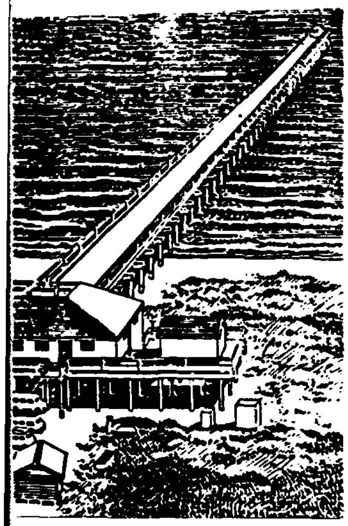
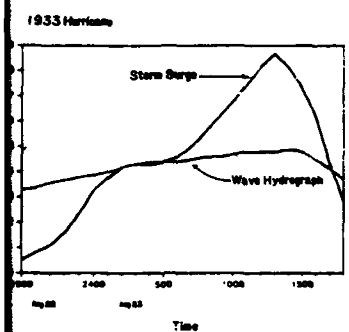
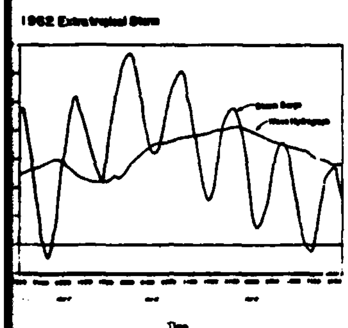
US Army Corps of Engineers







TECHNICAL REPORT CERC-88-1

COASTAL ENGINEERING STUDIES IN SUPPORT OF VIRGINIA BEACH, VIRGINIA, BEACH EROSION CONTROL AND HURRICANE PROTECTION PROJECT

Report 3

EVALUATION AND DESIGN OF BEACH FILL

by

Mark Hansen, Norman Scheffner

Coastal Engineering Research Center

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
3909 Halls Ferry Road, Vicksburg, Mississippi 39180-6199

April 1990

Report 3 of a Series

Approved For Public Release; Distribution Unlimited

Prepared for US Army Engineer District, Norfolk
Norfolk, Virginia 23510-1096

Under Intra-Army Order No. AD-86-3018

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE

REPORT DOCUMENTATION PAGE

Form Approved OMB No. 0704-0188

1a. REPORT SECURITY CLASSIFICATION Unclassified	1b. RESTRICTIVE MARKINGS
2a. SECURITY CLASSIFICATION AUTHORITY	3. DISTRIBUTION/AVAILABILITY OF REPORT Approved for public release; distribution unlimited.
2b. DECLASSIFICATION/DOWNGRADING SCHEDULE	
4. PERFORMING ORGANIZATION REPORT NUMBER(S) Technical Report CERC-88-1	5. MONITORING ORGANIZATION REPORT NUMBER(S)
6a. NAME OF PERFORMING ORGANIZATION USAEWES, Coastal Engineering Research Center 6b. OFFICE SYMBOL *(If applicable)*	7a. NAME OF MONITORING ORGANIZATION
6c. ADDRESS *(City, State, and ZIP Code)* 3909 Halls Ferry Road Vicksburg, MS 39180-6199	7b. ADDRESS *(City, State, and ZIP Code)*
8a. NAME OF FUNDING/SPONSORING ORGANIZATION USAED, Norfolk 8b. OFFICE SYMBOL *(If applicable)*	9. PROCUREMENT INSTRUMENT IDENTIFICATION NUMBER Intra-Army Order No. AD-86-3018
8c. ADDRESS *(City, State, and ZIP Code)* Norfolk, VA 23510-1096	10. SOURCE OF FUNDING NUMBERS PROGRAM ELEMENT NO. | PROJECT NO. | TASK NO. | WORK UNIT ACCESSION NO.

11. TITLE *(Include Security Classification)*
Coastal Engineering Studies in Support of Virginia Beach, Virginia, Beach Erosion Control and Hurricane Protection Project; Report 3, Evaluation and Design of Beach Fill

12. PERSONAL AUTHOR(S)
Hansen, Mark; Scheffner, Norman

13a. TYPE OF REPORT	13b. TIME COVERED	14. DATE OF REPORT *(Year, Month, Day)*	15. PAGE COUNT
Report 3 of a series	FROM ________ TO ______	April 1990	68

16. SUPPLEMENTARY NOTATION
Available from National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161

17. COSATI CODES			18. SUBJECT TERMS *(Continue on reverse if necessary and identify by block number)*
FIELD	GROUP	SUB-GROUP	See reverse.

19. ABSTRACT *(Continue on reverse if necessary and identify by block number)*

A study was conducted to determine beach-fill berm dimensions in order to provide backshore areas with sufficient protection from storm-induced flooding. The beach-fill design was part of a beach erosion control and hurricane protection project along approximately 6 miles of Virginia Beach, Virginia. A modified Kriebel cross-shore numerical model was used to evaluate various berm dimensions for several hurricane and northeaster storm events. Results of the modeling effort indicated a 100-ft-wide berm with an elevation of 5.4 ft National Geodetic Vertical Datum would provide sufficient protection for the 100-year storm event.

Grain size analysis of the native beach and seven borrow areas indicated that an overfill ratio of 1.37 was appropriate for the design. Applying this value, initial beach-fill volumes were calculated in addition to 2- and 5-year advanced nourishment quantities.

20. DISTRIBUTION/AVAILABILITY OF ABSTRACT ☒ UNCLASSIFIED/UNLIMITED ☐ SAME AS RPT. ☐ DTIC USERS	21. ABSTRACT SECURITY CLASSIFICATION Unclassified
22a. NAME OF RESPONSIBLE INDIVIDUAL	22b. TELEPHONE *(Include Area Code)* | 22c. OFFICE SYMBOL

18. SUBJECT TERMS (Continued).

Beach-fill design
Beach nourishment
Borrow material
Dune erosion
Erosion
Grain size analysis
Numerical modeling
Virginia Beach, VA

PREFACE

The US Army Engineer District, Norfolk (CENAO), requested the Coastal Engineering Research Center (CERC), US Army Engineer Waterways Experiment Station (WES), to assist in the design of a Beach Erosion and Hurricane Protection Project for Virginia Beach, Virginia. The study was divided into two major parts consisting of a seawall design and beach nourishment design. This report is the third in a series of three and addresses the evaluation and design of the beach fill. Funding authorizations by CENAO were granted in accordance with Intra-Army Order No. AD-86-3018.

This study was conducted at CERC under the general direction of Dr. James R. Houston, Chief, CERC; Mr. Charles C. Calhoun, Jr., Assistant Chief, CERC; Mr. Thomas W. Richardson, Chief, Engineering Development Division; Mr. H. Lee Butler, Chief, Research Division; Mr. Claude E. Chatham, Jr., Chief, Wave Dynamics Division; Ms. Joan Pope, Chief, Coastal Structures and Evaluation Branch; and Mr. D. D. Davidson, Chief, Wave Research Branch. This report was prepared by Mr. Mark Hansen and Dr. Norman Sheffner. Report editing was performed by Ms. Lee T. Byrne, Information Technology Laboratory.

This study was closely coordinated with Mr. Dave Pezza, CENAO Project Manager; Mr. Jerry Swean, District Geologist; and Mr. Paul Bowen, CENAO. Acknowledgment is made to all others involved at CENAO for their assistance in the study.

Commander and Director of WES during the publication of this report was COL Larry B. Fulton, EN. Technical Director was Dr. Robert W. Whalin.

CONTENTS

CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
cubic yards	0.7645549	cubic metres
feet	0.3048	metres
miles (US statute)	1.609347	kilometres
pounds (mass)	0.4535924	kilograms

COASTAL ENGINEERING STUDIES IN SUPPORT OF VIRGINIA BEACH, VIRGINIA BEACH EROSION CONTROL AND HURRICANE PROTECTION PROJECT

EVALUATION AND DESIGN OF BEACH FILL

PART I: INTRODUCTION

Project Description

1. The proposed Virginia Beach, Virginia, Beach Erosion Control and Hurricane Protection Project is one of the largest and most complex coastal erosion and flooding projects in recent US Army Corps of Engineers experience. The City of Virginia Beach is located on the east coast of the United States just south of the entrance to Chesapeake Bay (Figure 1). The project area consists of 6 miles* of heavily developed commercial and urban shoreline that extends north from Rudee Inlet to 89th Street (Figure 2). This shoreline is subject to severe damages from both hurricanes and extreme extratropical storms. Both the August 1933 hurricane and the March 1962 extratropical storm ("the Ash Wednesday storm") devastated this coastal area. Storm damages have included loss of the beach, destruction of the bulkhead and seawall system, damage to buildings, and inshore flooding. In addition, beach erosion has been a continuing problem. Since 1962, annual harbor dredging and pumping operations to by-pass sand at Rudee Inlet and/or the trucking in of sand from other sources have been sponsored by the Federal, State, and city governments to maintain a beach width of approximately 65 ft and a crest elevation of +5.4 ft.**

2. Existing protection consists of a combination of various bulkheads with crest elevations between 10 and 12 ft NGVD and nourished beach. In 1970 the US Army Engineer District, Norfolk (CENAO), completed a feasibility study that recommended construction of a sheet-pile seawall with a concrete cap at elevation 15 ft NGVD and heavy stone at the base. By 1983, results of the previous study had been reevaluated and incorporated into an initial (Phase I) seawall design and beach erosion concept. The seawall was designed with

---

* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 3.

** All elevations cited herein are in feet referred to the National Geodetic Vertical Datum (NGVD).

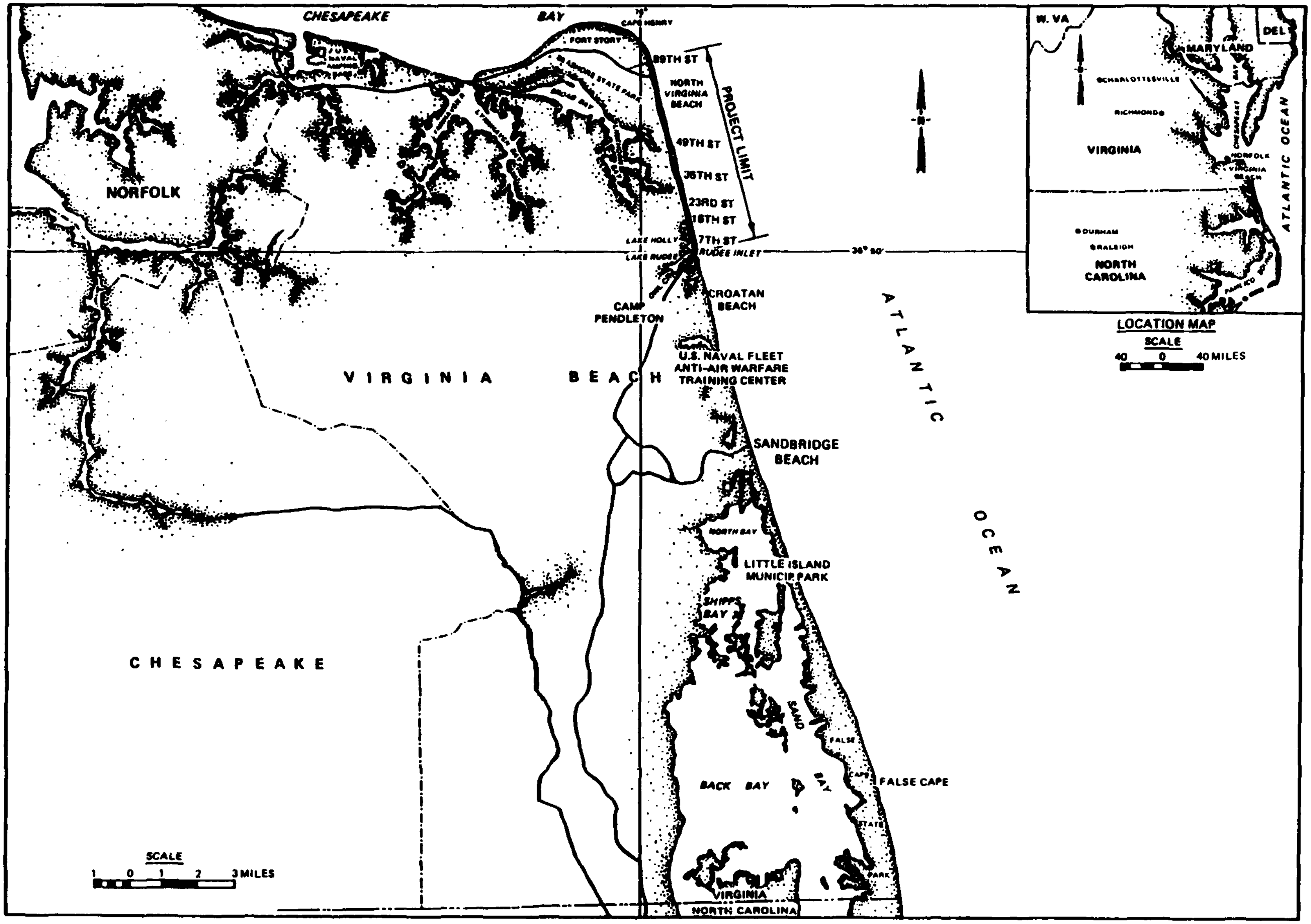


Figure 1. Location map of study area

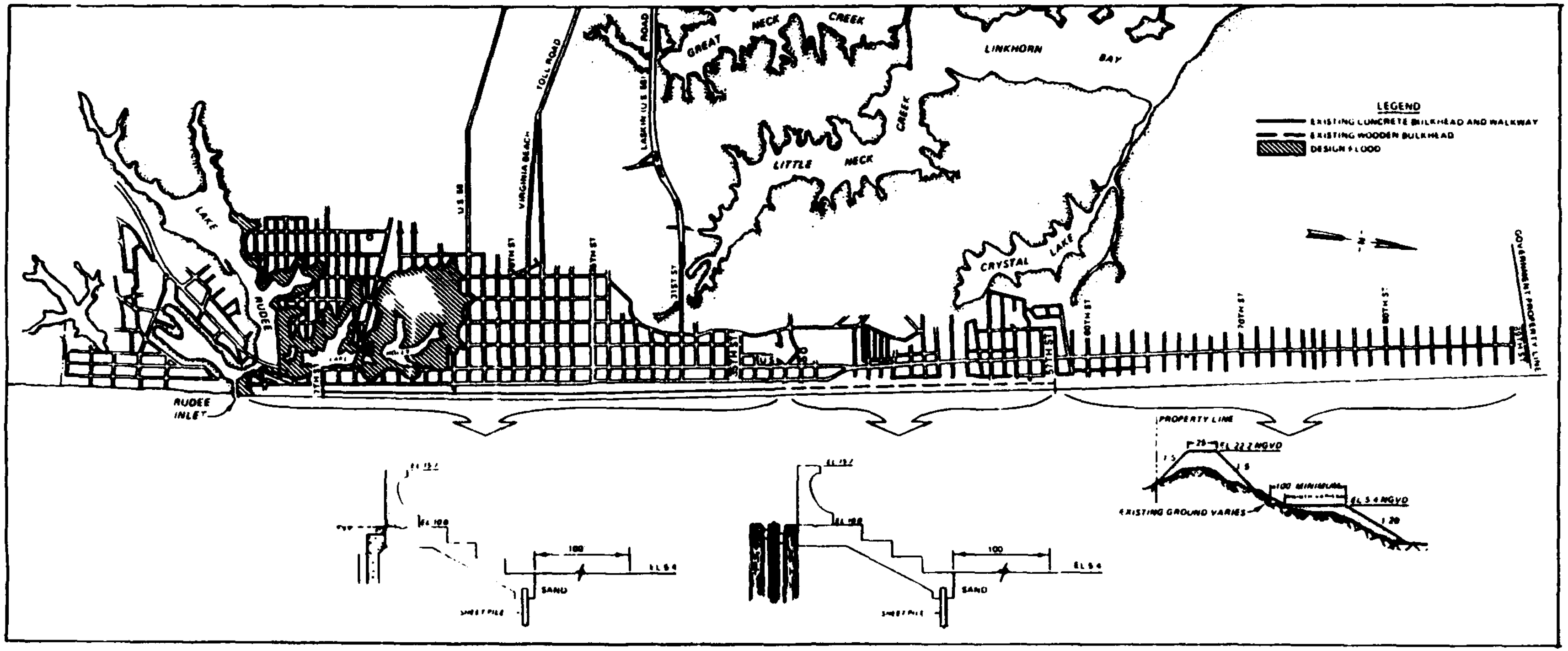


Figure 2. Project reach, Rudee Inlet to 89th Street

guidance from the Shore Protection Manual (SPM) (1984) based primarily on monochromatic wave theory. Adequate storm protection was to be provided by the seawall without sacrificing aesthetics of the ocean view.

3. The proposed construction project is a new stepped-face seawall with curved parapet, located just seaward of the existing seawall (between Rudee Inlet and 57th Street). The existing dune field will be raised and widened from 57th Street, north to 89th Street. Both flood-control structures will be fronted by a continuously maintained beach berm.

Study Background

4. This report is the last of a series of three reports on Coastal Engineering Studies which were conducted by the Coastal Engineering Research Center (CERC), US Army Engineer Waterways Experiment Station, to assist the CENAO in the Advanced Engineering and Design of the Virginia Beach, Virginia, Beach Erosion Control and Hurricane Protection Project. The other two reports (Heimbaugh et al. 1988; Lillycrop, Pope, and Abel 1988) consist of the seawall design (i.e., physical model, overtopping tests, and physical model pressure or wave loading tests) and selection of the design parameters and results of the physical model seawall overtopping tests. This report evaluates the effectiveness of various beach-fill designs and concludes with design of the selected plan. Figure 3 presents a flowchart of the coastal engineering studies.

5. Selection of design waves, storm surge hydrographs, and runup-overtopping rates was crucial to the development of the most hydraulically efficient seawall geometry and in defining the short-term beach stability. Coastal engineering studies consisted of selection of design storms from the historical record, simulating the wave field for each of these storms, establishing the design surge hydrographs, and developing a two-dimensional (2-D) hydrographic model to test and measure overtopping rates.

6. The seawall overtopping tests were concerned with hydraulically designing the most efficient seawall plan and developing overtopping rates that could be used in interior flooding design. The study used a 2-D physical model (Heimbaugh et al. 1988) over a range of sea state conditions to measure wave-induced overtopping. Parameters which required evaluation and were

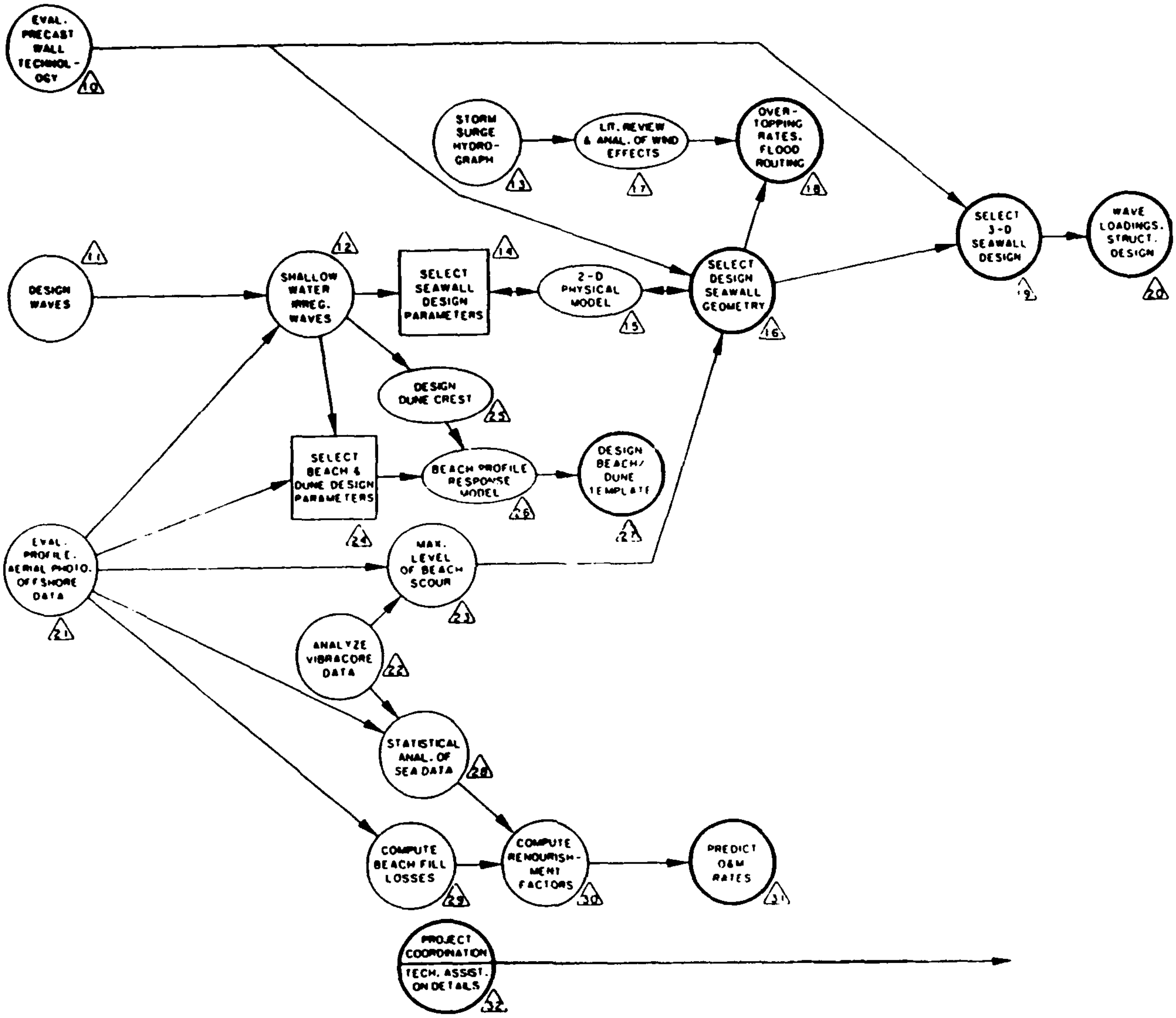


Figure 3. Flowchart for coastal engineering studies Virginia Beach, Virginia

incorporated into the model test were design wave conditions, the design storm surge levels, seawall geometry, and beach profile.

7. This report is divided into two parts. The first part discusses results of numerical modeling using a profile response model to evaluate the effectiveness of various beach-fill designs. Based upon results of the numerical modeling effort, standard empirical procedures, and field data, the second part contains engineering parameters of the recommended beach-fill design.

PART II. BEACH-FILL DESIGN EVALUATION

8. The objective of this section is to determine if the beach-fill and dune designs proposed by the CENAO are sufficient for protecting the low-lying backshore areas from storm-induced flooding. The area of concern extends from Rudee Inlet to 88th Street of Virginia Beach, Virginia. Two designs are evaluated: one with a protective seawall (Figure 4) and one without a seawall (Figure 5). In each case, a 100-ft flat berm is specified in the design profile.

9. The numerical model used to evaluate the proposed design is the modified Kriebel beach erosion model. The basic theory of the model is described in detail by Kriebel (1984a, b), who provides a program listing. Birkemeier et al. (1987) discuss model strengths and limitations. A modification was made at CERC to expand the capability of the model to include the simulation of vertical seawalls. This enhancement will be subsequently discussed. A first fundamental assumption of the model is that the offshore depth can be described by an equilibrium relationship of the following form:

$$h(x) = Ax^{2/3} \tag{1}$$

where

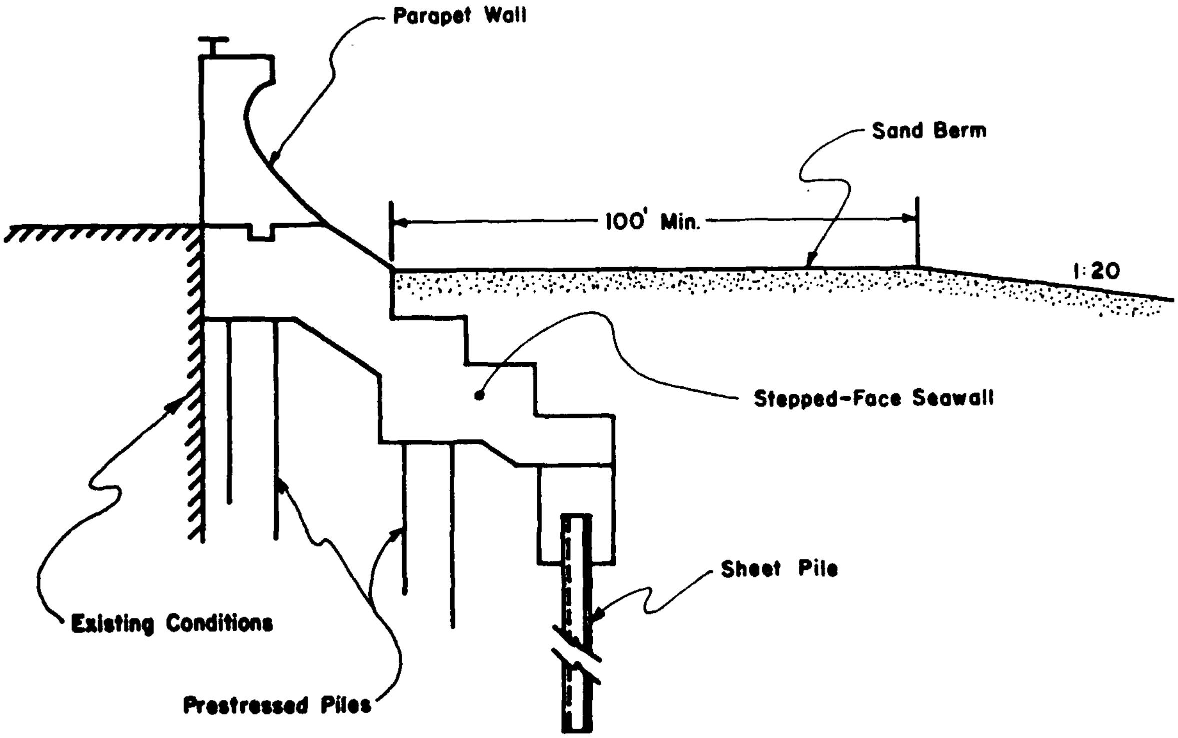
h = depth below mean sea level (MSL)

A = equilibrium coefficient

x = distance offshore

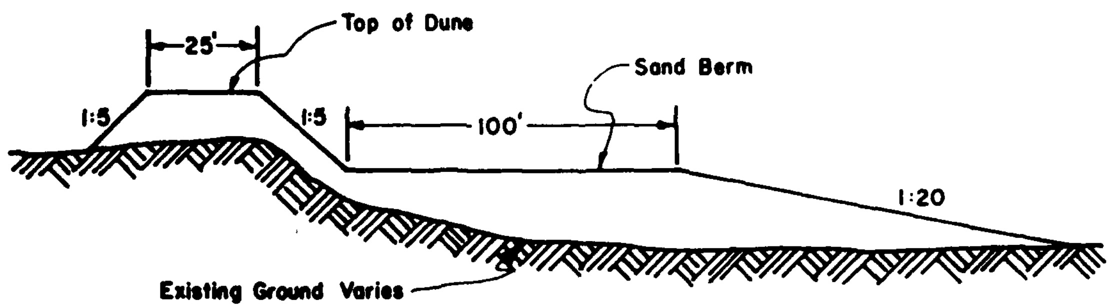
This relationship was first reported by Bruun (1954) and was substantiated by Dean (1977) in a study in which Equation 1 was shown to satisfactorily describe 502 offshore profiles measured along the eastern coast of the United States. Subsequent evaluations by Hughes (1978) and Moore (1982) have shown the equation to be valid for beach profiles in the laboratory and for beaches in various parts of the world.

10. The above studies have shown that the equilibrium coefficient A is correlated to the mean grain size of the beach material. For example, a grain size of 0.25 mm corresponds to an A value of approximately 0.13 $m^{1/3}$ (0.19 $ft^{1/3}$). Figure 6, reproduced from Moore (1982), shows the relationship of grain diameter to equilibrium coefficient. The preferred procedure for



RUDEE INLET TO 57th STREET

Figure 4. Typical design cross section with protective seawall with 100-ft-wide berm



57th TO 89th STREETS

Figure 5. Typical design cross section without protective seawall with 100-ft-wide berm

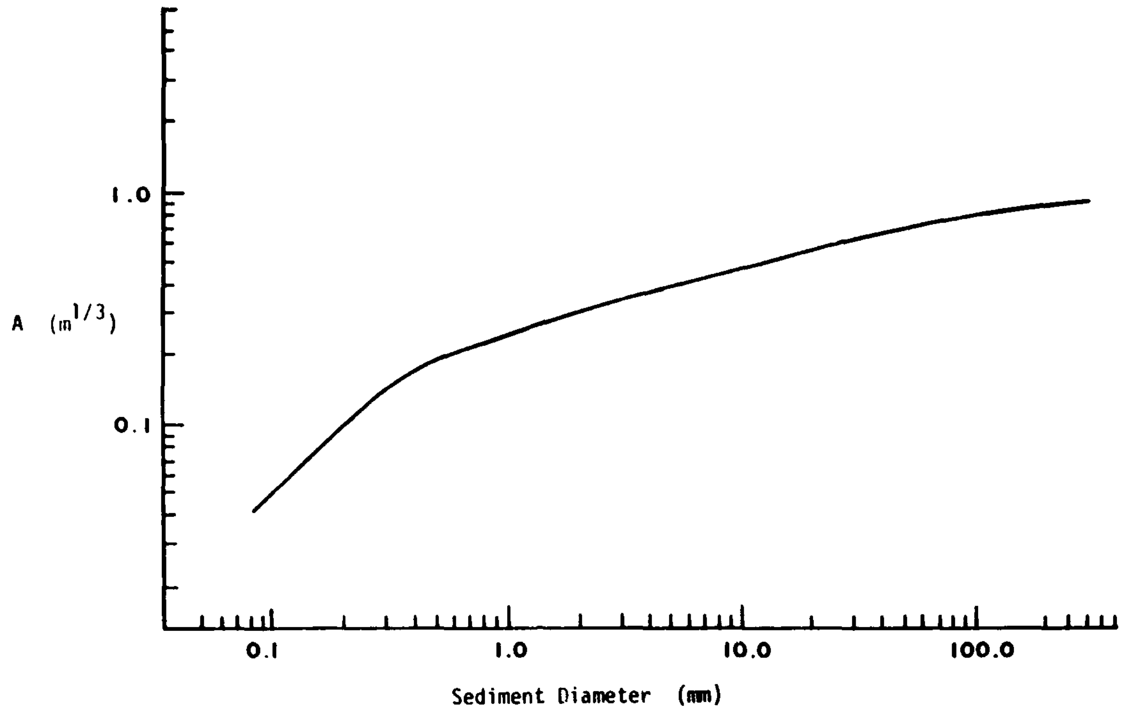


Figure 6. A versus sediment diameter (after Moore 1982)

determining an A value for a given location is to use the existing offshore bathymetry to compute an average, or best fit, coefficient. If the existing profile cannot be considered to be in equilibrium (i.e., a storm event has recently occurred or coastal structures are in the immediate vicinity) or if no profile data are available, the mean grain size (D_{50}) can be used to estimate a value for the coefficient A according to Figure 6.

11. The equilibrium profile of the form of Equation 1 was shown by Dean (1977) to result from the assumption that the equilibrium profile is a function of the dissipation of wave energy per unit volume of water in the active surf zone. Transport of sediment in the surf zone is assumed to be a primary function of the dissipation of the flux of this wave energy. This basic assumption is used to formulate a sediment transport equation in the following form:

$$Q_c = k(D - D_{eq}) \tag{2}$$

where

D = energy dissipation written as

$$D = \frac{1}{h} \frac{\partial(EC_g)}{\partial x}$$

E = total wave energy

C_g = group velocity

The parameter D_{eq} represents the dissipation for an equilibrium profile in which the depth and offshore distance are related according to Equation 1. The parameter k is an empirical coefficient found by Moore (1982) to have a value of 2.2×10^{-6} m^4/N (0.001144 ft^4/lb) . Linear wave theory defines the two dissipation relationships as follows:

$$D = \frac{5}{16} \rho \gamma g^{3/2} h^{1/2} \partial h/\partial x \tag{3}$$

$$D_{eq} = \frac{5}{24} \rho \gamma g^{3/2} A^{3/2} \tag{4}$$

In Equations 3 and 4, ρ represents the fluid density, g represents the acceleration of gravity, and γ represents the ratio of wave height H to depth h .

12. The above derivations assume the surf zone is dominated by spilling breakers such that the breaking wave height is a constant fraction of the depth, i.e. $H = \gamma h$. Equations 2, 3, and 4 are used to compute a net volume of erosion or deposition for the area bounded approximately by the shoreline (MSL) and the breaker line. Note that for a beach in equilibrium, no transport of material occurs, since $D = D_{eq}$. The basic premise of the model is that this volume of material is either supplied from the dune and berm or deposited on the berm depending on whether erosion or deposition is indicated offshore. Deposition on the dune face during poststorm recovery is not permitted. The assumption is therefore made that erosion of the dune is irreversible.

13. The equilibrium profile concept has one disadvantage; it indicates that the offshore depth monotonically increases with offshore distance. This assumption precludes the formation of offshore bars or troughs which are known to occur in many locations; however, the average profile has been shown to be

adequately described by this equilibrium profile concept (Dean 1977). The primary goal of the dune erosion model is not to compute offshore erosion patterns, but to compute the above MSL changes that result from a specific storm event based on the total volume of erosion (or deposition) occurring between the shoreline and the breaker zone. The use of an average offshore profile to compute this total volume of erosion is therefore a reasonable assumption for areas which are not dominated by offshore anomalies that produce highly variable longshore transport patterns.

14. The present equilibrium profile approach to dune erosion modeling also assumes that alongshore transport is constant and that erosion of the dune face is solely a function of the onshore-offshore transport. This one-dimensional (1-D) (onshore-offshore) approximation of a 2-D (onshore-offshore/alongshore) natural process is an acceptable one when alongshore transport is in balance. For example, sediment entering the test section is assumed to leave the test section in the identical spatial and temporal distribution such that the net alongshore effect is negligible on the profile. This assumption is a reasonable one for the Virginia Beach project, which is characterized by a relatively straight coastline free of major structures such as jetties or breakwaters. Although the 1-D simplification is well suited for the Virginia Beach area, the potential impact of the approximation on the computed results must be critically evaluated. For example, breaching of the dune complex was experienced during the 1962 northeaster. This undoubtedly occurred because of variations in the dune height along the dune crest, such as walk-over areas and locations of channelized wind erosion, in which the 2-D assumption is violated. Similarly, the net alongshore littoral drift has been reported to be to the north. The extreme southern limit of beach fill could become sediment starved as the newly placed material moves north with no additional material to replace it from the south. In this case, that 1-D assumption would also be violated since the alongshore inflow would not equal the alongshore outflow. These types of analyses must be considered in the evaluation of the results of any idealized numerical model. As with all numerical models, it is the responsibility of the user to correctly interpret the numerical results. This is especially true for sediment transport models in which the governing physics are not well understood, and empirical relationships are used to describe the transport process. This approach is acceptable as long

as the basic assumptions used in the determination of the relationships are not severely violated.

15. A quantitative evaluation of the dune erosion model was made by Birkemeier et al. (1987), in which 14 prestorm and poststorm surveys representing four separate storm events were used to test the capability of the model to realistically predict volumes of erosion resulting from known storm surges. Results of the comparison showed that the average percentage of deviation between calculated erosion volumes above MSL and measured volumes was 109 percent. This value represents the average of five underpredictions averaging 55 percent of the measured and eight overpredictions representing 145 percent of measured. One profile of the 14 was influenced by adjacent structures and was not used in the above percentages. The profiles used for this analysis did not include profiles with seawalls; therefore, an additional evaluation of the model was made by Kraus et al. (in preparation). Measured elevation changes in front of a seawall resulting from a documented storm surge were compared with elevation changes as computed by the model with the seawall modification. Three prestorm and poststorm surveys were used to test the model. Average/ predicted elevation changes for the three profiles were -0.31/-0.26, -0.50/-0.42, and -0.32/-0.04 m, respectively. Total volumes of erosion could not be compared since adequate prestorm and poststorm data were not available. Conclusions of both verification comparisons of the model show it to be capable of acceptably predicting storm-related erosion. In view of the fact that a natural variability in observed erosion occurs on what would appear to be a straight and homogeneous section of beach, the predictions of the model to be approximately ±50 percent of the observed values represent a positive feature of the model. A worst case scenario can be made by applying a "variability factor" of 2.0 to the model predictions. Support of a factor of this magnitude was reported by Birkemeier et al. (1987) and Chiu and Dean (1986). The dune erosion model has been shown to effectively predict quantities of erosion resulting from single storm events of known surge level and duration. The initial analyses that follow are also made for single storm events in which the initial beach profile is assumed to be the design profile. If multiple storm events occur between beach renourishment, a corresponding increase in erosion of the dune should be anticipated. A limited analysis of multiple events is made to address this additional volume.

16. Detailed profiles from 19 locations along Virginia Beach were used to compute an average equilibrium coefficient for use in the model. The individual values of the coefficients for each of the profiles are shown in Table 1. As can be seen, the coefficients indicate that all profiles are very similar in shape. This result demonstrates that the project area is well suited to the 1-D analysis of the dune erosion model. All of the above profiles were supplied to CERC by CENAO and represent profiles collected on 20 June 1986. The computed range of coefficient values is 0.165 to 0.190 $ft^{1/3}$, corresponding to a sediment diameter of 0.25 to 0.30 mm. An approximate average value of 0.18 $ft^{1/3}$ was used for all computations. This value corresponds to a mean sediment diameter of 0.28 mm, which is consistent with the sediment analysis provided CENAO by Waterways Surveys & Engineering Ltd. (1984) for the Cape Henry area. Material of this diameter is also consistent with the following potential sources of fill material reported by CENAO (1984): Linkhorn Bay (0.29 mm), The Narrows and Broad Bay (0.25 to 0.34 mm), Fort Story and Seashore State Park (0.26 mm), Lynnhaven Inlet (0.28 mm), Ocean Naval Air Station area (0.34 mm north, 0.26 mm south), Charity and Pungo Ridges (0.32 mm). Coarser material was reported for Bonney's Corner (0.39 mm) and the east end of the Thimble Shoal channel (0.42 mm), while finer material was indicated offshore of the Virginia Beach project (0.19 mm) and the Dam Neck disposal site (0.16 mm).

Table 1

Equilibrium Coefficients

Street	Coeff. $ft^{1/3}$	Street	Coeff. $ft^{1/3}$	Street	Coeff. $ft^{1/3}$
2	0.165	5	0.165	8	0.170
11	0.180	14	0.180	16	0.180
18	0.170	21	0.175	24	0.180
27	0.180	30	0.190	33	0.180
35	0.180	38	0.180	40	0.185
43	0.180	46	0.180	49	0.185
52	0.180				

17. The two beach-fill designs provided by CENAO were schematized to best represent the design profiles shown in Figures 4 and 5. These data were input to the dune erosion model for all simulations. The NGVD and MSL are equal in value for the project area, and all elevations are referenced to these datum.*

18. Four storm events of record were supplied by CENAO for evaluation of the two proposed beach-fill designs. A request was also made by CENAO to test the design dune system with a hurricane having a peak surge elevation of 9.5 ft. This test storm was assembled by scaling the 1933 hurricane such that the peak surge was linearly scaled up from 8.7 to 9.5 ft. The approximate return period for each storm event was extracted from the adopted Virginia Beach elevation-exceedance curves published by CENAO (1983) and shown in Figure 7. Pertinent storm-related data, representing three northeasters and two hurricanes, are shown in Table 2. The surge profiles are shown in Appendix A.

19. Each numerical simulation generates a complete updated cross section at each computational time step. To summarize these results in a concise and meaningful manner, two output values are reported for each simulation. First, the maximum computed poststorm recession of the dune/berm face is provided as a meaningful indicator of potential storm damage by structural undermining. The second output represents the net volume of erosion per foot of beach width (alongshore) that results from the total storm event. This figure reflects the fact that beach recovery does occur and indicates the amount of beach renourishment which would be required to restore the beach to prestorm conditions. Results of the initial simulations are shown in Table 3.

20. The prestorm and poststorm profiles for each of the above storm events for both the with and without seawall designs are shown in Appendix B. Note that for the storm events with surges greater than 5.5 ft, no erosion of the dune face is indicated in spite of the submergence of the berm. This is a consequence of the numerical algorithm in which the assumption is made that the flat berm section erodes first before the dune erodes. If the surge level remains in contact with the dune face for a substantial portion of the storm event without completely eroding the flat berm, this assumption may represent a poor approximation of the natural process. In this case, an evaluation of the results should be made as mentioned above. Note also in Table 3 that a

* Personal Communication, 1987, R. Owen Reece, Jr., CENAO, Norfolk, VA.

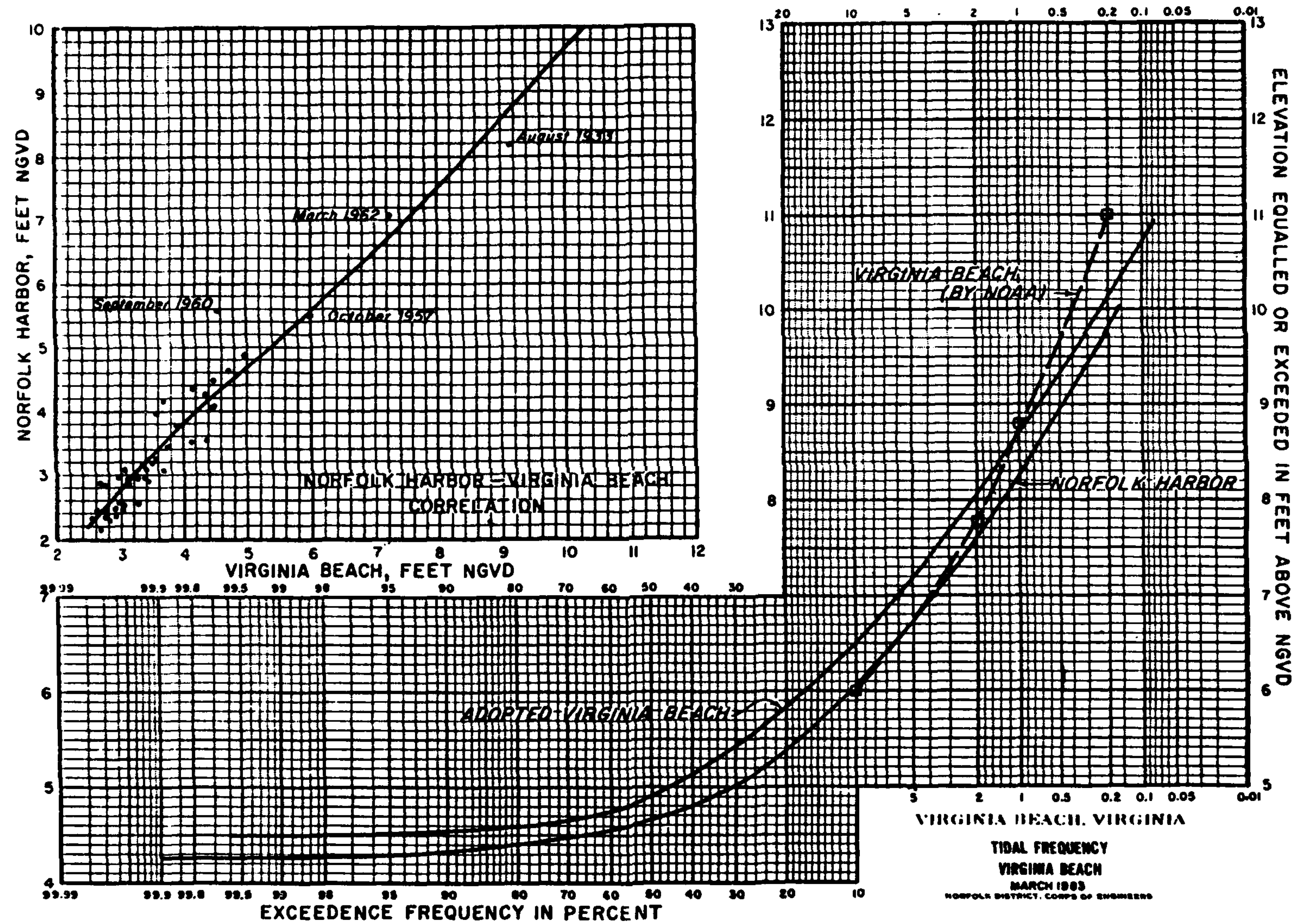


Figure 7. Exceedance frequency curves for Virginia Beach, Virginia

Table 2

Design Storms

Date	Type	Peak Surge ft	Duration hr	Return Period* years
August 1933	hurricane	8.7	23.0	100
Test storm	hurricane	9.5	23.0	200
April 1956	northeaster	6.3	71.0	10
March 1962	northeaster	6.7	78.0	15
April 1978	northeaster	5.8	56.0	5

* Based on still-water level of the surge.

Table 3

Model Results, 100-ft Berm

Storm	Design	Maximum Recession ft	Volume of Erosion yd^3/ft
August 1933	no seawall	-50.63	7.87
	seawall	-50.72	7.87
Test	no seawall	-56.76	8.50
	seawall	-56.89	8.50
April 1956	no seawall	-25.96	4.97
	seawall	-25.94	4.98
March 1962	no seawall	-42.27	9.45
	seawall	-42.27	9.45
April 1978	no seawall	-41.71	8.27
	seawall	-41.70	8.27

small discrepancy is shown in the with and without seawall results. This difference is a result of the fact that a flat berm cannot be specified as input to the model for the seawall case. Computationally, a 1:200 slope is used for this area, which introduces small discrepancies in value.

21. The numerical results shown in Table 3 indicate that the design storms do not cause erosion of the entire flat berm region. In fact, the 9.5-ft test storm resulted in a maximum recession of only approximately 57 ft.

As seen in figures from Appendix B, a recession of this magnitude does not reach the base of the dune or the face of the seawall. As previously stated, however, a worst case scenario can be surmised by applying a variability factor of 2.0 to these maximum recession values. Even with this factor included, the design profiles provided by CENAO are still shown to provide adequate protection to the dune and seawall since erosion of the base on the dune or face of the seawall will just have begun. To test this scenario, a multiple storm simulation was made. Four test storms were constructed by assuming that a single storm event was immediately followed by an additional storm event. The storms used were the 9.5-ft test hurricane and the March 1962 northeaster. The four permutations of these storms (33-33, 33-62, 62-33, and 62-62) were subjected to each of the CENAO designs. Results of these simulations are shown in Table 4. Appendix B represents the prestorm and poststorm profiles. In only two cases (33-33 and 62-33) are erosion of the crest of the dune indicated. These recession values are 1.94 and 1.78 ft respectively. In the respective seawall cases, a volumetric loss of the berm elevation up to 2 ft occurred over its entire width to the base of the seawall.

Table 4

Model Results, Multiple Simulation

Storms	Design	Maximum Recession ft	Volume of Erosion yd^3/ft
33-33	no seawall	-74.68	13.23
	seawall	-82.75	13.23
33-62	no seawall	-53.81	12.67
	seawall	-53.82	12.67
62-33	no seawall	-75.77	14.00
	seawall	-83.28	14.00
62-62	no seawall	-56.92	13.60
	seawall	-56.94	13.60

22. An additional set of simulations was made in which the berm was reduced to a 50-ft width in order to assess the computed erosion resulting from the 9.5-ft surge design storm. Without the protection of the seawall, approximately 5.0 ft of the dune crest would be eroded, with a maximum

recession on the dune/berm face of 34.75 ft. Again, results for the seawalled case indicate a volumetric loss from the berm resulting in an elevation loss up to 2 ft over its entire width to the base of the seawall. Two additional simulations were made to evaluate a case in which the berm sloped from the dune toe (elevation of 5.5 ft) to MSL. Similar results are obtained for these cases, erosion of 5.0 ft of the dune face and crest without a seawall and minor erosion of the berm to the face of the seawall.

23. Additional simulations were made to demonstrate the effects of the 9.5-ft test storm on variations of the foreshore slope of the duned design profile and of the equilibrium profile (which is an indirect measure of the effect of the grain size). Table 5 presents these data. As seen in Table 5, the effects of changing the subject parameters are qualitatively predictable. For example, a steeper foreshore absorbs more storm energy and is therefore subject to greater erosion. Similarly, a smaller grain size erodes more easily than a coarse grain, as is indicated in the table. None of the effects are seen to be dramatic, a result which indicates that the CENAO design is a stable one with respect to the geometric parameters and sediment size.

Table 5

Geometrical Effects on Volume of Erosion

Case of Erosion (with 9.5-ft Surge)	Design	Maximum Recession ft	Volume yd^3/ft
Slope = 1:20	no seawall	-56.76	8.50
Slope = 1:25	no seawall	-40.76	7.47
Slope = 1:30 with A = 0.18	no seawall	-29.69	6.67
A = 0.15 (D50 = 0.10 mm)	no seawall	-74.26	8.75
A = 0.18 (D50 = 0.28 mm)	no seawall	-56.76	8.50
A = 0.21 (D50 = 0.32 mm) with slope = 1:20	no seawall	-43.49	8.31

24. The numerical results presented in this report indicate that the dune/seawall profile design developed by CENAO is adequate for protection of the areas shoreward of the dune line. In no case was the dune crest significantly eroded, nor was the seawall uncovered even when multiple design storms were assumed to follow in succession. Use of the variability factor still shows the CENAO design to be capable of withstanding the design storms with

little or no damage to the dunes or seawall. Results indicate that the energy contained in the specified storms is not adequate for moving the volume of fill material contained in the design profiles.

PART III: BEACH-FILL DESIGN

25. Part II results indicated that a berm width of 100 ft at elevation +5.4 NGVD would provide adequate protection against extremal storm events. The purpose of this study was to design a beach nourishment project using the recommended berm dimensions. To design the project, this study statistically analyzed native and borrow sediment data, evaluated beach profiles for design purposes, calculated overfill and renourishment factors, and computed design and construction volumes and templates.

Methodology

26. The volume of material required to build the beach to the recommended dimensions, i.e. 100-ft berm at +5.4 ft NGVD, was computed based upon the profile translation method (Delft Hydraulics Laboratory 1987; Hansen and Lillycrop 1988). This method assumes that hydrodynamically stable fill material placed on the subaerial beach will eventually be transported seaward until a new equilibrium profile is established. The limit of significant onshore-offshore transport is assumed to be located at the average profile closure depth. Representative profiles of the study area are required in this procedure.

27. Three assumptions are made for this method to be viable. The first assumption is that all hydrodynamically unstable fill material will be winnowed and transported either seaward of the profile closure depth or downdrift from the project site. This winnowing process is assumed to produce an overall resultant grain size equivalent to the native material. The second assumption is that grain size generally controls profile steepness (Bascom 1951, Shepard 1963). If these two assumptions hold true, it can be assumed, in most cases, that the new equilibrium profile will be congruent to the existing profile, however translated seaward. The last assumption is that the translated profile will intersect, or "tie-in," to the existing profile at an average seaward depth of extreme onshore-offshore transport, often termed profile closure (Hallermeier 1981).

28. As a result of the continuous nourishment program at Virginia Beach, it is difficult to ascertain representative profiles that are in dynamic equilibrium with the local wave climate. Therefore, the approach

taken for this project was to use the profile data set which possessed the narrowest berm width. Translating profiles exhibiting a narrow berm width would provide the most conservative (greatest) volumetric requirements for the project. Four profile data sets provided by CENAO to document the typical seasonal variability and short-term trends (March 1986, September 1986, March 1987, September 1987). The March 1987 data set was generally found to have the narrowest berm width (Appendix C). Therefore, using the March 1987 profiles in the profile translation method may provide the maximum benefits in design of a storm protection structure during extreme storm events.

Data Analysis and Results

29. Unique compartments, or subreaches, are typically identified in beach nourishment projects to divide the beach into zones that may exhibit anomalous erosion rates or profile shape. Subreaches are often treated separately when computing design parameters, i.e. design volume, composite grain size. At the request of CERC, CENAO identified seven unique subreaches along the project length. Each compartment was represented by an established profile line which generally bisected the subreach. Table 6 identifies the seven profiles and length of each subreach, designated by street number, used for designing the hurricane protection project. The beach north of 65th Street has an average berm width that exceeds the design berm width (100 ft) computed in Part II of this report. Therefore, all design calculations were computed

Table 6

Length of Each Subreach Designated by Street Location

Profile Street		Length, ft
5th		2,500
11th		5,280
29th		5,300
38th		3,625
49th		1,975
55th		1,850
61st		2,560
	Total	23,090

for the beach area which extends from Rudee Inlet northward to 1,660 ft north of 61st Street, i.e. 65th Street. Total length of the beach-fill design was 23,090 linear ft.

30. A determination of profile closure depth is required when applying the profile translation method for beach-fill design. As stated previously, it is assumed the new equilibrium beach profile will "tie-in" to the existing profile at a depth of profile closure. Hallermeier's (1981) method was used to compute closure depth (d) at Virginia Beach. The nearshore limit of sediment transport as given by Hallermeier was determined by:

$$d = 2.28\, H_e - 68.5 \frac{H_e^2}{gT_e^2} \tag{5}$$

where

d = limit of intense onshore-offshore transport (Mean Lower Low Water (MLLW))

H_e = extreme significant wave height exceeding 12 hr/year defined by:

$$H_e = H_s + 5.6\sigma \tag{6}$$

where

H_s = significant wave height

σ = standard deviation of significant wave height

g = acceleration of gravity

T_e = corresponding wave period

31. Twenty years of wave hindcast data from the Wave Information Study (WIS) (Jensen 1983) Phase II, Station 77, were used in the computation of closure depth. For Virginia Beach, the average significant wave height was 1.8 ft, with a standard deviation equal to 1.87 ft and an average wave period of 5.7 sec. Substituting Equation 6 in Equation 5 and applying the 20-year WIS wave summary data yield an average annual profile closure depth of approximately -19 ft NGVD (-18 ft MLLW).

Design Volumetric Calculations

32. Profile data for the March 1986, September 1986, March 1987, and September 1987 surveys were digitized and recorded in the Interactive Survey Reduction Program (ISRP) (Birkemeier 1984) format. A computer program was developed to translate the representative profiles a specified distance from the baseline, at a given berm height, and intersecting the existing profile at a given closure depth. As specified by results from Part II of this study, a desired berm width of 100 ft from the construction baseline with a berm height of +5.4 ft above NGVD was applied to the seven profiles representing unique subreaches. The profile closure depth of -19 ft NGVD (Equation 5) was used in these calculations.

33. Using ISRP, a volumetric (yd^3/linear ft) difference was computed between the representative profiles and the translated profiles (Table 7) (Appendix D). This volume was then multiplied by the length of subreach represented by that particular profile. The analysis indicates that to build the 23,090 ft of beach to design dimensions with adjustment to depth of closure, 1,228,000 yd^3 of borrow material with a grain size similar to the native beach would be required. Theoretically, when redistribution of the beach fill has occurred across the active profile, forming a new equilibrium profile shape,

Table 7

Design Quantity Estimates for Native Beach Subreach

Subreach	Length, ft	yd^3/ft	yd^3	Ra*, yd^3
5th	2,500	8	20,000	31,000
11th	5,280	56	295,700	458,300
30th	5,300	57	302,100	468,200
38th	3,625	81	293,600	455,100
49th	1,975	57	112,600	174,500
55th	1,850	55	101,800	157,700
61st	2,560	40	102,400	158,700
Total	23,090		1,228,200	1,903,600

* Overfill ratio of 1.55 multiplied by cubic yards for each subreach.

the entire project would have a resulting berm width of approximately 100 ft from the baseline with a berm elevation of +5.4 ft NGVD.

Grain Size Analysis

Offshore borrow material

34. Dredged material from the Thimble Shoals navigation channel was selected by CENAO as the borrow for the Virginia Beach Hurricane Project (Figure 8). To characterize the borrow area, 24 cores were taken within Thimble Shoals. Channel sediment samples were extracted from each core based upon stratigraphic units or unique sediment characteristics. The raw grain size data from each unit for all cores was provided to CERC for grain size composite computations. A total of seven zones within Thimble Shoals were initially identified by CENAO as potentially suitable borrow areas. Table 8

Table 8

Cores Samples Used to Compute Borrow Area Grain Size Composites

Core Number	Elevation, ft (NGVD)	Borrow Area
83VC-52	-53.0 to -60.0	#1
84VC-162	-51.0 to -53.3	#1
83VC-83	-41.0 to -45.0	#1
83VC-53	-52.0 to -55.4	#1
83VC-54	-52.0 to -55.5	#1
84VC-164	-40.0 to -54.0	#1
84VC-165	-52.5 to -54.5	#1
83VC-84	-40.6 to -55.5	#1
83VC-55	-50.0 to -54.0	#1
85VC-228	-40.0 to -55.0	#2, #3, #7
85VC-229	-53.0 to -59.0	#2, #3, #7
83VC-56	-52.0 to -60.0	#2, #3, #7
83VC-85	-41.0 to -55.0	#2, #3, #7
84VC-167	-46.0 to -56.0	#2, #3, #7
84VC-166	-52.0 to -56.0	#2, #3, #7
83VC-57	-52.0 to -60.0	#2, #3, #7
83VC-58	-53.0 to -60.0	#2, #4, #6, #7
84VC-168	-50.0 to -58.0	#2, #4, #6, #7
84VC-169	-54.0 to -61.0	#2, #4, #6, #7
83VC-59	-53.0 to -60.0	#2, #4, #6, #7
83VC-60	-52.0 to -60.0	#2, #5, #6
84VC-170	-45.0 to -53.0	#2, #5, #6
84VC-171	-52.0 to -57.0	#2, #5, #6
83VC-61	-52.0 to -57.0	#5

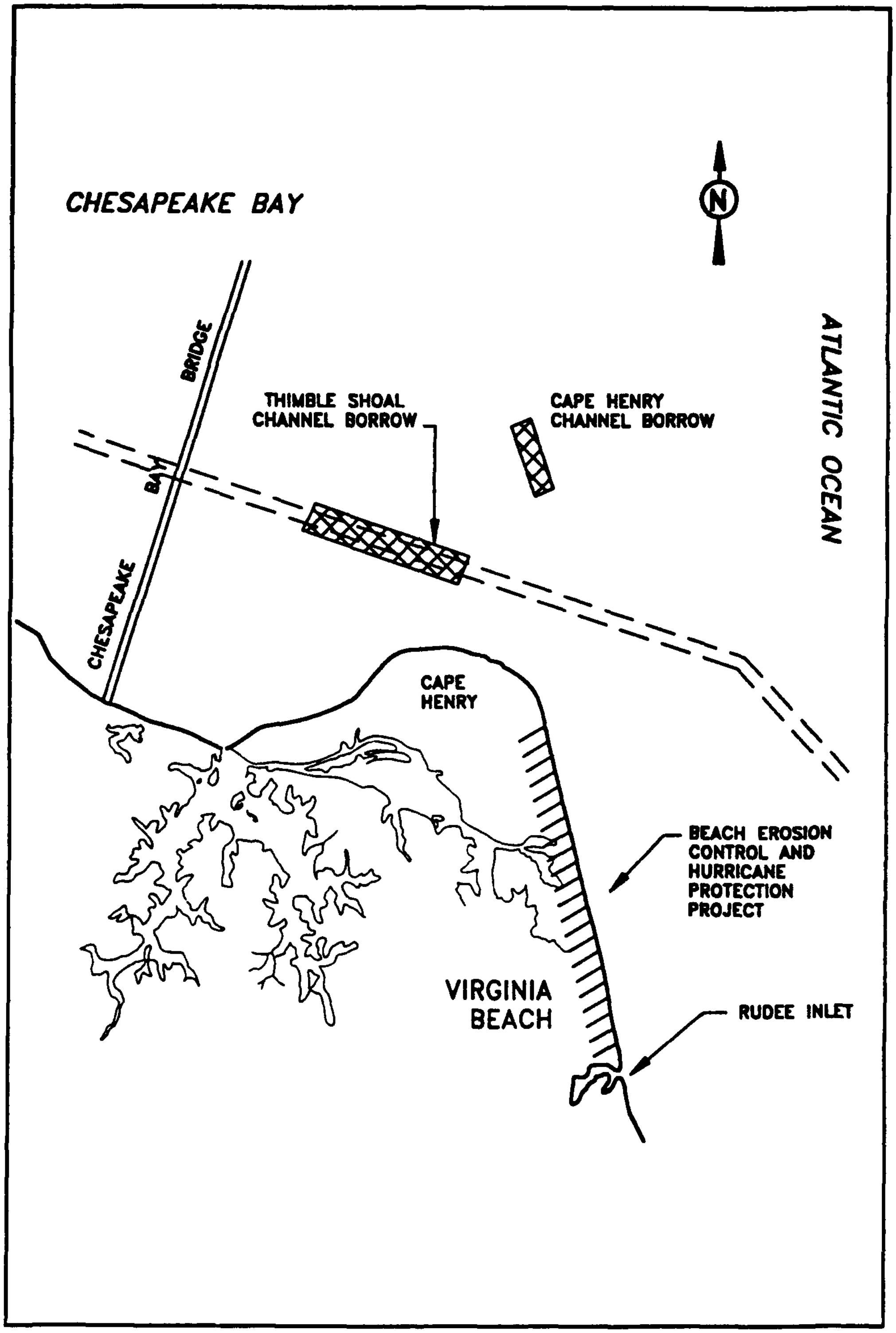


Figure 8. Location map of Thimble Shoals borrow area

indicates cores and depth locations where sediment samples were taken to statistically create composites for the seven zones.

35. A computer program was developed at CERC to compute composite sediment grain size distributions for any borrow area or native beach. Various schemes were developed to weight individual sediment samples to create statistically correct composites. Borrow area sediment composites could be computed in this program based upon core location, depth, beach profile location(s), or any combination of the three. For this particular project, borrow area composites were created by selecting individual samples within cores between defined depth intervals. The percentage of sample within the specified depth interval was multiplied by the sample's grain size distribution. An example would be to create a borrow area composite consisting of all sediment samples between depths of -50 to -60 ft NGVD taken from cores 83VC-52, 83VC-53, and 83VC-54. If a particular sample in one of the cores represented a stratigraphic unit from -50 to -55 ft, a weighting factor of 50 percent was multiplied by the individual weights of each sieve interval, i.e. (-50 - -55)/(-50 - -60) × 100 = 50 percent. Remaining samples located between the -50 to -60 ft depth interval would be weighted in a similar manner. Using this procedure, a single composite was created for each core with each core composite possessing a weighting factor of one. The result was a single grain size composite which represents that particular borrow area.

36. Table 9 represents the composite grain size distribution for the four borrow areas selected by CENAO. Based upon results from the seven borrow sources, four borrow sites were selected for further investigation. These sites were borrow areas designated #2, #3, #6, and #7.

Native beach material

37. Native beach sediment characteristics are required for beach-fill design. The sediment grain size characteristics of a beach vary: (a) across the beach profile, (b) along the beach, and (c) between seasons (Hobson 1977). To accurately describe this variability, temporal and spatial native beach sediment samples should be mathematically combined to create an overall composite grain size distribution. Using this procedure, one composite grain size distribution reflects the three components of variability. Accuracy of the composite is crucial as the native beach composite, as well as borrow composite, determines the project's overfill ratio and ultimate construction and maintenance costs.

38. Natural native sediment characteristics at Virginia Beach have been obscured by continuous beach replenishment efforts since the early 1950's.

Table 9

Overfill (Ra) and Renourishment (Rj) Factors

for Native Beach Subreaches

			Borrow Sources							
			#2		#3		#6		#7	
			Mean	SD	Mean	SD	Mean	SD	Mean	SD
			1.74	1.24	1.72	1.27	1.78	1.26	1.70	1.11
Subreaches	Mean	SD*	Ra	Rj	Ra	Rj	Ra	Rj	Ra	Rj
5th										
March	1.67	0.86	1.21	0.63	1.22	0.59	1.24	0.64	1.15	0.73
Sept	1.56	1.11	1.16	1.04	1.15	0.99	1.19	1.06	1.15	1.12
Both	1.64	0.99	1.16	0.83	1.16	0.79	1.19	1.06	1.15	1.12
11th										
March	1.68	0.84	1.22	0.60	1.22	0.55	1.25	0.60	1.15	0.69
Sept	1.49	1.09	1.23	1.09	1.21	1.03	1.26	1.10	1.23	1.18
Both	1.62	0.97	1.18	0.82	1.18	0.78	1.21	0.84	1.11	0.92
30th										
March	1.38	0.90	1.40	0.95	1.38	0.89	1.44	0.97	1.35	1.08
Sept	1.63	1.11	1.11	0.98	1.11	0.93	1.14	0.99	1.07	1.06
Both	1.50	1.03	1.23	1.01	1.22	0.95	1.27	1.02	1.20	1.11
38th										
March	1.52	0.85	1.31	0.74	1.31	0.68	1.35	0.75	1.24	0.86
Sept	1.44	1.20	1.32	1.24	1.27	1.19	1.36	1.26	1.37	1.32
Both	1.46	1.07	1.26	1.09	1.24	1.04	1.30	1.11	1.27	1.19
49th										
March	1.47	0.84	1.36	0.76	1.35	0.71	1.39	0.77	1.29	0.89
Sept	1.30	1.09	1.48	1.29	1.42	1.23	1.52	1.31	1.55	1.40
Both	1.38	0.98	1.36	1.07	1.34	1.01	1.40	1.09	1.35	1.19
55th										
March	1.46	0.82	1.38	0.74	1.37	0.68	1.41	0.75	1.31	0.87
Sept	1.40	1.04	1.33	1.12	1.30	1.06	1.37	1.14	1.34	1.23
Both	1.41	0.94	1.35	0.98	1.34	0.92	1.39	1.00	1.30	1.10
61st										
March	1.45	0.84	1.37	0.78	1.36	0.73	1.41	0.79	1.31	0.91
Sept	1.20	1.19	1.72	1.51	1.64	1.44	1.77	1.53	1.88	1.61
Both	1.29	1.06	1.48	1.27	1.43	1.21	1.53	1.29	1.55	1.39
Winter	1.59	0.85	1.27	0.68	1.26	0.63	1.30	0.69	1.20	0.79
Summer	1.42	1.13	1.32	1.20	1.28	1.14	1.36	1.22	1.36	1.29
All	1.55	1.00	1.30	0.95	1.27	0.87	1.34	0.94	1.25	1.02

* SD = standard deviation.

Traverse Group Inc. (1980) performed an historical sedimentological study at Virginia Beach using data from 1951 to 1977 and found several trends. The study found that the median grain size above mean low water (MLW) was coarsest (0.38 mm, 1.40) in 1951 gradually becoming finer with the finest (0.22 mm, 2.19) recorded in February 1977. However, the median grain size below MLW has remained fairly constant through time. The spatial variation for the five profiles surveyed is inconsistent through time. The study also found the beach sediment has become more uniform, i.e. lower standard deviation, since 1951. Sediment characteristics of the present beach reflect mainly the replenished material. These trends are probably a result of the continual beach replenishment practices.

39. To describe the present sedimentological condition at Virginia Beach, native beach sediment data for 13 profile lines were provided by CENAO. Surface sediment samples for the 13 lines were collected for four time periods: March 1986, September 1986, March 1987, and September 1987. Samples were collected at eight locations along each profile line: +3.8, 1.8, 0.1, -1.6, -6.6, -11.6, -16.6, and -21.6 ft NGVD.

40. Native composite grain size distributions were computed for the seven subreaches identified for the project. These seven subreaches were the same used for the volumetric analysis. Unlike the profile data where one profile represents a subreach, the sediment subreaches were represented by data from three profiles. Sediment data from three profile lines were used to decrease the natural variability in the grain size statistics. The center profile was the same used in the volumetric analysis with additional sediment data provided from adjacent profiles. Winter (March 1986 and 1987), summer (September 1986 and 1987), and winter-summer composites were computed. The winter and summer composites represent temporal variations of the native beach. It should be noted that winter-summer composites are not an average of the two, but, a composite of all individual samples for these periods. For native beach composites, each surface sample was assigned an equal weighting.

41. Typically, sediment samples from across the entire profile, i.e. +12 to -30 NGVD, are included in the grain size composite for the native material. Stauble, Hansen, and Blake (1984) found that the inclusion of offshore samples in the native composite tends to make the native composite finer, thereby reducing the project's overfill ratio. Subsequently, less volume is apparently needed to design the beach-to-project dimensions. Stauble, Hansen,

and Blake suggest using only the intertidal sediment samples, i.e. between mean high and low tide, to create the native beach composite.

42. The approach of Stauble, Hansen, and Blake (1984) was taken on this project since the purpose was to create a berm that could withstand extreme waves and run-up generated by hurricanes and extratropical storms. By using a native composite composed of material located in the intertidal zone, it is possible to characterize the particle sizes that compose the active wave run-up zone during extreme events. The chances of maintaining integrity of the berm are increased if these particle sizes are abundant during such an event. Therefore, only the sediment samples from +5.4, 3.4, 1.7, 0.0, -5.0 ft MLW were used to create the native beach composite.

43. Table 9 represents the composite mean, standard deviation for the seven subreaches on the native beach. Unlike the Traverse Group study, a definite spatial gradation in the mean grain size at Virginia Beach was observed during the 2 years in which sediment data were collected. A gradual increase in the composite mean diameter was identified from south to north with the standard deviation remaining fairly constant. Temporally, the March composites tended to be finer that the September composites.

Beach-Fill Design

44. Two equations are typically used to compute losses of borrow material for beach nourishment projects (SPM 1984). The fill factor (Ra) or overfill ratio estimates the volume of borrow material necessary to create one unit of stable beach material (James 1975). For example, an overfill ratio of 2.0 indicates that two units of borrow material are required to create one unit of native beach material after winnowing has occurred. A second equation, the renourishment factor (Rj), estimates the long-term stability of a particular borrow relative to the native beach material. A renourishment factor of two suggests that the borrow material is one-half as stable as native beach material or that renourishment would have to occur twice as often to maintain specific beach dimensions. This factor is often applied to select borrow sources with similar overfill ratios (Hobson 1977).

45. As previously stated, there was a gradual increase in the composite mean grain size diameter from Rudee Inlet to 61st Street. Consequently, overfill and renourishment factors computed for each subreach reflect this

gradation by increasing from south to north. Table 9 represents the overfill and renourishment factors for all subreaches in this study.

46. The overfill factors computed for each subreach could have been applied separately to compute project volume requirements. However, the area of most concern for this project was centered in the vicinity of 38th to 49th Streets where there has been historically accelerated erosion. Therefore, instead of applying overfill factors for each subreach, a more cautious approach was taken by applying the overfill ratio computed for the area of high erosion to the entire project. In cooperation with CENAO, an overfill ratio of 1.35 was selected for the entire project to compensate for an expected high loss of finer, unstable material during the initial profile adjustment period.

47. Computed renourishment factors for the seven subreaches ranged from 0.63 to 1.69 (Table 9). These values should be used with caution since the renourishment equation has not been fully verified. Assessment of the renourishment factors suggests borrow sites #2, #3, and #6 may be more suitable borrow sources as their renourishment factors were slightly lower compared with borrow site #7. An economic evaluation based upon overfill ratios, depth, distance to beach, and available volume should be performed prior to selecting the final borrow source(s) for this project.

Design volume requirements

48. Due to the physical location and distance of Thimble Shoals borrow area from Virginia Beach, it is most likely that borrow material will be excavated using a hopper dredge and then pumped out onto the beach via an offshore pumpout station. A certain percentage of borrow material is lost in this type operation because of multiple handling (transfers) of the material. To account for this anticipated loss, a handling loss factor (typically 10 to 25 percent) is added to the overfill ratio (Hobson and James 1979). In a cooperative decision with CENAO, a handling loss of 15 percent was applied to the project overfill ratio of 1.35. This brings the final overage factor for the hurricane protection project to 1.55. Multiplying this factor times the volume required to build the beach to design dimensions of 1,228,000 yd^3 yields 1,904,000 yd^3 as the final volume of borrow material required to obtain the specified design. The design construction profile extends 205 ft from the construction control line at an elevation of 6.4 ft NGVD (8.0 MLW) and then tapers to existing bottom with a 1:20 slope (Appendix D).

Advance nourishment estimates

49. At the request of CENAO, advanced nourishment volume quantities were estimated. Fortunately, many data have been collected at Virginia Beach concerning historic erosion rates and subsequent annual nourishment quantities. An estimated 220,000 yd^3 of material is annually bypassed from Rudee Inlet and placed south of 18th Street. North of 18th Street, approximately 150,000 yd^3 of material is annually truck hauled and deposited on the beach. This combined quantity of 370,000 yd^3/year tends to stabilize a majority of beaches for a period of approximately 1 year. The exception is an area in the vicinity of 38th Street that tends to historically erode at a greater rate than the adjacent areas.

50. For these calculations, it was assumed the design volume of 1,904,000 yd^3 would provide an adjusted, uniform beach width of 100-ft berm from the baseline after establishment of a new equilibrium profile. A 2-year advance nourishment will require an additional 740,000 yd^3 (370,000 yd^3 × 2 years) times an overfill ratio of 1.55 for a final 2-year advance nourishment quantity of 1,145,000 yd^3 (Table 10). A total of 3,049,000 yd^3 (1,904,000 + 1,145,000 yd^3) of material would be required for the entire project if the 2-year advance nourishment is performed at the time of initial construction. The construction profile cross sections for the design and 2- and 5-year advance nourishment quantities are represented in Appendix E. The 2-year advance nourishment construction profile extends 290 ft from the construction control line and then tapers to existing bottom with a 1:20 slope. An overfill ratio was applied to the advance nourishment quantity since it was assumed that losses similar to the design volume would occur. The 2-year advance nourishment will provide approximately an additional 32 yd^3/linear ft of beach after winnowing of fines and handling losses. This volume of material can be roughly translated into 32 ft of addition berm width using the rule of thumb where 1 yd^3 equals 1 ft of beach width.

51. Similar calculations were performed for a 5-year advance nourishment. A 5-year advance nourishment would require approximately 1,850,000 yd^3 (370,000 yd^3 × 5 years) times an overfill ratio of 1.55 for a 5-year advance nourishment quantity of 2,867,000 yd^3 (Table 11) (Appendix E). A total of 4,777,000 yd^3 of (1,904,000 + 2,867,000 yd^3) material would be required for the entire project if the 5-year advance nourishment is performed at the time of initial construction. This volume of material can be translated into an

Table 10

Two-Year Advance Nourishment Quantity Estimates for Native Beach Subreach

Subreach	Length ft	yd^3/ft	yd^3	Ra.* yd^3	Design & Advance yd^3
5th	2,500	32	80,000	124,000	155,000
11th	5,280	32	169,000	261,900	720,200
30th	5,300	32	169,600	262,900	731,100
38th	3,625	32	116,000	179,800	634,900
49th	1,975	32	63,200	98,000	272,500
55th	1,850	32	59,200	91,800	249,500
61th	2,560	32	82,000	127,000	285,700
Total	23,090		739,000	1,145,000	3,048,900

* Overfill ratio of 1.55 multiplied by cubic yards for each subreach.

Table 11

Five-Year Advance Nourishment Quantity Estimates for Native Beach Subreach

Subreach	Length ft	yd^3/ft	yd^3	Ra.* yd^3	Design & Advance yd^3
5th	2,500	80.1	200,300	310,400	341,400
11th	5,280	80.1	423,000	655,500	1,113,800
30th	5,300	80.1	424,600	658,000	1,126,200
38th	3,625	80.1	290,400	450,000	905,100
49th	1,975	80.1	158,200	245,200	419,700
55th	1,850	80.1	148,200	229,700	387,400
61th	2,560	80.1	205,100	317,800	476,500
Total	23,090		1,849,800	2,866,600	4,770,100

* Overfill ratio of 1.55 multiplied by cubic yards for each subreach.

additional 80 ft of berm width provided by this quantity. The 5-year advance nourishment construction profile extends 410 ft from the construction control line and then tapers to existing bottom with a 1:20 slope.

PART IV: SUMMARY AND CONCLUSIONS

52. Model simulations show that the proposed 100-ft berm of the design dune/beach profile for the no-seawall case is adequate for protection from either a single or multiple design storm event. Additional model results indicate that a minimum width of 50.0 ft would provide protection against breaching by the design storm although erosion of the crest of the dune would occur. This analysis assumes that the design berm configuration is maintained at all times and that the 1-D assumption is not severely violated. If the verification results reported by Birkemeier et al. (1987) are taken into consideration, a factor of safety of approximately 2.0 should be applied to the present analysis. This figure results from the fact that erosion in 5 of the 13 cases was underestimated by 55 percent or that the observed erosion volume was 1.82 times the predicted. A factor of safety of 2.0 was also recommended by Kraus et al. (in preparation) in a dune erosion model application to the shoreline of New Jersey. In view of these results and the fact that widespread damage may occur if the dune is breached, the 100-ft berm width proposed by CENAO is recommended. According to the stage-frequency curve computed by CENAO, this would provide protection for the 100-year storm (i.e. the August 1933 storm) and provide a reasonable margin of safety.

53. The proposed dune/beach profile with a 100-ft berm width for the design case with a seawall also provides adequate protection against the design storm. For the seawalled case, the berm will cease receding when recession reaches the seawall. At this point, scour at the face of the seawall will begin. Additional simulations were performed in which the berm width was reduced to 50.0 ft in order to evaluate the potential erosion in front of the seawall. Results of the multiple design storm simulation showed that the berm elevation was lowered over its entire width. However, the elevation loss at the seawall was not sufficient to endanger seawall integrity and was comparable to the value of approximately 2 ft of erosion used in computing overtopping quantities in the physical model study (Lillycrop et al. 1988). In view of this result and the variability factor, the 100-ft width at elevation +5.4 NGVD design is recommended as an effective measure of protection.

54. The profile translation method was used to determine the quantity of material needed to build the beach, in its present condition, to a beach possessing a berm width of 100 ft after the establishment of a new equilibrium

profile. The CERC was provided with four sets of profile data spanning a period of 2 years. The March 1987 profile data set was selected to calculate the design volume for the project. This data set provided the most conservative (greatest) volumetric results. Seven unique subreaches of the project beach were identified and represented by seven profile lines. Total volume necessary to construct the design berm without advance nourishment was calculated to be 1,227,000 yd^3.

55. Composite grain size analysis was conducted for seven borrow areas within Thimble Shoals, and four areas were selected for further investigation. In a similar manner, composite grain size analysis were computed for the same seven native beach subreaches used in the volumetric assessment. Overfill ratios and renourishment factors were calculated using all native beach and borrow area composites. Overfill ratios ranged from 1.07 to 1.88 for the various scenarios of native and borrow material. Since the area of accelerated erosion is in the vicinity of 38th Street, an overfill ratio of 1.35 was selected by CERC and CENAO to be applied for the entire project. Because of the type of dredging operation anticipated for Thimble Shoals, a handling loss of 15 percent was applied to the overfill ratio yielding a final overage factor of 1.55 for the entire project. Multiplying the final overage factor (1.55) times the volume of material required to build the beach to design dimensions (1,227,000 yd^3) yields 1,902,000 yd^3 of material to build the project to specifications. Evaluating the renourishment factors suggests borrow areas #2, #3, and #6 are probably the most suitable; however, further economic evaluation should be performed before making the final borrow site selection.

56. Currently, an estimated 370,000 yd^3/year of material is artificially placed in the project area on a yearly basis to stabilize the beach width. Based upon current stabilization practices, 2- and 5-year advance nourishment quantities were estimated. A 2- and 5-year advance nourishment would require an additional 1,147,000 and 2,867,500 yd^3 of material, respectively, to the design volume of 1,902,000 yd^3 giving a total of 3,049,000 and 4,769,500 yd^3, respectively. These quantities would theoretically stabilize the beach at 100 ft from the construction control line for the respective time interval. An overfill ratio of 1.55 is included in the advance nourishment.

REFERENCES

Bascom, W. H. 1951. "The Relationship Between Sand Size and Beach Face Slope," Transactions, American Geophysical Union, Vol 32, No. 6, pp 866-874.

Birkemeier, W. A. 1984. "The Interactive Survey Reduction Program: User's Manual to ISRP-PC," Instruction Report CERC-84-1, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Birkemeier, W. A., Kraus, N. C., Scheffner, N. W., and Knowles, S. C. 1987. "Feasibility Study of Quantitative Erosion Models for Use by FEMA in the Prediction of Coastal Flooding," Technical Report CERC-87-8, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Bruun, P. 1954. "Coast Erosion and the Development of Beach Profiles," TM-44, US Army Corps of Engineers, Beach Erosion Board, Fort Belvoir, VA.

Chiu, T. Y., and Dean, R. G. 1986. "Additional Comparison Between Computed and Measured Erosion by Hurricanes," Beaches and Shores Technical and Design Memorandum, Beaches and Shores Recourse Center, Institute of Science and Public Affairs, Florida State University, Gainesville, FL.

Dean R. G. 1977. "Equilibrium Beach Profiles: US Atlantic and Gulf Coasts," Ocean Engineering Report No. 12, Department of Civil Engineering, University of Delaware, Newark, DE.

Delft Hydraulics Laboratory. 1987. Manual on Artificial Beach Nourishment, Rijkswaterstaat.

Hallermeier, R. J. 1981. "Seaward Limit of Significant Sand Transport by Waves: An Annual Zonation for Seasonal Profiles," Coastal Engineering Technical Aid No. 81-2, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Hansen, M., and Lillycrop, W. J. 1988. "Evaluation of Closure Depth and Its Role in Estimating Beach Fill Volumes," Proceedings: Specialty Conference in Beach Nourishment, Florida Shore and Beach Preservation Society.

Heimbaugh, M. S., Grace, P. J., Ahrens, J. P., and Davidson, D. D. 1988. "Coastal Engineering Studies: Virginia Beach, Virginia, Beach Erosion Control and Hurricane Protection; Report 1, Physical Model Tests of Irregular Wave Overtopping and Pressure Measurements," Technical Report CERC-88-1, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Hobson, R. D. 1977. "Review of Design Elements for Beach-Fill Evaluation," Technical Report 77-6, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Hobson, R. D., and James, W. R. 1979 "Importance of Handling Losses to Beach Fill Design," Technical Report CERC-79-9, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Hughes, S. A. 1978. "The Variation in Beach Profiles When Approximated by a Theoretical Curve," M.S. Thesis, Coastal and Oceanographic Engineering Department, University of Florida, Gainesville, FL.

James, W. R. 1975. "Techniques in Evaluating Suitability of Borrow Material of Beach Nourishment," Technical Memorandum No. 60, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Jensen, R. E. 1983. "Atlantic Coast Hindcast, Shallow-Water Significant Wave Information," WIS Report 9, US Army Engineer Waterway Experiment Station, Vicksburg, MS.

Kraus, N. C., Scheffner, N. W., Gravens, M. B., and Cialone, M. "Coastal Processes at Sea Bright to Monmouth Beach, New Jersey," Technical Report in preparation, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Kriebel, D. L. 1984a. "Beach Erosion Model (EBEACH) User's Manual, Volume I: Description of Computer Model," Beaches and Shores Technical and Design Memorandum No. 84-5-I, Division of Beaches and Shores, Florida Department of Natural Resources, Tallahassee, FL.

__________. 1984b. "Beach Erosion Model (EBEACH) User's Manual, Volume II: Theory and Background," Beaches and Shores Technical and Design Memorandum No.84-5-II, Division of Beaches and Shores, Florida Department of Natural Resources, Tallahassee, FL.

Lillycrop, W. J., Pope, J., and Abel, C. E. 1988. "Coastal Engineering Studies; Virginia Beach, Virginia, Beach Erosion Control and Hurricane Protection, Report 2, Seawall Overtopping Evaluation," Technical Report CERC-88-1, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Moore, B. 1982. "Beach Profile Evolution in Response to Changes in Water Level and Wave Height," M.S. Thesis, Department of Civil Engineering, University of Delaware, Newark, DE.

Shepard, F. P. 1963. Submarine Geology. Harper and Row, New York.

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

Stauble, D. K., Hansen, M., and Blake, W. 1984. "An Assessment of Beach Nourishment Sediment Characteristics," Proceedings: Coastal Engineering 1984. American Society of Civil Engineers, New York.

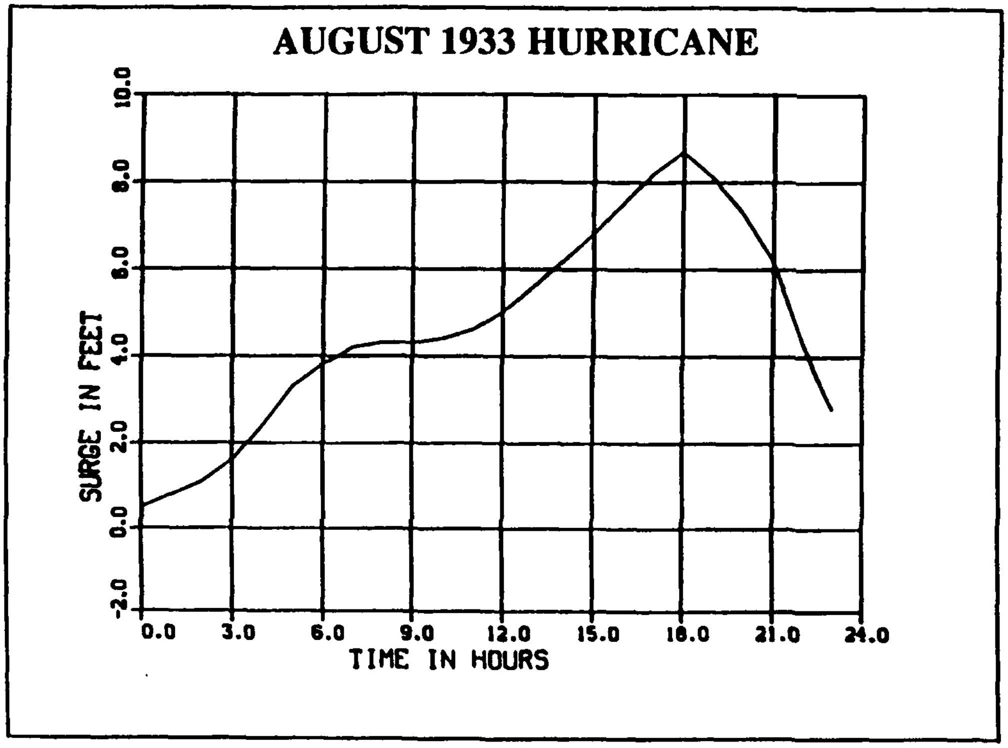
Traverse Group Inc. 1980. "Beach Erosion Control and Hurricane Protection at Virginia Beach, VA: Coastal Processes Evaluation," Ann Arbor, MI.

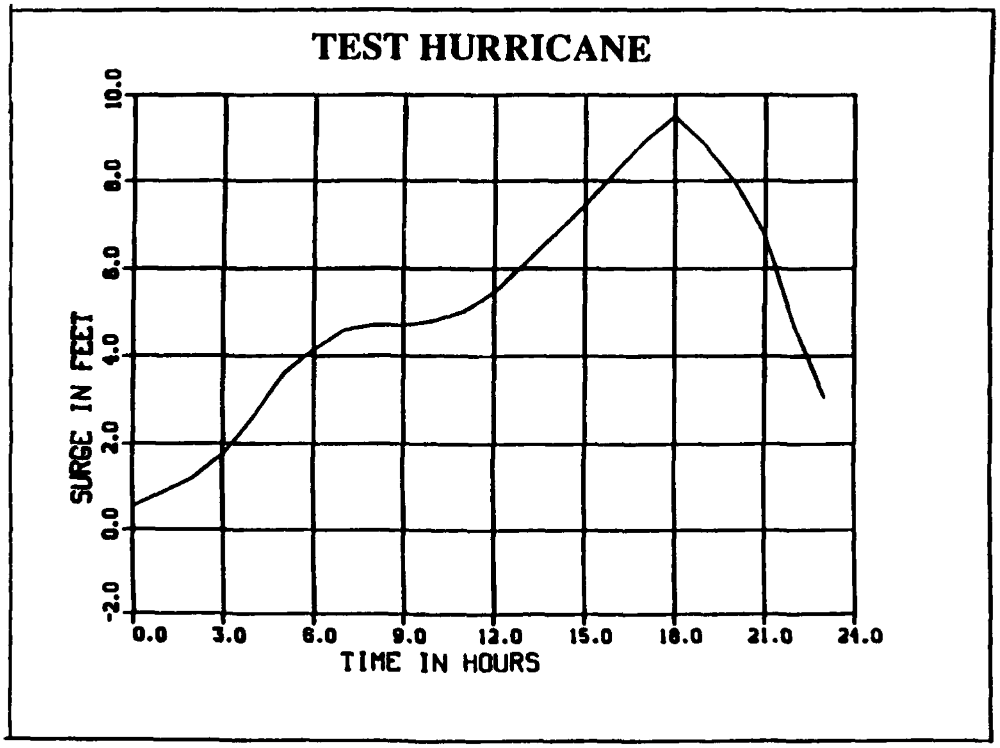
US Army Engineer District, Norfolk. 1983 (Nov). (Revised Jun 1984). "Main Report and Supporting Documentation, Phase 1-GDM and Supplemental EIS, Beach Erosion Control and Hurricane Protection," Norfolk, VA.

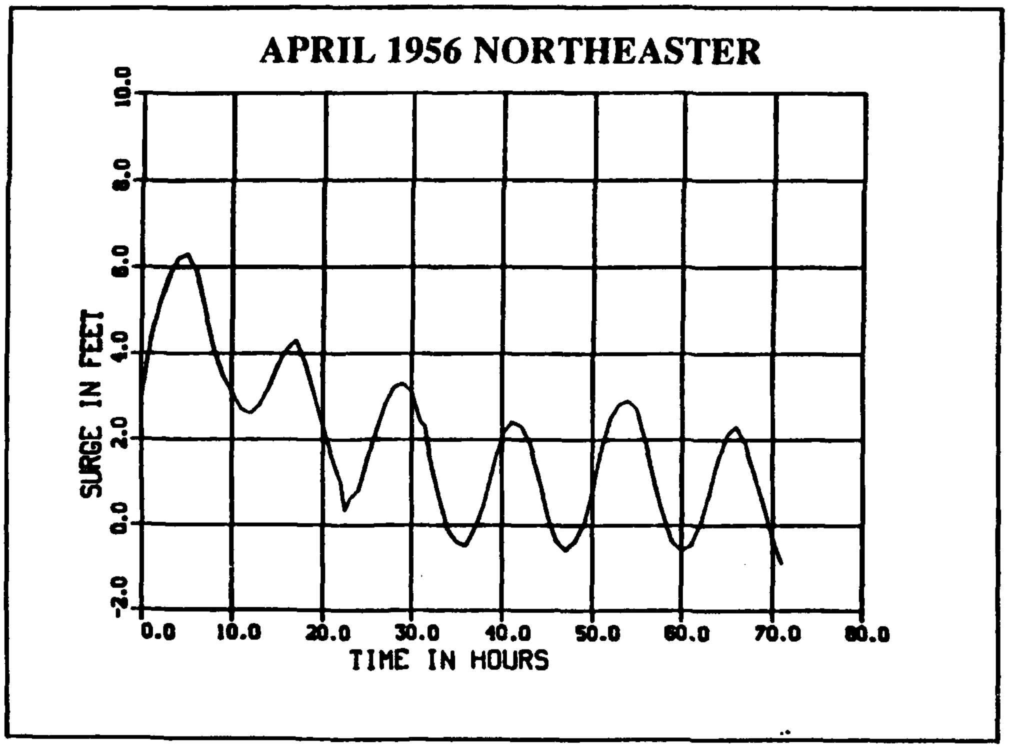
Waterways Surveys & Engineering Ltd. 1984. "Feasibility Study for Disposal of Dredged Material from Norfolk Harbor Channel Deepening on Fort Story Beaches at Cape Henry," Virginia Beach, VA.

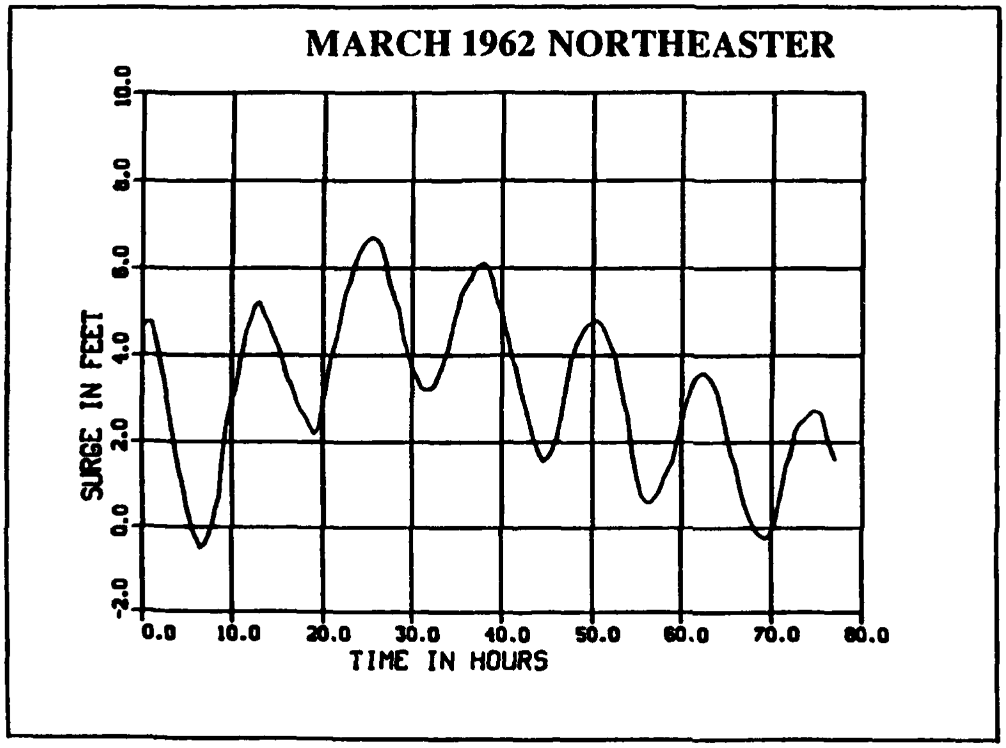
APPENDIX A: STORM SURGE CURVES

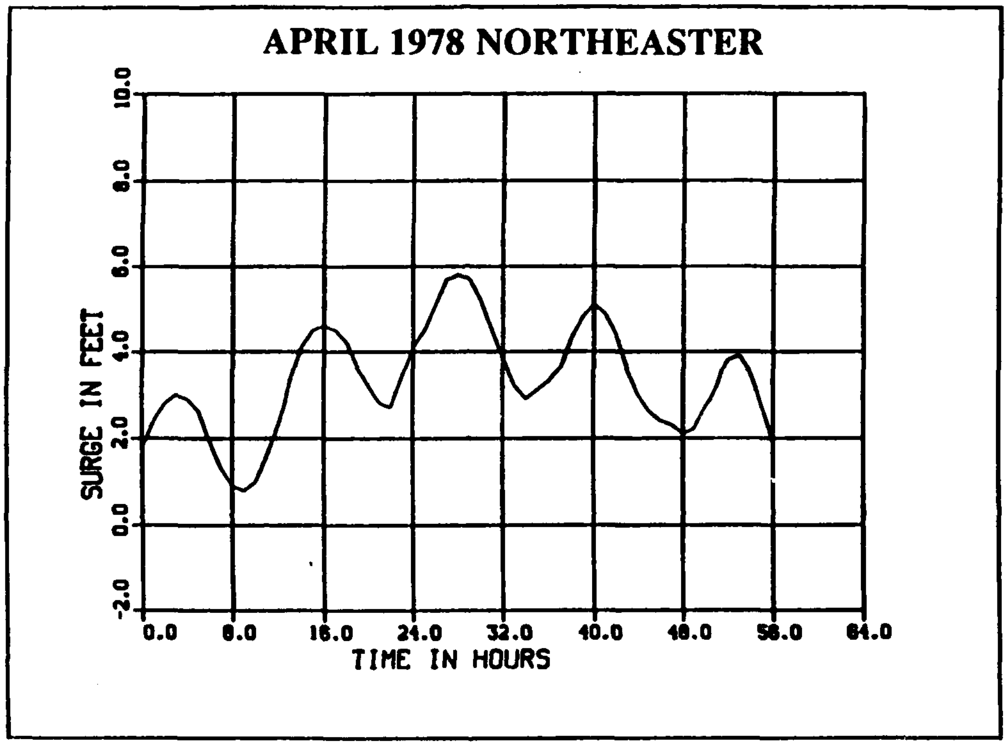
(Storm surge curves used in the numerical modeling test represent three northeasters and two hurricanes.)


AUGUST 1933 HURRICANE
SURGE IN FEET
TIME IN HOURS
10.0
8.0
6.0
4.0
2.0
0.0
-2.0
0.0
3.0
6.0
9.0
12.0
15.0
18.0
21.0
24.0


TEST HURRICANE
SURGE IN FEET
TIME IN HOURS
10.0
8.0
6.0
4.0
2.0
0.0
-2.0
0.0
3.0
6.0
9.0
12.0
15.0
18.0
21.0
24.0

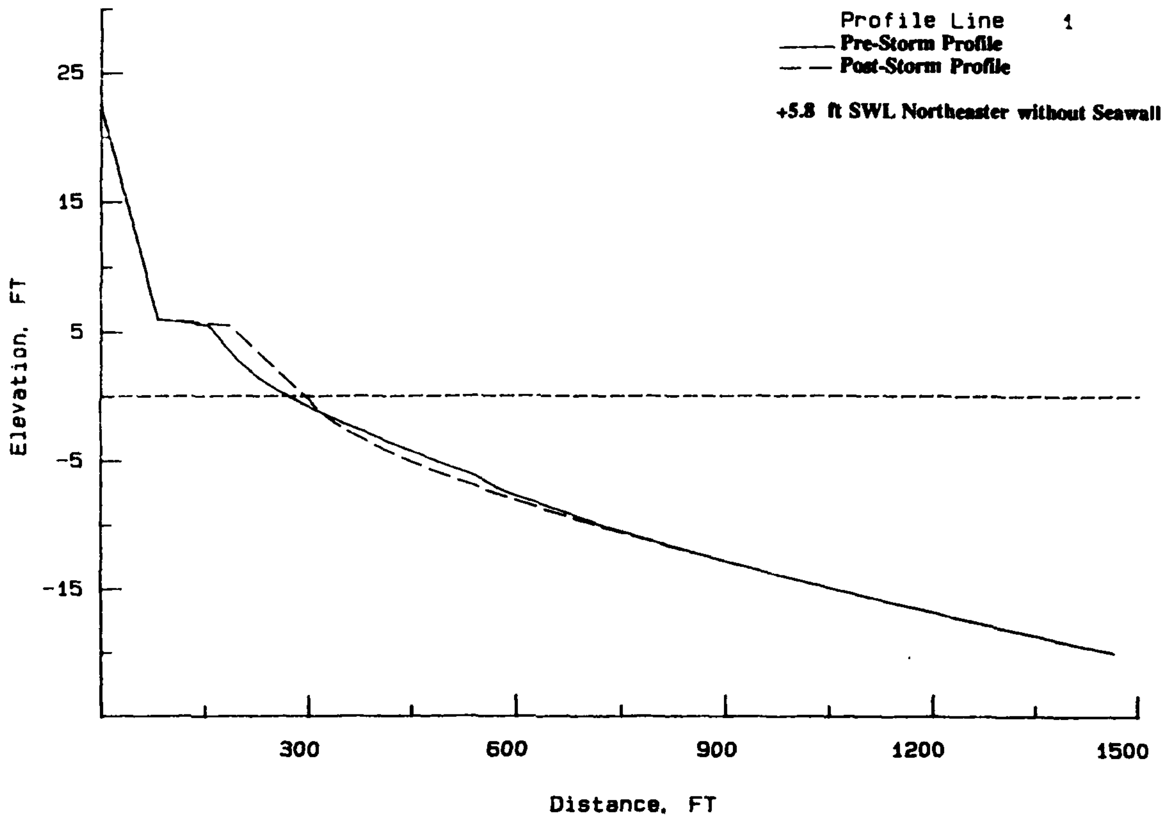

APRIL 1956 NORTHEASTER
SURGE IN FEET
-2.0
0.0
2.0
4.0
6.0
8.0
10.0
0.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
80.0
TIME IN HOURS


MARCH 1962 NORTHEASTER
SURGE IN FEET
-2.0
0.0
2.0
4.0
6.0
8.0
10.0
0.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
80.0
TIME IN HOURS

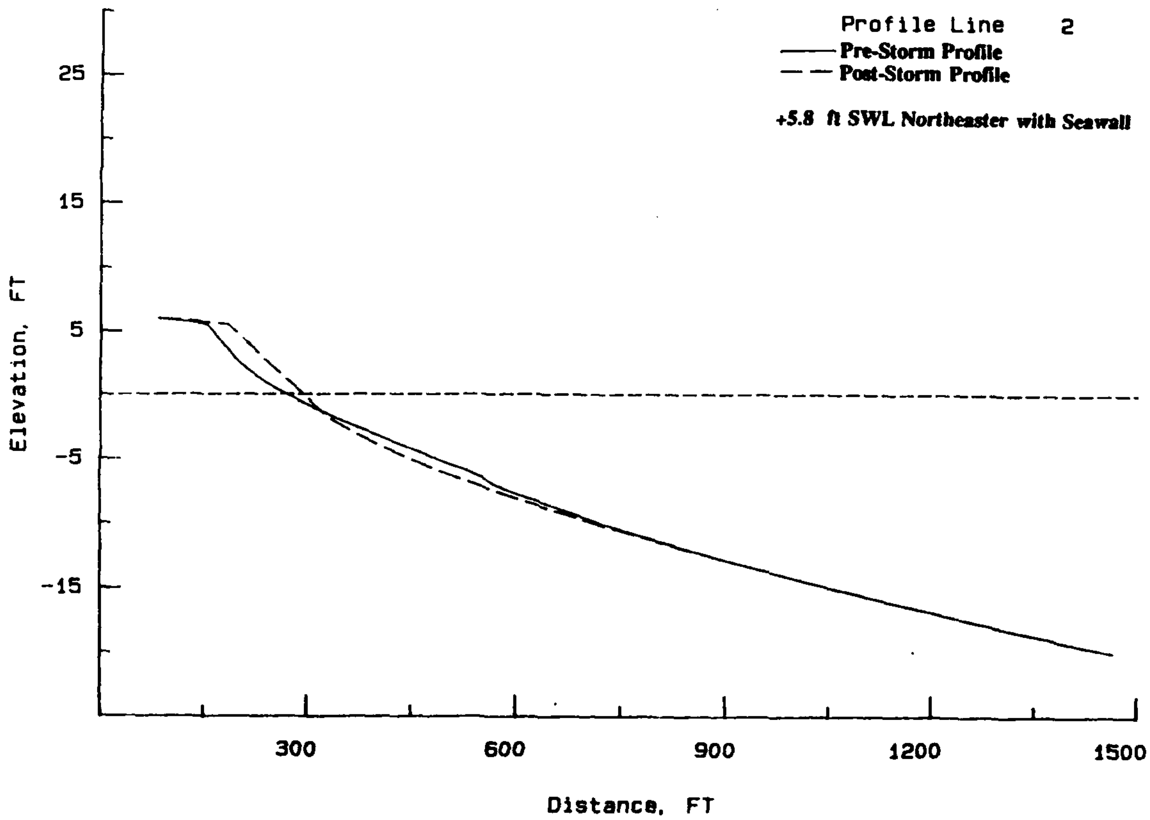

APRIL 1978 NORTHEASTER
SURGE IN FEET
10.0
8.0
6.0
4.0
2.0
0.0
-2.0
0.0
8.0
16.0
24.0
32.0
40.0
48.0
56.0
64.0
TIME IN HOURS

APPENDIX B: PRESTORM AND POSTSTORM PROFILES

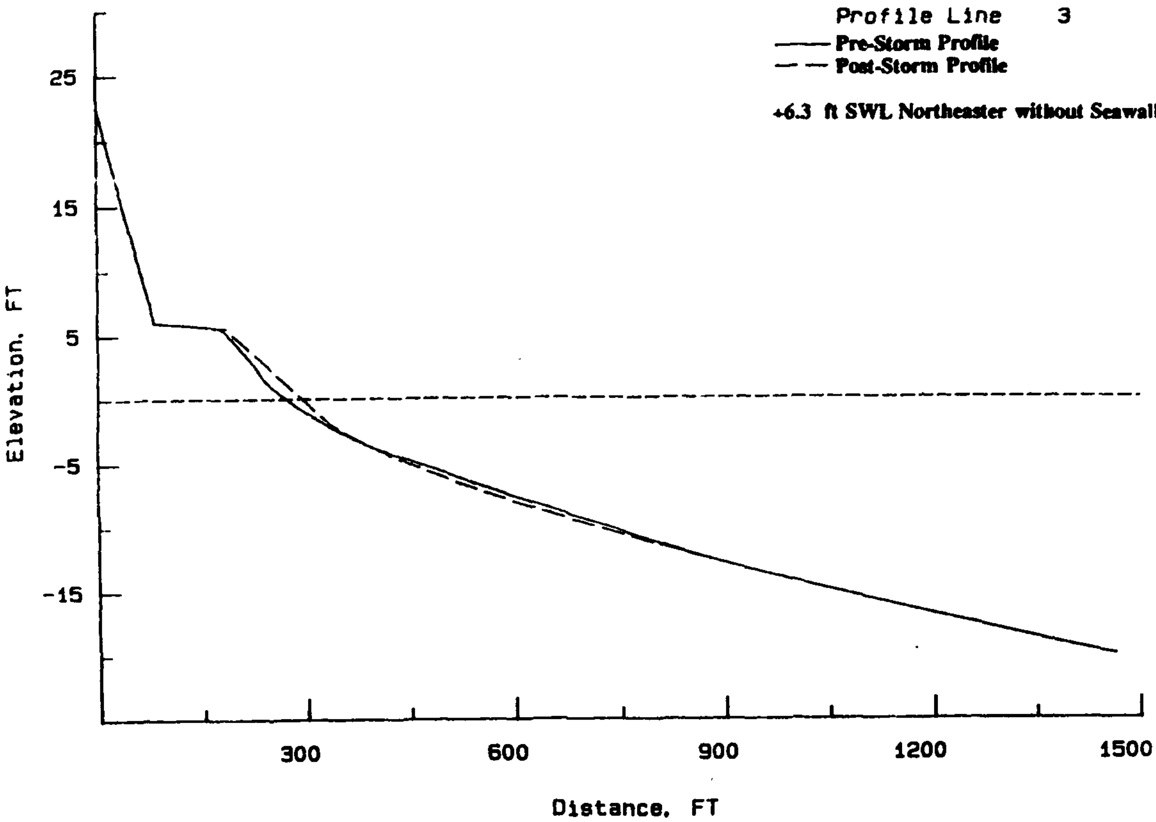
(Prestorm and poststorm profiles are presented with and without a protective seawall. Elevations are relative to National Geodetic Vertical Datum.)

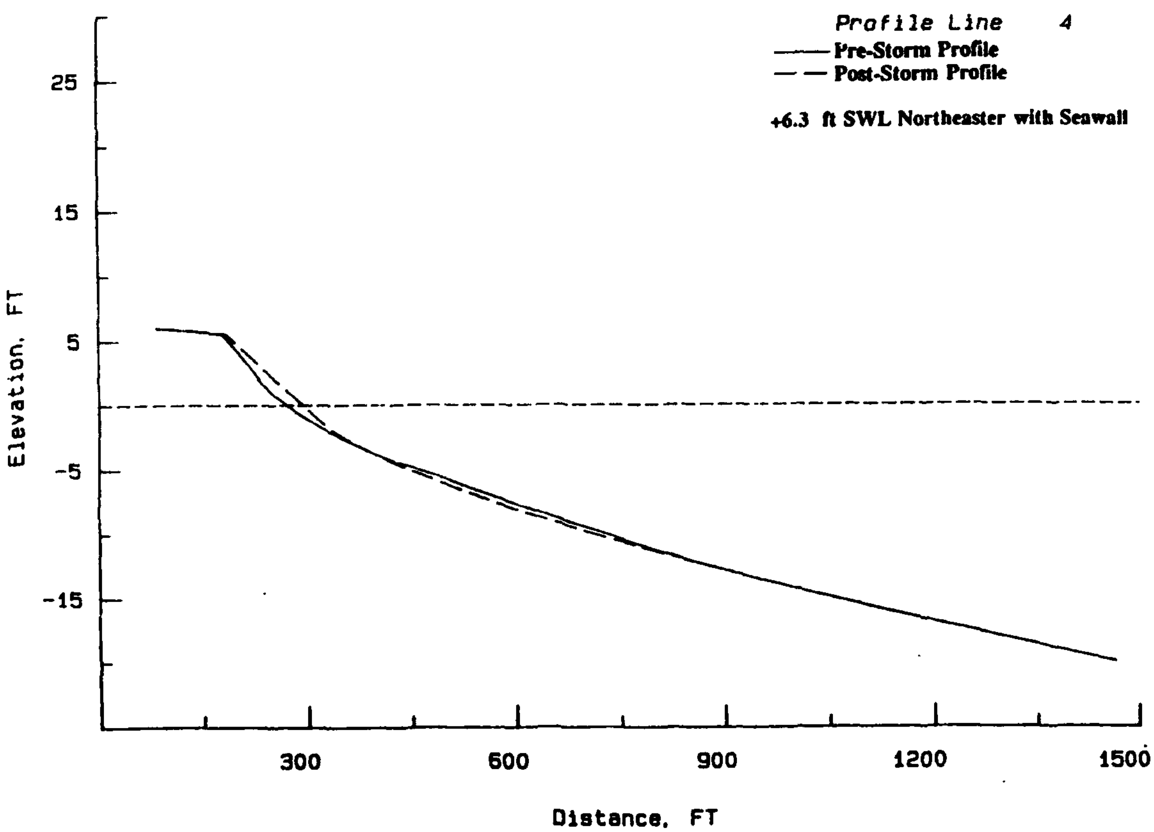


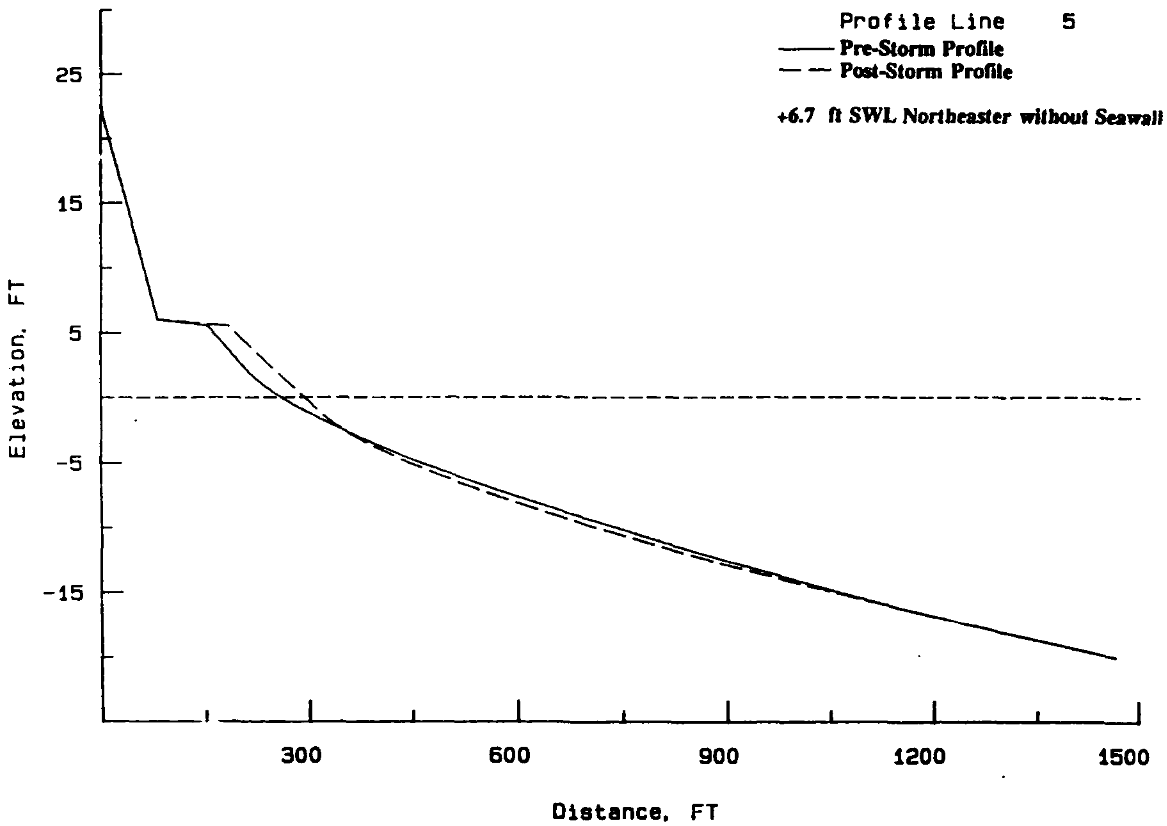
Profile Line 1
Pre-Storm Profile
Post-Storm Profile
+5.8 ft SWL Northeaster without Seawall
25
15
5
-5
-15
Elevation, FT
300
600
900
1200
1500
Distance, FT

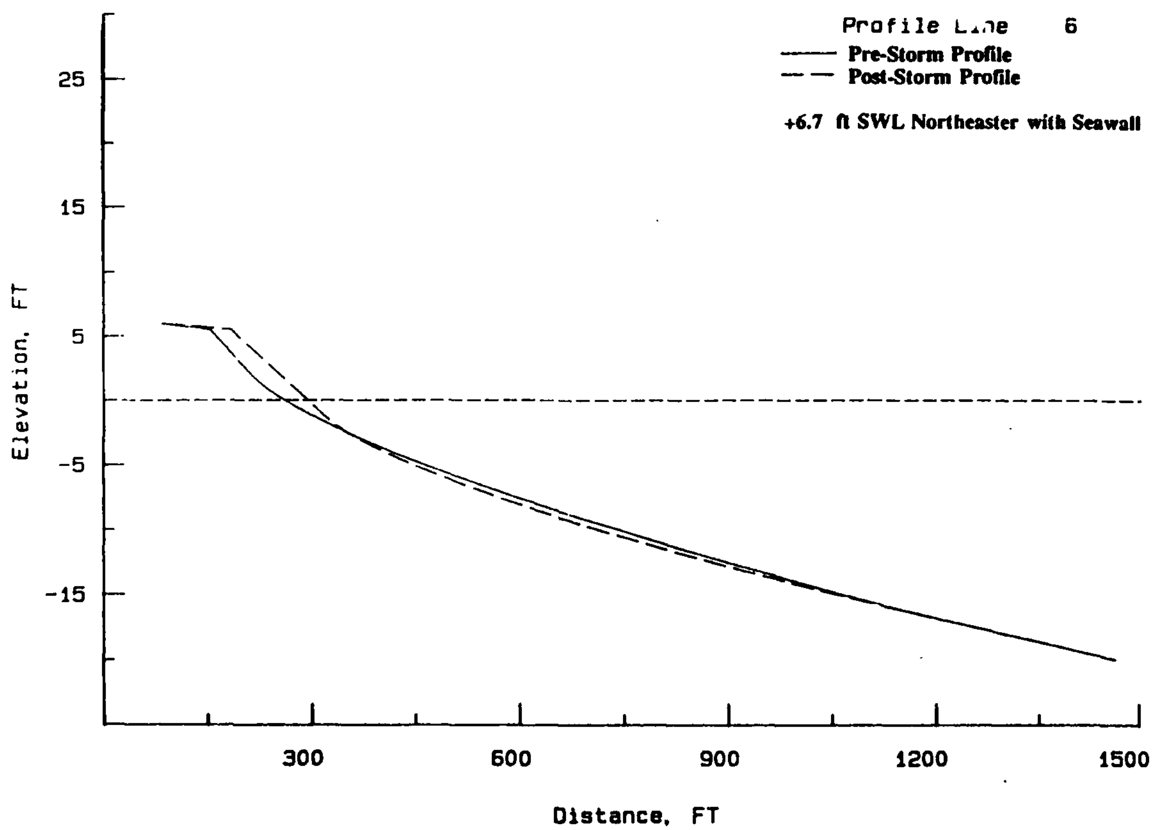


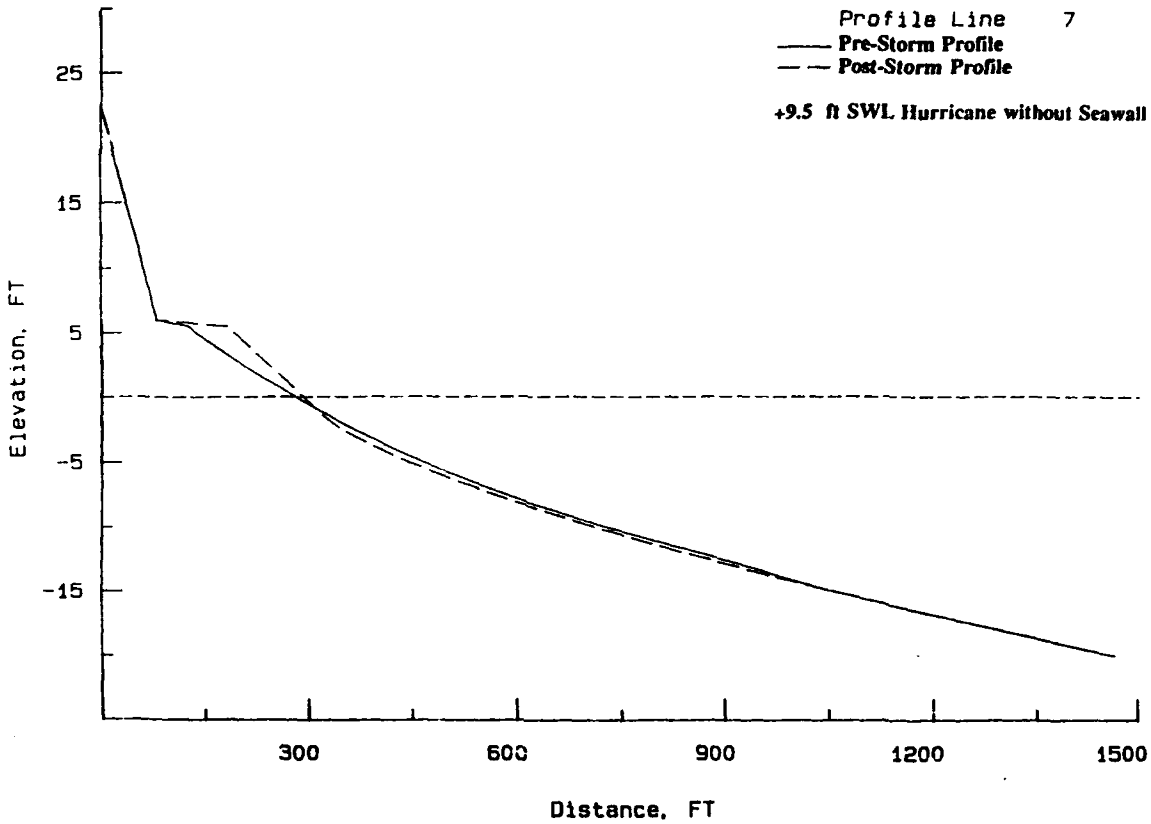
Profile Line 2
Pre-Storm Profile
Post-Storm Profile
+5.8 ft SWL Northeaster with Seawall
25
15
5
-5
-15
Elevation, FT
300
600
900
1200
1500
Distance, FT

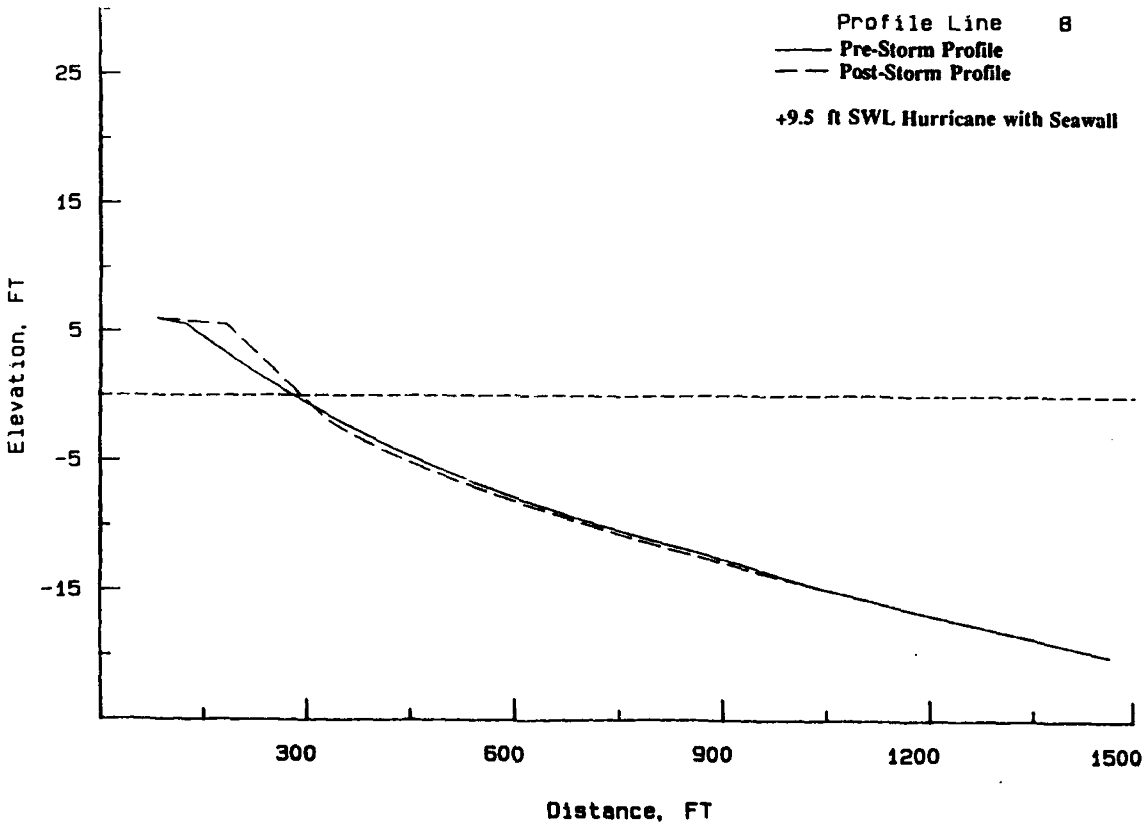

Profile Line 3
Pre-Storm Profile
Post-Storm Profile
+6.3 ft SWL Northeaster without Seawall
25
15
5
-5
-15
Elevation, FT
300
600
900
1200
1500
Distance, FT

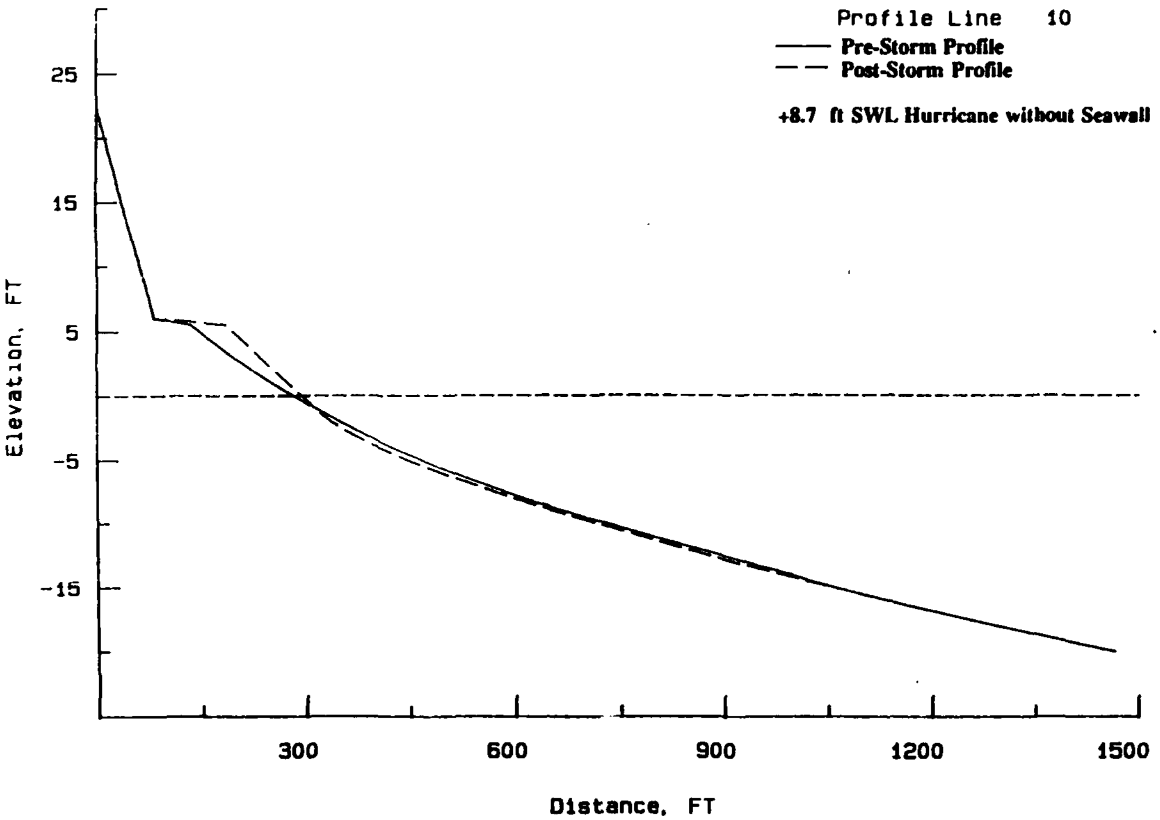

Profile Line 4
Pre-Storm Profile
Post-Storm Profile
+6.3 ft SWL Northeaster with Seawall
25
15
5
-5
-15
Elevation, FT
300
600
900
1200
1500
Distance, FT

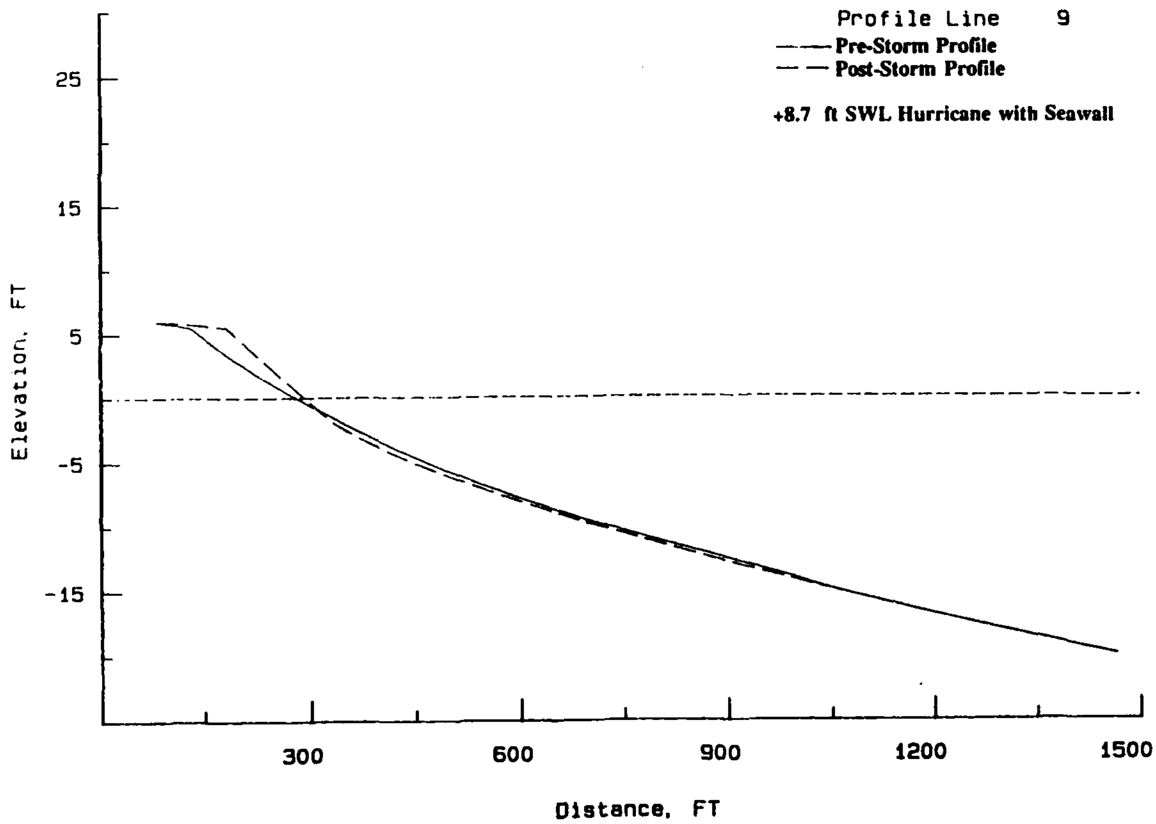

Profile Line 5
Pre-Storm Profile
Post-Storm Profile
+6.7 ft SWL Northeaster without Seawall
Elevation, FT
25
15
5
-5
-15
300
600
900
1200
1500
Distance, FT


Profile Line 6
Pre-Storm Profile
Post-Storm Profile
+6.7 ft SWL Northeaster with Seawall
Elevation, FT
25
15
5
-5
-15
300
600
900
1200
1500
Distance, FT


Profile Line 7
Pre-Storm Profile
Post-Storm Profile
+9.5 ft SWL Hurricane without Seawall
25
15
5
-5
-15
Elevation, FT
300
600
900
1200
1500
Distance, FT

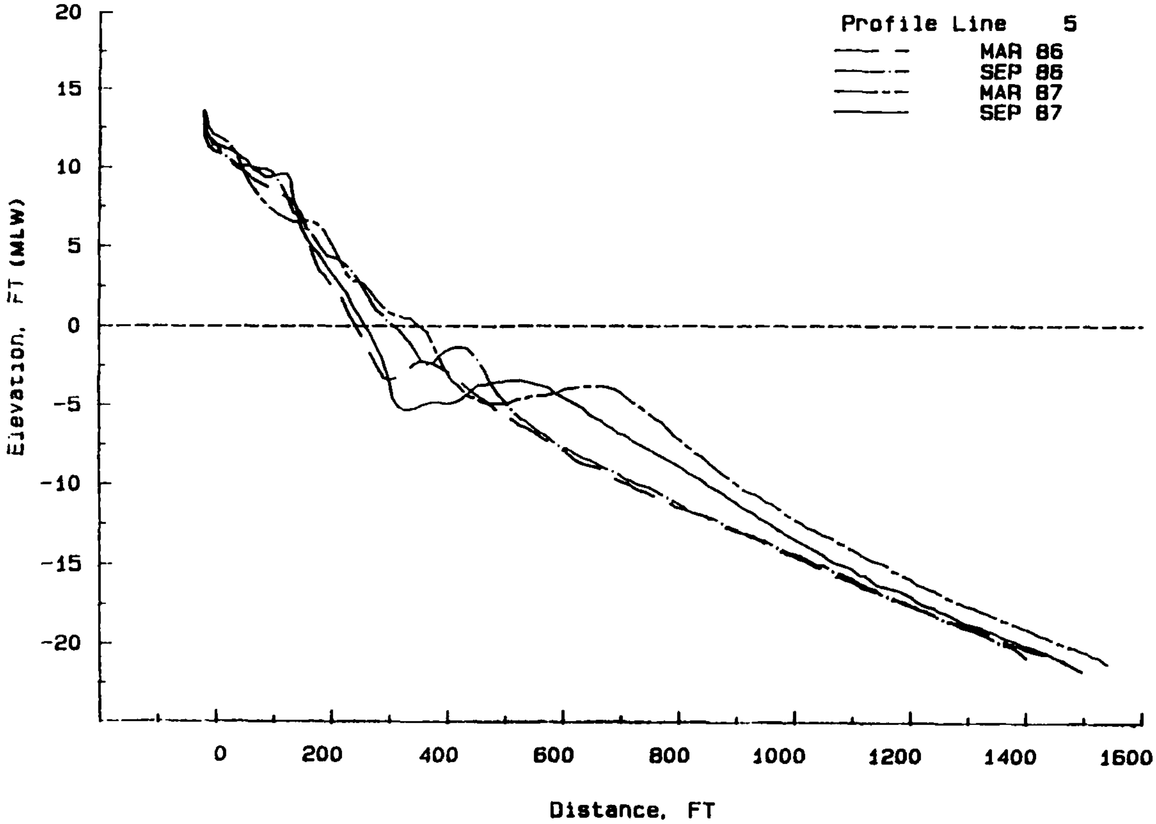

Profile Line 8
Pre-Storm Profile
Post-Storm Profile
+9.5 ft SWL Hurricane with Seawall
25
15
5
-5
-15
Elevation, FT
300
600
900
1200
1500
Distance, FT

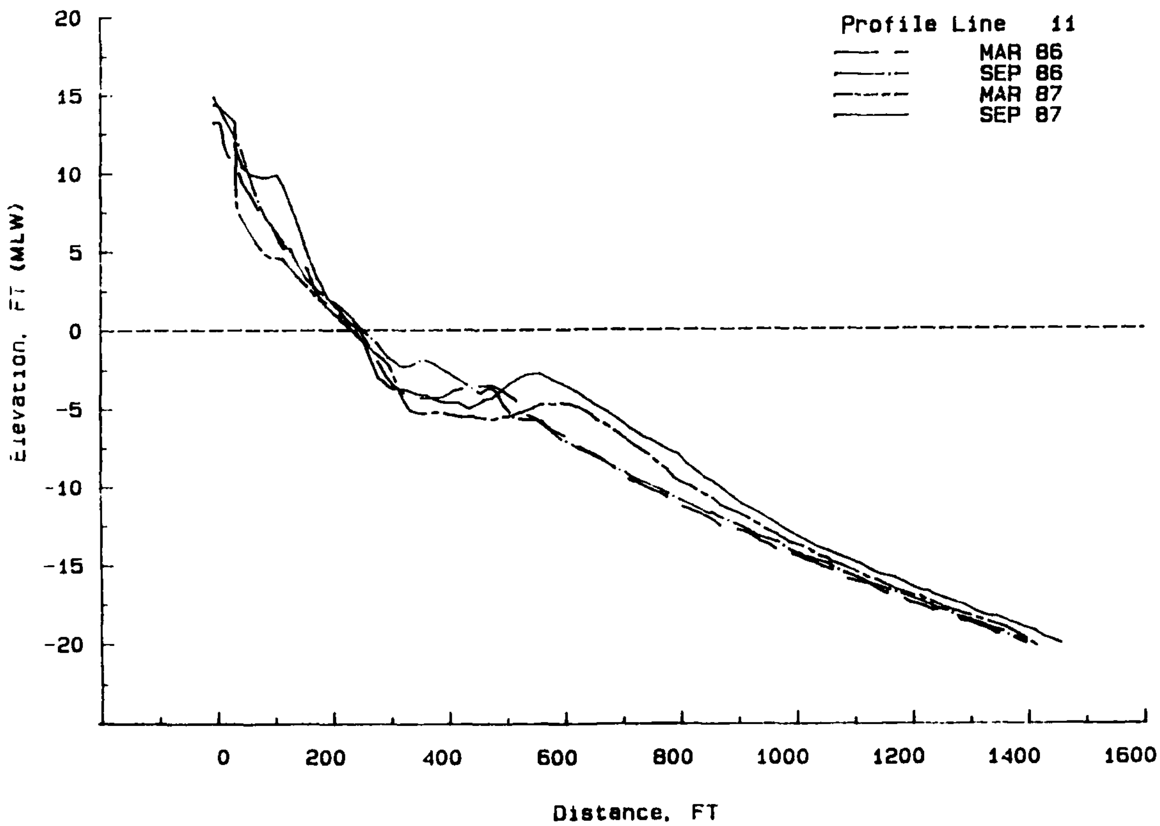

Profile Line 10
Pre-Storm Profile
Post-Storm Profile
+8.7 ft SWL Hurricane without Seawall
Elevation, FT
25
15
5
-5
-15
300
600
900
1200
1500
Distance, FT

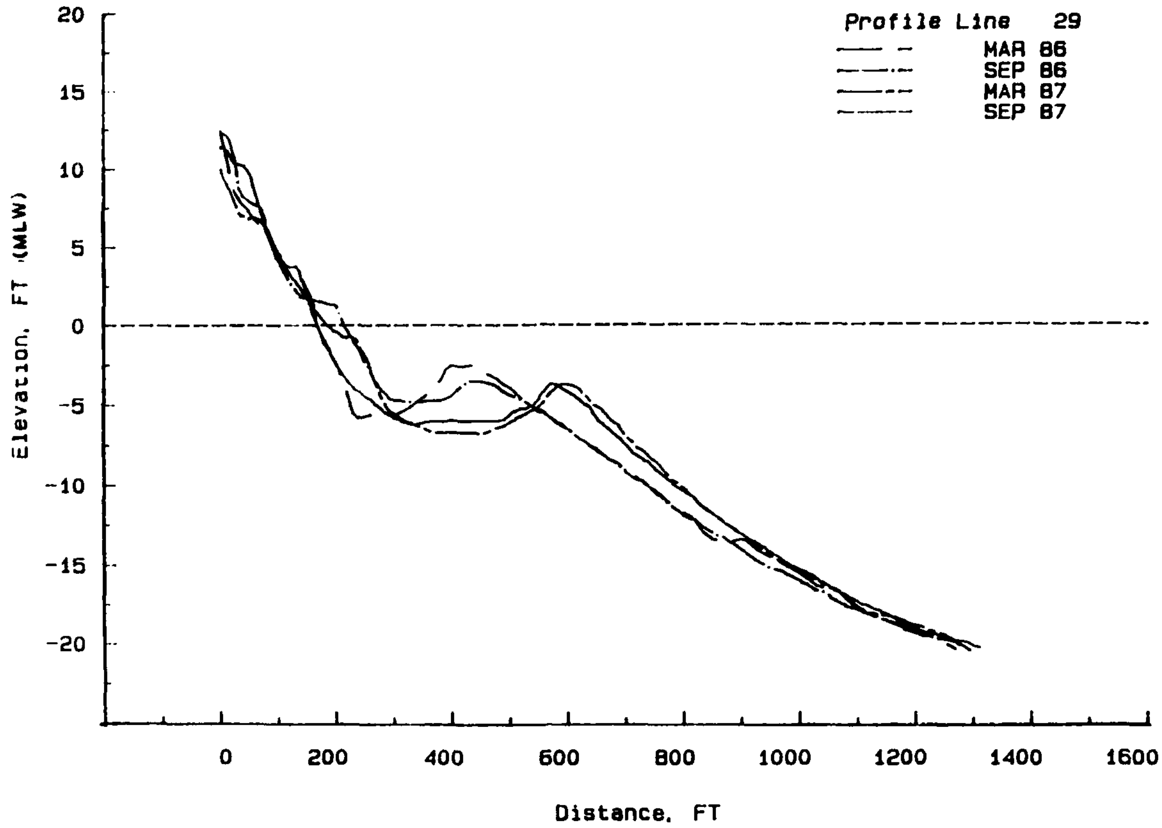

Profile Line 9
Pre-Storm Profile
Post-Storm Profile
+8.7 ft SWL Hurricane with Seawall
Elevation, FT
25
15
5
-5
-15
300
600
900
1200
1500
Distance, FT

APPENDIX C: REPRESENTATIVE PROFILES FOR SUBREACHES

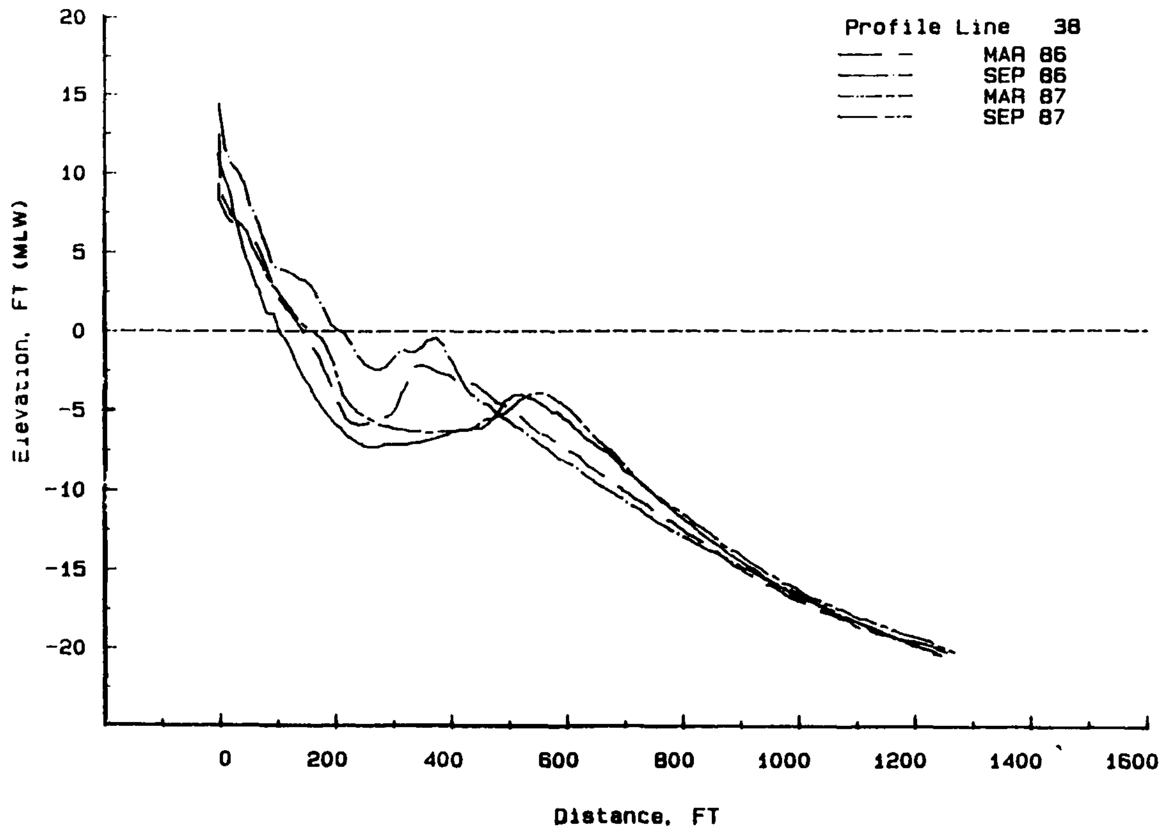
(Representative profiles for the seven native beach subreaches are presented. Horizontal distance is relative to the construction control line. Elevations are relative to mean low water (MLW).)


Profile Line 5
MAR 86
SEP 86
MAR 87
SEP 87
20
15
10
5
0
-5
-10
-15
-20
Elevation, FT (MLW)
0
200
400
600
800
1000
1200
1400
1600
Distance, FT

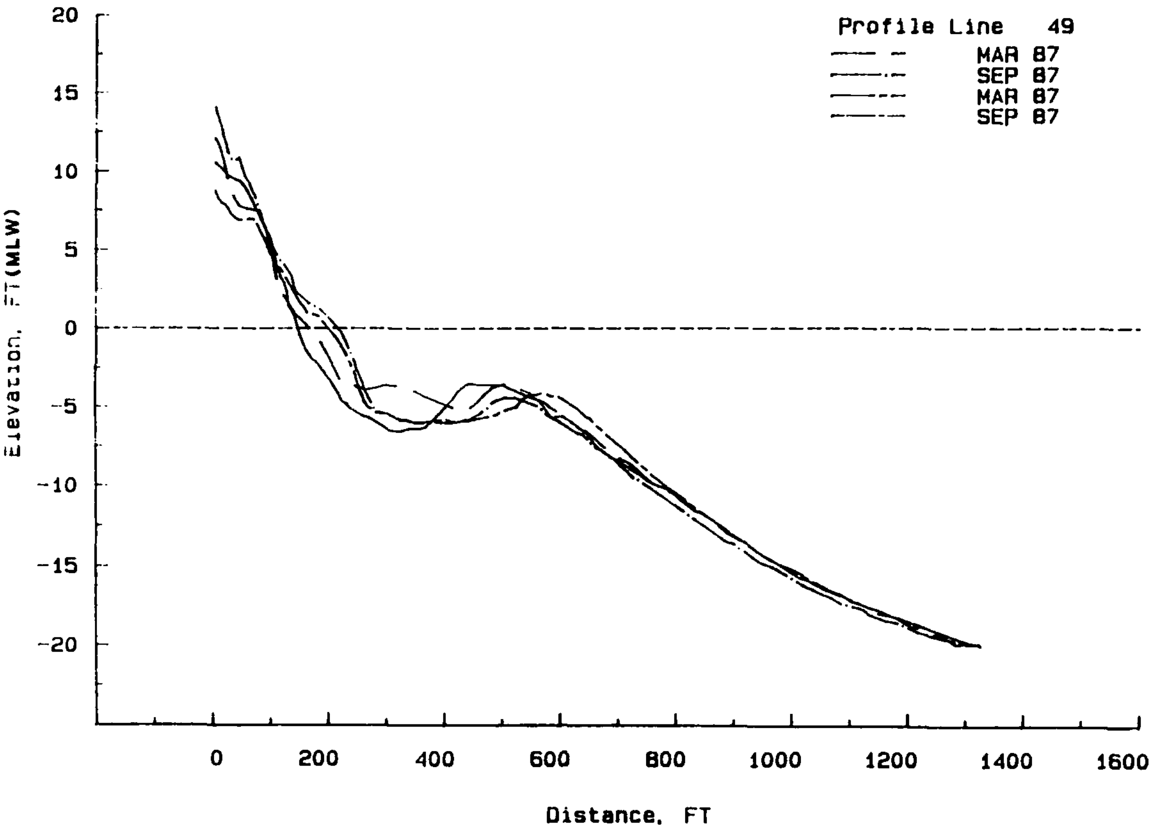

Profile Line 11
MAR 86
SEP 86
MAR 87
SEP 87
20
15
10
5
0
-5
-10
-15
-20
Elevation, FT (MLW)
0
200
400
600
800
1000
1200
1400
1600
Distance, FT

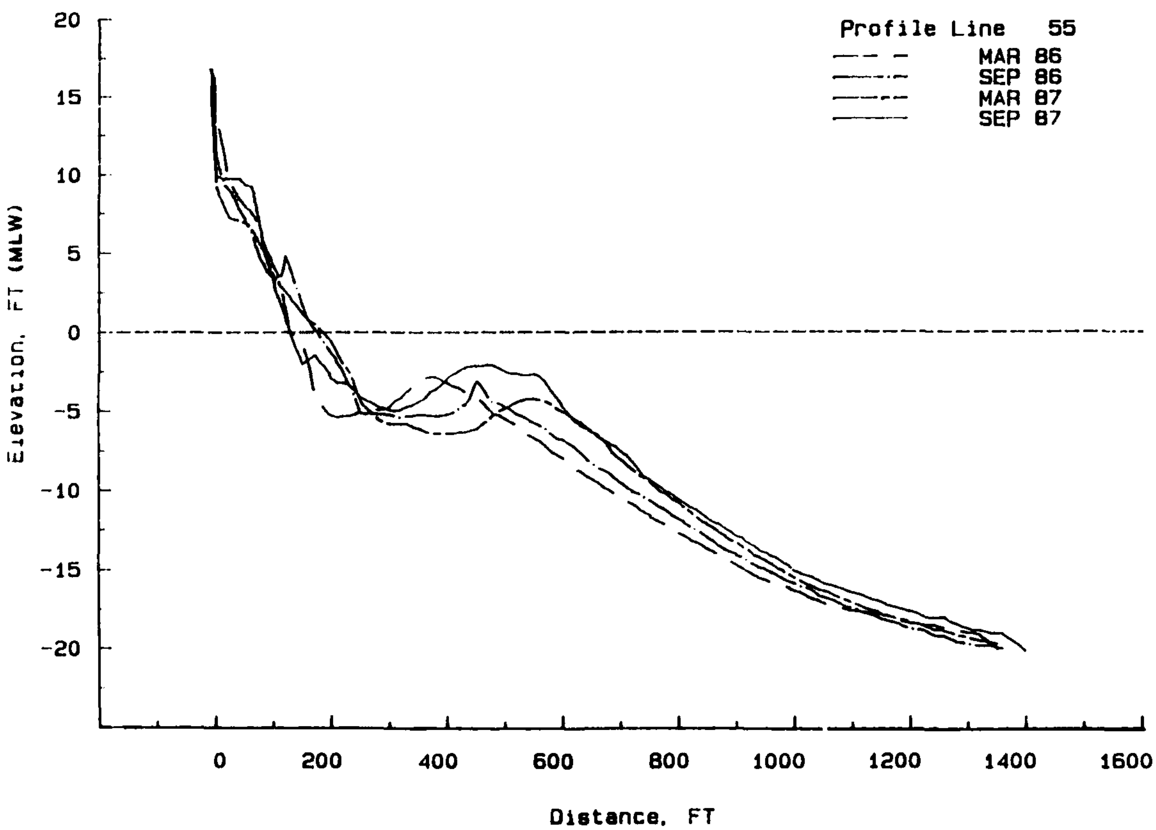


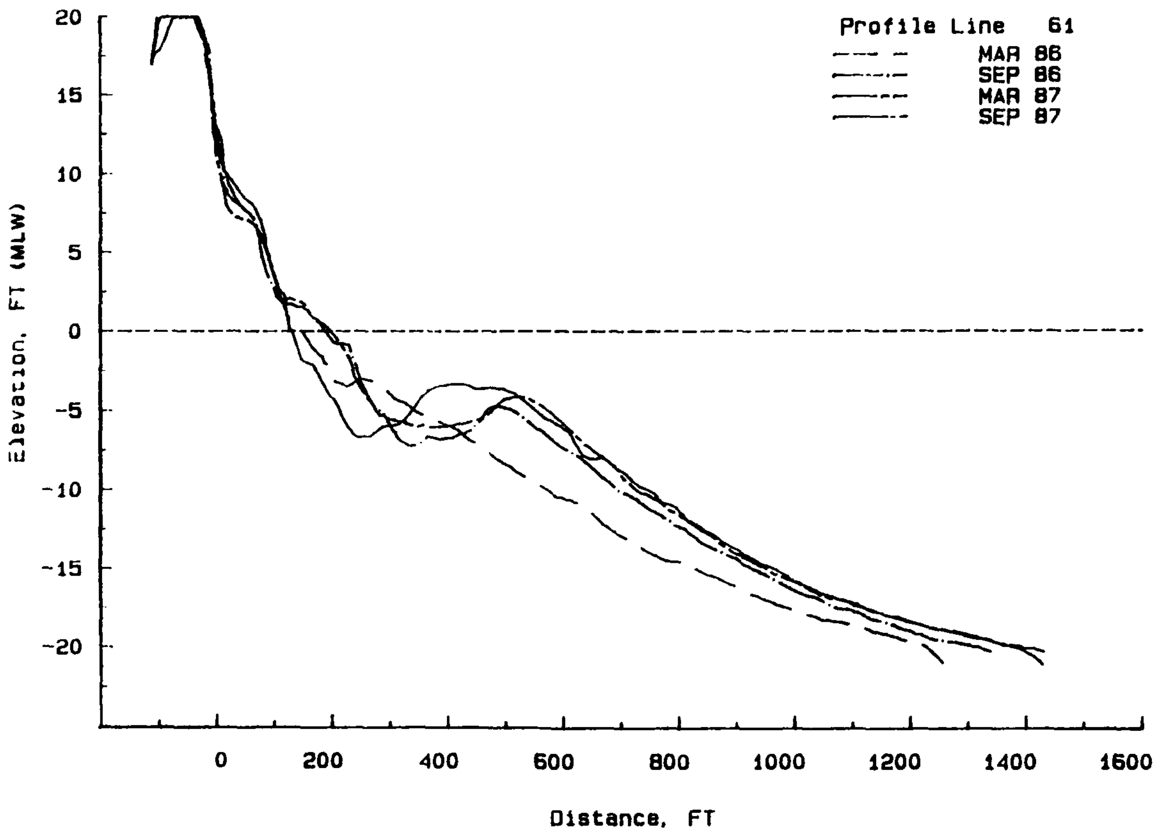
Profile Line 29
MAR 86
SEP 86
MAR 87
SEP 87
Elevation, FT (MLW)
20
15
10
5
0
-5
-10
-15
-20
0
200
400
600
800
1000
1200
1400
1600
Distance, FT



Profile Line 38
MAR 86
SEP 86
MAR 87
SEP 87
Elevation, FT (MLW)
20
15
10
5
0
-5
-10
-15
-20
0
200
400
600
800
1000
1200
1400
1600
Distance, FT

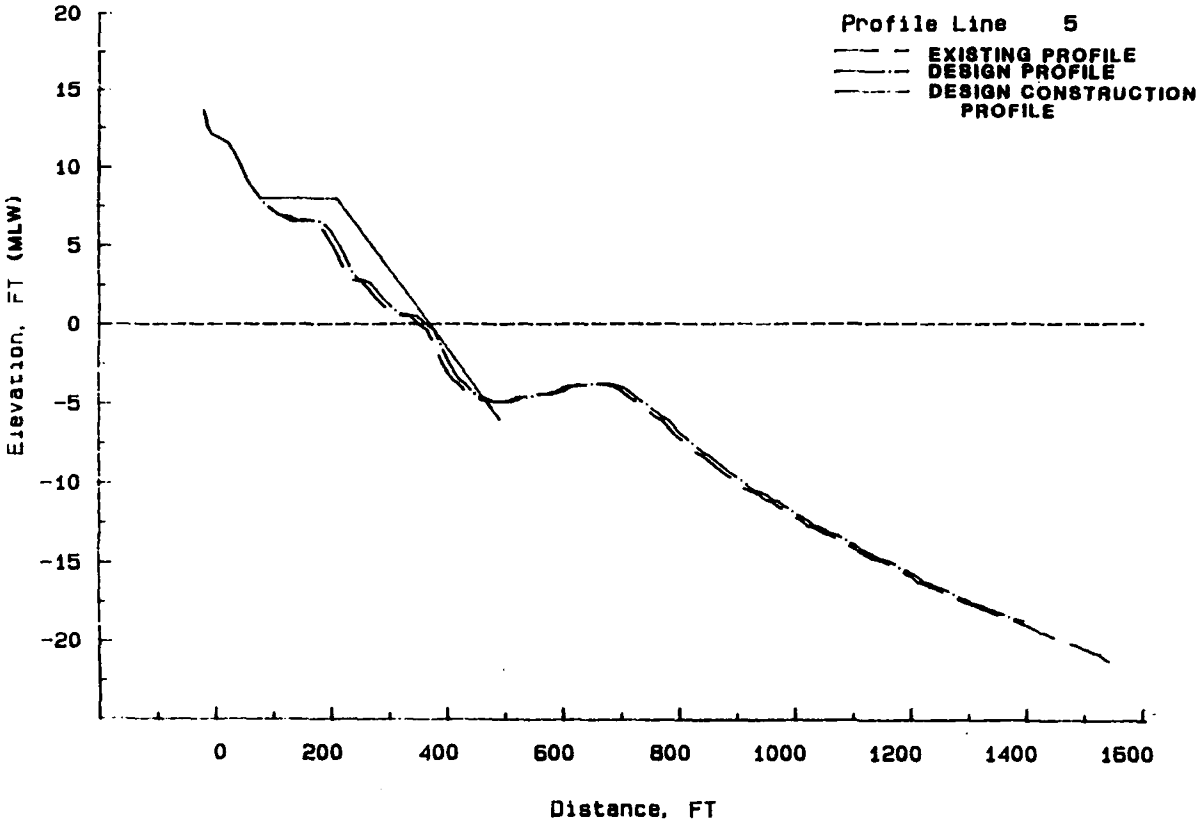

Profile Line 49
MAR 87
SEP 87
MAR 87
SEP 87
Elevation, FT(MLW)
20
15
10
5
0
-5
-10
-15
-20
0
200
400
600
800
1000
1200
1400
1600
Distance, FT

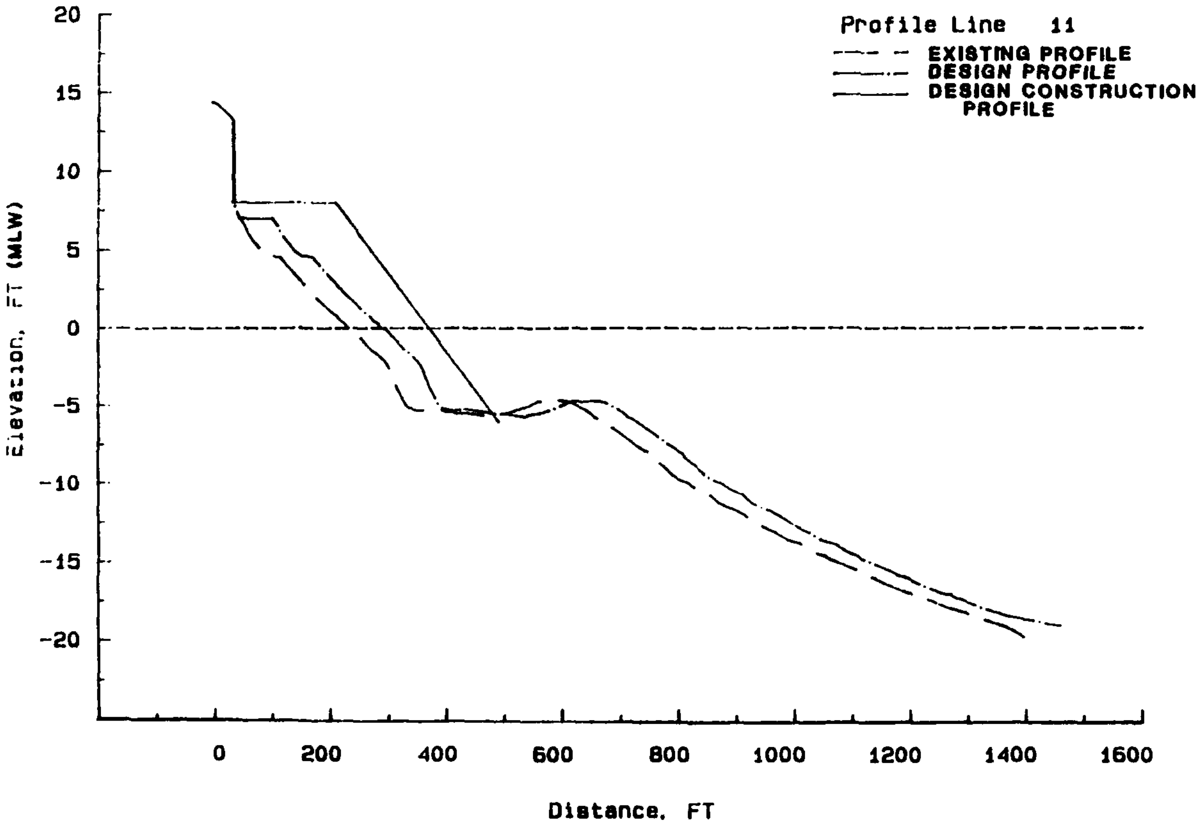

Profile Line 55
MAR 86
SEP 86
MAR 87
SEP 87
Elevation, FT (MLW)
20
15
10
5
0
-5
-10
-15
-20
0
200
400
600
800
1000
1200
1400
1600
Distance, FT

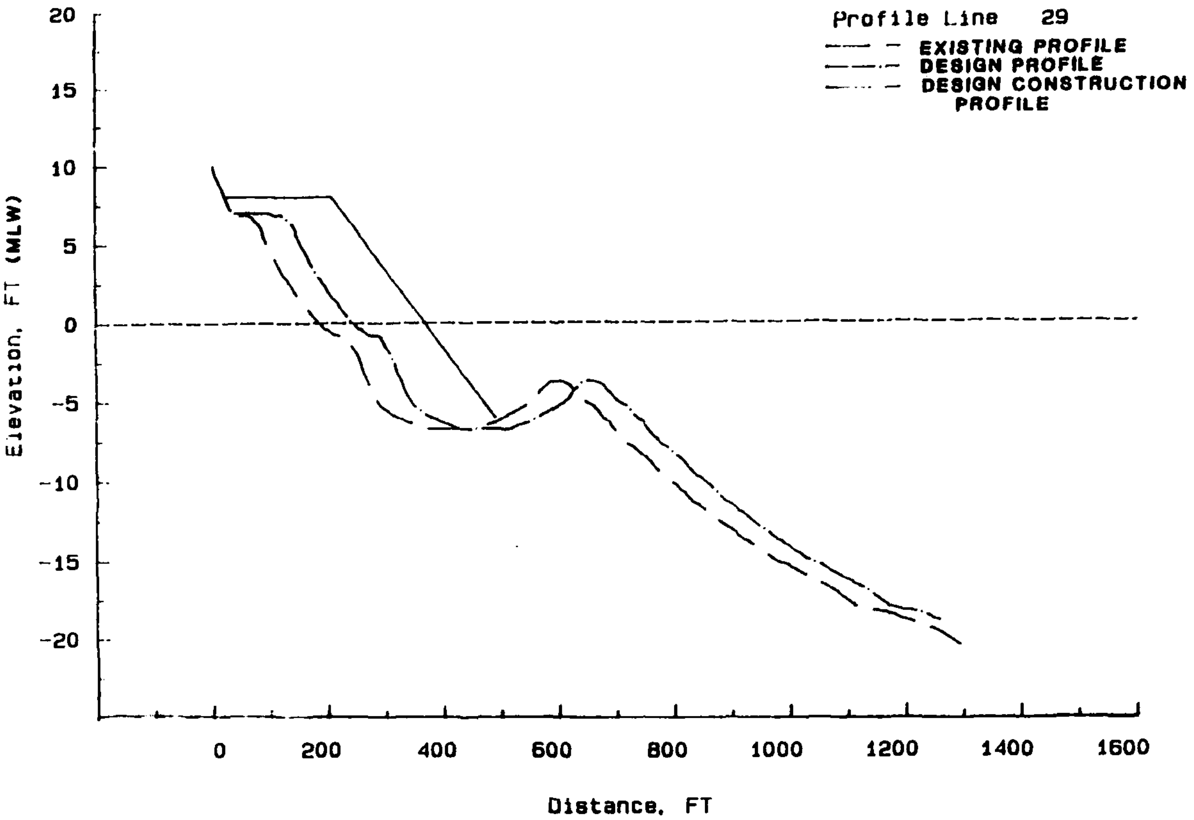

Profile Line 61
MAR 86
SEP 86
MAR 87
SEP 87
Elevation, FT (MLW)
20
15
10
5
0
-5
-10
-15
-20
0
200
400
600
800
1000
1200
1400
1600
Distance, FT

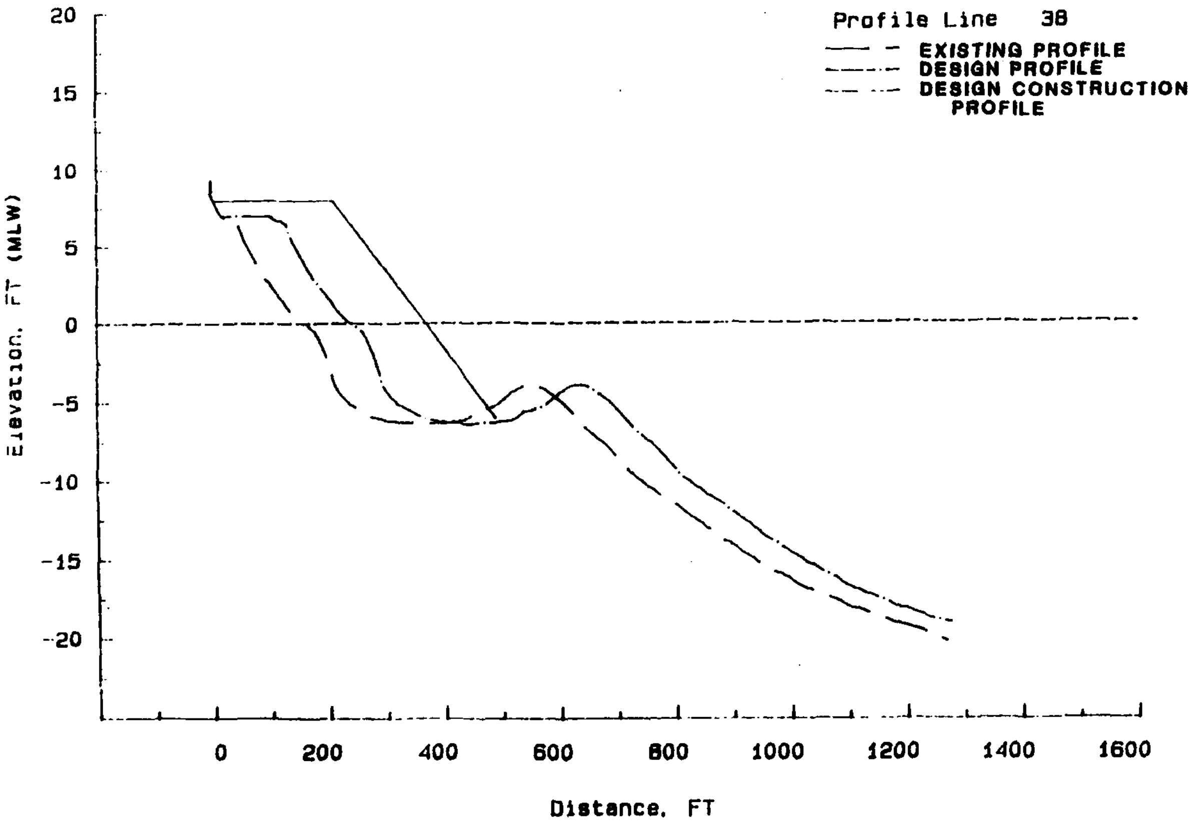
APPENDIX D: DESIGN AND DESIGN CONSTRUCTION PROFILES

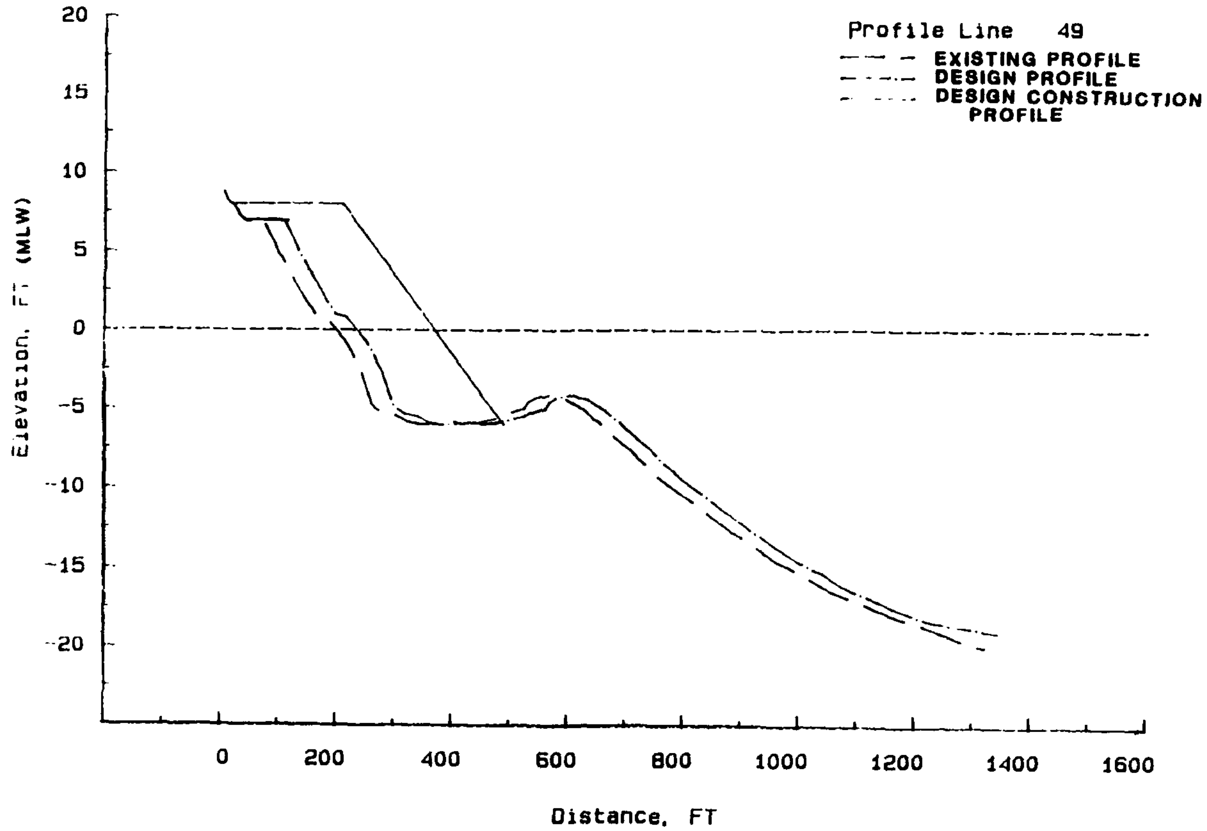
(Presented are the design profiles and design construction profiles for the seven native beach subreaches. Horizontal distance is relative to the construction control line. Elevations are relative to mean low water (MLW).)

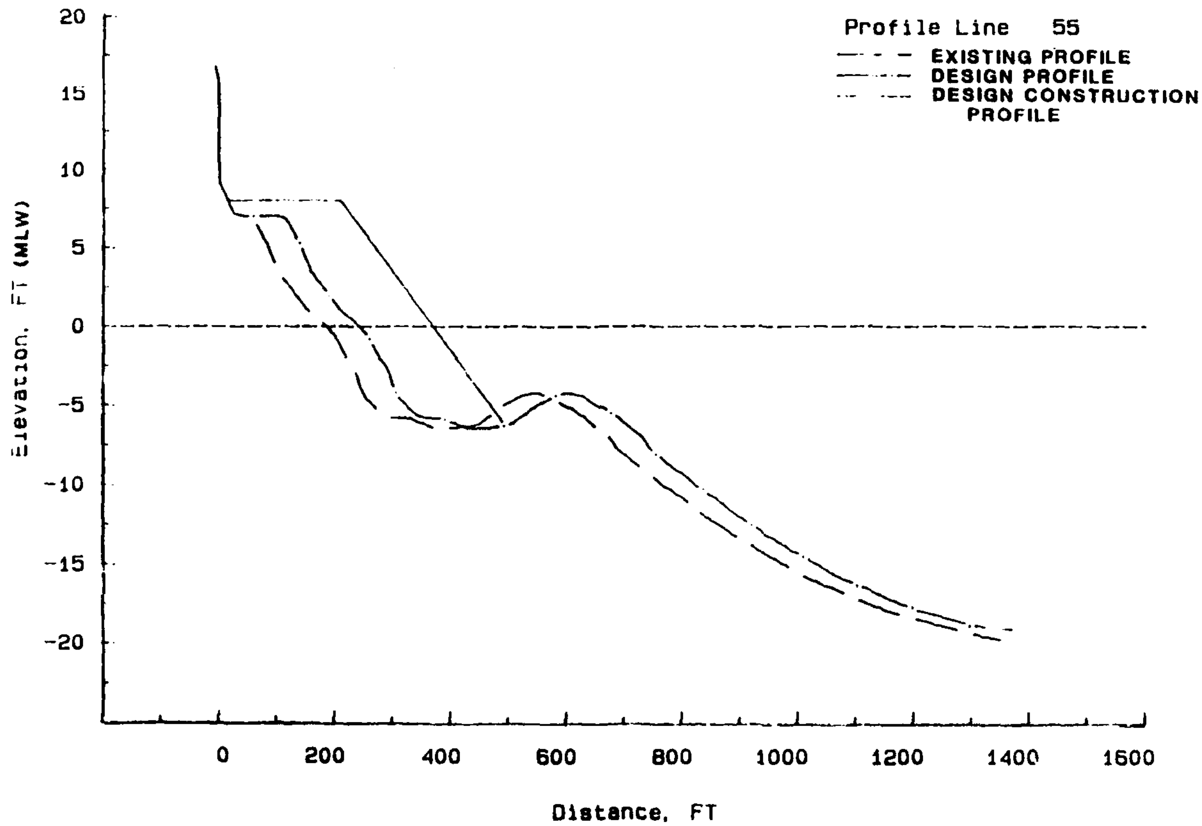

Profile Line 5
EXISTING PROFILE
DESIGN PROFILE
DESIGN CONSTRUCTION PROFILE
Elevation, FT (MLW)
Distance, FT

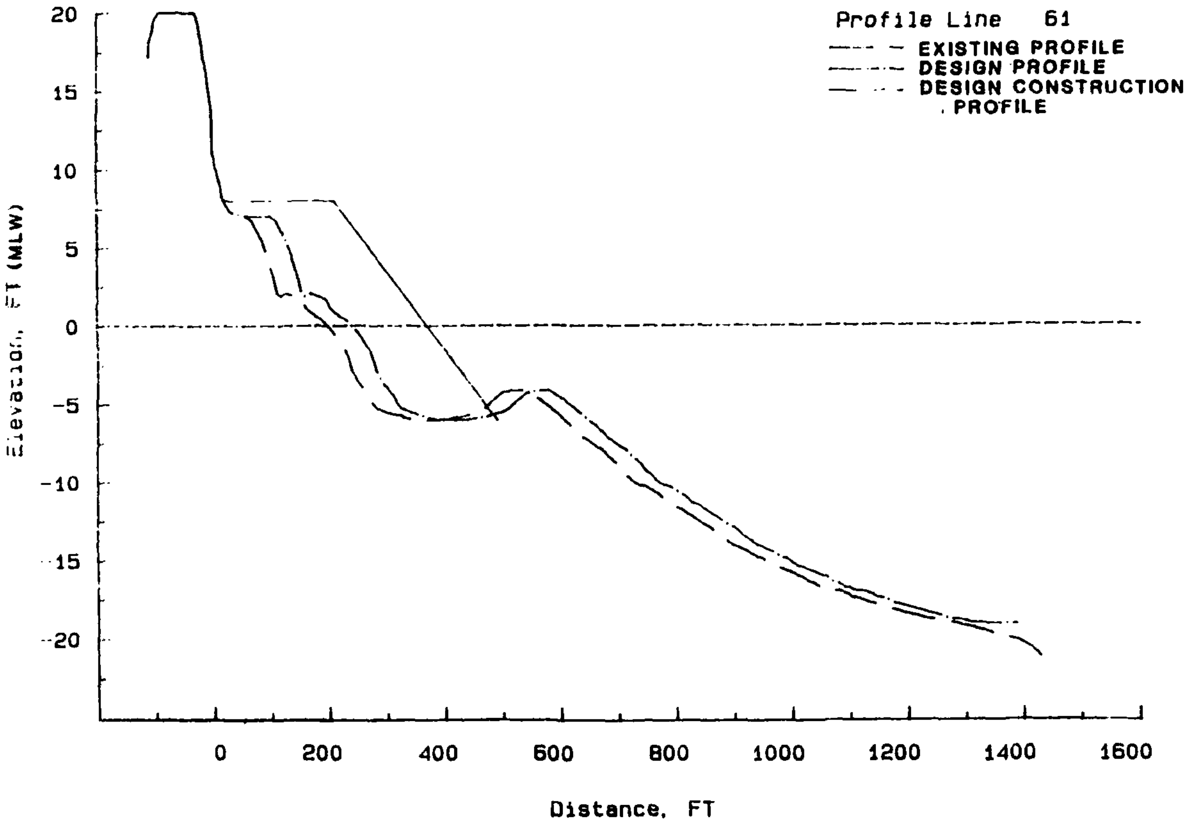

Profile Line 11
EXISTING PROFILE
DESIGN PROFILE
DESIGN CONSTRUCTION PROFILE
Elevation, FT (MLW)
Distance, FT


Profile Line 29
EXISTING PROFILE
DESIGN PROFILE
DESIGN CONSTRUCTION PROFILE
Elevation, FT (MLW)
Distance, FT
20
15
10
5
0
-5
-10
-15
-20
0
200
400
600
800
1000
1200
1400
1600


Profile Line 38
EXISTING PROFILE
DESIGN PROFILE
DESIGN CONSTRUCTION PROFILE
Elevation, FT (MLW)
Distance, FT
20
15
10
5
0
-5
-10
-15
-20
0
200
400
600
800
1000
1200
1400
1600


Profile Line 49
EXISTING PROFILE
DESIGN PROFILE
DESIGN CONSTRUCTION PROFILE
Elevation, FT (MLW)
20
15
10
5
0
-5
-10
-15
-20
0
200
400
600
800
1000
1200
1400
1600
Distance, FT

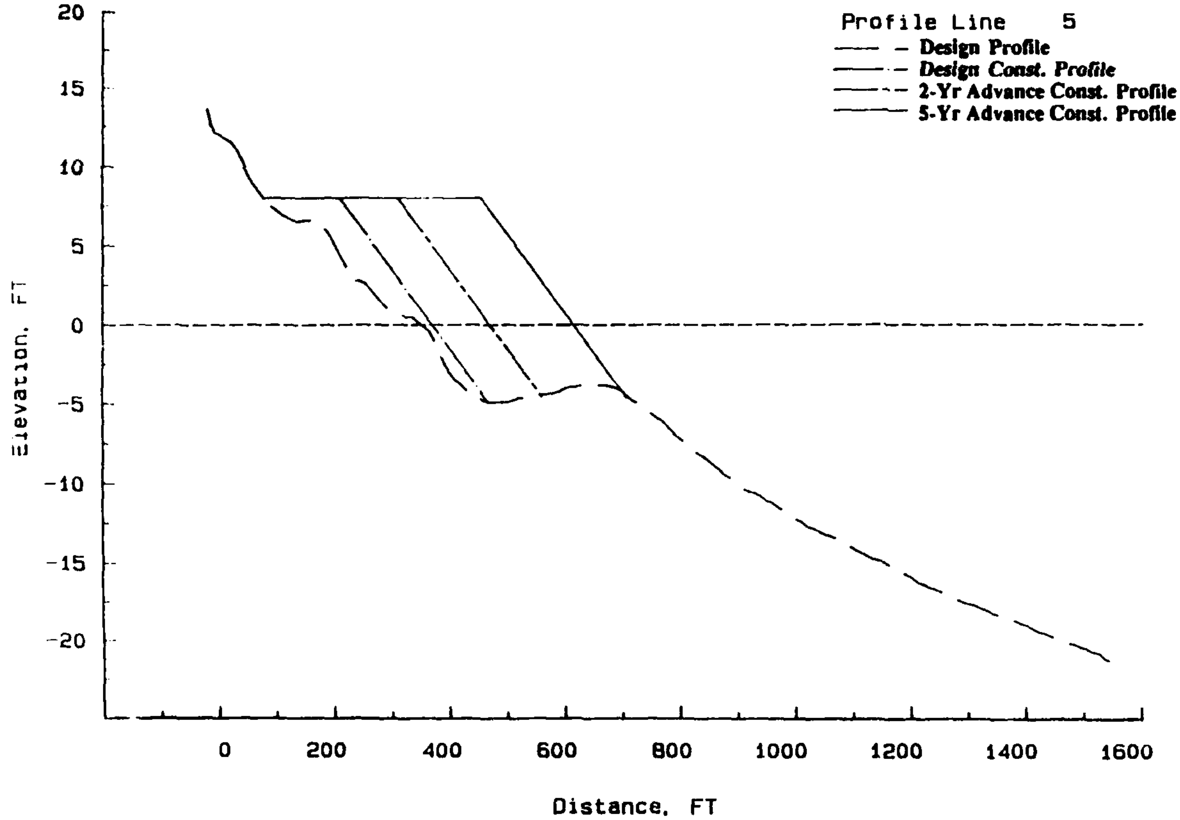

Profile Line 55
EXISTING PROFILE
DESIGN PROFILE
DESIGN CONSTRUCTION PROFILE
Elevation, FT (MLW)
20
15
10
5
0
-5
-10
-15
-20
0
200
400
600
800
1000
1200
1400
1600
Distance, FT



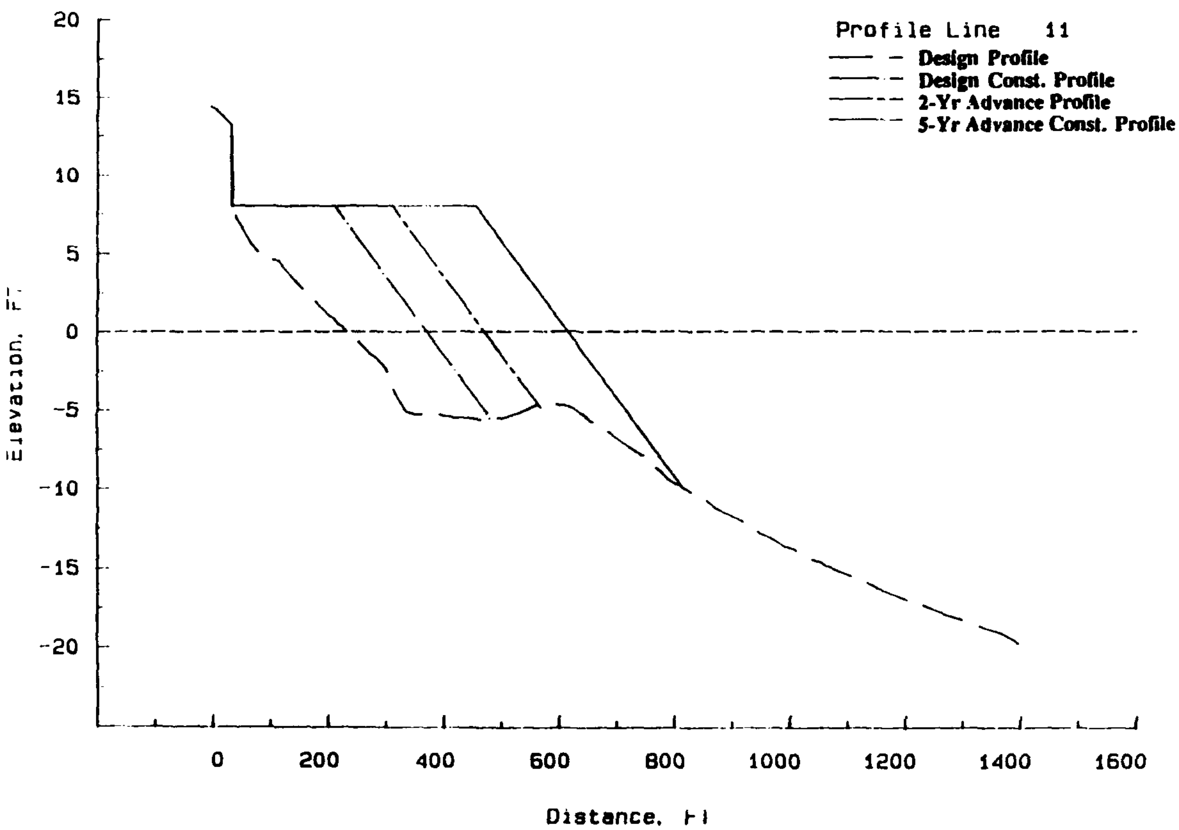
Profile Line 61
EXISTING PROFILE
DESIGN PROFILE
DESIGN CONSTRUCTION PROFILE
Elevation, FT (MLW)
Distance, FT
20
15
10
5
0
-5
-10
-15
-20
0
200
400
600
800
1000
1200
1400
1600

APPENDIX E: ADVANCE NOURISHMENT PROFILES

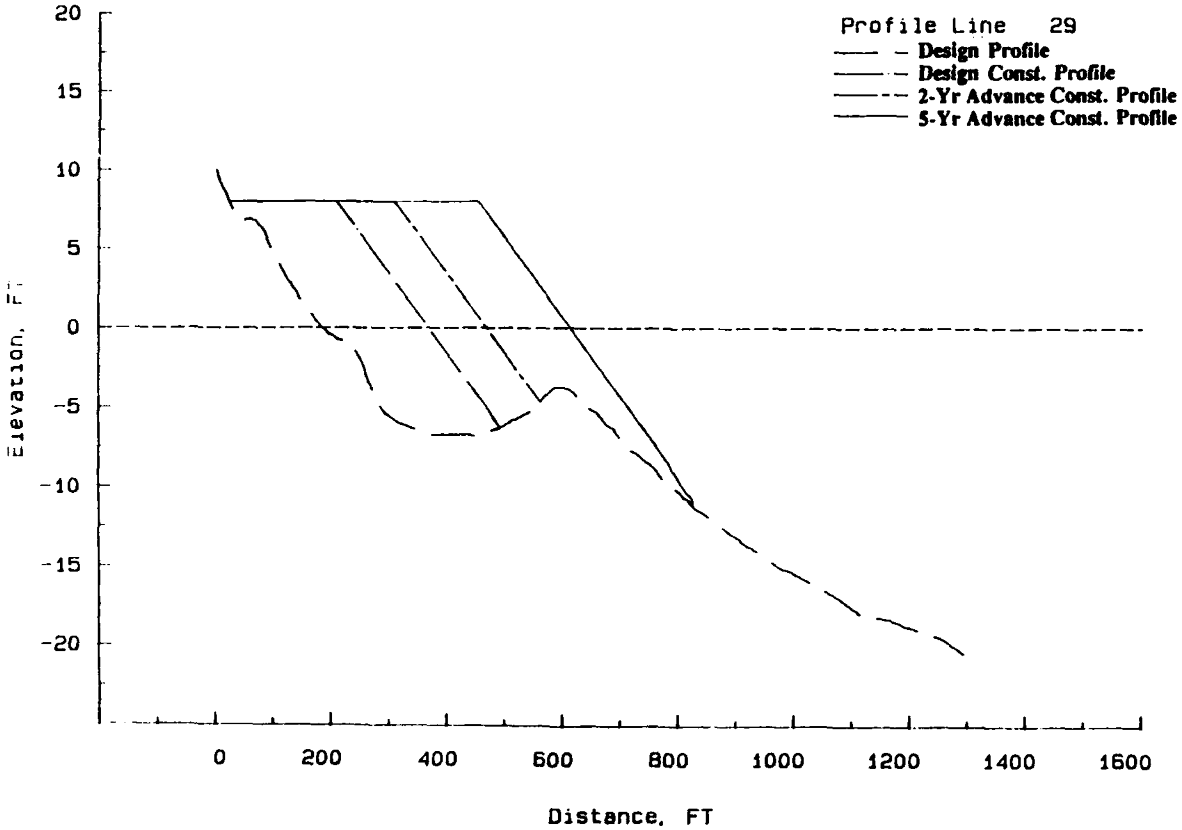
(Design construction profiles and 2- and 5-year advance nourishment profiles are presented for the seven native beach subreaches. Horizontal distance is relative to the construction control line. Elevations are relative to mean low water (MLW).)

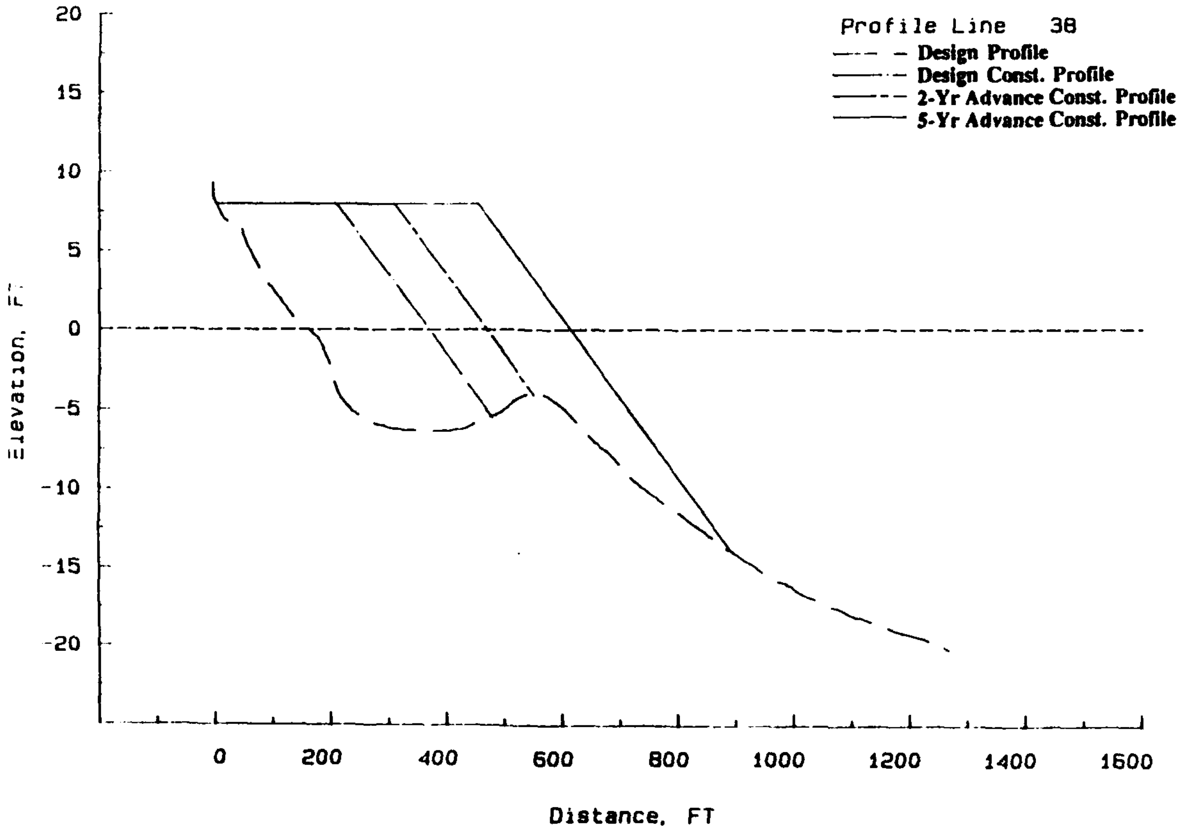


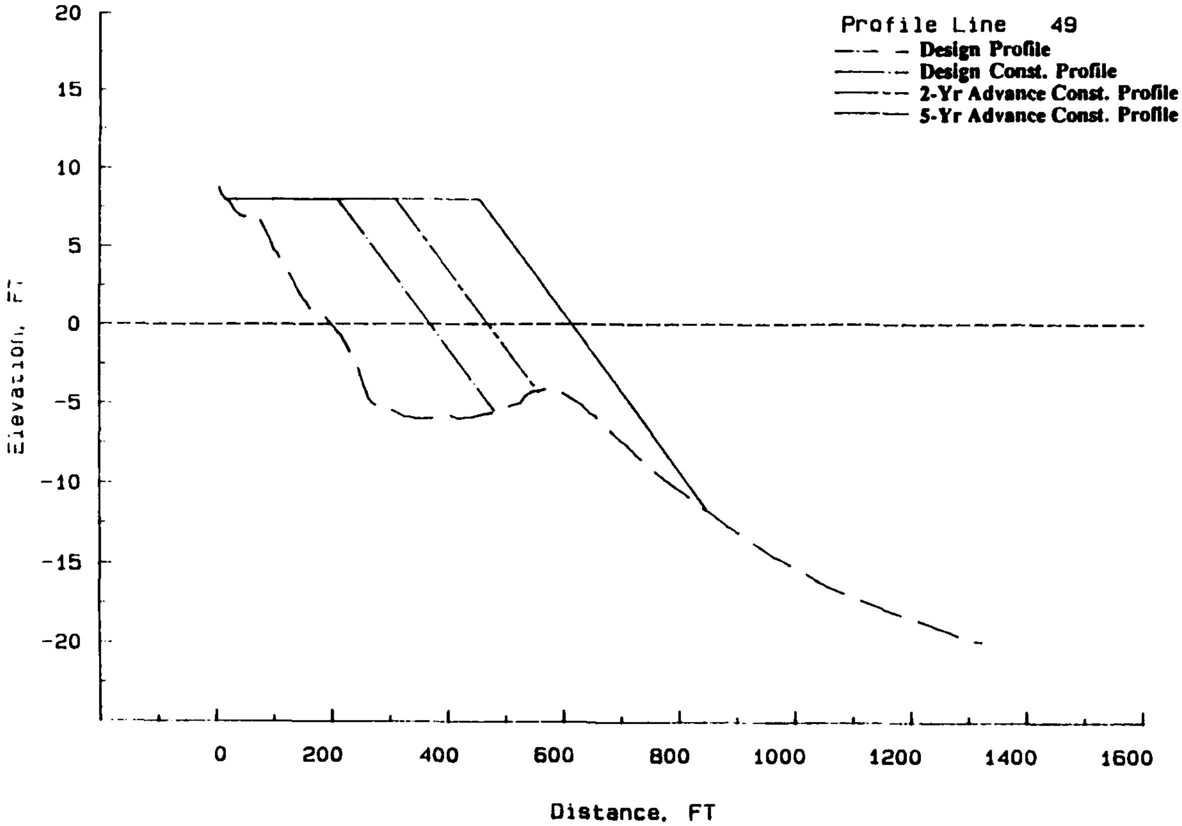
Profile Line 5
Design Profile
Design Const. Profile
2-Yr Advance Const. Profile
5-Yr Advance Const. Profile
Elevation, FT
Distance, FT
20
15
10
5
0
-5
-10
-15
-20
0
200
400
600
800
1000
1200
1400
1600



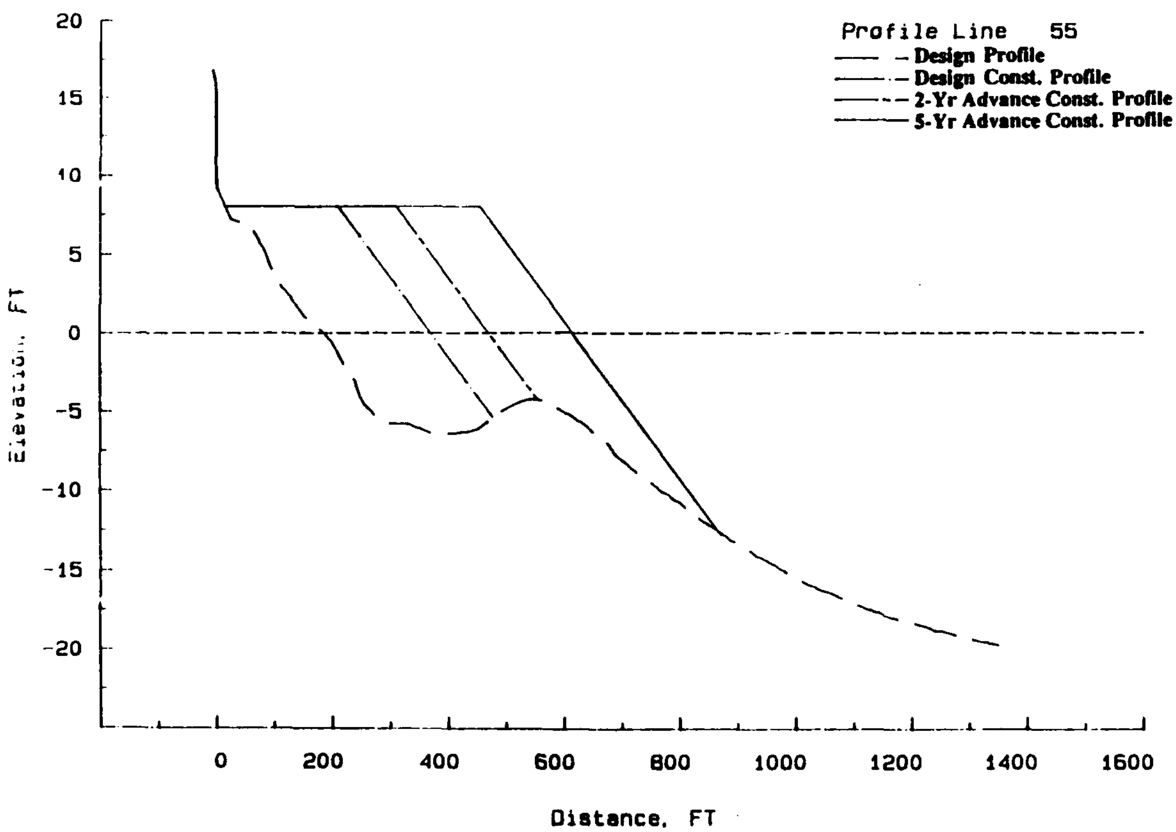
Profile Line 11
Design Profile
Design Const. Profile
2-Yr Advance Profile
5-Yr Advance Const. Profile
Elevation, FT
Distance, FT
20
15
10
5
0
-5
-10
-15
-20
0
200
400
600
800
1000
1200
1400
1600


Profile Line 29
Design Profile
Design Const. Profile
2-Yr Advance Const. Profile
5-Yr Advance Const. Profile
20
15
10
5
0
-5
-10
-15
-20
Elevation, FT
0
200
400
600
800
1000
1200
1400
1600
Distance, FT

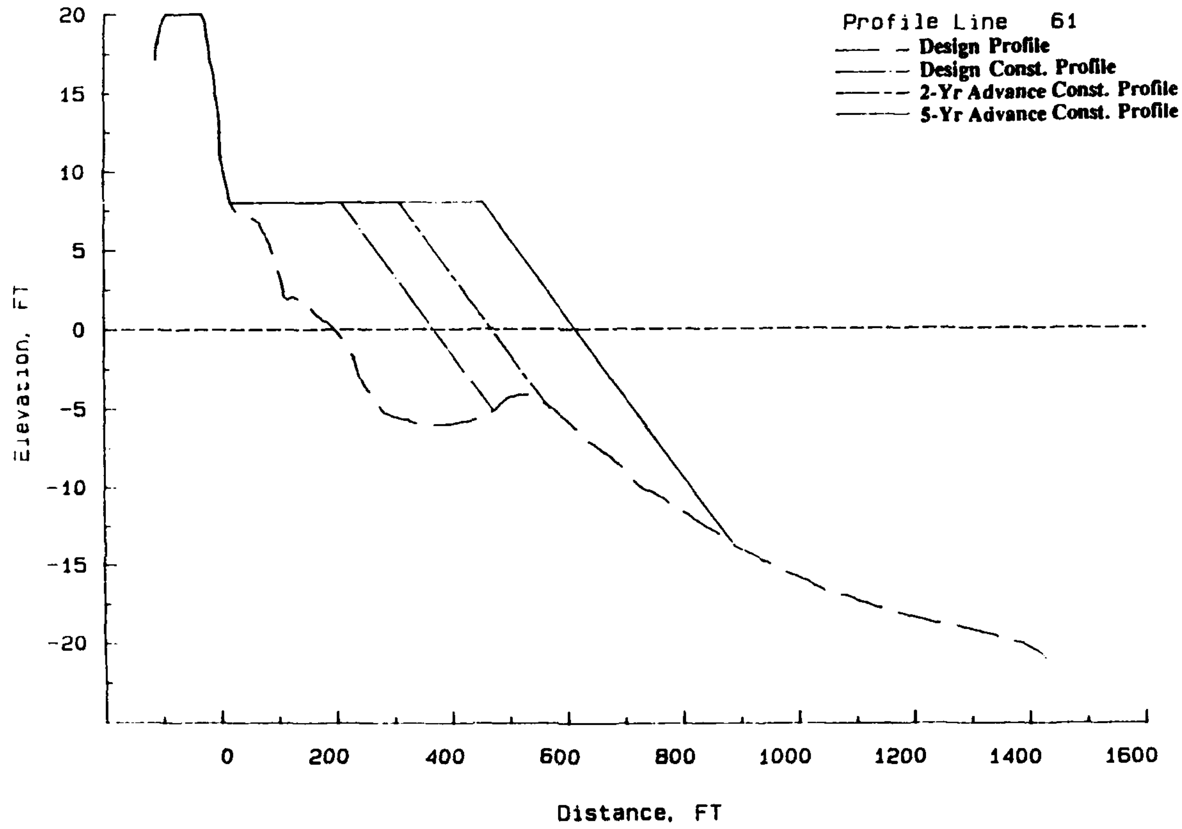

Profile Line 38
Design Profile
Design Const. Profile
2-Yr Advance Const. Profile
5-Yr Advance Const. Profile
20
15
10
5
0
-5
-10
-15
-20
Elevation, FT
0
200
400
600
800
1000
1200
1400
1600
Distance, FT



Profile Line 49
Design Profile
Design Const. Profile
2-Yr Advance Const. Profile
5-Yr Advance Const. Profile
Elevation, FT
Distance, FT



Profile Line 55
Design Profile
Design Const. Profile
2-Yr Advance Const. Profile
5-Yr Advance Const. Profile
Elevation, FT
Distance, FT



Profile Line 61
Design Profile
Design Const. Profile
2-Yr Advance Const. Profile
5-Yr Advance Const. Profile
20
15
10
5
0
-5
-10
-15
-20
Elevation, FT
0
200
400
600
800
1000
1200
1400
1600
Distance, FT