



Numerical Methods for Nearshore-Berm Evaluation, St. Johns County, Florida

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Numerical Methods for Nearshore-Berm Evaluation, St. Johns County, Florida

by Cheryl E. Pollock, William R. Curtis

U.S. Army Corps of Engineers
Waterways Experiment Station
3909 Halls Ferry Road
Vicksburg, MS 39180-6199

Hans R. Moritz

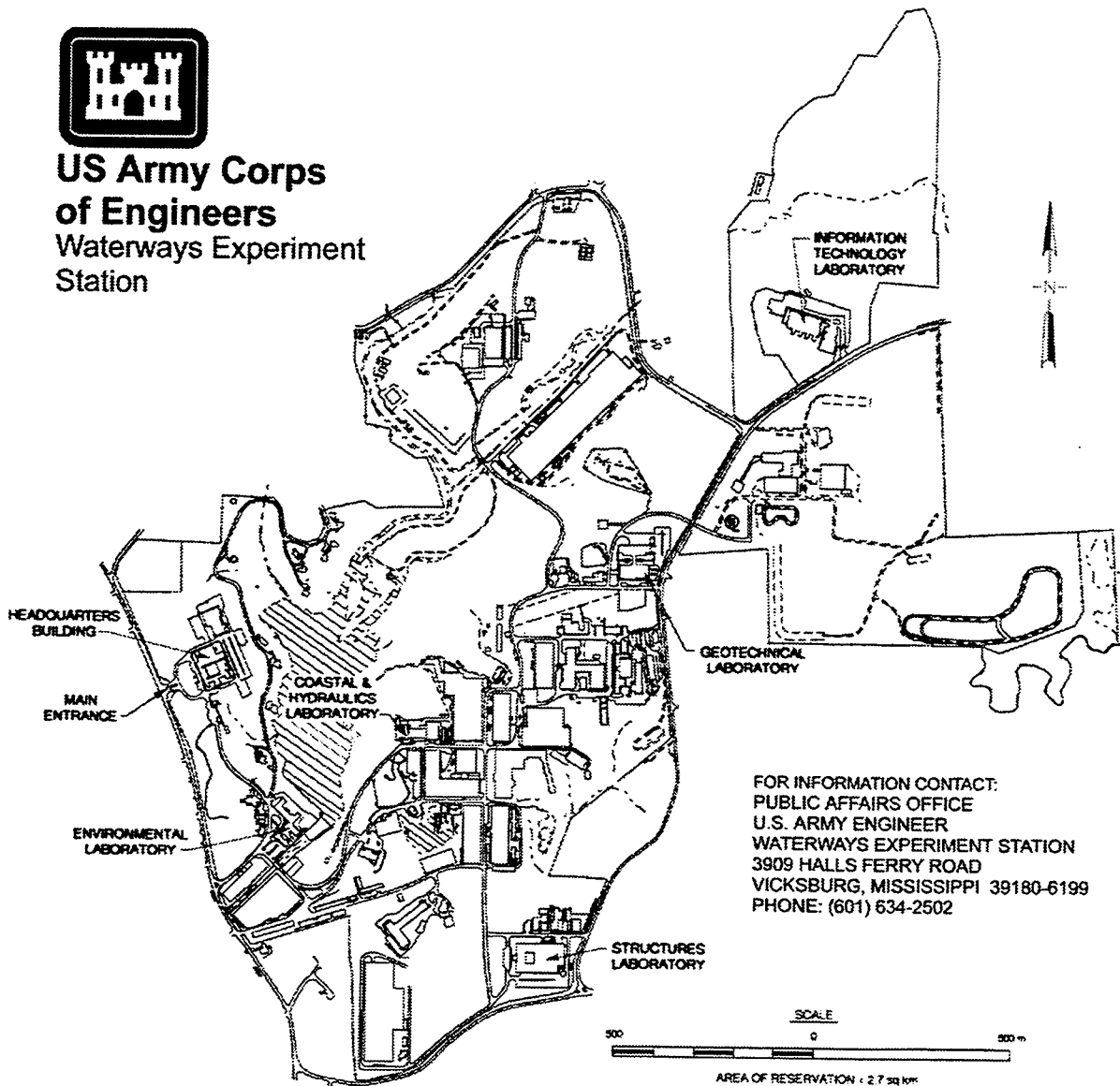
U.S. Army Engineer District, Portland
333 SW First Avenue
Portland, OR 97204-3495

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Station



FOR INFORMATION CONTACT:
PUBLIC AFFAIRS OFFICE
U.S. ARMY ENGINEER
WATERWAYS EXPERIMENT STATION
3909 HALLS FERRY ROAD
VICKSBURG, MISSISSIPPI 39180-6199
PHONE: (601) 634-2502

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Preface

The study summarized in this report was performed in support of the St. Johns County, Florida, Shore Protection project for the U.S. Army Engineer District, Jacksonville, and was conducted by the U.S. Army Engineer Waterways Experiment Station's (WES) Coastal and Hydraulics Laboratory (CHL). The CHL was formed in October 1996 with the merger of the WES Coastal Engineering Research Center (CERC) and Hydraulics Laboratory. Dr. James R. Houston is the Director of the CHL and Mr. Charles C. Calhoun, Jr., is Assistant Director. Numerical methods for addressing storm-related recession and renourishment are developed and applied to an existing nearshore profile and several nearshore berm options for St. Johns County.

Work was performed under the CHL general administrative supervision of Dr. Yen-hsi Chu, Chief, Engineering Applications Unit; Ms. Joan Pope, Chief, Coastal Structures and Evaluation Branch; Mr. Thomas W. Richardson, Chief, Coastal Sediments and Engineering Division; Mr. Calhoun, Assistant Director; and Dr. Houston, Director.

This report was prepared by Ms. Cheryl E. Pollock and Mr. William R. Curtis, CHL, and Mr. Hans R. Moritz, U.S. Army Engineer District, Portland. Assistance in planning this study was also provided by Ms. Pope and Messrs. Richardson, Bruce Ebersole, and Randy Wise, all of CHL, and Thomas Smith, Jacksonville District. Figures were prepared by Msses. Mary Allison and Laura Teaford, and Messrs. Robert Chain and Ronald Neihaus, all of CHL. Mr. Gerius Patterson, Student contractor, assisted in data analysis. Ms. Janie Daughtry, CHL, assisted with final report preparation.

Dr. Robert W. Whalin was Director of WES at the time of the publication of this report. COL Robin R. Cababa, EN, was WES Commander.

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1 Introduction

In support of the St. Johns County, Florida, Shore Protection Project under the jurisdiction of U.S. Army Engineer District, Jacksonville (herein referred to as Jacksonville District), the use of nearshore berms in combination with beach nourishment is investigated as shore protection features for mitigating storm damage to upland properties and to reduce shoreline erosion. Placement of a nearshore berm on the existing profile can initiate wave breaking offshore, decreasing wave energy at the shore and reducing storm impacts. Physical estimates for annual beach recession rates, project renourishment rates, and storm-related recession distance are required for economic evaluation of a nearshore berm project. This information is site specific and varies for different shoreline protection options. Documented in this report is the evaluation of numerically simulated storm-related beach recession rates and storm-related nearshore berm renourishment rates associated with nearshore berm profile geometries for a portion of St. Johns County.

The U.S. Army Corps of Engineers is a proponent of constructive use of clean dredged material. Beneficial uses of such material include creation of terrestrial and aquatic habitat, wetlands and placement of beach fills. The concept of placing clean dredged material in the nearshore in the form of shore-parallel linear berms has recently gained acceptance as a potential means of enhancing or nourishing the beach profile. Benefits of nearshore berms may include the addition of material to the littoral system and attenuation of wave energy incident on the beach profile. A precedent was established as a result of this study. Methods were developed for evaluating a nearshore berm's effect on the landward extent of the profile envelope. These methods are documented in this report. Results from these methods were applied by Jacksonville District to determine the economic feasibility of nearshore berms to the St. Johns County, Florida, Beach Erosion Control Project on an event-related basis.

Project Overview

This investigation developed methodologies for addressing storm-related berm recession and renourishment requirements to maintain the nearshore berm. Using the developed methodologies, site-specific predictions were made for the existing beach profile condition at St. Johns County and several engineered design

templates for nearshore berm alternatives. To accomplish this effort, a series of activities were required: developing forcing climate; designing profile templates; developing methodologies for predicting recession and renourishment; computing beach recession from selected storms for each profile template; and evaluating profile templates to compute nearshore berm renourishment rates. These activities, along with climatic information and existing numerical models were used to predict storm-related recession and storm-related renourishment for the nearshore berm profiles. Results from this study were used by Jacksonville District for event frequency correlation and economic evaluation of benefit provided by nearshore berms. A brief description of the study activities follows.

Activity 1 - develop forcing climate

The driving forces used in the numerical models were developed from measured and hindcast climatic information including waves, water levels, and storm duration. The suite of actual storms that occurred at the site between 1984 and 1991 exhibited a maximum deep-water wave height and water level of 6.8 and 1.4 m National Vertical Geodetic Datum (NGVD), respectively. Three more severe storms were synthesized resulting in a maximum wave height and water level of 6.2 and 2.71 m NGVD, respectively.

Activity 2 - creating design profile templates

The existing condition profile template used in this study is a generic profile that represents a typical profile of the St. Johns County unstructured shoreline. The design profile templates for nearshore berm and beach fill combinations are the result of adding material to the existing condition template. Template designs were based on existing guidance for nearshore berms and beach fills and are sensitive to stability and placement cost.

Activity 3 - develop methodologies for predicting recession and renourishment

Methodologies for predicting beach recession and renourishment rates for the templates using numerical models were developed.

Two paths for developing methodologies to estimate recession were pursued. This first approach was to place the berm on the profile and use the Storm Induced BEAch Change model, Version 2.0 (SBEACH 2.0) to predict beach recession (Rosati et al. 1993). The suite of storm events were individually applied to the profile templates to predict recession related to each storm event. Following Kraus and Larson (1991), the second approach used Numerical Model of the LONGshore Current (NMLONG) to predict wave characteristics in the lee of the berm and used these wave conditions on the without-berm profile in SBEACH 2.0 to predict beach recession rates. A comparison of results from the two approaches is presented in this report.

Nearshore berm renourishment volumes were predicted using both SBEACH 2.0 and the Long Term FATE model (LTFATE) (Scheffner et al. 1995). Separate testing was required to estimate the storm-event-related beach recession and nearshore berm renourishment rates.

Activity 4 - computing beach recession from selected storms for each profile template

The selected methodology was used to predict storm-related recession of the existing condition template and of the four design templates for beach fill and nearshore berm combinations for the measured, hindcast, and synthetic storms.

Activity 5 - evaluating profile templates to estimate nearshore berm renourishment rates

The selected methodology was used to predict berm renourishment rates for the four design nearshore berm profiles.

This report documents the work done for Jacksonville District in support of the St. Johns County Shore Protection Project. The methodology developed to compute reduction of beach erosion rates induced by nearshore berms has broad application to Corps-wide use of nearshore berms. In addition, Appendix A documents nearshore-berm profile template designs, Appendix B documents applied hydrodynamic events, Appendix C documents applied SBEACH 2.0 input parameters, Appendix D documents berm profile response to SBEACH 2.0 simulations, and Appendix E documents berm profile response to LTFATE simulations.

Site Description and Background

As outlined in "St. Johns County, Florida, Beach Erosion Control Project, Special Report, St. Augustine Beach Nourishment," prepared by the U.S. Army Corps of Engineers (1990):

The project area lies along a 13,200 ft [4,024 m] length of St. Johns County, Florida, beginning approximately 14,500 ft [4,421 m] south of the St. Augustine Inlet. The portion of the shoreline under study in this report is centered approximately on the St. Augustine Beach public pier, and extends from survey monuments DNR-137A southward to DNR-150. The study area is shown in Figure 1.

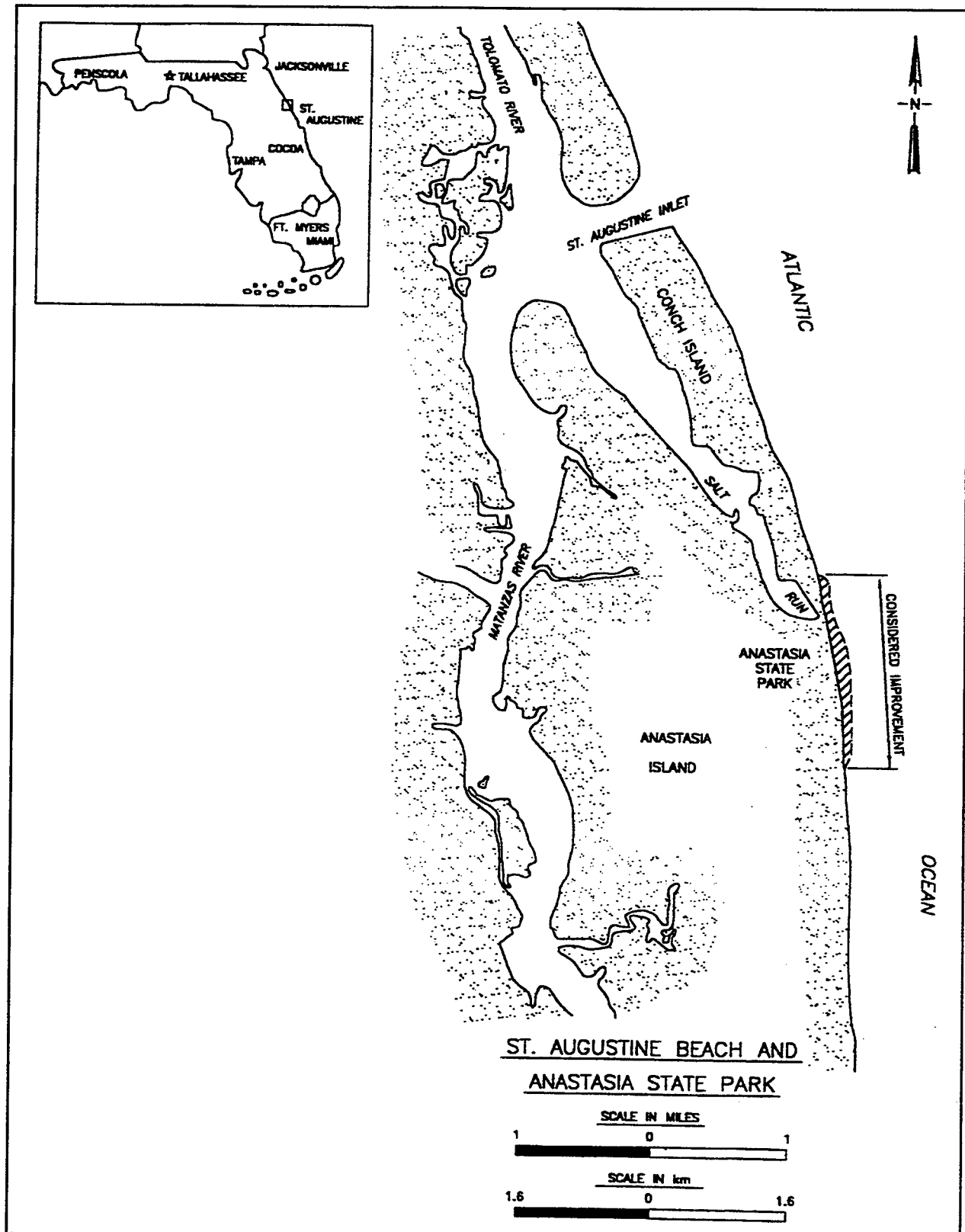


Figure 1. Project location: St. Augustine Beach and Anastasia State Park, St. Johns County, Florida

Due to the varied upland characteristics within the study area, the project length may be further divided into three reaches. The northernmost reach extends along 4,100 ft [1,250 m] of undeveloped, sandy shoreline. The southernmost 1,800 ft [549 m] of this reach is protected by an upland riprap revetment, and the northern portion of this reach extends into Anastasia State Park. The middle reach extends along a 4,800 ft [1,463 m] length of revetment and revetted seawall, some of which is inundated at higher tide levels. This revetment protects several multi-story hotels and condominiums, as well as many private residences and apartments. The southernmost reach is 4,300 ft [1,311 m] long and consists mostly of a wide sandy beach, with a well-developed dune line. Houses along this reach are generally set back at least 100 ft (30 m) from the dune line.

Historically, this portion of St. Johns County has been highly unstable, and in recent years it has experienced considerable beach erosion. Changes in shoreline position have been accurately documented since 1858, and coastal charts dating back to 1586 show approximate shoreline positions. Prior to stabilization of the St. Augustine Inlet, shoreline position fluctuated greatly as the inlet constantly underwent changes in depth, width, position, and alignment. Following the stabilization of St. Augustine Inlet beginning in the 1940's, the adjacent shorelines also stabilized, but the beaches to the south soon began to experience erosion.

In order to prevent erosional damage to upland property, and to restore the recreational value of the beaches in this eroded area, the shoreline benefits from wave attenuation and the economic feasibility of constructing various nearshore berm configurations are investigated and documented here.

2 Storm-Related Recession

Numerical Methods for Estimating Shoreline Recession

Two initial techniques for development of a methodology to estimate shoreline recession were pursued. The first technique was to place the berm on the profile and use SBEACH 2.0 to predict beach recession. Results from SBEACH 2.0 are event related and use a time series for storm surge and associated wave input. The suite of storm events was individually applied to profile templates to predict recession related to each event. Estimated recession rates yield relative comparisons for shoreline protection afforded by different nearshore berm geometries.

The second approach used NMLONG to predict the wave characteristics in the lee of the berm and used these wave conditions on the without-berm profile in SBEACH 2.0 to predict beach recession rates. NMLONG generates a series of linear waves that simulate a spectrum for a specific deep-water significant wave height (H_{mo}) and spectral peak wave period (T_p). When applying NMLONG, individual waves are transformed across a given profile after passing over the berm. Reformed spectral H_{mo} and T_p are calculated along the profile. NMLONG treats the berm as a hard structure (similar to a reef) of the minimum berm dimensions. The reformed wave in the lee of the structure is extracted and used in SBEACH 2.0 to predict profile changes.

Many questions arose in preparing input to the models. Should the minimum or maximum berm profile be used? Which model best predicts wave characteristics in the lee of the berm? Should the berm be allowed to erode or be treated as a hard structure? Can benefits be claimed for material migrating to the beach? These questions, and others, were addressed in the process of developing a methodology to predict profile recession.

Intermodel comparison: SBEACH 2.0 versus NMLONG

An intermodel comparison investigating wave transformation over a designed nearshore berm was conducted using SBEACH 2.0 and NMLONG. Wave

transformation in SBEACH 2.0 and NMLONG included wave shoaling, refraction, breaking, and reformation. Wave shoaling and refraction were calculated based on linear wave theory. Wave breaking and reformation were calculated based on theory presented by Dally, Dean, and Dalrymple (1985). Three profiles (see Figure 2) were selected for testing. A single monochromatic wave was input at the offshore boundary of the profiles and propagated across the profiles using SBEACH 2.0 and NMLONG. Several tests were conducted for each profile using varying input wave conditions. (Refer to Table 1 for input wave parameters.) A shore-normal direction of wave approach and a still-water elevation of 0.0 m NGVD were assumed. Model results were similar where wave breaking and energy dissipation occurred over the natural bar system in the surf zone. SBEACH 2.0-calculated breaking wave heights in the surf zone were approximately 0.5 m less than those calculated by NMLONG. This result can be attributed to the breaking wave height to depth ratio (H_b/d_b) parameter. H_b/d_b was input as a constant to NMLONG over the length of the profile, whereas H_b/d_b is calculated as a function of H_o/d_o by SBEACH 2.0 for each grid cell using the following equation:

$$\frac{H_b}{d_b} = 1.14[\tan(\beta)/\frac{H_o}{d_o}]^{0.21} \quad (1)$$

where

H_b and d_b = breaking wave height and depth of breaking

H_o and d_o = deep-water wave height and water depth

β = profile slope

SBEACH 2.0 was selected to transform waves from the seaward profile boundary across the design berm configuration. This selection was based on the fact that SBEACH 2.0 explicitly solves for H_b/d_b at each grid cell. In addition, SBEACH 2.0 has wave height randomizing capability. If the time between consecutive wave entries is long relative to the time-step, randomization may be applied to more realistically simulate in the wave field.

Numerical modeling processes modifications

Testing with SBEACH 2.0 under the prescribed procedures (termed Method I) resulted in little or no benefit from nearshore berms placed on the profile. Recession rates were insensitive to the effects of the features reducing incident wave height. This is due to the method in which SBEACH 2.0 calculates runup. Wave information initially entered into the model seaward of the berm is used to calculate wave runup and setup at the beach. The model does not recalculate runup and setup associated with the lesser height of the reformed wave in the lee of the berm. SBEACH 2.0 Method II and Method III were developed to address this problem. Methods II and III are unverified except for limited comparisons by

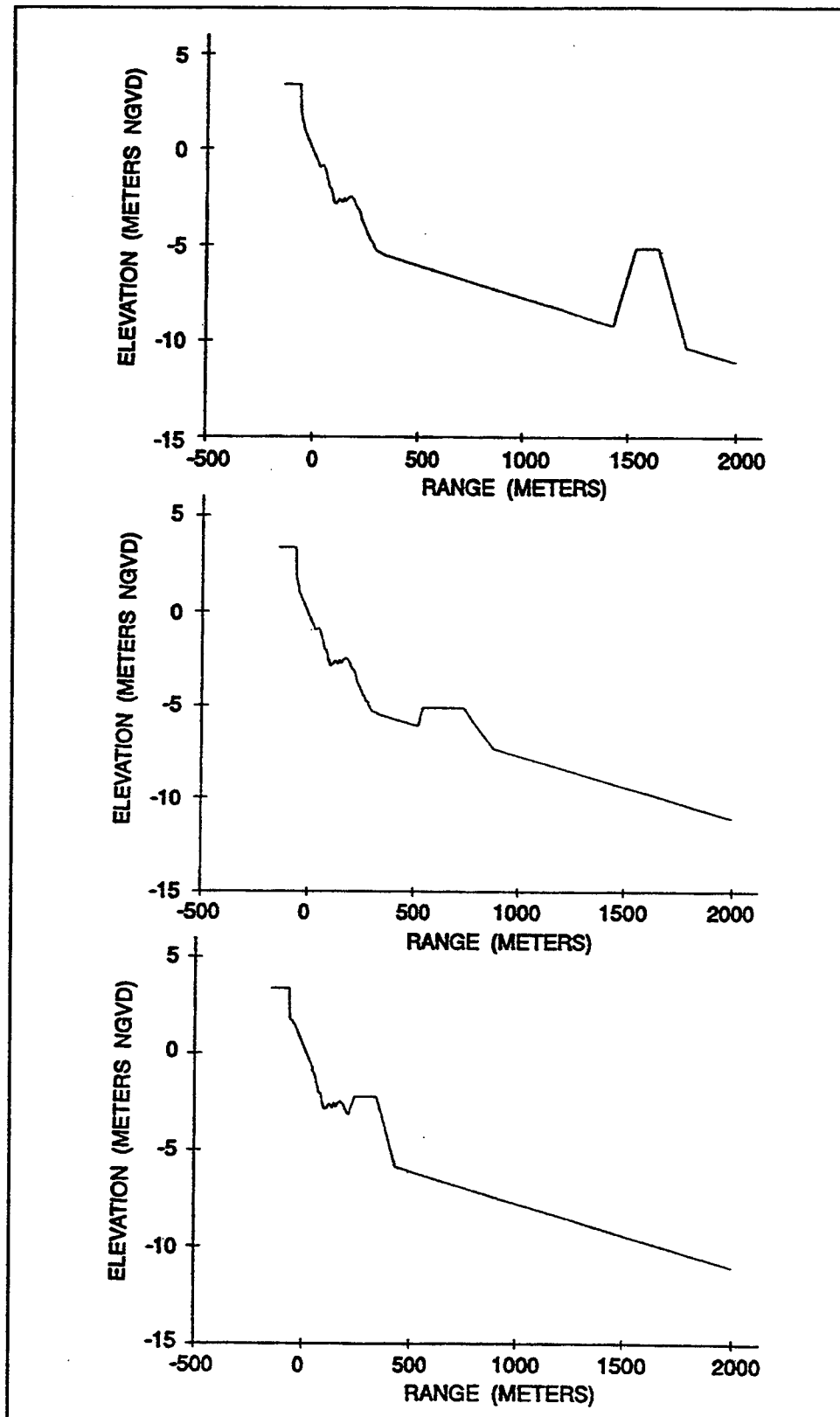


Figure 2. Intermodel comparison: SBEACH 2.0 versus NMLONG. Profile templates

Table 1
Input Wave Conditions for
SBEACH 2.0/NMLONG

H _s , m	T _p , sec
2	15
3	15
4	17
5	17
6	20
7	20

the author with data from the SUPERTANK Laboratory Investigation (Kraus and Smith 1993). This comparison substantiates the use of Methods II and III to better describe profile changes when a nearshore berm is present. Smith (1994) found similar results applying NMLONG to the SUPERTANK Laboratory Investigation data set.

For SBEACH 2.0 Method II, the output from SBEACH 2.0 Method I was used to visually select the cell location in the lee of the berm where the wave reformed and stabilized. At that cell location, the reformed wave information was saved at each time-step. The reformed wave data were then entered on the without-berm profile at the same cell location (in the lee of the berm). This method allows benefits (i.e., reductions in shoreline recession) to be claimed for the reduced runup and setup associated with energy reductions from the wave breaking over the berm and reforming at a reduced wave height.

SBEACH 2.0 Method III is identical to Method II except, in addition to wave information, water level information was saved at the selected cell location and entered on the without-berm profile. This method accounts for setup produced during wave breaking over the berm. Methods II and III yield results that more intuitively represent expected trends for profile recession in the lee of a nearshore berm. Prototype data are not available to discriminate between Methods II or III. Example SBEACH 2.0 input parameters for Methods I, II, and III are presented in Appendix C.

Nearshore Profile Templates

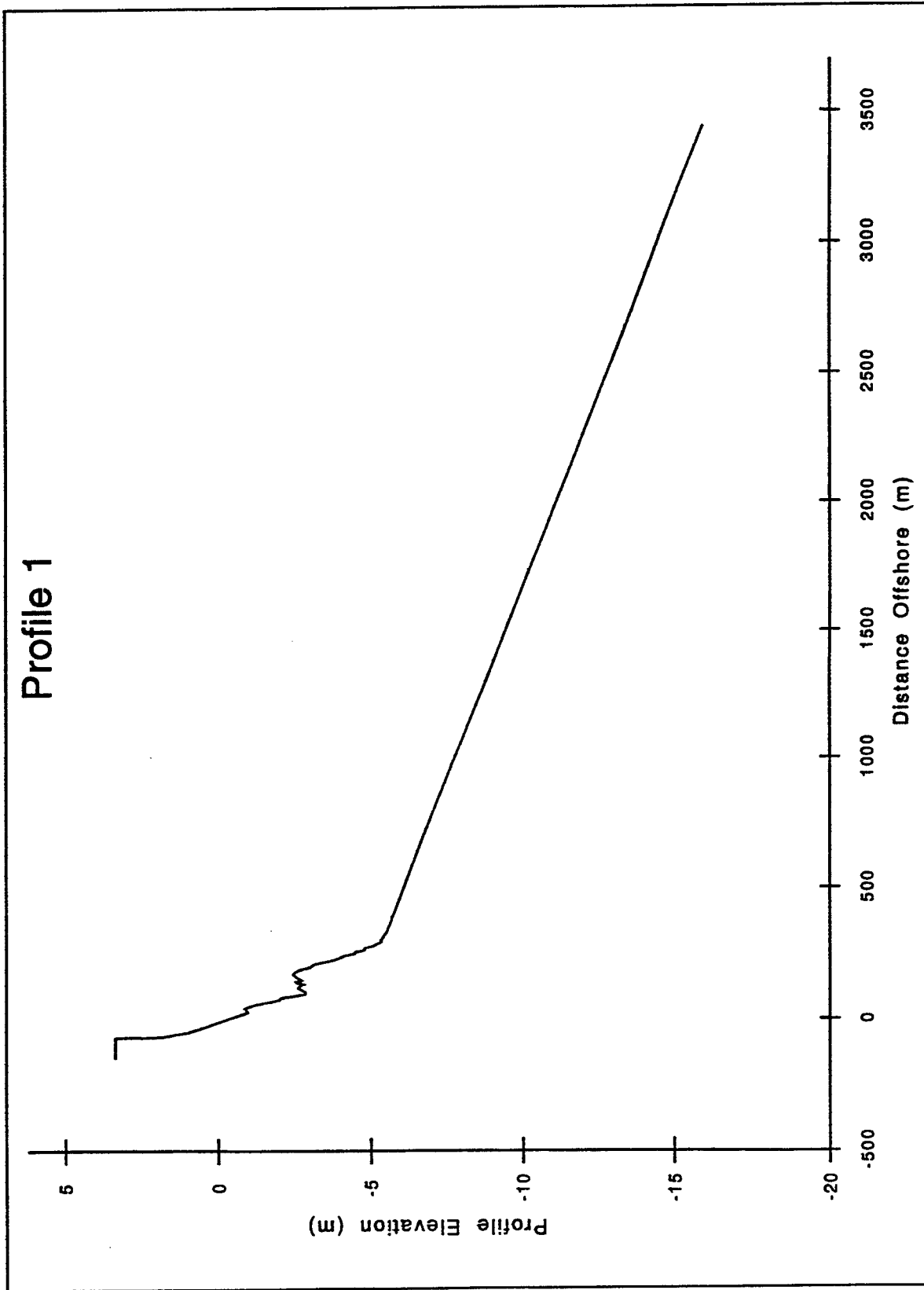
Several nearshore profile templates were developed for use in numerical model simulations. Simulation results were used to assess protection levels afforded by various berm configurations.

In the absence of detailed profile data, Profile 1 (Figure 3) was synthesized to generically represent the nonarmored existing conditions along the St. Johns County Beach. The nearshore portion of the profile represents a combination of profile R-139 surveyed by the Jacksonville District in April 1988 and April 1984 (Figure 4). Survey profiles were adjusted to NGVD, and elevations are presented in meters. The offshore profile is a 1 on 300 slope estimated from local National Oceanic and Atmospheric Administration nautical charts. A horizontal plane extends from the highest point of the dune landward to accommodate numerical model simulation landward boundary condition requirements. In addition, an equilibrium profile (Profile 11) was generated using $y = Ax^{2/3}$ (Dean 1991) and a grain size $D_{50} = 0.16$ mm (U.S. Army Corps of Engineers 1990) (Figure 5). Comparison of Profile 11 and Profile 1 storm-related recession rates showed Profile 11 to erode at a greater rate than Profile 1. Altering the nearshore baseline condition profile significantly altered the study results. Profile 1, approximated-existing condition profile, is used as the baseline condition for relative comparison of storm-related recession and renourishment of the nearshore berms.

For initial testing, nine two-dimensional nearshore berm profile templates (Profiles 2-10) were developed for consideration. The templates included a variety of nearshore berm geometries and beach fill combinations as listed in Table 2. Nearshore berm geometric parameters are represented in Figure 6. Figure 7 displays these nearshore berm/beach fill alternatives superimposed on the approximated existing profile template (Profile 1). Refer to Appendix A for the individual profile templates. Individual nearshore berms are located in three general offshore locations (300, 600, and 1,600 m) with two crest elevations (-2.25 and -5.1 m NGVD) and varied crest widths (30, 100, and 200 m). The crests closer to the sea surface were most effective in causing wave breaking. Longevity of the nearshore berms would be enhanced by increasing the width of the berm, requiring greater time to erode the feature. In final testing, the suite of profile templates was limited to five profiles, Profiles 1, 3, 4, 8, and 10. Selection of profile templates for final testing was based on a template's potential as an effective wave attenuator. A template's wave attenuation potential was estimated by SBEACH 2.0 (Method I) simulations.

Beach fills are represented as a 25-m seaward translation of the average beach face. During initial testing, profiles with beach fills exhibited greater erosion than the profiles without beach fill. A beach fill without the presence of a nearshore berm was not among the design templates. A possible reason for the increased erosion is that the prestorm beach slope is a planar slope, and not a concave slope more closely replicating the natural nearshore profile. Complexity introduced by the presence of a beach fill detracted from evaluation of the optimum nearshore berm geometry. Profiles with beach fills were not included in the suite of final test profile templates.

Nearshore berm designs are based on guidance developed through the Dredging Research Program (Pollock, Allison, and Williams 1993a,b). Although the objective of the study was to find nearshore berm designs that optimized storm protection regardless of location and size, consideration was given to



11 Figure 3. Profile 1: Without-berm profile (elevation in meters NGVD)

ST JOHNS COUNTY MEASURED PROFILES: R-139

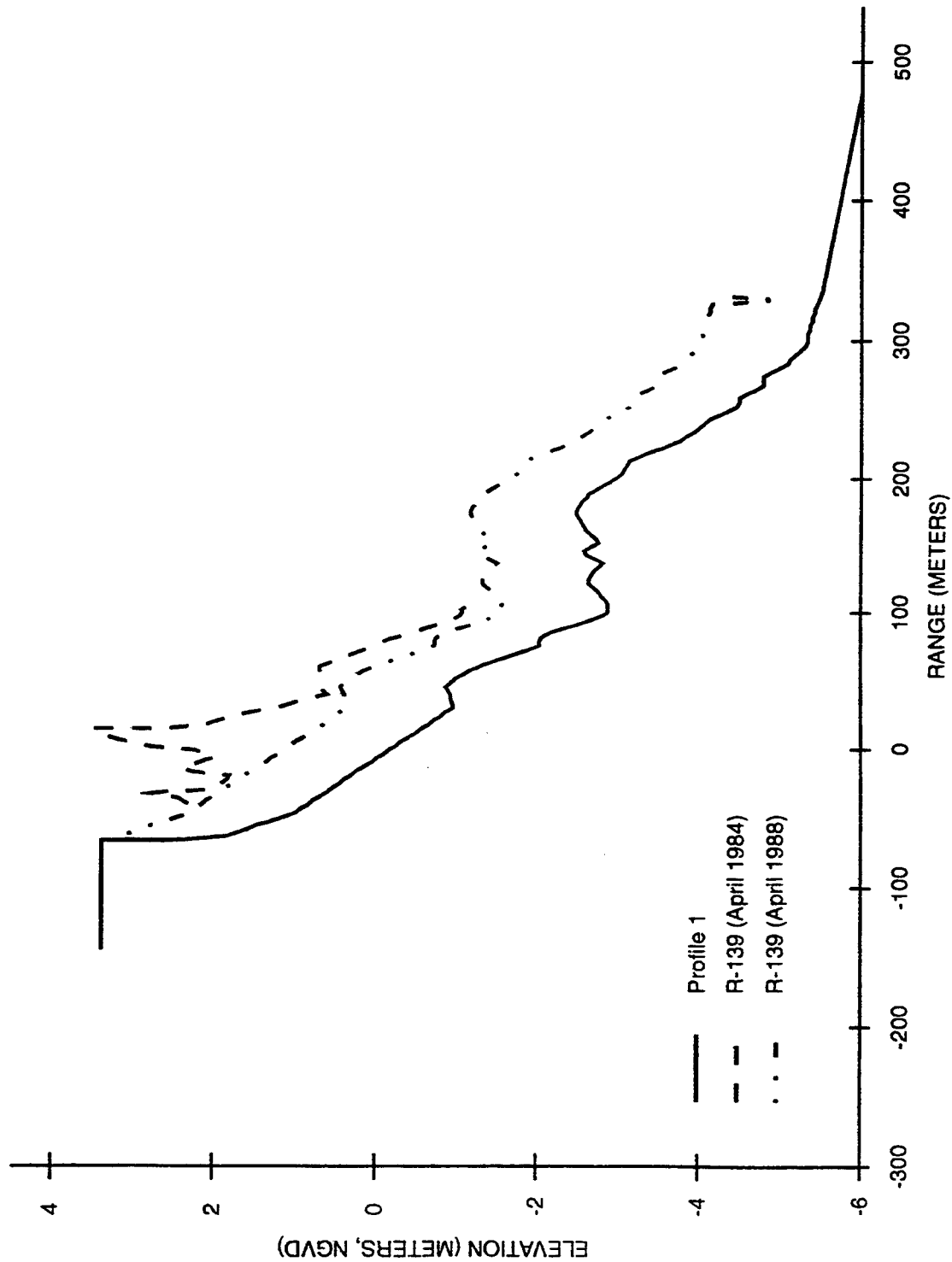
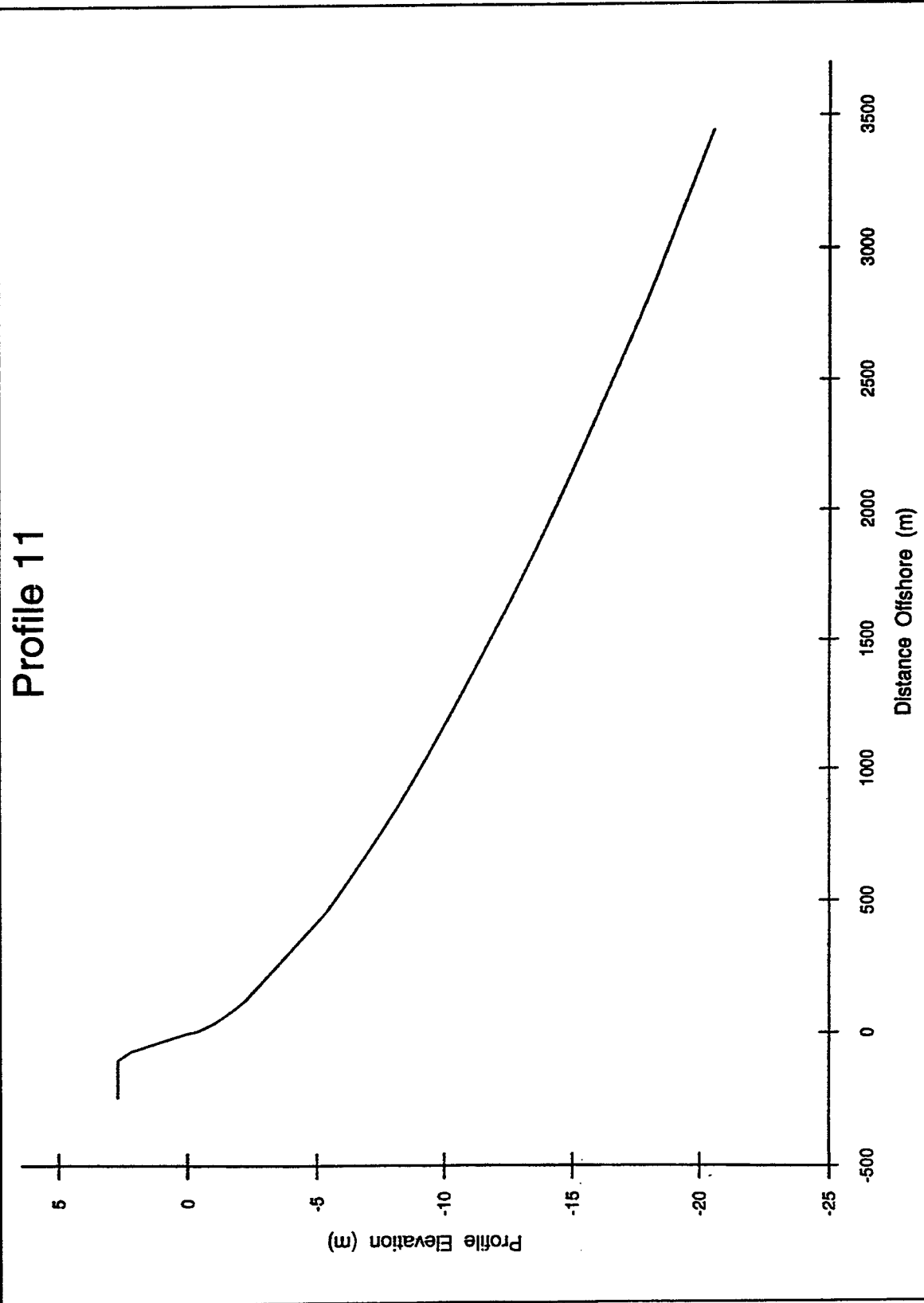


Figure 4. Measured nearshore representation of Profile 1 (elevation in meters NGVD)



13 Figure 5. Profile 11: Equilibrium profile (elevation in meters NGVD)

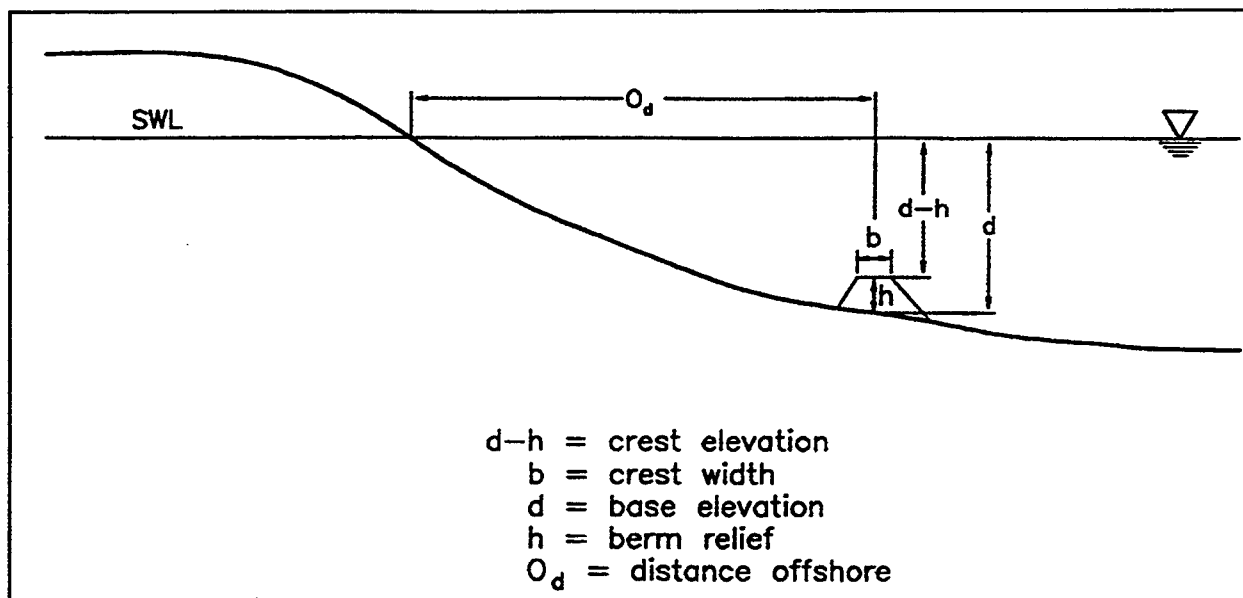


Figure 6. Definition of nearshore-berm geometric parameters

Profile No.	Crest Elevation m	Crest Width m	Base Elevation m	Berm Relief m	Distance Offshore m	Berm Volume m ³ /m	Berm Volume yd ³ /ft	Beach Fill ? Yes or No
1	Existing profile/no berm					0	0	
2	-2.25	100	-5.3	3.1	300			yes
3	-2.25	200	-5.3	3.1	300	475	189	no
4	-2.25	100	-5.3	3.1	300	465	185	no
5	-2.25	200	-5.6	3.3	350			no
6	-2.25	200	-5.6	3.3	350			yes
7	-5.1	30	-6.3	1.2	560			no
8	-5.1	100	-6.4	1.3	600	180	72	no
9	-5.1	200	-6.6	1.5	645			no
10	-5.1	100	-9.7	4.6	1600	983	392	no
11	Dean's equilibrium profile/no berm							

practicality, economics, and dredge availability. Balancing nearshore berm geometry with volume requirements and renourishment potential also influenced template design. Nearshore berm design templates were restricted to construction limitations of hopper dredges that have previously been used at this site.

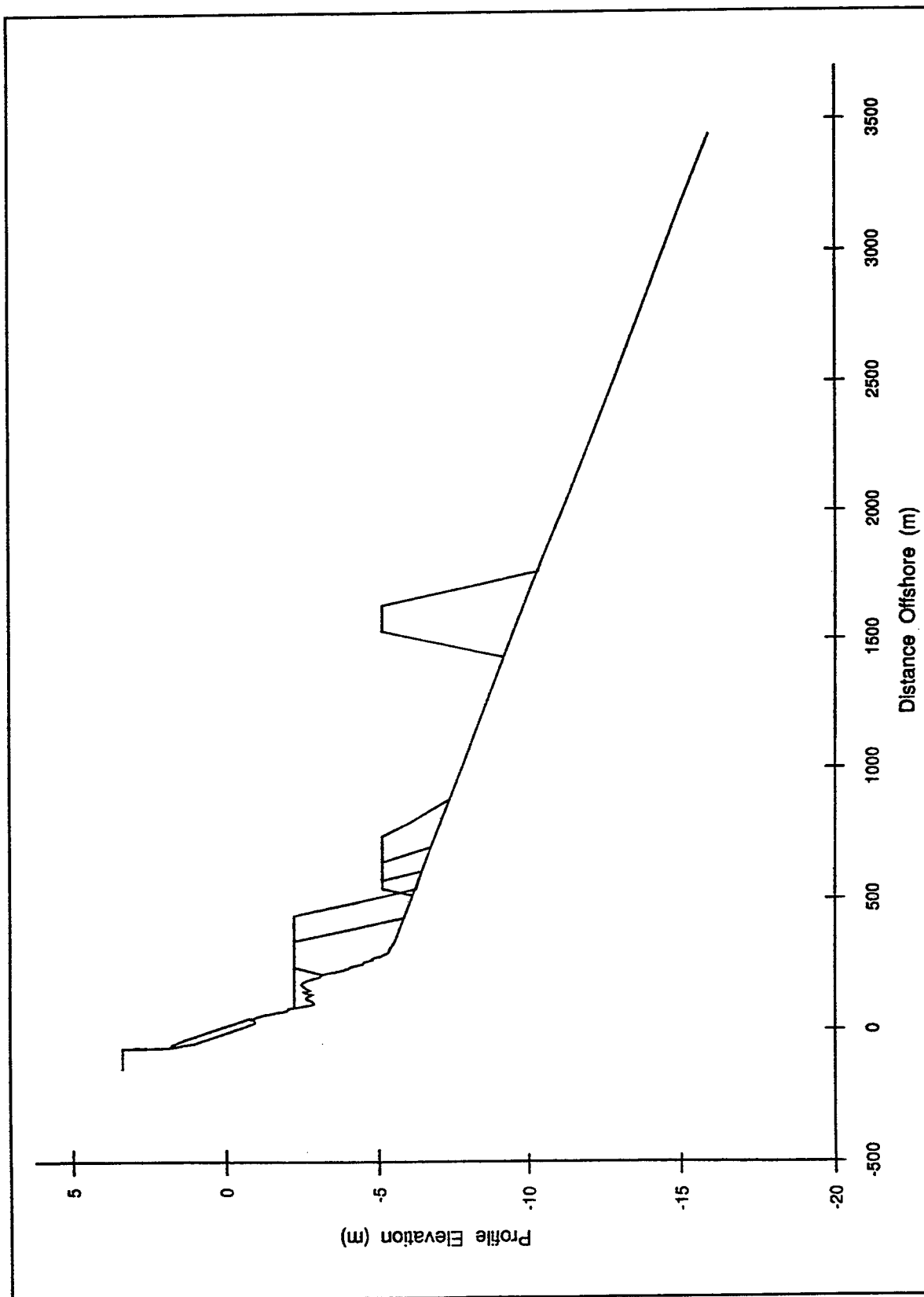


Figure 7. Nearshore-berm/beach-fill alternatives (Profiles 2-10) superimposed on Profile 1 (elevation in meters NGVD)

Hydraulic pumping is also a possible construction method, but is generally less cost effective. Hopper dredges previously used at this site include the Currituck, Sugar Island, and Northerly Island, drafting (fully loaded) 2.1, 4.6, and 5.8 m, respectively (McLellan 1990). The hopper dredges have placed nearshore berms with crest depths as shallow as -2.5 and -3 m (U.S. Army Corps of Engineers 1990).

Nearshore berm crest elevation or the depth of water over the crest may be the most significant parameter for initiating wave breaking. When waves become depth limited, increasing the crest elevation reduces the size of the wave that is allowed to propagate unbroken across the nearshore berm or reform in the lee of the berm. The depth of the crest may be limited by construction techniques, the draft of the placement vessel, navigation restrictions, and the volume of material available. For hopper dredge placement, water levels and loaded draft of the vessel generally limit the maximum elevation of the crest. Light loading of a hopper can reduce the draft limitations. Two crest elevations selected for testing were -2.25 and -5.1 m NGVD.

Nearshore berm crest width can influence attenuation of energy as waves propagate over the feature, directly influencing the longevity or renourishment period of the berm. Numerical testing suggests crest widths of approximately 45 and 100 m for the selected crest elevations of -2.25 and -5.1 m, respectively, to optimize wave attenuation.¹ Although guidance is not available to suggest crest widths to optimize stability and renourishment requirements, testing during the SUPERTANK Laboratory Investigation indicates that a wide nearshore berm maintains its structural integrity better than a narrow nearshore berm (Burke 1992). Crest widths greater than the suggested wave attenuation crest widths were included in the suite of tested templates to increase berm stability and reduce renourishment requirements.

Nearshore berm side slopes used in this study were 1 on 25. Monitoring studies of previously constructed nearshore berms indicate prototype side slopes range from 1 on 15 to 1 on 125, with most controlled construction projects yielding side slopes near 1 on 25. Shallower slopes would increase volume requirements.

Determining the optimum depth and distance offshore for locating the nearshore-berm design is generally a function of several elements. These elements include the site-specific wave climate, the nearshore slope, volume of material, bottom type, designated disposal area, required relief of the nearshore berm, stability requirements, increased potential for nearshore berm material to reach the shore, and geometric combinations cited above.

Estimating potential stability of a nearshore berm based on design templates can be addressed by two methods: depth of closure (Pollock, Allison, and Williams 1993a,b) or comparison of site-specific wave-induced bottom velocities

¹ Work expanded from Pollock, Allison, and Williams (1993).

with conditions that exist at constructed nearshore-berm sites (Hands and Allison 1991). Both methods only address whether the material is likely to move, and do not quantify the sediment movement. Using the estimated average of the highest 12 hr of waves per year $H_{12} = 4.5$ m and $T_p = 12.0$ s calculated from data obtained from the Wave Information Study (WIS) Atlantic Update Level II database for WIS location 25 (latitude = 30°N, longitude = 81°W, depth = 20 m), the inner and outer limits of the depth of closure were estimated to be 9.8 and 29 m NGVD (Brooks and Brandon 1995). All nearshore berms were located within the depth of closure and were not expected to be stable. Templates 8-10 had the deepest placement depths (-6.4 to -9.7 m NGVD) and were expected to remain more stable than the other berms.

Hydrodynamic Events

Extratropical wave events

Six extratropical storm events were selected from the WIS Atlantic Update Level II database for WIS location 25 (latitude = 30°N, longitude = 81°W, depth = 20 m). Events were selected from wave years 1989 through 1993 using the computer program "EVENT.FOR" (Brooks and Brandon 1995). Criteria for selection included a significant wave height exceeding 3.0 m and a duration exceeding 120 hr, where wave heights were at least 1 m. Information provided by WIS included time series of significant wave height (H_s), peak wave period (T_p), and mean wave direction (Θ) at 3-hr temporal resolution. For SBEACH 2.0 modeling purposes, wave information predicted by WIS for a depth of 20 m was transformed to a water depth of 15.4 m using computer program WAVETRAN.FOR.¹ Refer to Appendix B for a graphical representation of hydrodynamic event parameters.

Additional extratropical event information was provided by the Jacksonville District for the November 1984 storm. Time series of H_s and T_p were recorded at a non-directional wave gauge located 35 km south of St. Augustine Beach and 39 km offshore in a water depth of 9.14 m. For SBEACH 2.0 modeling purposes, a shore-normal mean wave direction was assumed for the November 1984 storm (see Appendix B).

Tropical wave events

Measured and hindcasted wave information was not available for tropical events occurring in the vicinity of the project area. Three tropical events were synthesized to represent extreme environmental conditions for SBEACH 2.0

¹ Personal Communication, 1994, R. Wise, U.S. Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, Vicksburg, MS.

modeling purposes. Maximum wave height and wave period values were fit to a Gaussian distribution using the following equation.¹

$$D = M e^{\left(\frac{-9.21}{24L}\right)^2} \left[3I - \left(\frac{24L}{2}\right)^2 \right] \quad (2)$$

where

D = wave height distribution

M = maximum value of wave parameter

L = duration of event in days

I = integer counter

Deep-water wave-height values were transformed to a water depth of 15.24 m using Fortran Code WAVETRAN.FOR. Maximum wave-period values were arbitrarily assigned, and a shore-normal direction of wave approach was assumed (see Appendix B).

Water level

For each extratropical wave event predicted by WIS, corresponding water level information was calculated for the St. Augustine Beach location. Water level information was obtained from the Mayport/Fernandina Beach Automated Real-Time Tidal Elevation System (ARTTES) gauge, located 93 km north of St. Augustine Beach, (latitude = 30°40.3N, longitude = 81°28.0W).² Water elevation data (measured in meters MLW) were converted to meters NGVD for the Fernandina Beach location using the equation:

$$STAGE_{NGVD} = STAGE_{MLW} - (2.7ft/(3.281ft/m)) \quad (3)$$

A mean value of water level was calculated for each event. Assuming that storm surge remained constant for the duration of the event, mean water level values were subtracted from the water level time series to remove the surge from the record:

$$TIDE = STAGE_{NGVD} - \text{mean}(STAGE_{NGVD}) \quad (4)$$

According to tables of tidal differences, there is a multiplicative conversion factor for tidal amplitudes of 0.85 between Fernandina Beach and St. Augustine Beach.

² Personal Communication, 1994, N. Scheffner, U.S. Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, Vicksburg, MS.

² Personal Communication, 1994, P.T. Puckette and W. Thompson, U.S. Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, Vicksburg, MS.

Fernandina Beach tide values corrected for surge were multiplied by the tidal amplitude correction factor:

$$TIDE(m) = TIDE * 0.85 \quad (5)$$

Mean water level (storm surge) values were added back to the tidal elevation records (see Appendix B).

For the November 1984 storm, water level information (measured in feet MLW) was provided by the Jacksonville District for a gauge located 35 km south of St. Augustine Beach (29° 40' 3" N and 81° 12' 17" W). The gauge was assumed to be located seaward of St. Augustine Beach. Water level data were converted from feet MLW to meters NGVD:

$$STAGE_{NGVD} = STAGE(ft)/(3.281 \text{ ft/m}) - 2.15 \text{ ft}/(3.281 \text{ ft/m}) \quad (6)$$

(see Appendix B). For extreme wave events, water level information was synthesized by fitting an arbitrarily chosen maximum stage value to a Gaussian distribution as described above.

Results of SBEACH 2.0 Methods I, II, and III

For the purpose of this project, the Jacksonville District has defined storm-induced recession based on the District benefit analysis methodology. Results of this analysis were used by the District as input to a coastal storm damage model (SDM). Recession (R_o), as simulated by application of individual storm events to SBEACH 2.0 and given profile templates, is defined as the distance between the prestorm simulation reference shoreline (MHW) and the furthest landward extent of the storm erosion envelope (see Figure 8). Additional criteria for evaluating recession can be established. For example, recession criteria may be arbitrarily defined as the distance between prestorm simulation (MWH) and the landward extent of the erosion envelope where 0.25 and 0.5 m of vertical change were observed.

As stated, SBEACH's apparent inability to faithfully represent reduced recession due to the wave-attenuating effect of nearshore berms stems from the method in which the model calculates wave runup on the beach face. This method is the prescribed procedure in the model documentation and is termed Method I. Lack of benefit from wave-attenuation effects when applying Method I can be observed in Table 3. Table 3 and Figures 9-10 present R_o values resulting from application of the prescribed SBEACH 2.0 method and the two modified SBEACH 2.0 methods. Referring to Method I results in Table 3 and Figure 9, for any given storm event, design berm template profiles (Profiles 3, 4, 8, and 10) afford little to no reduction in R_o when compared with the without-berm profile (Profile 1). For example, when considering the storm event September 1989, R_o values are reduced from 99 m (Profile 1) to 97, 91, 91, and 93 m for Profiles 3, 4, 8, and 10, respectively. In the case of that storm event, differences in R_o values

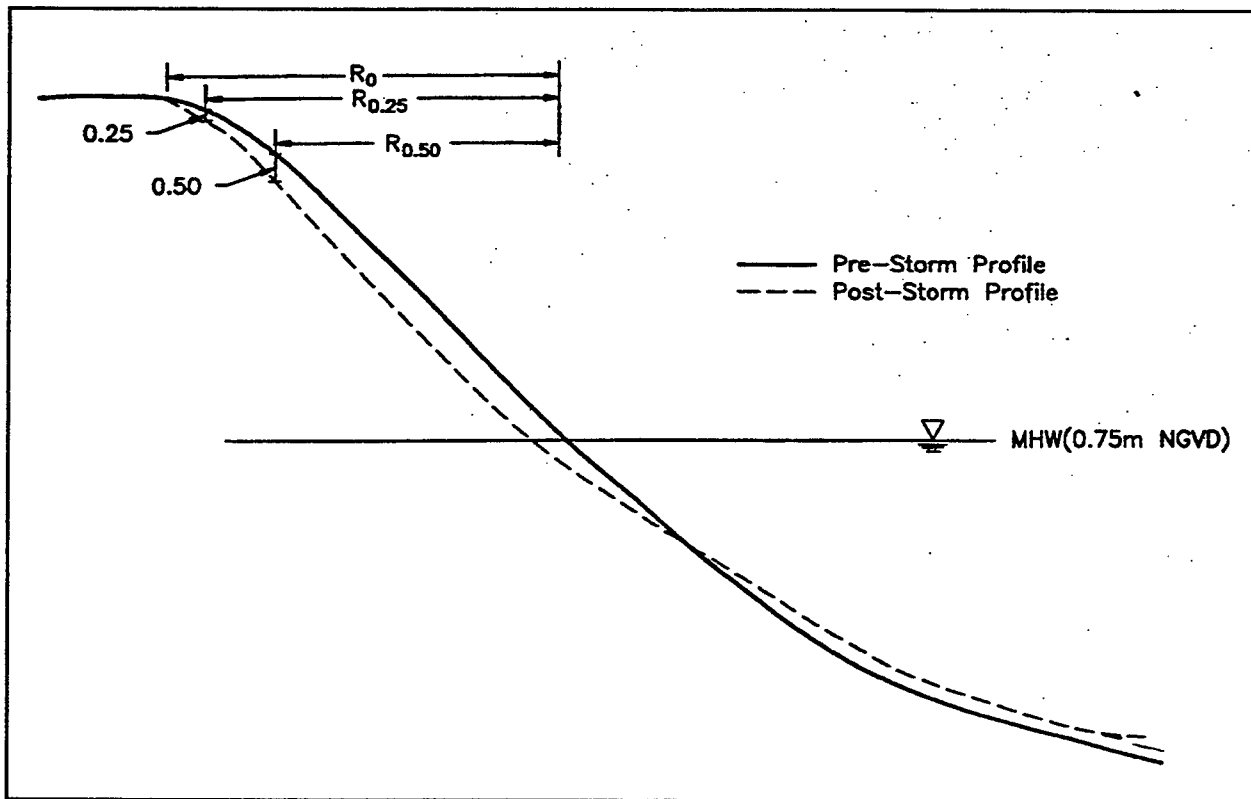


Figure 8. Landward extent of erosion envelope: R_o defined

are negligible when considering benefits gained in storm damage protection from a nearshore berm.

Method II and Method III recalculate wave runup and setup associated with using a wave of lesser height reforming in the lee of the nearshore berm. SBEACH 2.0 simulations were run in Method II and Method III modes for the without-berm profile and each nearshore berm configuration investigated. Observed results from applying Methods II and III (Table 3) indicate a reduction in R_o affected by the presence of the nearshore berm for each storm event (Figures 10 and 11). Example SBEACH 2.0 results for storm event January 1988 applied to Profile 8 for Methods I, II, and III are presented in Figure 12. When compared with Method I, R_o values were reduced from 63 to 47 and 49 m by application of Methods II and III, respectively. In general, when Method II and Method III are compared with each other, differences in R_o values are negligible. However, Method III tends to generate slightly greater values of R_o than Method II. Larger R_o values calculated by Method III can be attributed to the fact that Method III saves water elevation information in the lee of the nearshore berm. By saving water elevation information in the lee of the berm, Method III includes the effect of wave setup on total water elevation to serve as a contribution to wave runup on the beach face. By including wave setup in the lee of the berm, water levels saved are elevated when compared with water elevations at the same location when applying Method II. The overall observed result is an increase in wave runup over Method II, when applying Method III.

Table 3 Recession (R_o) Values: SBEACH Methods I, II, and III																
Events	Profile 1 Methods			Profile 3 Methods			Profile 4 Methods			Profile 8 Methods			Profile 10 Methods			
	I	II	III	I	II	III	I	II	III	I	II	III	I	II	III	
NOV84	29	29	29	29	13	27	27	27	27	29	29	29	27	29	29	
JAN88	73	73	73	65	27	37	63	29	37	63	47	49	63	51	53	
JAN89	69	69	69	63	25	35	61	29	37	61	31	45	63	43	27	
SEPT89	99	99	99	97	31	49	91	29	49	91	57	61	93	59	67	
OCT90a	57	57	57	71	29	39	49	29	39	49	47	35	51	45	33	
OCT90b	65	65	65	59	25	29	55	27	29	55	29	45	55	29	47	
HALLOW91	74	74	74	77	55	55	63	61	51	63	77	67	65	81	67	
TR25	145	145	145	115	55	89	125	61	91	121	77	93	121	81	137	
TR50	171	171	171	161	65	117	165	77	119	143	109	131	121	119	123	
TR100	175	175	175	131	99	119	141	111	119	133	133	123	91	151	89	

SBEACH: Method 1

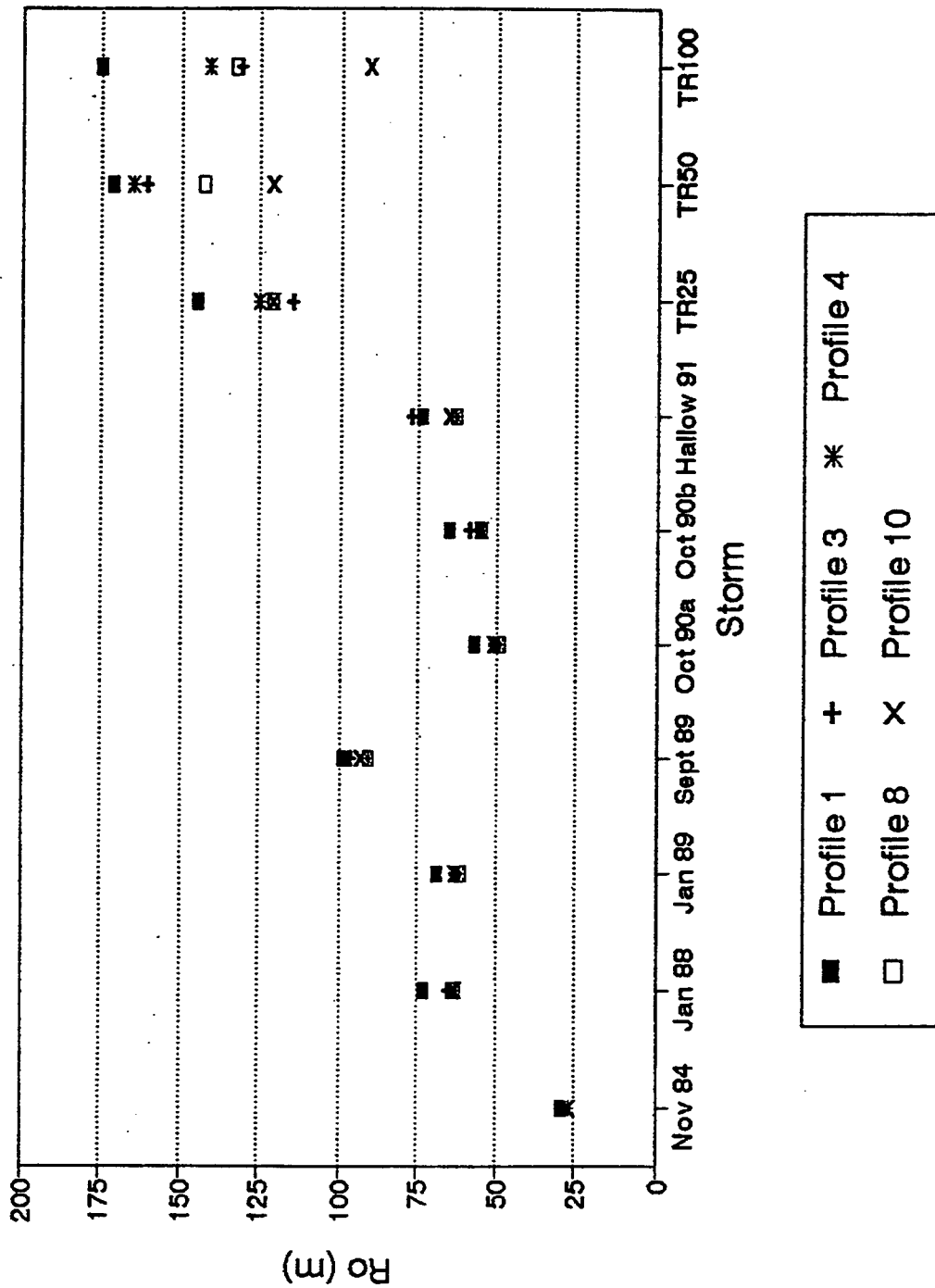


Figure 9. Recession (R_0) values: SBEACH 2.0 Method I

SBEACH: Method 2

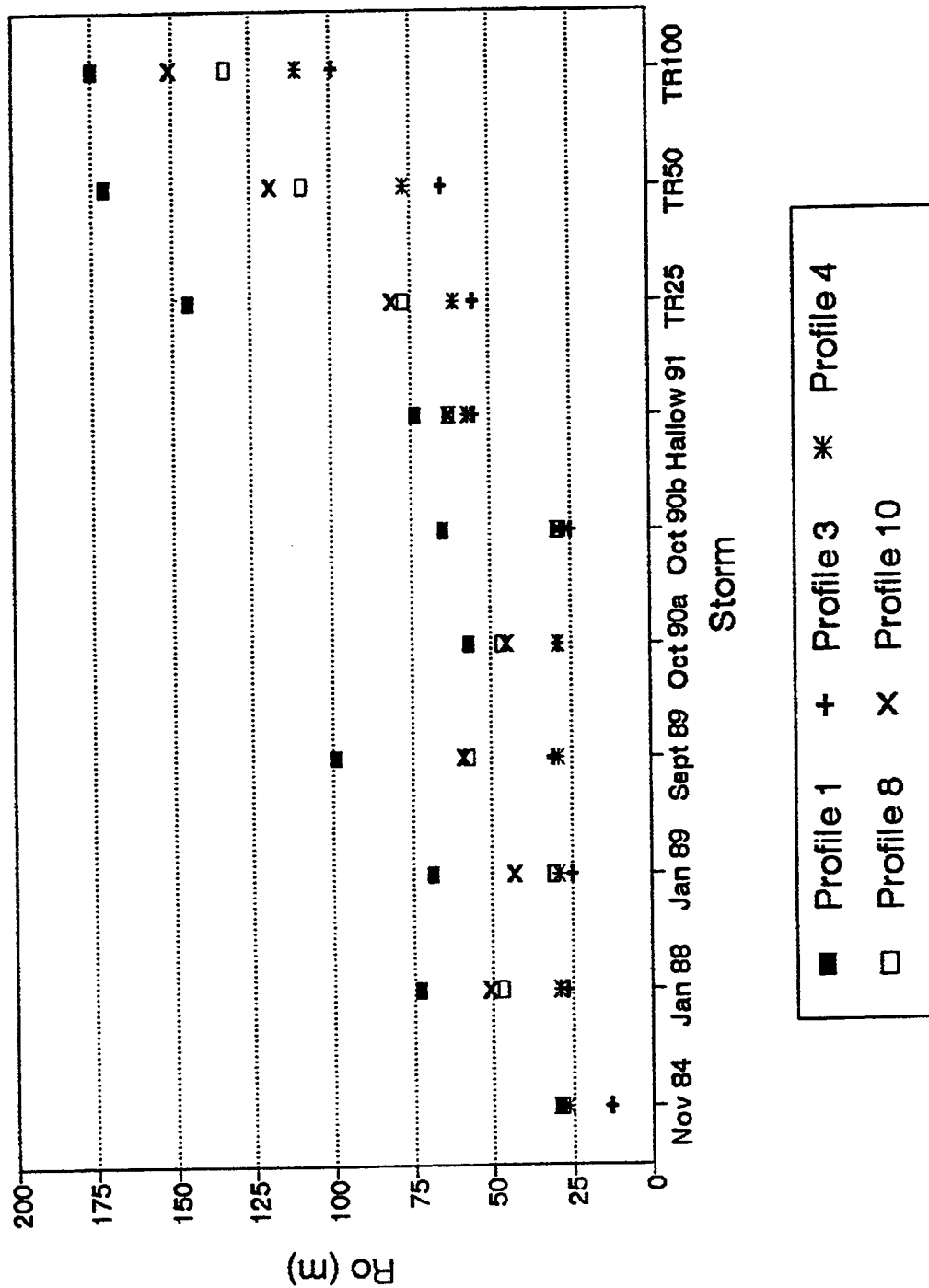


Figure 10. Recession (R_0) values: SBEACH 2.0 Method II

SBEACH: Method 3

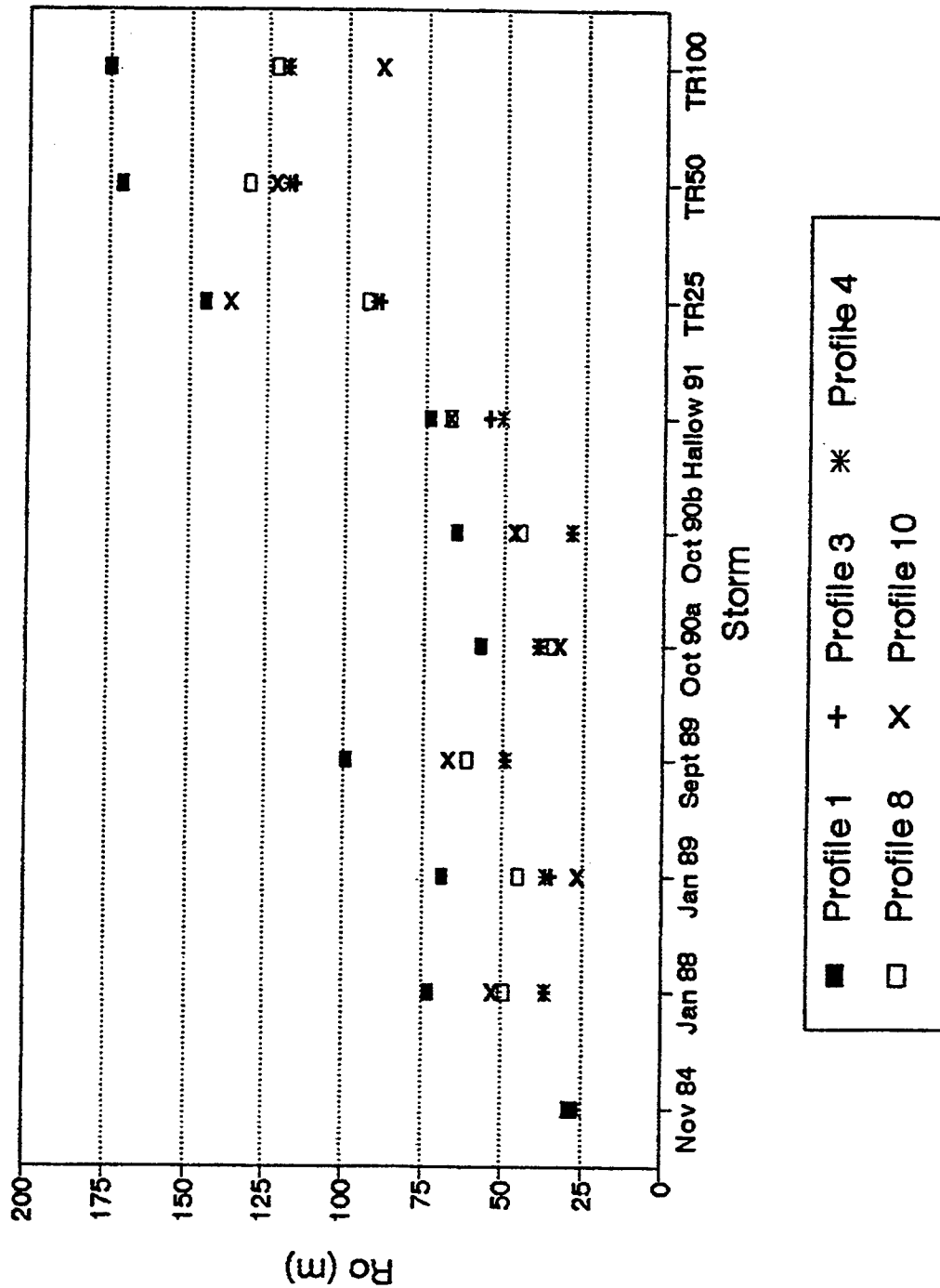


Figure 11. Recession (R_d) values: SBEACH 2.0 Method III

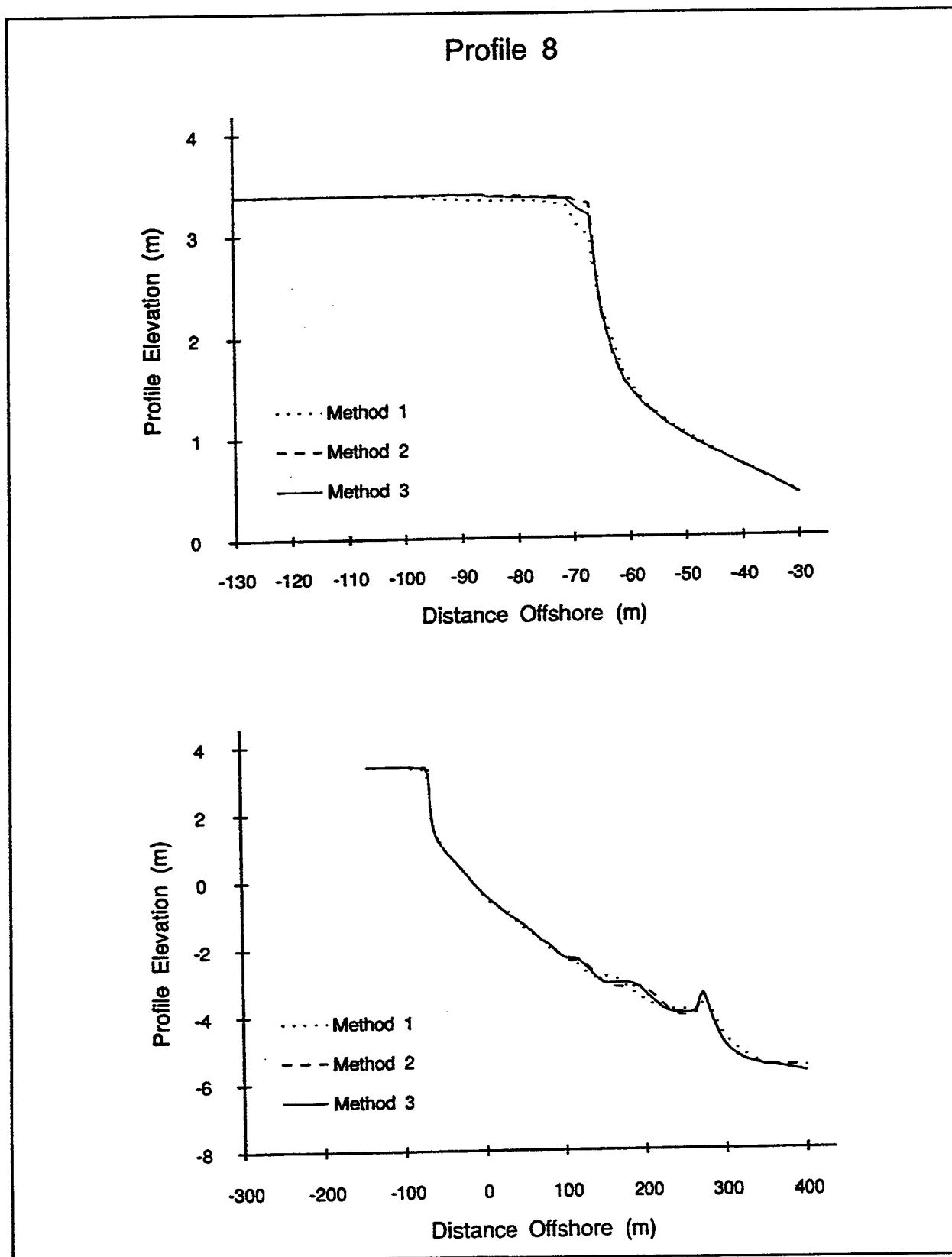


Figure 12. Storm JAN88 applied to Profile 8 example results: SBEACH 2.0 Methods I, II, and III (elevation in meters NGVD)

3 Storm-Related Renourishment

Nearshore berm renourishment requirements were predicted using two numerical models, SBEACH 2.0 and LTFATE. Two-dimensional volumetric comparisons were made between prestorm and poststorm predicted cross-shore berm profiles for each of the models, assuming a 1-m longitudinal berm section.

Storm Response of Nearshore Berm: SBEACH 2.0

A suite of storm events (Appendix B) was applied to four nearshore berm design profiles (Profiles 3, 4, 8, and 10) using the SBEACH 2.0 Method I approach. A storm-associated renourishment volume for each nearshore berm template was estimated following application of each storm event. Renourishment volumes were estimated based on geometric comparisons between prestorm and poststorm profiles. Table 4 presents storm-associated renourishment volumes based on SBEACH 2.0 results. Examples of nearshore berm profile responses for a given storm (HALLOW91) are presented in Figure 13. For storm event HALLOW91, associated nearshore berm renourishment volumes are 81, 63, 0, and 0 m³/m for Profiles 3, 4, 8, and 10, respectively. In general, the two berms positioned further offshore (600 and 1,600 m) required less renourishment maintenance on an event-related basis. Comprehensive nearshore berm profile responses for the suite of 10 storms are presented in Appendix D.

Storm Response of Nearshore Berm: LTFATE

LTFATE is a site-evaluation tool for estimating dispersion characteristics and subsequent stability for dredged material placed in open water over periods of

Table 4
Storm-Event-Associated Renourishment Volumes: SBEACH 2.0
Method I (m³/m)

Event	Profile 3	Profile 4	Profile 8	Profile 10
NOV84	67	63	22	28
JAN88	66	75	15	69
JAN89	58	56	0	31
SEPT89	75	98	22	113
OCT90a	69	88	0	19
OCT90b	61	56	0	0
HALLOW91	81	63	0	0
TR25	17	18	0	0
TR50	23	21	0	19
TR100	22	15	0	19

time ranging from days (storm events) to years (ambient conditions). For purposes of this study, LTFATE was applied to simulate storm-induced nearshore berm erosion. LTFATE was used to calculate storm-related renourishment volume for six storm events and two nearshore berm profiles. Storm events were selected to simulate low-water (set-down) conditions above the berm crests. A set-down criterion was chosen to enhance erosive effects of storm events on the nearshore berm design templates during numerical simulations. To ensure numerical stability, Profiles 8 and 10 were selected for testing, as they are located offshore, outside the surf zone. Numerical evaluation of Profiles 3 and 4 using LTFATE may have resulted in numerical instabilities, as the nearshore berm may be located in the saturated surf zone region during a storm event. Table 5 presents storm-associated renourishment volumes as calculated by LTFATE. Examples of nearshore berm profile responses for a given storm (HALLOW91) are presented in Figure 14. Comprehensive nearshore berm profile responses for the suite of six storms are presented in Appendix E. In addition, Appendix E presents a detailed summary of the LTFATE nearshore berm profile response modeling effort.

Storm Response of Nearshore Berm: Observations

A subjective two-dimensional criterion for renourishing the nearshore berms would be when the berm crest has eroded to less than 80 percent of the minimum design crest width at an elevation of 0.5 m below the design crest elevation (McLellan 1990). SBEACH 2.0 modeling results predicted Profiles 8 and 10 to require very little if any renourishment related to the suite of test storm conditions. Based on the small volume lost and the sustained poststorm

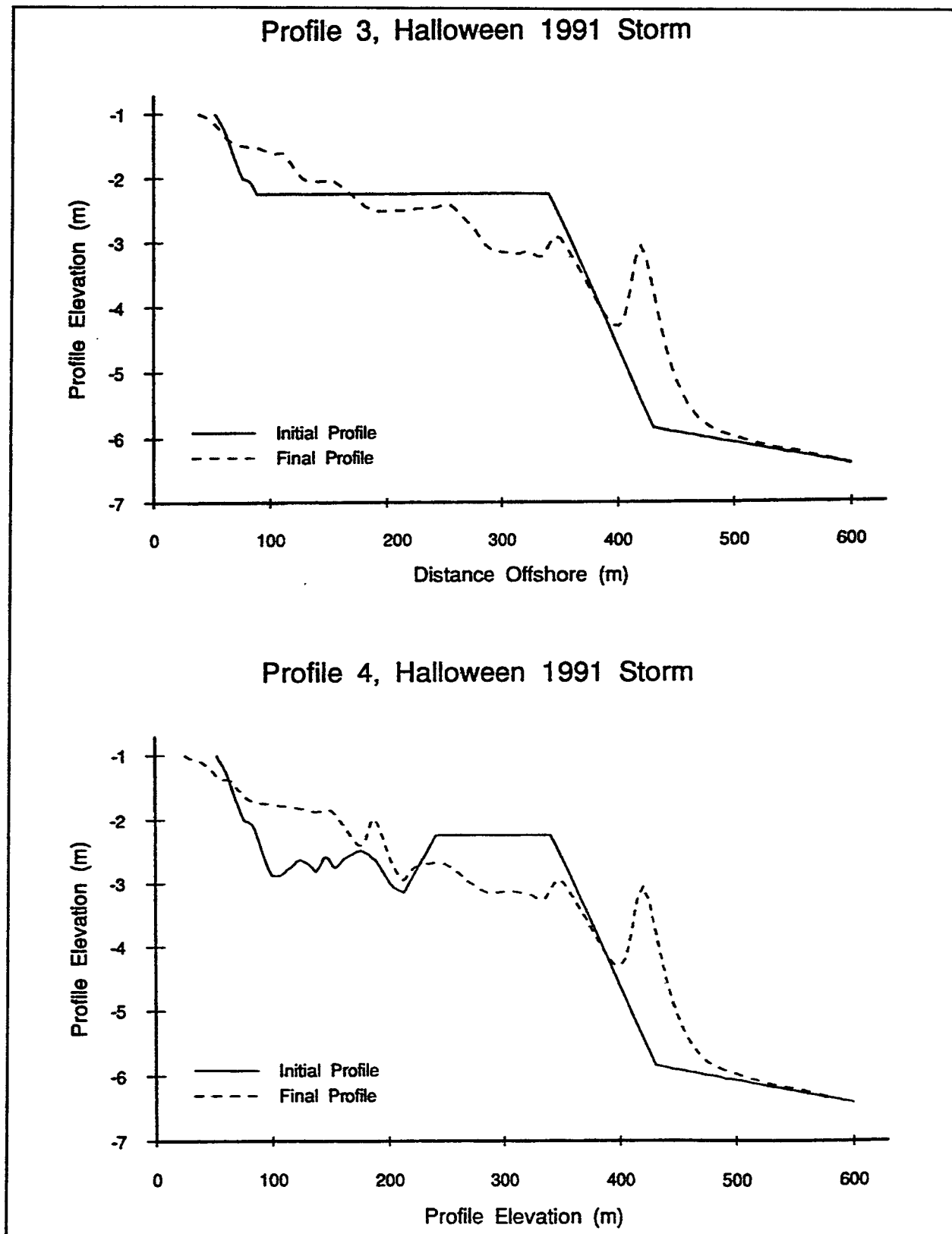


Figure 13. Nearshore-berm profile response to storm event HALLOW91: SBEACH 2.0 (elevation in meters NGVD) (Continued)

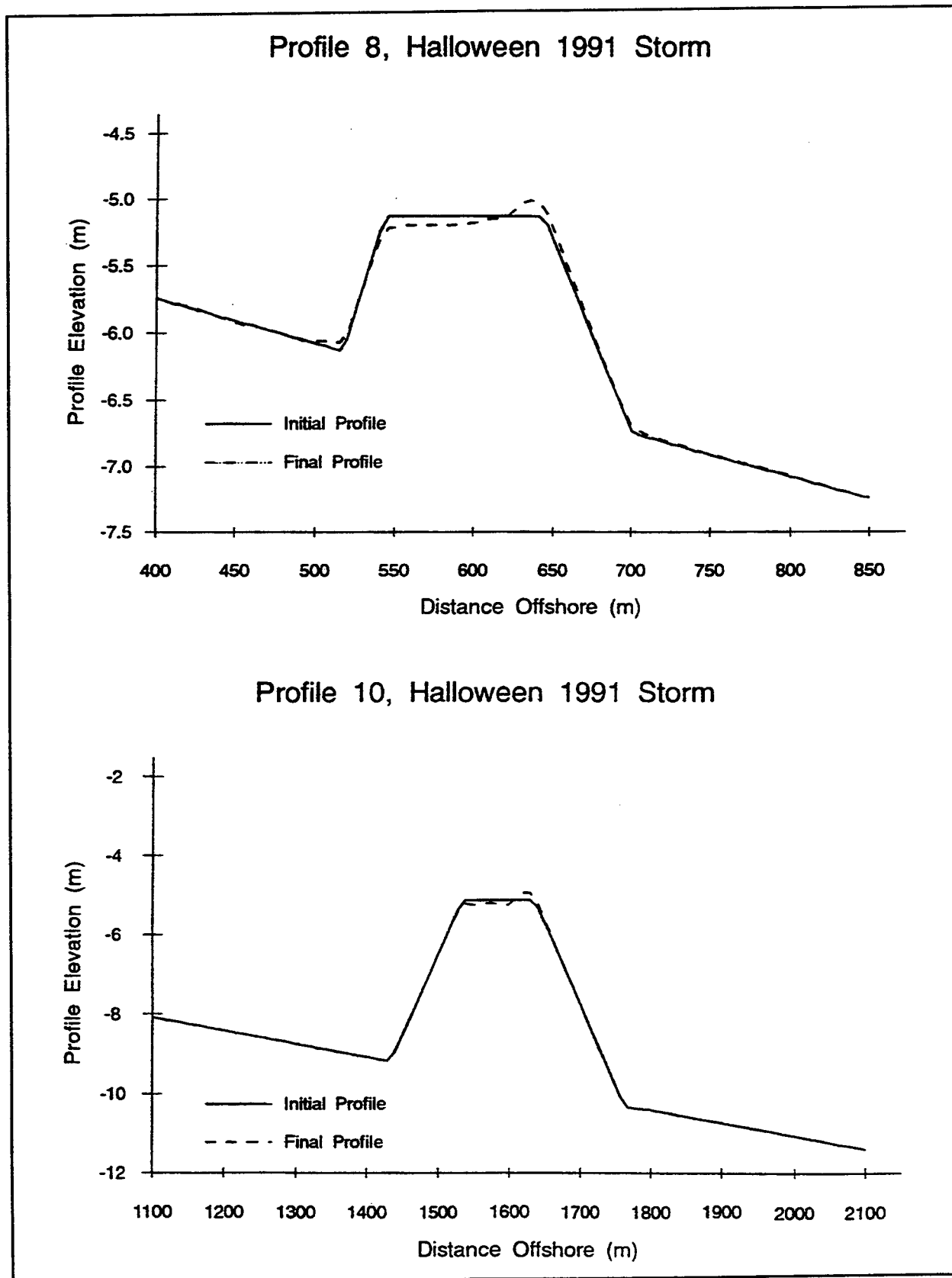


Figure 13. (Concluded)

Table 5 Storm-Event-Associated Renourishment Volumes): LTFATE (m³/m)		
Event	Profile 8	Profile 10
JAN88	4.3	46.4
JAN89	6.0	61.7
SEPT89	4.0	46.4
OCT90a	1.76	18.3
OCT90b	0.0	1.3
HALLOW91	5.0	45.7

geometry, the nearshore berms' wave-attenuating properties were not deteriorated, and no significant changes in recession rates were predicted for poststorm Profiles 8 and 10. LTFATE modeling results for Profiles 8 and 10 confirm the storm responses of the nearshore berms observed in the SBEACH 2.0 modeling effort, i.e., the berms were minimally affected by the storms.

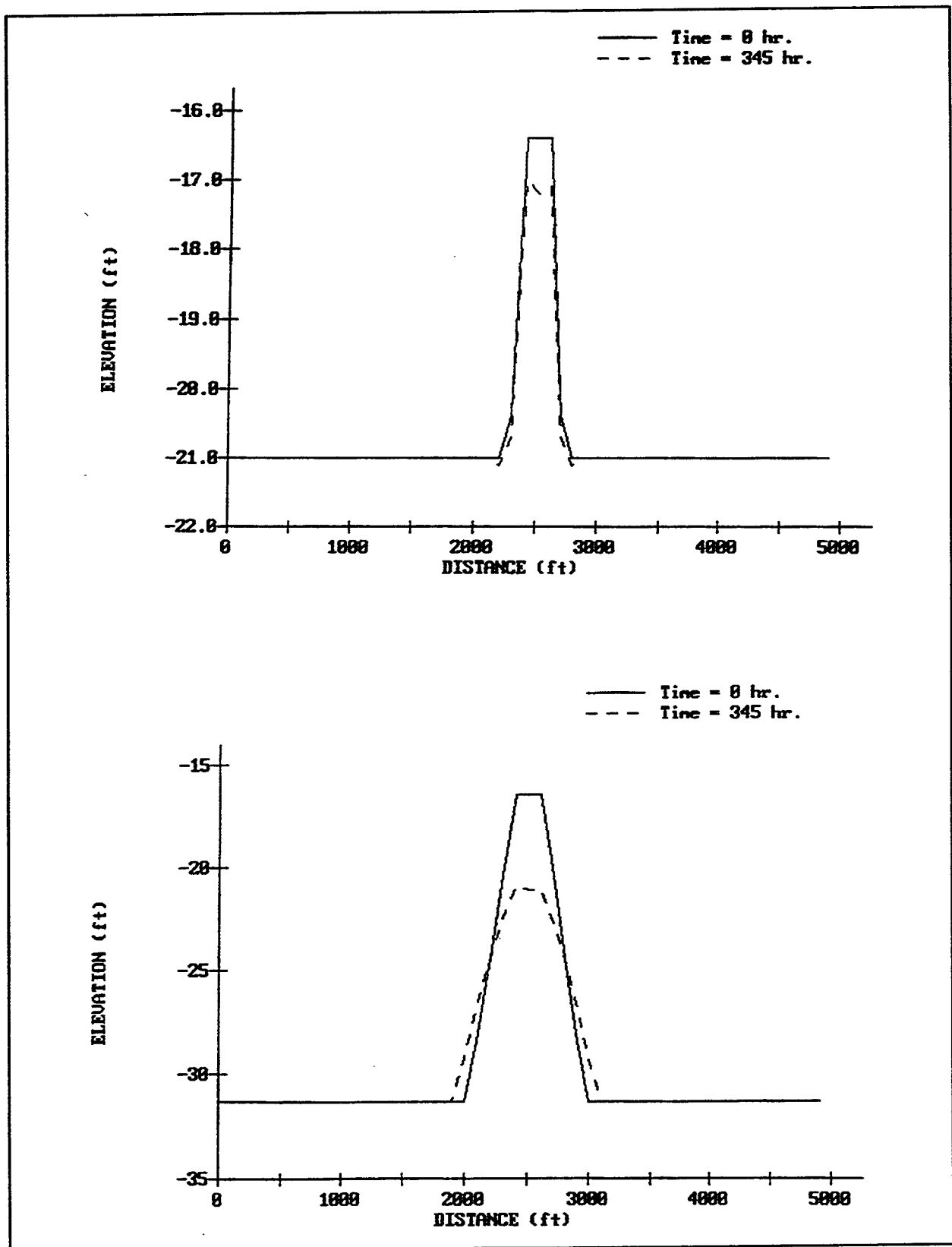


Figure 14. Nearshore-berm profile response to storm event HALLOW91: LTFATE (Profile 8 (top), Profile 10 (bottom), elevation in meters NGVD)

4 Summary and Conclusions

Summary

This report presents results of a study investigating use of numerical techniques to predict shoreline response as influenced by the presence of nearshore berm features located on the beach profile. Specifically, stable and unstable nearshore berms are investigated as shore protection features for mitigation of coastal storm damage to upland properties and to reduce shoreline erosion. Nearshore berm placement on an existing beach profile has the potential to initiate offshore wave breaking, decreasing wave energy at the shore and thereby reducing storm impacts. Estimates of annual beach recession rates, project renourishment rates, and storm-related recession distance are required for economic evaluation of a nearshore berm project. This information is site-specific and varies for different shoreline protection options. The methods applied in this study were initiated under the Dredging Research Program and the Dredging Operations Technical Support Program.

Predictions were made for a generalized existing profile and several engineered design templates for nearshore berm alternatives superimposed on the existing profile. For comparison purposes, the study used climatic information and numerical models to predict storm-related recession and storm-related renourishment requirements for the nearshore berm profiles. The objective of the study was to perform a comparison of shoreline response as influenced by varying nearshore berm design configurations, and explore the potential for storm damage mitigation to the beach profile using an engineered berm feature.

Results from this study indicate nearshore berms could provide measurable shoreline protection from storm events. By initiating wave breaking on the berm, a less energetic wave re-forms in the lee, resulting in lower wave runup at the shore and reduced coastal erosion. When compared to the without-berm profile, shoreline recession estimates decreased by tens of meters per event as the result of placing a nearshore berm on the profile. If the coastal manager has confidence in the numerical techniques applied, recession estimates may be used as input for event frequency correlation and economic evaluation.

St. Johns County, Florida, Shore Protection Project

In support of the St. Johns County, Florida, Shore Protection Project under the jurisdiction of U.S. Army Engineer District, Jacksonville, the U.S. Army Engineer Waterways Experiment Station applied methods for addressing storm-related recession and renourishment. The portion of shoreline under investigation is located in St. Johns County, Florida, and is centered on the St. Augustine Beach public pier (U.S. Army Corps of Engineers 1990). The project area extends along 4,024 m of open-coast shoreline and is located approximately 4,421 m south of St. Augustine Inlet. Historically, this reach of shoreline has been unstable, and in recent years has experienced significant coastal erosion. Coastal erosion is encroaching on public and private elements of community infrastructure.

Predicting storm-related recession

The Storm Induced Beach Change Model, Version 2.0 (SBEACH 2.0) was used to estimate storm-related recession of the generalized St. Johns County beach profile and recession of the generalized profile with nearshore berm design templates superimposed. Model forcing functions included a suite of measured, hindcasted and synthesized hydrodynamic events, which were individually applied.

Profile templates. Development of the profile templates used in this study is discussed in Chapter 2 and the templates are presented in Appendix A of this report. The existing condition profile template used is a generalized beach profile derived from a combination of measurements, and represents the St. Johns County unstructured (i.e., non-armored) shoreline (Figure 3). The offshore portion of the profile has a 1 on 300 slope and was estimated from local nautical charts. A horizontal plane extends landward from the highest dune elevation, and was included to accommodate landward boundary condition requirements of the model.

For initial consideration, nine two-dimensional nearshore berm profile templates were developed and superimposed on the generalized profile. Individual berms were located in three general offshore locations (300, 600, and 1,600 m measured from the dune crest) with two berm crest elevations (-2.1 and -5.1 m NGVD) and varied crest widths (30, 100, and 200 m). For final testing, the suite of berm templates was limited to four configurations, Profiles 3, 4, 8, and 10 (Appendix A). Template designs are based on existing design guidance for nearshore berms developed through the U.S. Army Engineer Dredging Research Program (McLellan 1990; McLellan et al. 1990; Burke and Allison 1992; Pollock et al. 1993a).

The berm templates evaluated are sensitive to practicality, economics, and dredge availability. Berm crest elevation, or the depth of water over the crest, may be the most significant parameter for initiating wave breaking. The depth of

the crest may be limited by construction techniques, the draft of the placement vessel, navigation restrictions, and volume of available material. Therefore, the berm templates were restricted to construction limitations of hopper dredges that have been previously used at this site (U.S. Army Corps of Engineers 1990; McLellan 1990). Hydraulic pumping is also a viable construction method, but is generally less cost-effective.

Nearshore berm crest width can influence attenuation of energy, as waves propagate over the feature, affecting the longevity or renourishment period of the berm. Preliminary numerical testing suggested that crest widths of approximately 45 and 100 m be applied to enhance offshore wave breaking for selected crest elevations of -2.25 and -5.1 m, respectively. Crest widths equal to or greater than the suggested wave attenuation crest widths were included in the suite of evaluated templates to increase berm stability and reduce renourishment requirements. Additional numerical simulations suggested crest widths to optimize stability and renourishment requirements. These simulations were supported by SUPERTANK Laboratory Investigation results, which indicated that a wide nearshore berm maintains its structural integrity better than a narrow nearshore berm (Burke 1992). Renourishment suggestions are partially based on numerically generated post-storm profiles meeting the minimum crest width and elevation criteria to optimize wave attenuation.

Hydrodynamic events. Due to lack of available measured wave data for the project area, six hindcasted extratropical storm events were selected for use as model forcing functions from the WIS Atlantic Update Level II database (latitude = 30°N, longitude = 81°W, depth = 20 m) (Brooks and Brandon 1995). Events were selected from wave years 1989 through 1993 and included maximum significant wave height values exceeding 3 m, and durations in excess of 120 hr of minimum wave height values of 1 m. Information provided by WIS included time series records of significant wave height, peak wave period, and mean wave direction at 3 hr temporal resolution. WIS wave parameters were transformed to a water depth of 15.4 m for input to SBEACH 2.0 at the offshore computational boundary.

For each event hindcasted by WIS, corresponding water level information was estimated. Water level information recorded 93 km north of the project site at Mayport/Fernandina Beach was numerically transformed to the St. Augustine Beach location for model input. Refer to Chapter 2 and Appendix B for further discussion and presentation of the hydrodynamic events used in this study.

SBEACH 2.0 simulations. For the purposes of this study, storm-induced recession was defined based on a parameter used for economic benefit analysis. The recession parameter R_o is defined as the variation in distance estimated between the prestorm reference shoreline and the furthest landward extent of the storm erosion envelope on the profile. The mean-high-water shoreline is the designated reference shoreline used for the determination of R_o in this study (Figure 8).

Applying SBEACH 2.0 under the normal prescribed procedures (Rosati et al. 1993) resulted in little or no benefit from nearshore berms placed on the profile. Recession rates were insensitive to the effects of offshore features in breaking waves and the re-forming of reduced wave heights incident on the shoreline. Refer to simulated R_o values for the prescribed SBEACH 2.0 method presented in Figure 9. When considering benefits gained from storm protection afforded by a nearshore berm, reductions in R_o values for the October 1990 storm are negligible. Differences in R_o values ranged from 6 m to 10 m, when compared to the without-berm case for the storm. Lack of wave attenuating benefit predicted by the model for nearshore berm conditions results from the process in which the model calculates wave runup on the beach face. Wave information initially entered into the model at the computational boundary seaward of the berm is used to calculate wave runup and setup at the beach. The model does not recalculate runup and setup associated with the lesser height of the re-formed wave in the lee of the berm.

Using the prescribed SBEACH 2.0 method to transform a wave over a nearshore berm and predict subsequent shoreline change produced results contrary to expectation. These results prompted two modified versions to the application of the model, Method II and Method III. For a given hydrodynamic event and berm template superimposed on the generalized profile, output from the prescribed SBEACH 2.0 method was used to visually select the cell location in the lee of the berm where broken waves re-formed and stabilized. For Method II, the re-formed wave information was saved at each computational time step at the selected cell location in the lee of the berm. For Method III, both the re-formed wave and water level information were saved at that cell location. The re-formed hydrodynamic data were then entered on the without-berm profile at the same cell location (in the lee of the berm) and propagated shoreward. These modifications to the prescribed method account for variations in wave runup and setup produced by waves breaking over the berm. Storm-related recession values, as predicted by modified methods, are significantly less than those predicted by the unmodified SBEACH 2.0. Recession values are presented in Chapter 2 and Appendix D.

The modified application of SBEACH 2.0 yields results which intuitively represent expected trends for profile recession in the lee of a nearshore berm. It should be noted that the modified model results are unverified, except for limited comparisons with data from the SUPERTANK Laboratory Investigation by the authors and a similar comparison by Smith (1994) using the NMLong model for wave transformation over the berm. However, these modified model-measurement comparisons substantiate the use of the modified SBEACH 2.0 methods to better describe cross-shore profile response changes, when a nearshore berm is present on the profile. It should be noted that the SBEACH 2.0 model simulations did not include accretionary mechanisms for sediment deposition on the landward extent of the profile. Rather, the model was used to predict seaward loss of material from the landward extent of the profile.

Predicted storm-related renourishment rates

Potential berm stability based on design templates is generally estimated by two methods: depth of closure (Pollock et al. 1993a) or comparison of existing site-specific wave-induced bottom velocities and conditions (Hands and Allison 1991). Both methods address whether the sediment is likely to move from the design template, rather than quantifying the volume of sediment movement. All design templates evaluated in this study were located within the depth of closure, and were not expected to be stable. For comparison, the volume of sediment movement from the berm templates was estimated, applying the SBEACH 2.0 and LTFATE models on a storm-event-related basis (Chapter 3 and Appendixes D and E)

SBEACH 2.0 storm response. Two-dimensional volumetric comparisons were made between prestorm and predicted poststorm cross-shore profiles, assuming a 1-m longitudinal berm section. Examples of storm-related berm template renourishment volumes based on SBEACH 2.0 simulations are presented in Table 4.

As indicated by values presented in Table 4, volume loss estimates varied per event for a given berm template. In general, the cross-shore loss of material predicted by SBEACH 2.0 from a berm template was minimal for each storm scenario. Poststorm profiles were compared to minimum berm crest width and elevation requirements for optimizing wave attenuation. The resulting berm geometries rarely suggested need for renourishment due to a single storm event.

LTFATE storm response. LTFATE is a three-dimensional coupled hydrodynamic-sediment transport model. LTFATE was developed to serve as a site-evaluation tool for estimating dispersion characteristics and subsequent stability for dredged material placed in open water over periods of time ranging from days to years (Scheffner et al. 1995). LTFATE was used to predict storm-related renourishment volumes for the suite of extratropical storm events and two selected nearshore berm design templates. To ensure numerical stability, Templates 8 and 10 were selected for testing, as the berm features are located seaward of the surf zone. Numerical evaluation of Templates 3 and 4 using LTFATE may have resulted in numerical instabilities, as the berm features are located in the surf zone region during a storm event.

To accurately simulate combined tide and storm-induced nearshore currents, hydrodynamic data sets were modified to produce depth-averaged current information for input to LTFATE. It was assumed that for Templates 8 and 10, the berm features were located offshore of the inner depth of closure. This assumes wave action to be an agitation force to suspend sediment, rather than a mechanism of net transport. Net transport was dependent on the tidal and storm-induced current, with wave action serving to augment the magnitude of sediment transport. For the simulations, each berm feature had a longitudinal length of 762 m. Table 5 presents storm-associated renourishment volumes as predicted by LTFATE. Volumes presented in Table 5 represent predictions for a cross-shore transect at the mid-point of a feature's longitudinal axis.

For all hydrodynamic events evaluated, the net volume change of the berm templates was negligible. Material from one end of the feature was predicted to migrate in the longshore direction to be deposited at another location on the feature. The migration of material was unidirectional, due to the character of the synthesized currents associated with each storm. Although migration of material was predicted, poststorm berm configurations remained similar in form to prestorm configurations, negating renourishment requirements on an event-related basis.

For each storm scenario evaluated with LTFATE, Template 10 experienced greater volume loss than Template 8. The difference in volume loss can be attributed to Template 10 being located in deeper water than Template 8, where larger waves can act on the surface area of the feature.

Conclusions

Estimates of annual beach-recession rates, project renourishment requirements, and storm-related recession distance are parameters required for economic evaluation of a nearshore berm project. For comparative purposes, methods for evaluating a nearshore berm's effect on the landward extent of the profile's recession envelope and storm-event-related project renourishment requirements were investigated. Numerical methods were applied by the U.S. Army Engineer Waterways Experiment Station under the authority of the Jacksonville District, to estimate parameter values that may be applied to an economic feasibility evaluation of nearshore berm configurations to the St. Johns County, Florida, Beach Erosion Control Project.

Results from this study indicate that the nearshore berm design configurations evaluated for a given suite of hydrodynamic events may provide some measure of storm damage mitigation of the shore. Placement of a berm feature on the nearshore profile is likely to initiate wave breaking over the feature, resulting in a less energetic wave re-forming between the berm and the shoreline. The less energetic wave is expected to result in reduced wave runoff and reduced coastal erosion.

Profile recession values for a suite of hydrodynamic events and berm design templates were predicted using a modification to the prescribed application of the SBEACH 2.0 model. The modification produced shoreline change results that are intuitively correct for the presence of a nearshore berm feature on the profile. Model results are substantiated by limited measured laboratory data, but are unverified due to a lack of prototype-scale measurements for comparison. Simplified simulations using SBEACH 2.0 and LTFATE indicated that berm templates selected for testing would require minimal or no renourishment on an event-related basis for the applied suite of storms.

Environmental and physical limitations of the dredging and placement process must be assessed to select the optimum profile template. Based solely on the

numerical predictions of physical aspects, it is evident that three characteristics of berm geometry enhance wave attenuation and reduce R_o : increased crest elevation or reduction in the depth of water column over the crest; increased crest width for a given elevation; and proximity to the shore. The number of test profiles was reduced to four cases which optimize these factors and considered historic use of dredges in the region. All four berms resulted in significant reductions in R_o and required minimal renourishment, if any, for the suite of test storms. Categorizing the cases by crest elevations, the two landward berms (Profiles 3 and 4) outperformed the seaward berms (Profiles 8 and 10) for every storm condition. Profile 3 and 4 results were comparable, with Profile 3 performing slightly better than Profile 4 for some storm conditions; however, Profile 3 required more material and mechanical maneuvering for placement. For the deeper class of berms, Profile 8 only slightly outperformed Profile 10. Profile 8 required less material to construct and was less likely to require renourishment.

Study Contribution to Coastal Engineering and Recommendations

It is the objective of this study to present a reconnaissance-level simplified and innovative approach to provide the physical parameters needed to translate wave attenuation benefits induced by a nearshore berm feature to economic benefit calculations. Use of the modified SBEACH 2.0 model application and the LTFATE model to provide these parameters is a preliminary step that may enable the coastal manager to evaluate the potential for storm mitigation benefits of nearshore berms as compared to other engineered coastal management techniques. This is only a first step in the development of methods to evaluate shoreline protection options. Additional collection and analyses of laboratory and prototype-scale data are necessary to validate and improve the numerical methods used in this investigation.

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Appendix A

Nearshore-Berm Design Profile Templates

Profile elevations in meters National Geodetic Vertical Datum.

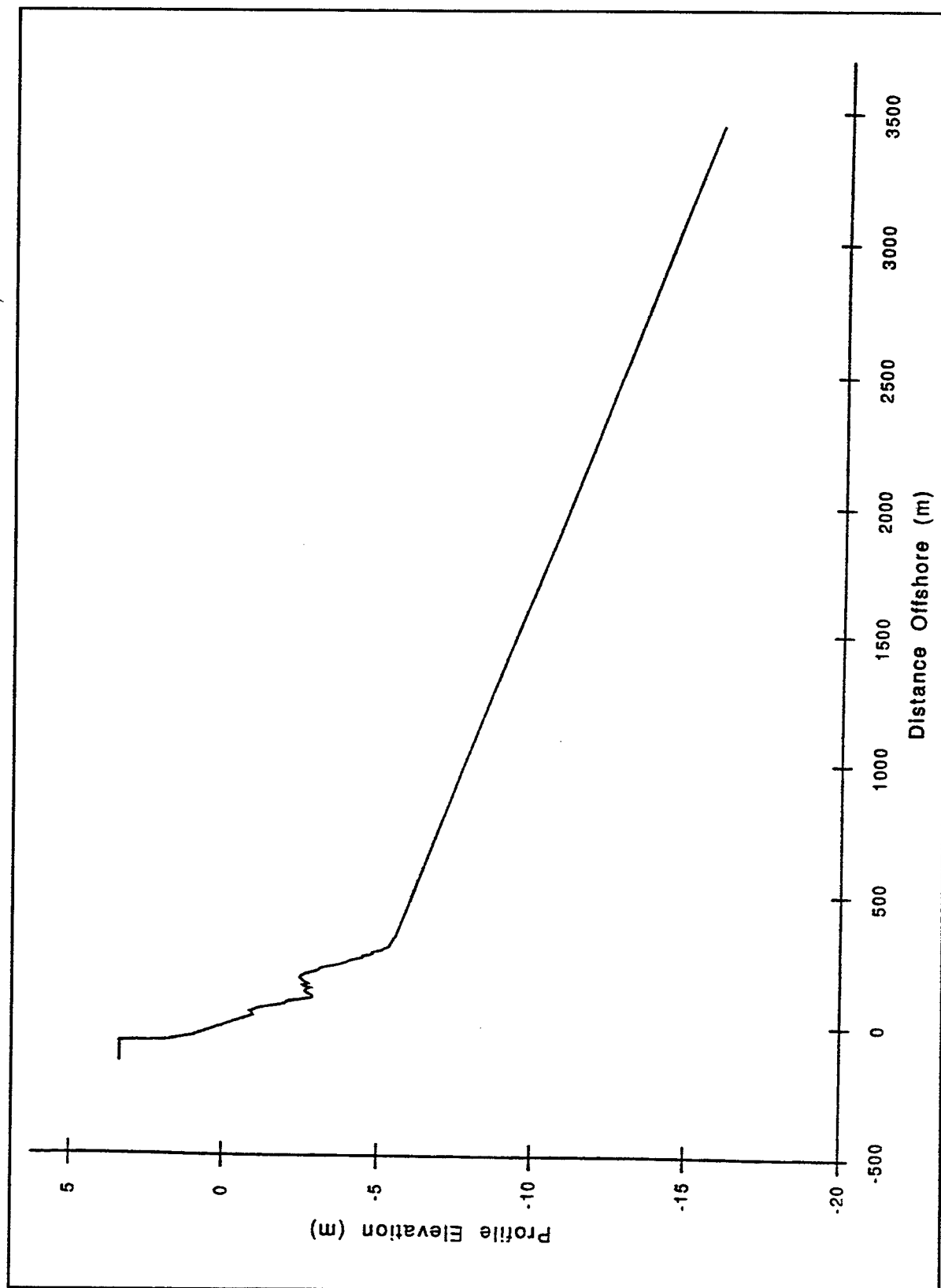


Figure A1. Design profile template 1

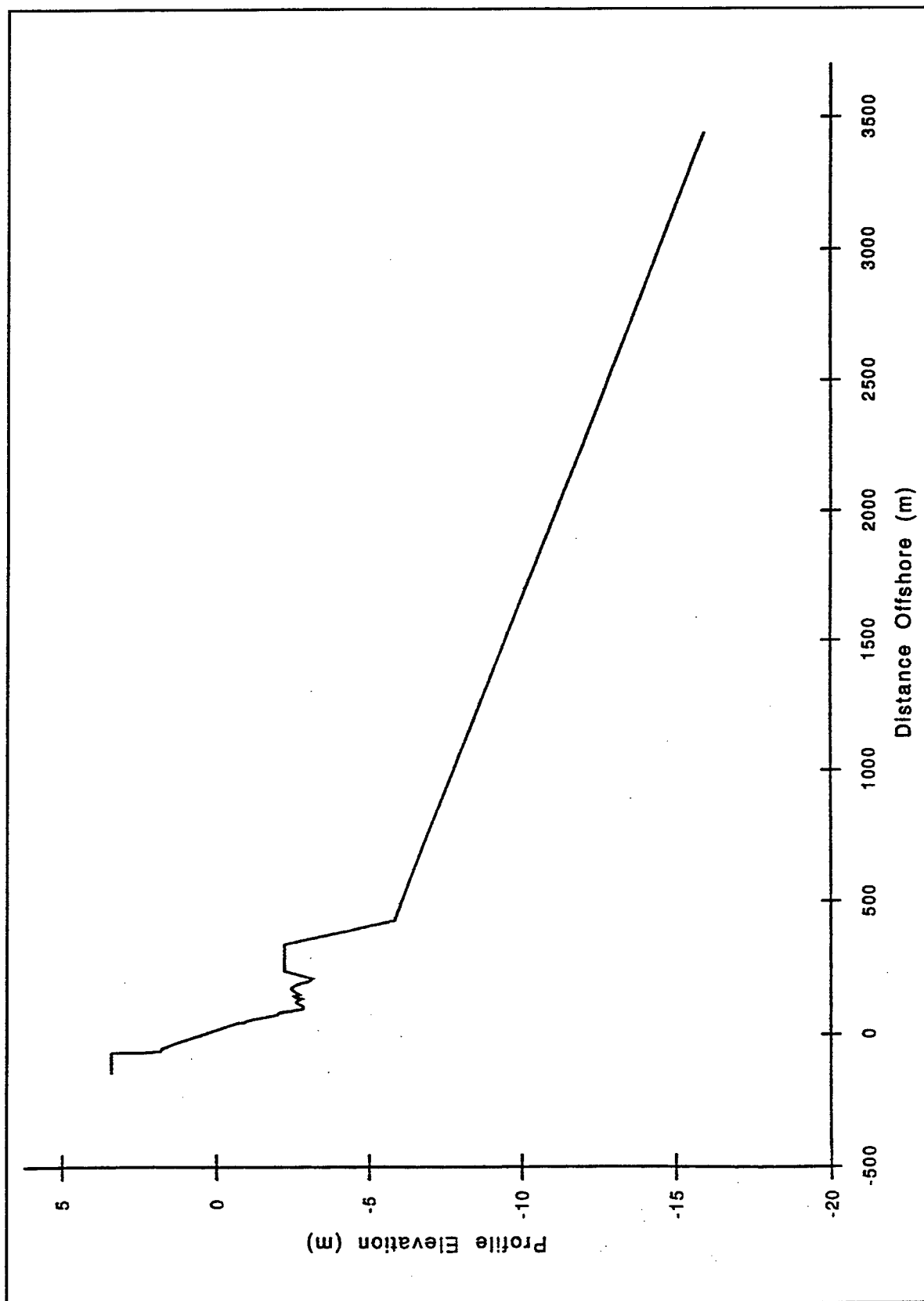


Figure A2. Design profile template 2

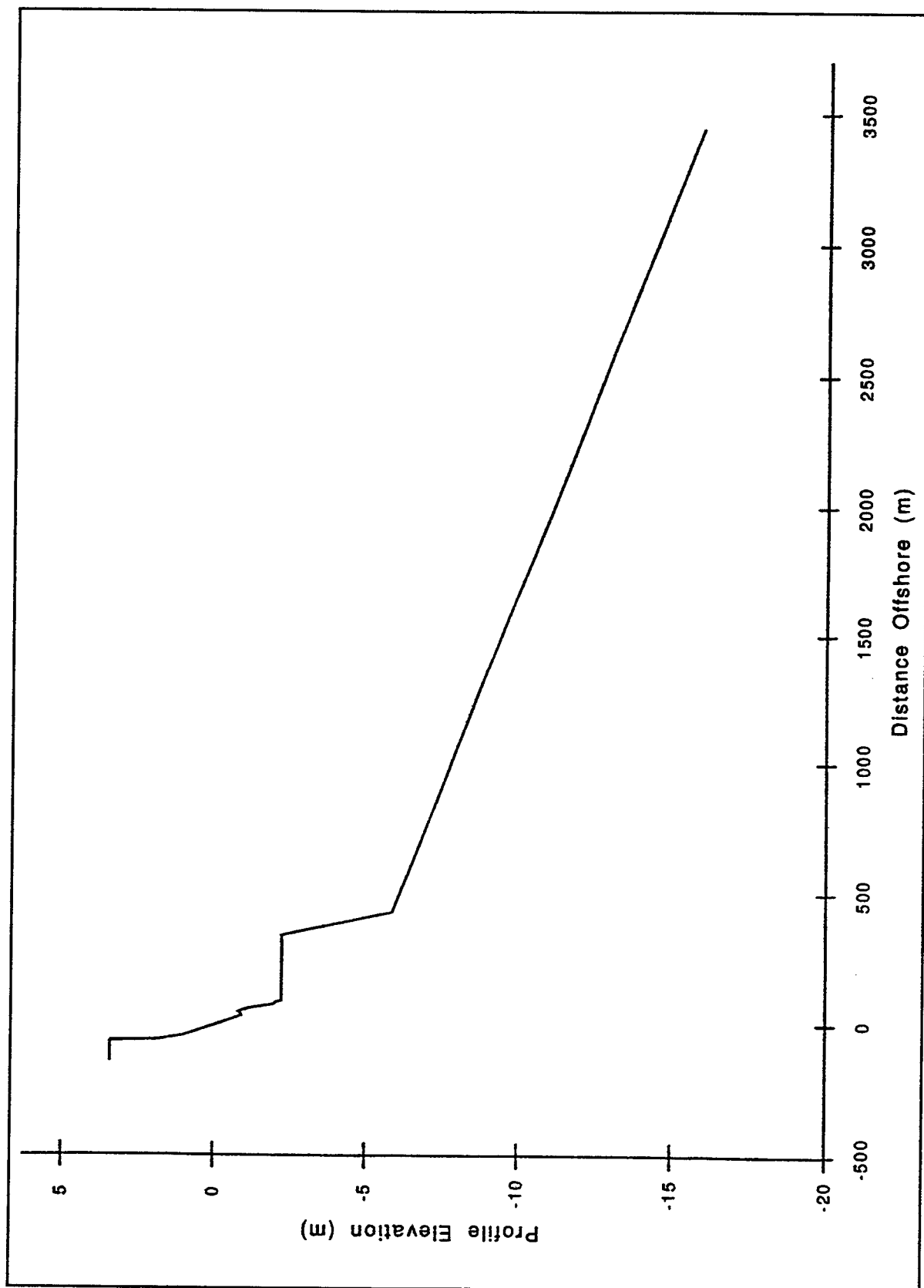


Figure A3. Design profile template 3

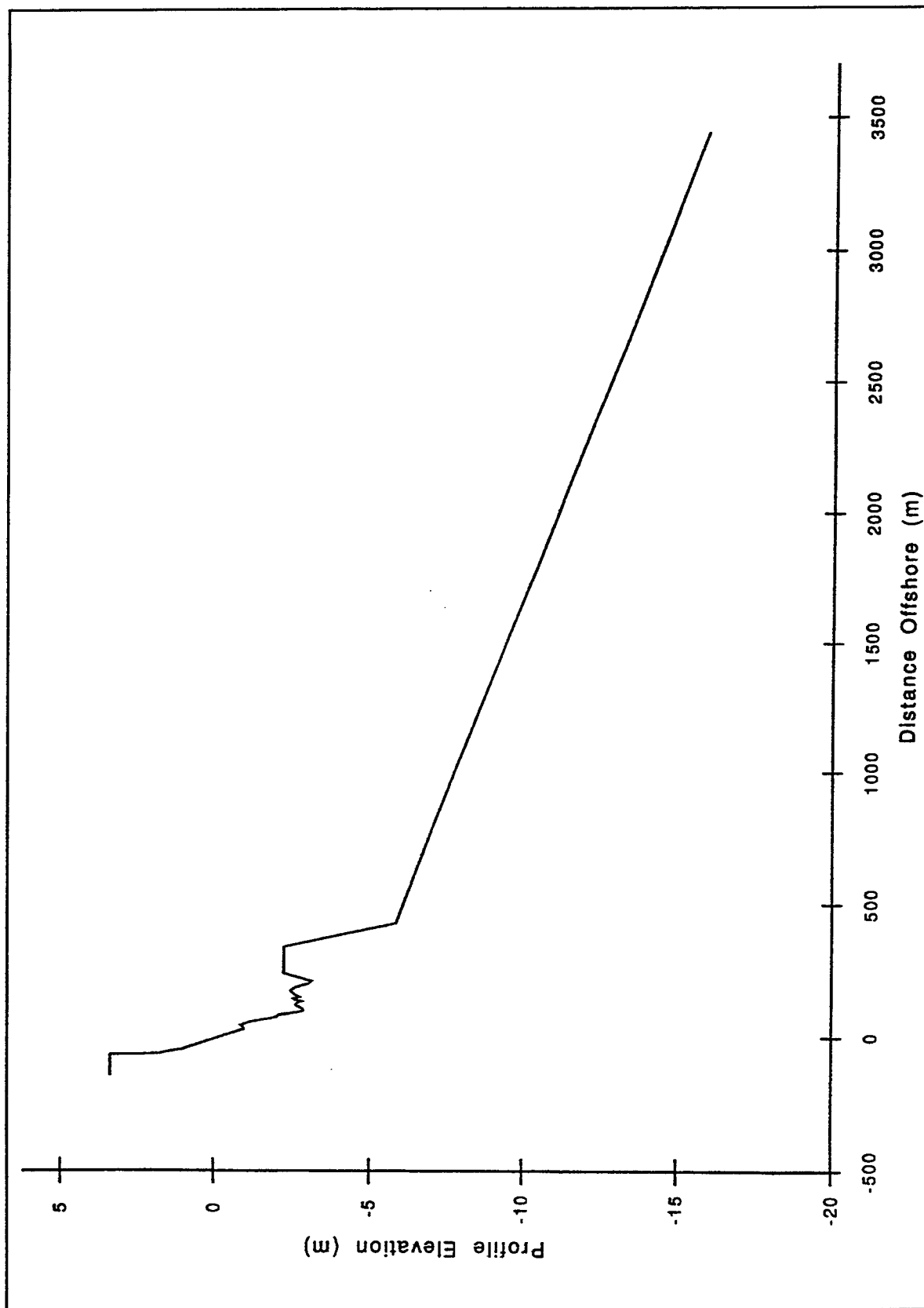


Figure A4. Design profile template 4

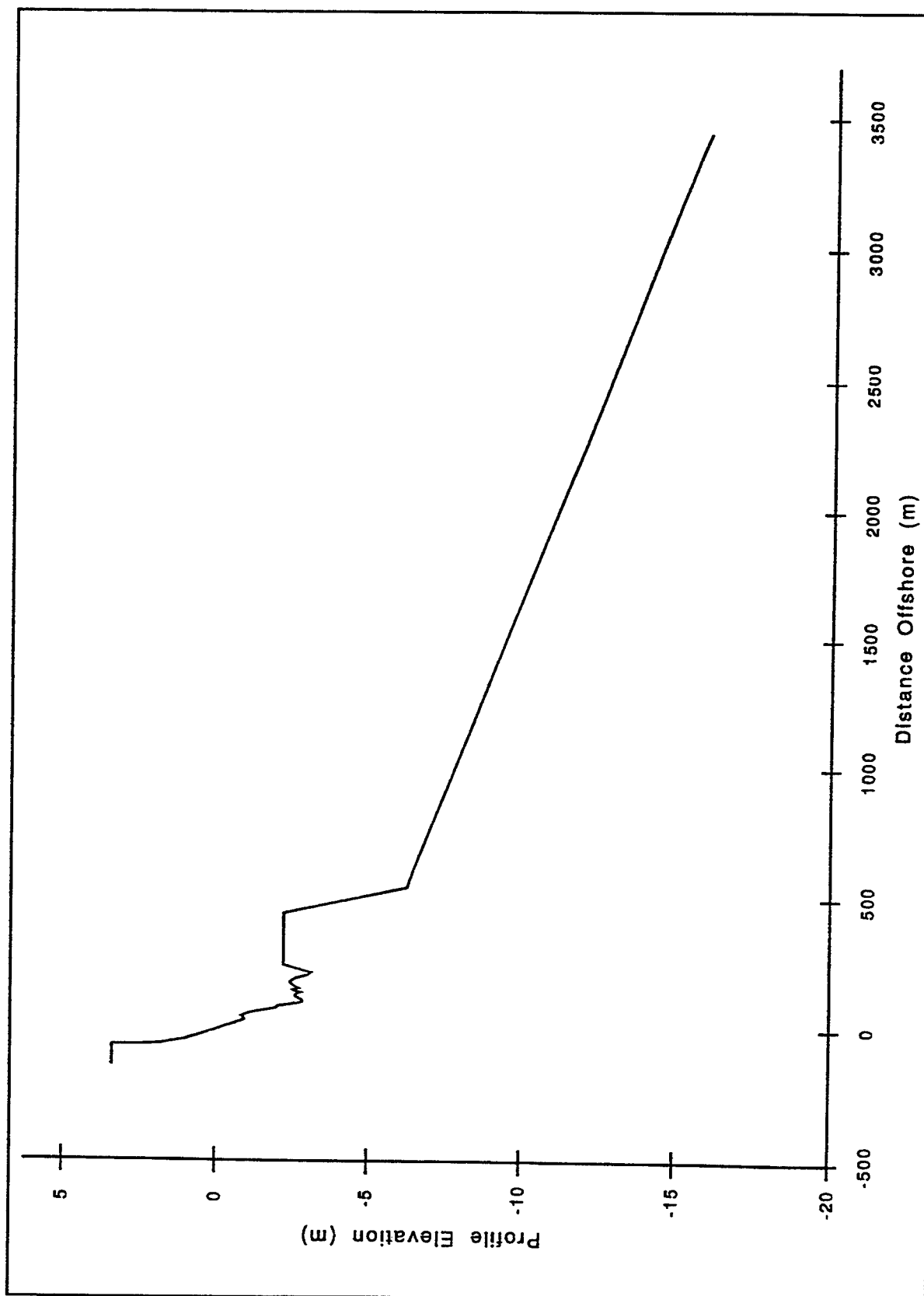


Figure A5. Design profile template 5

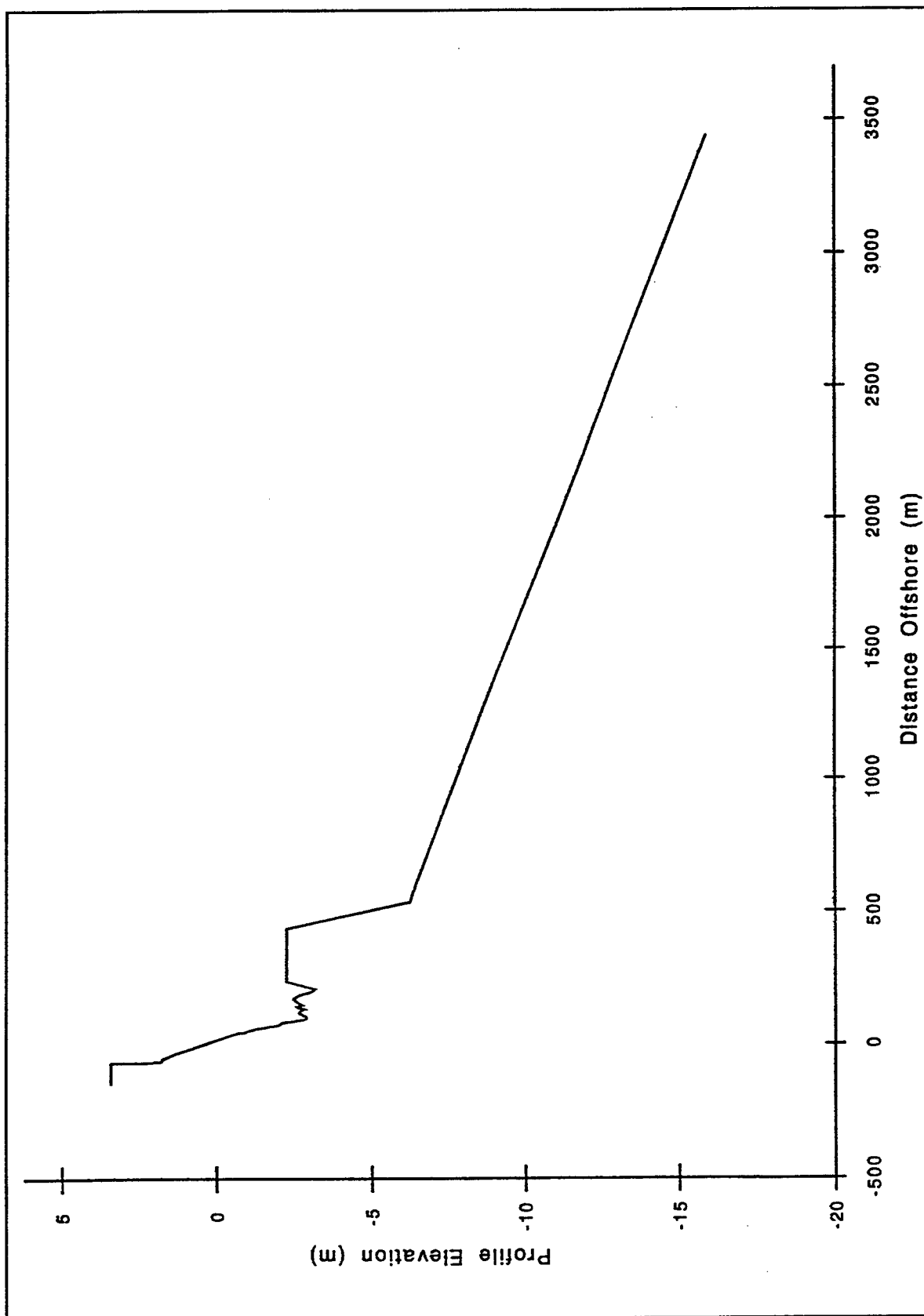


Figure A6. Design profile template 6

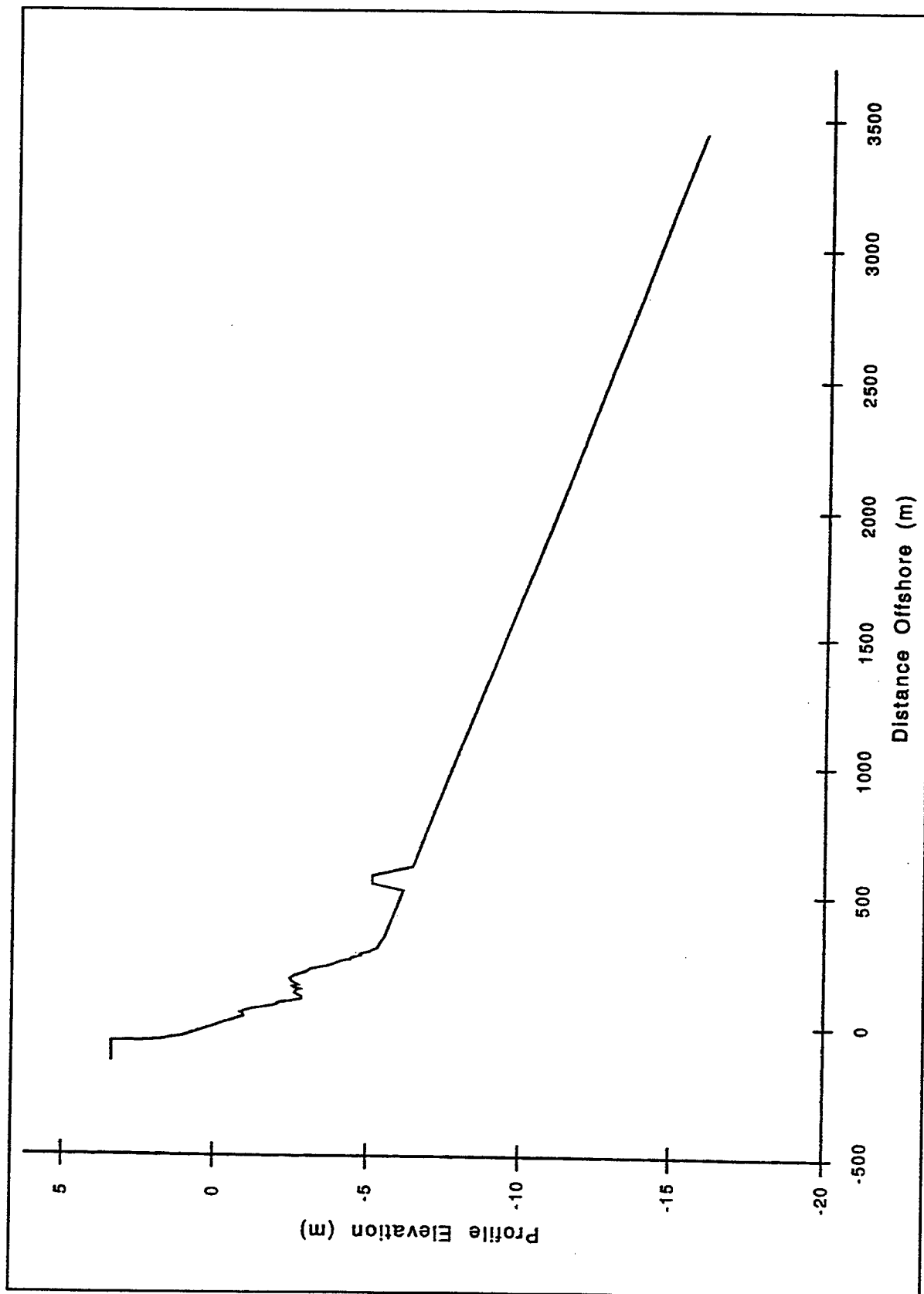


Figure A7. Design profile template 7

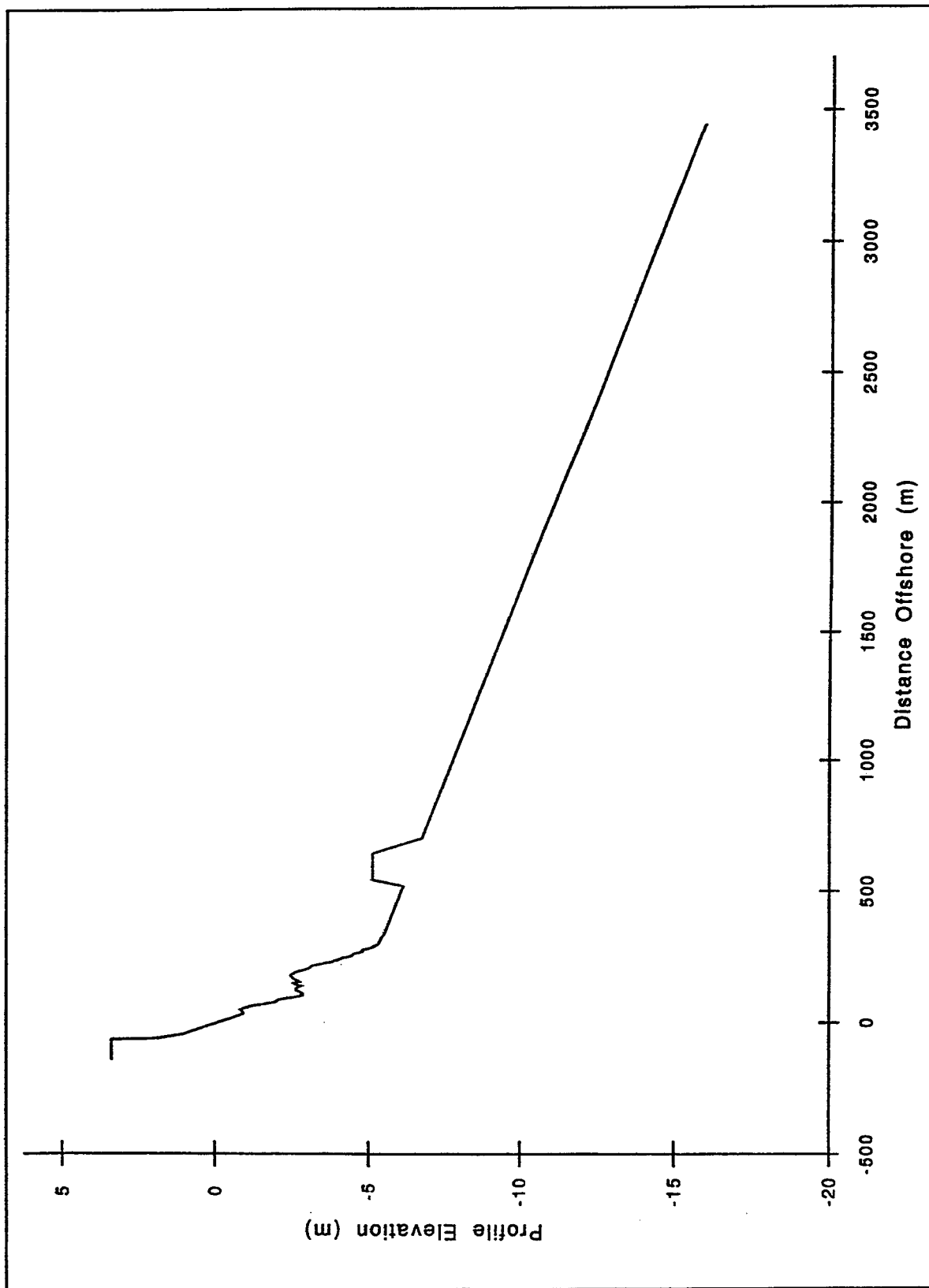


Figure A8. Design profile template 8

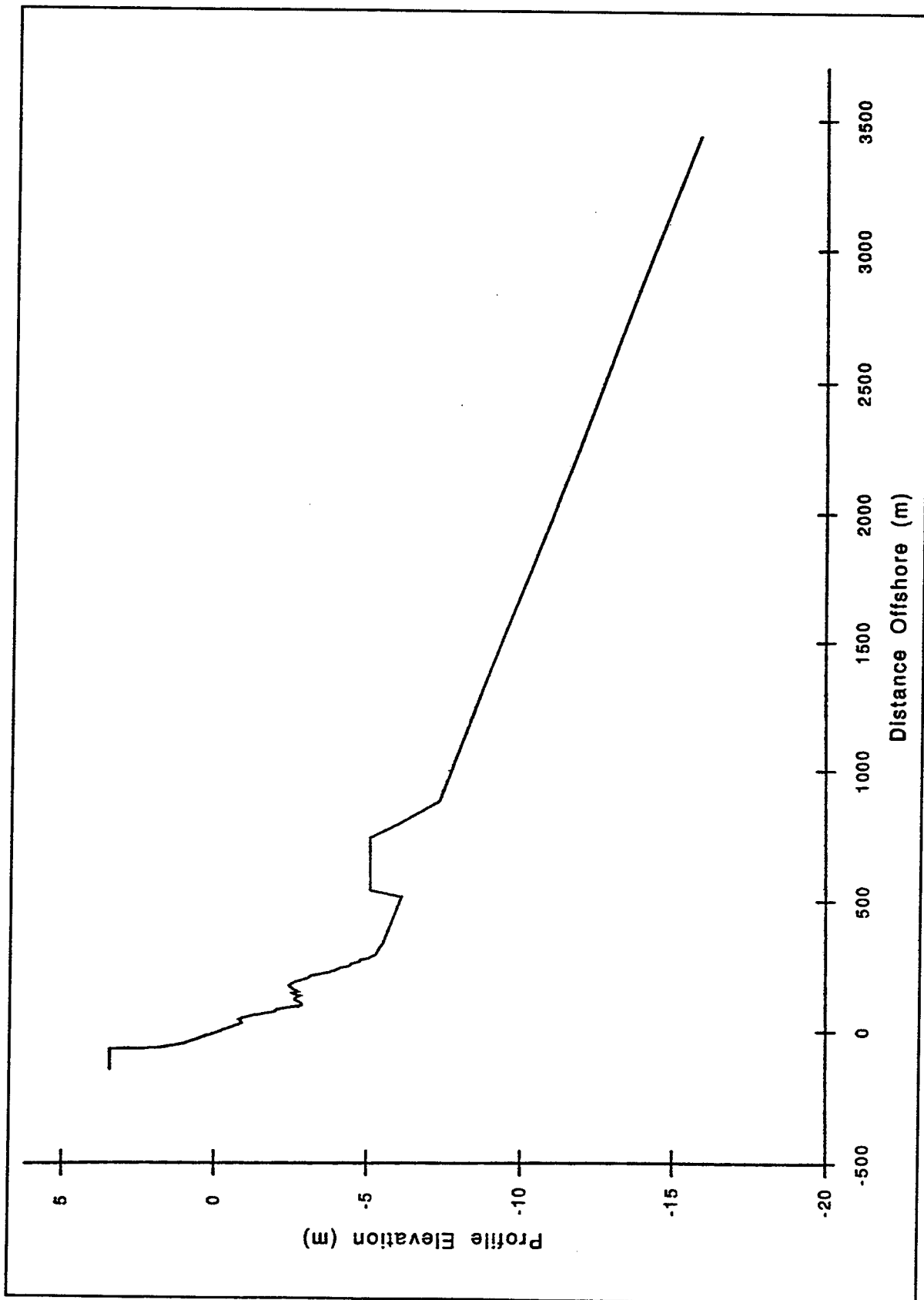


Figure A9. Design profile template 9

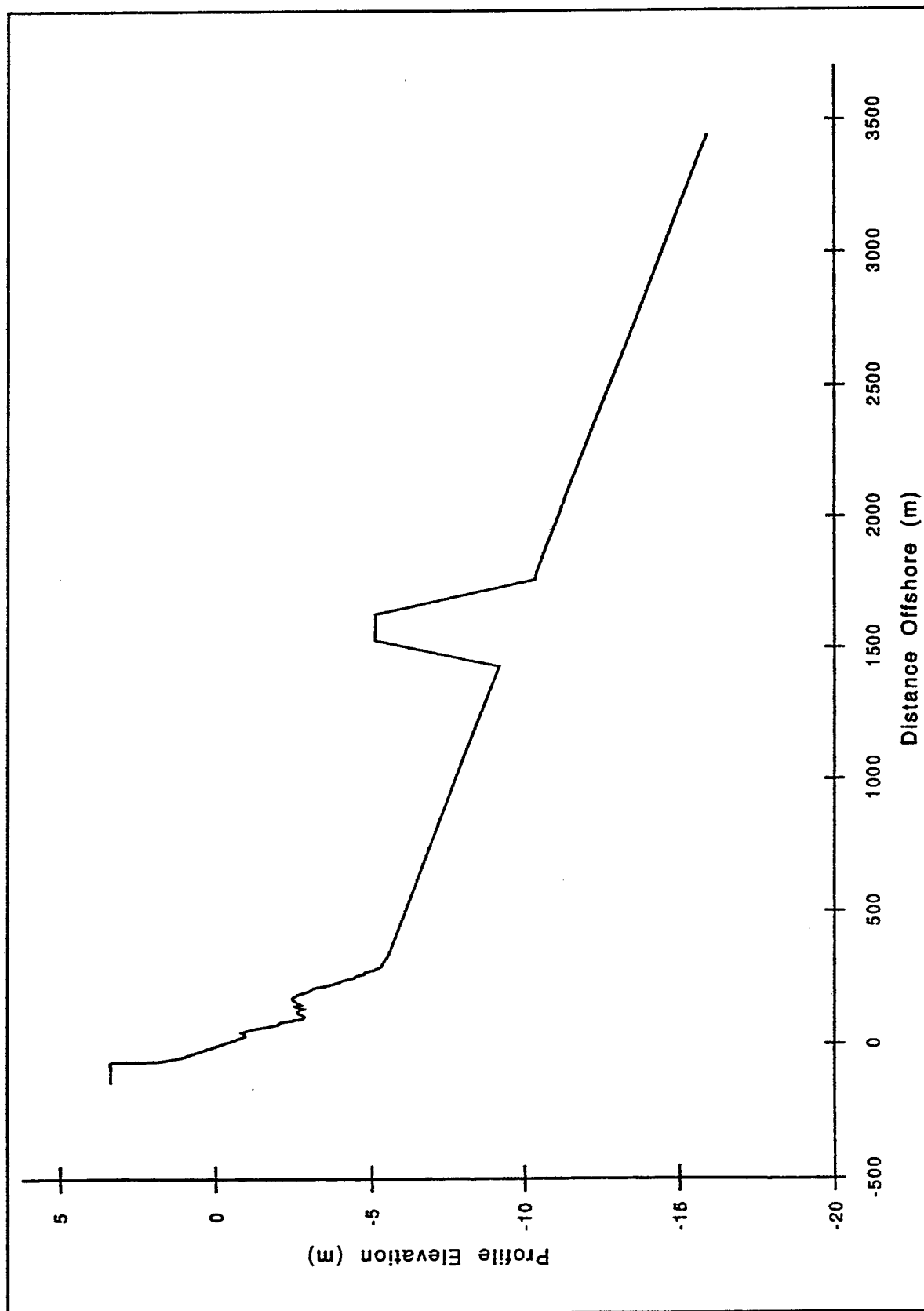


Figure A10. Design profile template 10

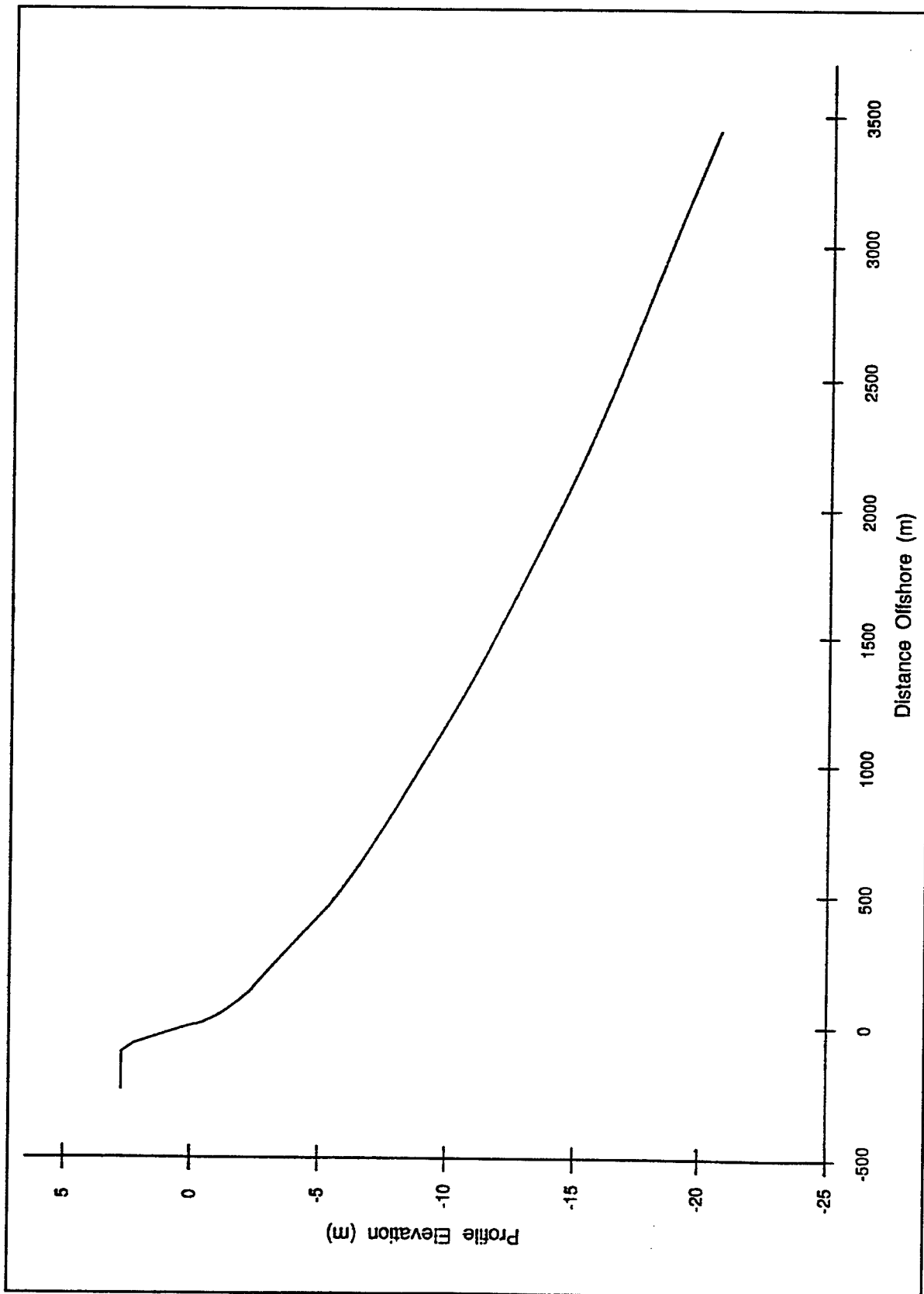


Figure A11. Design profile template 1: Equilibrium profile

Appendix B

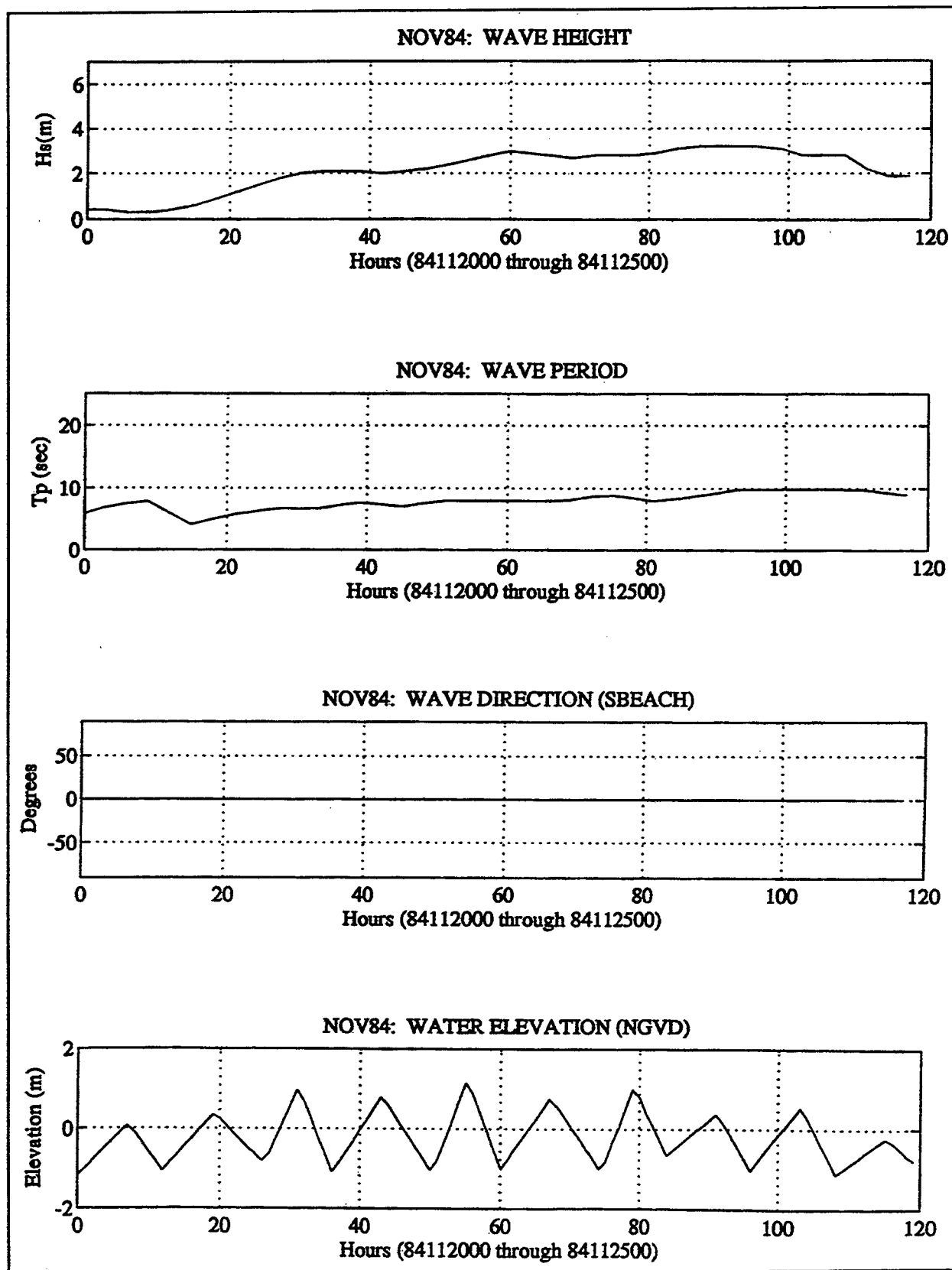
Hydrodynamic Events

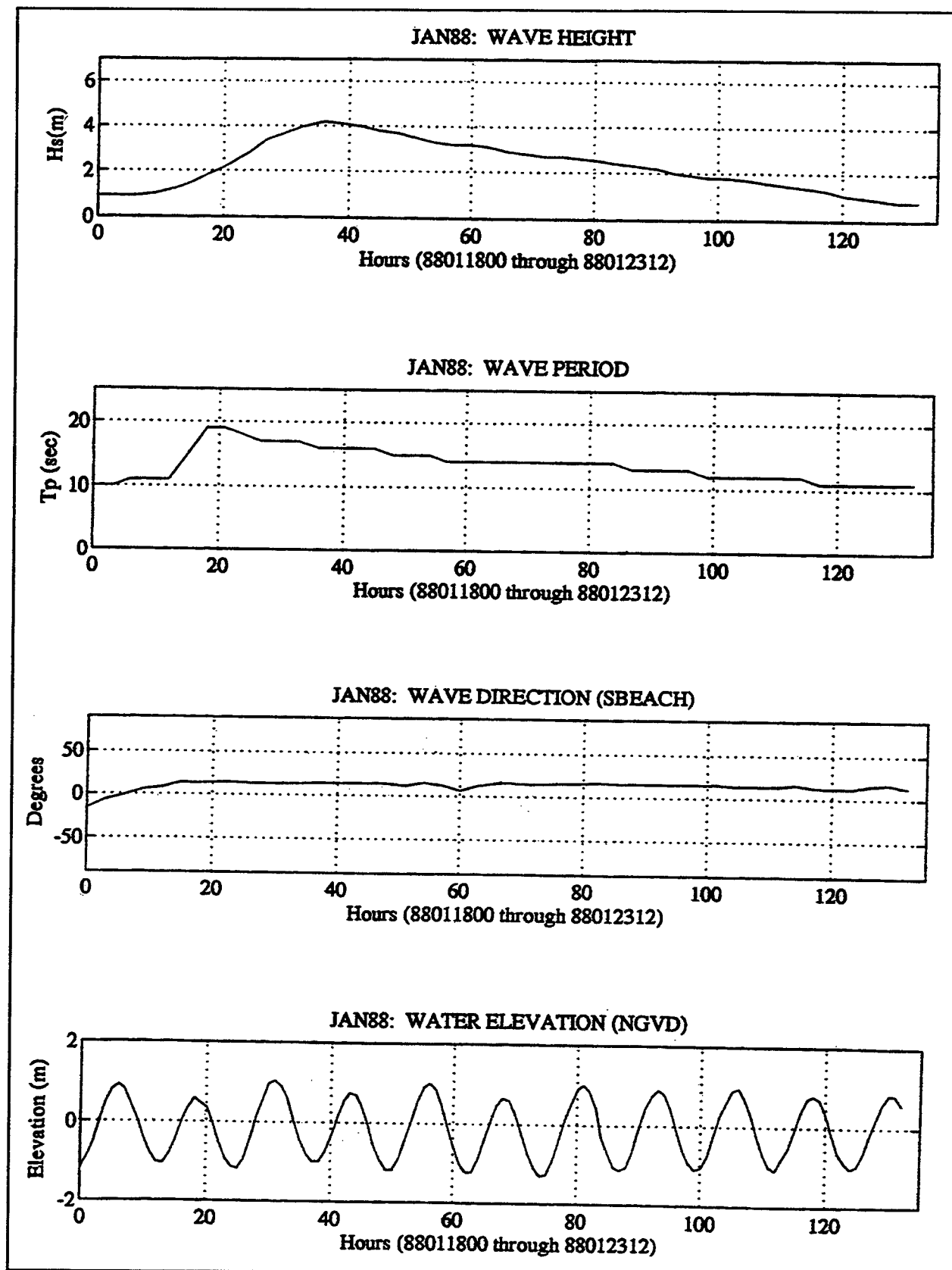
With exception of the November 1984 storm and synthesized storm events (TR25, TR50, and TR100), wave events were selected from the Wave Information Study (WIS) Revised Hindcast Level II database (WIS location 25 / 30°N, 81°W / depth = 20 m). Selection criteria were based on wave events in excess of $H_{s(max)} = 3$ m and a duration in excess of 120 hr with wave heights exceeding 1 m. Assuming straight and parallel contours, waves were transformed to a depth of 15.24 m for input to SBEACH 2.0. Water elevations corresponding to WIS events were obtained from the Fernandina Beach location Automated Real-Time Tidal Elevation System (ARTTES) database. ARTTES data were corrected for the St. Augustine Beach location and applied to SBEACH 2.0.

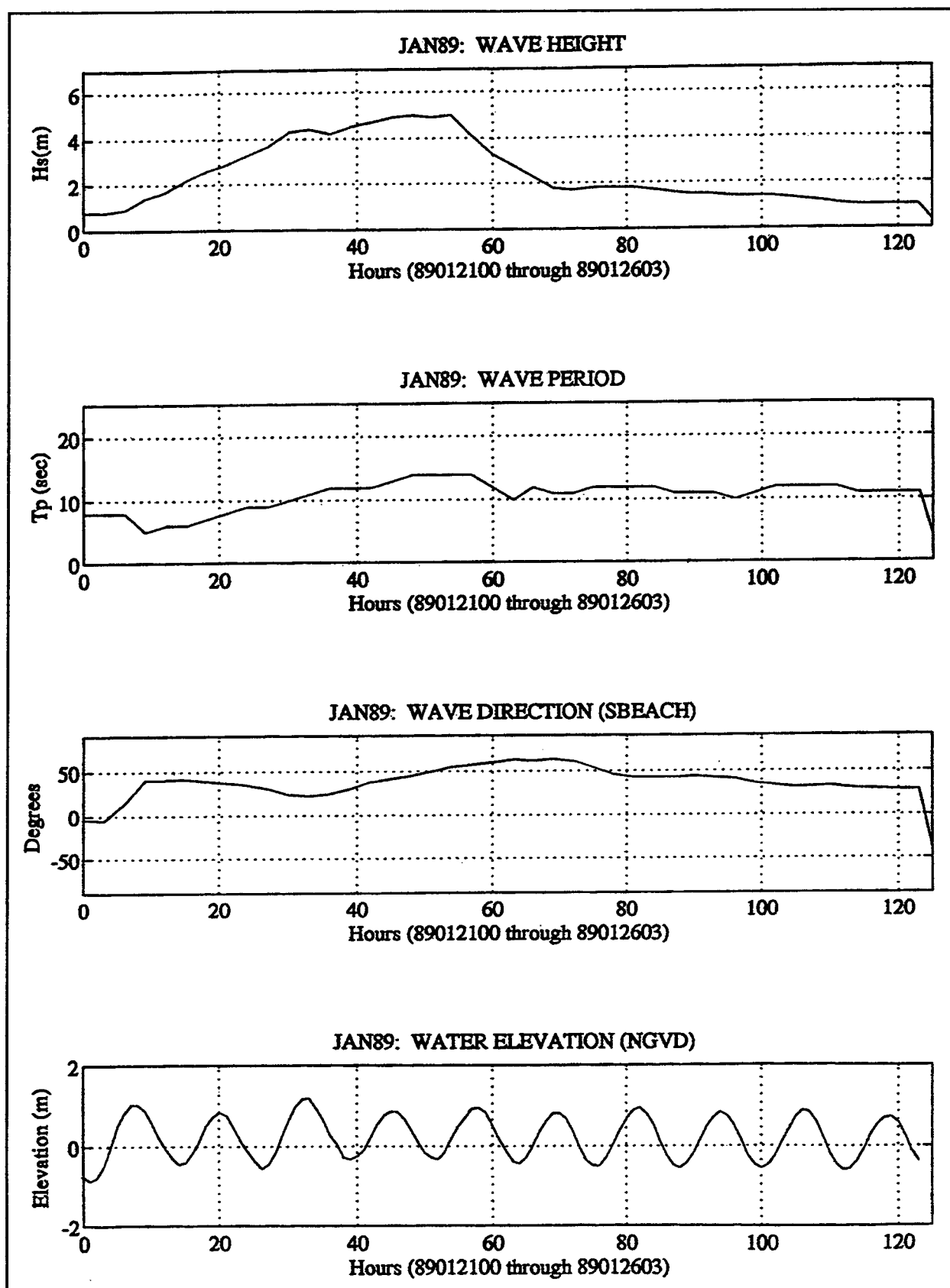
The November 1984 storm wave data were provided by the U.S. Army Engineer District, Jacksonville, (accessed from the Coastal Data Network Marineland Gage). Water level data were also provided by the Jacksonville District (selected from a Florida DNR monitoring report). Marineland gauge wave data is nondirectional. A shore-normal direction of wave propagation was assumed when applied to SBEACH 2.0.

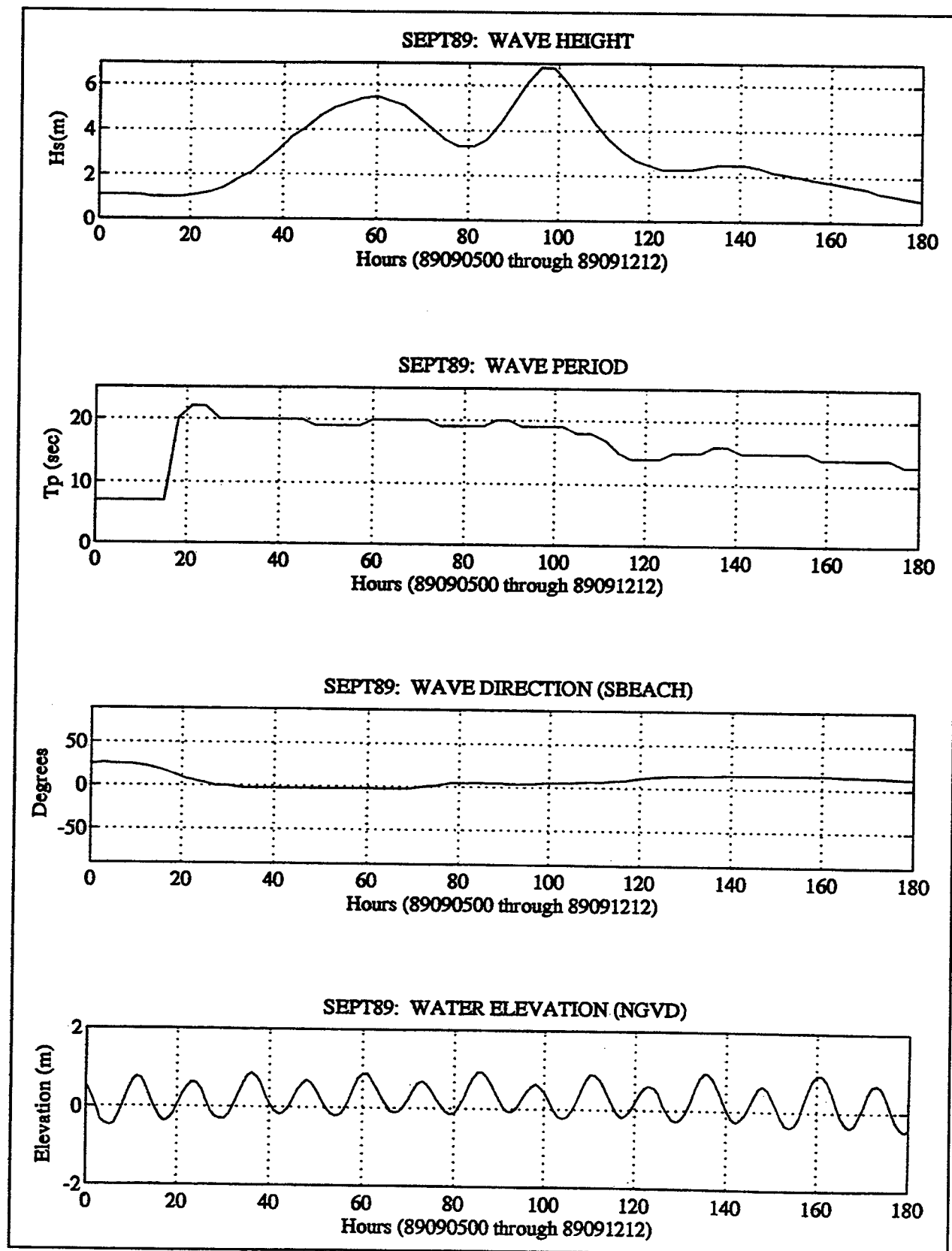
Extreme event water level values were arbitrarily chosen to represent tropical events. These tropical events are represented by storms TR25, TR50, and TR100. A deep-water wave height was arbitrarily chosen to represent a maximum tropical storm wave height. Assuming straight and parallel contours, the wave was transformed to a depth of 15.24 m. A corresponding value of 20 sec for maximum peak period was arbitrarily chosen. A Gaussian-type distribution was fit to water elevation, wave height, and wave period values to generate wave and water level time series of 24-hr duration. A shore-normal direction of wave propagation was assumed when applied to SBEACH 2.0.

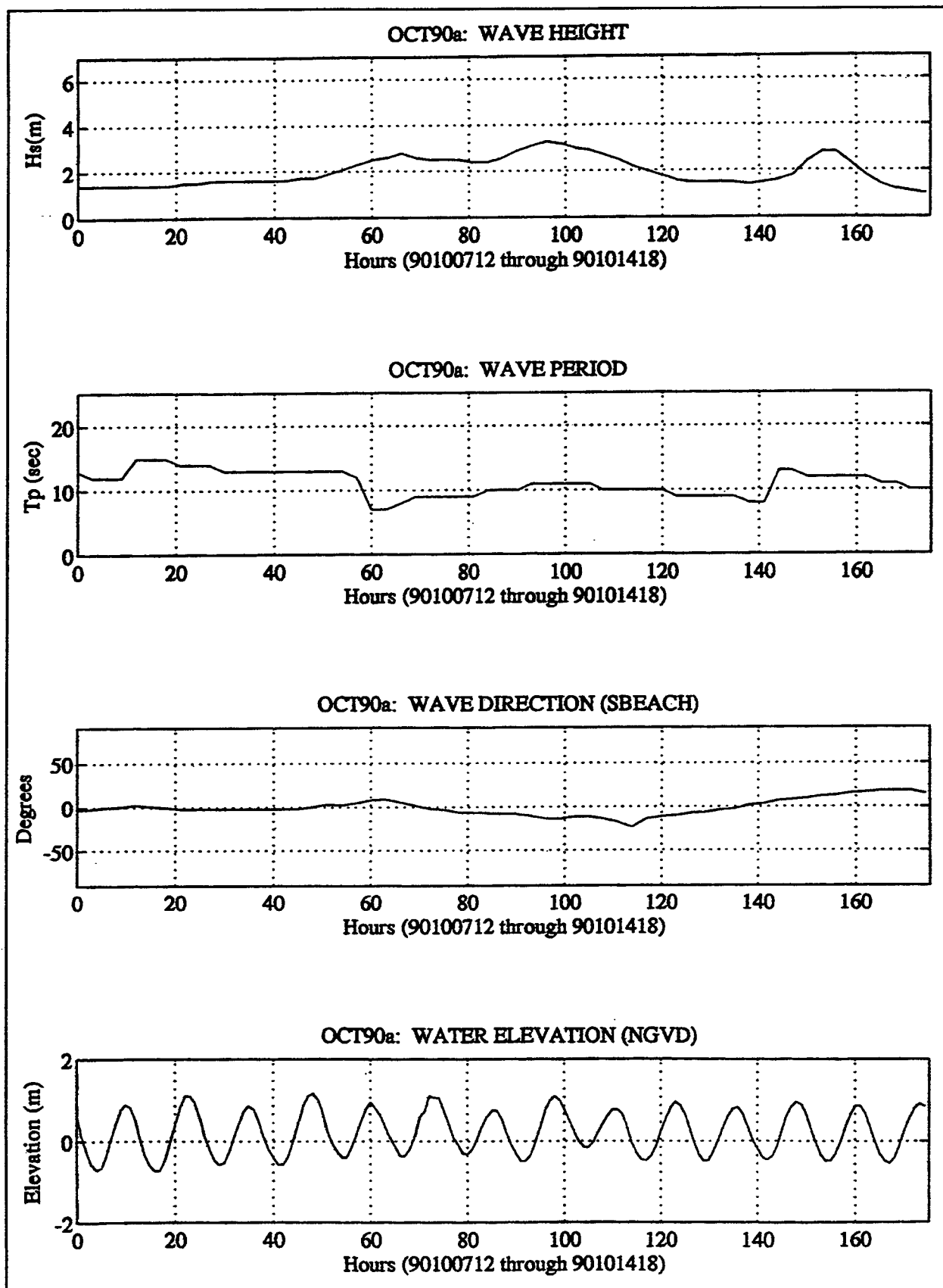
Table B1 Input SBEACH 2.0 Storm Events		
Event	Date (YYMMDDHH)	Reference
NOV84	84112000 - 84112500	SAJ
JAN88	88011800 - 88012312	WIS/ARTTES
JAN89	89012100 - 89012603	WIS/ARTTES
SEPT89	89090500 - 89091212	WIS/ARTTES
OCT90a	90100712 - 90101418	WIS/ARTTES
OCT90b	90101906 - 90102321	WIS/ARTTES
HALLOW91	91102112 - 91110421	WIS/ARTTES
TR25	24-hr duration	Scheffner 1994 ¹
TR50	24-hr duration	Scheffner 1994 ¹
TR100	24-hr duration	Scheffner 1994 ¹
¹ Personal Communication, 1994, N. Scheffner, U.S. Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, Vicksburg, MS.		

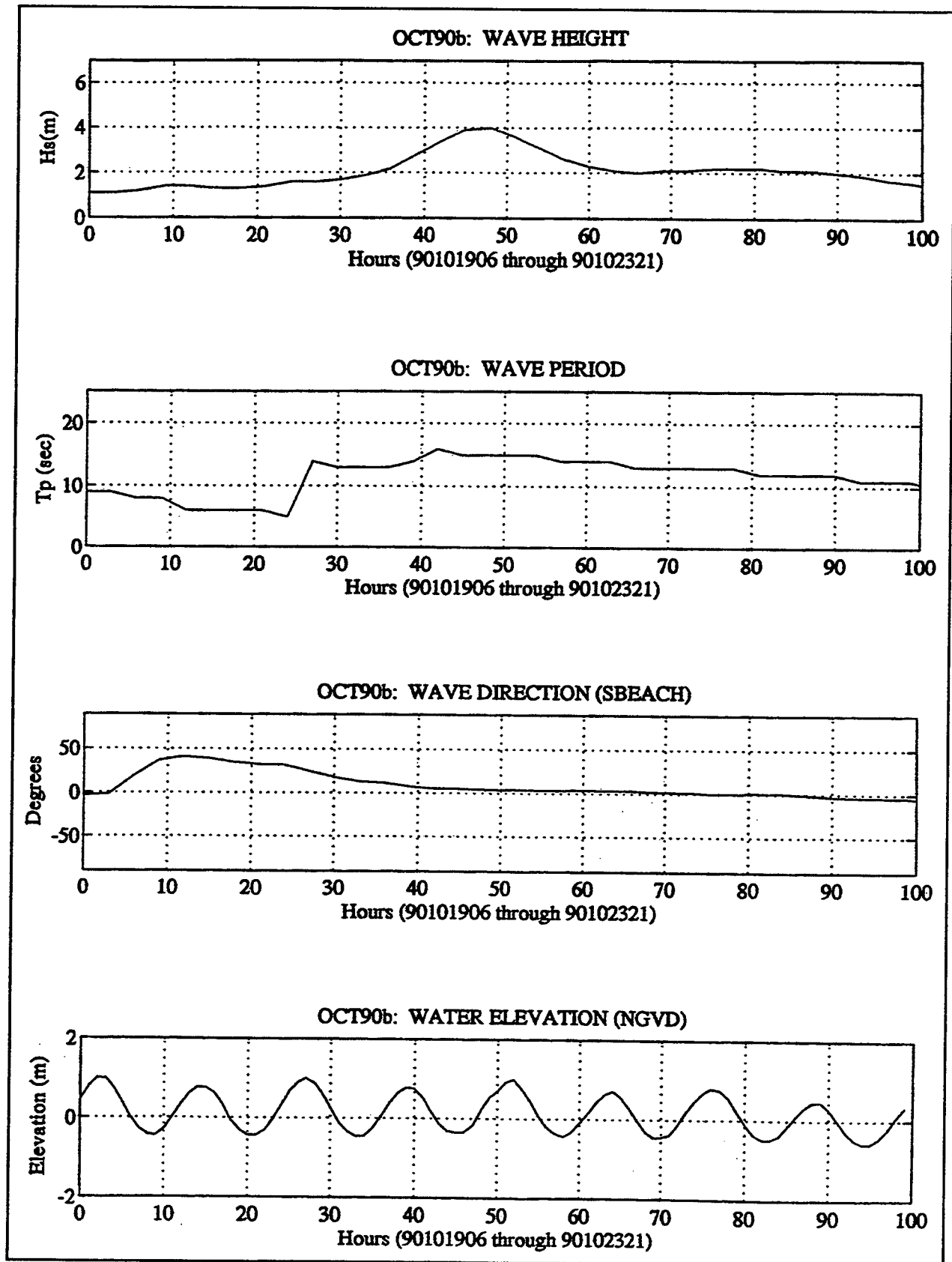


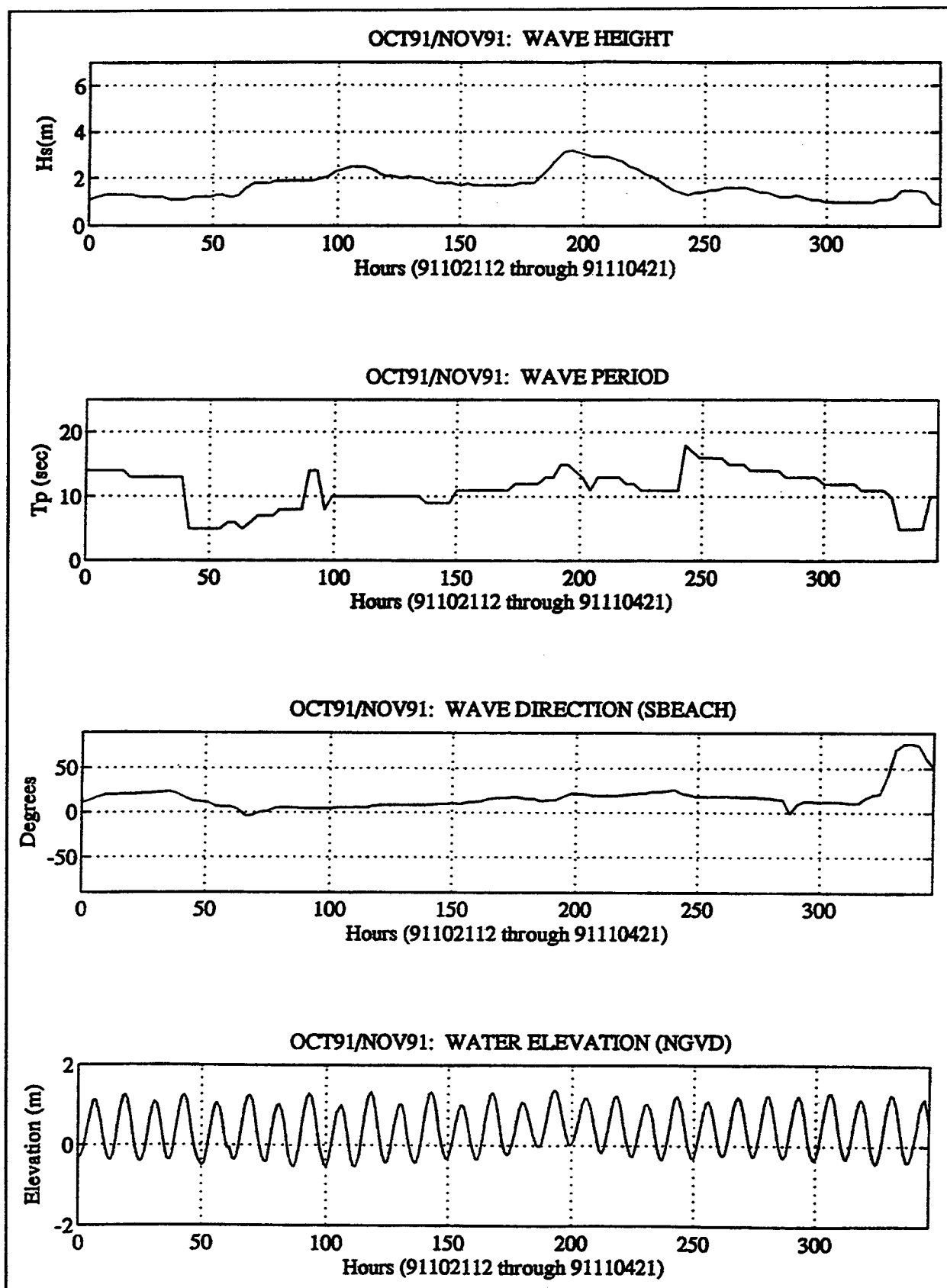




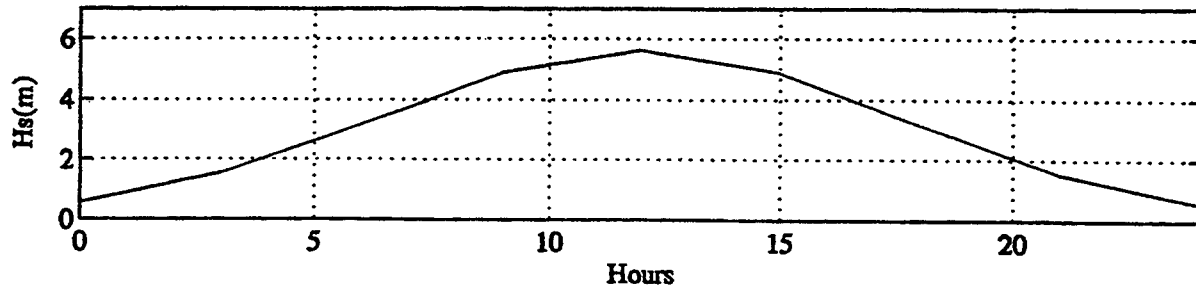




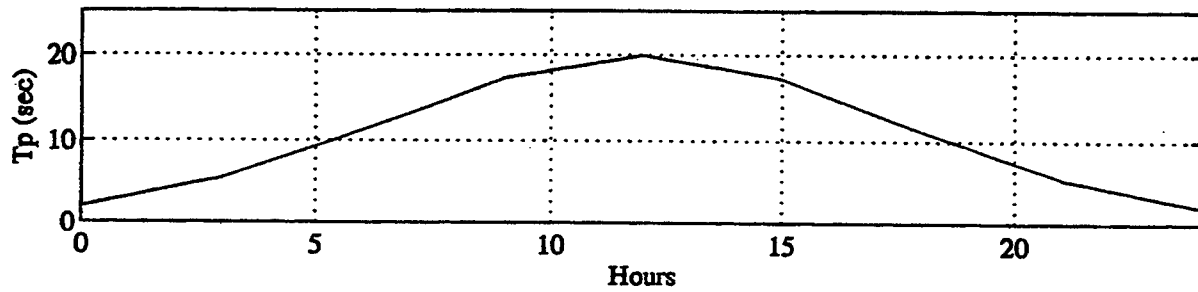




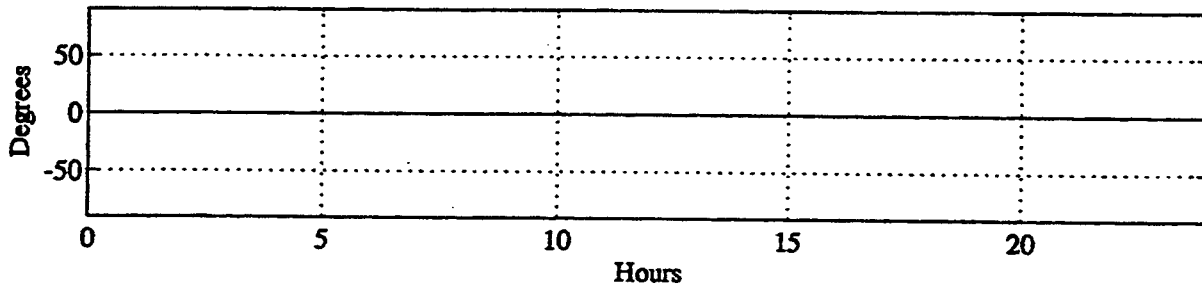
TR25: WAVE HEIGHT



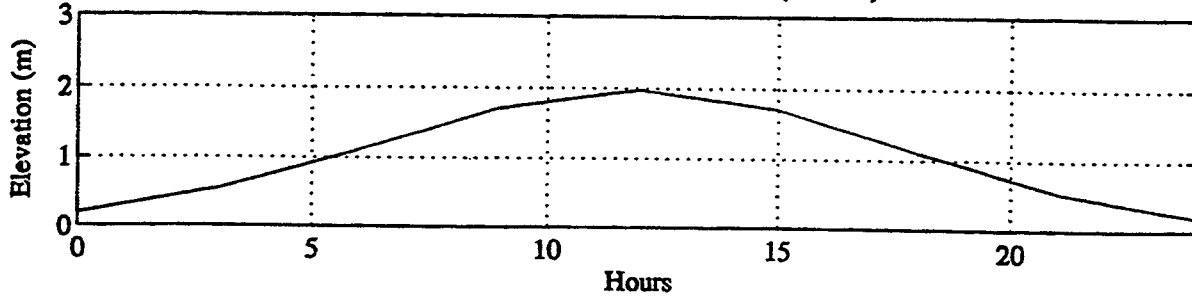
TR25: WAVE PERIOD

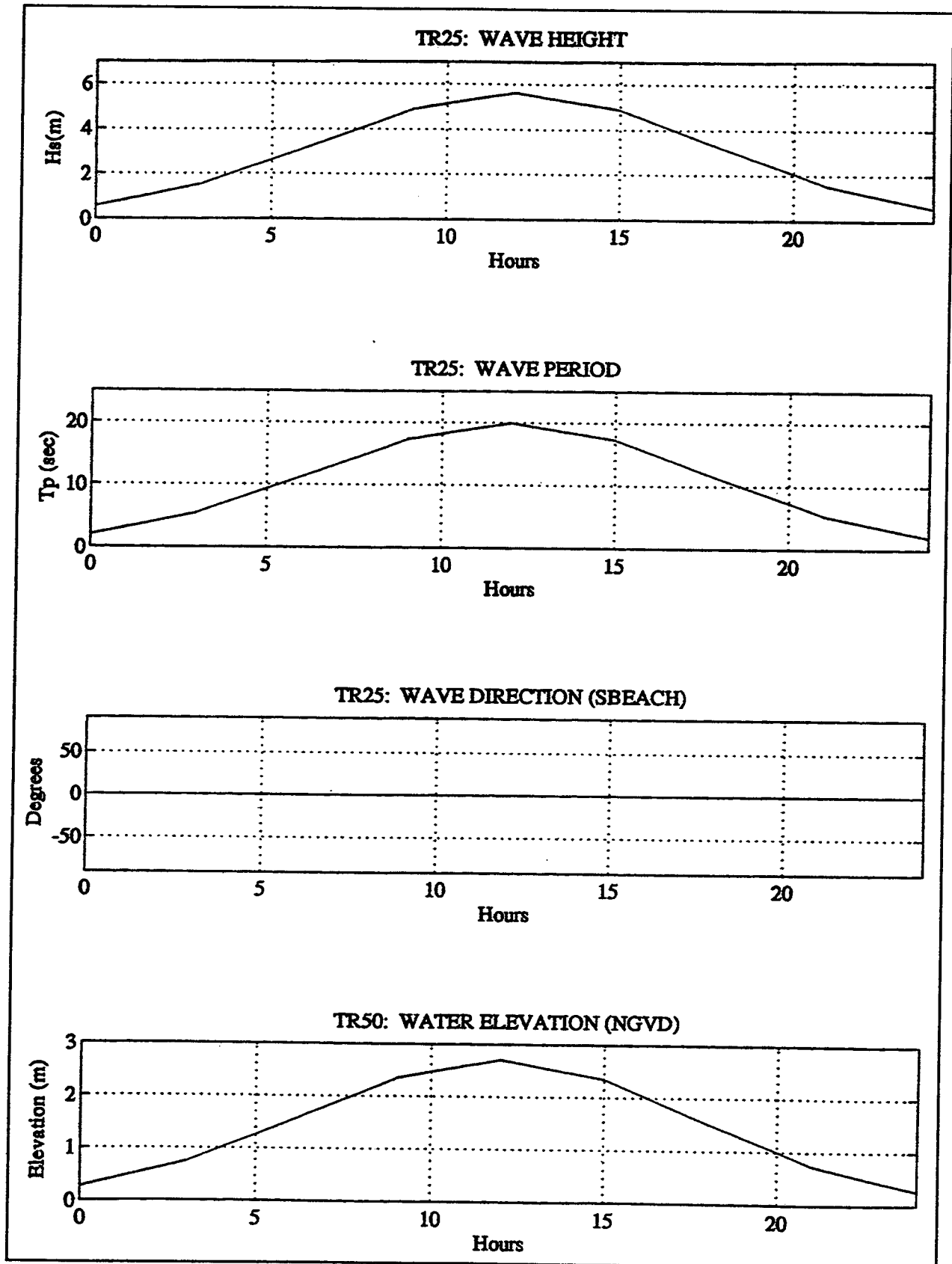


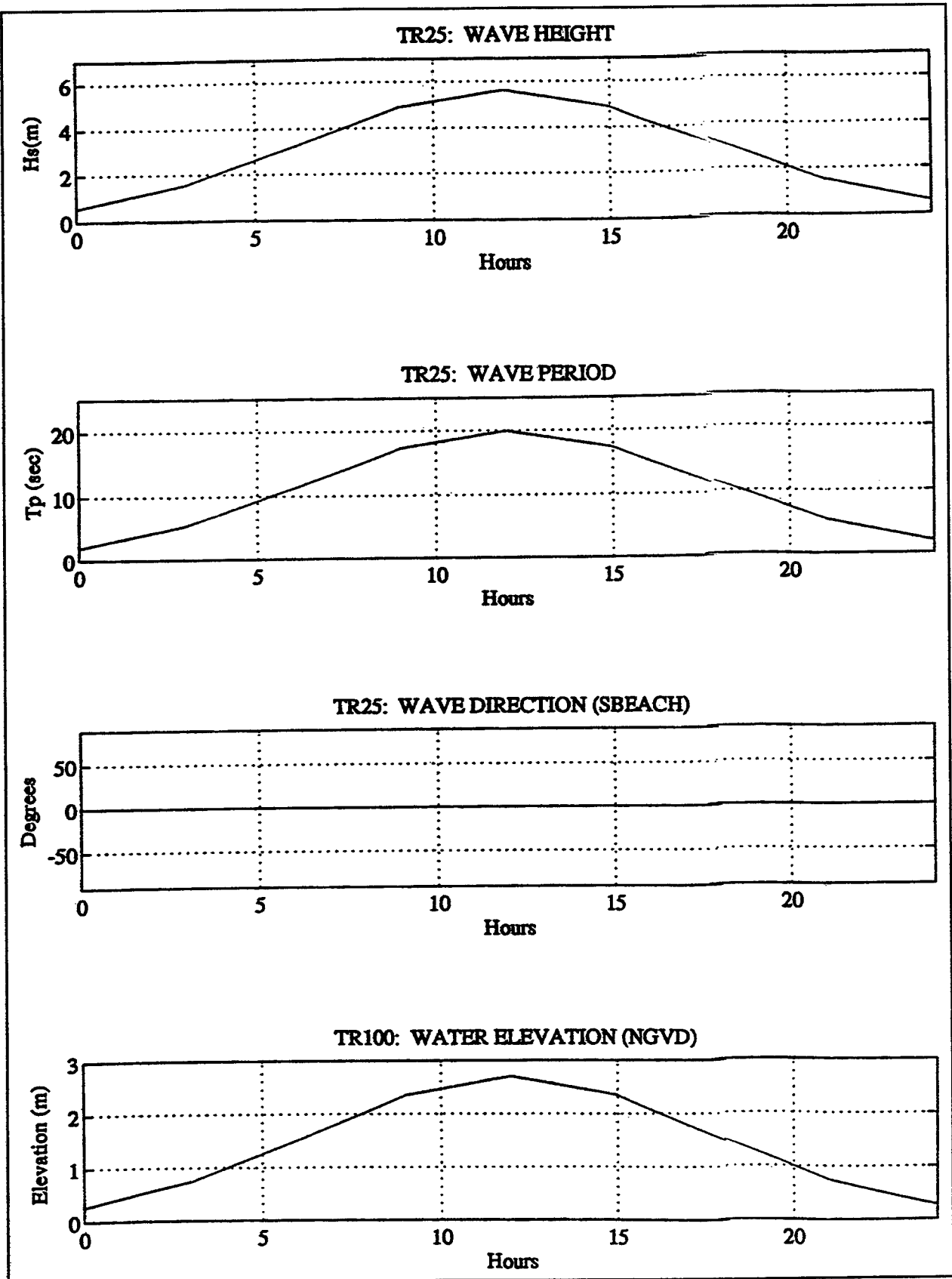
TR25: WAVE DIRECTION (SBEACH)



TR25: WATER ELEVATION (NGVD)







Appendix C

SBEACH 2.0 Input Parameters

Table C1
Method I. SBEACH 2.0 Configuration Parameters for Storm Jan88 and Profile 8. Input
Units: SI and Minutes

Parameter	Value	Reference
TITLE	Profile 8 JAN88	
UNITS	1	
NDX; XSTART	521; -144.98	
IDX	1	
NGRID	4	
WDXV; NDXV	2, 212; 5, 145; 10,85; 20,79	
NDT; DT	1584; 5	
ELV1; ELV2; ELV3	0.75; 0; -0.65	
EDP1; EDP2; EDP3; REFELV	0.75; 0.5; 0.25; 0.75	
K	1.5E-6	Rosati, Wise, and Kraus 1993 ¹
EPS	0.002	Rosati, Wise, and Kraus 1993
LAMM	0.2	Rosati, Wise, and Kraus 1993
TEMPC	15	Rosati, Wise, and Kraus 1993
WVTYPE	1	
IWAVE	1	
DTWAVE	180	
INANG	1	
DTANG	180	

¹ References cited in this appendix are located at the end of the main text.

Table C2
Method II. SBEACH 2.0 Configuration Parameters for Storm
Jan88 and Profile 8 (Rosati, Wise, and Kraus 1993). Input Units:
SI and Minutes

Parameter	Value	Reference
TITLE	Profile 8 JAN88	
UNITS	1	
NDX; XSTART	280; -144.98	
IDX	1	
NGRID	2	
WDXV; NDXV	2, 212; 5, 68	
NDT; DT	1584; 5	
ELV1; ELV2; ELV3	0.75; 0; -0.65	
EDP1; EDP2; EDP3; REFELV	0.75; 0.5; 0.25; 0.75	
K	1.5E-6	Rosati, Wise, and Kraus 1993
EPS	0.002	Rosati, Wise, and Kraus 1993
LAMM	0.2	Rosati, wise, and Kraus 1993
TEMPC	15	Rosati, Wise, and Kraus 1993
WVTYPE	1	
IWAVE	1	
DTWAVE	5	
IANG	1	
DTANG	5	
DMEAS	5.63	
IRAND	1	
ISEED; RPERC	7878; 20	Rosati, Wise, and Kraus 1993
IELEV	1	
DTELV	60	
D50	0.15	St. Johns County, 1990
DFS	0.3	
BMAX	30	Wise 1994 ¹

¹ Personal Communication, 1994, R. Wise, U.S. Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, Vicksburg, MS.

Table C3
Method III. SBEACH 2.0 Configuration Parameters for Storms
Jan88 and Profile 8 (Rosati, Wise, and Kraus 1993). Input Units:
SI and Minutes

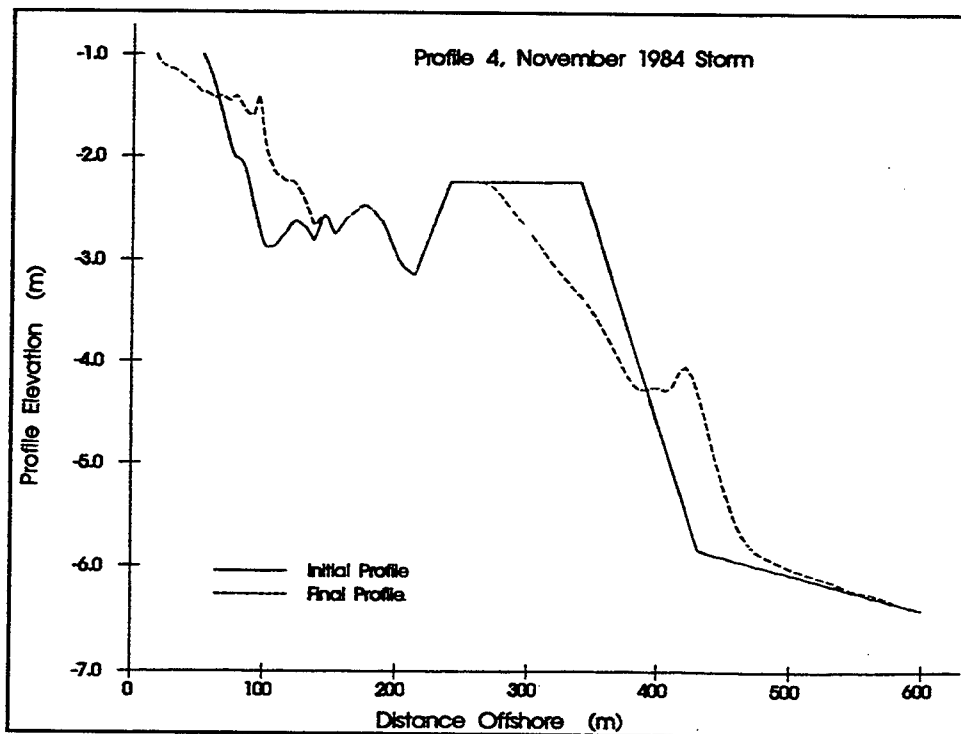
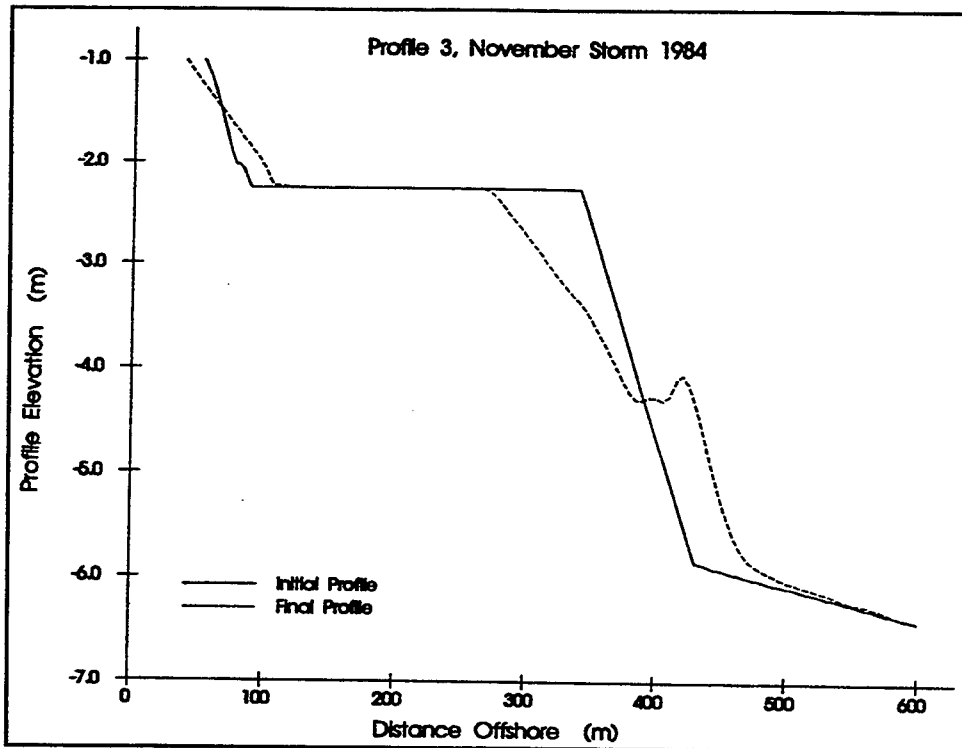
Parameter	Value	Reference
TITLE	Profile 8 JAN88	
UNITS	1	
NDX; XSTART	280; -144.98	
IDX	1	
NGRID	2	
WDXV; NDXV	2, 212; 5, 68	
NDT; DT	1584; 5	
ELV1; ELV2; ELV3	0.75; 0; -0.65	
EDP1; EDP2; EDP3; REFELV	0.75; 0.5; 0.25; 0.75	
K	1.5E-6	Rosati, Wise, and Kraus 1993
EPS	0.002	Rosati, Wise, and Kraus 1993
LAMM	0.2	Rosati, Wise, and Kraus 1993
TEMPC	15	Rosati, Wise, and Kraus 1993
WVTYPE	1	
IWAVE	1	
DTWAVE	5	
IANG	1	
DTANG	5	
DMEAS	5.63	
IRAND	1	
ISEED; RPERC	7878; 20	Rosati, Wise, and Kraus 1993
IELEV	1	
DTELV	5	St. Johns County, 1990
D50	0.15	
DFS	0.3	
BMAX	30	Wise 1994 ¹

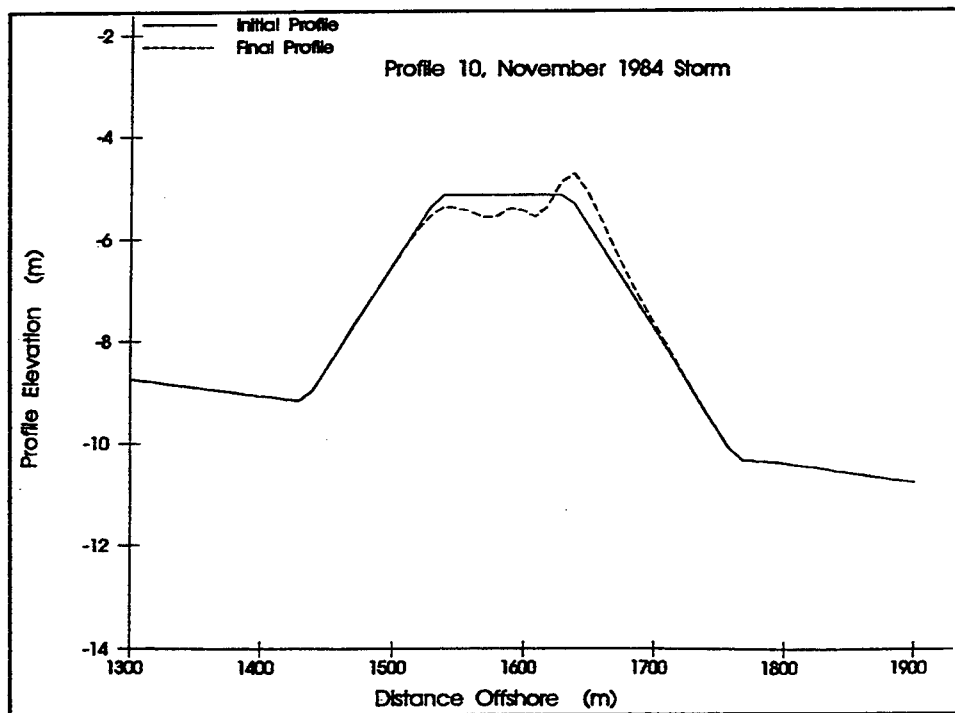
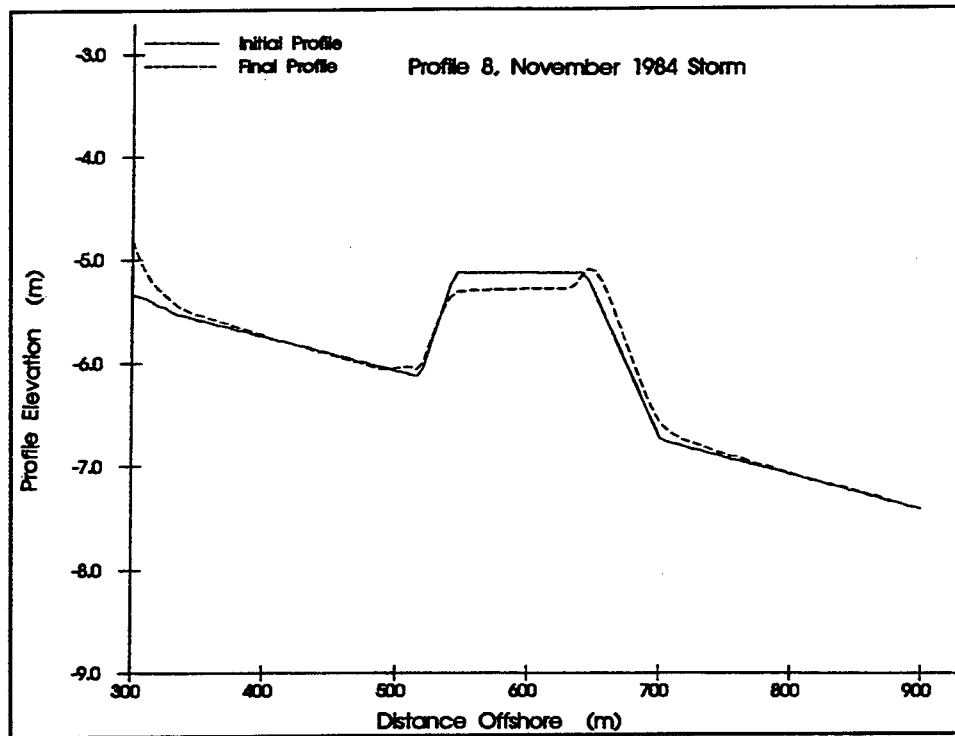
¹ Personal Communication, 1994, R. Wise, U.S. Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, Vicksburg, MS.

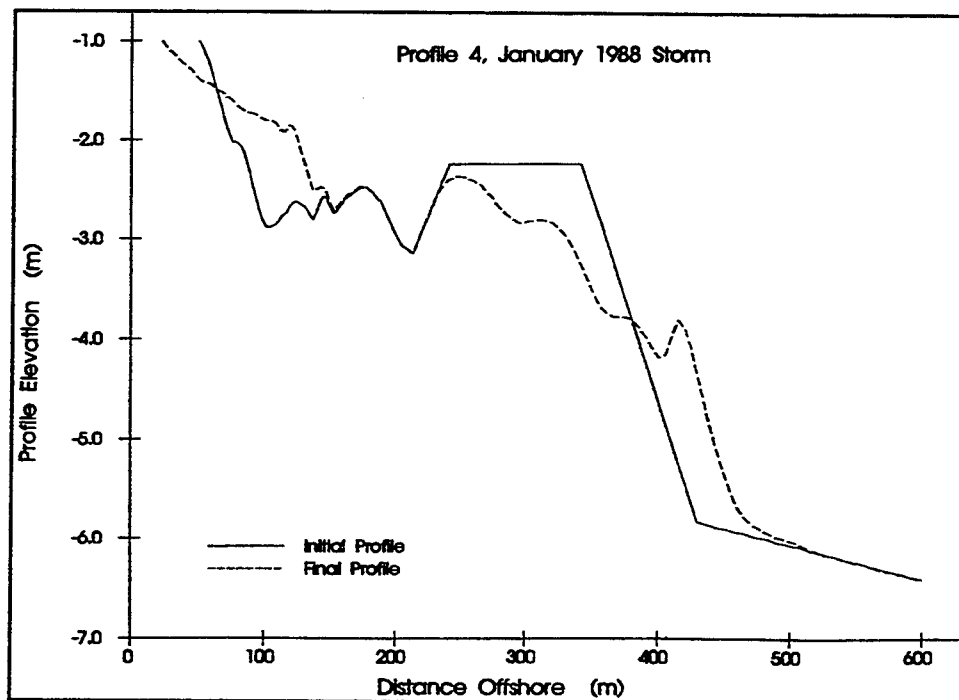
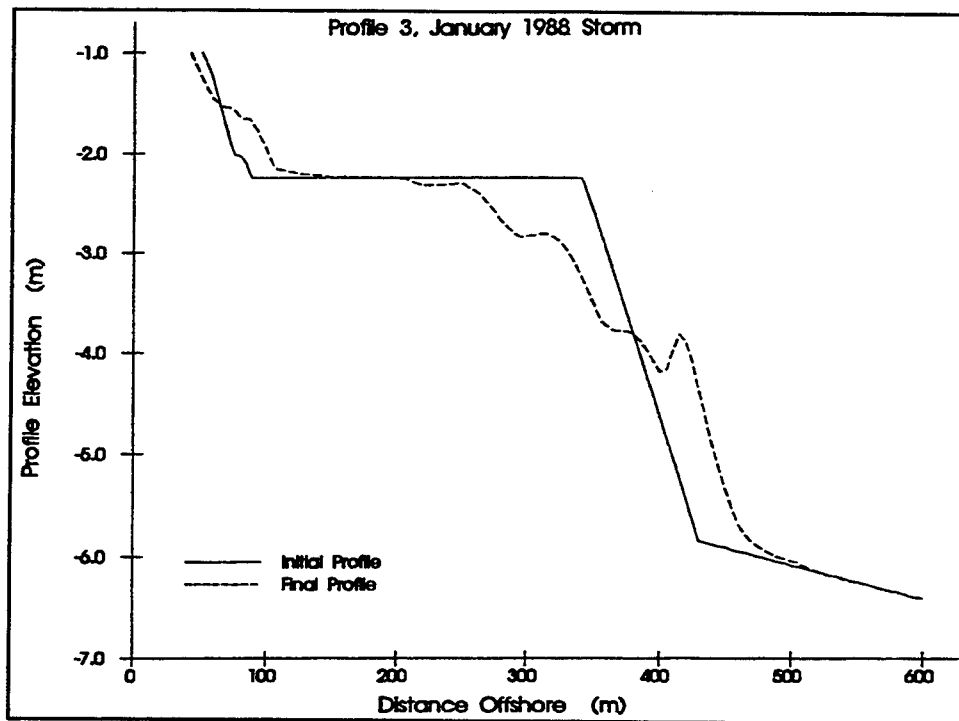
Appendix D

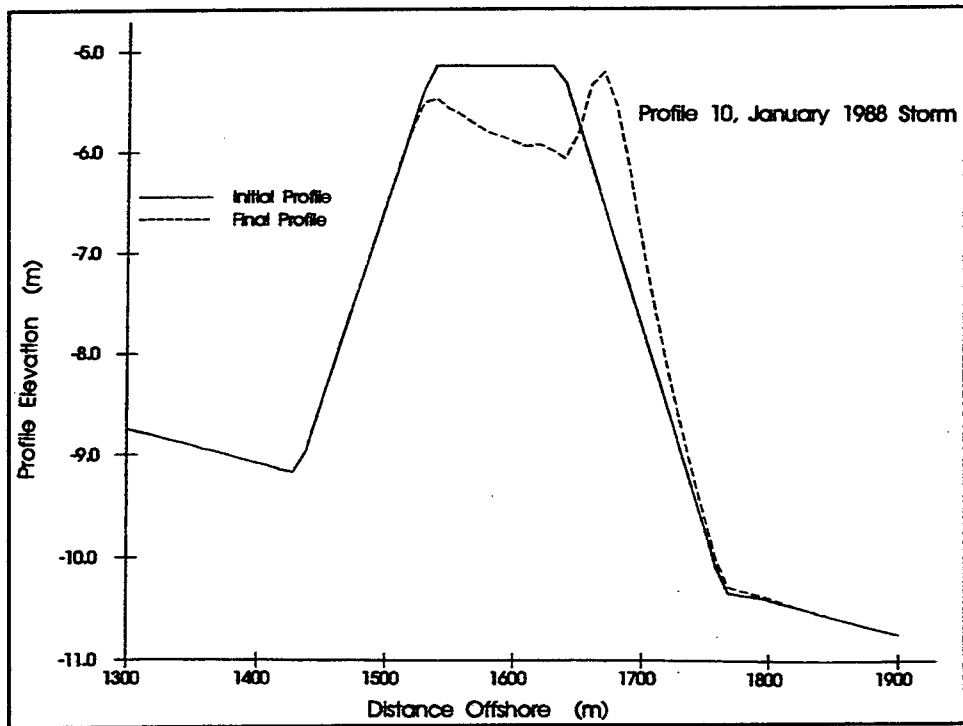
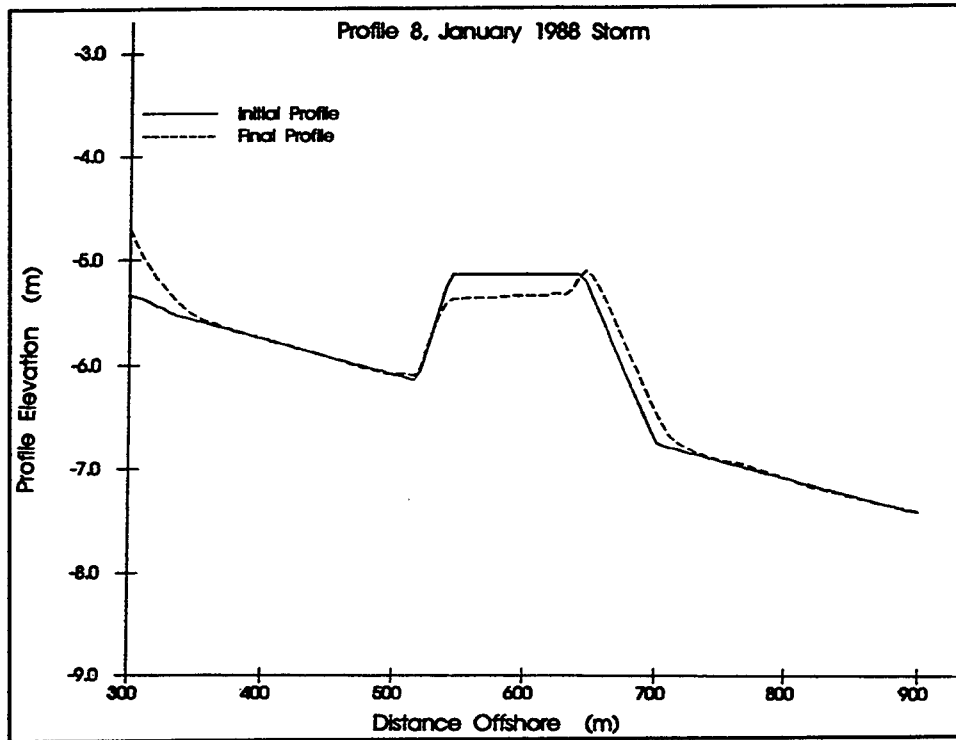
Storm-Event-Associated Nearshore-Berm Profile Response: SBEACH 2.0

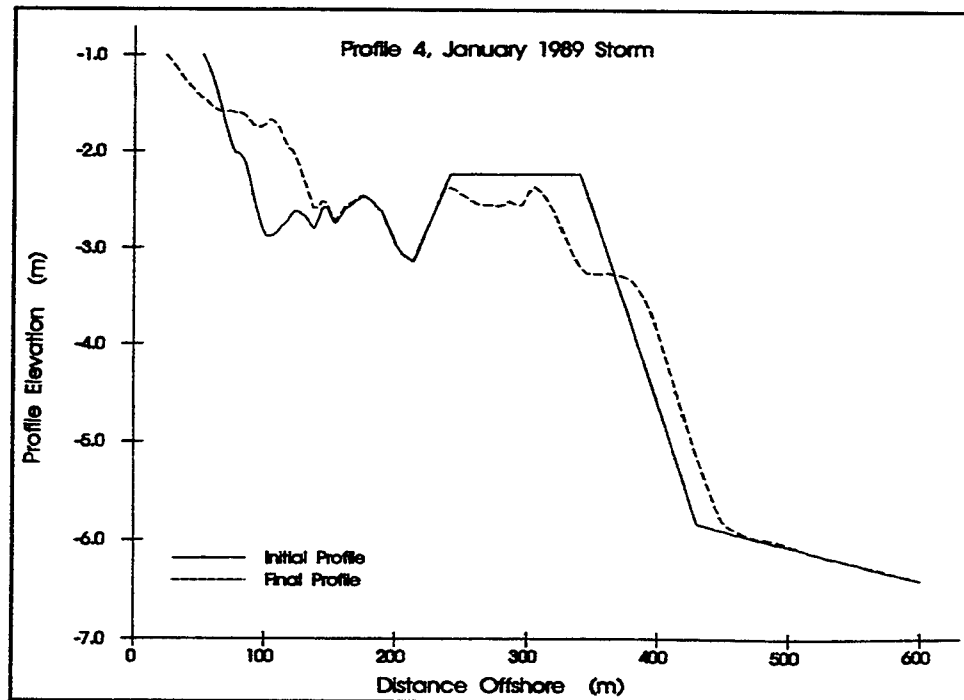
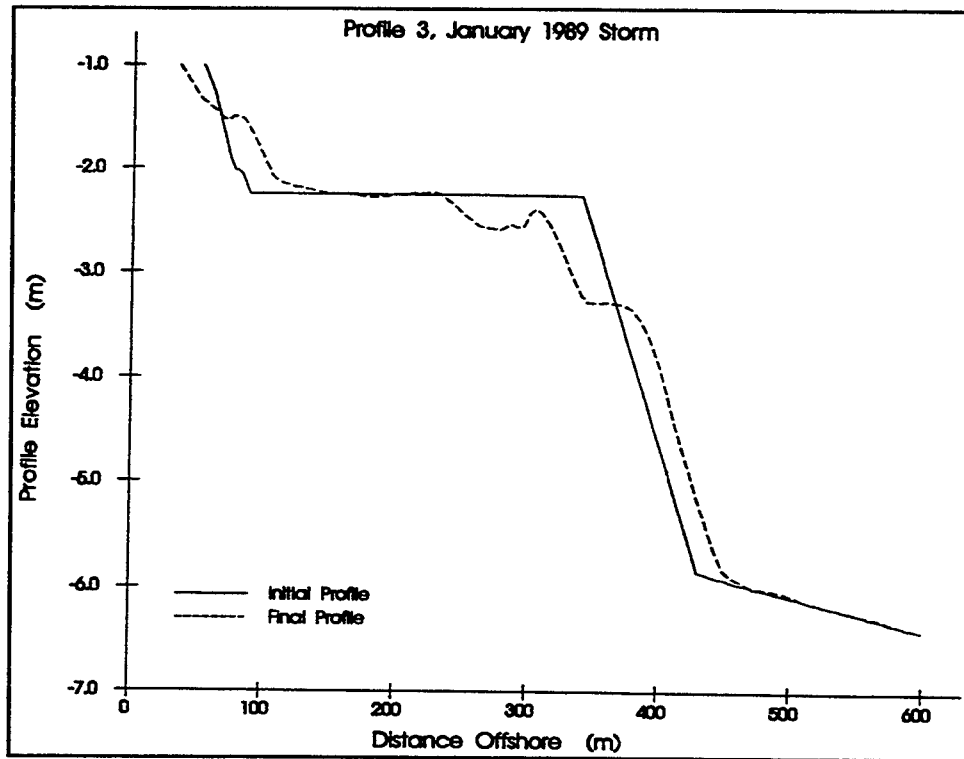
Profile elevations in meters National Geodetic Vertical Datum.

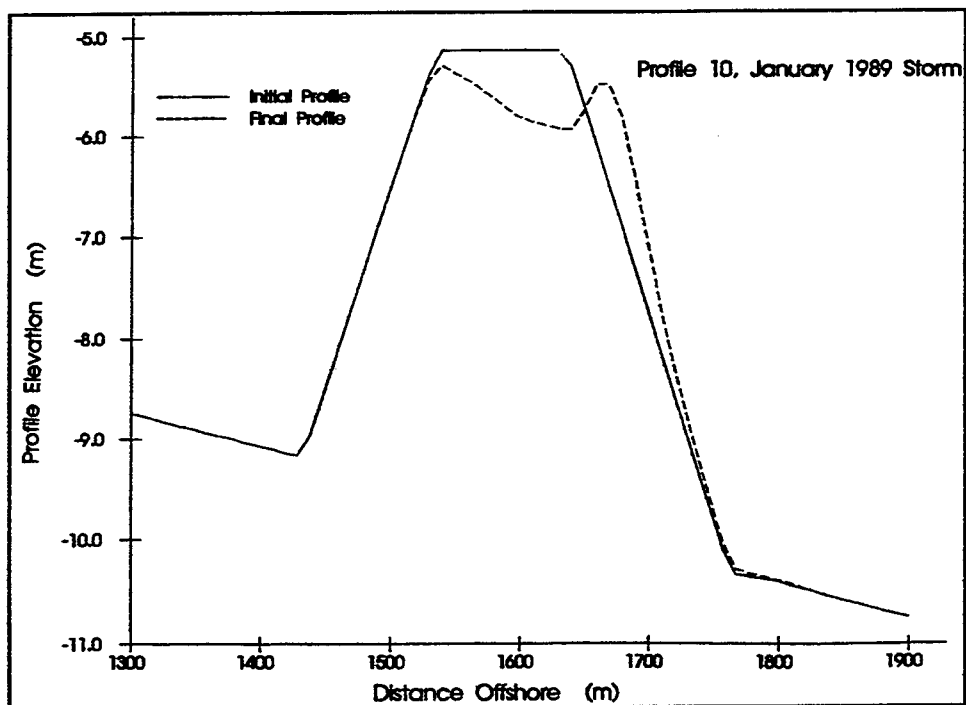
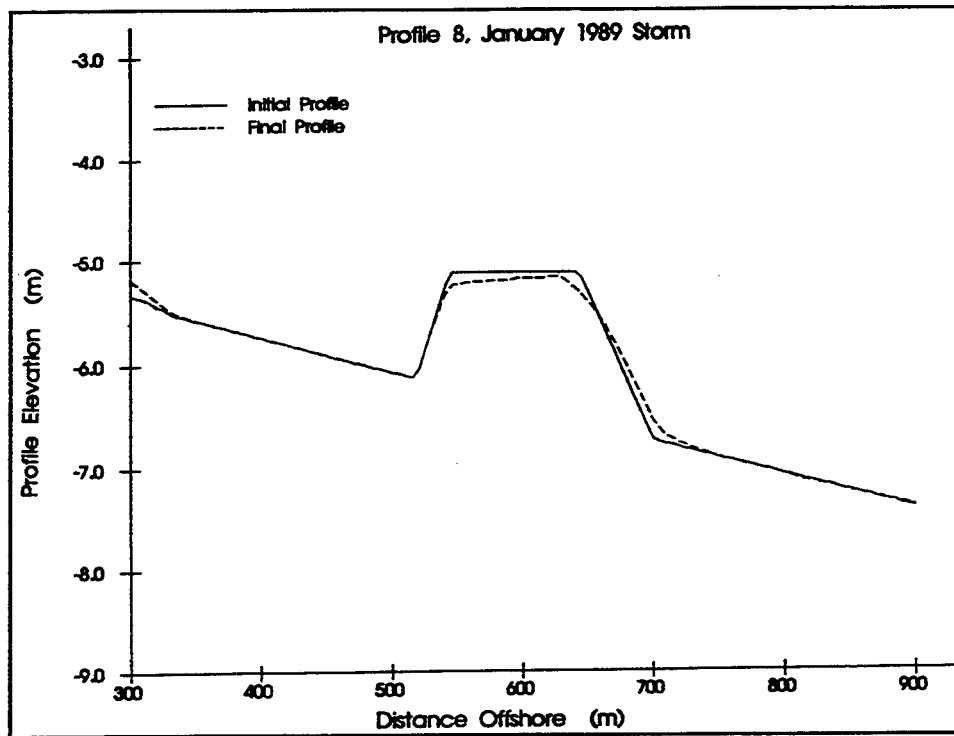


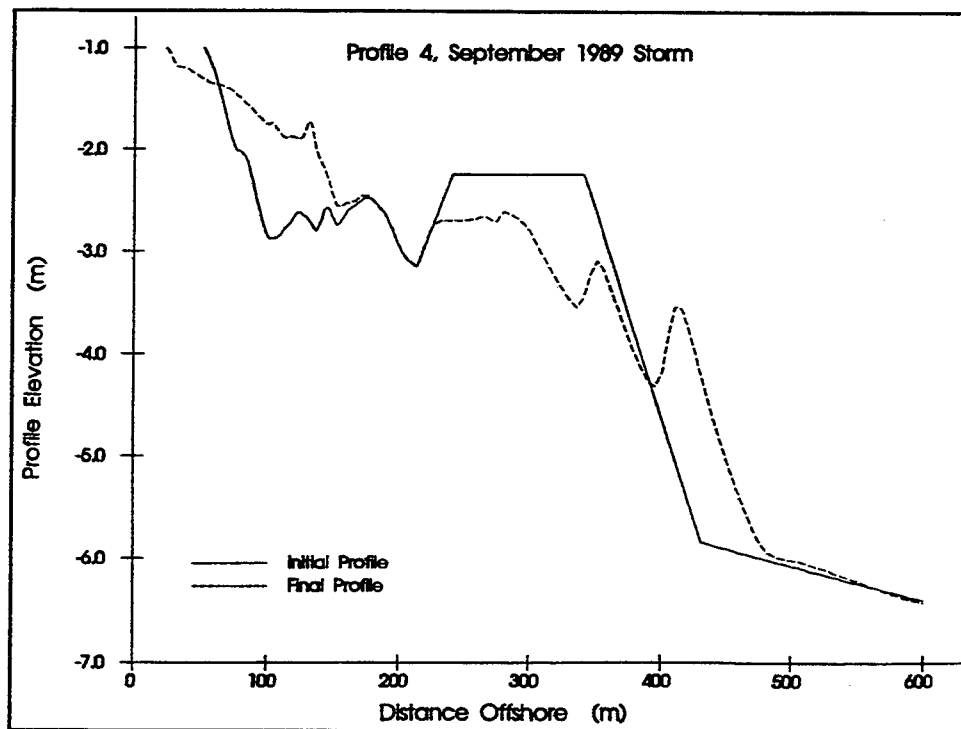
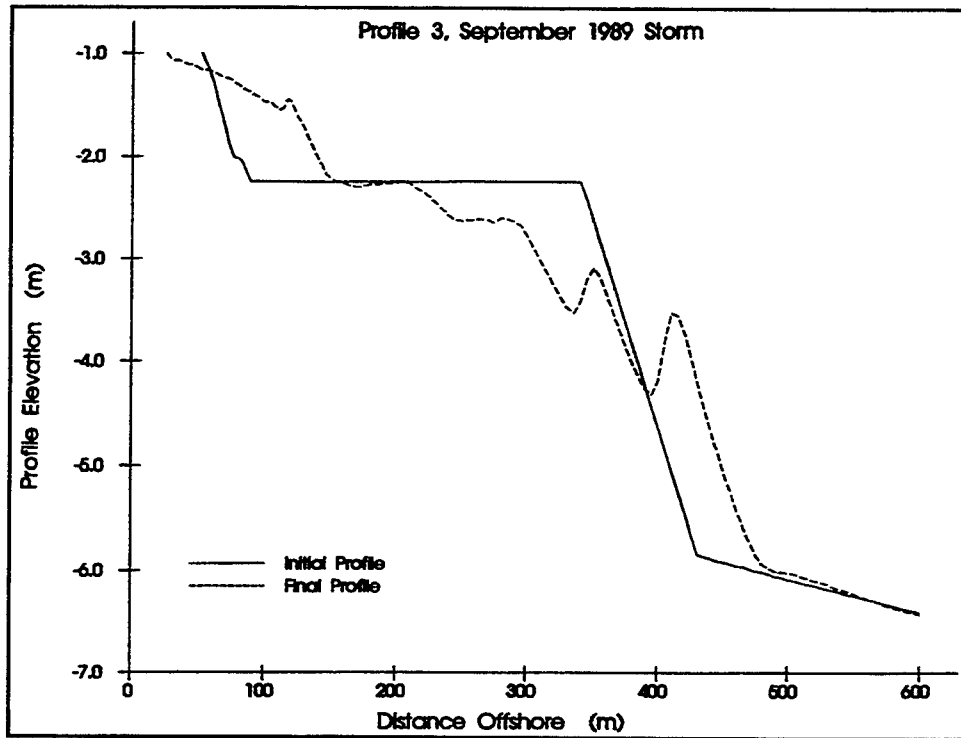


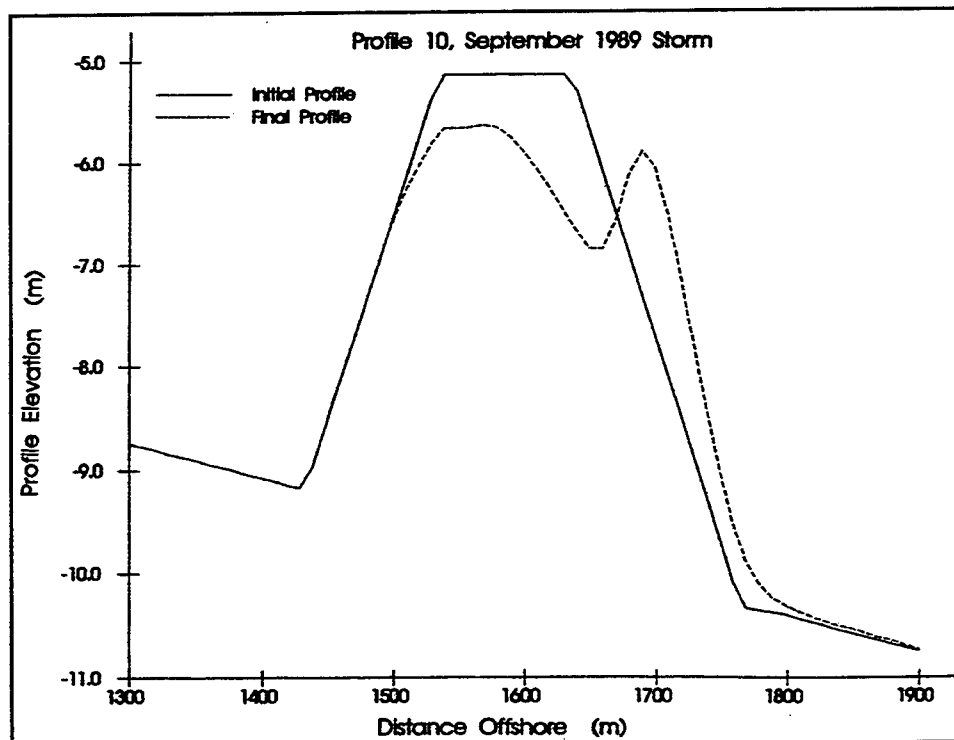
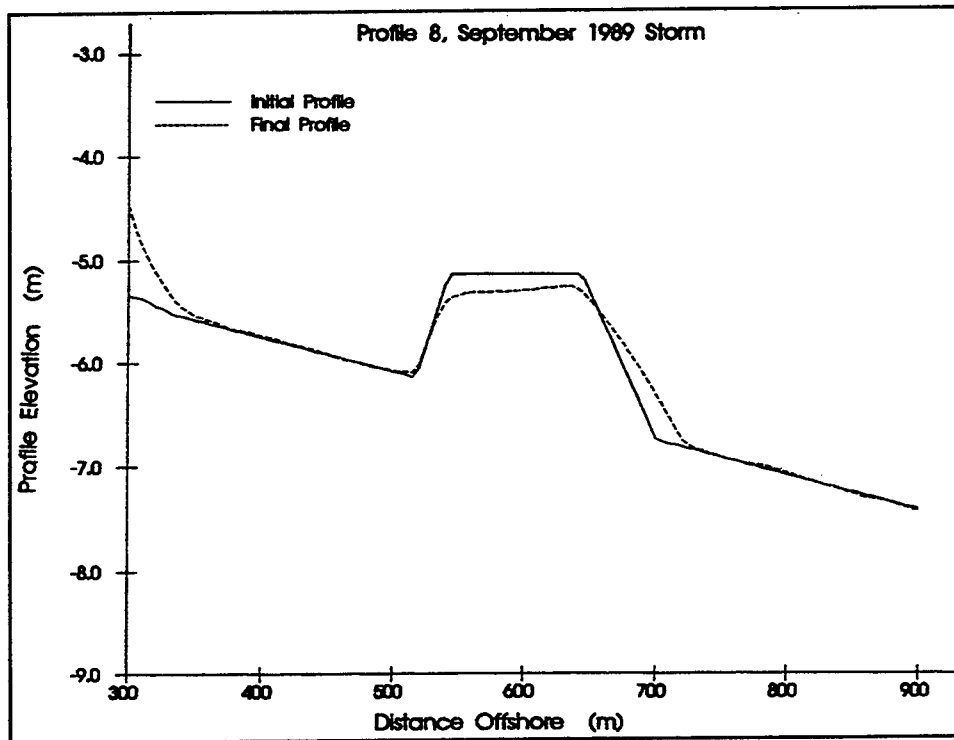


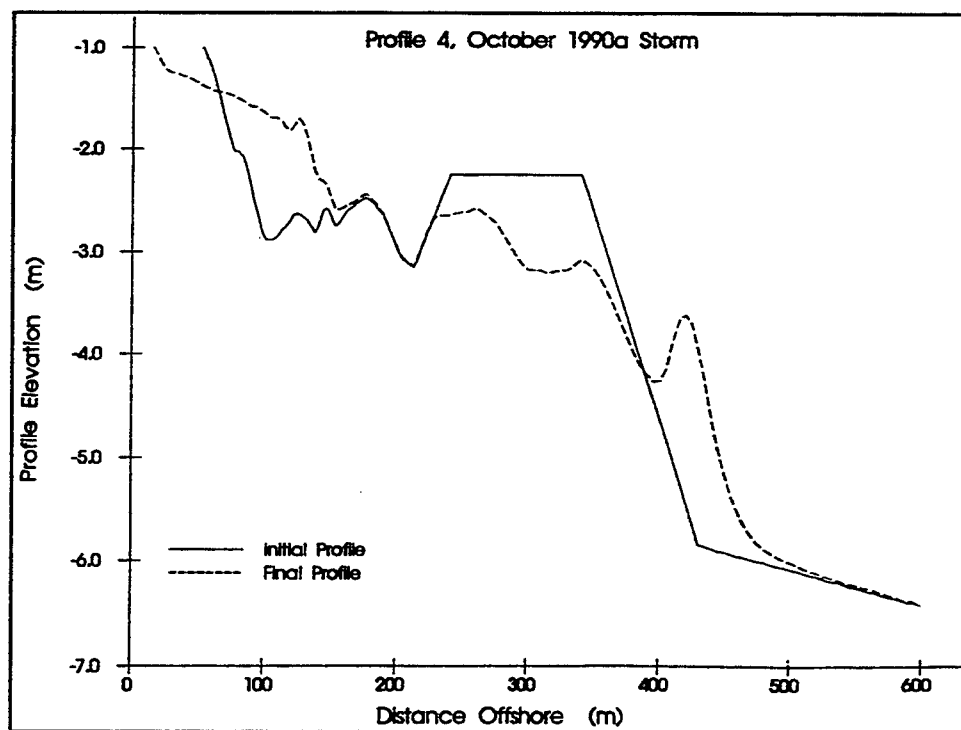
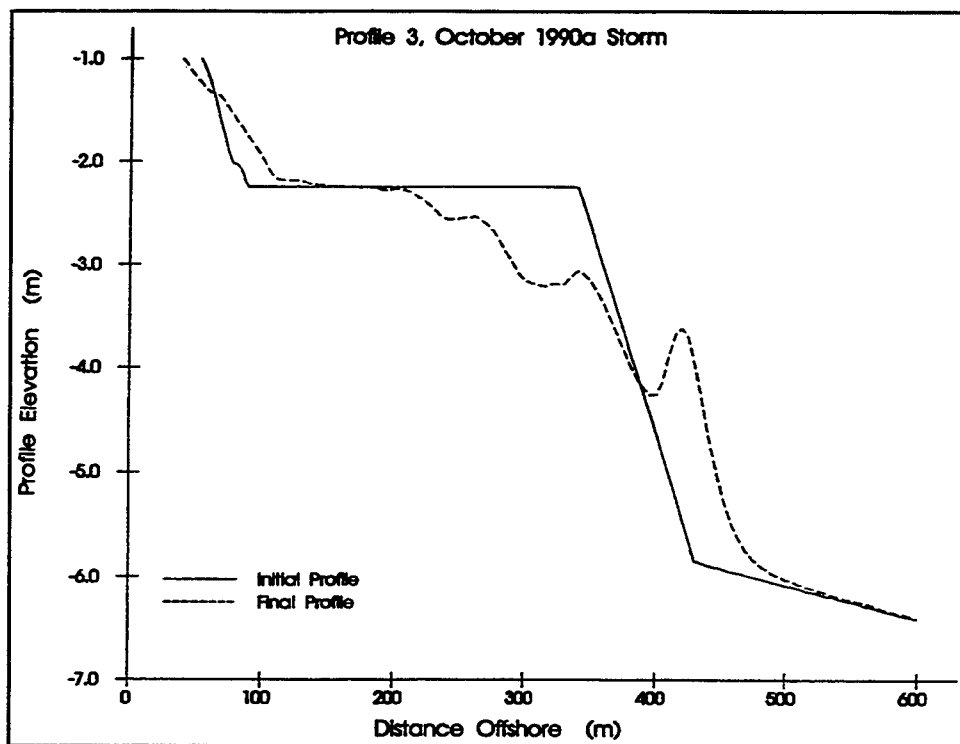


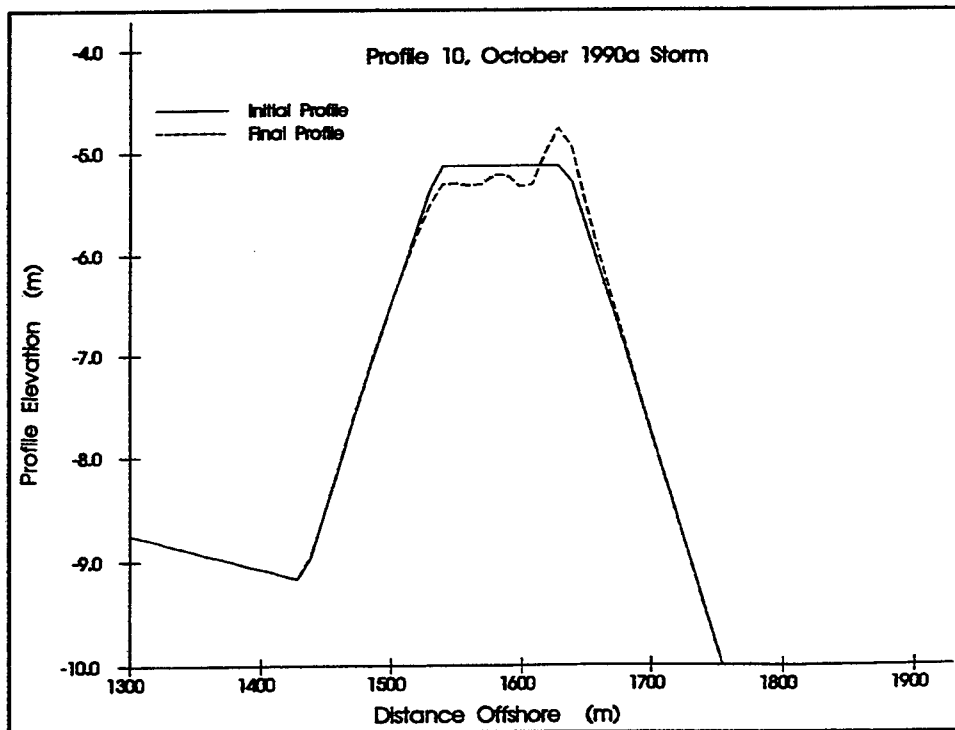
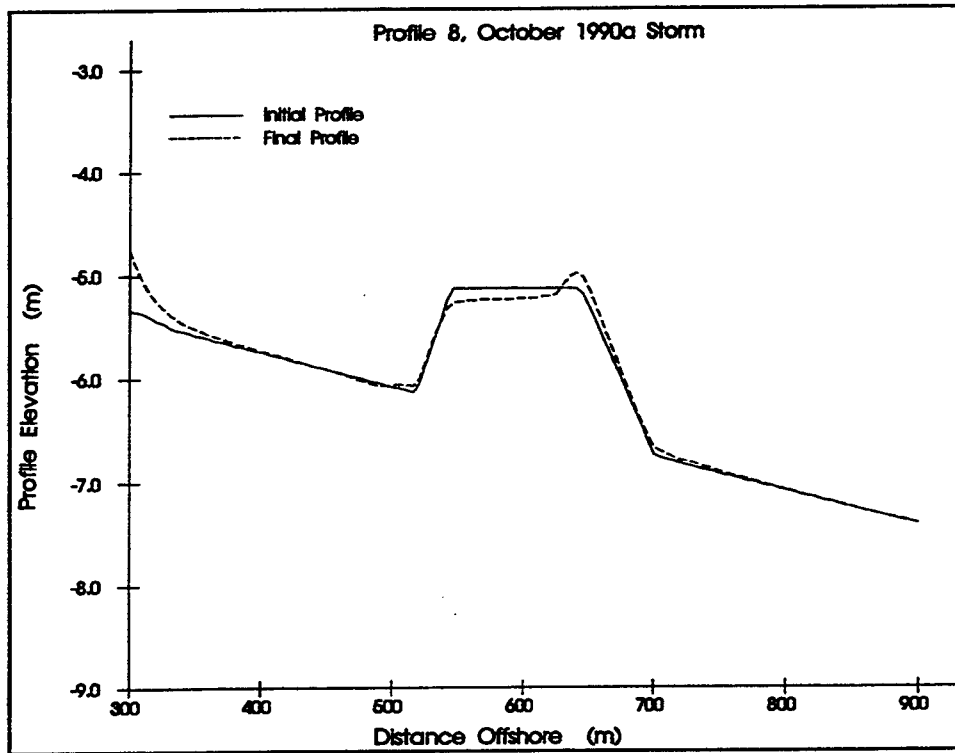


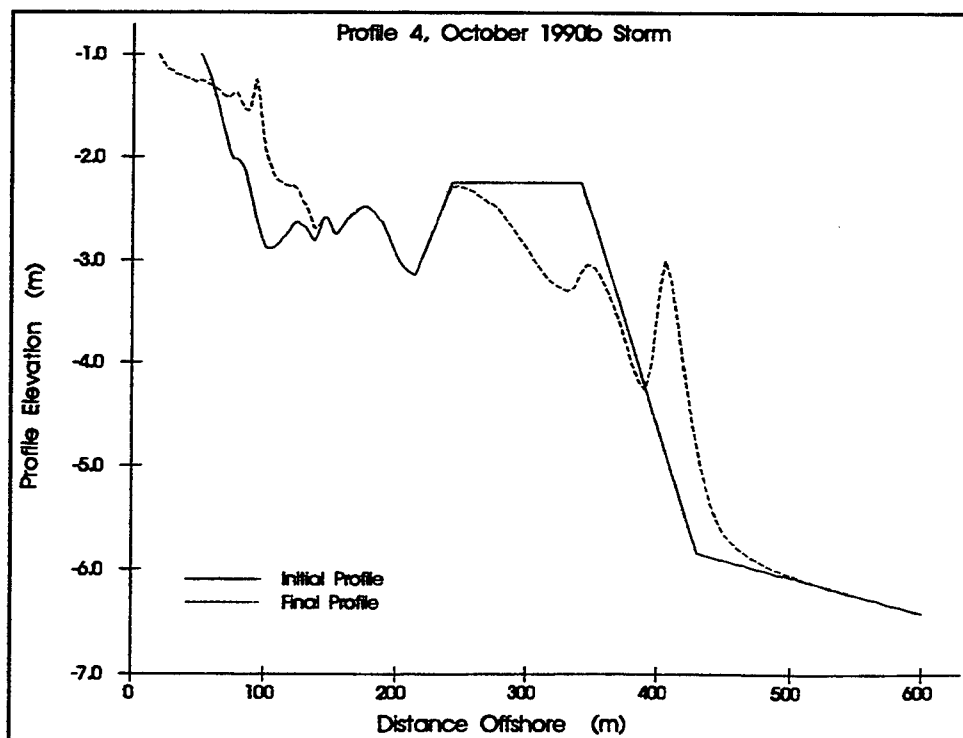
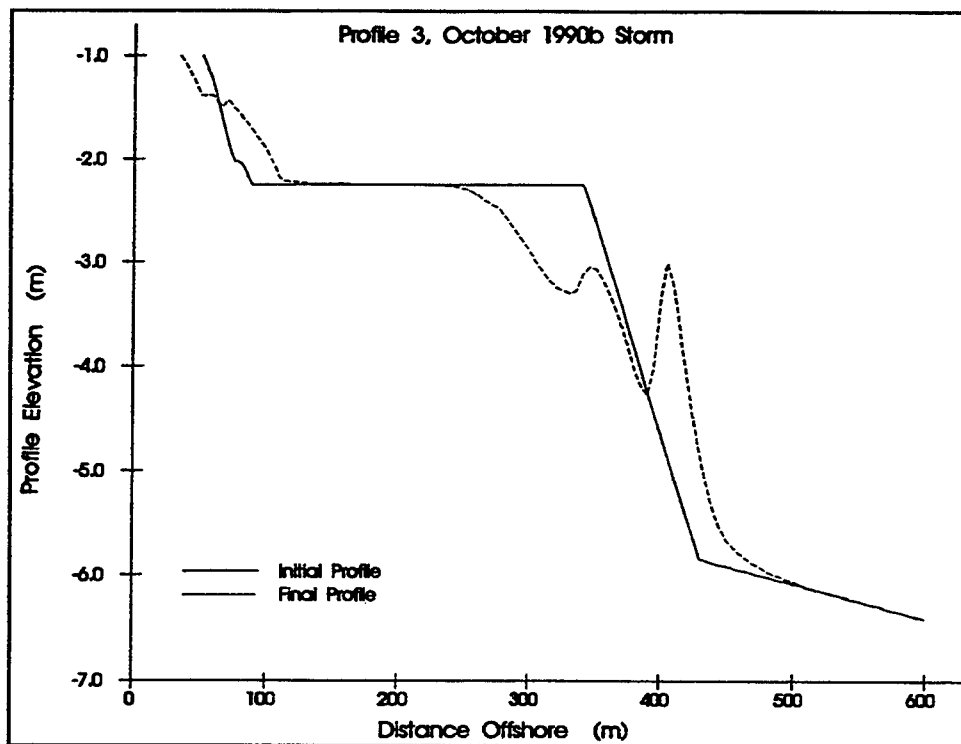


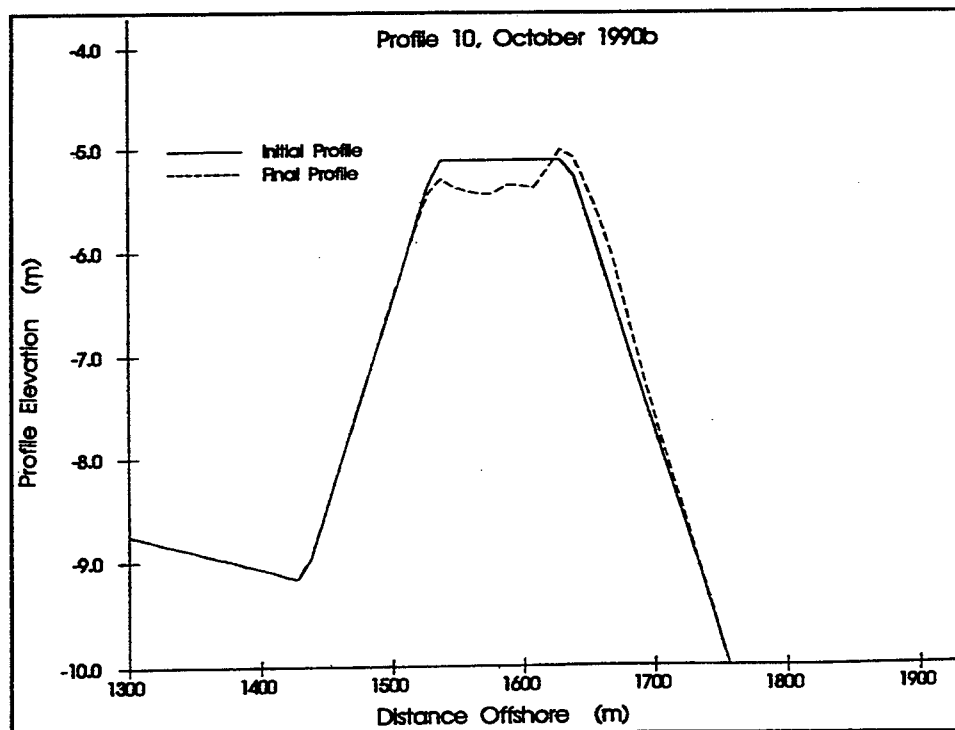
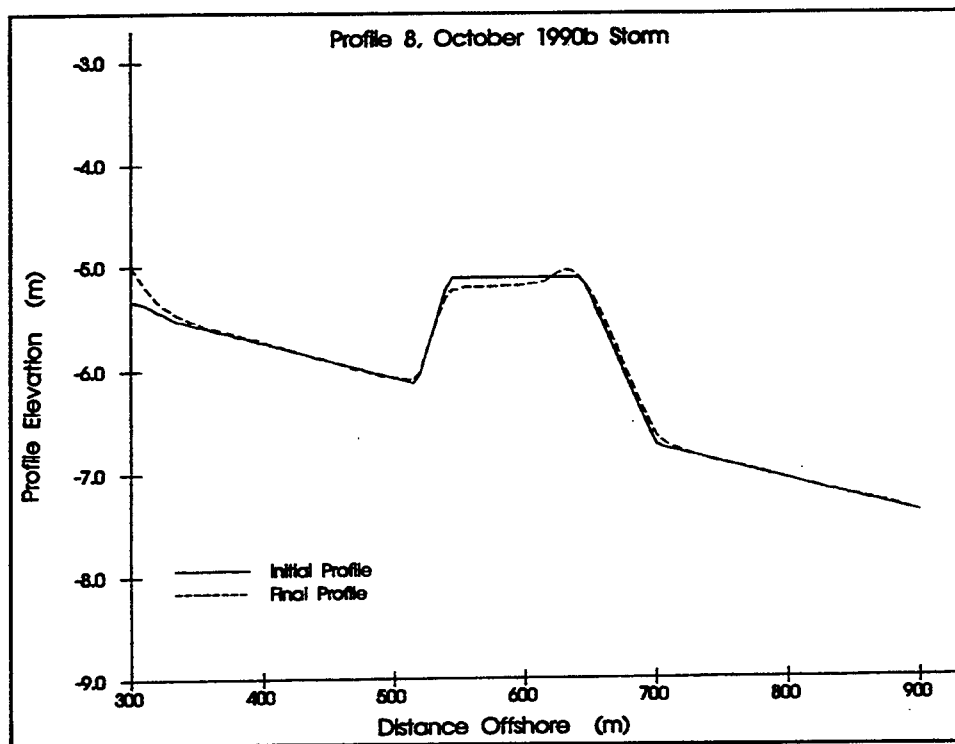


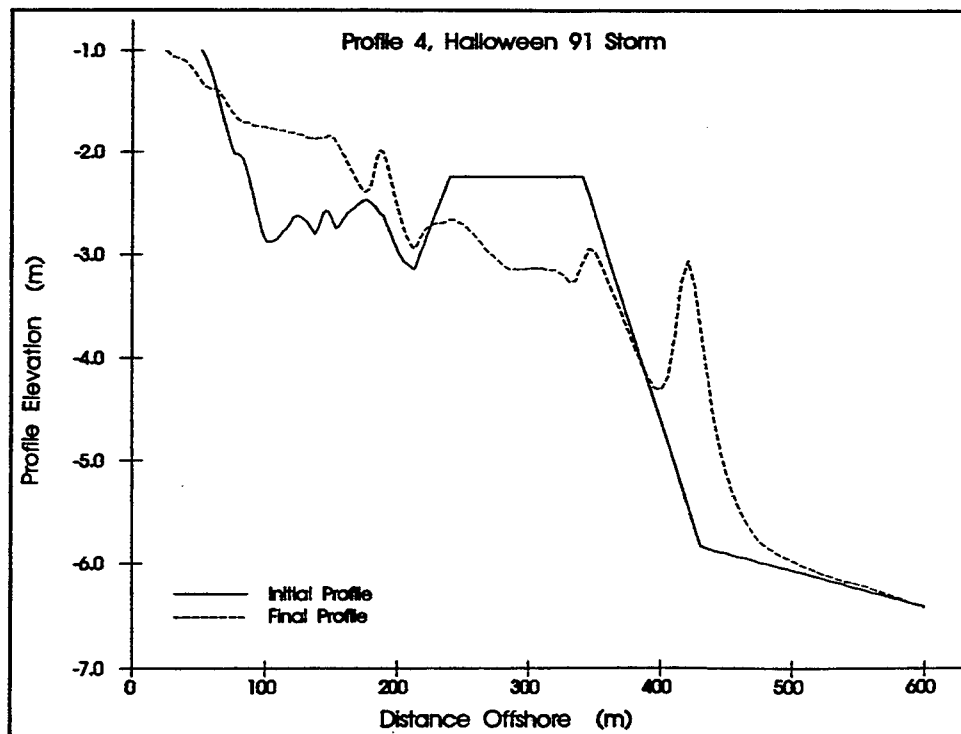
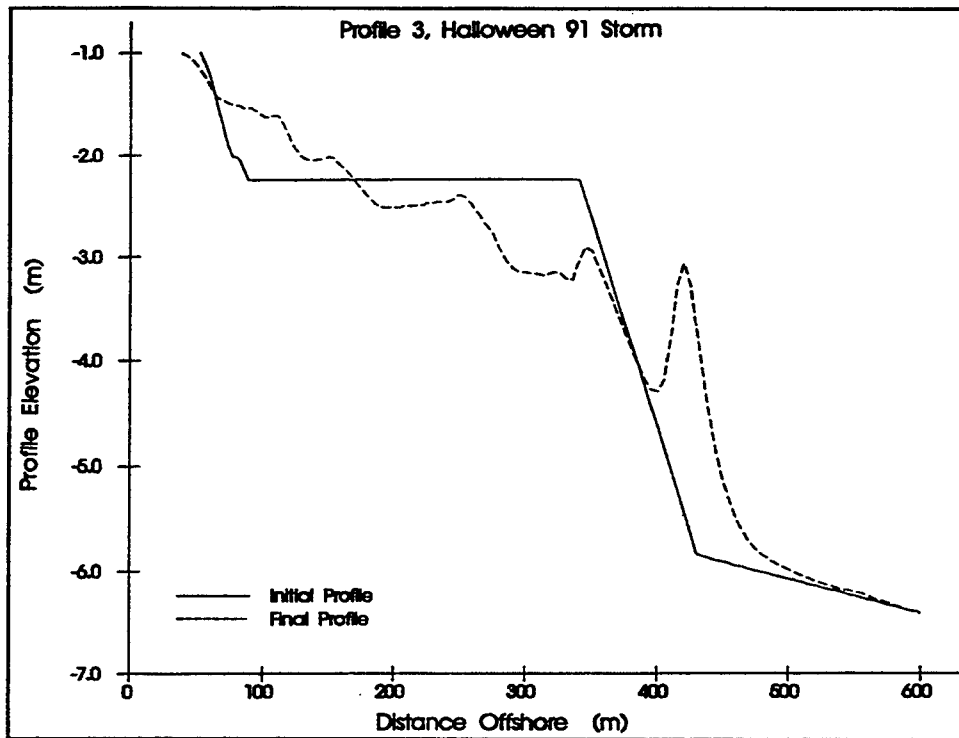


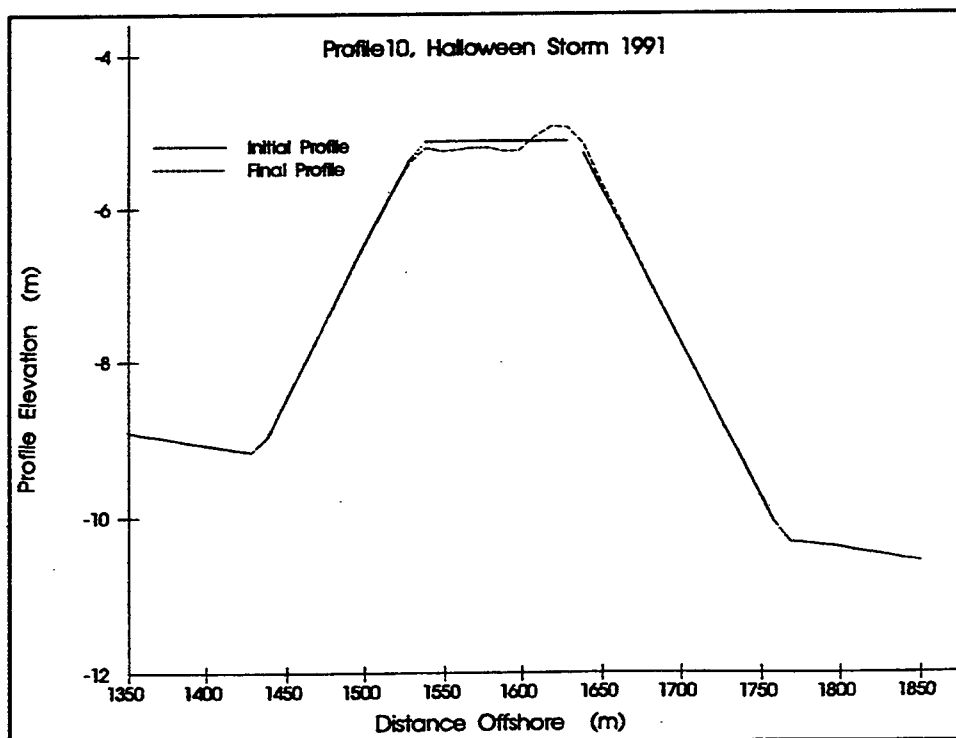
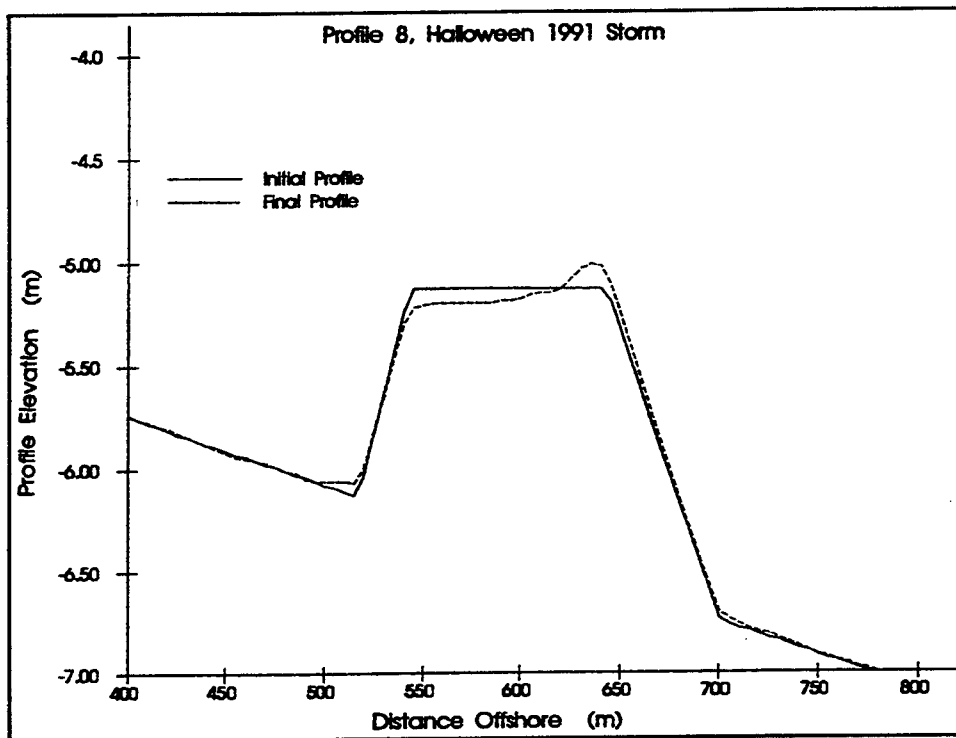


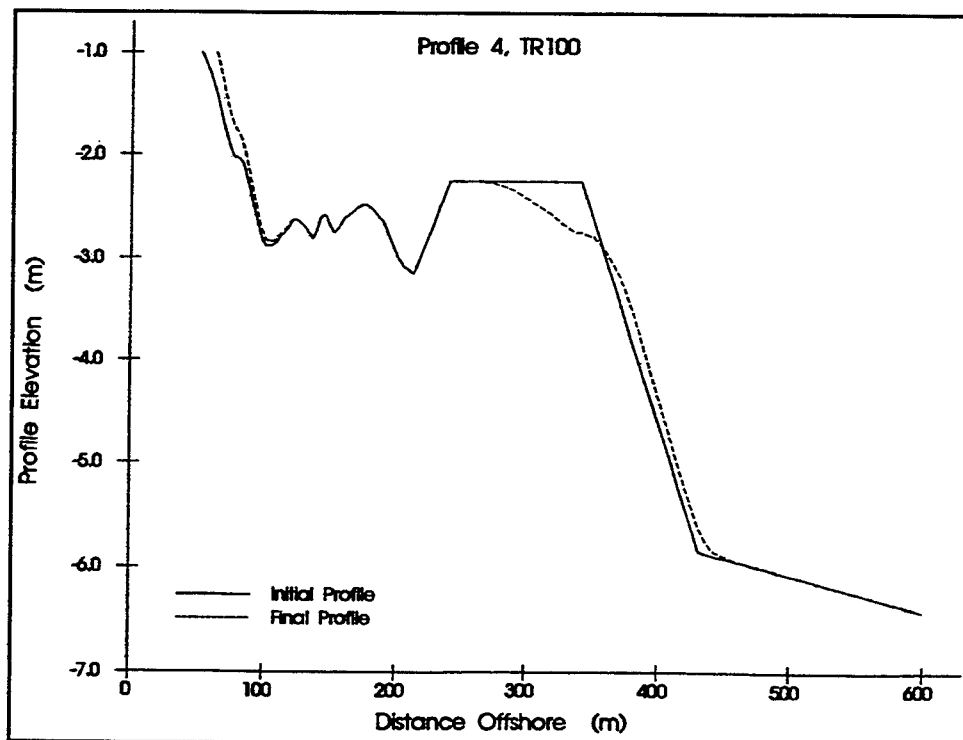
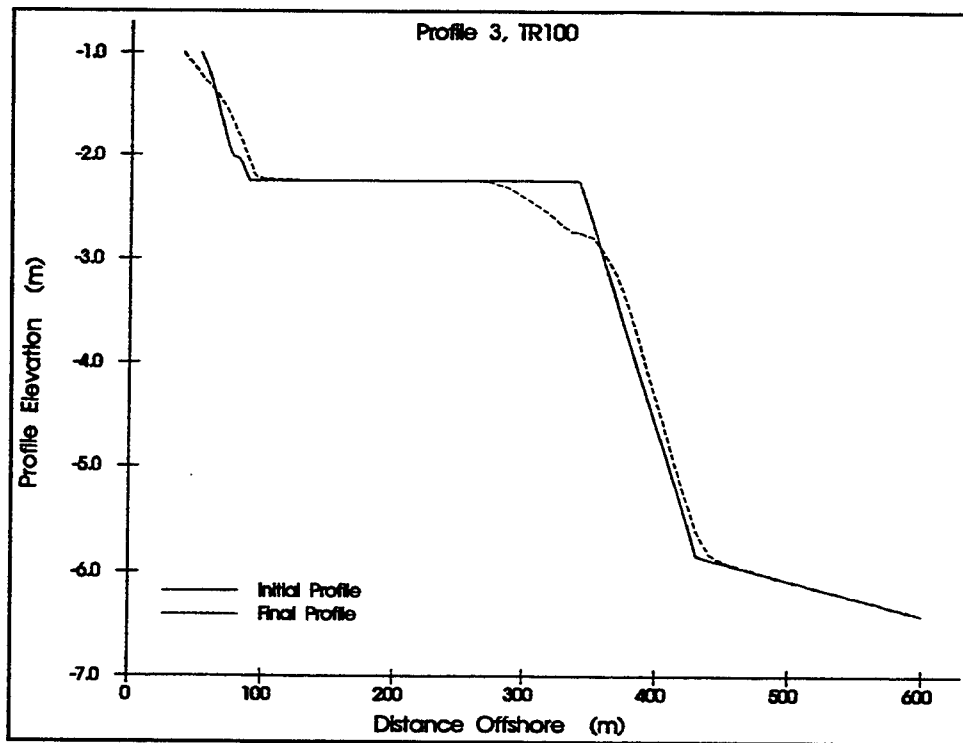


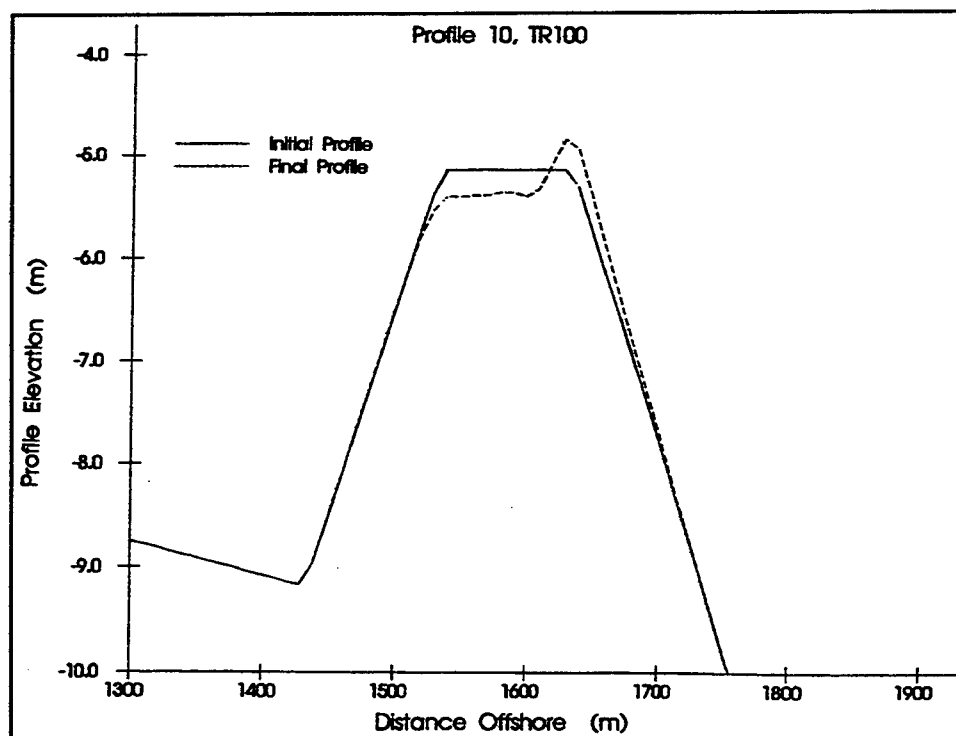
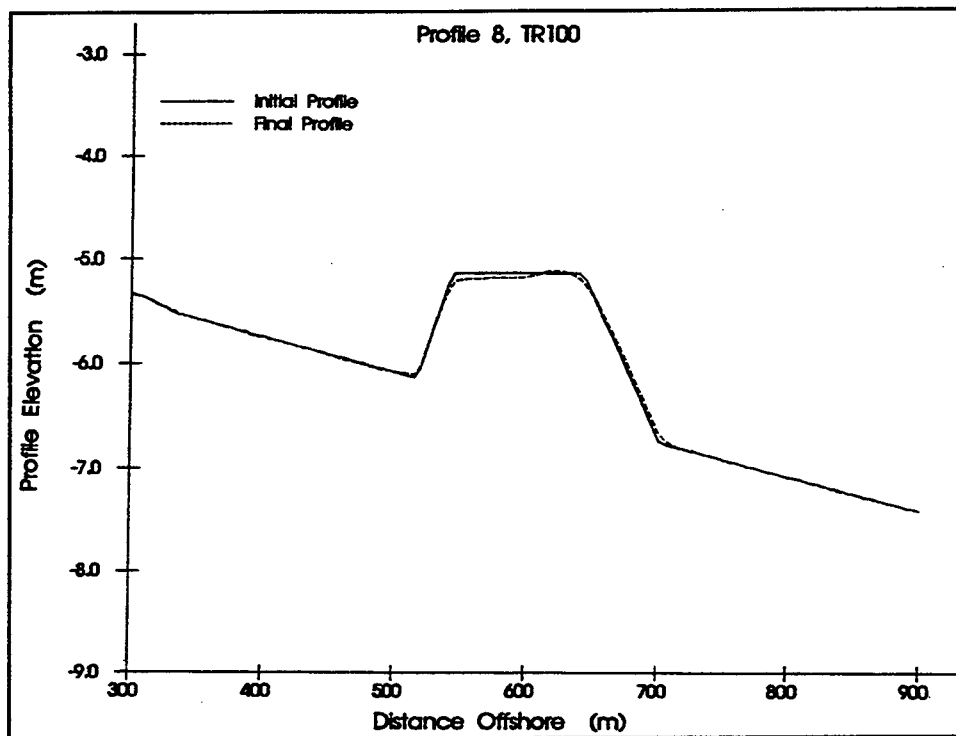


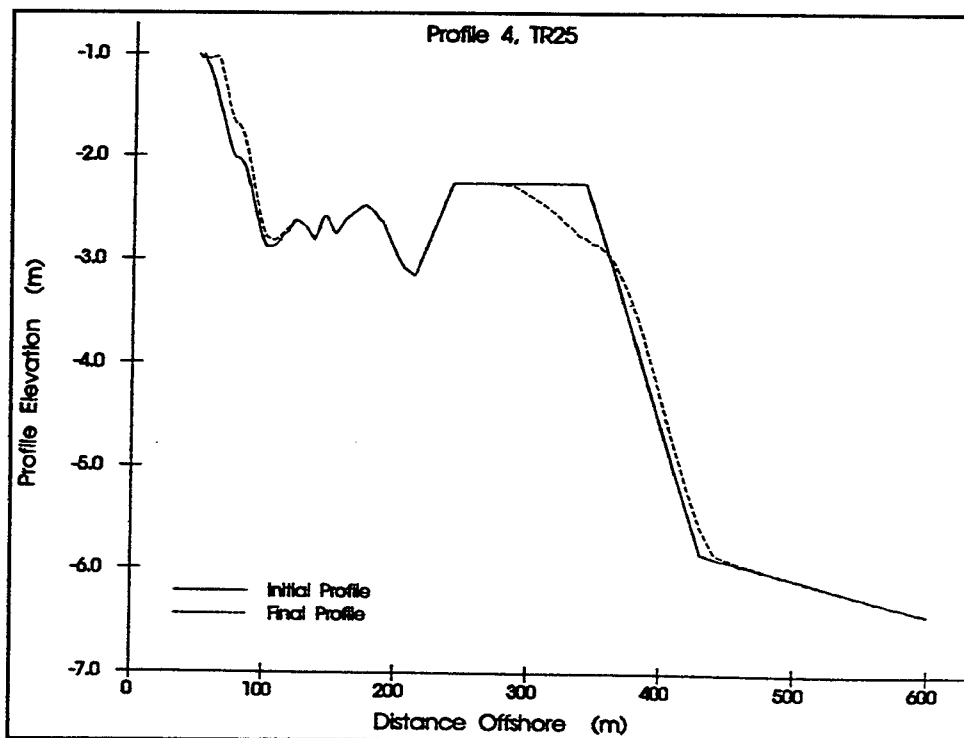
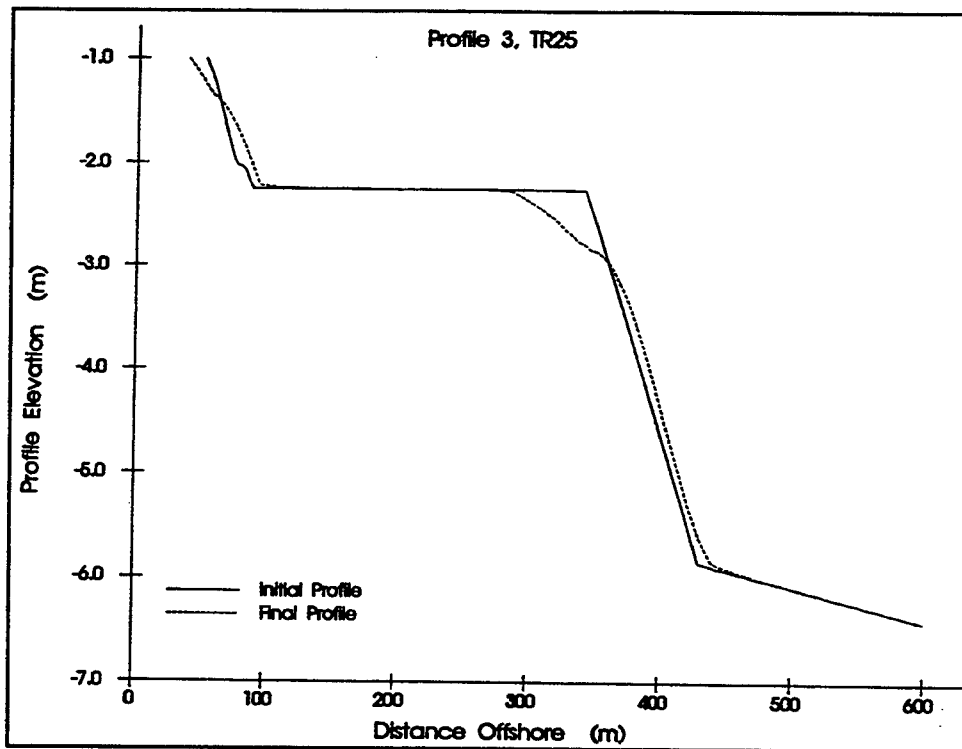


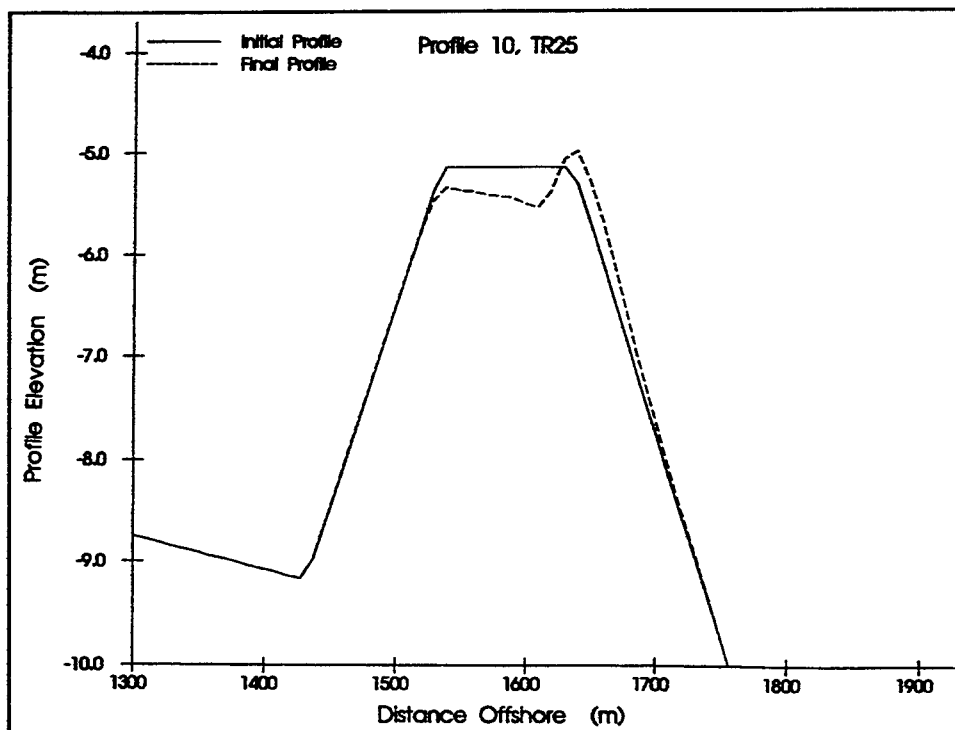
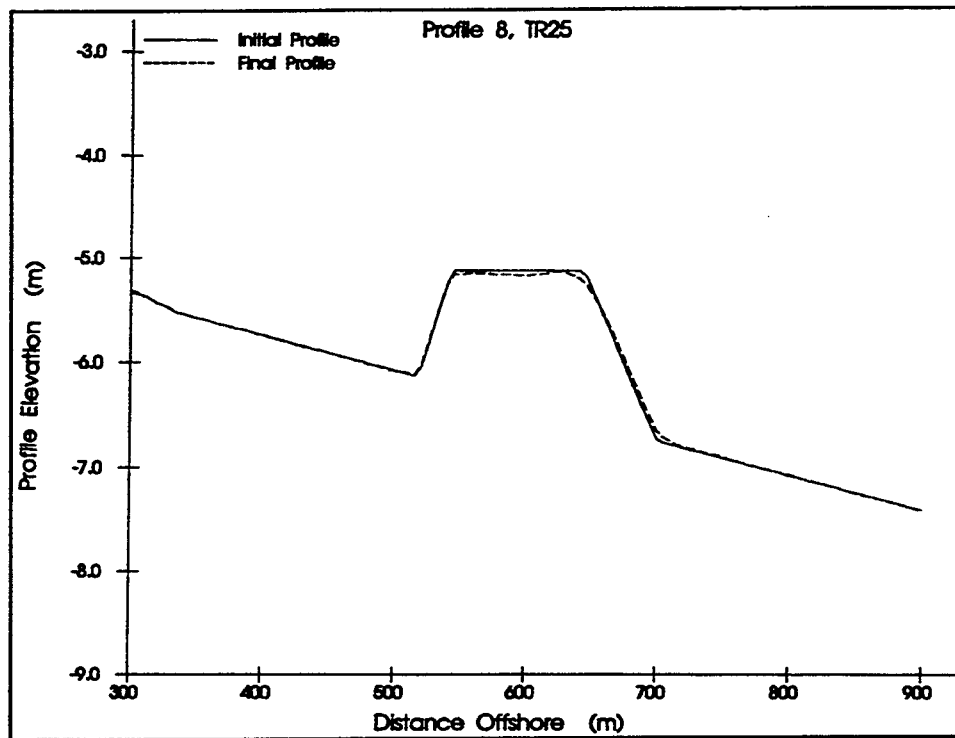


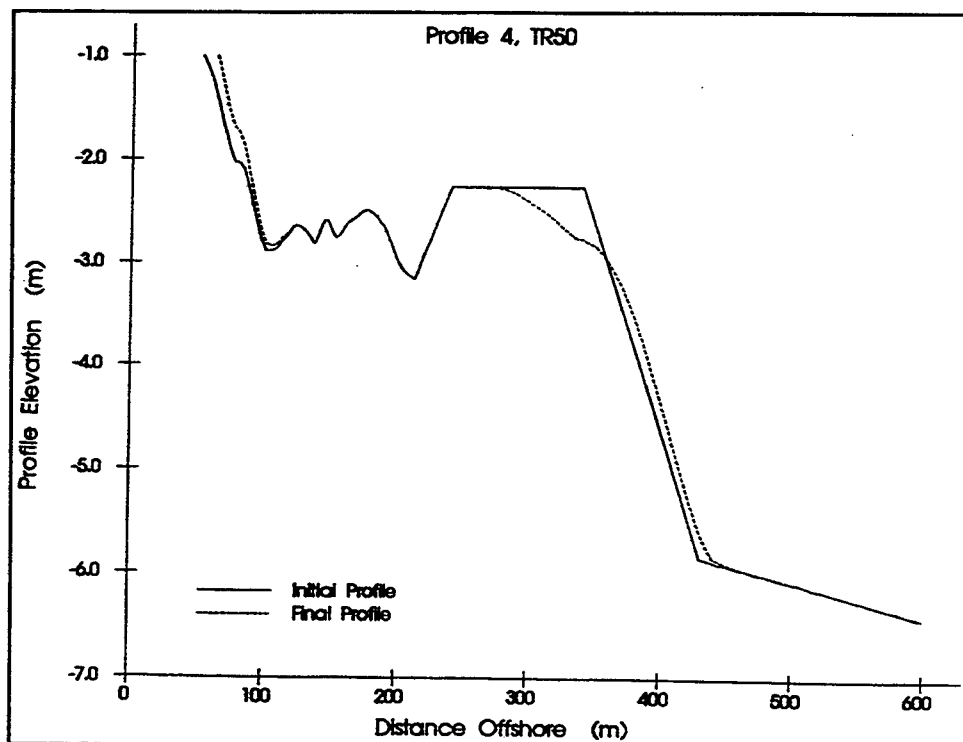
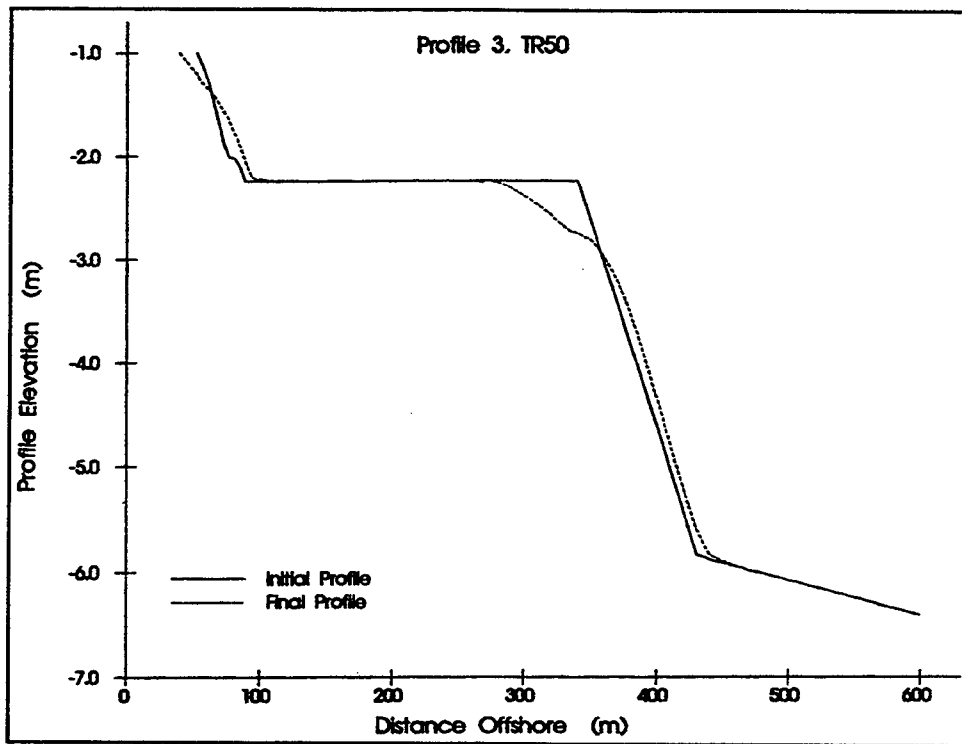


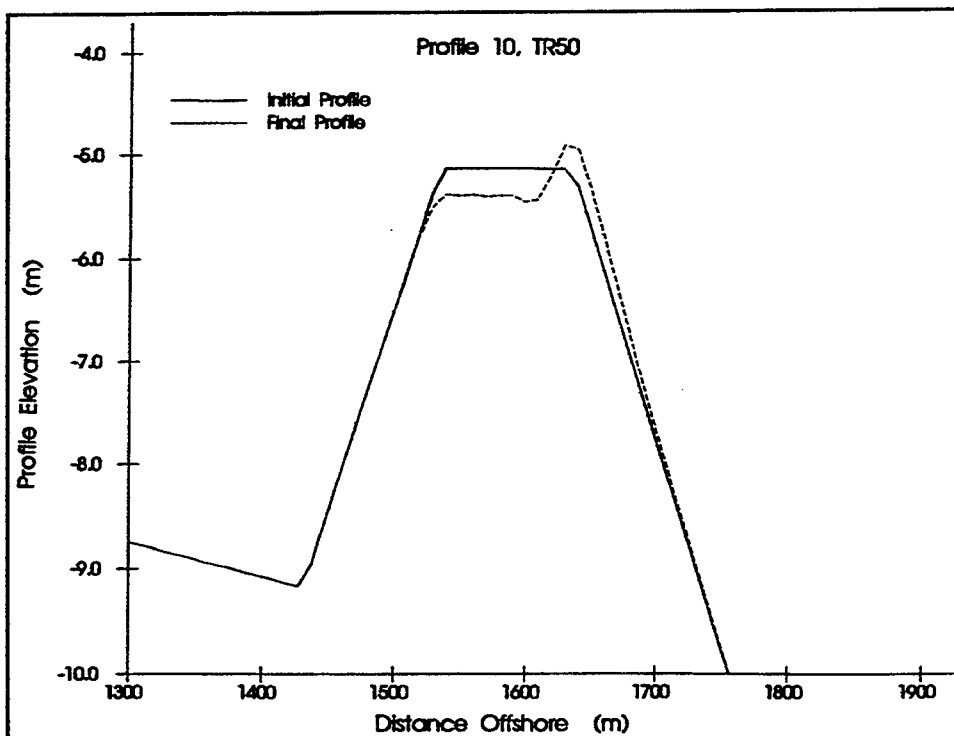
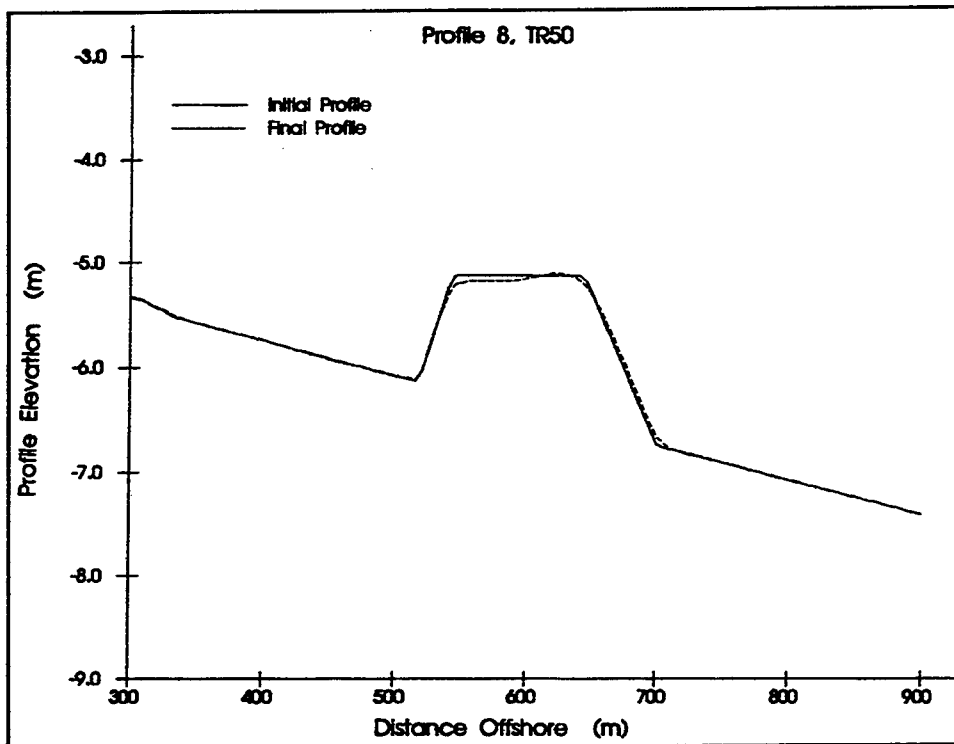












Appendix E

Storm-Event-Associated Nearshore-Berm Profile Response: LTFATE

Longterm-Stability Predictions for Nearshore-Berm Design Templates: Profiles 8 and 10

Stability and renourishment requirements were predicted for two nearshore berm configurations. A numerical model was used to predict the stability (berm movement) for each berm alternative plan in terms of the magnitude and direction of net berm movement. The applicable numerical model used in this investigation was the LongTerm FATE (LTFATE) model, which was developed under the U.S. Army Corps of Engineers, Dredging Research Program (Scheffner et al. 1995).¹

Numerical Methods

LTFATE is a PC-driven two-dimensional coupled hydrodynamic and bathymetric change numerical model. LTFATE simulates sediment transport on bathymetric features due to the combined effect of waves and currents. The model also simulates sediment avalanching, due to sediment transport, and self-weight consolidation for cohesive sediments. LTFATE has been used at several varying application sites with consistent results in each case. The model is limited to locations seaward of the surf zone.

Input to the LTFATE model includes time series representations for waves, water surface elevations, and currents. Together, these data define the forcing environment for the bathymetric feature of interest. When applying LTFATE, the

¹ References cited in this appendix are located at the end of the main text.

sediment (berm) material is characterized by median grain size (D_{50}) and cohesive material (silt or clay) content. The forcing environment used to assess berm stability consisted of average annual conditions at the proposed berm construction site and a suite of six site-specific storm scenarios.

The amount of material (sediment) transported off each berm alternative plan was estimated for the suite of forcing environments by calculating a volume difference between the baseline condition (prestorm) and the poststorm condition. This volume difference represented the renourishment required for a given storm event or average annual condition.

Other than ensuring that the material parameters are specified within reasonable limits of the actual case, the most important parameter affecting sediment transport in LTFATE is the current regime. The larger the depth-averaged current, the higher the rate of sediment transport (mound movement) in the direction of the current.

Physical Parameters

The two berm alternative plans are similar in terms of crest elevation and cross-section geometry. The only significant difference is due to Profile 10 being sited at a location further offshore than Profile 8 (600 and 1,600 m), where the water depth is approximately 3 m deeper. The two alternative berm plans are described in Table E1:

Table E1 Nearshore Berm Design Parameters		
Parameter	Profile 8	Profile 10
D_{50}	0.16 mm	0.16 mm
Crest Elevation	-5 m NGVD	-5 m NGVD
Crest Width	100 m	100 m
Construction Side Slope	1V:25H	1V:25H
Steepest Permitted Side Slope (Simulated)	1V:24H	1V:24H
Post-avalanche Side Slope (Simulated)	1V:38H	1V:38H
Base Elevation	-6.4 m NGVD	-9.5 m NGVD
Prototype Berm Length	3962 m	3962 m
Model Berm Length	762 m	762 m
Note: NGVD = National Geodetic Vertical Datum.		

Modeling Simplification

Although planned for application on a bathymetric gradient of 1H:300V, both berms were modeled on a flat bottom. This simplification was required to avoid corruption of boundary conditions in the LTFATE model and is not expected to affect the applicability of model results to the prototype case.

Profiles 8 and 10 were modeled as 762 m long. This was necessary in order to increase the across-berm resolution of LTFATE, which is limited to 50 X 50 elements. This results in a ratio of length:width of 3.6 (assuming average berm width of 244 m at the base). With an L:W ratio of 3.6, the modeled mound hydrodynamics should be similar to the prototype case. Simulated volume changes based upon the 762-m berm configuration were multiplied by a factor of 5.2 (3,962 m/762 m) to obtain an estimate for volume change applicable to the full length prototype berm(s).

The numerical grid for the LTFATE model was set up according to a fictitious coordinate system oriented in a similar manner as the state plane coordinate system. Each element within the LTFATE model was sized at 30.48 by 30.48 m. The longitudinal axis of the berm was assumed parallel with the isobaths at the site, which was oriented 13° west of north. This defined the orientation of the modeled berm configurations with respect to the LTFATE grid: 13° west of north.

Ambient Hydrodynamic Conditions

Average annual conditions for waves, tides, and currents were predicted for the project site. An annualized wave environment was simulated using the HPDSIM program (Borgman and Scheffner 1991). The wave height, period, and direction are based on the WIS database and apply at a depth of -18 m NGVD. An example of the simulated wave environment (1-year duration, time = 0 corresponds to May) that was used in the LTFATE model is shown in Figure E1. Note that the waves are more severe during the fall and winter (time = 150 - 300 days).

The average annual tidal environment for the study area was generated using the program TIDE employing eight tidal constituents for water elevation and tidal current (u,v). The tidal constituents were generated from the ADCIRC-derived database for the Western North Atlantic Coast (Westerink, Luetich, and Scheffner 1993). The time series shown in Figure E2 (top) represents an equilibrium tide for the project site. An equilibrium tide is harmonically correct to the actual case, but is not referenced to a specific date or time. Tidal-induced currents were similarly produced. HPDSIM and TIDE were developed at CERC and are part of the LTFATE package.

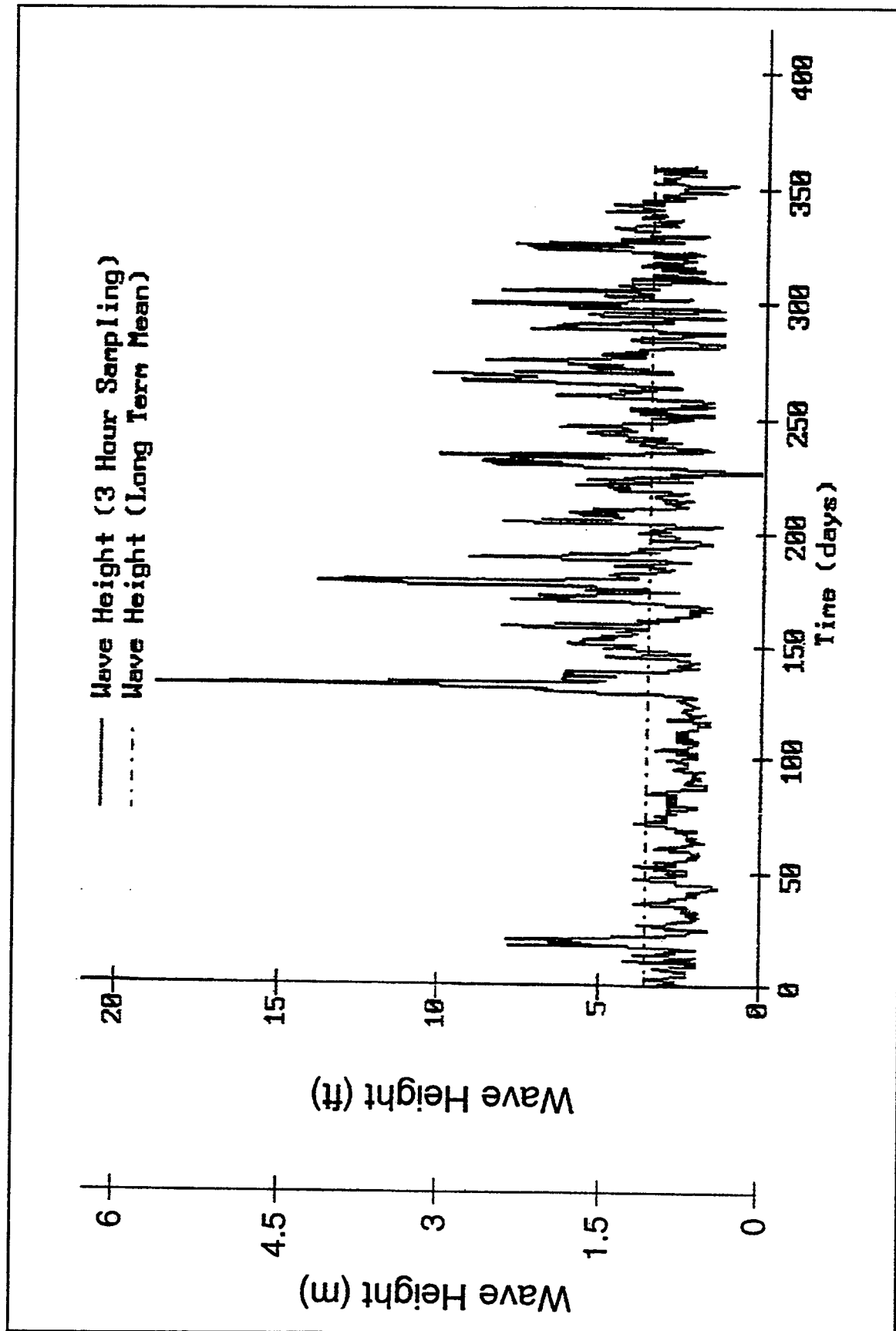


Figure E1. Simulated wave environment at St. Augustine Beach, Florida, for 1-year duration (Time = 0 is May. Water depth = 18 m)
(Sheet 1 of 3)

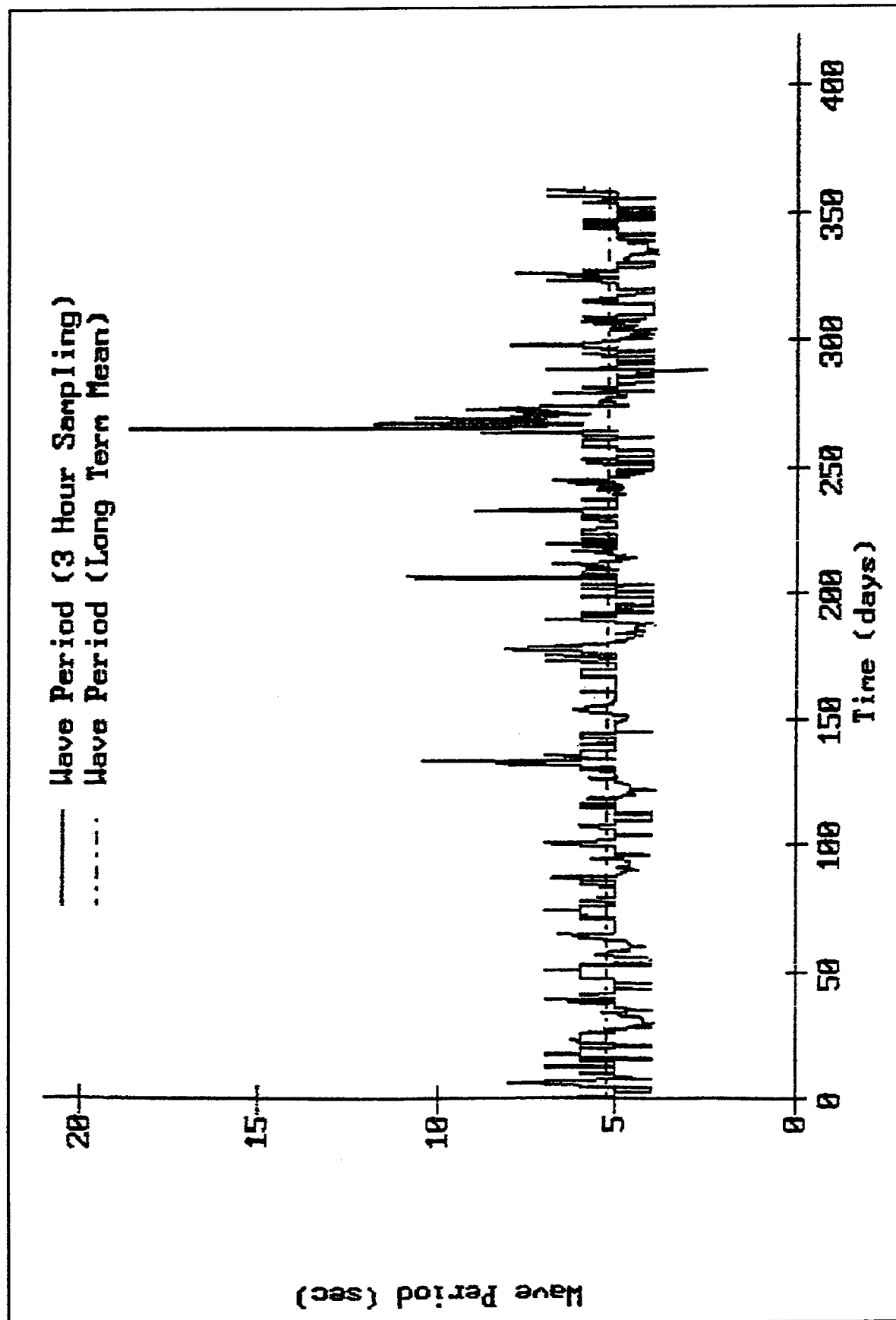


Figure E1. (Sheet 2 of 3)

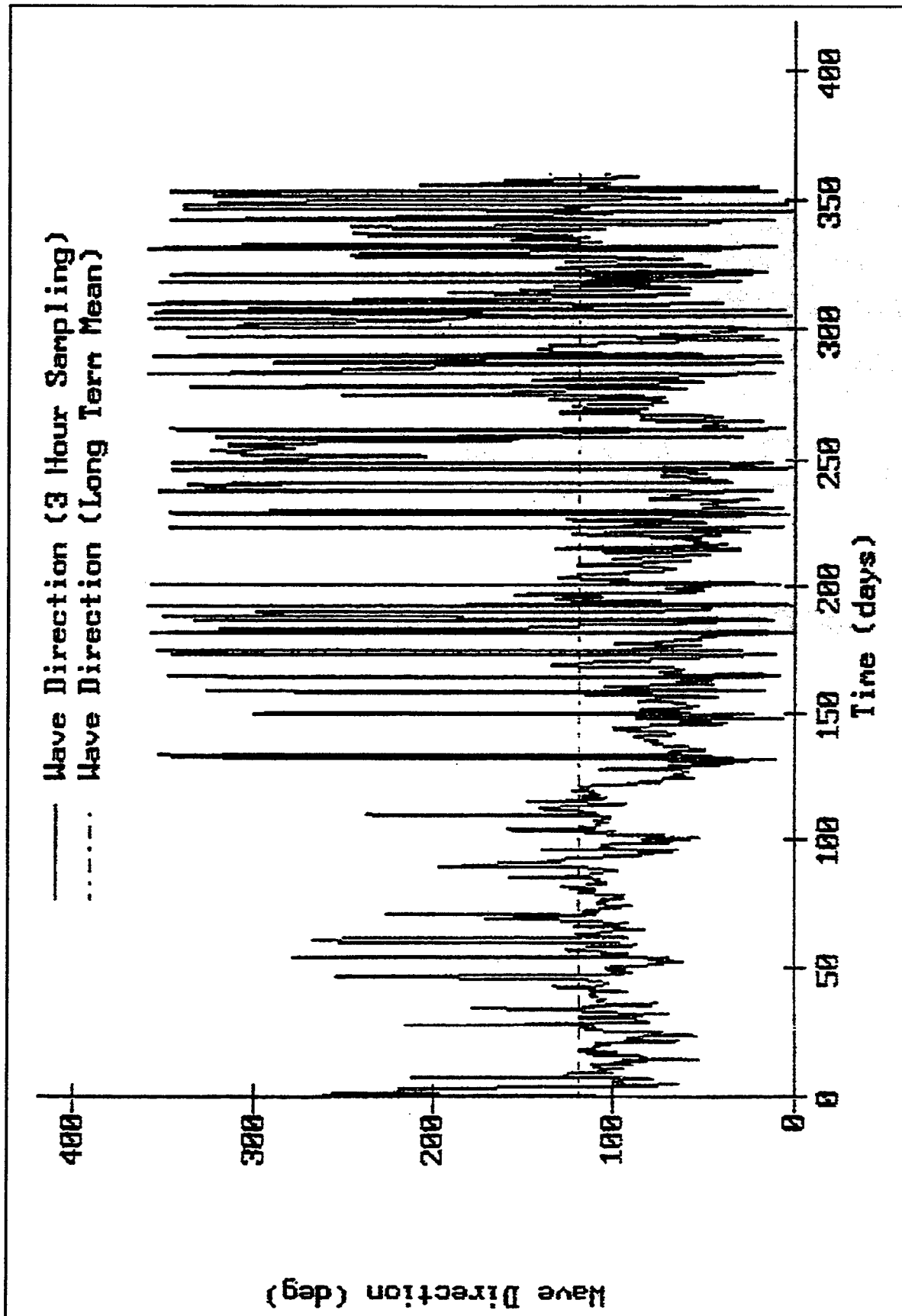


Figure E1. (Sheet 3 of 3)

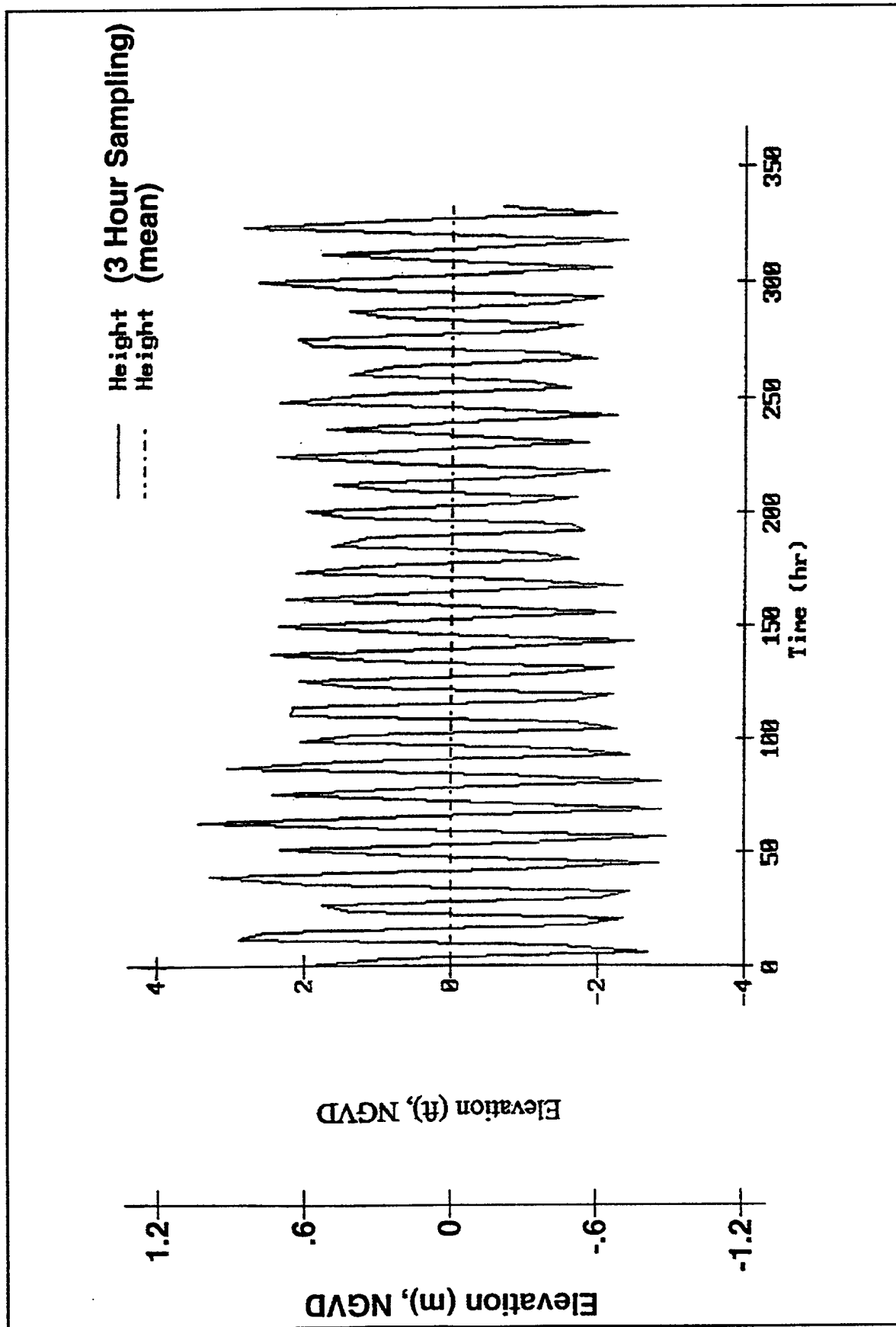


Figure E2. Simulated tide environment of 2-week excerpt from a typical year, St. Augustine Beach, Florida (Sheet 1 of 3)

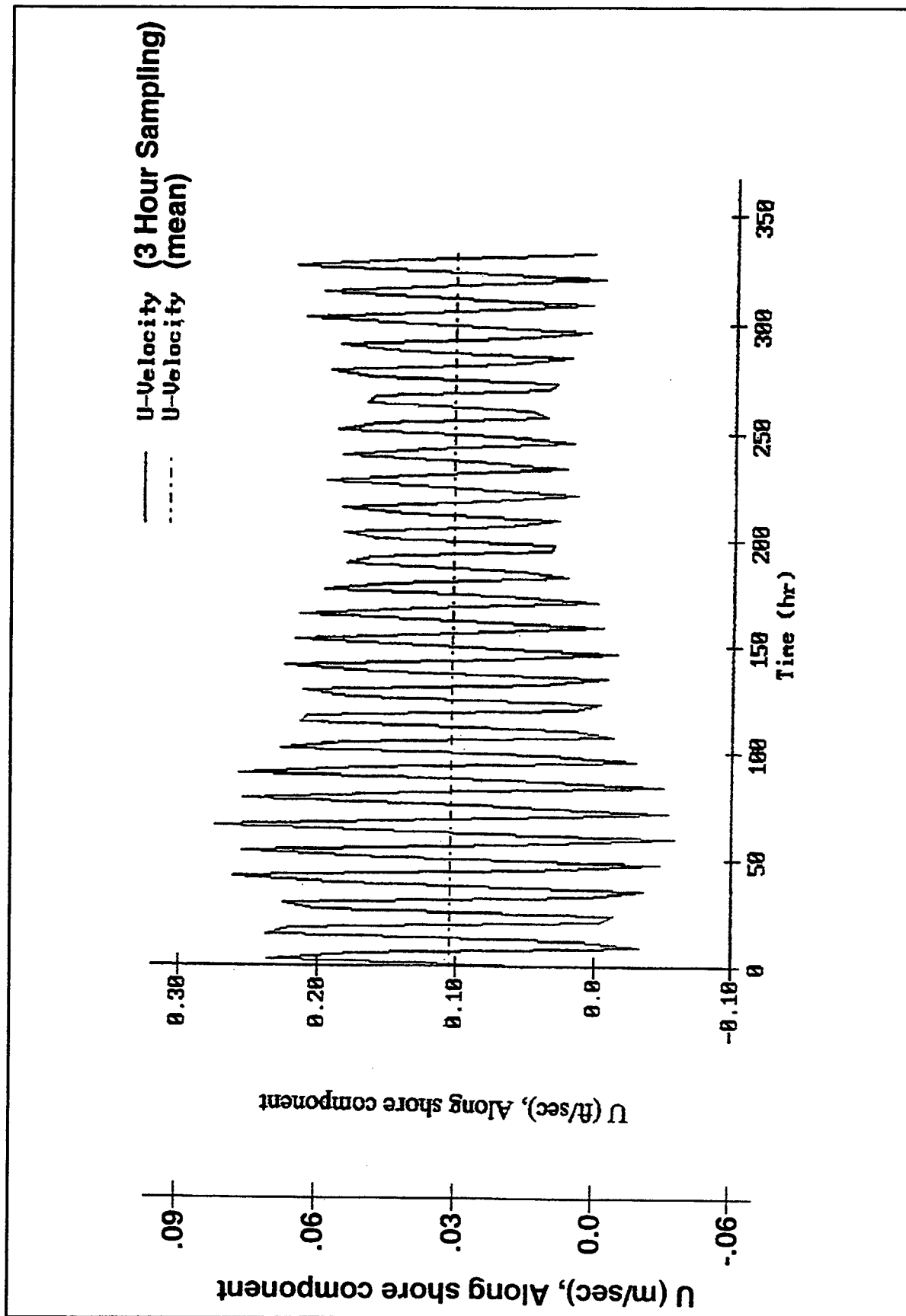


Figure E2. (Sheet 2 of 3)

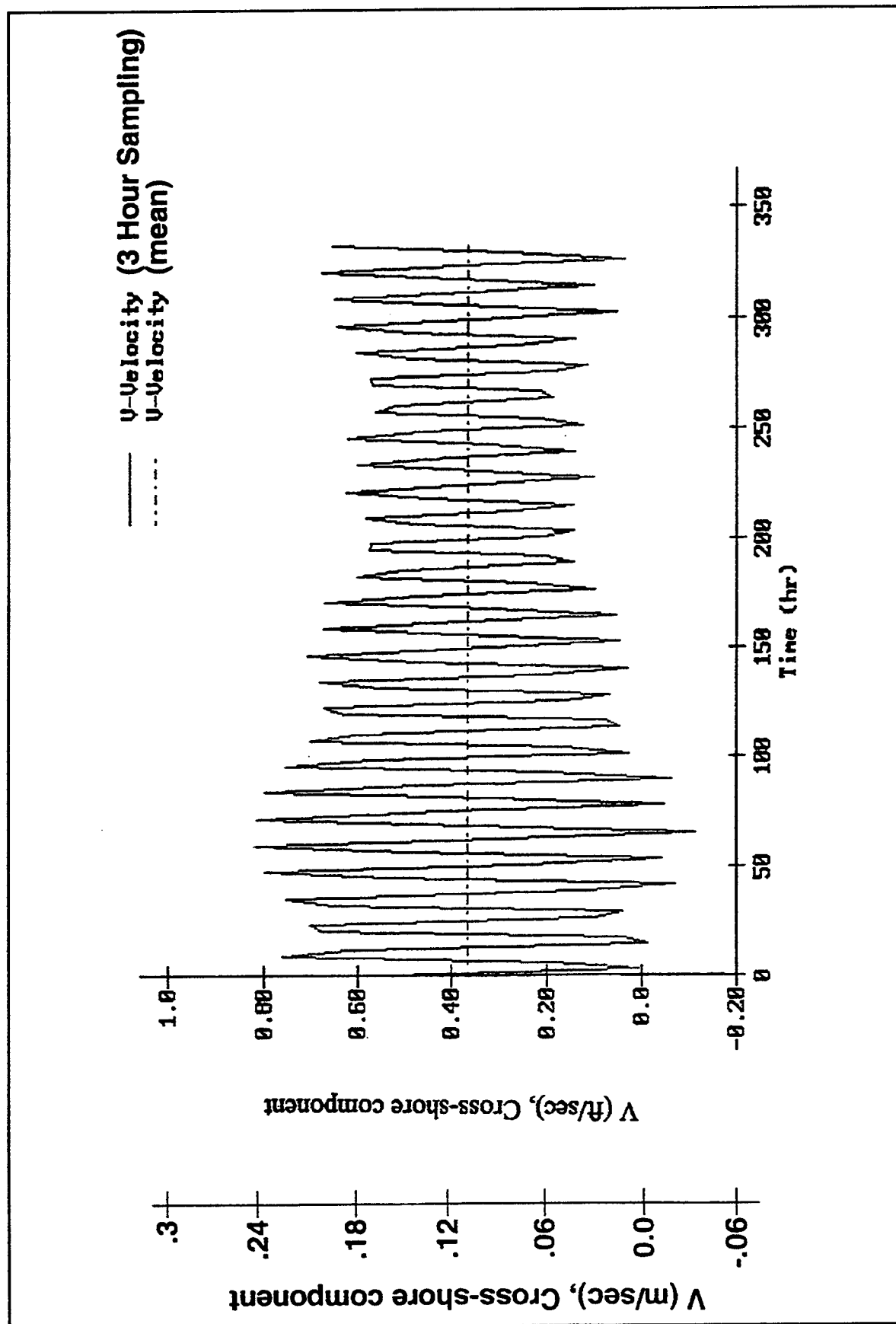


Figure E2. (Sheet 3 of 3)

Annualized Depth-Averaged Currents

The residual (nontidal) current (u,v) was obtained from measured data presented by Lee and Atkinson (1983) and Williams and Thomas (1987). It is noted that large-scale oceanic currents decrease rapidly with distance landward from the shelf-break. At the project water depth (approximately -9 m NGVD), the nontidal current is influenced by the nearshore windfield and circulation patterns. The measured mean (residual) current as reported above tends to be greater than 0 for both u and v components. The current components u and v are perpendicular and parallel to the isobaths, respectively. Table E2 shows summary statistics for the residual current measured near the project site during the fall of 1987.

Table E2				
Measured Residual Current Statistics				
Current Component	Mean Value cm/sec	Standard Deviation cm/sec	Minimum m/sec	Maximum cm/sec
U cross-shore	1.14	2.02	-2.80	11.58
V along-shore	3.92	7.29	-12.04	21.62

Due to the high variation of residual current, it was assumed that a value greater than the "mean" value be used to characterize the residual current. Since the residual current is distributed on the positive side of 0, for both u and v, the current was represented as:

$$U_{res} = \text{mean value (u)} + \sigma = 3.16 \text{ cm/sec}$$

$$V_{res} = \text{mean value (v)} + \sigma = 11.21 \text{ cm/sec}$$

Residual current magnitude = 11.65 cm/sec (at 3° east of north)

To completely specify a time-series representation of annualized current, the residual components were added to the tidal-induced current. The result for the annualized current at the project site is shown in the bottom two plots (u and v) in Figure E2.

Hydrodynamic Events

Six storm scenarios were selected as extratropical storm events. The storms were events that had previously occurred at the study area. Wave and water elevation data for all six storm scenarios were developed through direct measurement (for water levels) and hindcasting (for waves). Statistics for the six selected extratropical storm events are shown in Table E3. Figure E3 shows the

Table E3 Peak Storm Parameters for Selected Extratropicals						
STORM EVENT	PEAK HEIGHT m	WAVE PERIOD sec	WATER ELEVATIO N m, NGVD	SIMULATE D CURRENT U m/sec	PEAK ENVELOPE V m/sec	STORM DURATIO N hrs
JAN88	4.18	19	1.0	0.06, 0.09	0.34, -0.27	132
JAN89	5.0	14	1.19	0.09, -0.05	0.08, -0.43	123
SEPT89	6.8	22	0.94	0.09, -0.04	0.08, -0.34	180
OCT90a	3.29	15	1.12	0.08, -0.06	0.20, -0.34	174
OCT90b	4.0	16	1.00	0.05, -0.05	0.19, -0.11	99
HALLOW91	3.2	18	1.33	0.10, -0.07	0.27, -0.47	345

wave environment and water surface elevation during the 1991 Halloween storm. The water level data for the extratropical storm events include storm-related and background tidal effects.

The extratropical storm data sets were modified in order to produce derivative data for depth-averaged currents (u,v). At the time of this investigation, current information was not available for the six selected extratropical storms. However, hydrographs and current data were available for several hurricane events. In order to accurately simulate combined tide and storm-induced currents, current information was synthesized for each extratropical storm as described below.

Open coast hydrographs and depth-averaged current data (u,v) were available for several hurricanes of record that had passed nearby the study area. The hurricane data do not include the effect of background tidal conditions. The hurricane events, in terms of year, include 1899, 1933, 1972, and 1989. Data for the hurricanes were produced by hindcast. The hindcasted hydrograph and depth-averaged currents (u,v) for the 1972 hurricanes are shown in Figure E4. Note that u and v are considered perpendicular and parallel to shore, respectively.

Extratropical depth-averaged currents. The hydrographs for the hurricanes were compared with the six extratropical storms of interest. The best matching hurricane hydrograph was chosen for each of the six extratropical storms. The central idea was to match hydrographs for a given extratropical and hurricane, and use the depth-averaged currents (u, v) for the matching hurricane to represent the current regime for the corresponding extratropical.

The hurricane current data were then scaled in magnitude based upon the ratio of extratropical surge to hurricane surge. The scaled current (u,v) was added to the normal tidal current (u,v) to obtain a time-series representation for

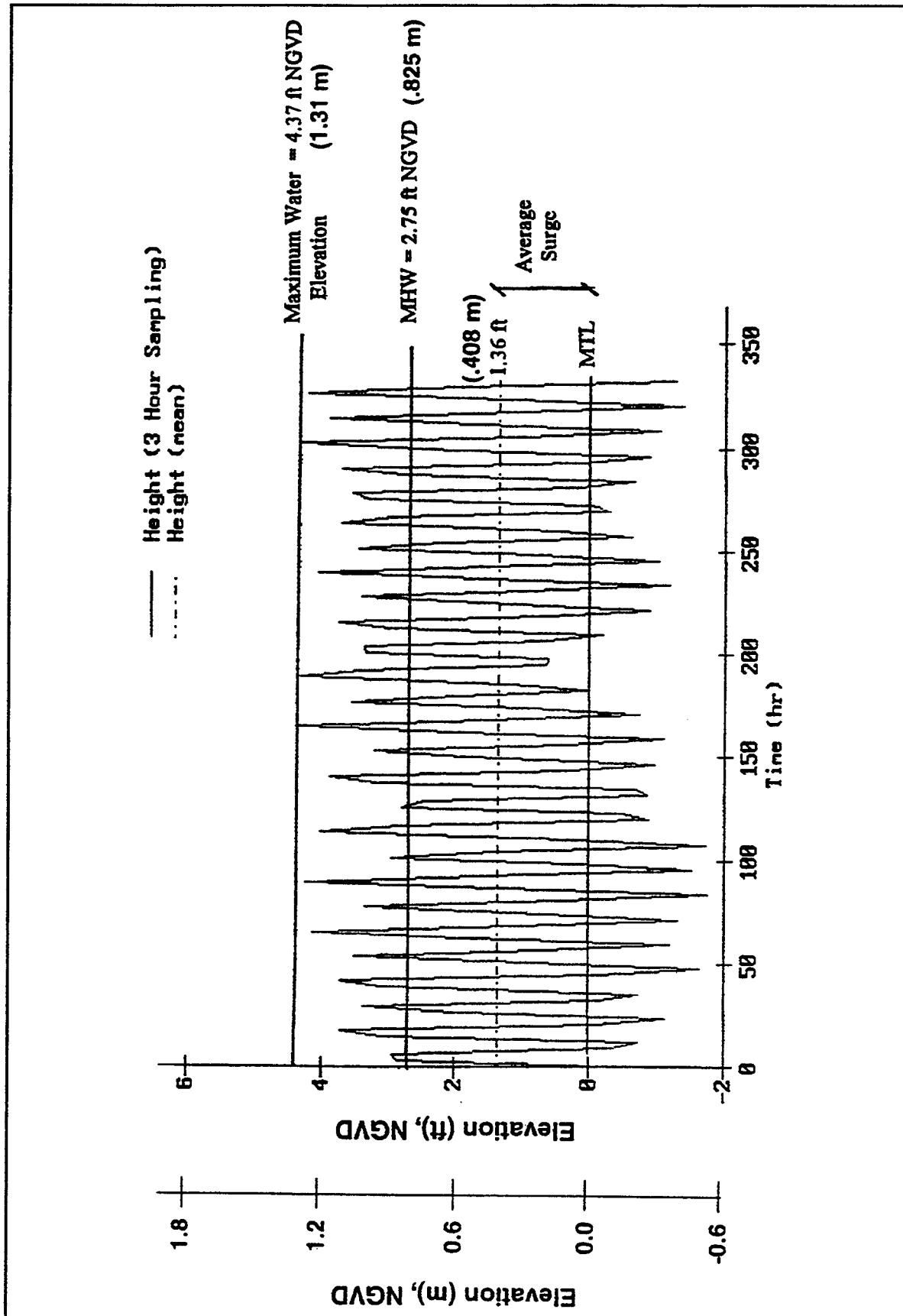


Figure E3. Hindcasted hydrograph and wave environment for Halloween 1991 storm event at St. Augustine Beach, Florida (Continued)

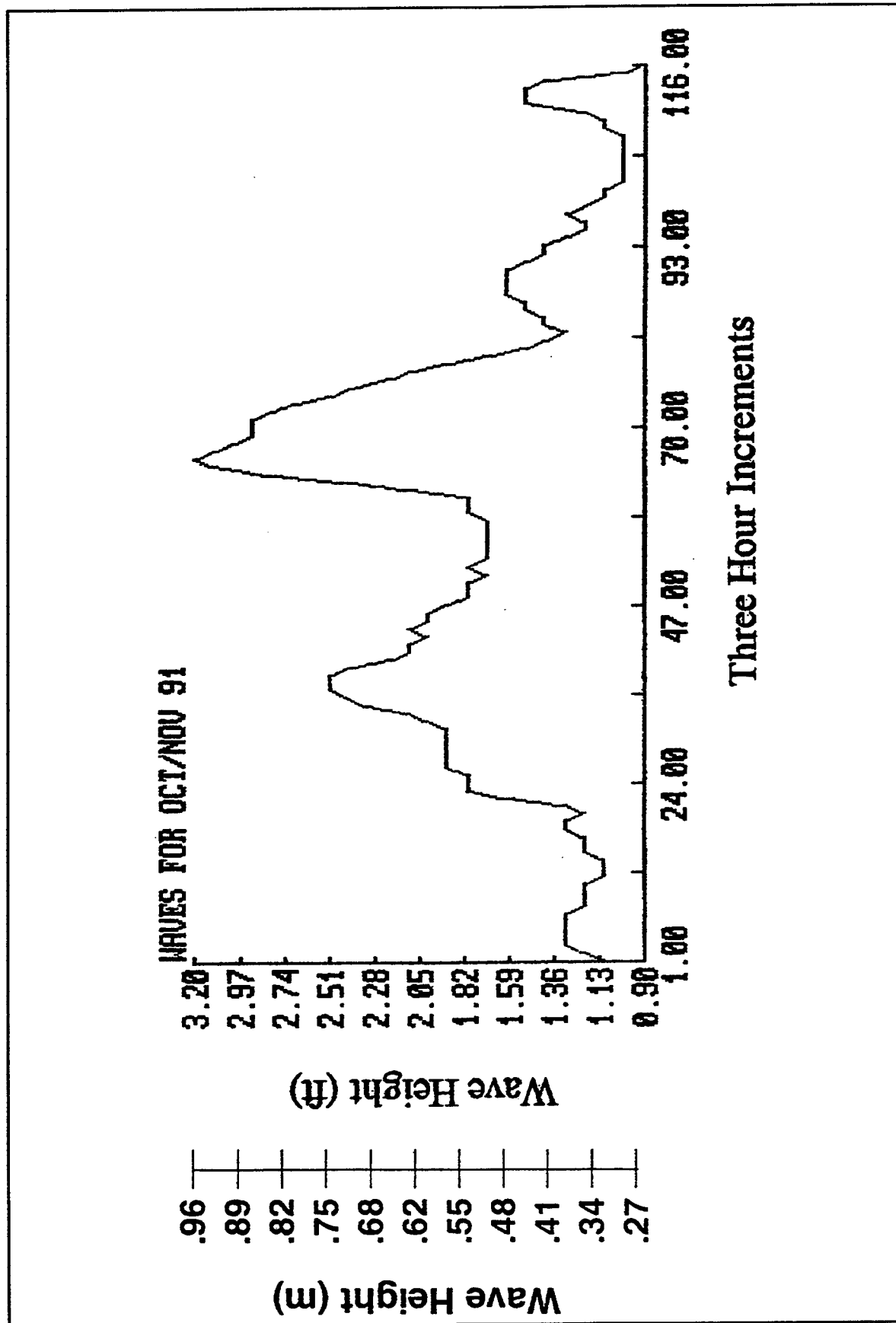


Figure E3. (Concluded)

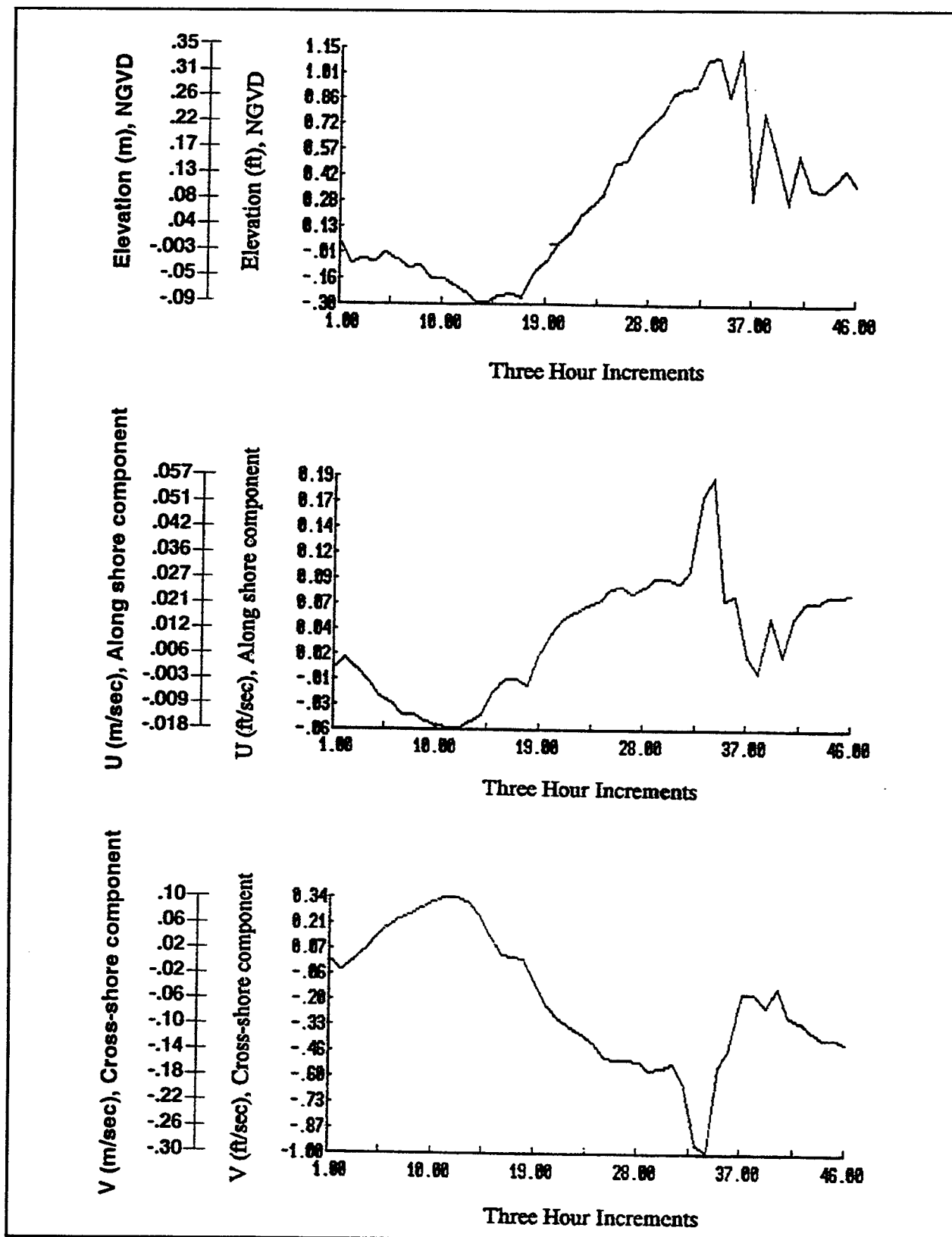


Figure E4. Hindcasted hydrograph and depth-averaged current for 1972 hurricane passing nearby St. Augustine Beach, Florida (tidal effects not included)

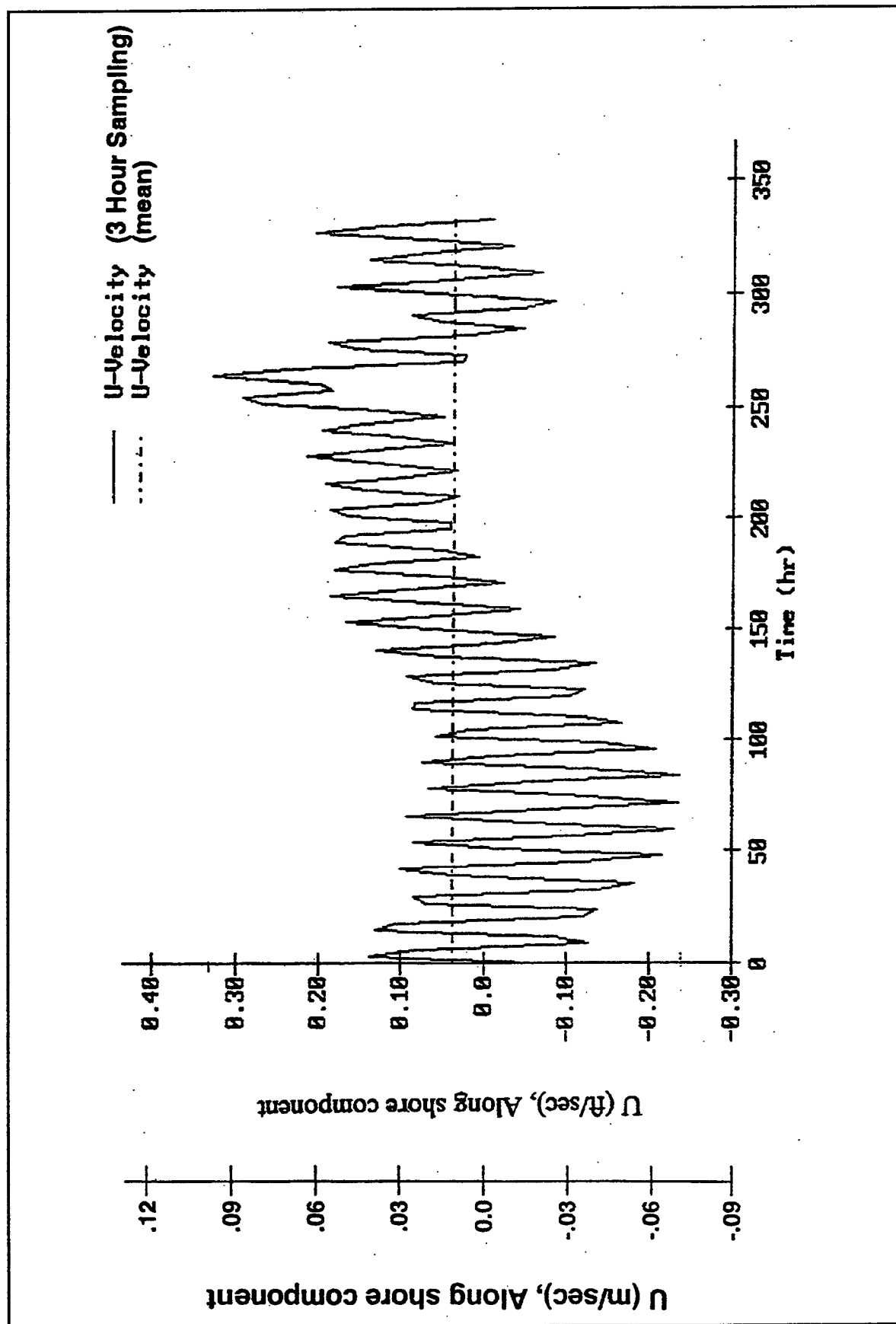


Figure E5. Reconstructed depth-averaged currents at St. Augustine Beach, Florida, for Halloween 1991 storm event (Continued)

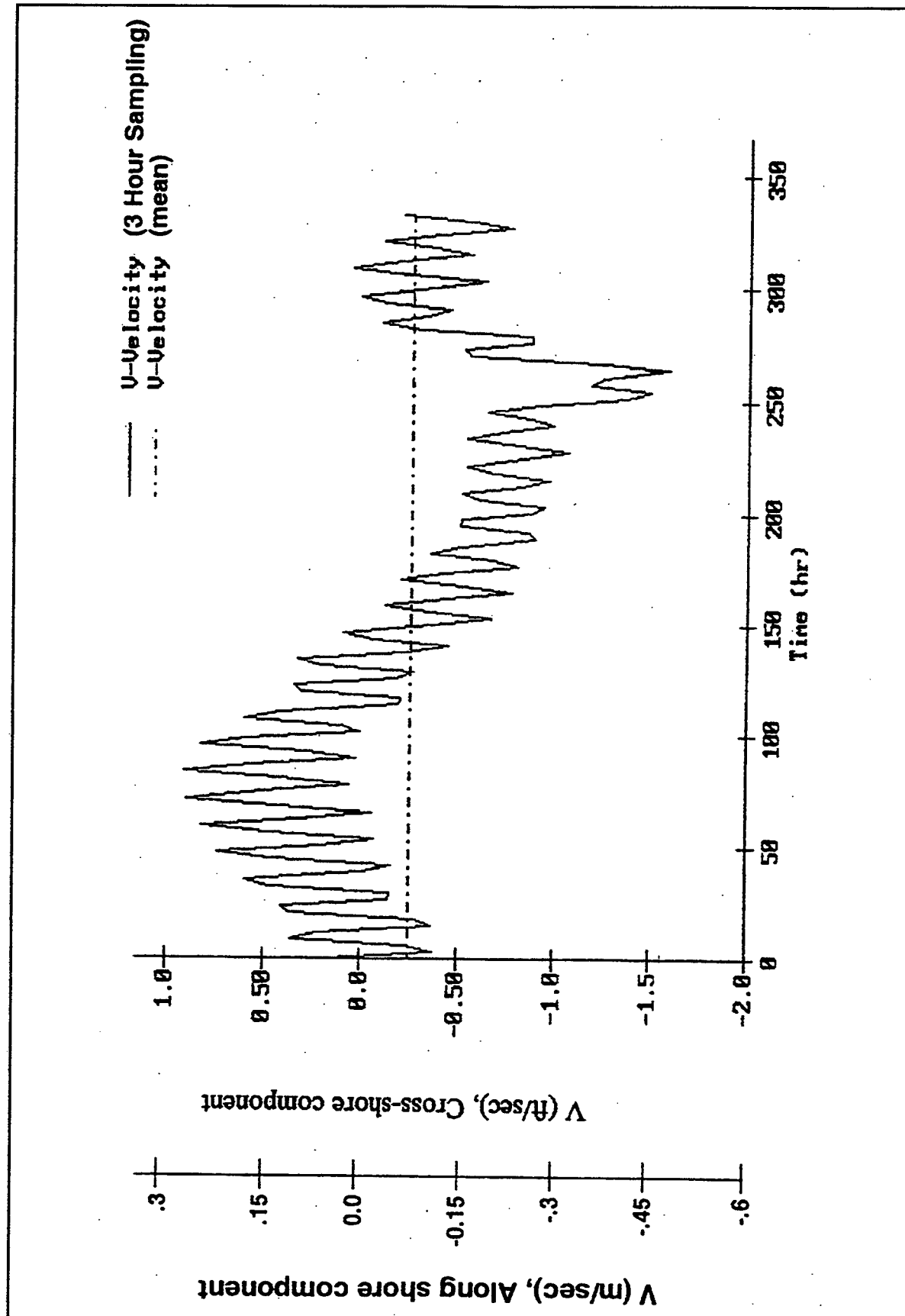


Figure E5. (Concluded)

extratropical current accompanied with background tidal effects. The resultant depth-averaged current for the Halloween 1991 extratropical is shown in Figure E5. The hurricane current data are algebraically adapted to the extratropical storms for the case of the Halloween 1991 extratropical storm below.

Surge for the Halloween 1991 Storm = average surge

Surge for the Halloween 1991 Storm = maximum surge - MHW

In the case of the Halloween 1991 Storm (Figure E3):

Average surge = 0.415 m

Maximum surge - MHW = $1.33 - 0.84 = 0.49$ m

Matching peak hurricane (1972) surge = 0.35 m

Current scaling factor = $0.49/0.35 = 1.40$

Depth-averaged currents (u,v) = [currents (u,v) in Figure A4 * 1.4] + [normal tide (u,v)]

Results are shown in Figure E5. This procedure was followed for each of the six storm scenarios. Results of the current generation for all six storms and other storm-related parameters are shown in Table E3.

Results. The LTFATE model was run using the physical and environmental data described above. Model runs were segregated into seven separate conditions. One result was obtained for the average annual conditions (one-year duration), and a set of six model runs was performed based on the suite of six extratropical storms.

Graphical results are shown for only Profile 10. Summary statistics for both alternatives, Profile 8 and Profile 10, are shown in Table E4 for average annual conditions and Tables E5 and E6 for storm-related berm movement. The initial condition for Profile 10 is shown in Figure E6. Note that the berm is portrayed with an exaggerated vertical scale. The crest for Profile 10 is 4.5 m above the ambient bathymetry. The grid shown in Figure E6 is oriented N-S and E-W. The berm is oriented 13° west of north.

Simulated average annual berm response. Statistics summarizing the LTFATE-predicted response of Profile 10 due to the average annual wave/tidal environment are shown in Table E4. Graphical results for Profile 10 subjected to the average annual condition are shown in Figures E7 and E8. Note the migration of the berm in the northern direction, at the expense of the berm's southern flank. In Table E4, this trend is quantitatively described for both Profile 8 and Profile 10; the net movement was north-northeast. This is opposite to the documented net southward longshore movement of littoral material for the site.

Table E4
Profile 10 Response for Average Annual Conditions

Berm Option	Net Movement of Berm Centroid ¹		Change in Berm Crest Elevation			Volume Loss Due ² to Berm Movement	
	Alongshore Parallel to Berm m	Cross-shore Perpendicular to Berm m	w.r.t. 335m North m	East-West C.L. m	Center Line 335 m South m	Total m ³	Per Unit Length m ³ /m
Profile 8	69.8	8.7	0.27	-0.24	-1.92	-101,722	-133.5
Profile 10	63.4	6.7	0.73	0.09	-0.43	-14,045	-18.6

¹ Negative values for alongshore/cross-shore movement indicate southward/westward migration.

² Net volume change of contiguous berm = 0. Volume loss is associated with movement of 762-m berm from original location. For a 3,962-m berm, total volume loss (m³) would be per unit length loss*3,962.

Table E5
Profile 10 Response for Six Individual Storm Scenarios

Storm Events	Net Movement of Berm Centroid ¹		Change in Berm Crest Elevation			Volume Loss Due ² to Berm Movement	
	Alongshore Parallel to Berm m	Cross-shore Perpendicular to Berm m	w.r.t. 335 m North m	East-West C.L. m	Center Line 335 m South m	Total m ³	Per Unit Length m ³ /m
JAN88	33.8	-0.9	0.49	0	-1.04	-35,324	-46.4
JAN89	-47.5	-0.8	-2.29	0	0.55	-47,022	-61.7
SEPT89	-36.0	0.5	-1.80	0	0.34	-35,401	-46.4
OCT90a	-10.7	0.3	-0.58	0	0.15	-13,915	-18.3
OCT90b	2.5	-0.06	0.15	0	-0.15	-902	-1.3
HALLOW91	-20.4	-2.00	-1.40	0	0.49	-34,789	-45.7

¹ Negative values for alongshore/cross-shore movement indicate southward/westward migration.

² Net volume change of contiguous berm = 0. Volume loss is associated with movement of 762-m berm from original location. For a 3,962-m berm, total volume loss (m³) would be per unit length loss*3,962.

Table E6
Profile 8 Response for Six Individual Storm Scenarios

Storm Events	Net Movement of Berm Centroid ¹		Change in Berm Crest Elevation			Volume Loss Due ² to Berm Movement	
	Alongshore Parallel to Berm m	Cross-shore Perpendicular to Berm m	w.r.t. 335 m North m	East-West C.L. m	Centerline 335 m South m	Total m ³	Per Unit Length m ³ /m
JAN88	22.6	-0.43	0.21	0	-0.12	-3,173	-4.3
JAN89	-32.0	-0.30	-0.30	0	0.30	-4,618	-6.0
SEPT89	-22.4	0.21	-0.24	0	0.12	-3,134	-4.0
OCT90a	-10.1	0.15	-0.09	0	0.09	-1,383	-1.8
OCT90b	1.0	-0.03	0	0	0	0	0
HALLOW91	-19.8	-1.52	-0.21	0	0.09	-3,746	-5.0

¹ Negative values for alongshore/cross-shore movement indicate southward/westward migration.

² Net volume change of contiguous berm ≈ 0 . Volume loss is associated with movement of 762-m berm from original location. For a 3,962 m berm, total volume loss (m³) would be per unit length loss*3,962.

It was assumed that the "offshore" berms would be placed offshore of the inner closure depth for the surf zone. This renders wave action only as an agitation force that acts to temporarily suspend bottom sediments, with no net transport. Net transport (direction) is due exclusively to current, with wave action augmenting only in the magnitude of sediment transport. At the 10-m isobath, it was determined that the mean residual current direction is north-northeast and is related to Gulf Stream eddies and related phenomena.

It is shown in the next section that storm-generated berm movements can produce net longshore movement of the berm to the south. A storm-generated wind/wave field can induce a coincident current that may overshadow Gulf stream influences at nearshore locations.

Response due to storms. Statistics summarizing LTFATE predicted response of Profile 10 due to each of the six selected storm events are shown in Tables E5 and E6. Graphical results for Profile 10 subjected to the average annual condition are shown in Figures E9 and E10. The "center line," referred to in Figure E10 and Tables E5 and E6, is oriented east-west across the middle of the grid. Note that for the HALLOW91 storm, Profile 10 migrated southward. The south end of the berm "gained" material at the expense of the north end. For example, the northern end of Profile 10 was lowered by 1.4 m by the HALLOW91 storm (crest elevation reduced to -6.4 m NGVD). This is the worst case value for the Profile 10 - HALLOW91 event and should be used to assess wave transmission predictions (if conducted) along the entire berm.

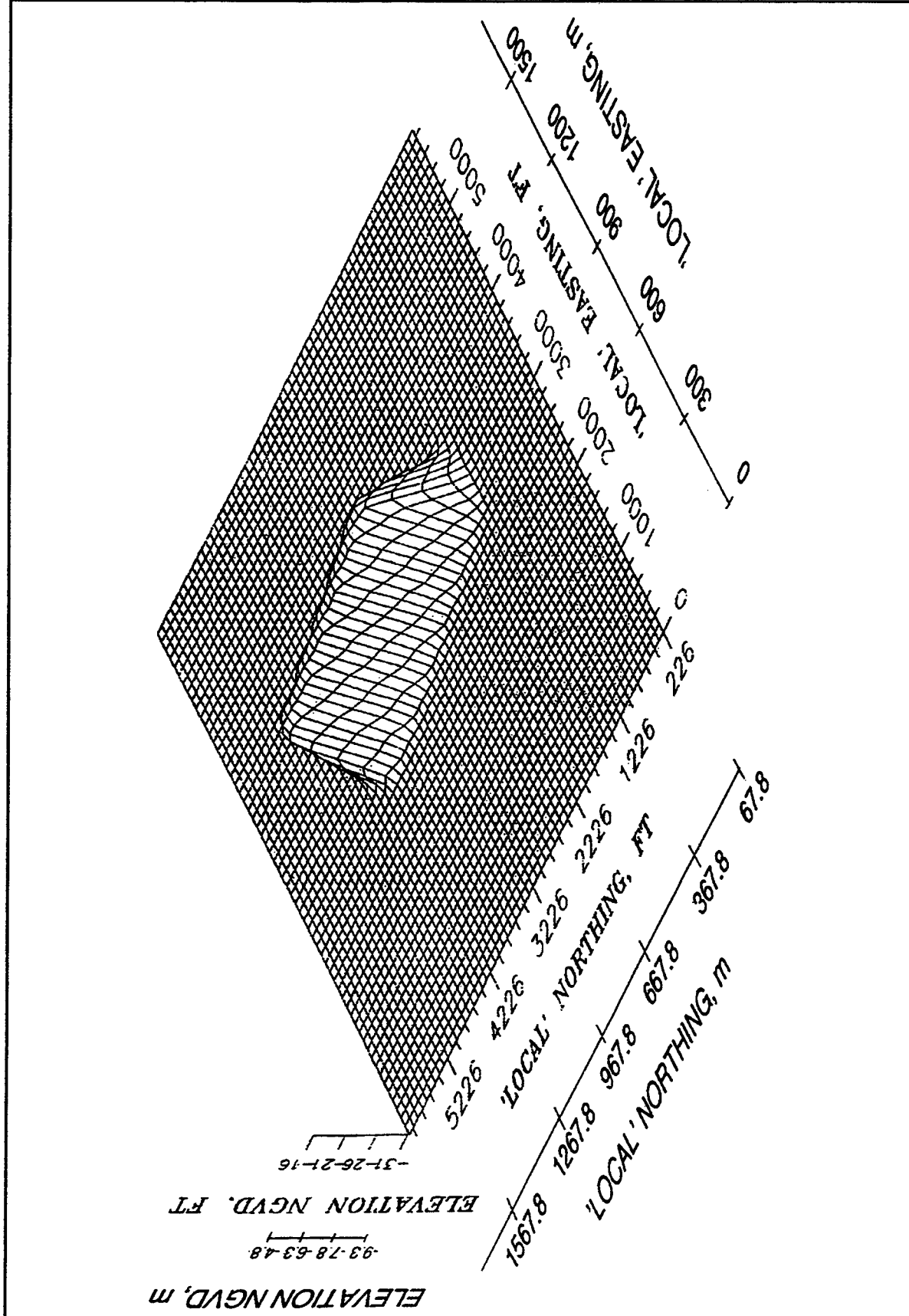


Figure E6. Initial condition for berm located on Profile 10 (Topographic contours in units of feet) (Sheet 1 of 3)

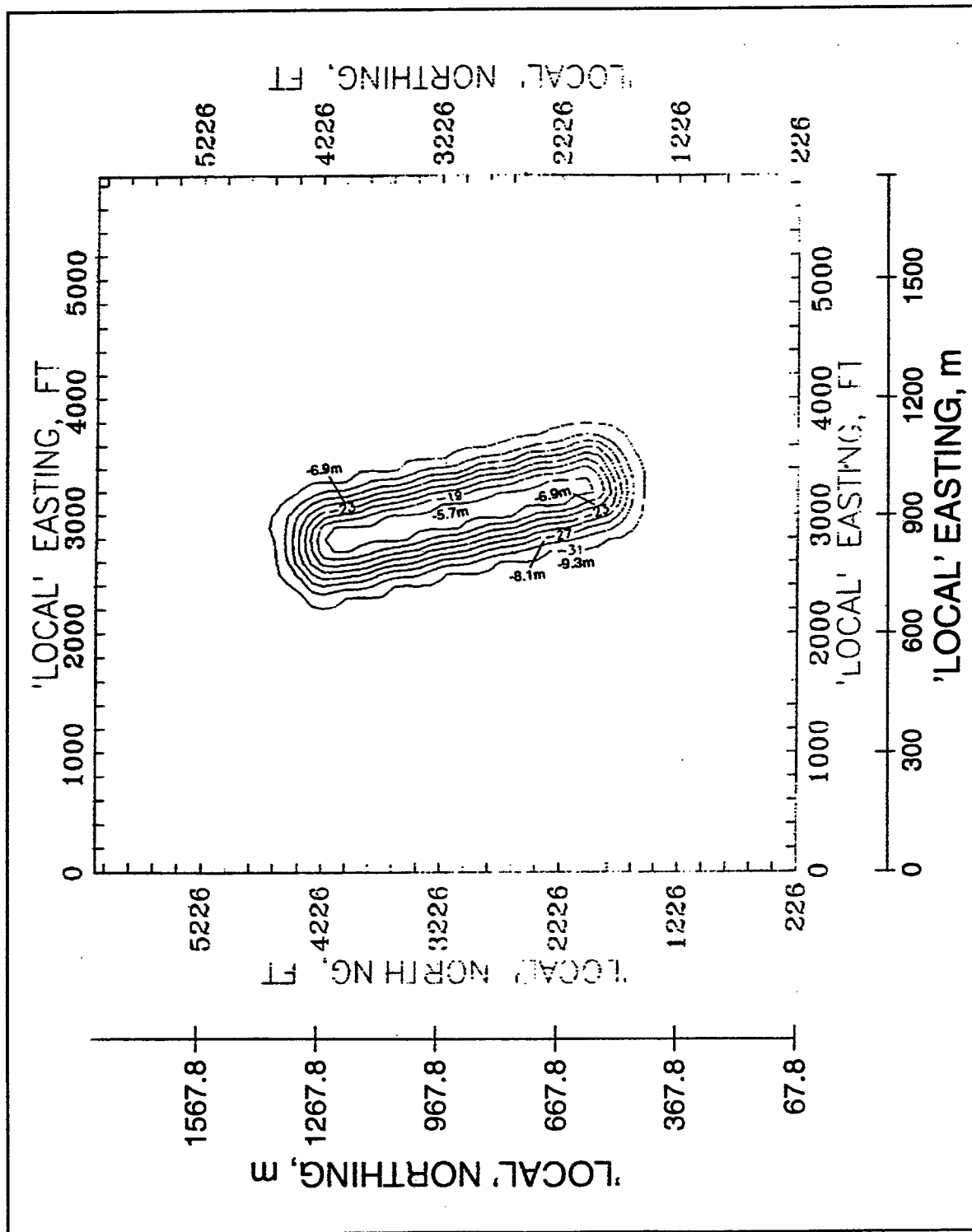


Figure E6. (Sheet 2 of 3)

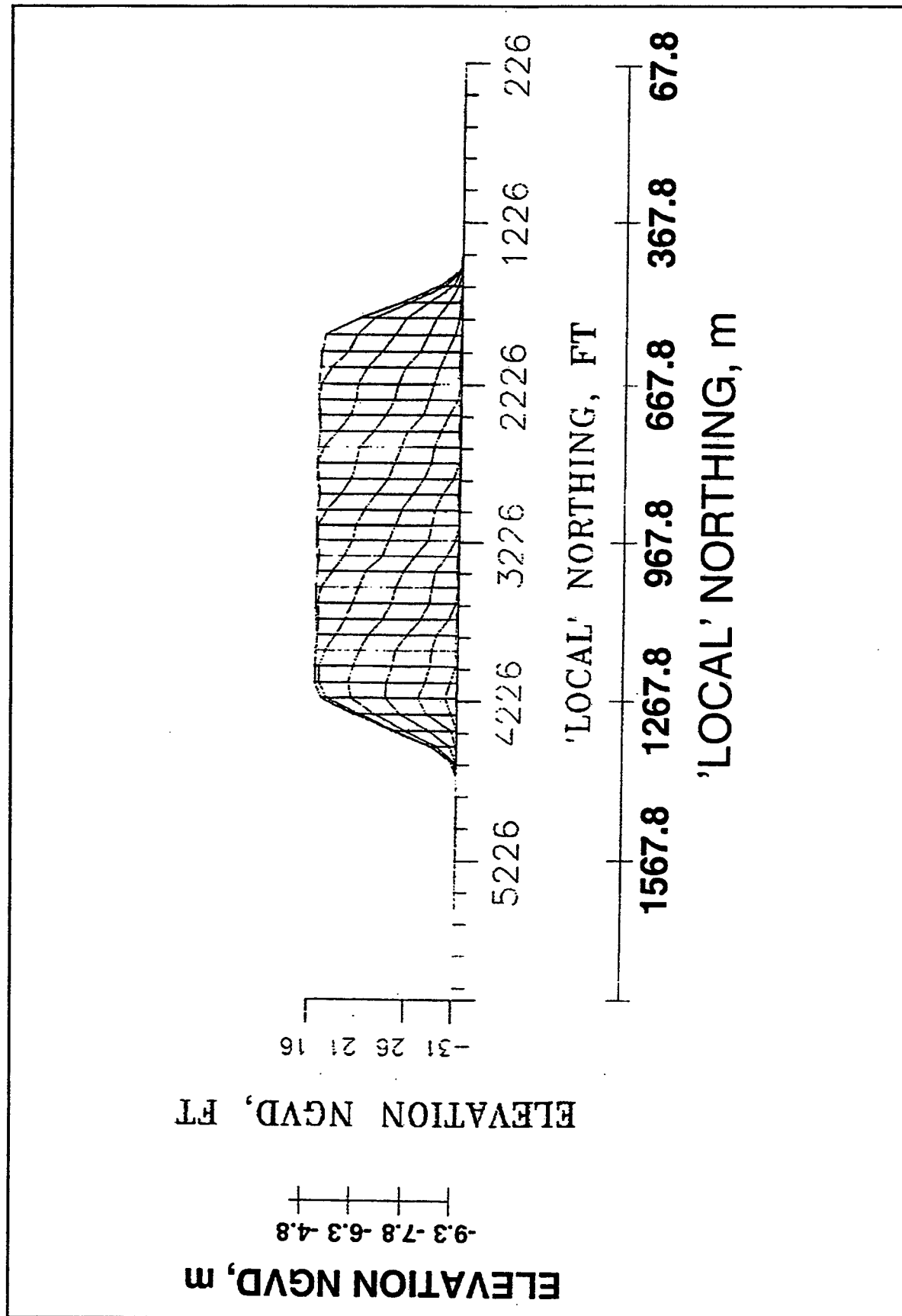


Figure E6. (Sheet 3 of 3)

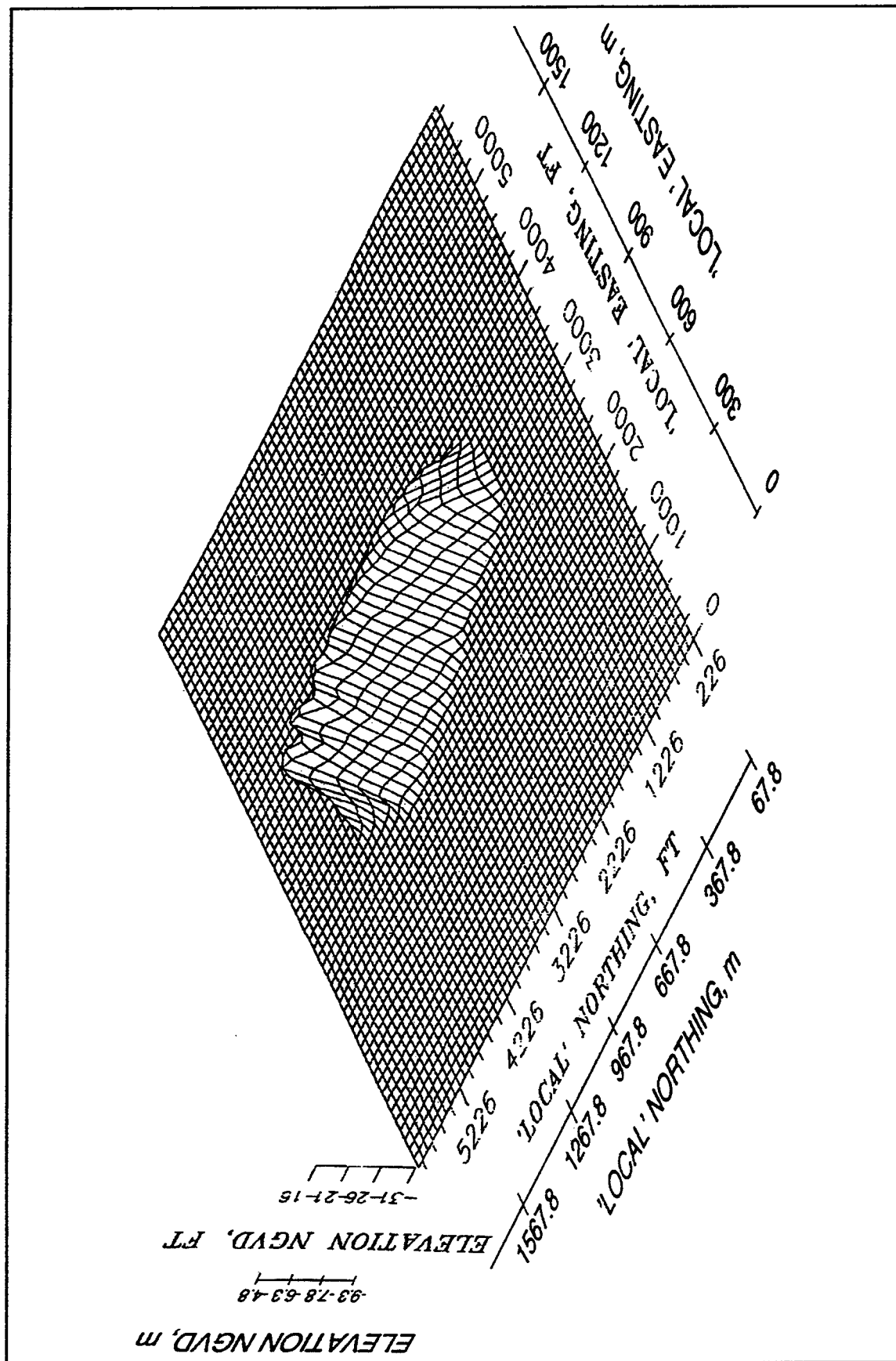


Figure E7. Simulated condition for berm located on Profile 10 after 1 year of average annual conditions (topographic contours in units of feet)
(Sheet 1 of 3)

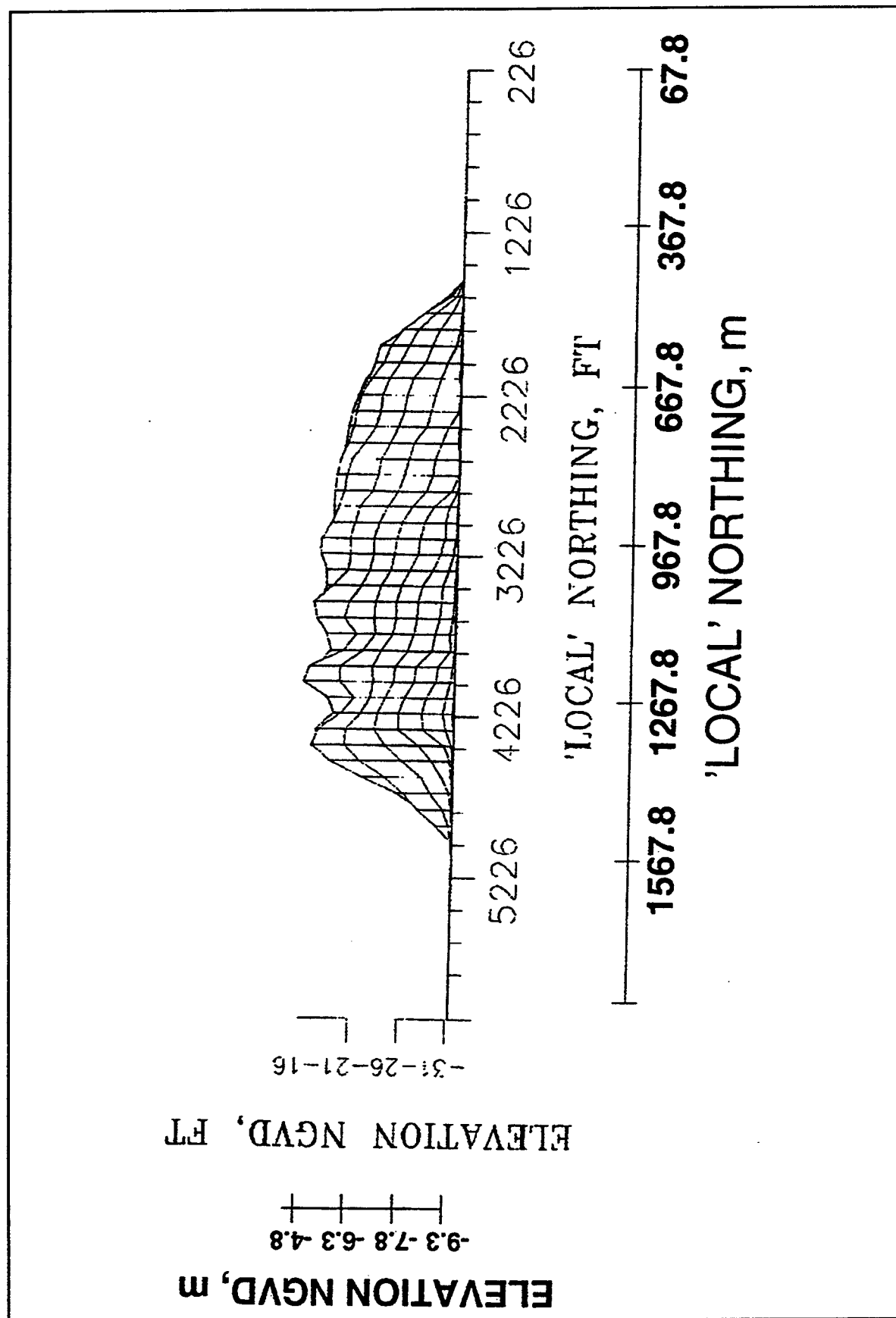


Figure E7. (Sheet 3 of 3)

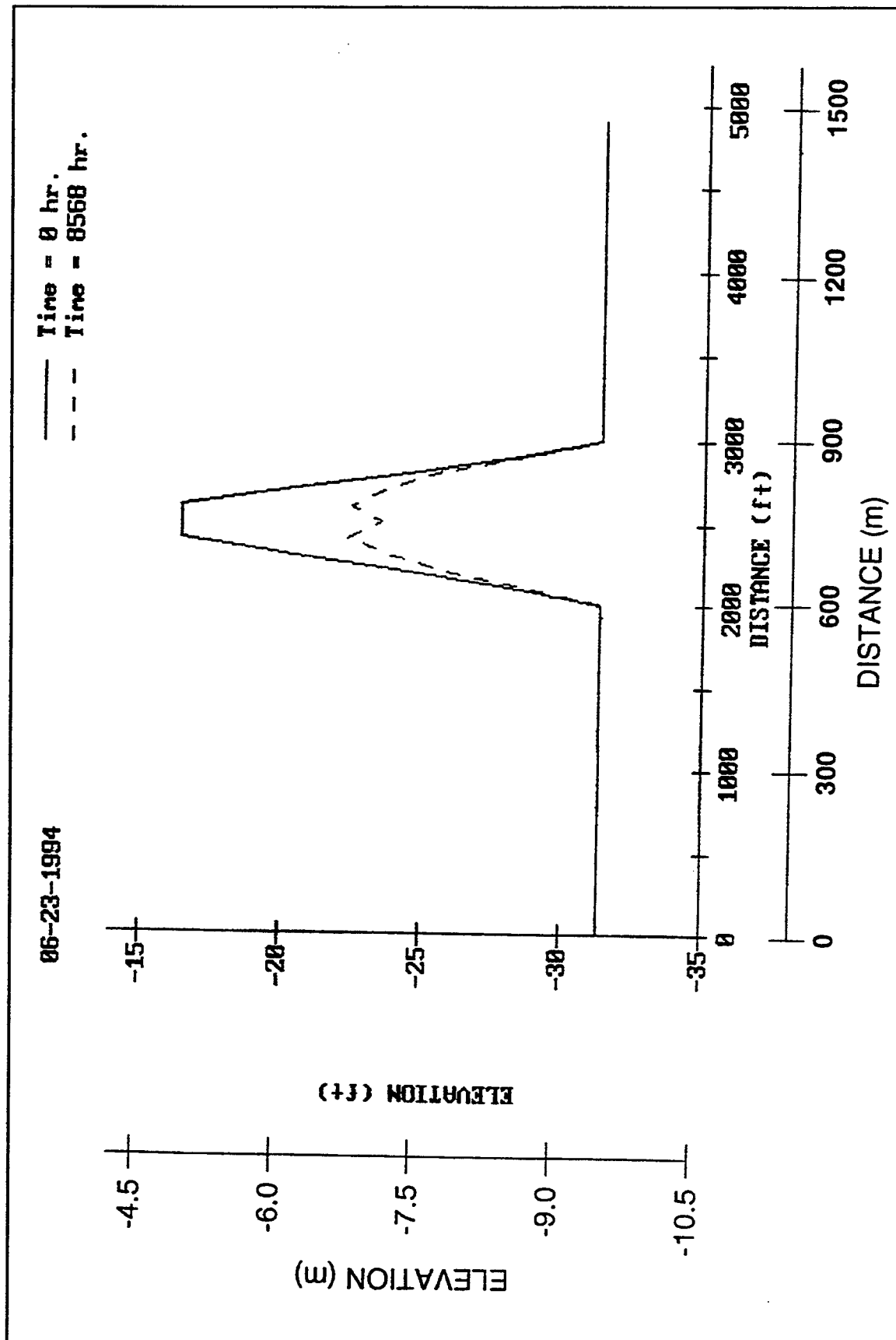


Figure E8. Predicted cross section: 335 m north and south of center line for berm located on Profile 10, after 1-year duration of average annual conditions (Continued)

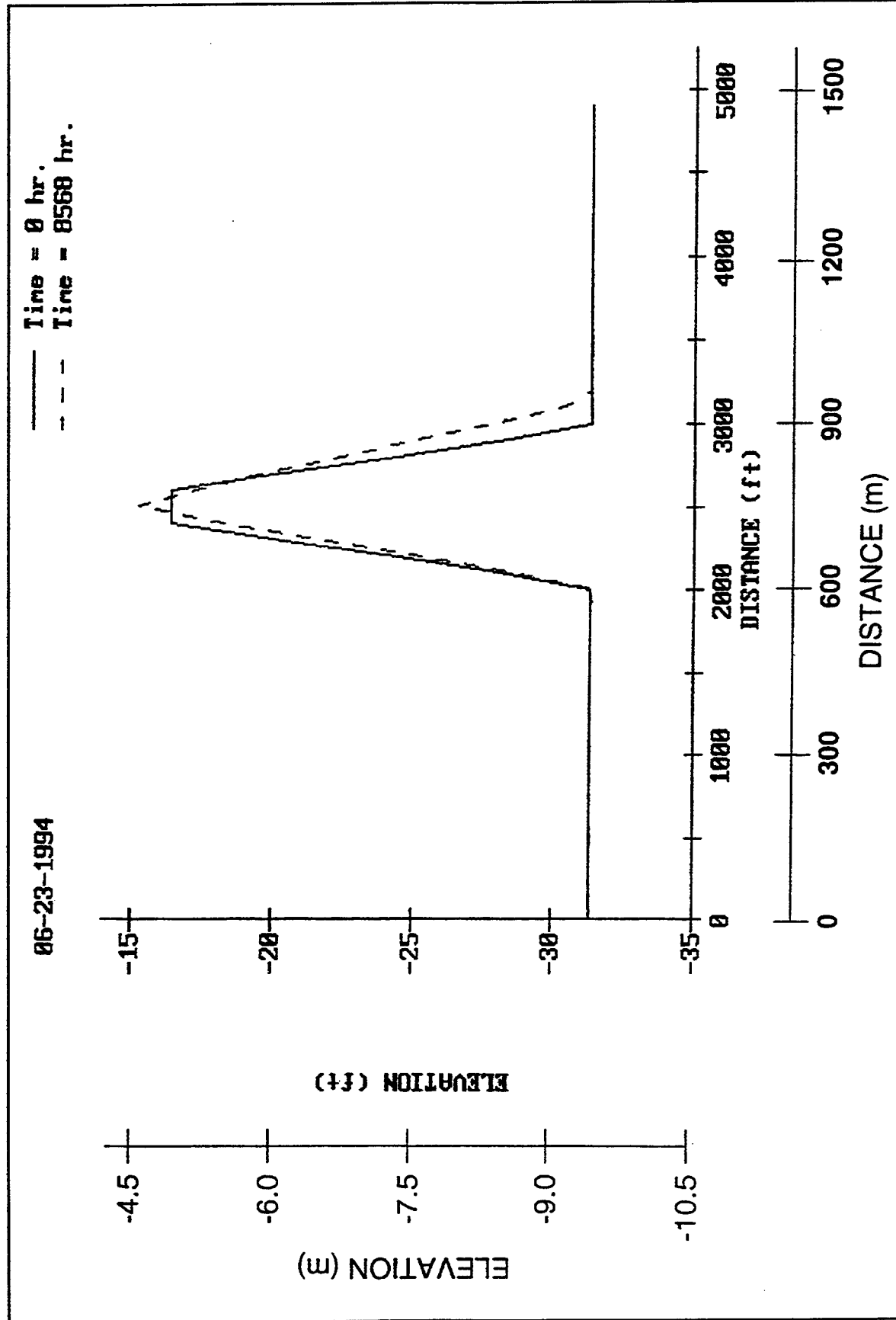
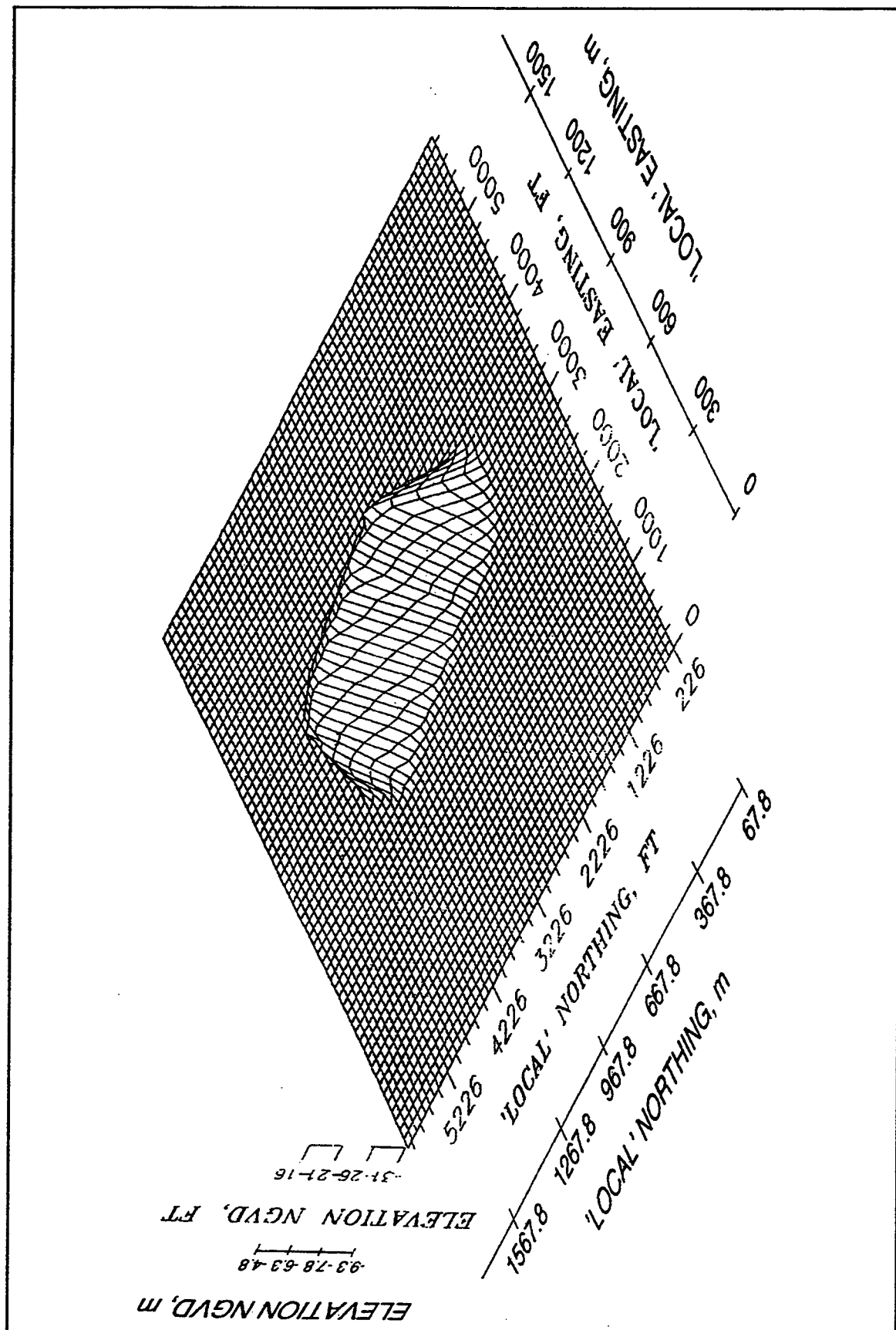


Figure E8. (Concluded)



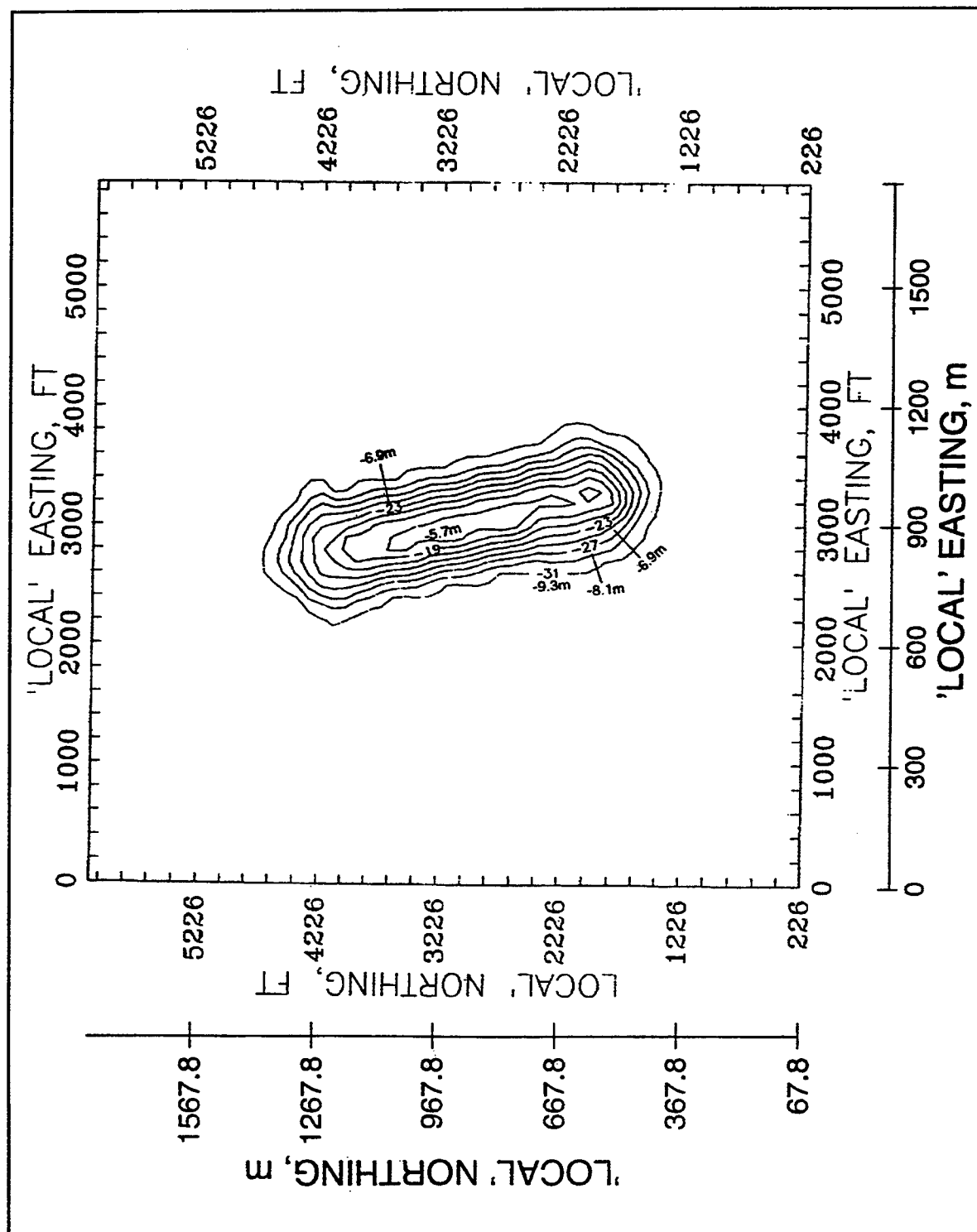


Figure E9. (Sheet 2 of 3)

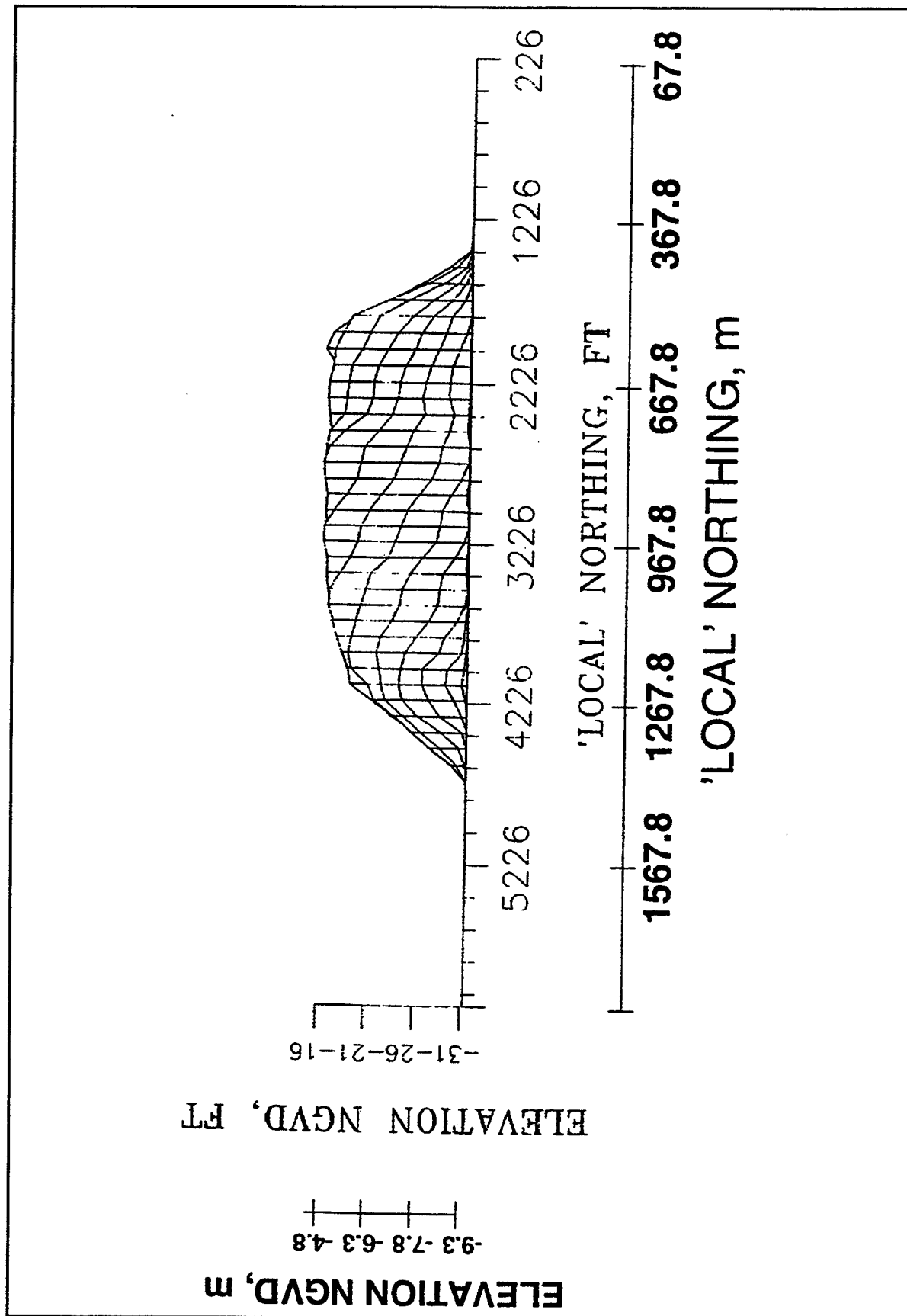


Figure E9. (Sheet 3 of 3)

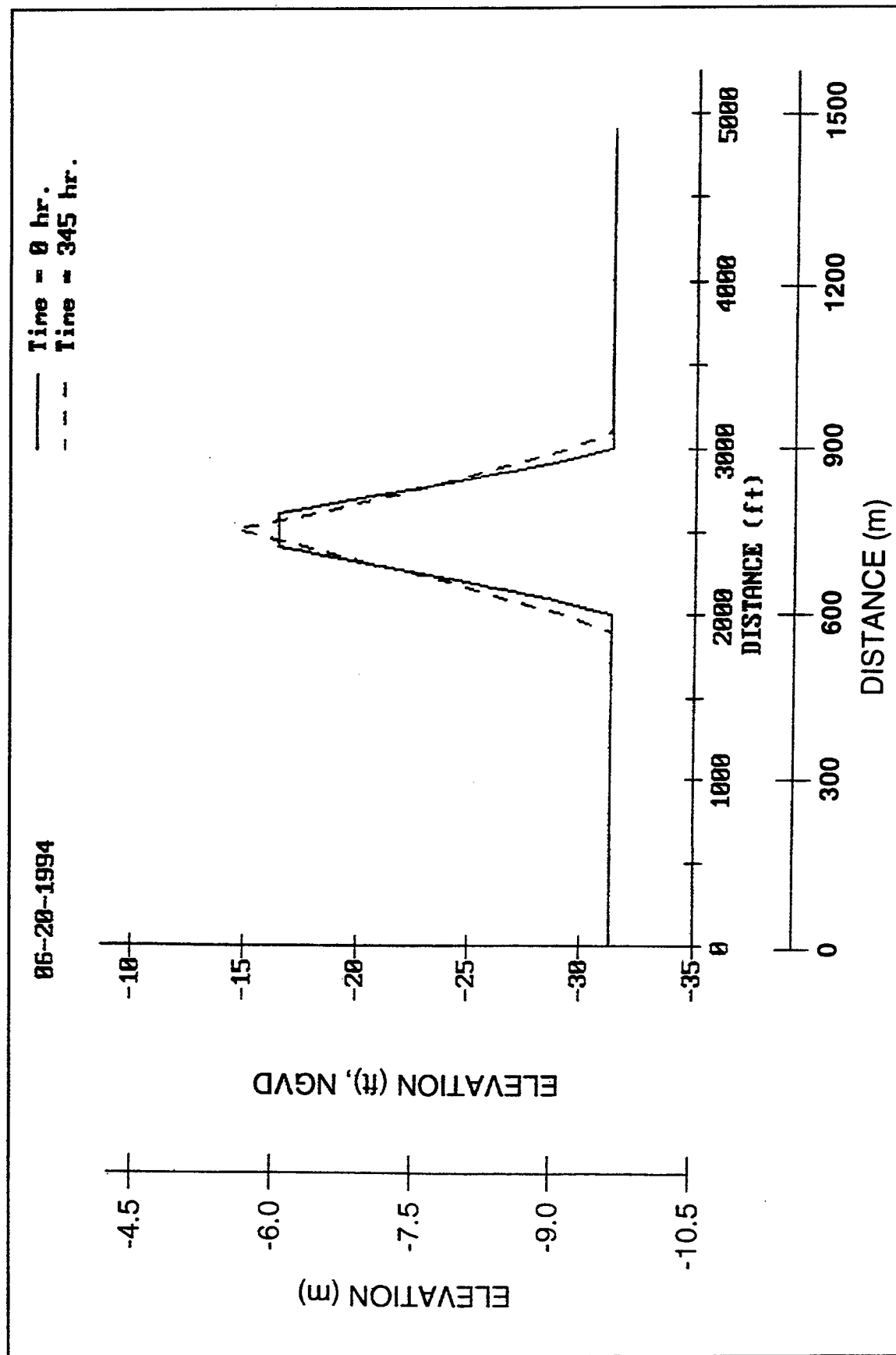


Figure E10. Predicted cross section: 335 m north and south of center line for berm located on Profile 10 resulting from Halloween 1991 storm event (Continued)

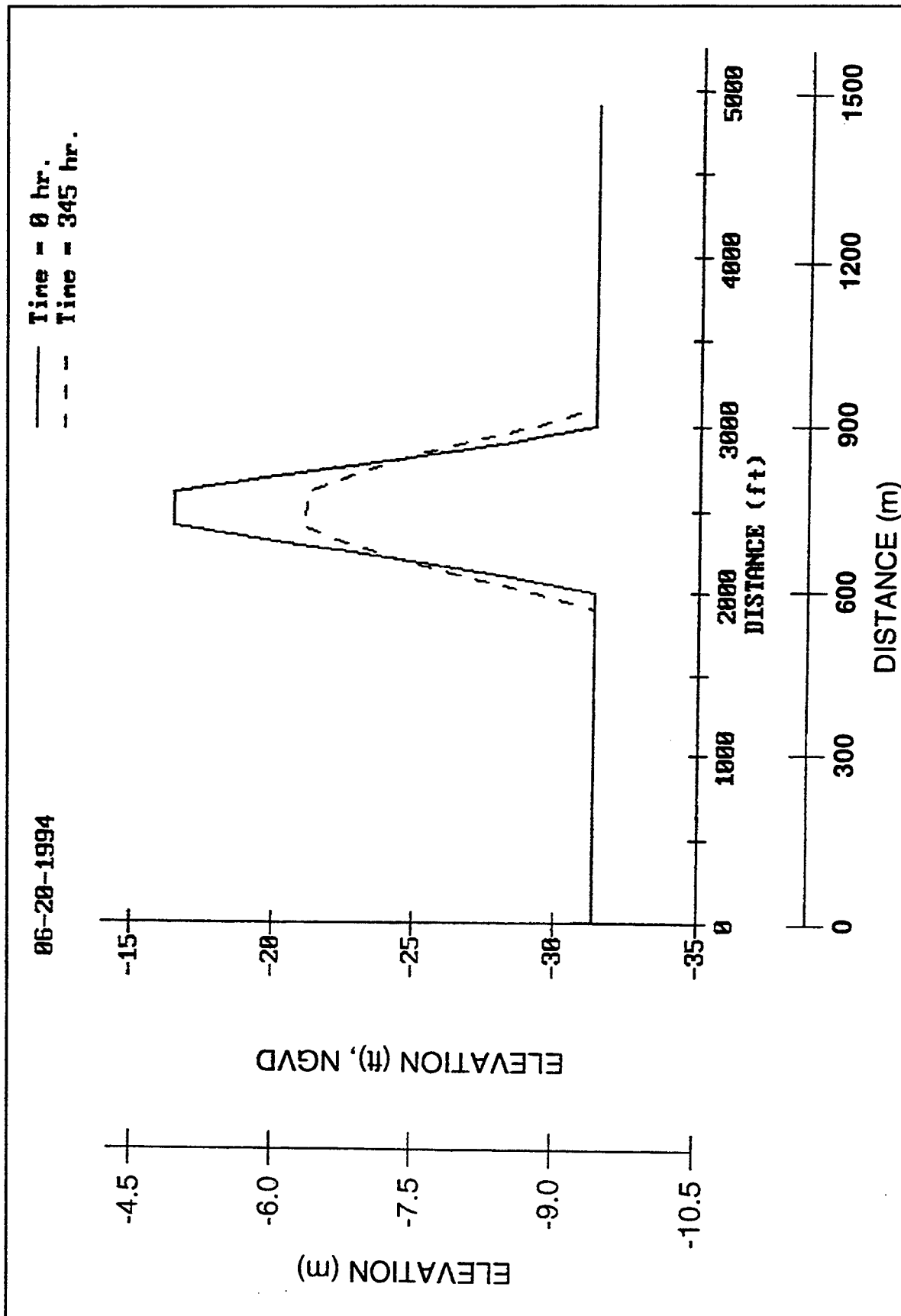


Figure E10. (Concluded)

In all six storm scenarios tested, net volume change of the berm was essentially 0. Berm material remained within the same basic configuration as the initial condition, except that one end of the berm migrated in the alongshore direction at the expense of the opposite end. This occurred because of the unidirectional character of the water column current for each storm. In LTFATE, the direction of sediment transport is governed exclusively by current direction.

Qualitative Results: Profile 10 and Profile 8. Profile 10 experienced more volume change because of the deeper water application, where larger waves can act on the extensive surface area of the berm. While Profile 10 will cost more to maintain its construction orientation, Profile 10 should produce a higher return in benefits than the other berm plan by reducing larger offshore waves (the wave transmission coefficient should be lower).

Profile 8 experienced less volume change than the other plan. Although Profile 8 will cost less to maintain than Profile 10, the wave reduction benefits from Profile 8 may not be as large as for Profile 10.

It is evident from Tables E4, E5, and E6, that Profile 10 will cost more to maintain than Profile 8. Results from a wave transmission study should determine the degree of wave-reduction benefits for each plan.

Summary Statement

The two most critical parameters used in this simulation were current (speed and direction) and berm sideslope (steepest permissible and postavalanched). Development of currents (u,v) for each storm scenario and the average annual condition is explained earlier in this appendix. The berm sideslope envelope was developed in part from existing conditions at the site. The steepest bathymetric gradient encountered on the first offshore bar (existing profile) was 1V:38H. This value was used to define the postavalanched berm sideslope. The steepest permissible sideslope for the berm was assumed to be just less than the construction sideslope, or 1V:24H.

It is assumed in the design and modeling for berm alternative plans 8 and 10 that if a berm is constructed, it will not re-form to a shallower slope than 1V:38H. If the constructed berm does in fact exhibit a shallower slope than that originally assumed, the berm effectiveness will be significantly reduced.

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shoreline erosion. When compared to the without-berm profile, shoreline recession estimates decreased by as much as 70 m per event as the result of placing an engineered nearshore berm on the profile. Following *with-project* and *without-project* numerical simulations, recession estimates may be used as input for event frequency correlation and economic evaluation.