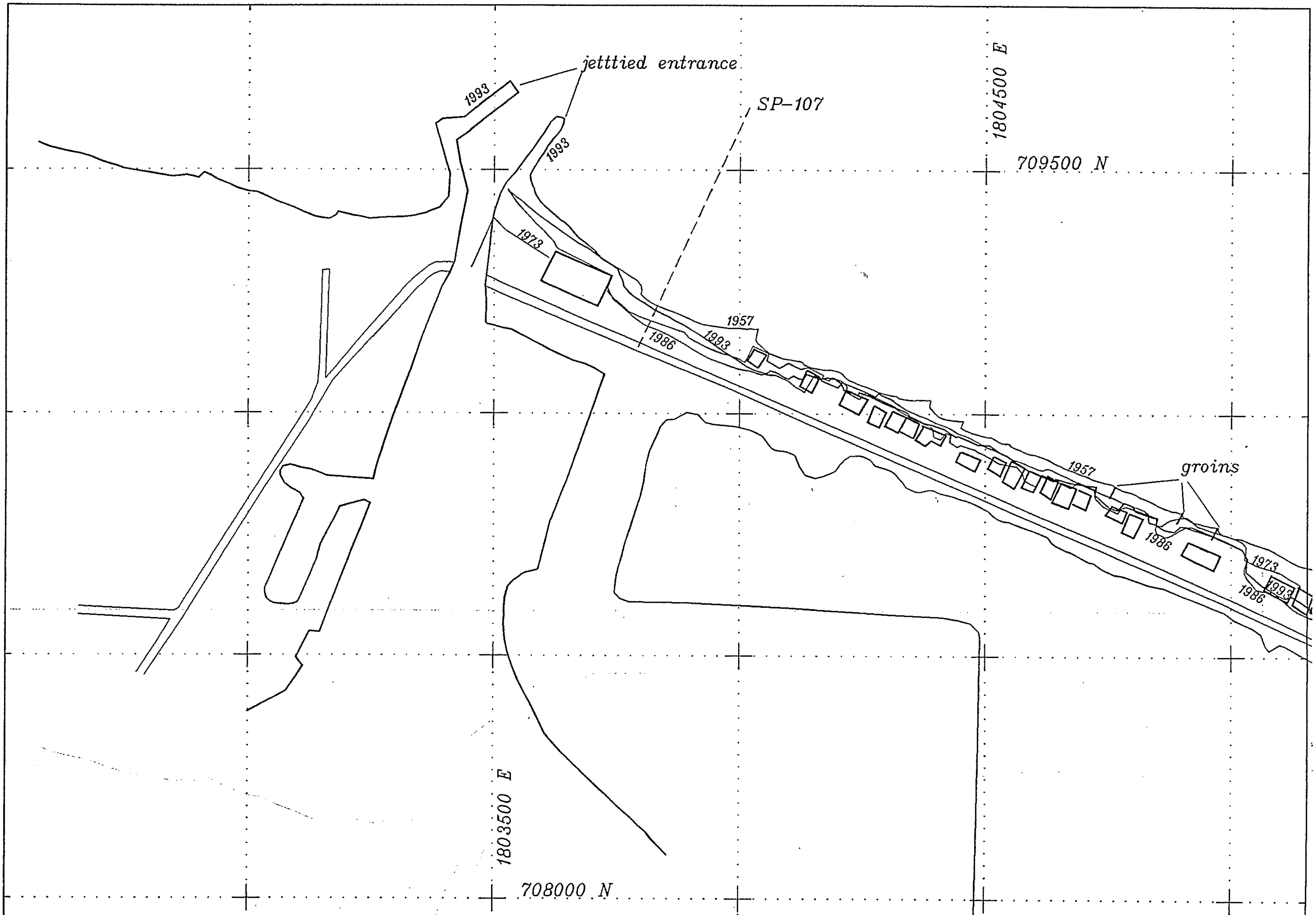


Figure 9. Shoreline positions from 1957, 1973, 1986, and 1993 at the west end of Sand Beach. House and road locations digitized from 1957 aerial photographs.

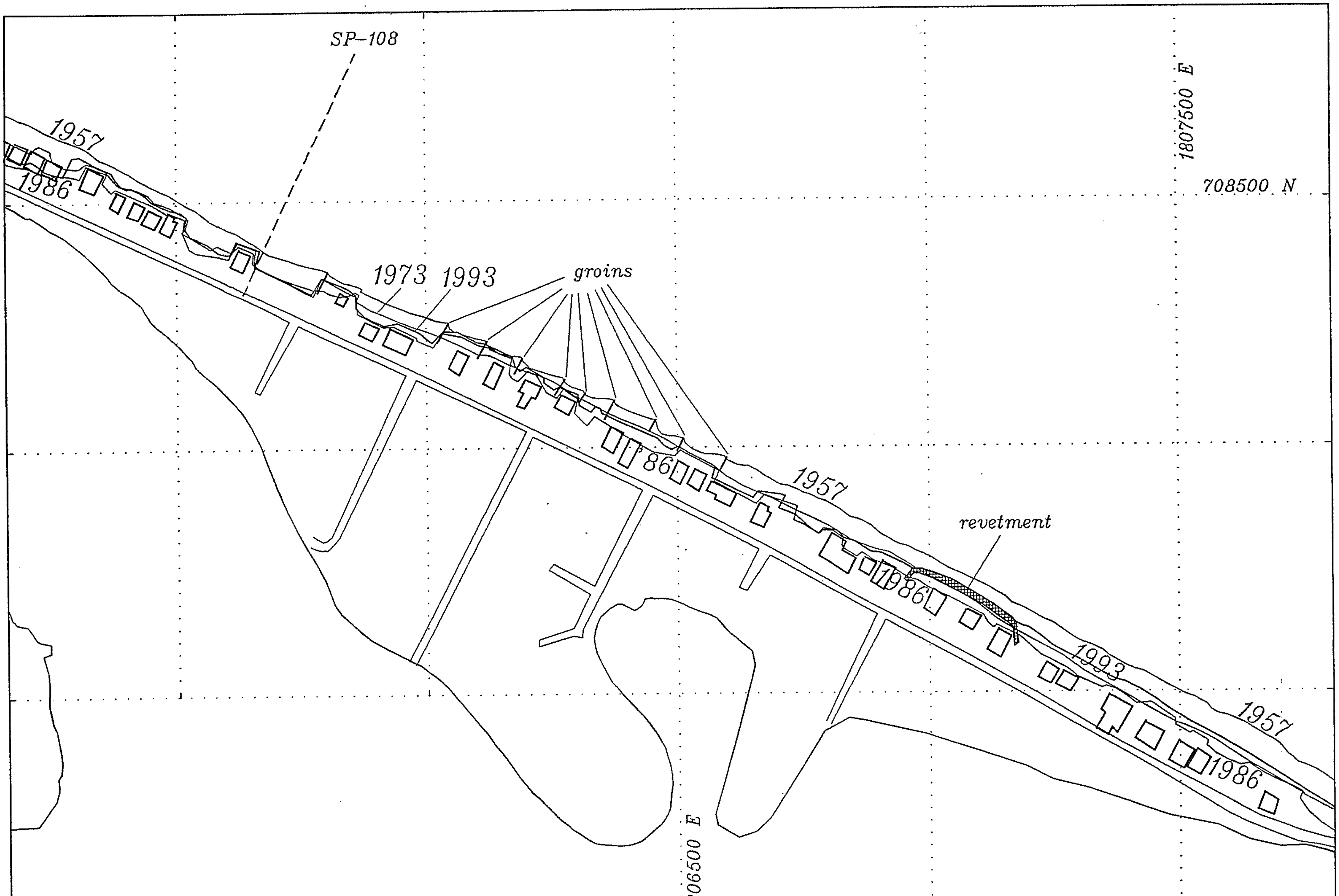


Figure 10. Shoreline positions from 1957, 1973, 1986, and 1993, at central portion of Sand Beach. House and road locations digitized from 1957 aerial photographs. Revetment installed after 1986.

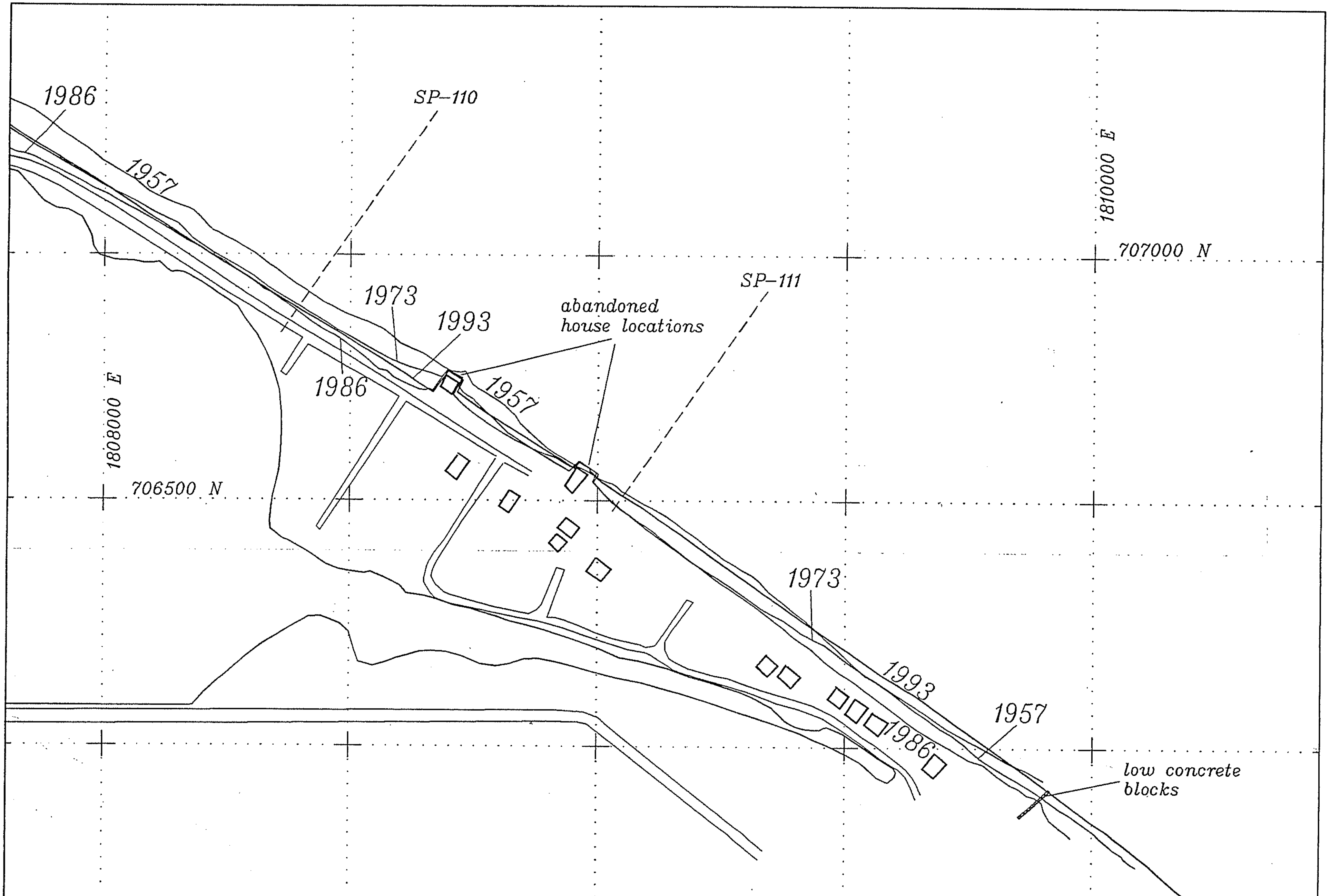


Figure 11. Shoreline positions from 1957, 1973, 1986, and 1993 at east end of Sand Beach. House and road locations digitized from 1957 aerial photographs.

ODNR made up only half of the entire set and so while they provided complete coverage the photographic distortion at the photo ends was significant. This tilt distortion also affected the position of shorefront homes which also cannot be considered reliable. Finally, the need to use poorly-defined road intersections for horizontal control also induced error in the process.

The following trends were observed. In 1957 there was a relatively wide beach along the entire Sand Beach shoreline. The next available shoreline position, 1973, shows that a large amount of erosion had occurred. This was probably the result of the November 1972 storm which had a peak water level of nearly 576.0 ft IGLD with the peak lasting for about six hours. The winds were from the northeast, a direction allowing for the greatest amount of offshore wave growth. Up to 10 feet of wave-induced erosion at Sand Beach due to this storm was cited by Carter (1973).

From 1973 to 1986 generalized shoreline erosion continued. Of the digitized shoreline positions, 1986 generally represents the most eroded condition. It is apparent that during this period even more erosion would have occurred were it not for the extensive shoreline armoring that was done by individual homeowners. This is reflected by the way the 1986 shoreline moves in and out to mirror the shorefront position of homes.

From 1986 to 1993 the shoreline changes over the central portion of Sand Beach have generally been insignificant. At the east end of Sand Beach however, considerable accretion occurred during this period. The western limit of this area appears to be at the large revetment shown in Figure 10. There was also some build up of the shoreline at the west end of Sand Beach adjacent to the marina's inlet jetty during this period.

V.B. Historic Beach Profile Changes

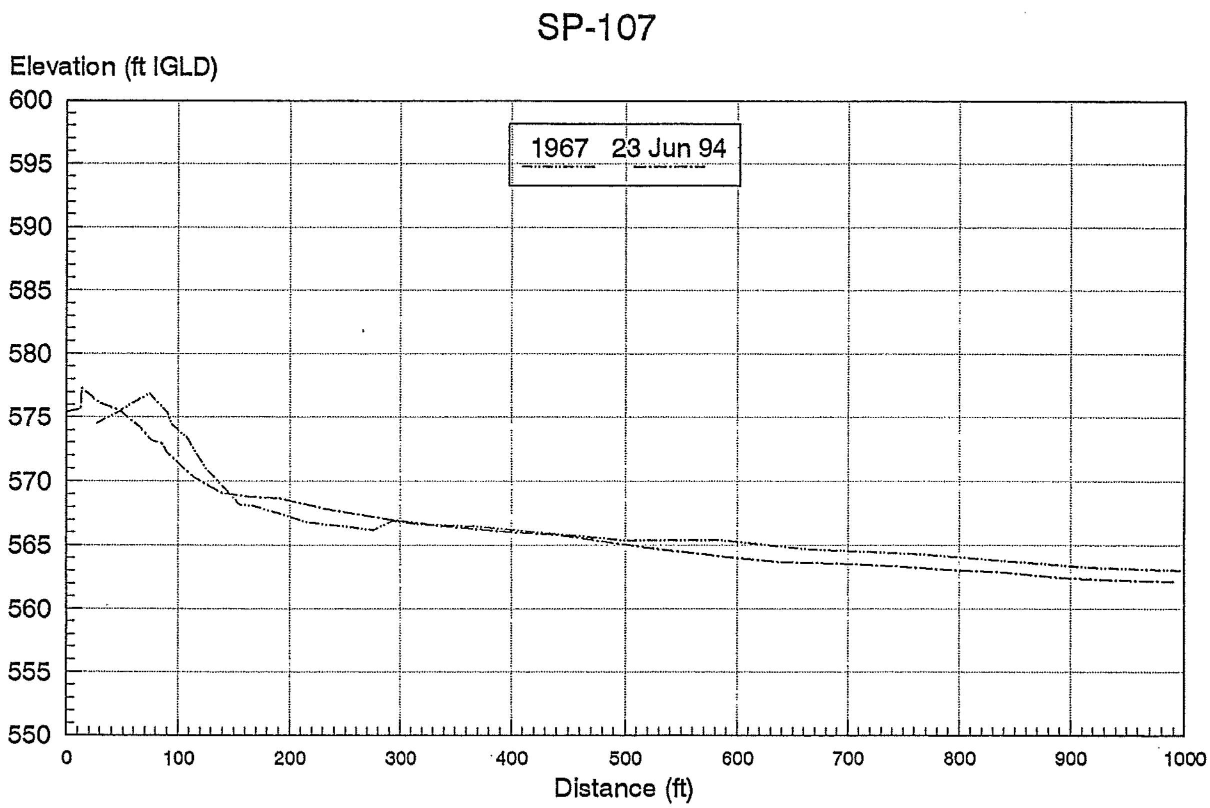
Beach profile transects along the western Lake Erie shoreline were established by USACOE sometime prior to 1943. Benchmarks were established with the transect locations referred to as "SP-#" where "SP" stands for shore point. Copies of the available plotted profiles were obtained from ODNR for this study. Figure 7 shows the transect locations in the Sand Beach area.

The profile data were digitized so they could be plotted together for comparison and for calculating unit volumes. The most recently available quality data was from 1973. To update the dataset, the transects were resurveyed in July 1994. Besides transects SP-107, SP-108, and SP-110, an additional transect was surveyed in the eastern portion of Sand Beach which is referred to as SP-111. This transect is located in the area having the widest above-water beach area.

ODNR provided survey notes describing the locations of the SP points in the area. The actual benchmarks could not be located in the field so the transects were surveyed from the centerline of the shorefront road. Standard rod and level survey techniques were used for the above water portion and a small boat equipped with a sounding line and a graduated nylon rope to measure distance was used to survey the underwater profile. The water line was surveyed to provide vertical control. The transects were surveyed to a distance of about 900 ft from shore in a shore-perpendicular direction.

Horizontal juxtaposition of the July 1994 data with historic data was accomplished by measuring the distance between the road centerline and the described location of the SP point benchmarks. The available historic data, together with the July 1994 data are plotted in Figures 12 and 13. The zero position in these figures corresponds to the centerline of the shorefront road. At SP-111, the zero position was determined by extending the centerline of the east end of the shorefront road to its intersection with the transect.

Profile unit volumes were calculated to examine the historic



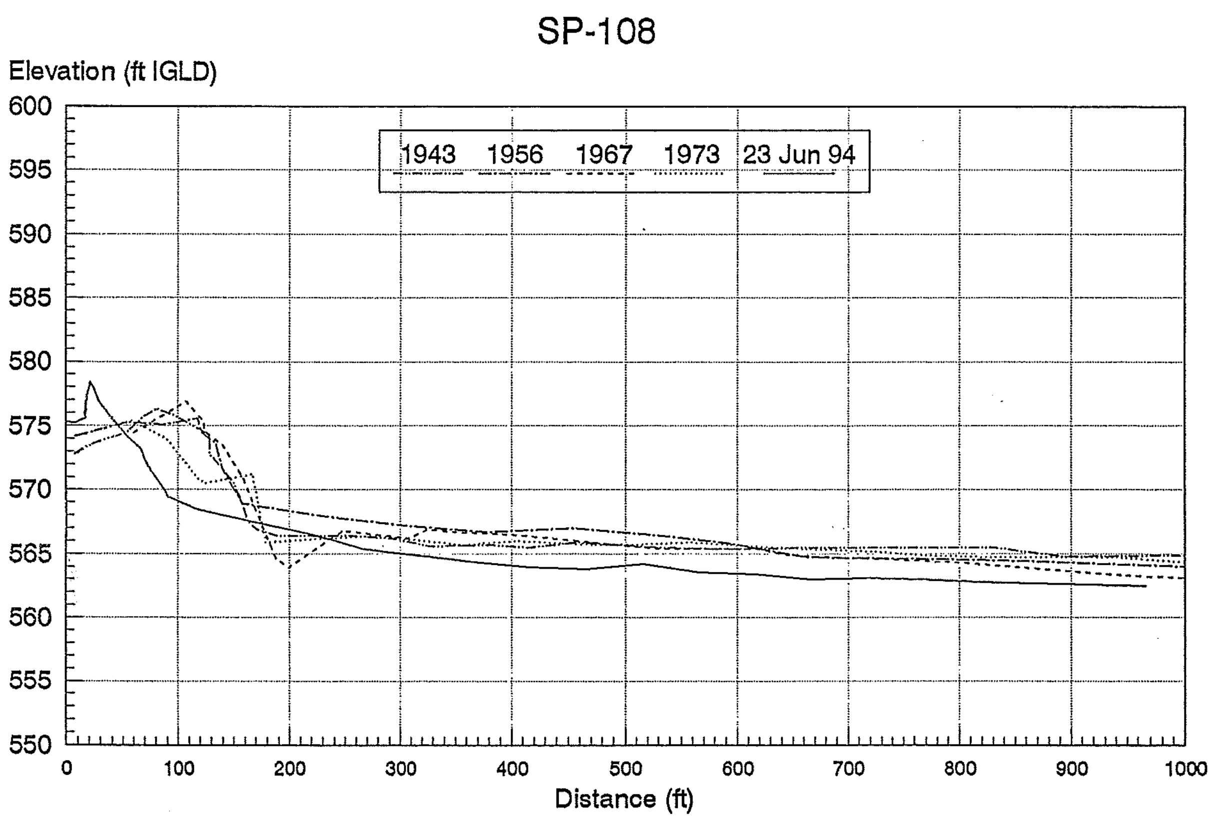
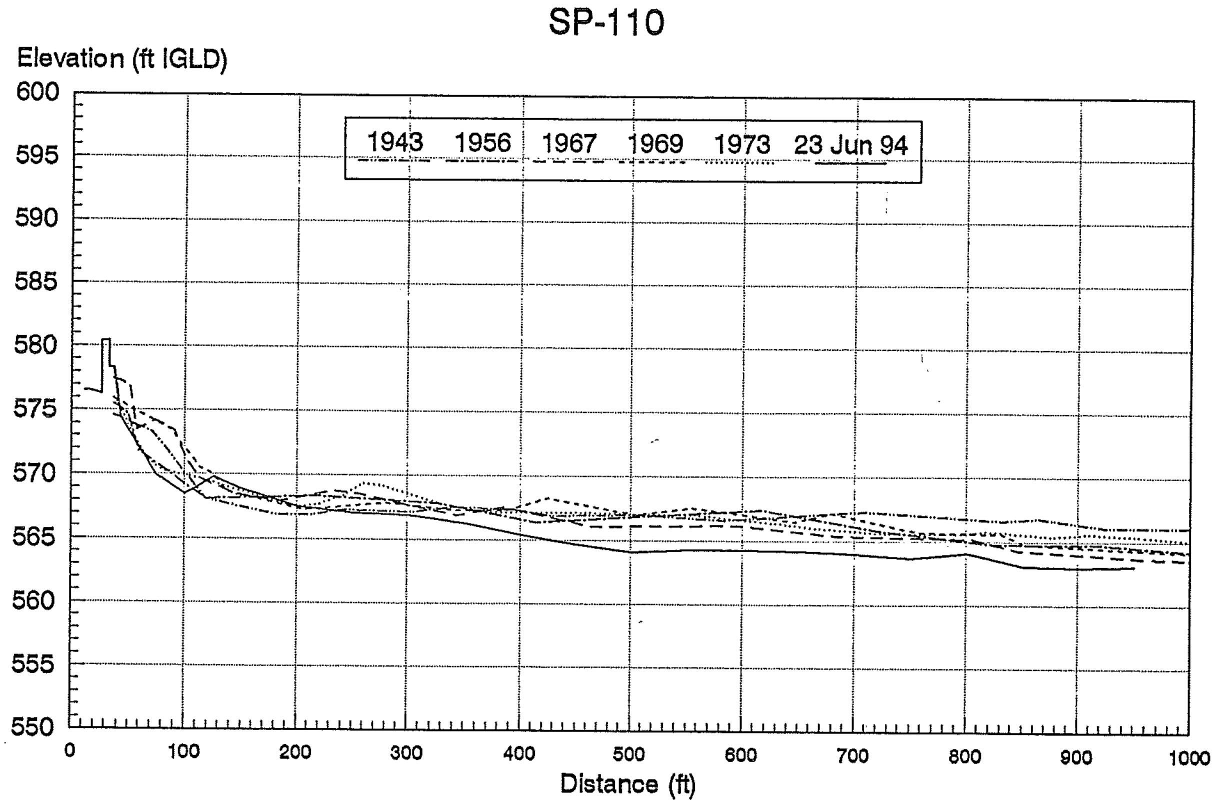


Figure 12. Beach profiles at transects SP-107 and SP-108.



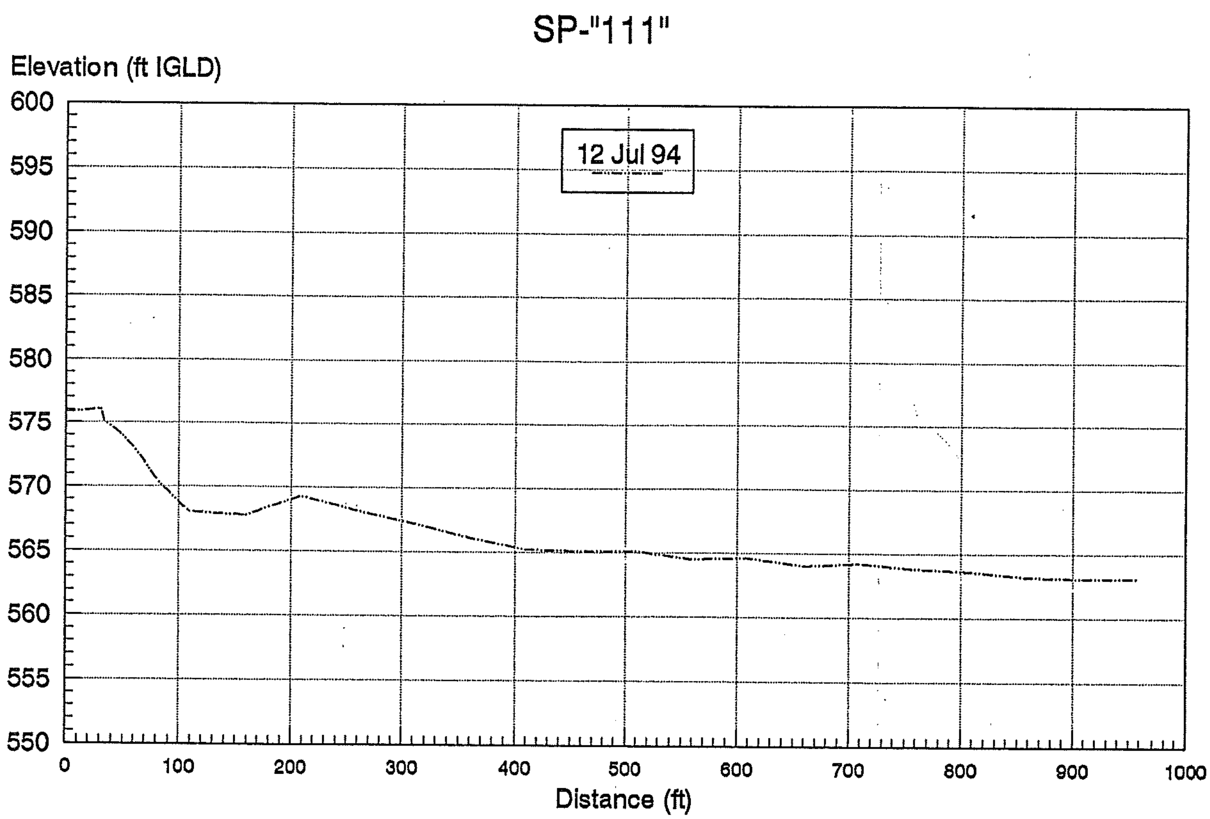


Figure 13. Beach profiles at transects SP-110 and SP-111.

trends and compare the existing profile conditions alongshore. The profile unit volume is an area whose upper boundary is the beach profile, the lower boundary was 562.0 ft IGLD, the shoreward limit was the most landward common point not including revetments, and the lakeward limit was chosen as 900 ft from the starting point. Based on analysis of the offshore wave heights, it was determined that the bottom sediments are active at 900 feet and beyond.

The unit volumes are plotted versus time in Figure 14. The data for transects SP-108 and SP-110 should be considered as representative of Sand Beach. SP-107 is not representative due to its proximity to the east marina entrance jetty which partially shelters the area and blocks westerly transport from leaving the area. Only one point is available at SP-111 since it was first established for this study. At risk of overgeneralizing, estimating average change rates from the data in Figure 14 indicates that there was a low erosion period from 1943-1967 (-0.5 cy/ft/yr average) and a high erosion period from 1967-1994 (-2.5 cy/ft/yr). The *Design Criteria* section discusses how water level controls the maximum breaking wave heights that reach the shore and erode the profile. The high and low average erosion rates correlate with average water levels of 570.5 and 571.6 ft IGLD, respectively.

The beach profiles show that the erosion has not been limited to the upper profile at the waterline but has occurred over the entire surveyed profile. There are not enough profile data available to determine whether the modest shoreline accretion that occurred east of the central revetment since 1986 was accompanied by an eroding, stable, or accreting underwater profile.

Figure 15 plots the 1994 beach profiles with starting distances matched at the centerline of the shorefront road. Also reported in the figure are the unit volumes (cy/ft) and upper beach slopes for each of the profiles. The unit volumes differ slightly from those of Figure 14 due to differences in the starting point for the calculations. The starting point for unit volume calculations in Figure 15 is 40 ft, at the lakeward base of the block revetment at SP-110. The ending point and base elevation are the same as for the previous unit volume calculations.

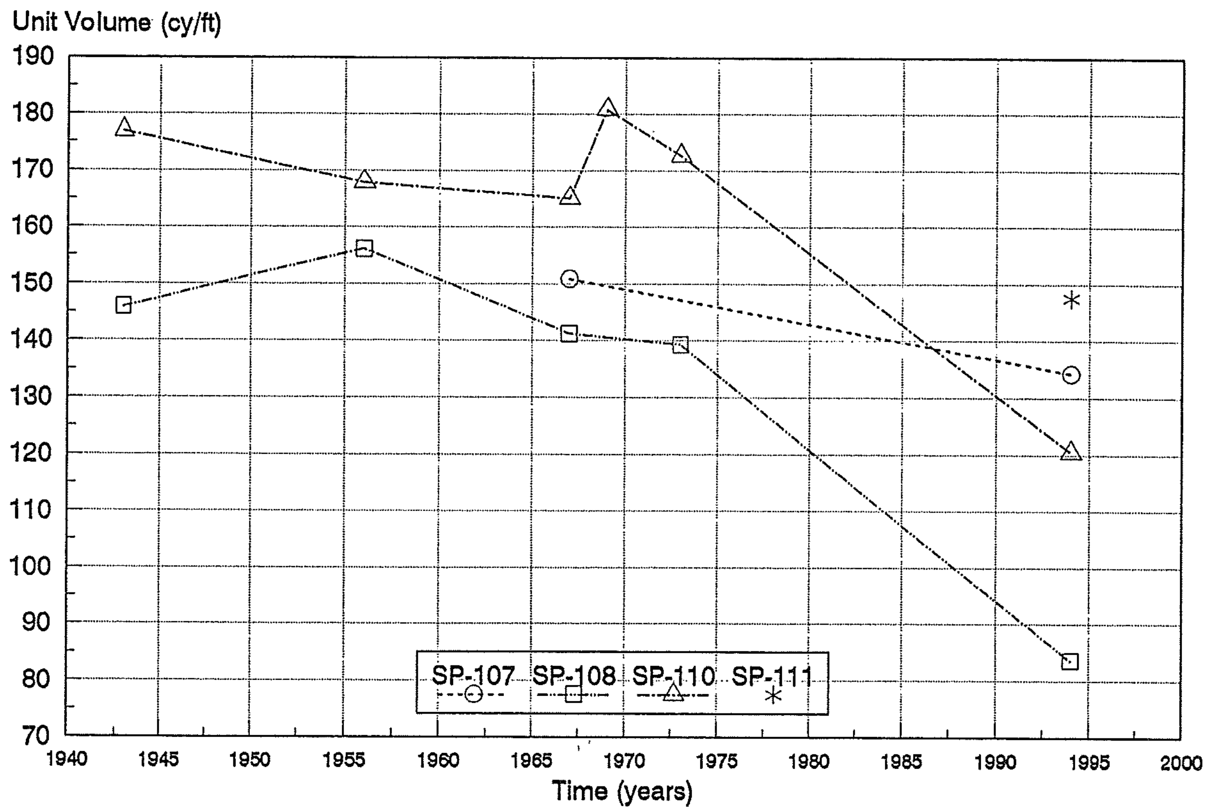


Figure 14. Unit volumes versus time at Sand Beach transects.

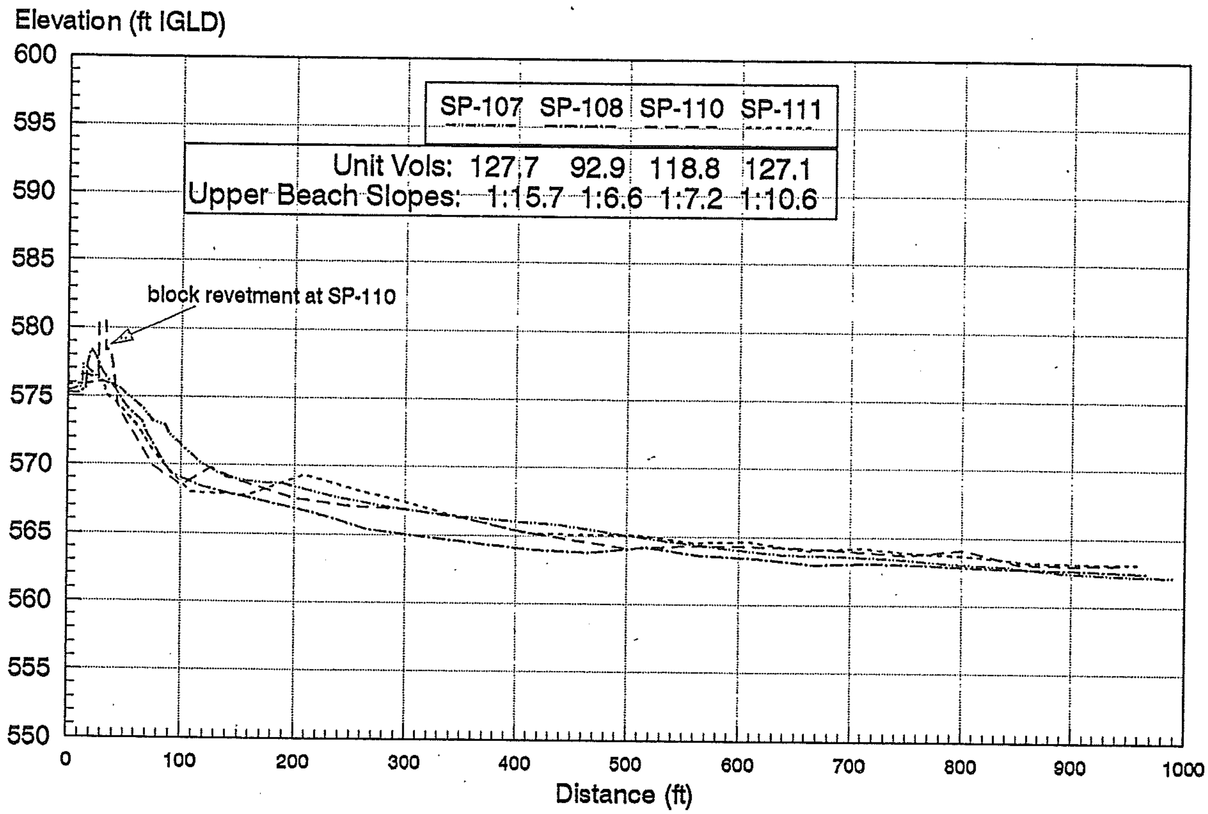


Figure 15. Comparison of July 1994 beach profiles. Starting point is centerline of shorefront road.

VI. EROSION SUMMARY

The eroded condition of the Sand Beach shoreline is fundamentally the result of sand supply deficits. The jetty at the west end of Sand Beach and the bulkhead remnants of previous house locations at the east end have also played a role in the relatively recent shoreline evolution. Based on the results of the potential longshore transport analysis, it is concluded that the historic erosion has predominantly resulted from storm wave erosion coincident with high water levels.

Figure 7 shows the results of the potential longshore transport analysis. The arrows plotted in that figure predict that on average sediment should tend to be transported toward the center of Sand Beach from the adjacent shorelines. This tendency results from Sand Beach's location on Locust Point with the offshore bathymetry focusing the waves toward the point from both directions. However, at the west end the entrance jetties are a barrier to transport from the west thereby depriving the west portion of Sand Beach of some sediment. At the east end the shoreline position data suggests that the two relic sheet-pile bulkheads where houses were once located are acting as groins to intercept transport moving from the east into that area. This would account for the wider, more stable beach area to the east of these structures.

The western portion of Sand Beach (cell 25 in Figure 7) has a net tendency for transport to the east. With a minimal net transport potential at cell 24 to the east and the littoral barrier that the jetties represent to the west, this area would be expected to erode even in the absence of a general sediment deficit. From a longshore transport potential perspective, the area within cell 24 in Figure 7 would be expected to be stable or accretionary if an adequate sediment supply existed. This area is an example of how the lack of sediment overwhelms the longshore tendency of average accretion and instead results in episodic erosion due to storm waves causing cross-shore transport (erosion).

Cross-shore transport in the lakeward direction (erosion of the upper beach) results from the combined effects of steep breaking waves and/or higher than average water levels. Without developing a detailed analysis of the cross-shore transport potential of Sand Beach area sediments, the following erosion mechanism is suggested by the available evidence and previously discussed analysis.

Due to the relative paucity of sediments and the storm-driven removal of beach quality sediments to deeper waters (witness the profile deepening evident in Figures 12 and 13) combined with the relatively high lake levels since about 1970, the average wave conditions are apparently inadequate to return storm-eroded sediments to the beach. As a result the beach profile continues to erode with each storm until either the erodible sediment is sealed off due to armoring, the lake level falls, or the storm waves are reduced with breakwaters. Another option is the placement of a sacrificial beach of borrowed sediments.

An exception to this general trend is the area at the east end of Sand Beach. From a longshore transport potential perspective the stability and accretion from 1986 to 1993 in this area is not surprising. In fact, the accretion of the area east of the revetment built after 1986 in the central portion of Sand Beach (Figure 10) supports the longshore transport analysis results indicating that sediment should tend to move from east to west here. The surprising aspect to the shoreline here is the presence of sediment to transport. Apparently there is a deposit of sand to the east of cell 23 in Figure 7 which supplies this area.

VII. ENGINEERING ALTERNATIVES

VII.A. Design Criteria

The objective of engineered alternative(s) is the creation and stabilization of a recreational beach and partial protection of the beach and shorefront structures from damaging storm waves. The project lifetime is estimated at 30 years. The length of shoreline requiring nourishment and stabilization was restricted to areas having shorefront homes. This represents an approximate length of 4300 ft.

The range of alternatives considered included breakwaters, groins, and beach fill. A revetment alternative was rejected due to the existing heavy armoring of much of the Sand Beach shoreline. Due to the lack of sediment supply, a beach fill component will be required in any alternative consideration to create a recreational and protective beach.

To stabilize the beach fill, a series of segmented breakwaters or groins should be constructed as part of the selected alternative. A segmented breakwater installation has the advantage over groins of providing storm wave reduction which will reduce erosion losses and possibly structural damages. A groin field is not recommended at Sand Beach since their principal function, the interception of longshore transport, is essentially meaningless at Sand Beach with its sediment supply deficit. Also, groins will not significantly reduce the height of storm waves which cause beach erosion. Properly placed groins can be used advantageously however to minimize the longshore transport of beach fill to areas outside the project limits.

VII.A.1. Design Conditions for Nearshore Structures

For purposes of structural design, the combination of water level and wave height yielding the most conservative requirements from a stability standpoint is required. The 30-year total water level and offshore H_s values are 575.8 ft IGLD and 8.9 ft, respectively. The offshore location is Station 2, previously described in *Offshore Wave Conditions*.

The weighted average predominant offshore storm wave direction at Station 2 is 60.2° Az. Using this direction and the average maximum annual T_p value of 6.7 sec in RCPWAVE yields a nearshore H_s amplification coefficient of 1.38 and a nearshore angle of 37.6° Az. The 38% amplification coefficient does not account for possible wave height limitations due to water depth. Using the most frequent offshore storm wave direction in RCPWAVE yields a nearshore direction of 36.0° Az, and an amplification coefficient of 1.35. Averaging these values to weight the most frequent storm wave direction yields a wave direction of 36.8° Az and an amplification coefficient of 1.36.

To determine the maximum breaking height (H_b) which can occur as a function of water depth, a breaking criteria is needed. The criteria developed by Weggel (1972) was used in this study:

$$H_b/d_b = b/[1 + ba/(gT^2)] \qquad (2)$$

where: $a = 43.75[1 - e^{(-19.0m)}]$

$b = 1.56/[1 + e^{(-19.5m)}]$

m = bottom slope

d_b = breaking depth (ft)

T = wave period (sec)

Using this breaking criteria to calculate H_b and comparing it to the H value determined from the RCPWAVE results for the same nearshore locations yields the maximum possible wave height at that location for the water depth considered.

Since the high waves which cause shore erosion and can damage shorefront structures usually occur coincident with elevated lake levels, two design conditions were considered:

1. 30 year water level with average annual extreme storm condition.

2. 30 year offshore wave height with average water level.

The condition yielding the highest possible wave height at the nearshore is #1.

VII.B. Beach Nourishment

A beach nourishment or beach fill component is recommended to create a recreational beach at Sand Beach. The primary design considerations with a beach fill are the size characteristics of the borrow sand, the total volume to be placed, the alongshore distribution of the placement, and the renourishment schedule, if any.

The SPM (1984) and James (1975) present criteria for determining the suitability of a potential borrow sediment based on the characteristics of the native beach sediments. The only existing data on the grain size distribution (GSD) of native beach materials is the sample collected by ODNR at SP-108 in 1974. Based on that sample's GSD and James' criteria, the range of sizes shown in Figure 16 is recommended for beach fill.

The volume of fill required was estimated based on an average increase in beach width of 50 ft relative to the assumed mean shoreline position (571 ft IGLD). Using the July 1994 profile at SP-108 as a guide, an addition of approximately 20 ft of berm is required. The recommended berm elevation is 576.0 ft, just above the 30 year total water level and consistent with historic berm elevations. The slope of the nourished beach face is estimated at 1:15, milder than the existing upper beach slopes for SP-108 and SP-110 (Figures 12 and 13) to provide a transition from the nearly flat lakeward profile slopes.

It is anticipated that the fill will be confined to the portion of Sand Beach having shorefront homes, approximately 4300 ft of shoreline. SP-108 is the only transect within this reach. Using SP-108 as the pre-construction profile, a typical fill template was constructed to determine a unit fill requirement of 14.3 cy/ft. The SP-108 profile and fill template are shown in Figure 17. Specific fill volumes will vary depending on the specific shoreline coverage of the chosen alternative.

It is not possible to accurately estimate the rate at which the beach fill can be expected to erode. The historic beach profile change data is too sparse and complicated by the area's sediment

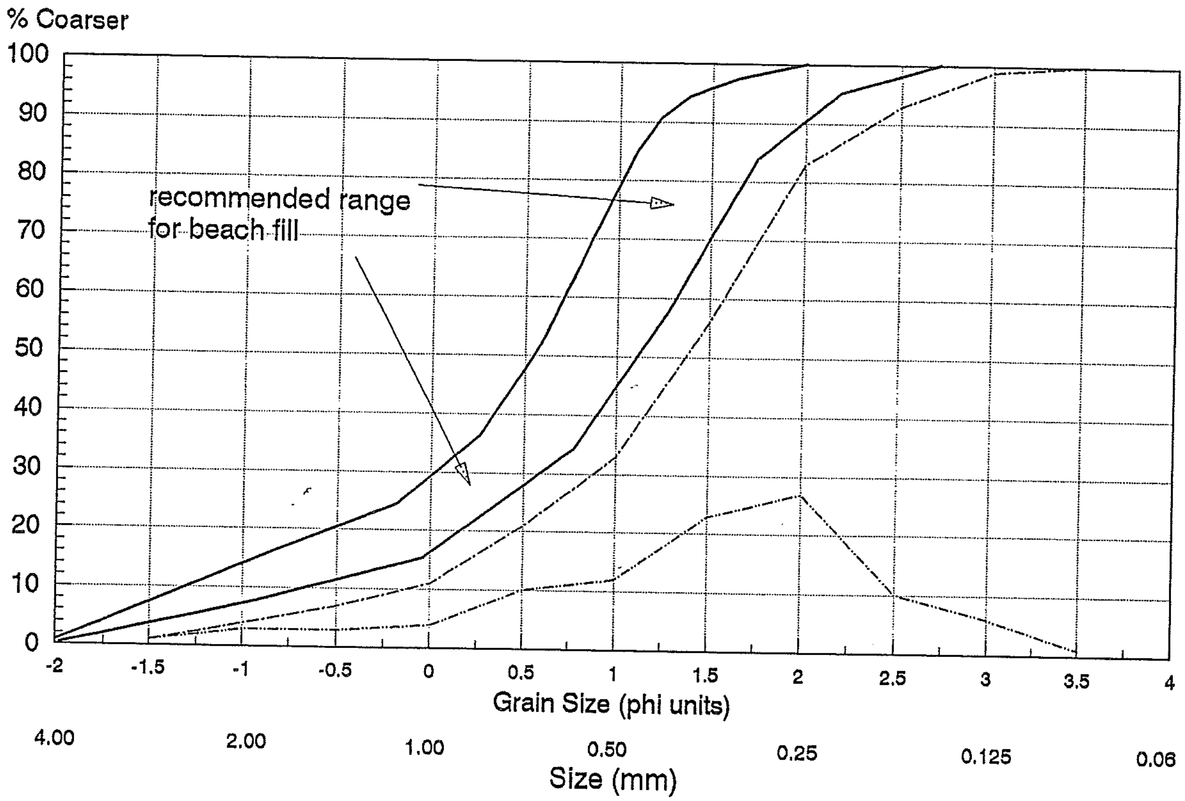


Figure 16. Cumulative and discrete GSD's for beach sample at SP-108 and recommended range for GSD of beach fill.

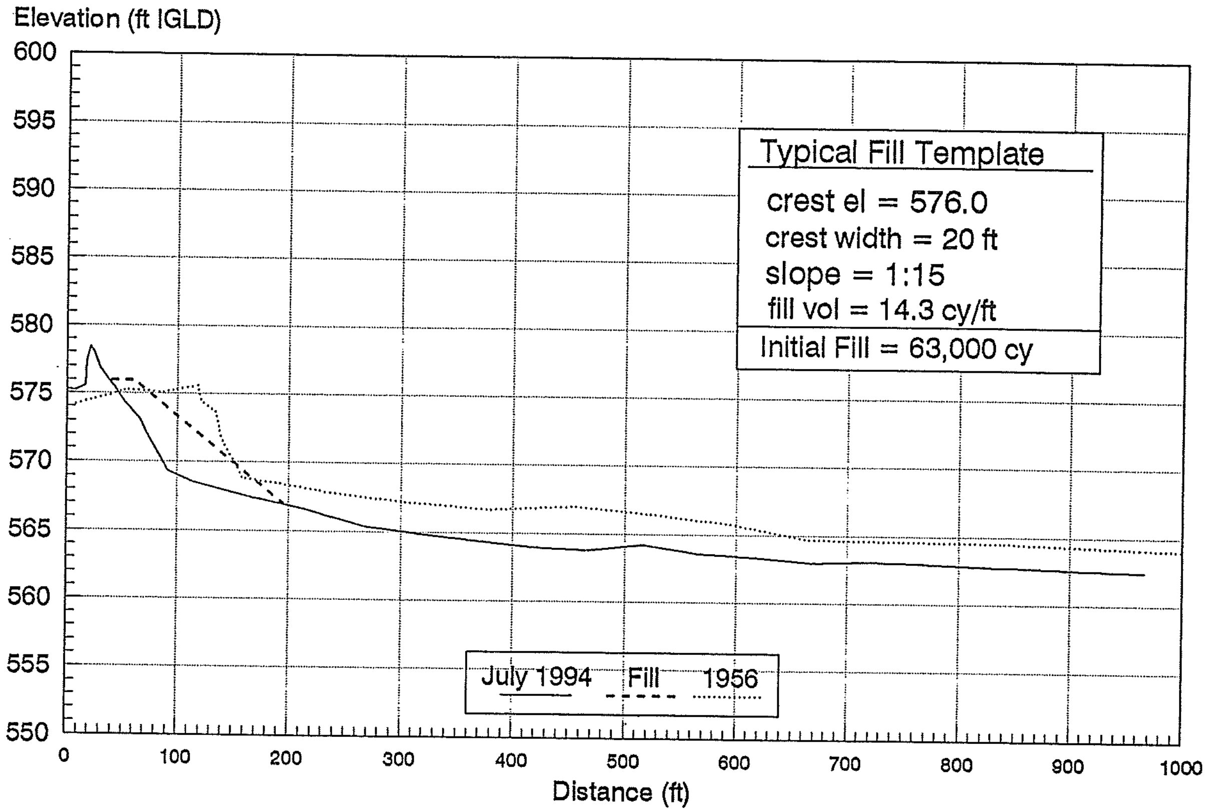


Figure 17. 1994 beach profile and typical fill template from SP-108. Initial fill volume is an average for the 2 alternatives presented. 1956 profile shown for comparison.

deficit to provide meaningful background erosion rates. The erosion of the fill will also depend on the type of structural protection provided and its characteristics. Nevertheless, the volumetric erosion rates for transects SP-108 and SP-110 in Figure 14 yield a historic range of -0.5 to -2.5 cy/yr as discussed in *Historic Beach Profile Changes*. This corresponds to an annual erosion range of approximately 2,000 to 11,000 cy/yr for the entire project area.

VII.C. Alternative 1 - Segmented Breakwaters with Beach Fill

Segmented breakwaters have been used successfully to stabilize eroded shorelines at three Lake Erie locations (Lakeview Park, Lorain, OH; Lakeshore Park, Ashtabula, OH; Presque Isle State Park, Erie, PA). These projects are described in USACOE (1984). Segmented breakwaters are used throughout the world, with the greatest use by far occurring in Japan. A description and brief discussion of the various guidance arising from laboratory and prototype installation performance is provided in Rosati (1990).

The following is a list of the primary design variables for a segmented breakwater installation.

L_s = length of breakwater segment (ft)
L_g = length of gap between breakwaters (ft)
X = distance from original shoreline to breakwater (ft)
θ = angle between breakwater axis and shoreline

Beside these variables unique to segmented breakwater installations, there are additional variables pertinent to the structural stability of quarrystone breakwaters in general. The following is a list of these variables and design considerations:

Crest elevation (ft)
Side slopes
d_s = average water depth at structure (ft)
b = crest width (ft)
W = weight of armor stone (tons)
n = number of stones in armor layer
Rock size gradation
Width of toe berm (ft)
Width of scour apron (ft)
Bedding/Filter layer requirements

As a general criteria for non-harbor sites, the breakwaters should be located in water as shallow as possible to minimize the rock volume requirement. Based on the July 1994 profiles at transects SP-108 and SP-110, the 567.0 ft contour was chosen for breakwater location (4 ft average depth). This is about 155 ft from a 571 ft IGLD shoreline position, or about 165 ft from the July 1994 shoreline. This is considered an approximate minimum to provide a recreational swimming area.

The breakwaters should be oriented to be perpendicular to the predominant storm wave direction (36.8° Az).

The design water depth is the 30 year total water level minus the bottom elevation at the breakwater (575.8 - 567.0 = 8.8 ft). The limiting H_b for this water depth is 8.2 ft, which is the controlling H_b since it is less than the depth unlimited H using the RCPWAVE results (7.6 x 1.38 = 10.5 ft).

The SPM (1984) provides guidance on the size of armor stone required for breakwater stability as a function of design wave height, quarrystone unit weight (155 pcf), structure side slope, and a stability coefficient whose values were developed in large scale model tests. Based on this guidance an armor stone weight of approximately 3 tons is recommended for both the head and trunk sections. This size assumes an allowable damage percentage of armor units of up to 10% under design conditions. Damage is defined as the displacement of individual stones. The recommended gradation for armor stones is given in Table 5.

% Smaller	Size (lbs)
100	7500
85	6750
75	6500
25	6000
15	5000
0	4500

Table 5. Recommended gradation for breakwater stones.

The crest width is recommended to be at least 2 armor units, or approximately 7 ft wide.

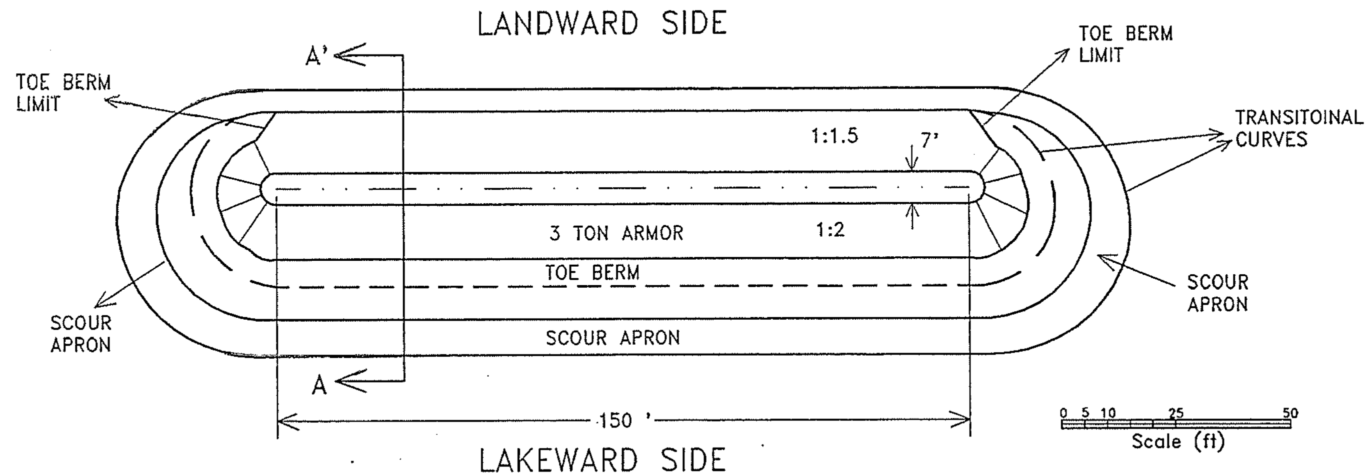
The crest elevation is the single largest determinant of rock volume requirement since it controls the structure geometry, given the water depth, side slopes, and crest width. The crest elevation along with factors controlling wave runup on the structure determine the amount of wave overtopping that can be expected during the design event. Oftentimes, only minor overtopping is allowable so as to minimize the wave heights in the lee of the structure due to overtopping. However, at Sand Beach the primary effect of an increase in allowable overtopping is the potential for greater amounts of beach fill erosion. The crest elevation required to allow only minor overtopping is 581 ft whereas a crest elevation that will allow an overtopped wave of approximately one-half the design wave (~ 4 ft) is 576 ft. Allowing for the larger overtopping results in an approximate 75% savings in rock volume.

With a crest elevation of 576 ft and a bottom elevation of 567 ft, the breakwaters will be 9 ft high. The minimum thickness of armor stone recommended is 2 layers (SPM, 1984), or approximately 7 ft. Since the armor layer occupies the majority of the breakwater section, no specific core material is recommended in order to facilitate construction operations.

Other considerations include a toe berm and scour apron to protect the lakeward structure toe from being displaced which would destabilize the entire lakeward face. Scour protection is especially important where $d_s \leq 2H_{design}$ (Eckart, 1983). A buried toe is recommended. The toe depth should extend ~H_{design} (7.5 ft) below bottom. Alternatively, an extended horizontal toe 15 ft wide can be substituted for the buried toe to facilitate construction operations.

If the bottom where the breakwaters are located is silty/clayey, either a geotextile fabric or layer of sand should be placed as a filter to prevent the migration of fines into the structure, resulting in structure settlement. This is especially important in an area with a history of profile erosion. A bedding layer of gravel should be placed on top of the filter to keep it in place and evenly distribute the bearing load to prevent differential settlement and/or puncture of the geotextile fabric if one is used. The bedding layer should be a minimum of 1 ft thick. If sand is used for a filter, the same sand as used for the beach fill can be used. The sand filter should be approximately 4 in

Plan View – Segmented Breakwater



Section A–A'

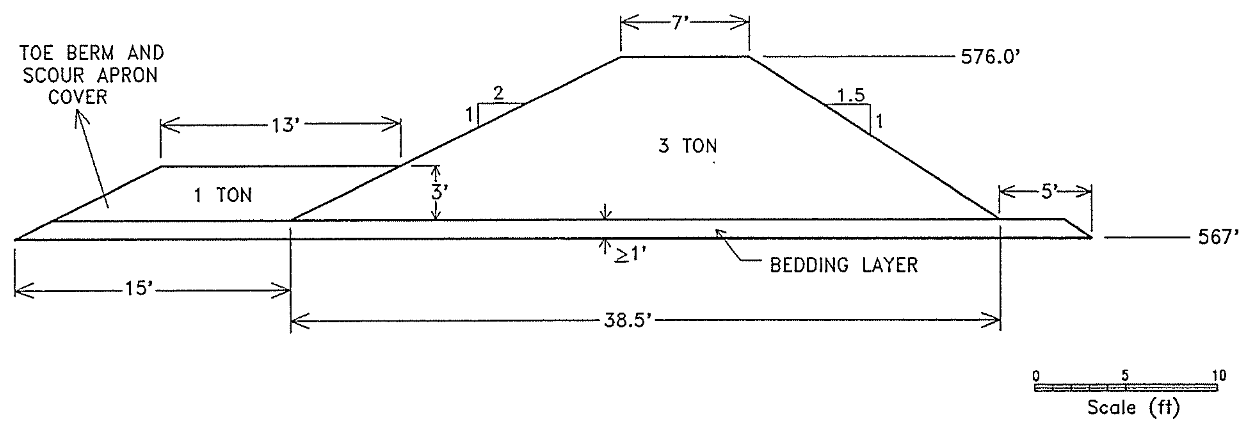


Figure 18.

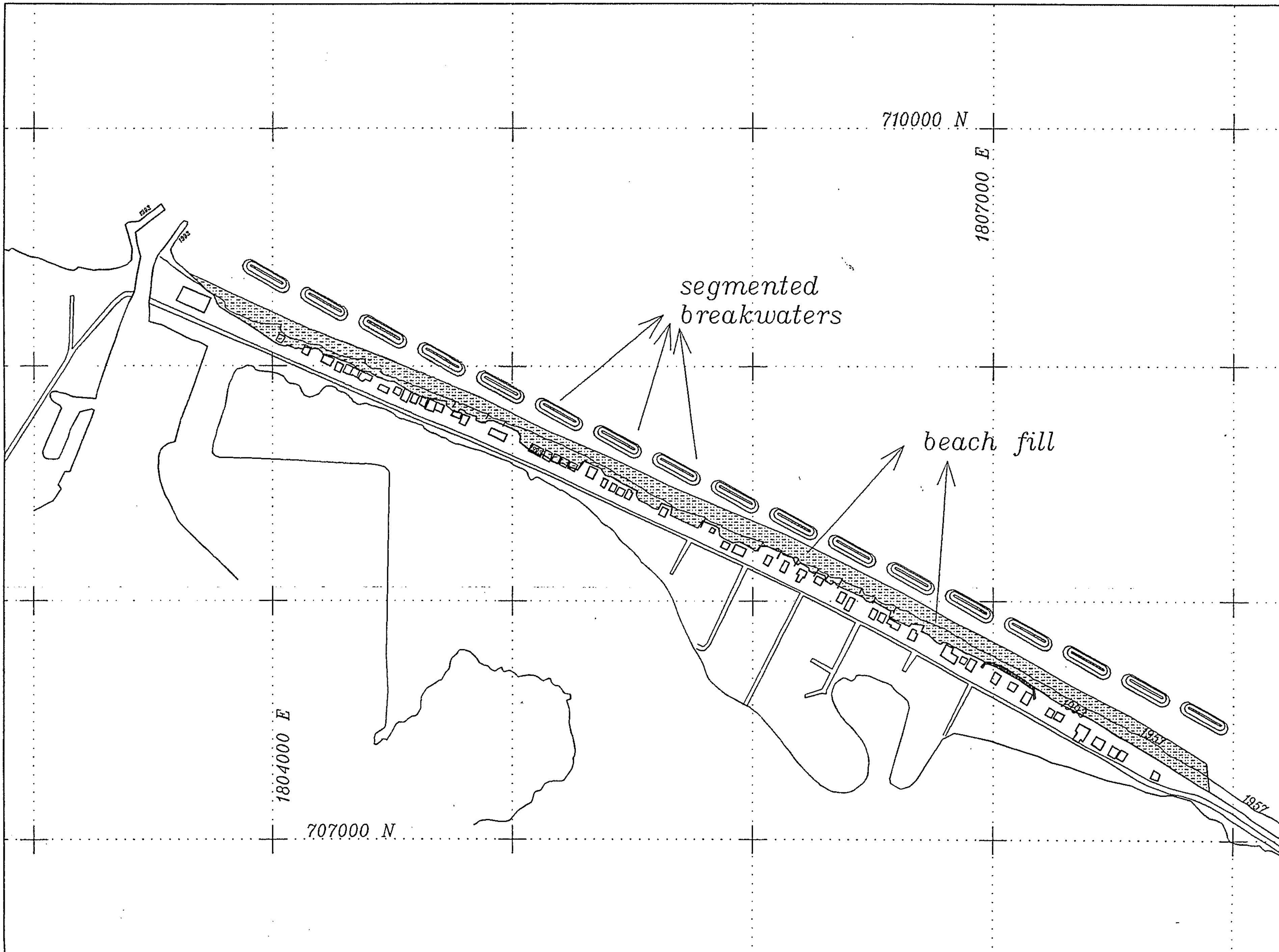


Figure 19. Plan view of project area for Alternative 1.

thick.

Figure 18 shows the breakwater plan and section geometry, dimensions, and stone sizes.

The length of each breakwater segment, L_s, is typically recommended as a function of the distance from the initial shoreline, X. In the absence of definitive guidance, $L_s=X$ is recommended. The average X value from transects SP-108 and SP-110 is approximately 150 ft.

The remaining design variable is the gap length, L_g. Rosati (1990) found after an examination of prototype installations including two on Lake Erie that the criteria of Seiji, Uda, and Tanaka (1987), ($L_g/X < 0.8$) was a good predictor of the maximum gap length to prevent shoreline erosion opposite the gap. This results in L_g=120 ft at Sand Beach. Figure 19 is a plan view of Alternative 1 including the beach fill. Depending on project performance, a terminal groin at the east end of the project could be added to help retain the beach fill. A preliminary construction cost estimate for Alternative 1 is shown on page 52.

VII.D. Alternative 2 - Beach Fill with Groins

The second alternative consists of a beach fill with a groin field to stabilize the fill. This alternative has the benefit of reduced initial construction costs. However, with little reduction of wave heights at the shore, storm erosion of the fill and possible structural damages are not reduced significantly. For this alternative, periodic beach nourishment will be required.

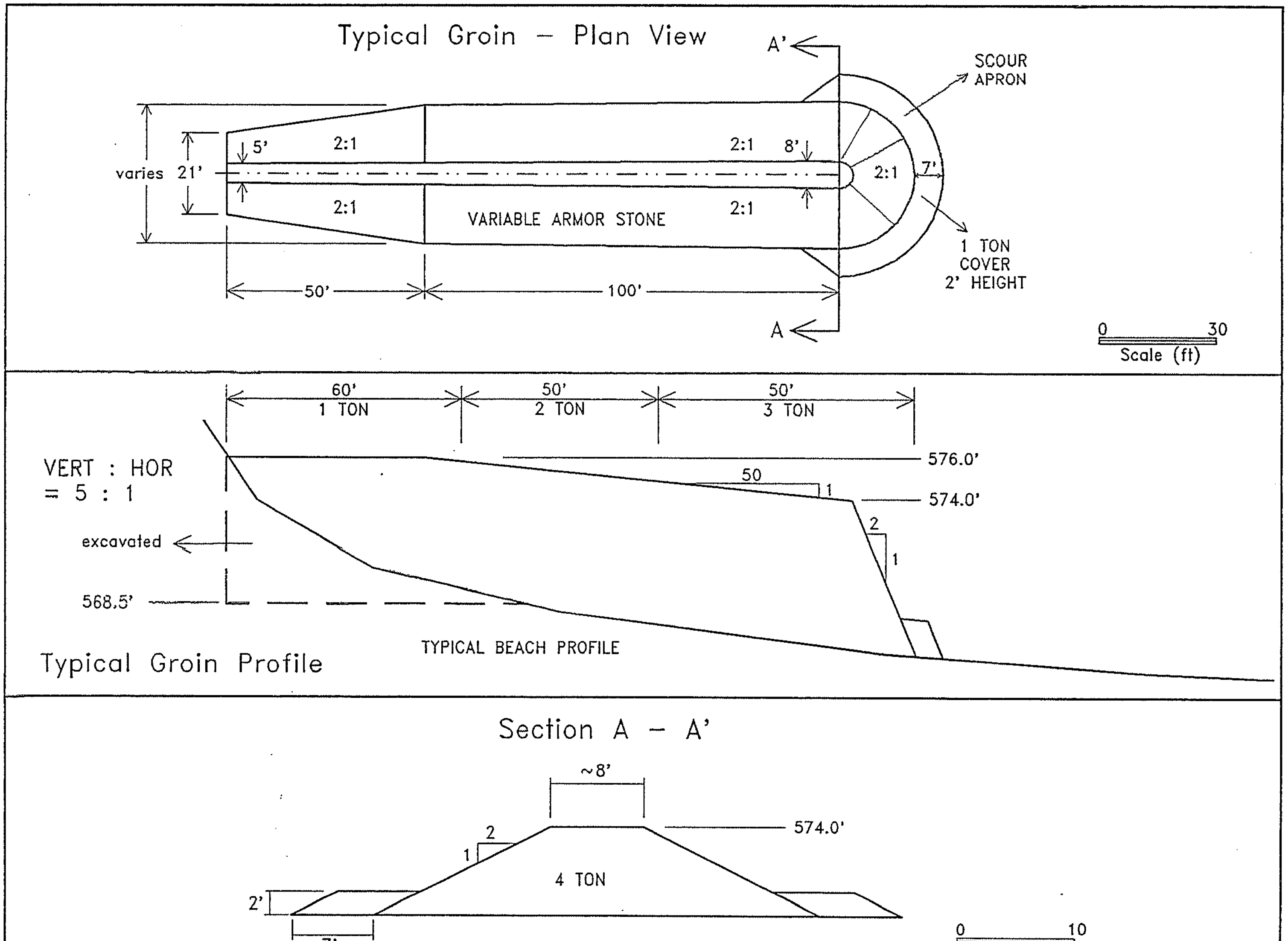
The available evidence suggests that if the average water level remains high, high erosion rates of the fill can be expected. If however, a trend of lower water levels ensues, then erosion rates should be correspondingly lower. Also, given a constant wave climate, erosion rates will be highest during the first few years after construction and will gradually taper off as the profile unit volumes are reduced. Conservatively, an annual average erosion rate of 5,000 cy/yr is estimated.

Available guidance on groin design (SPM, 1984; CSE, 1993) indicates that the ratio of groin spacing to length should not exceed 3.0. Groin spacing is the distance between groins. Groin length is the distance between the mean shoreline position in the groin cell and the groin head. Spacing:length values > 3.0 usually result in excessive erosion in localized areas within the groin cell.

For the groins to be effective in retaining sand within the cells an impermeable core is required. This can be accomplished with a graded structure having a core of small stone (1-2 in). Alternatively, for construction efficiency, one size stone gradation can be used and the structure's interstices can be grouted with a cement or bitumen based grout. This technique has been shown to be effective (USACOE, 1992) in making previously porous rubble mound structures impermeable. Detailed guidance on various grout mixes and their application is provided in Simpson (1989) and Simpson et al (1990).

The groin heads should be located at the toe of the beach fill. Using the beach profile at SP-108 as a guide (Figure 17), a head position approximately 200 ft from the centerline of the road is found. Actual distances will vary depending on the existing shoreline positions at each groin. Generally, using the beach profile of SP-108, the physical groin length is assumed to be 150 ft. Based on the average 571 ft IGLD shoreline, the effective groin length is about 120 ft. Thus, the groin spacing should be 360 ft. To provide coverage for 4300 ft of shoreline, 13 groins are required, covering 4320 ft. The beach fill requirement is 61,800 cy.

Using the same stability analysis as was described for the segmented breakwater cross-section, the groin plan, profile, and sections are shown in Figure 20. Figure 21 shows the project plan. A preliminary construction cost estimate for Alternative 2 is shown on page 53.



Typical Groin – Plan View
A'
A
SCOUR APRON
1 TON COVER 2' HEIGHT
varies
21'
5'
8'
7'
2:1
2:1
2:1
2:1
2:1
VARIABLE ARMOR STONE
50'
100'
0
30
Scale (ft)
60'
1 TON
50'
2 TON
50'
3 TON
VERT : HOR = 5 : 1
576.0'
574.0'
50
1
2
1
excavated
568.5'
TYPICAL BEACH PROFILE
Typical Groin Profile
Section A – A'
~8'
574.0'
2
1
4 TON
2'
7'
0
10
Scale (ft)

Figure 20.

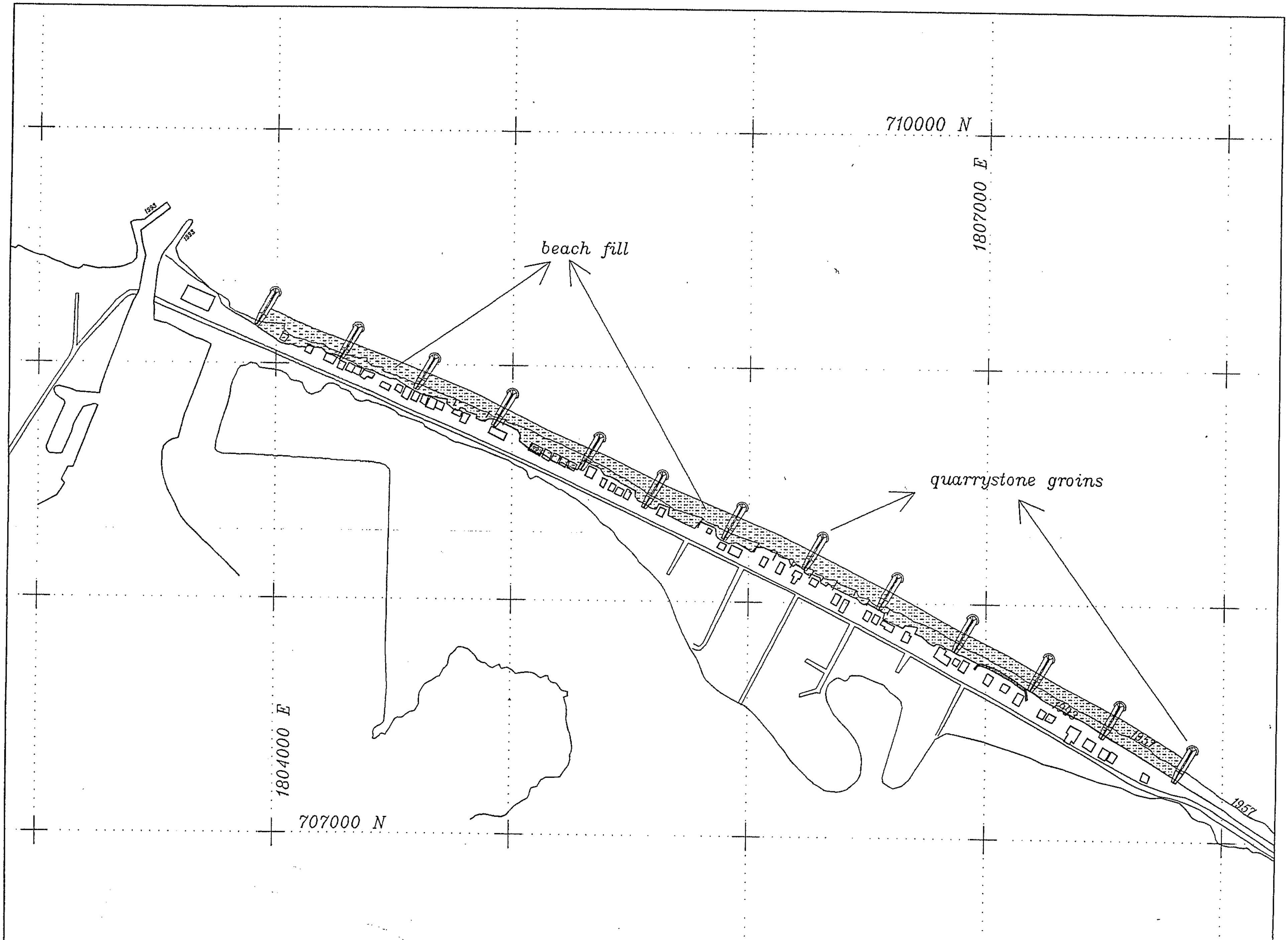


Figure 21. Plan view of project area for Alternative 2.

VII.E. Alternative 3 - Pilot Segmented Breakwater Project

The third project alternative is actually a small fraction of Alternative 1, the segmented breakwaters with beach fill project. The advantage of this alternative is that it costs only a fraction of the entire segmented breakwater alternative and provides valuable performance data upon which to base decisions concerning additional construction. The data gathered during the monitoring period could be used to fine tune the original plans which could result in cost savings.

The recommended pilot project consists of three breakwater segments and a beach fill component. The breakwater sections are the same as those in Figure 18. The cross-shore location and orientation are as indicated in Figure 19. The shoreline length covered by this alternative is 690 ft. Since erosion losses at the ends of an area this small may be high, rock groins are recommended to minimize erosion of the fill out of the pilot project area. Depending on the location of the pilot project, only one terminal groin may be required.

For this alternative, a reduced unit beach fill of 7 cy/ft is recommended. By placing the beach fill at a 1:10 slope, with an 18 ft berm width, the required groin length can be reduced to 75 ft. The groin section and profile characteristics are as shown in Figure 20 with the sloping portion of the profile reduced to 25 ft. Since the groins are temporary and relatively short, no scour apron is required. A preliminary construction cost estimate for Alternative 3 is shown on page 54.

VII.F. Pre-construction Recommendations

Prior to construction, beach profiles should be surveyed along the entire project shoreline at 200 to 300 ft intervals. These profile data will provide the coverage necessary to accurately define the beach fill requirement. The initial fill volume estimated here was based on only one transect in the project area surveyed in July 1994 (SP-108). The calculated fill quantity is sensitive to the number of profiles surveyed and their alongshore variation. Additional survey data may indicate that the initial fill volume can be reduced.

A detailed collection of bottom sediment samples along the 567.0 ft contour should be obtained to determine the underlayer requirement for the segmented breakwaters. Preliminary indications are that the bottom is composed of significant fractions of silt and clay which would necessitate the use of a sand layer or geotechnical fabric below the breakwater's bedding layer.

ALTERNATE 1

CONSTRUCTION COST ESTIMATE	DATE PREPARED 8/25/94	SHEET 1 OF 3
PROJECT NO. EROSION PLAN- 17 SEGMENTED BREAKWATERS AND BEACH FILL	CONTRACT NO.	
LOCATION SAND BEACH CONSERVANCY DISTRICT, OAK HARBOR, OH		
DRAWING NO.	ESTIMATOR JSM	CHECKED BY

TRADE	QUANTITY		LABOR		MATERIAL/EQUIP.		TOTAL COST
	NO. UNITS	UNIT MEAS	PER UNIT	TOTAL	PER UNIT	TOTAL	
Mobilization	LS	LS			LS	$ 10,000	$ 10,000
Armor Stone (3.0 Ton)	29,000	TN	10.00	$ 290,000	15.00	435,000	725,000
Toe Stone (1.0 Ton)	7,200	TN	10.00	72,000	15.00	108,000	180,000
Bedding Stone	9,400	TN	6.00	56,400	10.00	94,000	150,400
Bedding Sand	3,600	TN	4.00	14,400	6.00	21,600	36,000
Sand Fill	96,000	TN	2.00	192,000	6.00	576,000	768,000
						TOTAL =	$1,869,400

CONSTRUCTION COST PER LINEAR FOOT OF SHORELINE:

$1,869,400 / 4,470 L.F. = $418.21/L.F. of Shoreline

For a 40' Lot: $418.21 X 40 L.F. = $16,728

ANNUAL BEACH SAND NOURISHMENT COST:

Assume: 3,000 Tons/yr Total Erosion; Renourishment when approximately half project is gone; Therefore assume renourishment every 15 yrs of 45,000 tons.

Annual Cost: 3,000 Ton/yr X $8.00/Ton = $ 24,000/yr

24,000/ 4,470 L.F. = $ 5.37/L.F. per yr

$5.37/L.F. X 40' = $ 214.80 per 40' Lot

ALTERNATE 2

CONSTRUCTION COST ESTIMATE	DATE PREPARED 8/25/94	SHEET 2 OF 3
PROJECT NO. EROSION PLAN - 13 GROINS AND BEACH FILL	CONTRACT NO.	
LOCATION SAND BEACH CONSERVANCY DISTRICT, OAK HARBOR, OH		
DRAWING NO.	ESTIMATOR JSM	CHECKED BY

TRADE	QUANTITY		LABOR		MATERIAL/EQUIP.		TOTAL COST
	NO. UNITS	UNIT MEAS	PER UNIT	TOTAL	PER UNIT	TOTAL	
Mobilization	LS	LS			LS	$ 5,000	$ 5,000
Armor Stone (3.0 Ton)	6,700	TN	10.00	$ 67,000	15.00	100,500	167,500
Armor Stone (2.0 Ton)	5,100	TN	10.00	51,000	15.00	76,500	127,500
Armor Stone (1.0 Ton)	6,400	TN	10.00	64,000	15.00	96,000	160,000
Sand Bedding	1,400	TN	4.00	5,600	6.00	8,400	14,000
Concrete Grout	2,400	CY	50.00	120,000	50.00	120,000	240,000
Sand Fill	93,000	TN	2.00	186,000	6.00	558,000	744,000
						TOTAL =	$1,458,000

CONSTRUCTION COST PER LINEAR FOOT OF SHORELINE:

$\frac{\$\ 1,458,000}{4,320 \text{ L.F.}}$ = $337.50/ L.F. of Shoreline

For a 40' Lot : $337.50 X 40 L.F. = $13,500

ANNUAL BEACH NOURISHMENT COST:

Assume: 5,000 Tons/yr Total Erosion; Renorishment when approximately half project is gone; Therefore assume renourishment every 6 years of 45,000 tons

Annual Cost: 9,000 tons/yr X $ 8.00/ton = $ 72,000/yr

72,000/ 4,320 L.F. = $ 16.67/L.F. per yr

$16.67/L.F. X 40 L.f. = $ 666.80/yr

<u>ALTERNATE 3</u>

CONSTRUCTION COST ESTIMATE	DATE PREPARED 8/25/94	SHEET 3 OF 3
PROJECT NO.	CONTRACT NO.	
EROSION PLAN - PILOT PROJECT OF 3 BREAKWATERS, 2 GROINS, & BEACH FILL		
LOCATION: SAND BEACH CONSERVANCY DISTRICT, OAK HARBOR, OH		
DRAWING NO.	ESTIMATOR: JSM	CHECKED BY

TRADE	QUANTITY		LABOR		MATERIAL/EQUIP.		TOTAL COST
	NO. UNITS	UNIT MEAS	PER UNIT	TOTAL	PER UNIT	TOTAL	
Mobilization	LS	LS			LS	$ 10,000	$ 10,000
<u>Breakwaters</u>							
Armor Stone (3.0 Ton)	5,120	TN	10.00	51,200	15.00	76,800	128,000
Armor Stone (1.0 Ton)	1,270	TN	10.00	12,700	15.00	19,050	31,750
Bedding Stone	1,660	TN	6.00	9,960	10.00	16,660	26,560
Bedding Sand	650	TN	4.00	2,600	6.00	3,900	6,500
<u>GROINS</u>							
Armor Stone (2.0 Ton)	120	TN	10.00	1,200	15.00	1,800	3,000
Armor Stone (1.0 Ton)	440	TN	10.00	4,400	15.00	6,600	11,000
Sand Fill	18,000	TN	2.00	36,000	6.00	108,000	144,000
						TOTAL =	$ 360,810

CONSTRUCTION COST PER LINEAR FOOT OF SHORELINE:

$$\frac{\$\ 360{,}810}{740\ \text{L.F.}} = \$\ 487.58/\text{L.F. of shoreline}$$

For a 40' Lot: $487.58 X 40 L.F. = $ 19,503

IX. REFERENCES

Carter, C.H. 1973. "The November 1972 Storm on Lake Erie," Information Circular No. 39, Ohio Department of Natural Resources, Columbus, OH.

CSE, Inc. 1993. "Edisto Beach Groin Study," published by Coastal Science & Engineering, Inc., Columbia, SC.

Driver, B.D., et al,. 1991. "Hindcast Wave Information for the Great Lakes: Lake Erie," WIS Report 22, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Ebersole, B.A. et al,. 1986. "Regional Coastal Processes Numerical Modelling System, Report 1, RCPWAVE-A Linear Wave Propagation Model for Engineering Use," Technical Report CERC-86-4, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Eckert, J.W., 1983. "Design of Toe Protection for Coastal Structures," Proceedings of Coastal Structures '83, pp 331-341, ASCE, Alexandria, VA.

James, W.R., 1975. "Techniques in Evaluating Suitability of Borrow Material for Beach Nourishment," TM-60, Coastal Engineering Research Center, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Pore, N.A, et al., 1975. "Climatology of Lake Erie Storm Surges," NOAA Technical Memorandum NWS TDL-54, Techniques Development Laboratory, Silver Spring, MD.

Rosati, J.D., 1990. "Functional Design of Breakwaters for Shore Protection: Empirical Methods," Technical Report CERC-90-15, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Seiji, M., Uda, T., and Tanaka, S., 1987. "Statistical Study on the Effect and Stability of Detached Breakwaters," Coastal Engineering in Japan, Vol 30, No 1, pp 131-141.

Shore Protection Manual. 1984. 4th ed., 2 Vols, US Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, US Government Printing Office, Washington, DC.

Walker, J.R., et al,. 1980. "A Detached Breakwater System for Shore Protection," Proceedings, 17th International Coastal Engineering Conference, Sydney, Australia, pp 1968-1987.

Weggel, J.R., 1972. "Maximum Breaker Height," Journal of the Waterways, Port, Coastal, and Ocean Division, Vol 78 (No WW4), pp 529-548.

US Army Corps of Engineers, 1961. "Lake Erie Shoreline from the

Michigan-Ohio State Line to Marblehead, Ohio, Beach Erosion Control Study," US Government Printing Office, Washington, DC.

US Army Corps of Engineers, 1984. "Use of Segmented Offshore Breakwaters for Beach Erosion Control," CETN-III-22, US Army Engineer Waterways Experiment Station, Vicksburg, MS.