Grand Marais Harbor
Rehabilitation Design Alternatives

Final Report

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1. INTRODUCTION

1.1. Brief History and Problem Definition

The existence of the harbor at Grand Marais was first mentioned in 1660 in the journals of two French travelers and first appeared on a map in France in 1745. Located approximately 55 miles west of the St. Mary's River, Grand Marais is a rare example of a natural and picturesque Great Lakes deep water harbor. It is an important and critical harbor of refuge (River and Harbor Acts of June 14, 1880 and May 17, 1950), along a 90-mile stretch of shoreline deemed Lake Superior's "Shipwreck Coast" (Figure 1.1). Grand Marais Harbor is currently comprised of two bays: West Bay and East Bay. West Bay serves as a small boat harbor with a navigational channel between two parallel jetties. A wide sandy beach is located to the west of the navigational channel, inside West Bay (Figure 1.2).

A federally authorized project consisting of two parallel jetties (about 3,617 feet long and 500 feet apart), and a timber pile breakwater (about 5,770 feet long) was completed in 1903 to provide access into the natural harbor for commercial use that prevailed in the region. Initially, the west jetty was 1,917 feet long, and the east jetty was 1,695 feet long. The Federal navigation channel located between the parallel jetties has authorized depths of 20 feet (outer entrance), and 18 feet (inner channel). The virtual disappearance of the logging and commercial fishing industries around the 1930's resulted in the lack of economic justification to maintain the Grand Marais Harbor breakwater. The last maintenance funds were expended on the breakwater in 1943. Since that time, the channel has been maintained to 15 and 17 feet below datum, which is adequate for the current light draft vessel use.

Upon completion of the harbor jetties, Grand Marais Harbor acted as a complete block of the predominant west to east longshore sediment transport in this area. By 1960, the shoreline on the west side of the of the west jetty had progressed 900 feet lakeward at a rate of about 33 ft/year (from 1894 to 1912), and at a rate of 5.5 feet/year (from 1912 to 1945). The decrease in this rate was attributed to sand bypassing around the end of the west jetty. The US Army Corps of Engineers recommended the construction of an 802-foot long cellular steel sheet pile jetty extension to the west pier to abate the accretion of the bypassed sediments that were accumulating in the navigation channel. The River and Harbor Act (May 1950) provided for this extension and the construction of the modification under this act was completed in 1961.

Concurrently, the bluff at Lonesome Point (immediately east of the navigational channel) eroded 780 feet in length from 1867 to1966. An analysis of the littoral transport is detailed by Bajorunas (1960). It was estimated that the net littoral transport rate was 60,000 cubic yards per year from west to east. Saylor (1968) estimated that 74,000 cubic yards of sediment accreted in the harbor area every year.

A study of the pile dike breakwater (extending Eastward from the entrance channel), conducted in 1960 by the U.S. Army Corps of Engineers (USACE, 1960) found the pile dike had deteriorated to the point where it had lost its functional purpose. A later report prepared for the USACE (Dalton et al, 1975) described that the deterioration of the pile dike breakwater had resulted in increased vulnerability of the harbor to storms emanating from northeast. To date, there has been no further action to restore or repair this structure.

The citizens of Grand Marais, shoreline property owners, and numerous boaters indicate that there is a need for further improvements. With the absence of an essential component of the original structure (timber pile breakwater), the maximum depths inside the east end of West Bay have decreased from 55 feet (mid 1960's) to 25 feet (today). The harbor and the surrounding shoreline have been affected by the continued neglect of maintenance of the harbor structure and the downdrift shoreline, resulting in the loss of valuable aquatic habitat, a deep-water harbor, as well as profound shoreline changes.

1.2. Purpose of Study

The goals of this study are to provide viable and long-term solutions to address the needs and uses of Grand Marais Harbor. This multitask effort involves a comprehensive analysis of all factors affecting the harbor and harbor structures as well as their influence on the surrounding shoreline. Efforts are directed toward determining the best scenario for the future of Grand Marais Harbor including an evaluation of potential structural designs and resultant potential influence on the surrounding shoreline.

Towards these goals, the following tasks were undertaken:

- A literature review of existing information and past reports on the Grand Marais region was undertaken to delineate the history and prior findings with reference to this site. Also, acquisition of historical survey data allowed an updated evaluation of the volume of material currently entering the West Bay shoaling areas.

- An onsite field investigation of Grand Marais Harbor was conducted including detailed bluff, shoreline and offshore bathymetric surveys. These nearshore hydrographic surveys were completed through a combination of conventional land survey techniques and precision acoustic techniques.

- A new aerial photographic survey was acquired to accurately determine the current morphology of the Grand Marais region and the nearshore zone.

- A detailed underwater survey of the existing harbor structure was conducted. This included video, photography and high-resolution acoustic imagery obtained by Michigan's Remote Operated Vehicle for Education and Research, M-ROVER.

- A wind and wave climatology was developed to provide environmental characteristics necessary for input to numerical modeling efforts.

- The execution of a numerical model of wave transformation in shallow water was completed to provide estimates of potential longshore sediment transport and design wave information for structural design specifications.

- The execution of a numerical model of sediment transport in the vicinity of coastal structures was utilized to determine how alternate designs would affect the bathymetry.

- The execution of a numerical model of wave response in and around harbors was performed to evaluate wave conditions due to resonance within the harbor for the alternate designs.

The results of these activities are presented in the following sections. Section 2 summarizes the field and historical data collection activities outlined above. Section 3 describes the nearshore processes computational modeling effort. Section 4 presents five design alternatives to address the needs and uses of Grand Marais Harbor. Section 5 summarizes the design evaluation.

2. FIELD DATA COLLECTION PROGRAM

2.1. Site Selection

In order to address the goals of this research effort, a comprehensive field data collection program was initiated. To evaluate the nearshore processes, and the current condition of harbor and surrounding region, 12 survey lines were established perpendicular to the general trend of the shoreline. Temporary benchmarks (TBM's) were placed at the shoreward end of the survey lines and each site was surveyed using precision nearshore hydrographic survey methods. The 12 selected survey lines are located at 500; 1,500; 3,000; 6,000; 9,000, and 12,000 ft east and west of the two parallel jetties (Figure 2.1).

2.2. Precision Nearshore Hydrographic Surveys

Profile data for the beach and nearshore zone were collected using conventional precision hydrographic survey methods. The survey lines were established perpendicular to the general trend of the shoreline over benchmarks with azimuthal angles turned from easily observable landmarks in the vicinity of the site.

The subaerial and nearshore portion of the survey, extending through wading depth, was conducted using a total station and stadia rod. Horizontal distance and vertical elevation with respect to the benchmark were recorded at intervals that captured the general features of the topography. A buoy was placed shoreward of the offshore end of the wading portion of the survey providing an overlap region with the hydrographic survey. The most accurate hydrographic survey method for the offshore portion of the survey consists of a technique employing a total station and computer-networked fathometer/Differential Global Positioning System (DGPS) system. A survey vessel provided a stable platform from which to conduct this portion of the survey. The fathometer was calibrated for accurate depth recording at approximately the mean depth of the profile. The boat crew placed a second buoy on range with the assistance of the shore crew and total station as well as the DGPS system. In order to assure a near constant speed survey, the boat commenced the fathometer run a sufficient distance offshore of the second buoy thus reaching survey speed prior to the starting point. Tick marks for each buoy location were entered electronically in the real-time survey fathometer data file. The two records, offshore and nearshore, were combined in the laboratory and data quality was assured through matching of the nearshore overlap region. The hydrologic survey profiles for the region can be found Appendix A.

2.3. Random Access Survey

In addition to the conventional precision nearshore hydrographic survey described above, a composite survey grid consisting of soundings obtained along several closely spaced cross-shore transects was obtained for West Bay (see Figure 2.1). The purpose of this survey was to establish the current detailed bathymetry within the basin and provide a basis for the depth grid necessary for the computational modeling effort.

A region of approximately 4500 by 2000 ft was surveyed. This survey began at the boat launch ramp and extended through West Bay and terminated in line with the east jetty. This region ranges in depth from 6 to 53 ft. The results of this survey are presented in Figure 2.2.

2.4. Aerial Photography

Historical aerial photographs of the Grand Marais Harbor area exist for 1939, 1953, 1964, 1975 and 1993. The OEL was successful in located a copy of all but the 1953 air photo. In addition a new set of air photos was obtained in July, 1999 as part of this investigation. These photos are included in Apendix B of this report. Figure 2.3 shows the condition of the site in 1968 as per the U.S.G.S. topographic sheet. This map was derived from the 1964 air photos, the 1967 U.S. Lake Survey chart and was field checked in 1968.

The 1939 aerial photography shows the first aerial image of the Grand Marais harbor including the pile dike, constructed in the late 1800's. By 1939, the pile dike is no longer connected to land at Lonesome Point due to erosion of the point and there are shoals apparent inside the pile dike. The image shows a large accretion fillet west of the harbor jetties and exposed, vegetated land to the east of the jetties. In addition, there is evidence of erosion along the south shore of the bay.

Twenty-five years later, the 1964 air photos show no evidence of the pile dike or shoals. The 1960-61 extension to the west jetty is complete and the accretion fillet remains large. The south shore of the bay also shows an accretionary region through which several small stream channels are cut. The vegetated land to the east of the east jetty remains and shows on the 1968 topographic sheet under the name of Lost Island.

The 1975 aerial photos again show no pile dike. In addition, Lost Island is absent. The accretion on the south shore has continued, spreading further west and East Bay is now open directly to the lake.

In 1993, the shoreline was imaged by a color infrared imagery system to produce a USGS digital orthophoto quadrangle. In this image, it is apparent that material continues to deposit along the shoreline of west bay. The shoreline in the vicinity of east bay has receded 750 feet in some areas and a large portion of East Bay has been filled. The accretion fillet west of the West Jetty has also continued to grow.

The 1999 aerial photography, obtained as part of this investigation, shows a growth of material in the vicinity of what used to be the entrance to East Bay . This accretion has served to smooth out a morphological depression in the shoreline in this area which existed in the 1993 image. This photo set also clearly shows the accumulation of material in the shallow water around the margins of West Bay and in the region to the lee of the dilapidated crib structure.

Based solely upon this photography, it is evident that the presence of the large west jetty serves to almost completely block sediment flow from west to east along this shoreline. This large jetty also acts as a wave barrier for storm waves approaching from the northwest and thus, to some extent, decreases the wave energy attack on the shoreline for a distance of about one mile to its east. This distance is dependent upon the actual incident wave direction and period. Beyond this "shadow" region, the littoral drift is locally maximized with a high erosion potential. For littoral transport to the west, the material which is eroded from Lonesome Point and other areas is transported into West Bay along the shoreline seaward of East Bay and results in the deposition of material which is observed along the bay perimeter, as well as the deeper shoals.

Sedimentology

To characterize the nature of the nearshore sediments, 60 samples were obtained from the 12 established survey lines. About 100 grams of each sample consisting of sediments from the mid beach, the swash zone (subaerial samples), and from 5, 10, and 15 ft of water depth (subaqueus samples) were analyzed. Each sample was cleaned and dried. The individual samples were then dispersed through a mechanical sieve (US Standard Sieve numbers 8 to 230, with mesh openings from 0.097 to 0.0024 inches). The sand size classifications are summarized in Table 2.1. An initial comparison of sedimentological characteristics east and west of the harbor jetties is provided in Figures 2.4 and 2.5. At both sites the deeper water samples consist primary of fine sand. The beach and 5 ft sample east of the jetty are characterized by a much higher percentage of medium sand reflecting the depositional, lower energy regime within the shallow portion of West Bay. The swash samples show a greater trend towards coarse granular material outside of the bay, west of the structures.

Table 2.1 Sedimentological Analysis Grain Size Comparison Chart

Sieve Stamp	Grain Size (inches)	Grain Size (mm)	Grain Size (phi)	Wentworth Size Class	Descriptive Name
29	0.003	0.074	4.00	Silt and Clay	Silt
15	0.015	0.381	1.25	Fine to Med. Sand	Fine Sand
40	0.040	1.016	0.00	Med. to Coarse Sand	Medium Sand
72	0.072	1.829	-1.00	Very Coarse Sand	Coarse Sand
>72	>0.072	>1.829	>-1.00	Granule	Granular

2.6. Underwater Inspection

A detailed underwater inspection using the M-ROVER was performed to examine the existing coastal structures. M-ROVER is equipped with high resolution, low light color video imaging, still photography and three modes of high resolution sonar imaging. During the entire inspection operation, the Ocean Engineering Survey Vessel "Blue Traveler" was used for safety reasons (protecting the M-ROVER cable in the channel), and for providing exact positioning for recording eventual structural problems in the jetty walls. M-ROVER was deployed at the middle of the west jetty, and was navigated south along the jetty. Depth near the wall was 6 feet. The west jetty wooden cribs with iron plates appeared to be in good condition. After reaching the southern end of the west jetty, the M-ROVER crossed the channel to inspect the east jetty, moving north. The timber crib section was in relatively good condition. At location: 46°40.734'N, 85° 58.170'W, in front of survey mark 9 on the jetty wall, a gap in the jetty structure was observed, the joints were open, and rock was filling the cavity (Figure 2.6). A relatively large spacing between the cribs was observed in front of survey mark 12 on the jetty wall. After reaching the north end of the east jetty, the M-ROVER inspected the lakeward end and east side of the jetty, and both ends were found to be in good condition. After crossing the channel back to the west jetty, the northern part of this jetty was inspected and found to be in good condition. The steel cribbing had tight seams, and the toe protection was in good shape. At location 46°40.858'N, 85°58.293'W, there was a gap again recorded between two sections of the timber crib.

2.7. Wind/Wave Climatology

In order to characterize the physical environment of the nearshore and harbor regions, an accurate wind and wave climatology of the area was developed. This data provides a statistical estimate of the mean and extreme wave conditions dominant at the study site. The results of this analysis are used as input to the wave refraction/diffraction and reflection analysis as well as for the littoral calculations.

Data on the wind and wave climate is provided in table form by the Wave Information Study (WIS) of the US Army Corps of Engineers, Coastal Engineering Research Center (Driver et al, 1992). This is a thirty-two year database of marine wave conditions derived from a hindcast model exercised on a three hour interval for all of Lake Superior. The period of record spans the years 1956 to 1987. Statistical summaries of wave conditions were compiled at 91 coastal sites and 4 mid-lake sites. The site closest to Grand Marais Harbor is S53, located at 46°48'N, 86 °00'W (see Figure 1.1).

The statistical values provided in the WIS for station S53 are summarized in Figure 2.7 with the numerical values for the waves incident at Grant Marais supplied in Table 2.2. The summary provides mean and maximum wave height and associated wave period in 22.5° incident directional bins. The table also contains the percentage of waves occurring for each direction.

Table 2.2. WIS Wave Hindcast Statistical Values for Station S 53 near Grand Marais Harbor

Direction (DTN)	Mean Height (ft)	Maximum Height (ft)	Mean Wave Period (s)	Largest Wave Period (s)	Percent Frequency
0.0	4.27	22.97	4.9	10.5	8.5
22.5	3.28	21.33	4.5	10.5	5.2
45	3.28	15.42	4.5	8.5	4.3
67.5	3.28	16.08	4.4	9.5	3.5
292.5	3.94	22.64	4.9	11.0	11.8
315.0	4.27	25.92	5.3	11.0	11.2
337.5	4.92	23.95	5.3	11.0	8.1

This data agrees well with the known climatology of the region based on observational fact. Due to the location of Grand Marais Harbor near the eastern extreme of Lake Superior, it is expected that the dominant direction of approach of severe storm waves and resultant net littoral transport be from the north west quadrant. The mean and maximum values generally decrease as the direction of wave origin moves from west to east. The largest hindcast wave at this site was 25.92 ft with an 11.1 second period from 312°.

The WIS data set also provides information on the breakdown of frequency of occurrence of wave period for each site. The primary peak wave period incident upon this site is in the range of 4.0 to 4.9 seconds. This information is quite useful in determining the potential for deleterious effects of wave resonance within a harbor.

3. COMPUTATIONAL MODELING – NEARSHORE PROCESSES

3.1. Nearshore Wave Refraction and Diffraction Analysis

The previous section summarizes the deep water wave statistics for Grand Marais Harbor. When deep water waves approach a coast, the waves undergo significant modifications due to the interaction of the wave motion with the nearshore bottom. Through this process the wave's height, length, phase speed and direction of travel changes. In the simplest case, as a wave approaches a straight coastline, with planar and parallel depth contours at an angle, that portion of the wave travelling in the shallowest water will move the slowest. This phenomena results in a bending of the wave crests such that they tend to become parallel to the bottom contours and shoreline. This is known as topographic refraction. For a more complex nearshore bathymetry, through this bending of the wave crests, wave energy is concentrated over ridges and dispersed over troughs and depressions.

Wave diffraction is a phenomenon through which energy is transferred laterally along a wave crest. It is most apparent when a regular train of waves is interrupted by a surface piercing barrier, such as a breakwater, jetty, or island. If this phenomenon did not occur, there would exist a perfectly calm area in the lee of a breakwater. Through the process of diffraction, wave energy will move along a wave crest from a region of high wave energy to low wave energy thus "smoothing out" the effects of topographic refraction.

In order to determine the characteristics of the waves interacting with the nearshore region near Grand Marais Harbor, the Regional Coastal Processes Wave model (RCPWAVE) was exercised (Ebersole et al, 1986). The model was developed by the US Army Corps of Engineers as part of the Regional Coastal Processes Numerical Modeling System to predict coastal processes around man-made structures. This model utilizes a detailed nearshore bathymetry grid and deep water wave conditions as input. It produces a detailed wave height, number and direction grid based upon its calculation of the interaction of the incident deep water waves with the nearshore bathymetry.

In order to initiate the RCPWAVE modeling portion of this study, it was first necessary to establish the geometry and bathymetry of the study site. The coordinate system, datum, grid spacing, and nearshore bathymetry were defined based on the precision nearshore hydrographic surveys completed east and west of the jetties, the random access survey performed inside West Bay and digital NOAA nautical charts. The offshore wave climatology was derived from WIS statistical summary described in Section 2.7.

The numerical rectilinear grid cell size was set at 50 feet in the cross-shore direction and 100 feet in the longshore direction. This grid spacing was dictated by the necessity of fully characterizing the wave field with at least 3 data points per wavelength for the shortest analyzed wave. Figure 3.1 provides a contour plot of the nearshore bathymetry utilized in this investigation.

Each incident WIS wave direction was analyzed to determine the significant wave height and associated period, as well as percent frequency of occurrence. The model input is limited to wave approach directions less than 67.5° off normal.

Under conditions of average waves approaching from the northwest, the results of the numerical wave refraction/diffraction program show that the wave energy is decreased in lee of the harbor jetties and this results in decreased wave heights over the expanse of the region of shoreline previous protected by the pile dike breakwater. The results of the storm conditions from the northwest show a region of decreased wave heights, but the distance of shoreline over which the sheltering occurs is smaller. There is a small sheltered area due to the west jetty from waves from the northeast under both average and storm conditions. The marked line of elevated wave heights within West Bay are an unrealistic remnant of the limitations with complex bathymetric cases. Selected plots of the RCPWAVE output are presented in Appendix C.

3.2. Potential Longshore Sediment Transport

Longshore transport is defined as the movement of sediment in the nearshore zone resulting from waves breaking. When a wave breaks, it stirs up sediment which is carried parallel to the shoreline by the currents created by the breaking process. The rate of potential longshore transport depends on the incident wave angle, duration and wave energy. The potential sediment transport rate is empirically related to the Longshore Energy Flux Factor (P_{ls}) by the following expression:

$$Q\left[\frac{yd^3}{yr}\right] = 7500\left[\frac{yd^3 - s}{lb - yr}\right]P_{ls}\left[\frac{ft - lb}{ft - s}\right]$$

The units associated with each quantity are inside the brackets. The Longshore Energy Flux Factor is related to breaking wave height and angle by the following equation:

$$P_{ls} = 0.0884\rho g^{3/2} H_{sb}^{5/2} \sin 2\alpha_b$$

where ρ is the density of water, g is the gravitational constant, H_{sb} is the significant wave height at breaking and α_b is the wave breaking angle.

The significant wave height and approaching angle at breaking are estimated by the RCPWAVE numerical model output for each 100 ft alongshore grid cell. The contribution of each of the seven average incident offshore wave conditions to the longshore transport rate was calculated. The net potential longshore sediment transport in each bin was evaluated based on the percent frequency of occurrence of each of the average incident wave conditions. Figure 3.2 shows the annual potential longshore sediment transport obtained from this analysis.

In general, this area is subject to significant waves from both the east and west. The smaller, less frequent waves from the east act to move sediment from the east into West Bay, while the larger more frequent waves from the west cause sediment to accumulate west of the jetties and act to erode material from Lonesome Point and carry it further east. The net result is a dominant transport of material to the east, with a smaller return flow into West Bay.

The results of this numerical analysis show that the average potential net longshore sediment transport rate for the entire region is approximately 1,350,000 cubic yards per year to the east.

This value reflects the net amount of material which would pass a shore perpendicular plane in the course of one year provided an infinite source of sand-sized material was available for transport in the nearshore zone. Since this is not the case in this area, the actual amount of longshore transport is much smaller. However, relatively speaking, this high value of potential transport is indicative of a highly energetic coast. Due to the shadowing effect of the harbor structures, the most frequent and severe waves incident upon this site, those from the northwest, do not directly act upon the shoreline within about 1500 feet of the east jetty. This can be seen as a reduction in the amount of longshore transport to the east within this region (Figure 3.2), and an increase in that to the west. Average numerical model predictions of sediment transport along this section of shoreline (both updift and downdrift of the harbor structures) is approximately 125,000 cubic yards per year. This is the mechanism by which material is deposited within the bay.

Actual longshore sediment transport values were estimated by comparing bathymetric data from historical nautical chart surveys. After the deterioration of the pile dike breakwater, the central bay region began to fill with sediment. A comparison between bathymetric data from the 1967 and 1997 navigational charts allows for a calculation of sediment inflow into this region over this thirty year time span. This volume is 3.1 million cubic yards resulting in an average rate of 103,000 cubic yards per year. This value agrees well with previous work of the USACE estimating that between 1966 and 1978, West Bay had been shoaling at a rate of over 100,000 cubic yards per year (USACE, 1980). This value is also very similar in magnitude to the potential longshore sediment transport to the west at a point about 1000 feet east of the east jetty (as per Figure 3.2).

4. PROPOSED DESIGN ALTERNATIVES

Potential alternatives suggested by either the U.S. Army Corps of Engineers or by the Ocean Engineering Laboratory of the University of Michigan are presented in this section. From this array of possible alternatives, only those which provide a viable and environmentally sound solution are considered.

The U.S. Army Corps of Engineers Section 111 study for Grand Marais Harbor (USACE,1980) examined nine proposed design alternatives:

1. No mitigative action
2. Shoreland regulation
3. Restoration to pre-project conditions
4. Construction of a continuous revetment
5. Construction of protective beaches
6. Establishment of a sand-bypassing program
7. Construction of a detached breakwater (replace pile dike)
8. Construct groin field
9. Headland protection

Option 2, the shoreland regulation action provides insight to the problem for future shoreline development and construction, but does not address the needs and uses of Grand Marais Harbor itself. Restoration of the area to pre-project conditions (option 3) would ultimately render this Harbor of Refuge useless and leave a 90 mile stretch of coastline known for severe storms without protection for small watercraft. The proposed options 4, 8 and 9 address the erosion of Lonesome Point and either do not address or negatively impact the sedimentation problem in West Bay. The construction of protective beaches in West Bay from the east jetty to Lonesome Point (alternative 5) would increase the material already available for transport into West Bay.

The following sections examine the three remaining feasible design alternatives with estimated current costs. Cost estimates for each of these design alternatives as well as the design cross-sectional sketches of the three breakwaters were graciously provided by Prein and Newhof Engineering of Grand Rapids, Michigan.

4.1. No Mitigative Action

The no action alternative would allow the conditions of Grand Marais Harbor and the surrounding shoreline to continue to deteriorate. If no action is taken, the bluffs at Lonesome Point will continue to erode, West Bay would continue to fill and take on the shape of a crenulate-shaped bay, and the length of the shoreline affected by erosion will increase with time. The area around the delta formed in East Bay by the Sucker River will be exposed to a greater wave action, which will eventually redistribute the eroded material within the bay. This will protect the shoreline along some properties and may erode some others, having an effect in increased property values in accretion areas and decreased property values in erosion areas. It is estimated that a total of about 70 acres of land east of Lonesome Point will erode during the next 30 years (USACE, 1980). West Bay could remain functional, but dredging maintenance may increase dramatically. If West Bay continues to fill with sediment, this will result in less erosion of Lonesome Point, as there will be some sediment available for the downdrift beaches. The area in front of the old pile dike will continue to erode, creating a deeper zone which will allow larger waves to come into the harbor, also affecting the dock area.

4.2. Establishment of a Sand Bypassing Program

The establishment of a sand bypassing program can be implemented in two ways; annual maintenance dredging or a continuous permanent bypass facility. The first option consists of periodic maintenance dredging of material trapped by the west jetty as well as dredging of the entrance channel. This material would then be placed either directly on the beach or in the nearshore zone as feeder material to a downdrift location. Proper placement of the feeder beach would limit the amount of sand available to travel back to the west into West Bay. The quantity of bypassed sediment is estimated to be approximately 100,000 cubic yards per year. Maintenance dredging requires the periodic use of a hydraulic dredge and piping as needed. The cost estimate for this activity is found in Table 4.1. This alternative would disrupt harbor traffic unless a permanent pipe facility was constructed. The construction of a permanent pipe facility to avert the need to close the harbor channel during dredging is also quoted in Table 4.1. This one time cost estimate does not include the construction of a permanent pump station for continuous bypassing. The quoted "allowance" is estimated to cover construction contingencies, engineering and project administration.

Table 4.1 Breakdown of Costs for Sand Bypassing Systems

Annual Maintenance Dredging	Cost
Dredging 100,000 cu. yd. Per yr. @ $4/cu. yd.	$0.40 M
Mobilization/De-Mobilization	0.02
Allowances	0.08
TOTAL	**$0.50 M**
Permanent Pipe Facility	
24" channel crossing by directional drilling 2,000 lineal feet @ $400/ft.	$0.80 M
Concrete Junction Boxes (2)	0.20
Allowances	0.18
TOTAL	**$1.00 M**

4.3. Reconstruction of the Pile Dike Breakwater

The US Army Corps of Engineers evaluated two potential detached breakwater designs. The first re-establishes the line of the original pile dike breakwater and the second extends on a line at 15° to the original breakwater (Figure 4.1). The current research suggests a third breakwater design alternative consisting of a breakwater at 55° to the original breakwater. The designs presented and priced herein are for rubble mound wall, the dimensions of which were determine based on the average water depth available from the most recent bathymetric information from the Grand Marais area. Each breakwater would be constructed using a geotextile base layer with three stone layers. All three alternatives allow a 1.5 to 2.0 foot freeboard height above the design wave height to allow for the existence of larger waves, slight structural settling or inaccuracies in construction placement. For each structural alternative it is recommended that a sand by pass scheme be considered.

Table 4.2 provides basic design considerations and wall dimensions for each of the three breakwater design options. Cost estimates are provided in Table 4.3 and structure cross-sections in Figures 4.2, 4.3 and 4.4. As expected, construction of the structure in the original location has the highest cost estimate at $32.8 million. The 15° wall has the second highest cost estimate at $19.5 million and the 55° wall has the lowest cost estimate at $5.6 million. The large cost items are the rock wall and sand excavation.

Table 4.2 Grand Marais Breakwater Design: Rubble Mound Wall

	Basic Data				**Wall Dimensions**			
	Design Wave Height (ft)	**Wall Length (ft)**	**Sand to be Moved (cu yd)**		**Area (sf)**	**Length (ft)**	**Volume (cf)**	**Weight***
Reconstruction Original Location	10	7,000	2,000,000		500-900 Avg 800	7,000	5.6 M	325,000
15° Wall	8	4,800	1,500,000		250-730 Avg 600	4,800	2.9 M	170,000
55° Wall	5	2,500	100,000		300-600 Avg 500	2,500	1.3 M	75,000

*Assume 30% voids & weight of rock = 165 lb/cf

Table 4.3 Cost Estimate (Amounts in $1,000,000)

Reconstruction				**15° Wall**		**55° Wall**	
Item	**Unit Price**	**Qty**	**Amt**	**Qty**	**Amt**	**Qty**	**Amt**
Mob/De-Mob	$100,000	1	0.10	1	0.10	1	0.10
Rock	$55/T	325,000	17.88	170,000	9.35	75,000	4.13
Geotextile	0.20/sf	1.4M	0.28	900,000	0.18	400,000	0.08
Sand Excavation	$4/cy	2.0M	8.00	1.5M	6.00	0.1M	0.40
Allowance for Construction Contingencies, Engineering, Project Administration			6.54		3.87		1.19
Totals			**$32.80 M**		**$19.5 M**		**$5.90M**

It is important to note that the large rock volumes for the two higher cost alternatives may require more than one construction season in order to develop an adequate volume of rock from the mine. Also, the high cost for the same two alternatives may limit the number of contractors that are available to perform the work. It may, however, be possible to reduce the cost of construction at the original location by salvaging rock from the original wall. Also, should the construction of a concrete walkway and railing atop the wall be desirable, this would add approximately $300 per lineal foot and $40 per lineal foot, respectively.

5. COMPUTATIONAL MODELING - DESIGN EVALUATION

5.1. Bathymetric Response to Coastal Structures

Man-made structures, such as jetties and breakwaters, disturb longshore sediment transport by trapping sediments up-drift and behind the structure causing erosion down-drift of the structure. A numerical model of bathymetric evolution was used to estimate the bathymetric response to time-varying wave conditions and shoreline modification by including the effects of wave refraction and diffraction and accounting for both longshore and onshore/offshore sediment transport.

The model utilized in this investigation, N-line is a numerical model that uses a combination of analytical and empirical sediment transport equations to simulate sediment transport in the vicinity of coastal structures (Perlin and Dean, 1983). The sediment transport equations were employed using a fixed longshore and depth grid system where the cross-shore distance of bathymetric contours is allowed to change. This method estimates the bathymetric changes due to a littoral barrier in terms of its effect on the depth contours.

The bathymetry of the study area is represented by long shore cells of equal spacing, Δx, and a fixed number of contour lines, n, each at a specified water depth, h. The simulations of nearshore change in this application involved modeling 10 bathymetric contour depths: 1, 2, 3, 5, 10, 15, 20, 25, 30, and 32.8 ft. The cross-shore position of these contours, y, was determined by the equilibrium profile which best fit this region and its sedimentology. The equilibrium profile is a depth profile of the form

$$h = Ay^{2/3}$$

where h is the water depth, A is the parameter which is used to fit the profile data to this ideal profile and y is the distance offshore.

The sediment transport algorithm consists of two components, a longshore component, Q_x, and an onshore/offshore component, Q_y. The net longshore and cross-shore transport into or out of each grid cell determines the bathymetric response (Figure 5.1). The model has fixed boundary conditions; therefore, the region being evaluated must extend far enough away from the structure to limit the interactions between the boundary and the structure.

The numerical model was exercised for the present conditions as well as the three alternate breakwater designs presented in Section 4. The wave field utilized in this phase of the modeling effort was a three-year time history from the WIS time series, selected to represent typical Lake Superior wave conditions.

The interpretation of the results of this analysis are limited by the ability of the model to adequately account for the variable bathymetry in this region with a single value for A. As a result, Lonesome Point appears to erode rapidly during the first month to one year time period. This is a result of the complex shoreline orientation and the tendency for the numerical model to smooth shoreline bumps. This is followed by more slowly varying changes to the bathymetry which provide more information for comparison between the different structures. Thus, the results provide a relative rather than absolute indication of the structural impact. Table 5.1 summarizes the results of the numerical shoreline change model analysis for the four design options. Figure 5.2 shows the basis for the measurements presented in table 5.1 as well as a guide to interpretation of the numerical model output included in Appendix D of this report.

Table 5.1 Summary of results of numerical shoreline change model.

	Downdrift Impact (ft)					
	1 Month		**1 Year**		**2 Years**	
	Long-shore	Cross-shore	Long-shore	Cross-shore	Long-shore	Cross-shore
No BW	7000	1100	9000	1400	15000*	1500
55° BW	4000	600	4000	600	6000	500
15° BW	4000	700	6000	600	6000	600
0° BW	3000	250	6500	600	7000	500

** reached boundary of computational grid*

The table shows the extent of the impact downdrift from Lonesome Point in the longshore and cross-shore directions. It can be noted that following the initial 1 to 12 month adjustment period, the largest changes to the erosional area at Lonesome Point occur for the current configuration reaching the outerboundary of the studied area (15,000 ft). All three breakwaters show very limited growth of the erosional region following the initial adjustment period.

5.2. Harbor Resonance Model (HARBD)

A harbor is dynamically similar to a mechanical or acoustical system, where certain wave oscillation phenomena produce resonance (the amplification of the wave amplitude at a frequency unique to the geometry). In the case of a harbor, this can result in areas of extremely high wave height within the harbor. When the incident wavelength is on the same order of magnitude as one of the characteristic dimensions of the harbor basin, the wave amplitude in the harbor basin can be significantly larger than the amplitude of the incident wave. This phenomenon is commonly referred to as “Harbor Resonance”, “Seiche”, or “Harbor Surging”.

Some important factors to consider when studying harbor resonance include: large amplitude waves and storm waves, frictional energy loss at narrow passages and on the sea bottom, topographic features in the surrounding areas, such as natural beaches, and bathymetric changes. Natural beaches and non-vertical walls allow damping of energy and reduce harbor resonance.

HARBD is a harbor resonance model based on a numerical model by Chen and Mei (1974) and modified by Purdue University’s Great Lakes Coastal Research Laboratory. This model uses a finite element analysis approach for the solution of the conservation of energy equations. This approach provides the means of including the effects of wave refraction, diffraction and reflection as well as accounting for variable depths and boundary conditions.

The finite element mesh upon which the model was exercised was generated from the Nearshore Precision Hydrographic Surveys and the Random Access Survey. Additional shoreline and structure information was garnered from the NOAA Nautical Chart. The mesh had positive valued depths with the still water level on the day of survey used as the zero reference. There was a local coordinate system with (0,0) being located in the middle of the entrance channel in line with the seaward end of the west jetty. Each grid was composed of triangular elements, and the structures are modeled as voids in the grid with the surrounding elements containing the material properties.

HARBD was exercised for the three proposed design alternatives. The wave climatology and geographical orientation of the region were analyzed to determine each incident wave direction and periods of interest. Seven wave directions were input into the model ranging from 67.5 to –67.5 degrees relative to North, at 22.5 degree increments, and the wave periods ranged from 3 to 6 seconds in 0.5 second increments, resulting in 147 simulation runs. The model produces maps of the wave amplitude factor at each nodal point within the harbor. The wave amplitude factor is the number by which the wave height is multiplied to obtain the local resonant wave height at that point. Sample model output is supplied in Appendix E, along with summary graphs as described below.

To quantitatively determine the effect that each design alternative induced in the wave amplitude within the harbor, five sites within the harbor channel and West Bay were examined for wave amplitude factor under the various incident wave conditions (see Figure 4.1):

A. the center of the channel entrance (Lake Superior side)
B. a point in the center of the channel half way between the entrance and exit
C. the center of the channel exit (West Bay side)
D. a point opposite the channel near the south shore
E. a point in the middle of West Bay west of the channel

The numerical wave amplitude factors for each period at each of these points was plotted for each design alternative and incident wave direction. These graphs are included in Appendix E. To present a summary of this information, the mean wave amplitude factor over the spectrum of wave periods exercised in this model evaluation was calculated for each design alternative and location and plotted versus the direction of wave approach (Figure 5.3). The mean wave amplitude factors within the harbor are all less than 0.4, indicating that very little resonance occurs within this harbor at the wave periods examined. The strongest response, although small in comparison to typical resonant amplitude factors, occurs within the existing channel for waves approaching from the east northeast with the 0° and 55° breakwall. Also, the channel suffers a stronger response under directly incident waves (from the North) with the 55° breakwall. However, within the harbor, itself, the response is very low with mean wave amplitude factors less than about 0.05 inside the basin and 0.1 at the south shore. The response within the channel is quite likely due to a characteristic frequency which is related to the channel width or length itself, which is not modified by any of the proposed design alternatives. Thus, it is not anticipated that any one of the breakwall configurations will adversely impact the wave climate within West Bay.

6. CONCLUSIONS AND RECOMMENDATIONS

The goal of this study is to provide viable and long-term solutions to address the needs and uses of Grand Marais Harbor. This multitask effort involved a comprehensive analysis of the factors affecting the harbor and harbor structures as well as their influence on the surrounding shoreline.

Based upon historical data analysis, it is evident that the presence of the large west jetty serves to almost completely block the dominant longshore sediment flow from west to east along this shoreline. This large jetty also acts as a wave barrier for storm waves approaching from the northwest and thus, to some extent, decreases the wave energy attack on the shoreline for a distance of about one mile to the east. This distance is dependent upon the actual incident wave direction and period. Beyond this "shadow" region, the littoral drift is locally maximized with a high erosion potential. For littoral transport to the west, the material which is eroded from Lonesome Point and other areas is transported into West Bay along the shoreline seaward of East Bay and results in the deposition of material which is observed along the bay perimeter, as well as the deeper shoals.

The field data acquisition consisted of bathymetric and sedimentological data which were necessary to build the bathymetric grids upon which the numerical modeling portion of this study was exercised. In addition, an on-site investigation of the condition of the existing harbor structures was completed.

The results of the numerical model of shallow water wave transformation support the observations discussed above. The longshore sediment transport calculations performed by use of the wave model, estimations from bathymetric data and USACE estimates all show a shoaling rate of just over 100,000 cubic yards per year in West Bay.

The most reasonable mitigation options are described in section 4 of this study. They include a "no mitigation action" (section 4.1), establishment of a sand bypassing program (section 4.2), and reconstruction of all or portions of the pile dike breakwater (section 4.3). The consideration of these options is predicated on the concept that a stable shoreline must be maintained to the east of the harbor entrance. This can only be accomplished through a concerted sediment bypassing commitment. As indicated, through all phases of this study, approximately 100,000 cubic yards of nearshore material per year must be supplied to the downdrift (easterly) beaches to provide a stable nearshore environment.

The choice of mitigation option is also dependent upon the local desire to maintain and/or preserve the Grand Marais harbor. Each of the construction options preserves the basic nature of the harbor of refuge with varying costs of construction and long term maintenance. Without these modifications, all indications are that, the former deep water harbor will continue to fill with sediment at a rate of approximately 100,000 cubic yards per year and erosion of Lonesome Point, to the east, will accelerate.

Based upon the numerical analysis of the design options, the no action alternative represents the largest danger to the continued erosion of Lonesome Point. The presence of a breakwater will serve to decrease the shoaling of material within West Bay and thus decrease the loss of sediment to the longshore system in the vicinity of Lonesome Point. The analysis of the model output shows that one month after the implementation of the design alternatives the least amount of shoreline change corresponds to the 0° wall design. On the other hand, after an adjustment period of about two years, there are no significant differences in the simulated performance of the suggested designs. According to this, and taking into count the difference in cost of the three alternative designs, about 13.5 million dollars, the 15° wall or the 55° wall design will be the most practical and cost effective solutions. In addition, based upon this analysis, none of the proposed breakwaters pose an obvious threat to the wave conditions within the harbor.

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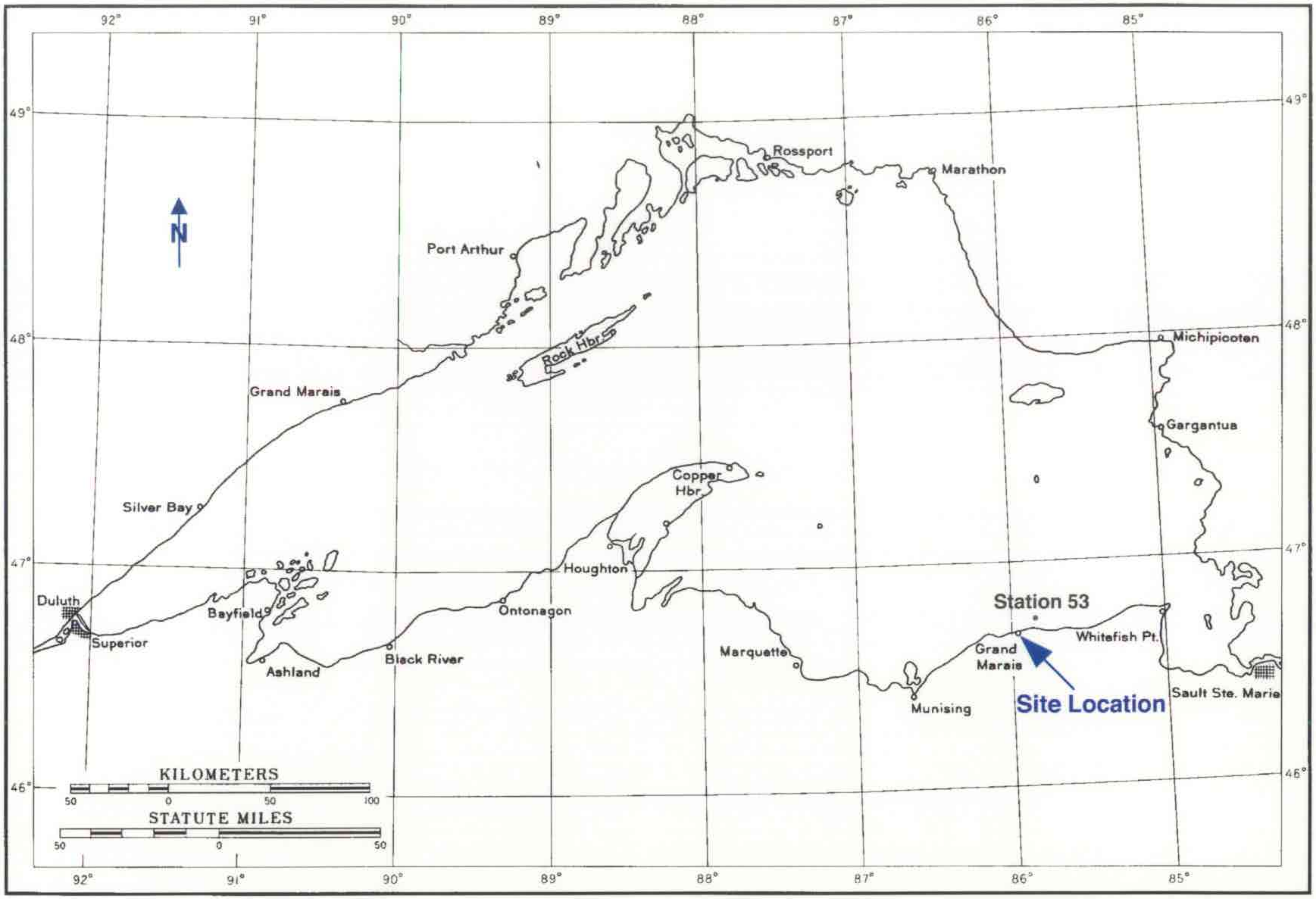


Figure 1.1 Map of Lake Superior showing study site location and Wave Information Study Station S53.

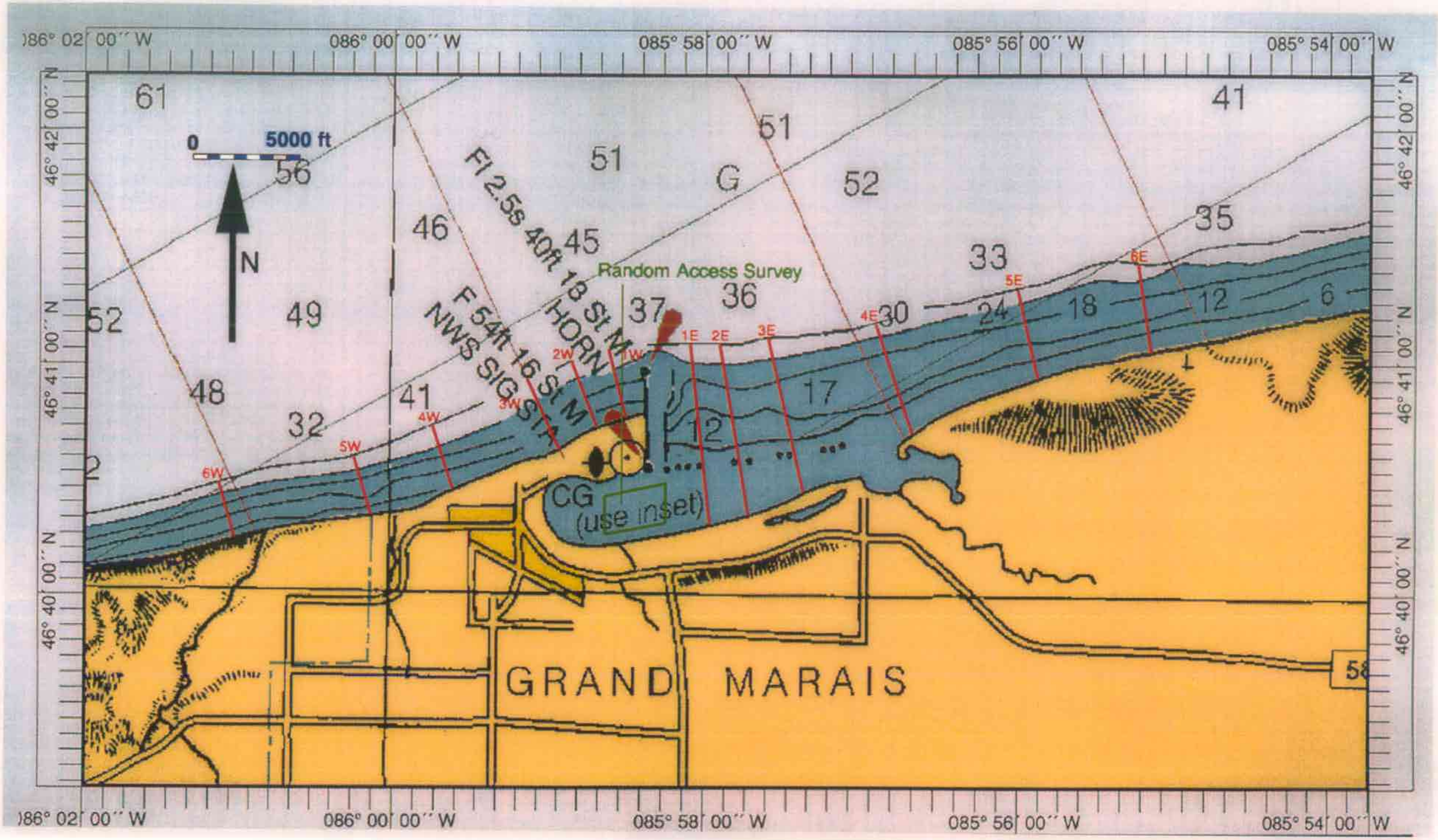


Figure 2.1 1997 nautical chart of Grand Marais Harbor showing precision nearshore hydrographic survey ranges and random access survey region.

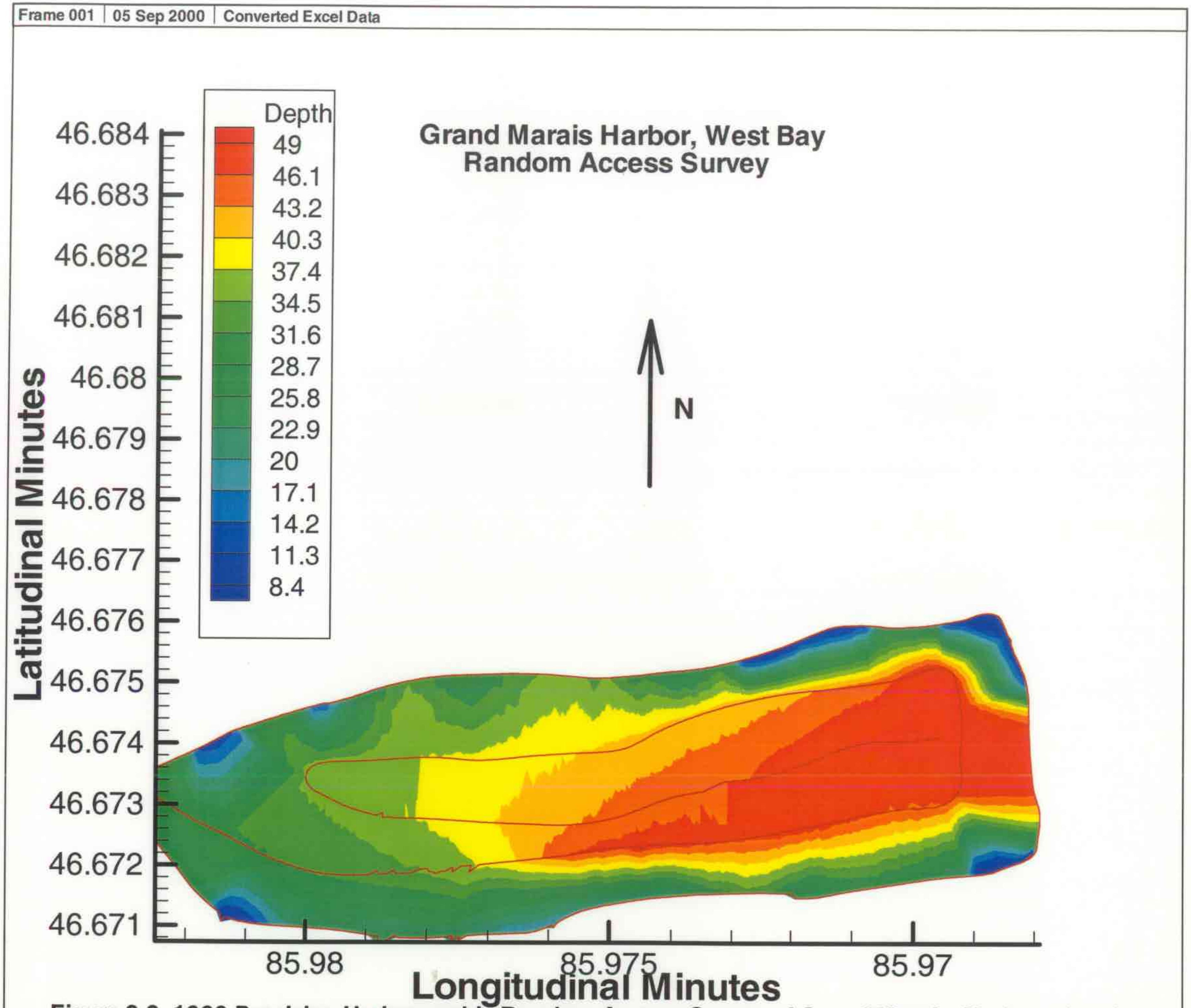


Figure 2.2 1999 Precision Hydrographic Random Access Survey of Grand Marais Harbor showing boat track line and depth contours within harbor with respect to still water level on August 10, 1999.

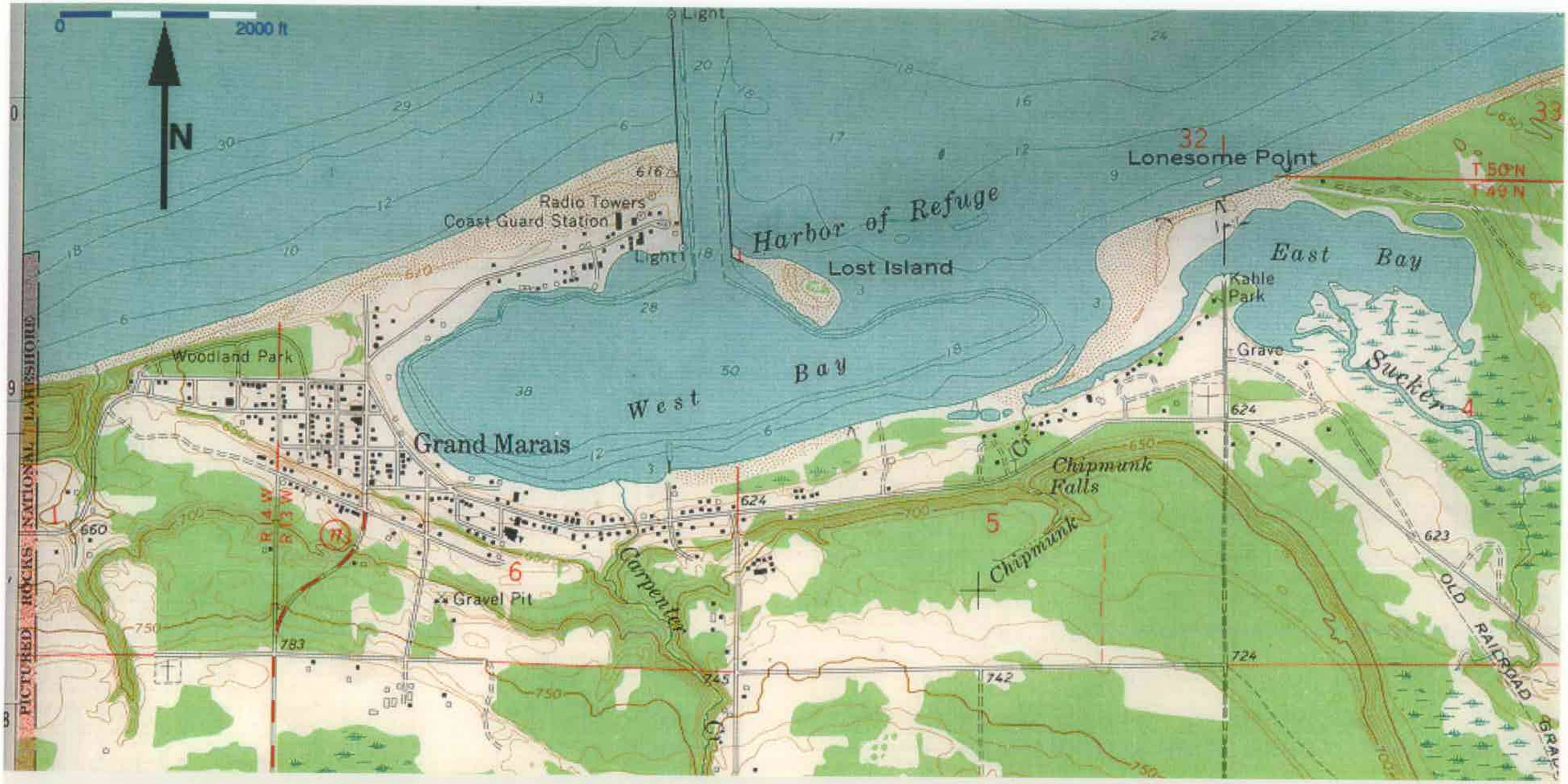


Figure 2.3 1968 USGS topographic map of Grand Marais Harbor region.

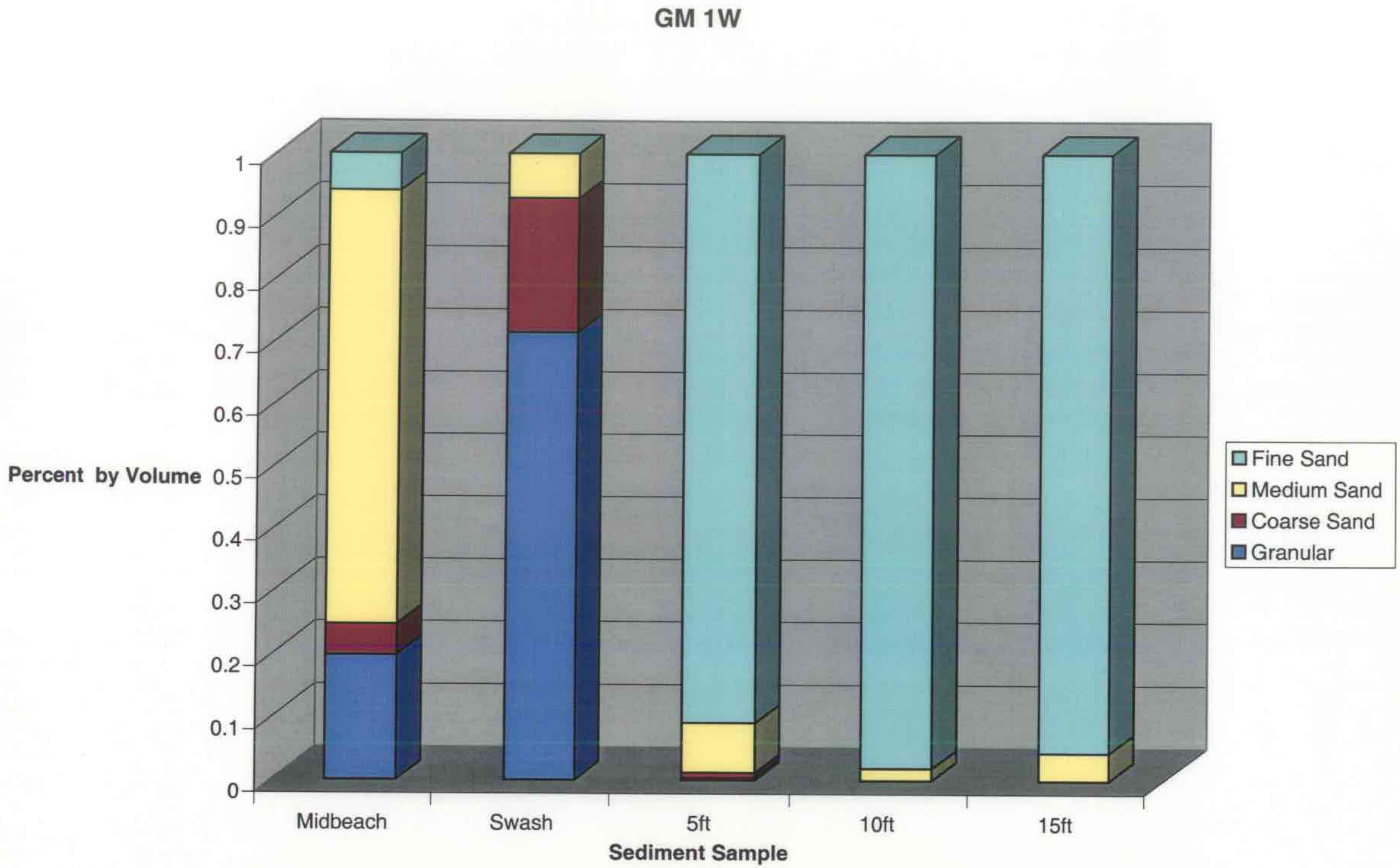


Figure 2.4
Bar chart of beach and nearshore sedimentology 500 ft west of the West Jetty, Grand Marais Harbor.

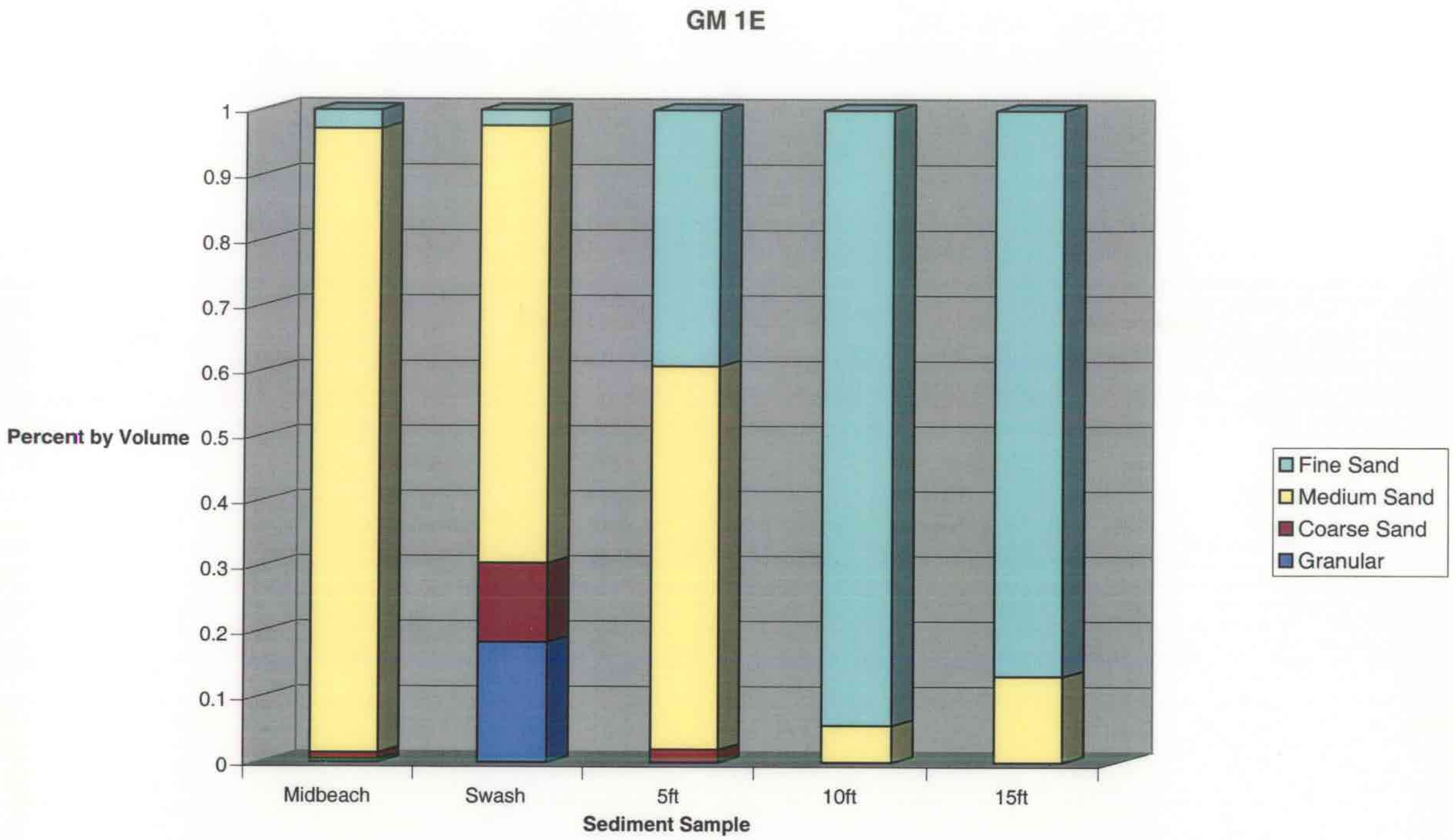


Figure 2.5
Bar chart of beach and nearshore sedimentology 500 ft east of the East Jetty, Grand Marais Harbor.

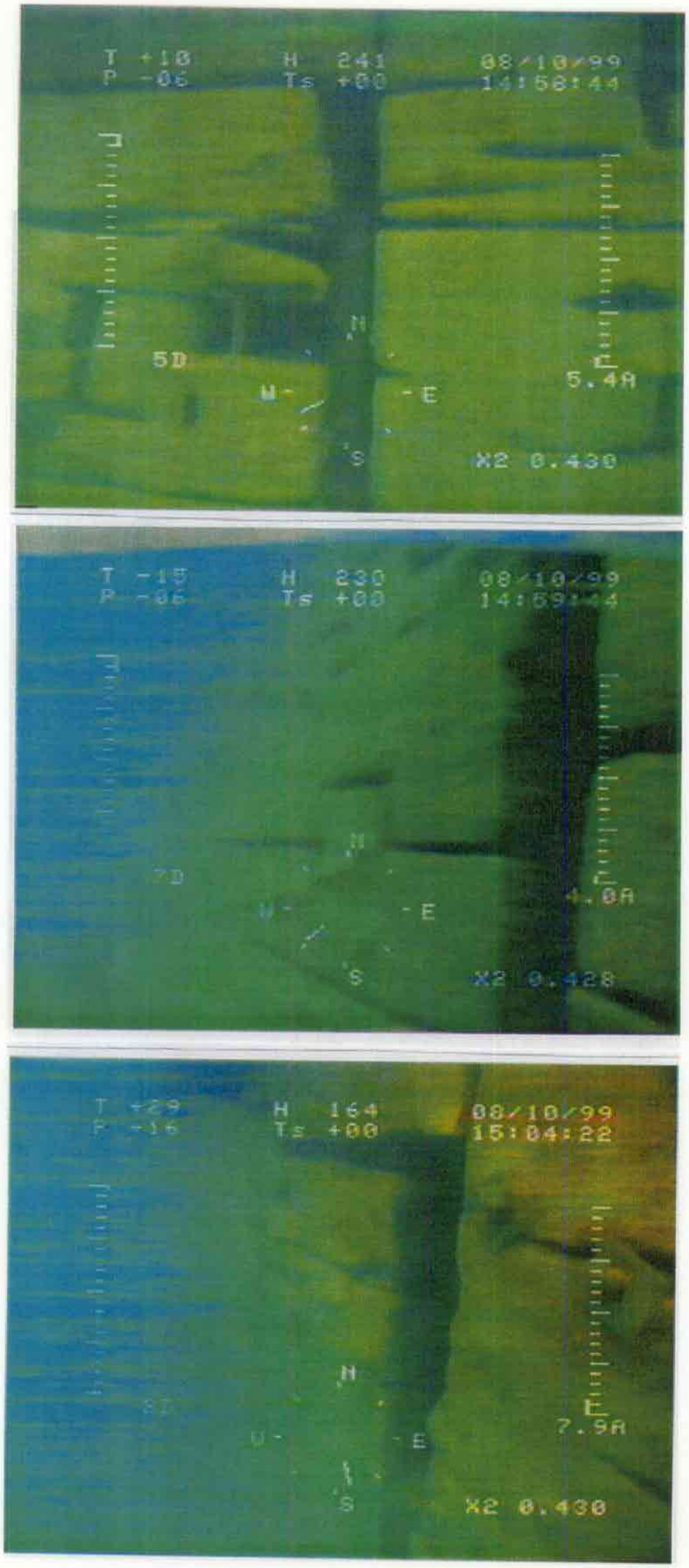


Figure 2.6 Video frame showing east jetty interior wall with open joints (1999).

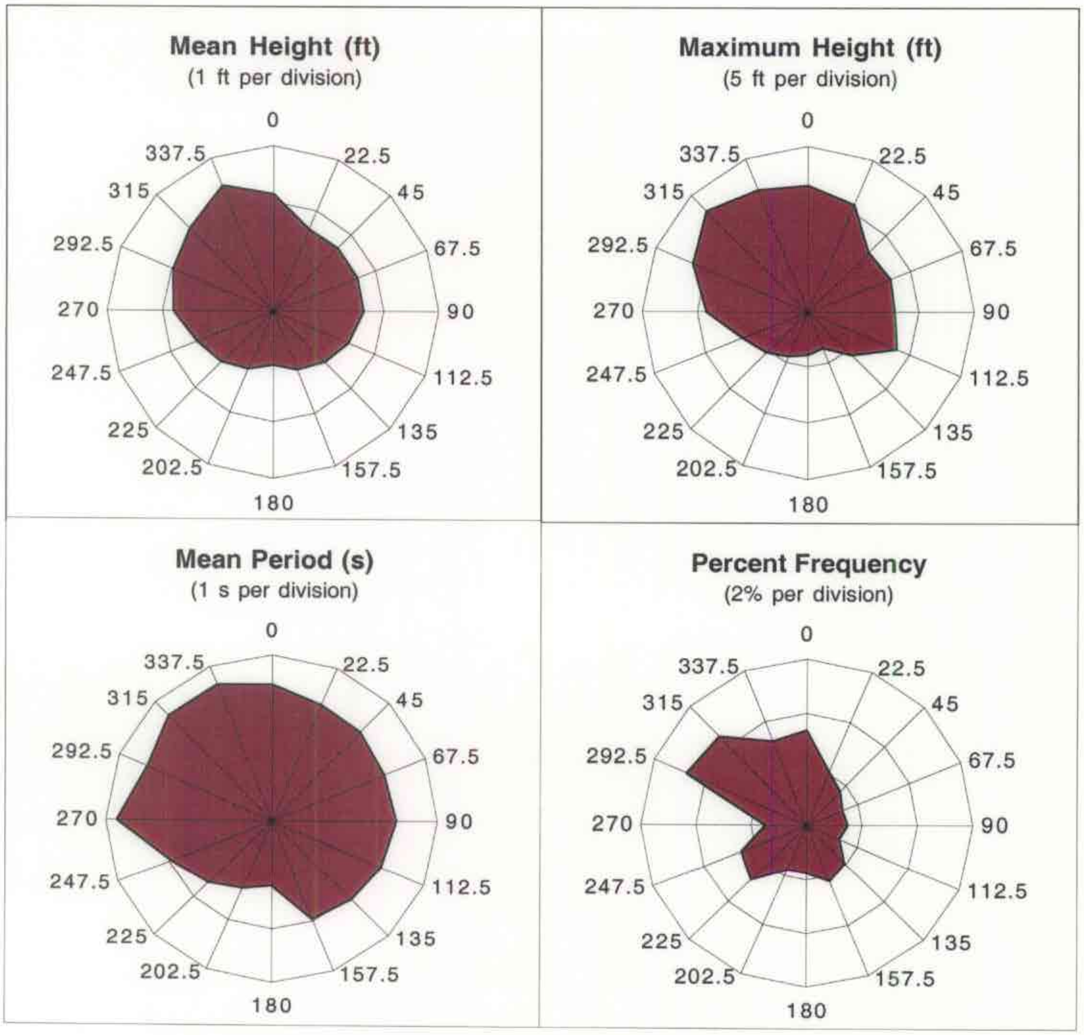


Figure 2.7 WIS station S53 wave hindcast statistics near Grand Marais, Michigan

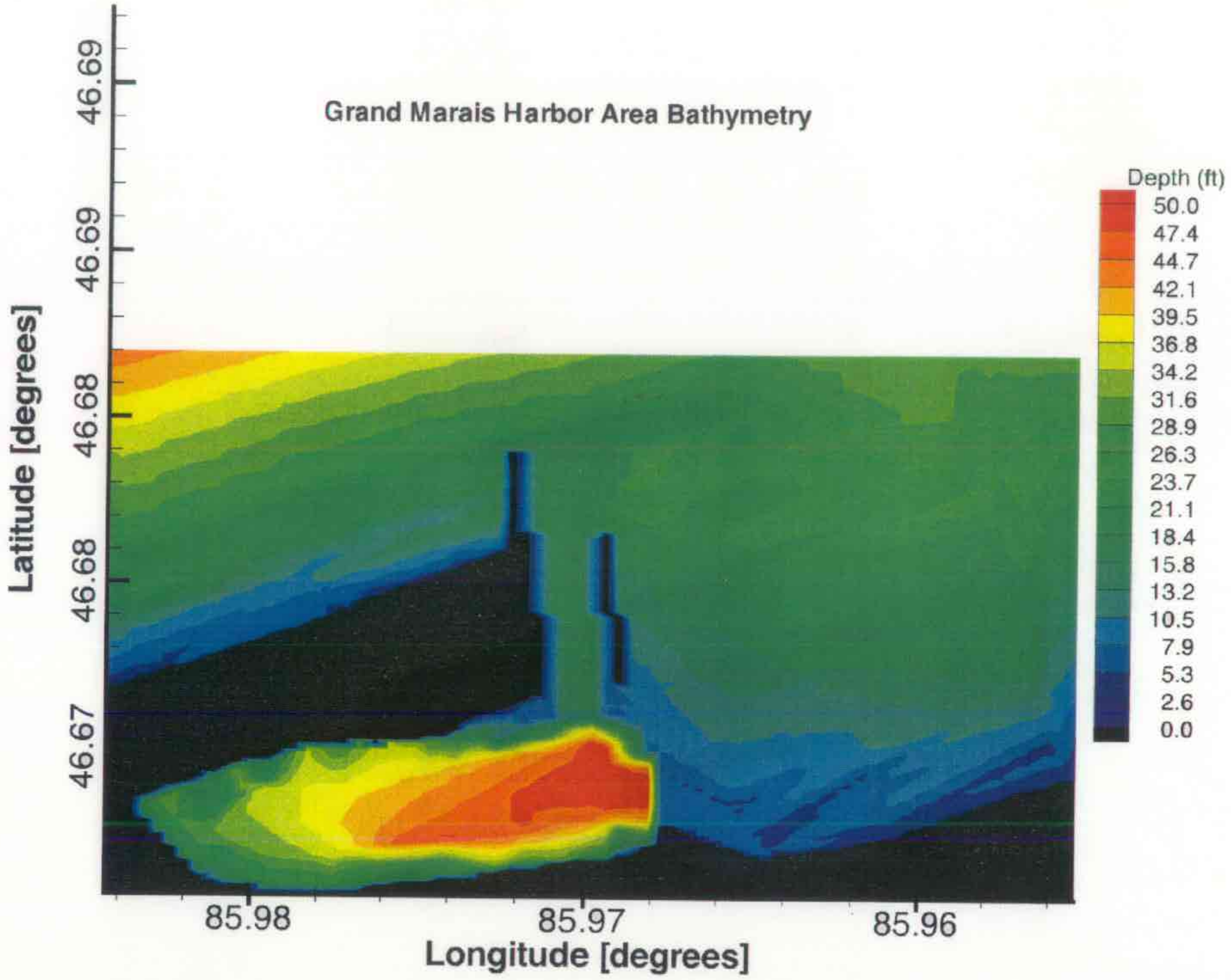


Figure 3.1 Contour diagram of Grand Marais Harbor area bathymetric grid used as input for numerical model of wave refraction and diffraction.

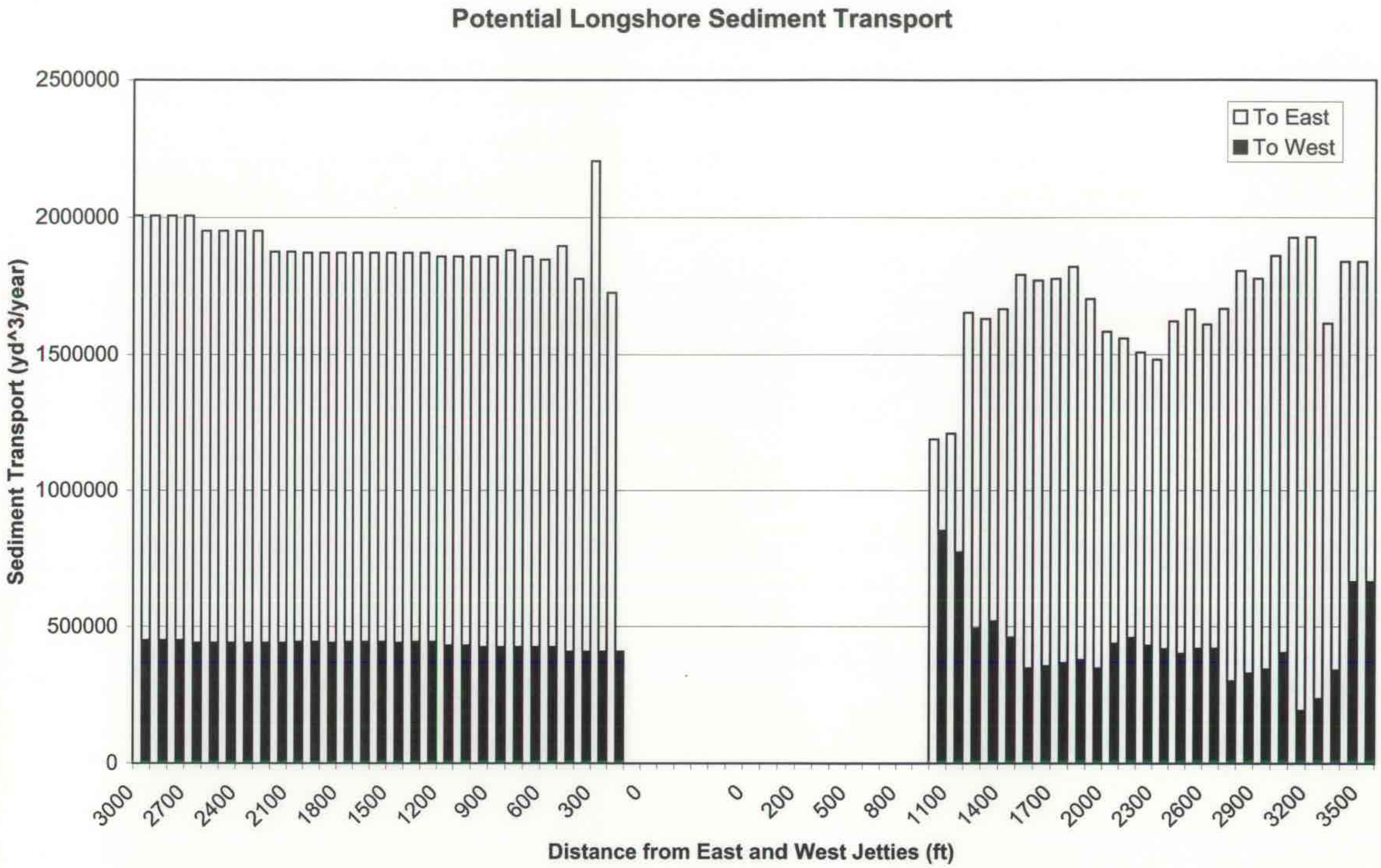


Figure 3.2 Annual potential longshore sediment transport at Grand Marais, Michigan.

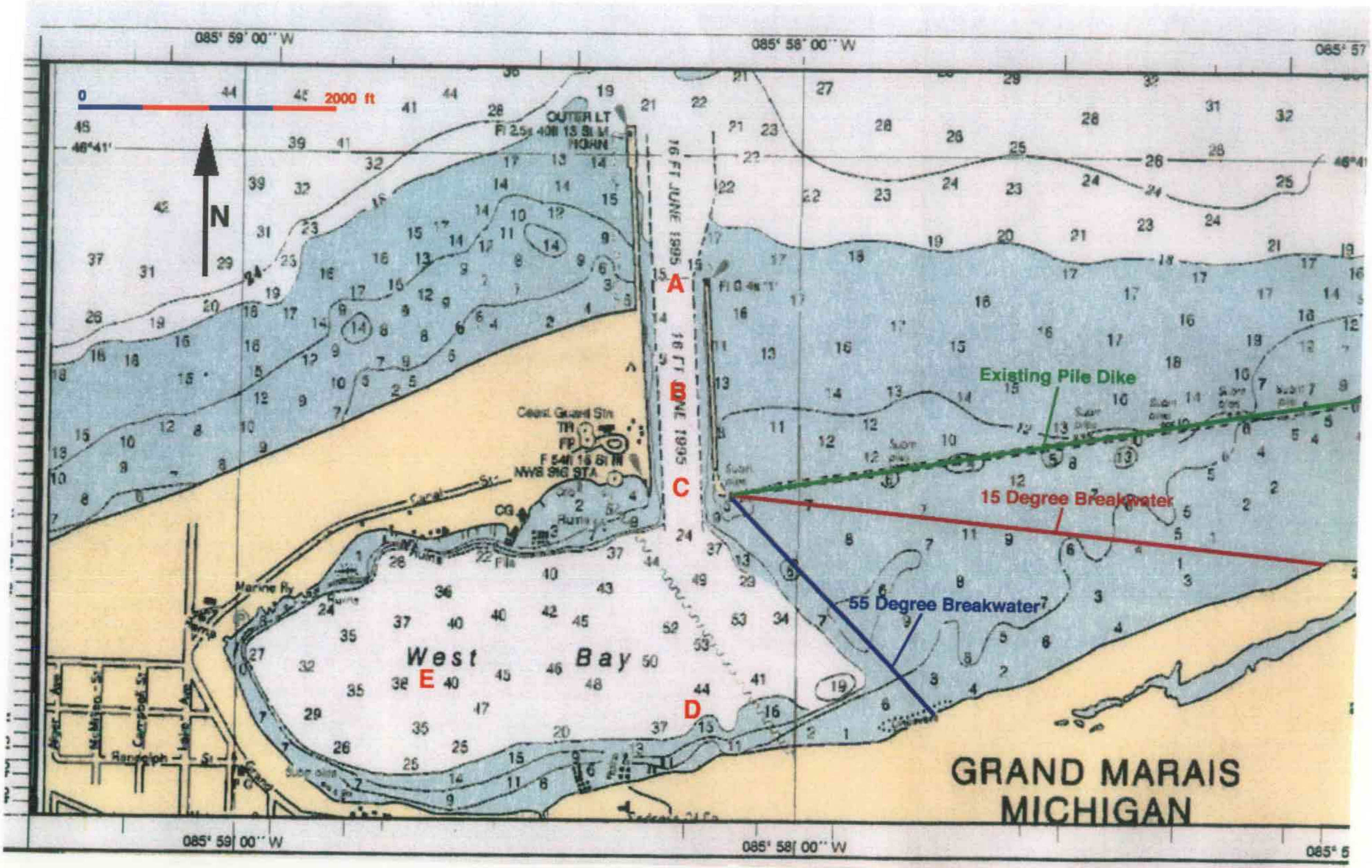
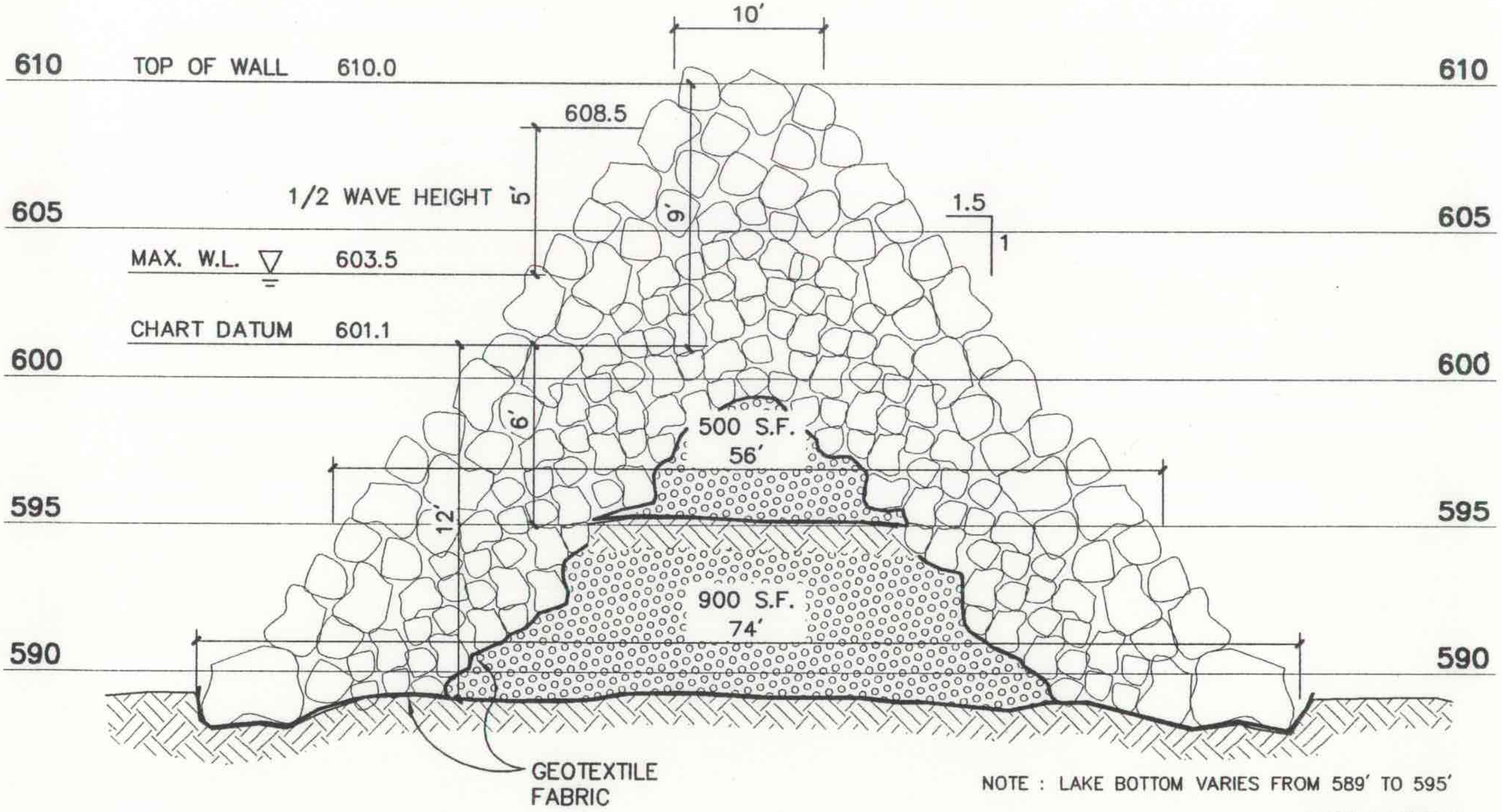


Figure 4.1 Map of Grand Marais Harbor showing three alternative breakwater locations: atop the existing pile dike, 15°, and 55° clockwise.

GRAND MARAIS
ALGER COUNTY, MICHIGAN

RECONSTRUCTION OF PILE DIKE

FIGURE 4.2



NOTE : LAKE BOTTOM VARIES FROM 589′ TO 595′

SCALE: HORIZ. 1″ = 10′
VERT. 1″ = 5′

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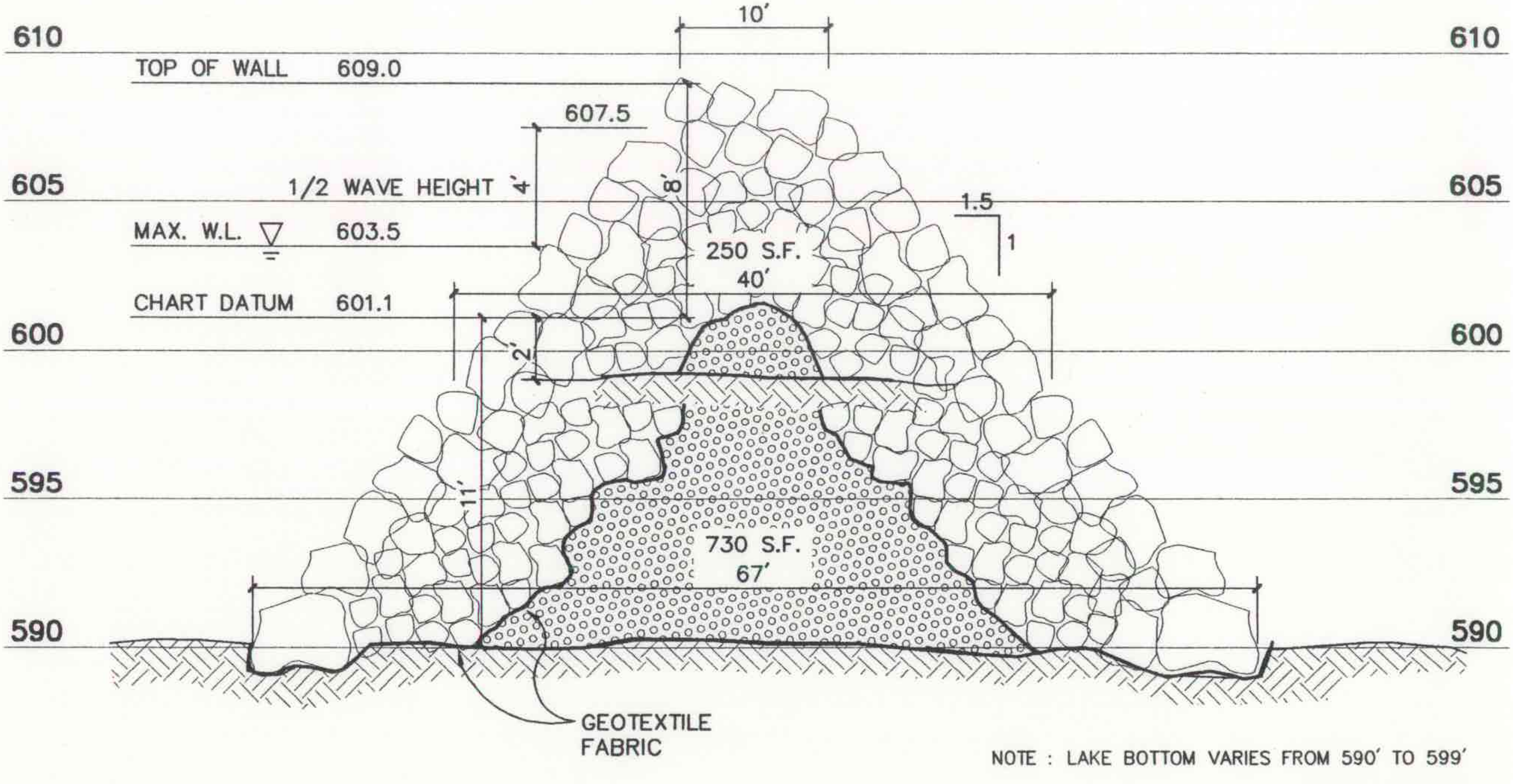
2000238

GRAND MARAIS
ALGER COUNTY, MICHIGAN

15° BREAKWATER

FIGURE 4.3



NOTE : LAKE BOTTOM VARIES FROM 590′ TO 599′

SCALE: HORIZ. 1″ = 10′
VERT. 1″ = 5′

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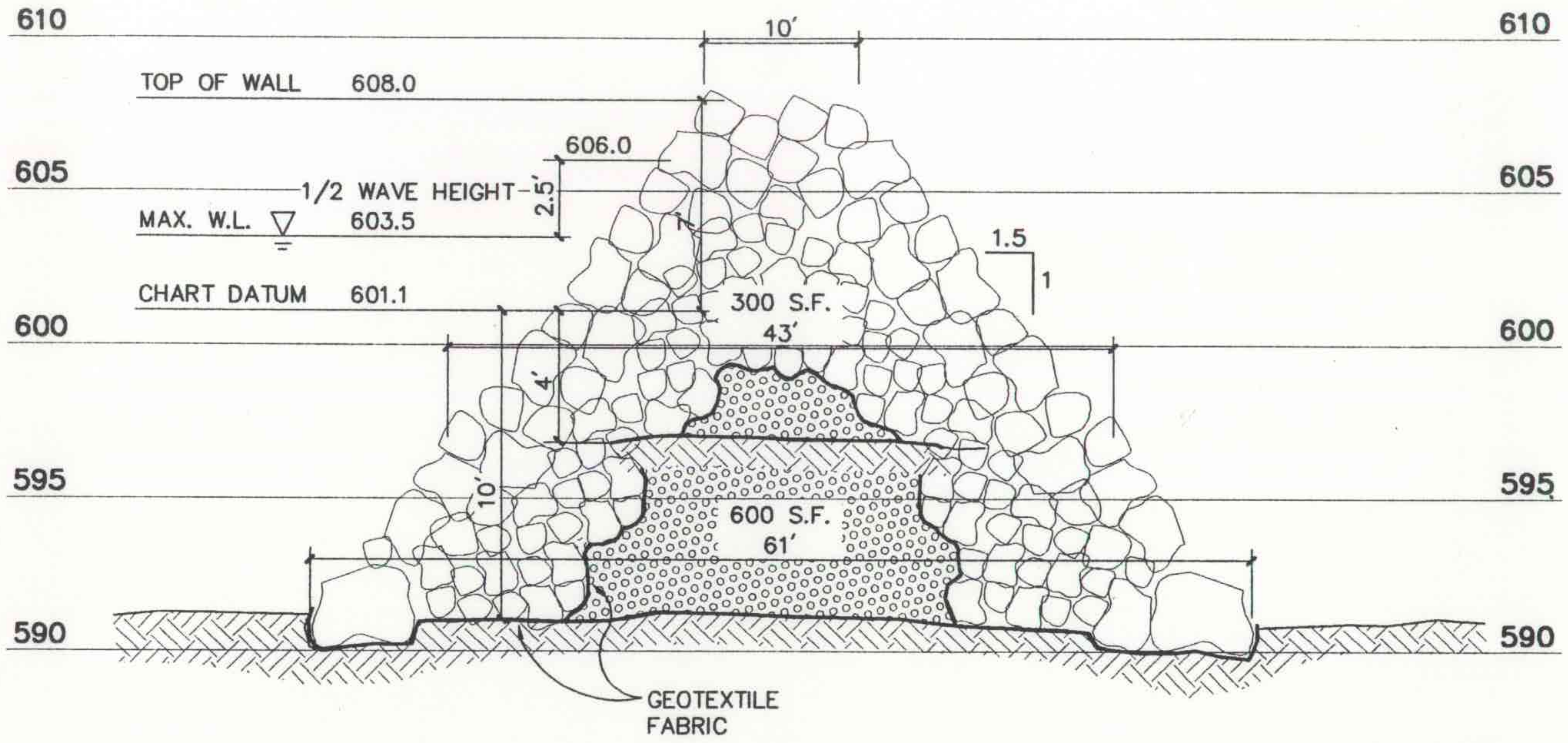
GRAND MARAIS
ALGER COUNTY, MICHIGAN

55° BREAKWATER

FIGURE 4.4



NOTE : LAKE BOTTOM VARIES FROM 591′ TO 597′

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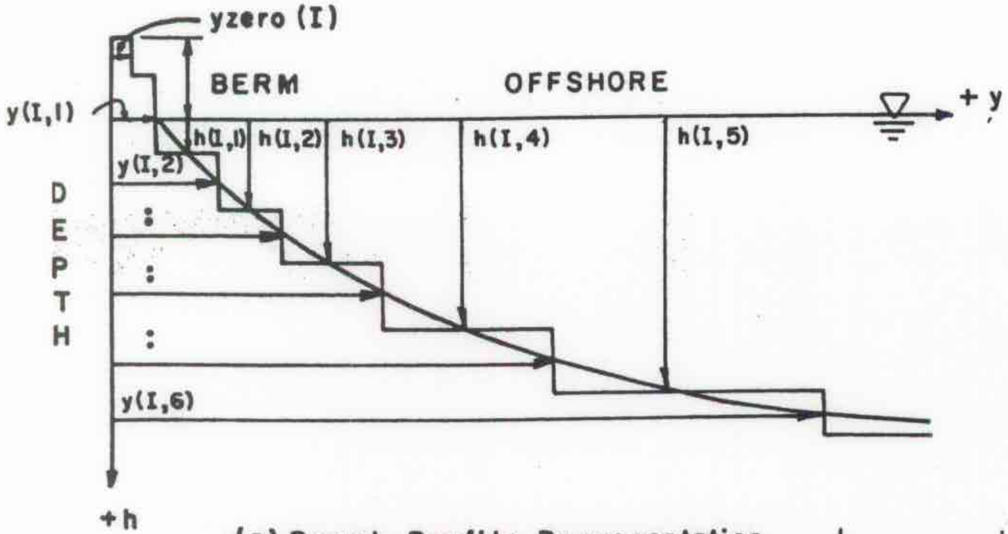
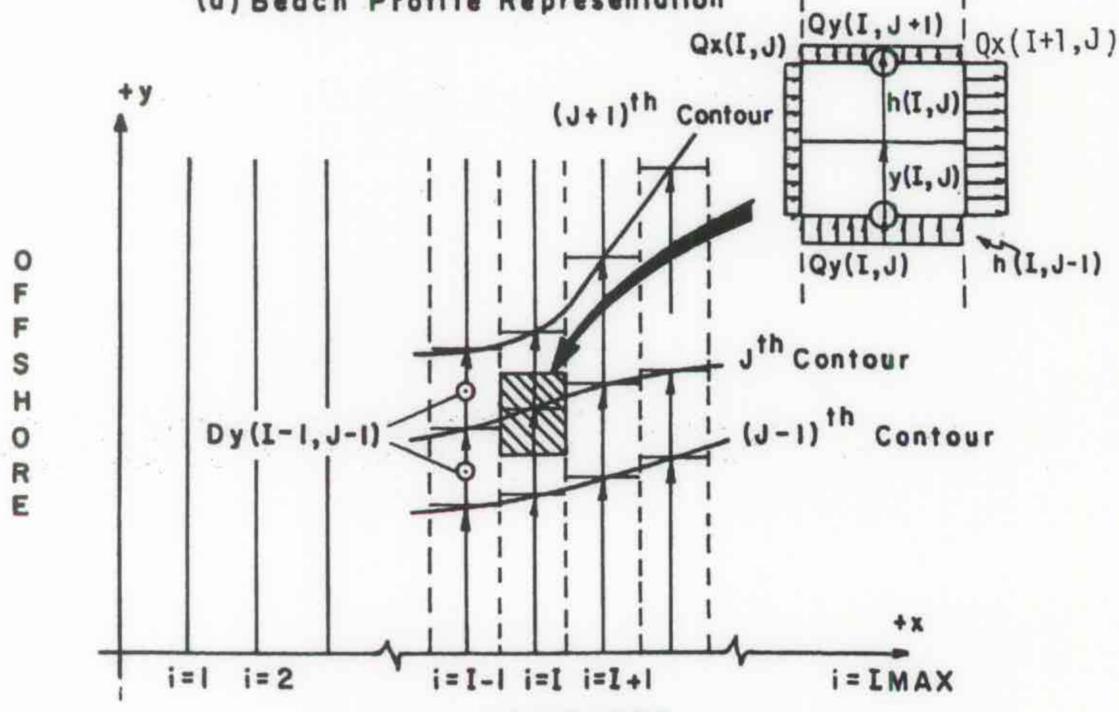


(a) Beach Profile Representation



(b) Beach Planform Representation

Figure 5.1 N-Line model numerical grid cell geometry

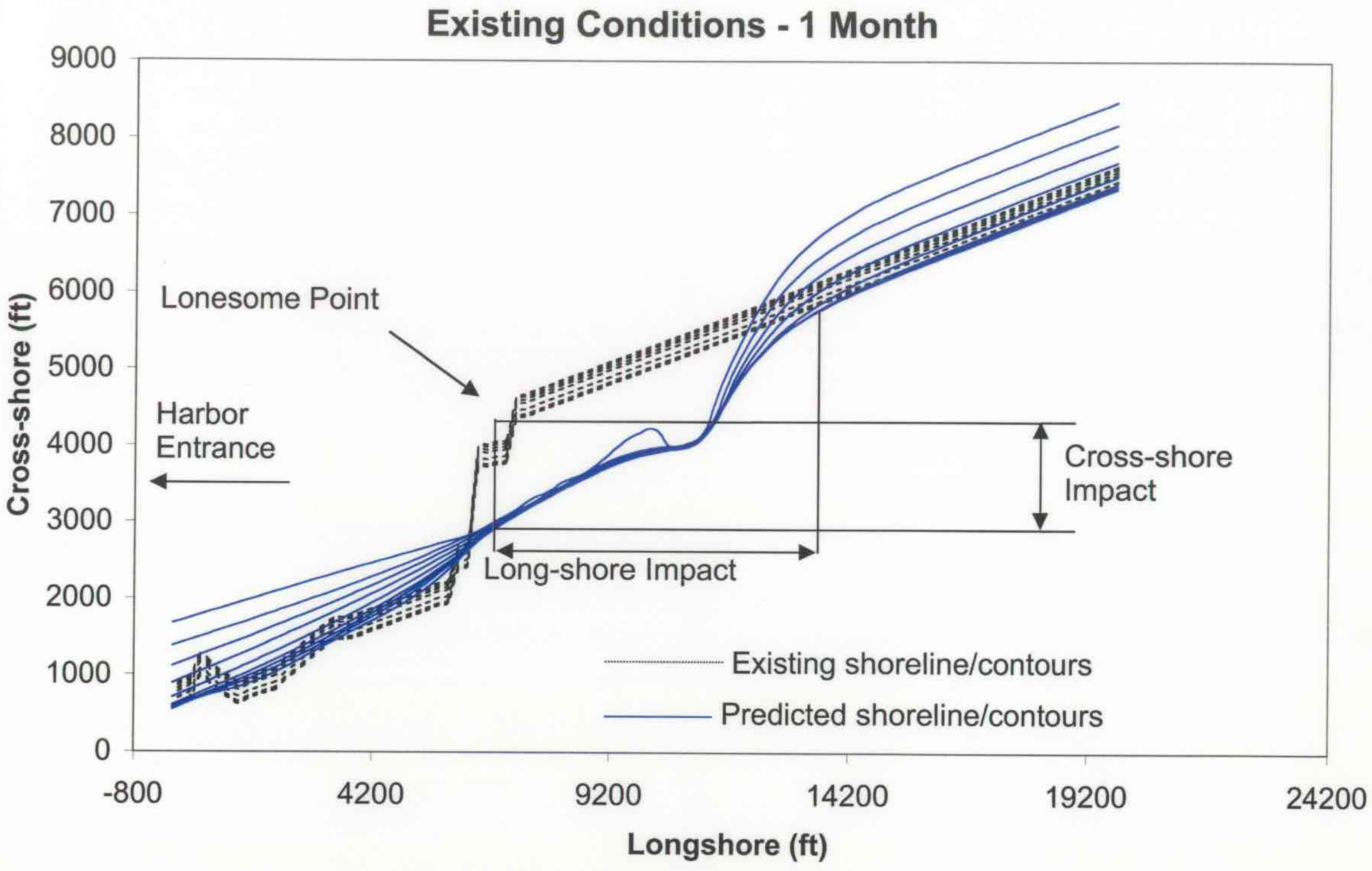


Figure 5.2. Sample N-Line output for Grand Marais showing shoreline evolution after one month for existing harbor conditions.

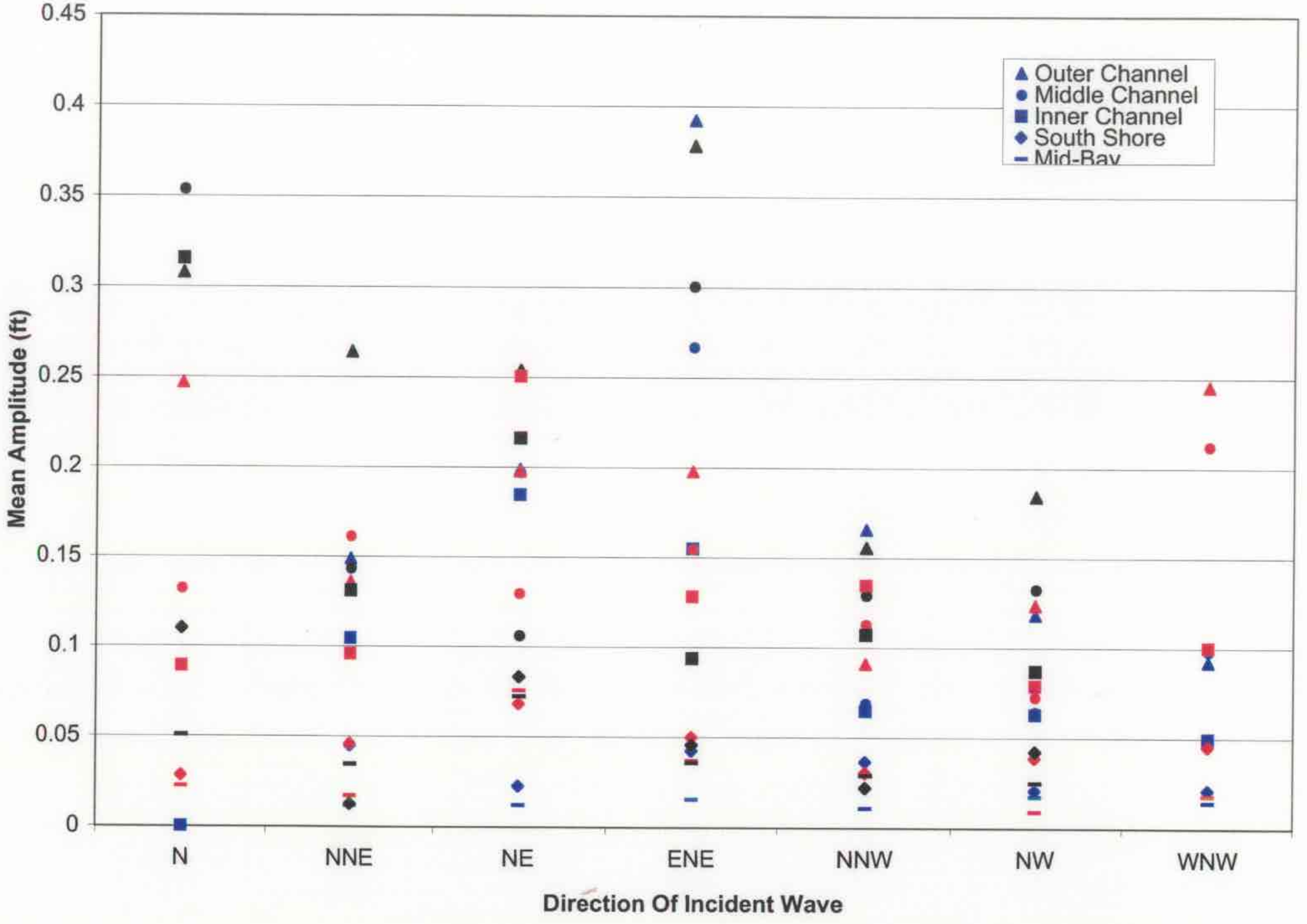
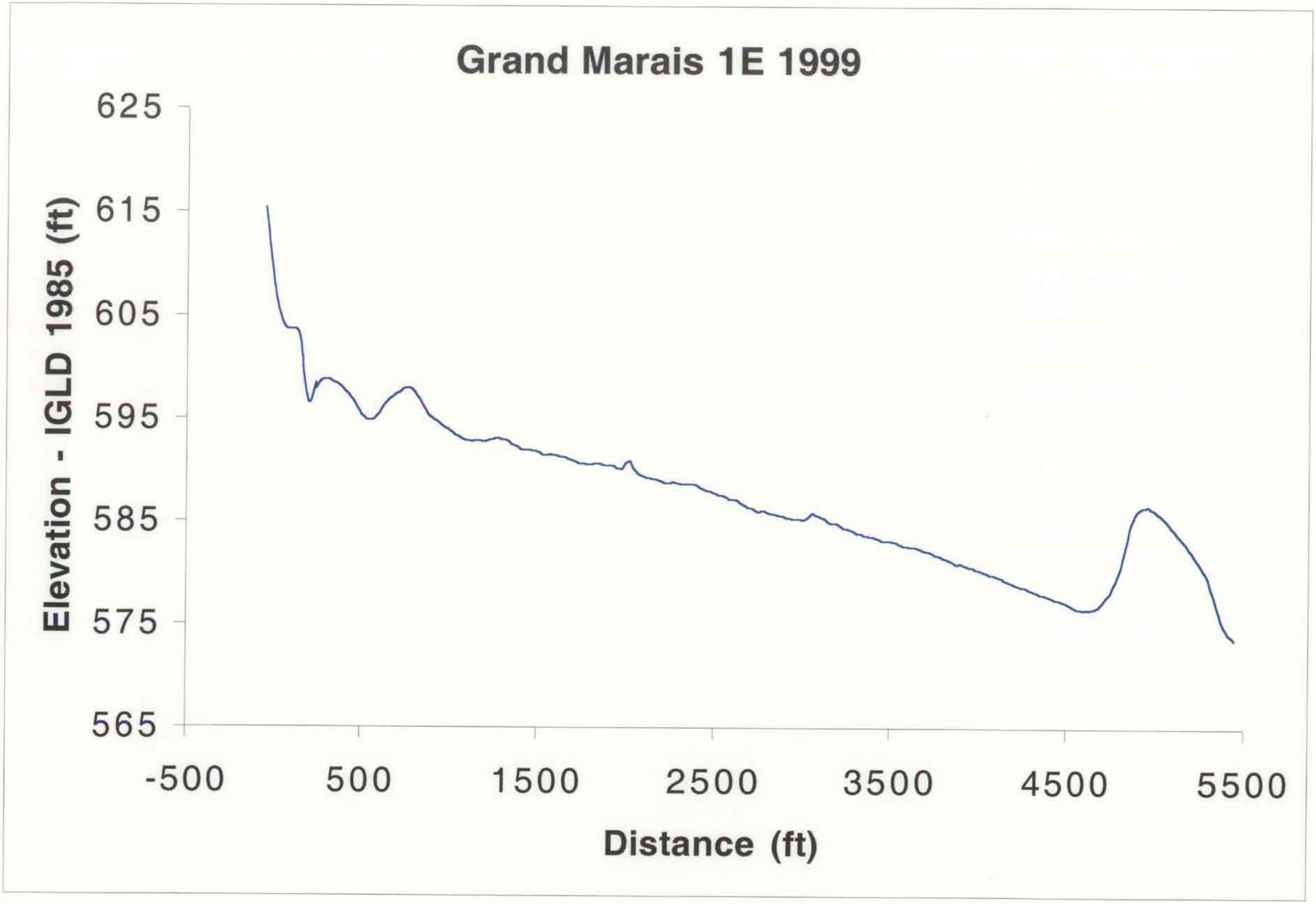
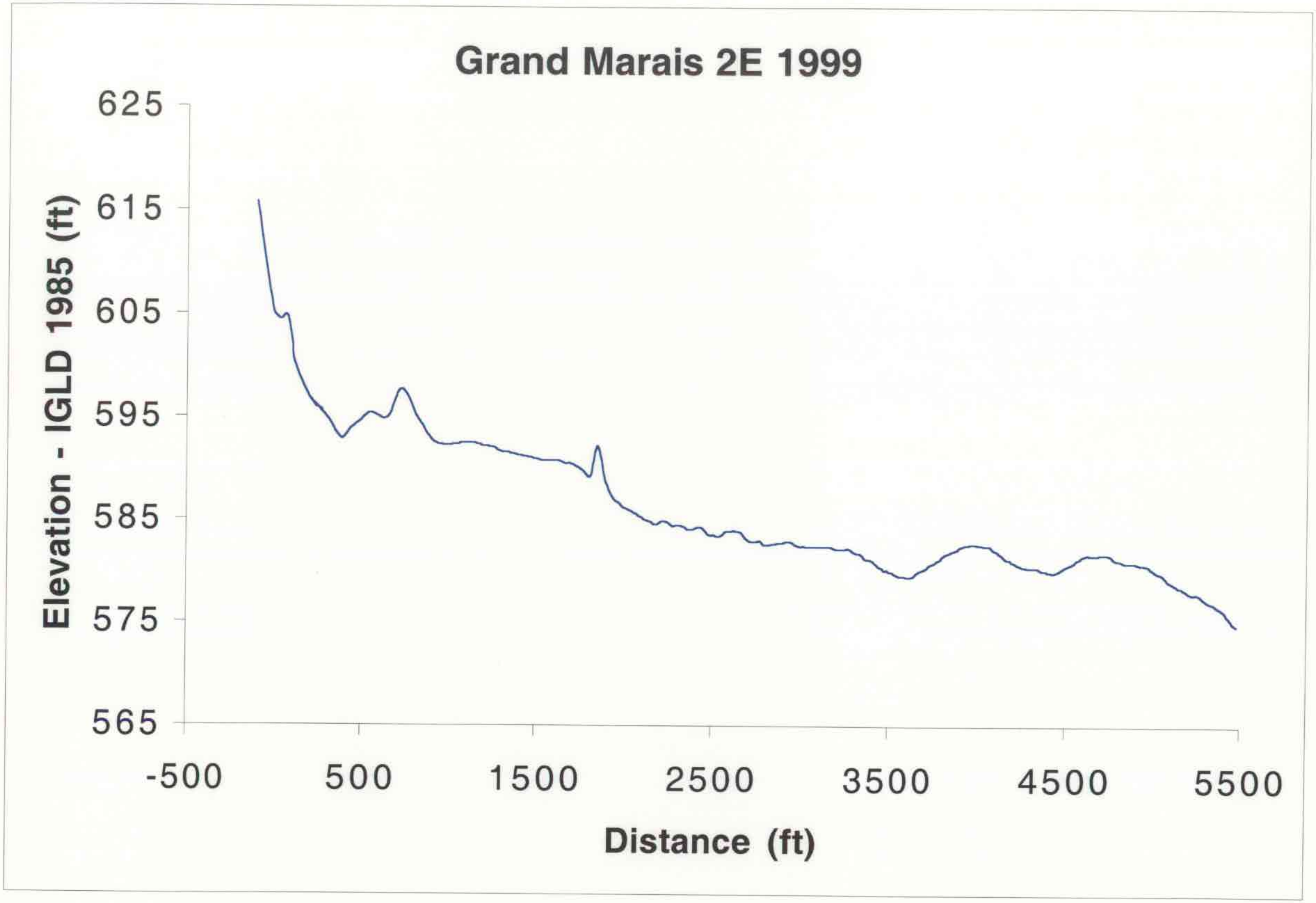


Figure 5.3. Summary of HARBD harbor resonance model results (blue: 0° breakwater, pink: 15° breakwater, black: 55° breakwater)

Appendix A:

Survey Data


Grand Marais 1E 1999
Elevation - IGLD 1985 (ft)
625
615
605
595
585
575
565
-500
500
1500
2500
3500
4500
5500
Distance (ft)


Grand Marais 2E 1999
Elevation - IGLD 1985 (ft)
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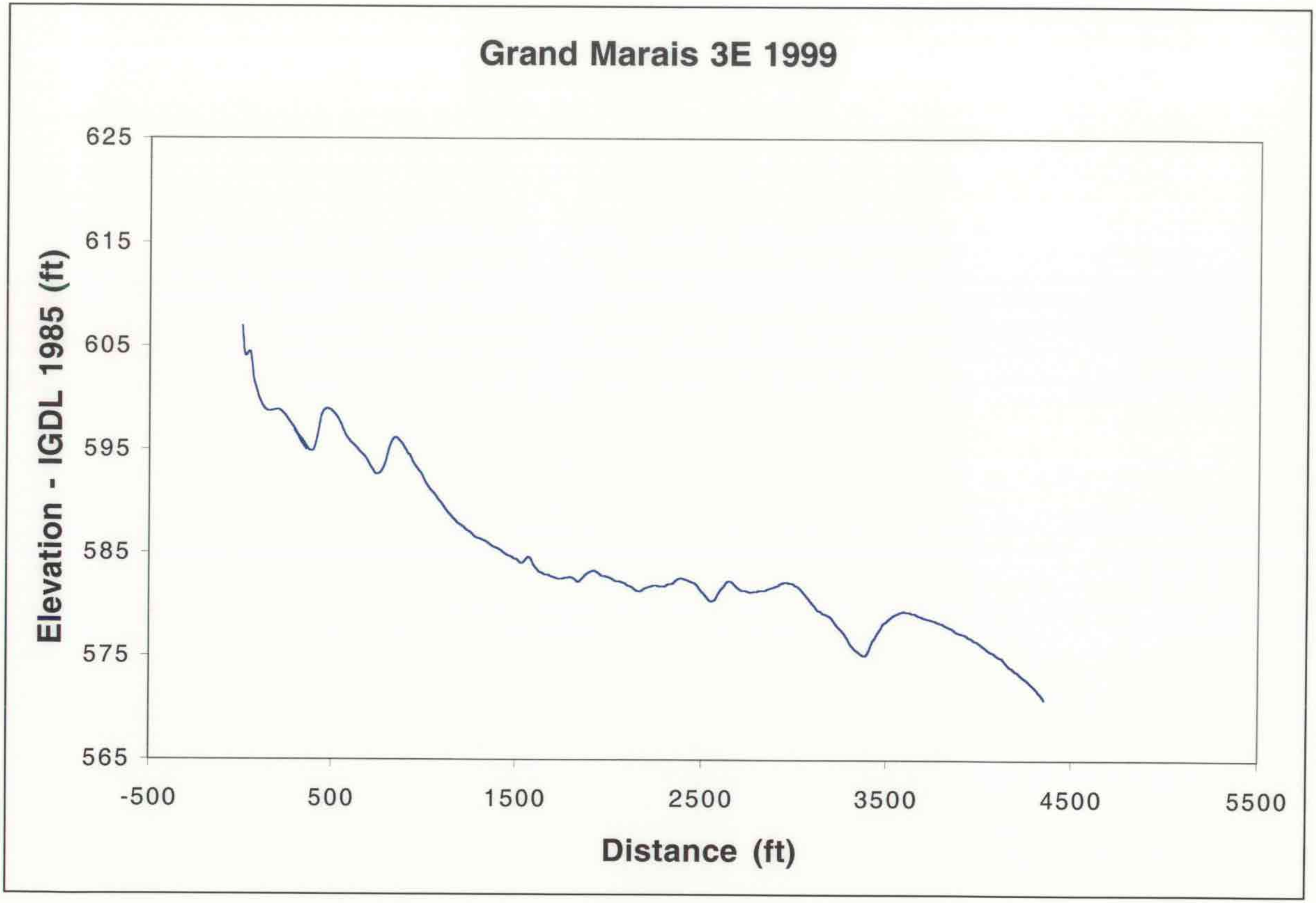
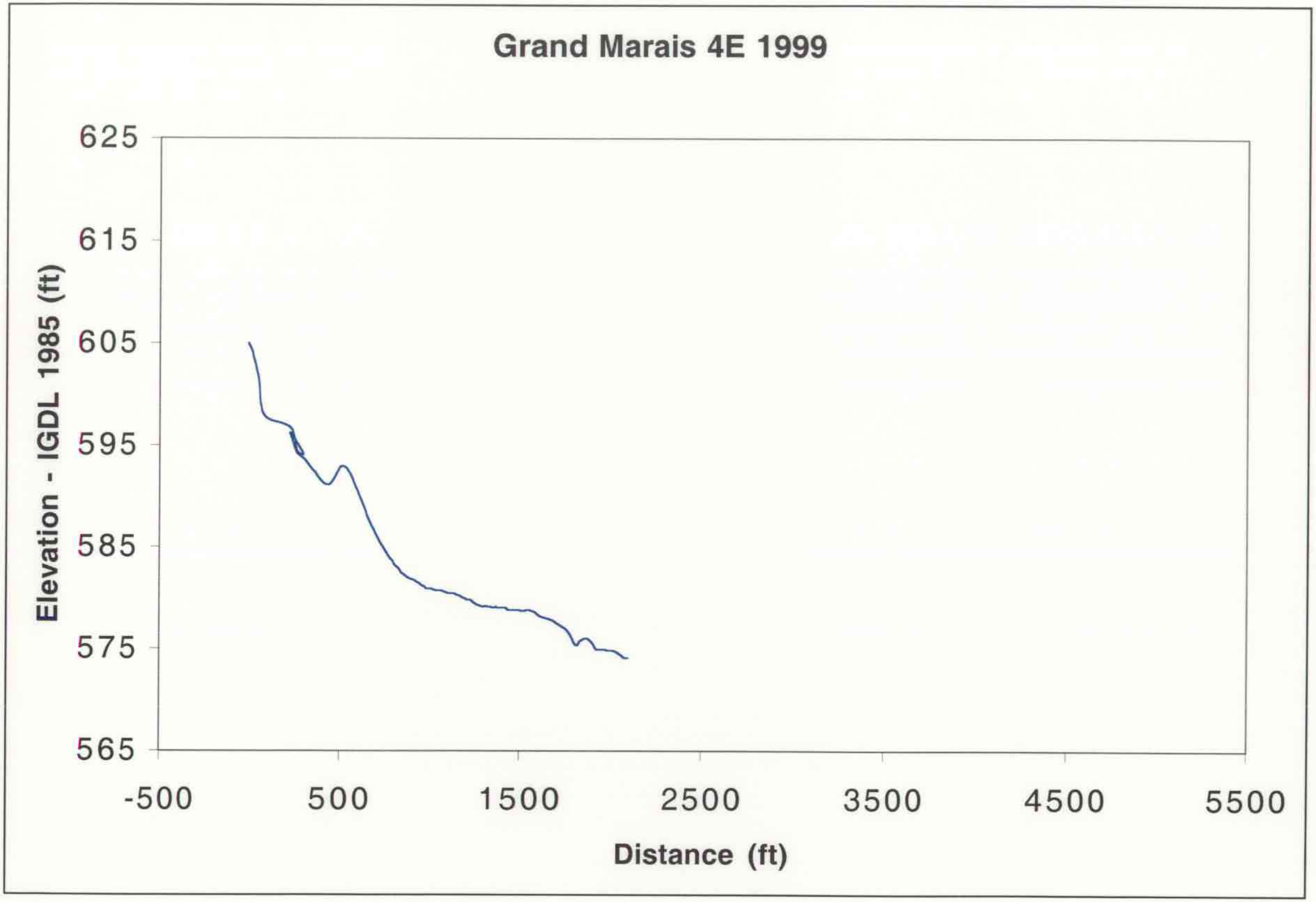
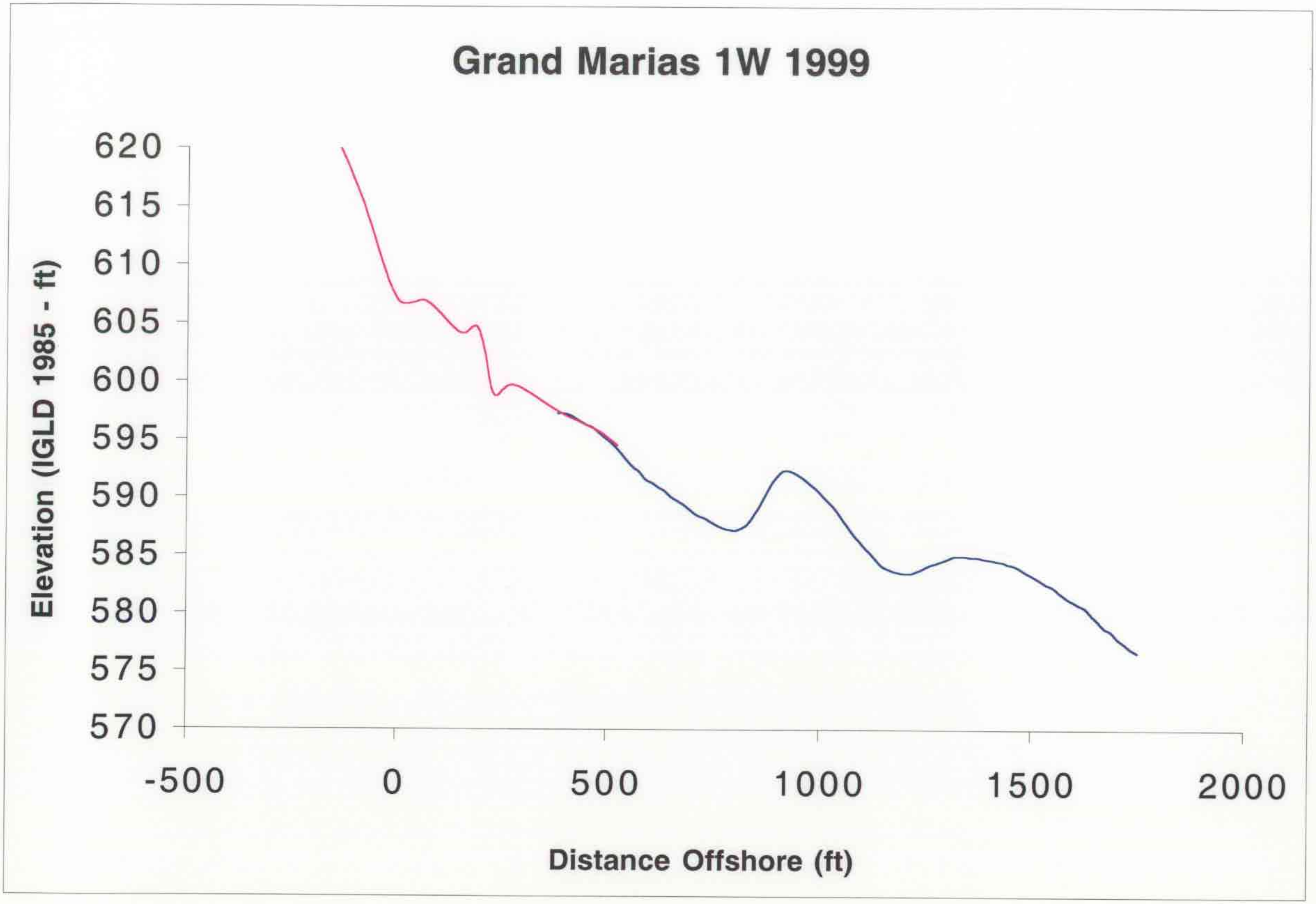
615
605
595
585
575
565
-500
500
1500
2500
3500
4500
5500
Distance (ft)

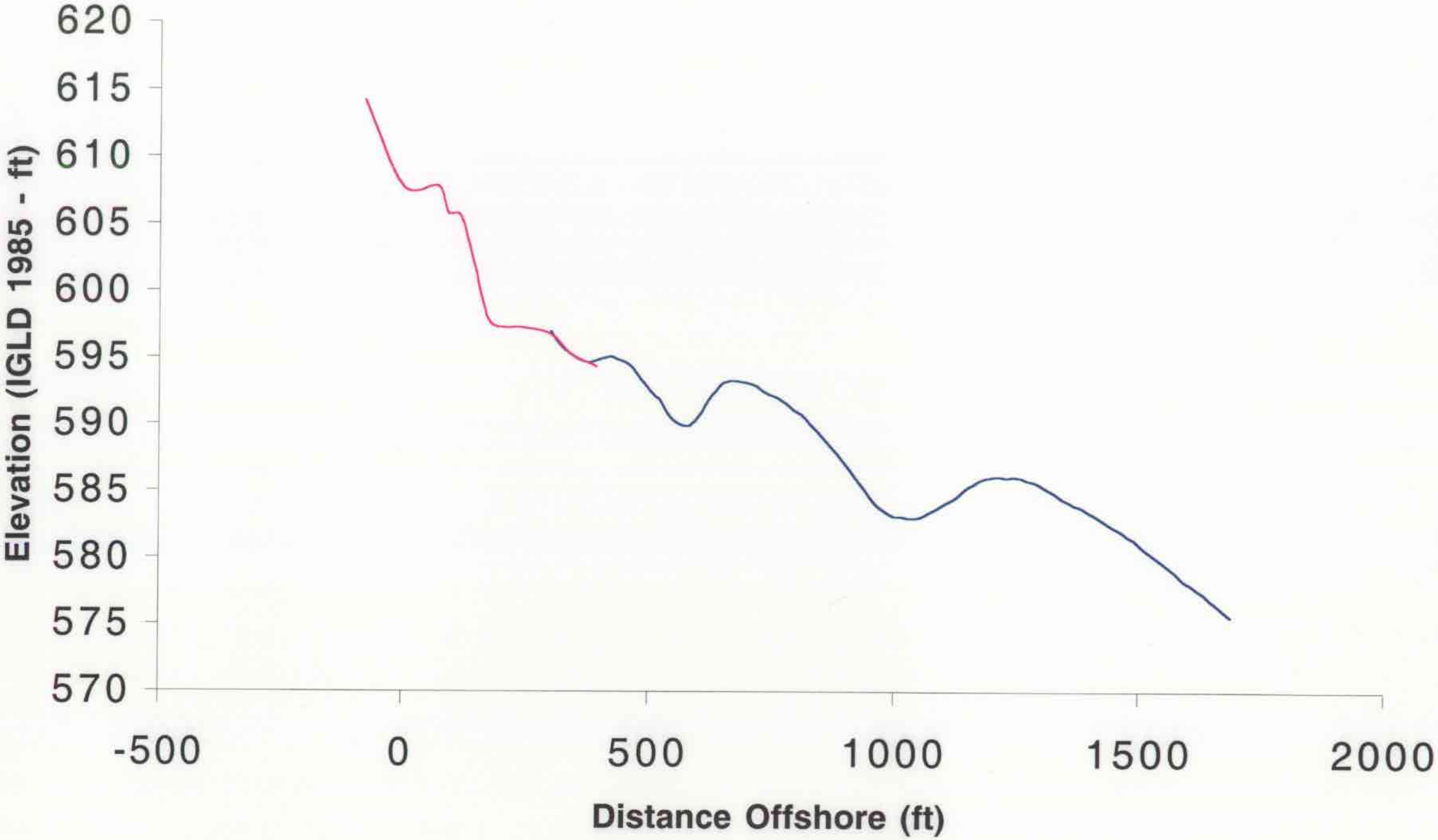


Grand Marais 3E 1999


Grand Marais 4E 1999
Elevation - IGDL 1985 (ft)
625
615
605
595
585
575
565
-500
500
1500
2500
3500
4500
5500
Distance (ft)


Grand Marias 1W 1999
Elevation (IGLD 1985 - ft)
620
615
610
605
600
595
590
585
580
575
570
-500
0
500
1000
1500
2000
Distance Offshore (ft)

Grand Marais 2W 1999



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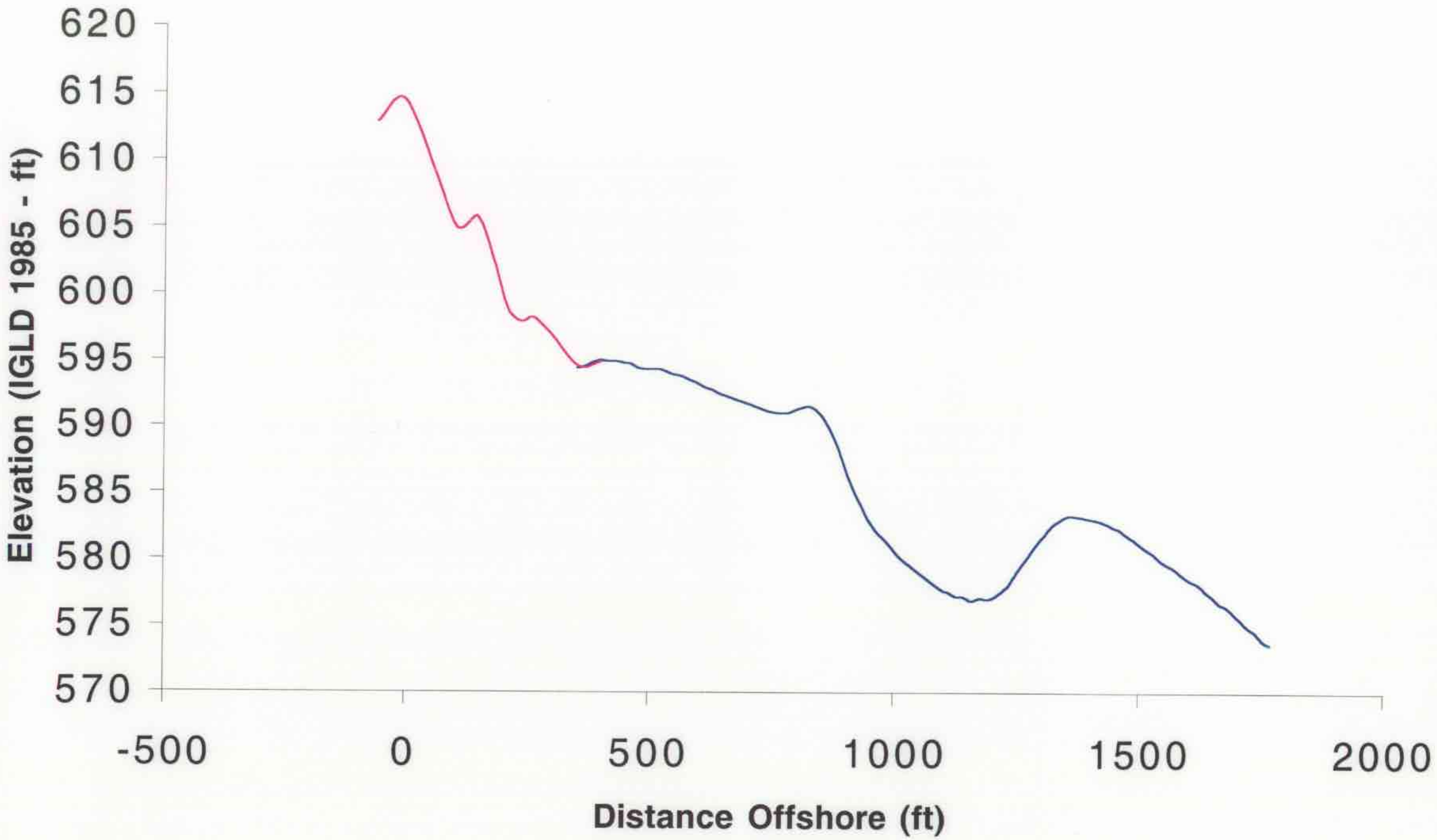
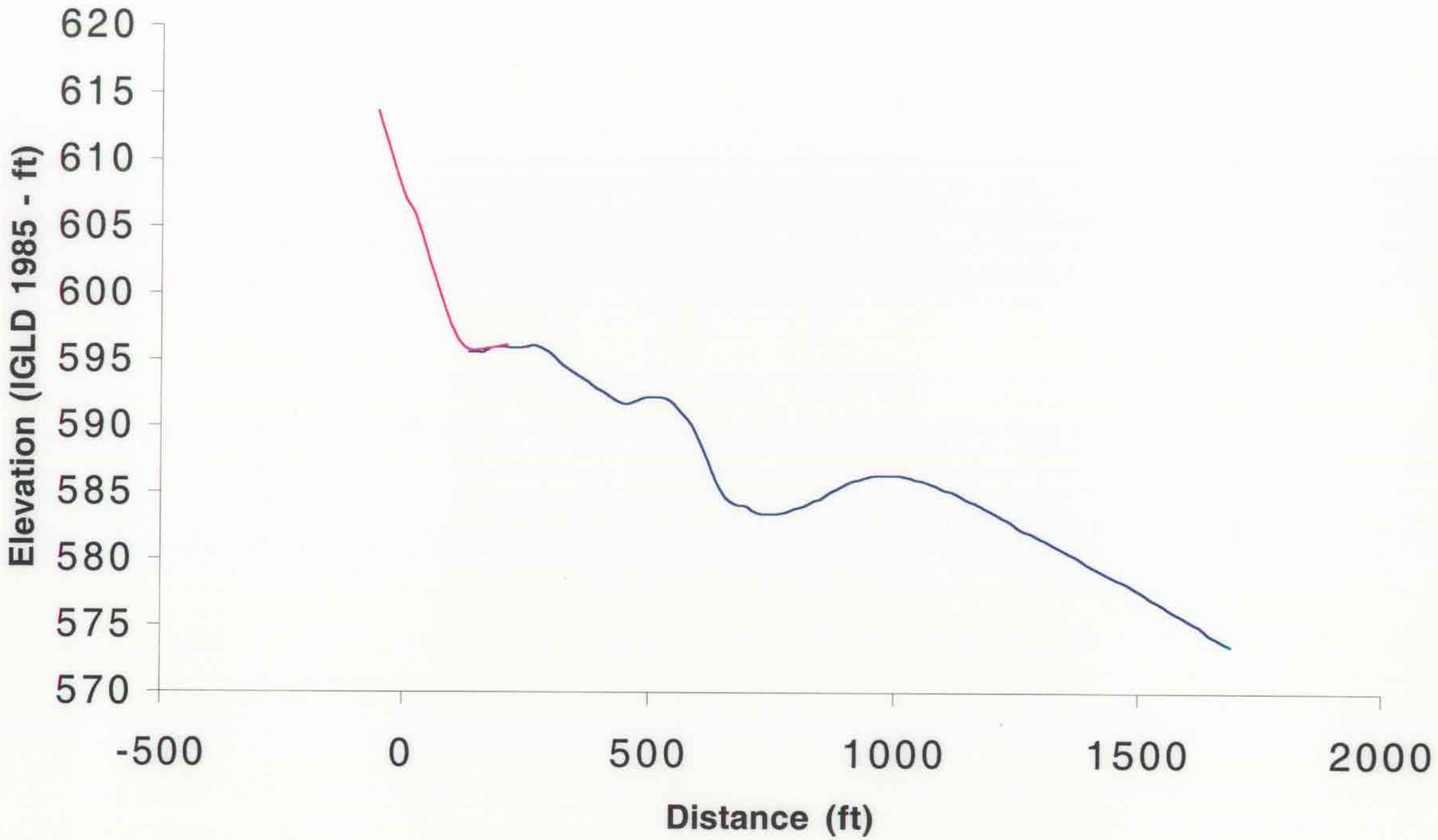
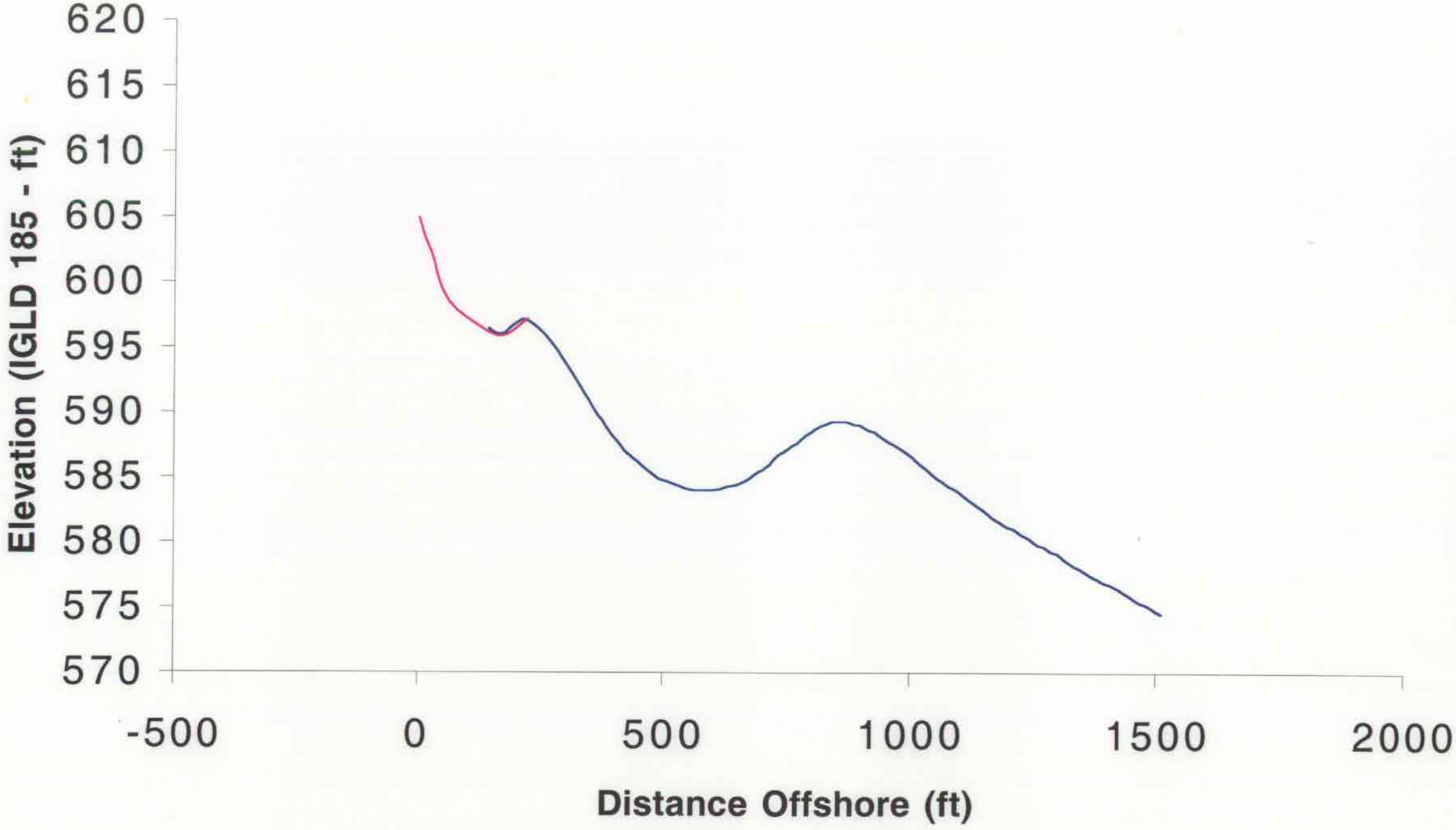
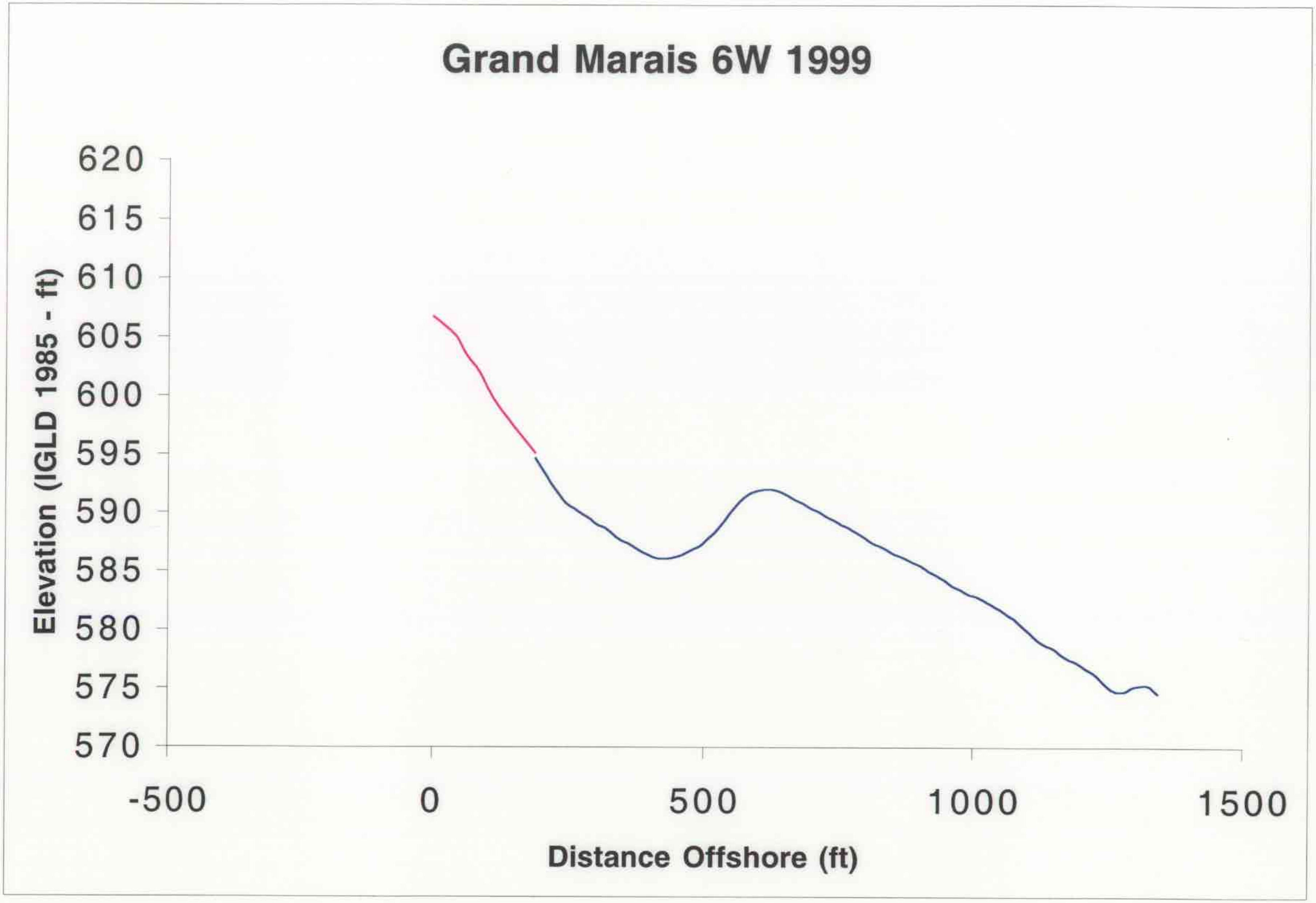


Grand Marais 4W 1999



Grand Marias 5W 1999



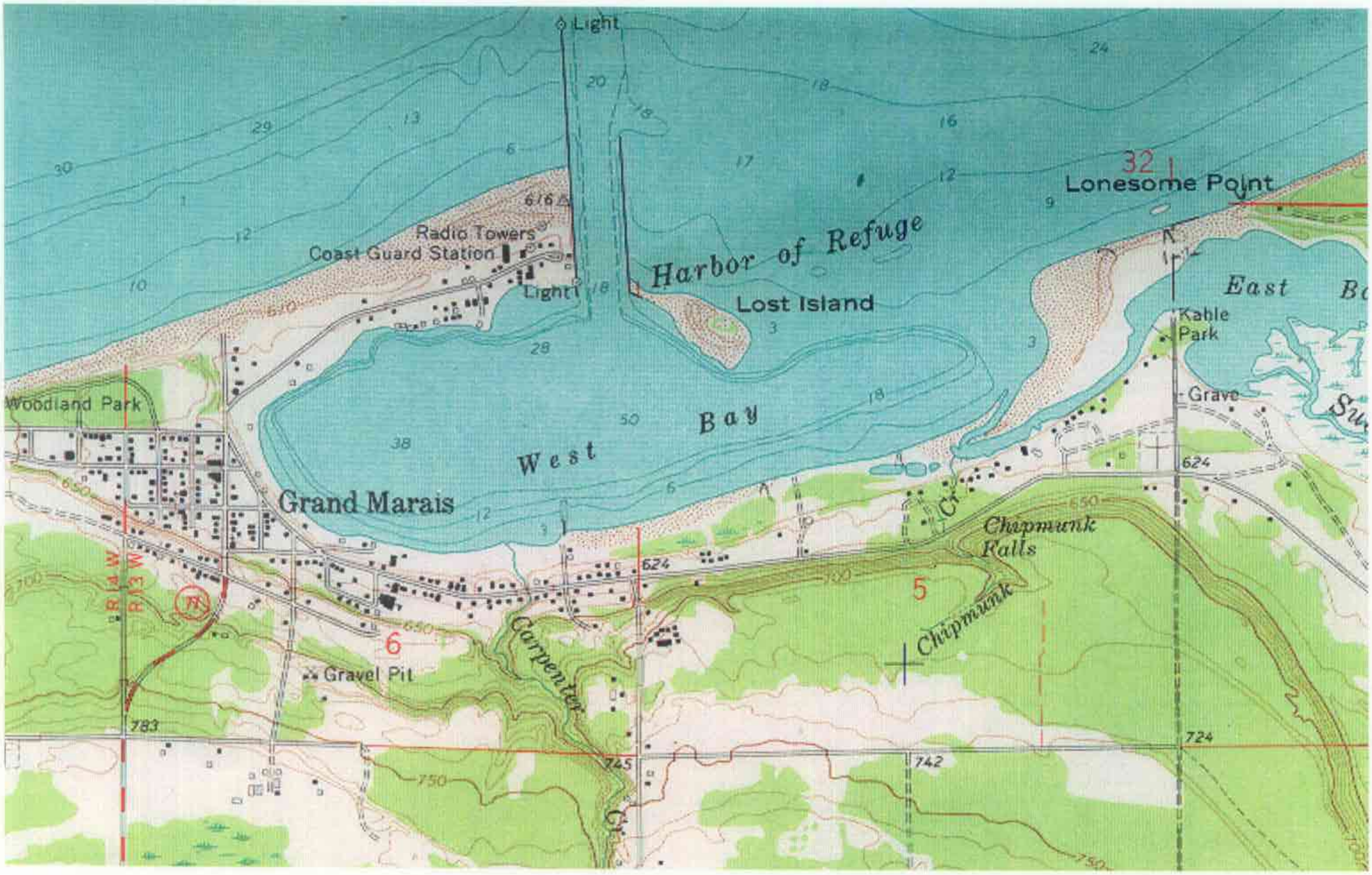


Grand Marais 6W 1999

Appendix B:

Aerial Photography

1968


Light
Harbor of Refuge
Lost Island
Lonesome Point
Radio Towers
Coast Guard Station
Light
East Ba
Kahle Park
Grave
Woodland Park
West Bay
Grand Marais
Chipmunk Falls
Chipmunk
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Gravel Pit
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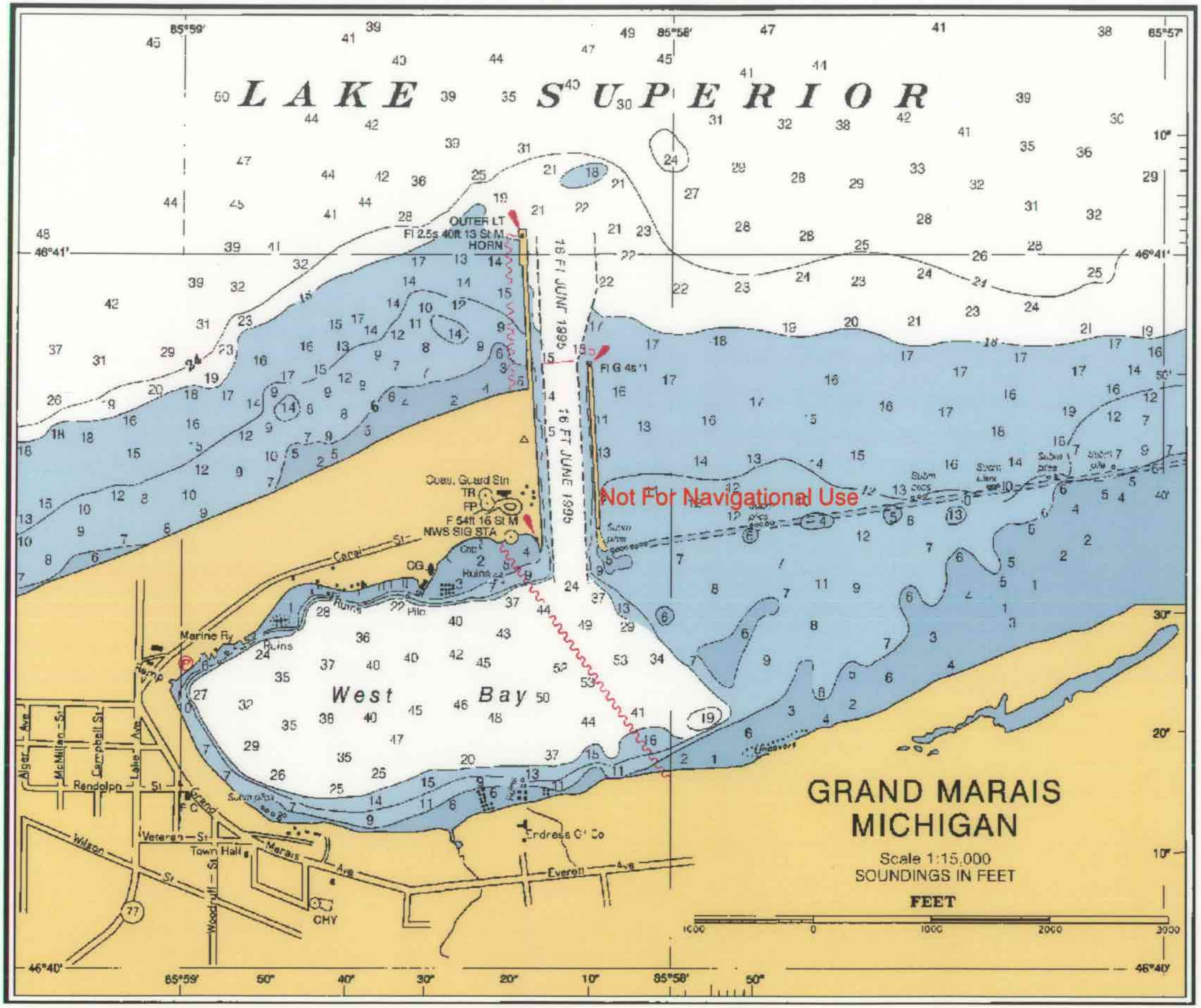
783
745
742
724
616

1997


LAKE SUPERIOR
GRAND MARAIS
MICHIGAN
Scale 1:15,000
SOUNDINGS IN FEET
FEET
1000
0
1000
2000
3000
Not For Navigational Use
West Bay
OUTER LT
Fl 2.5s 40ft 13 St M
HORN
Fl G 4s 1
16 FT JUNE 1995
Coast Guard Stn
TR
FP
F 54ft 16 St M
NWS SIG STA
CG
Canal
Ruins
Pile
Marine Ry
Ramp
Endress O' Co
Everett Ave
Town Hall
Marais Ave
Grand
Veteran St
Wilson St
Woodruff St
Randolph St
Alger Ave
McMillan St
Campbell St
Lake Ave
CHY
77
Subm piles
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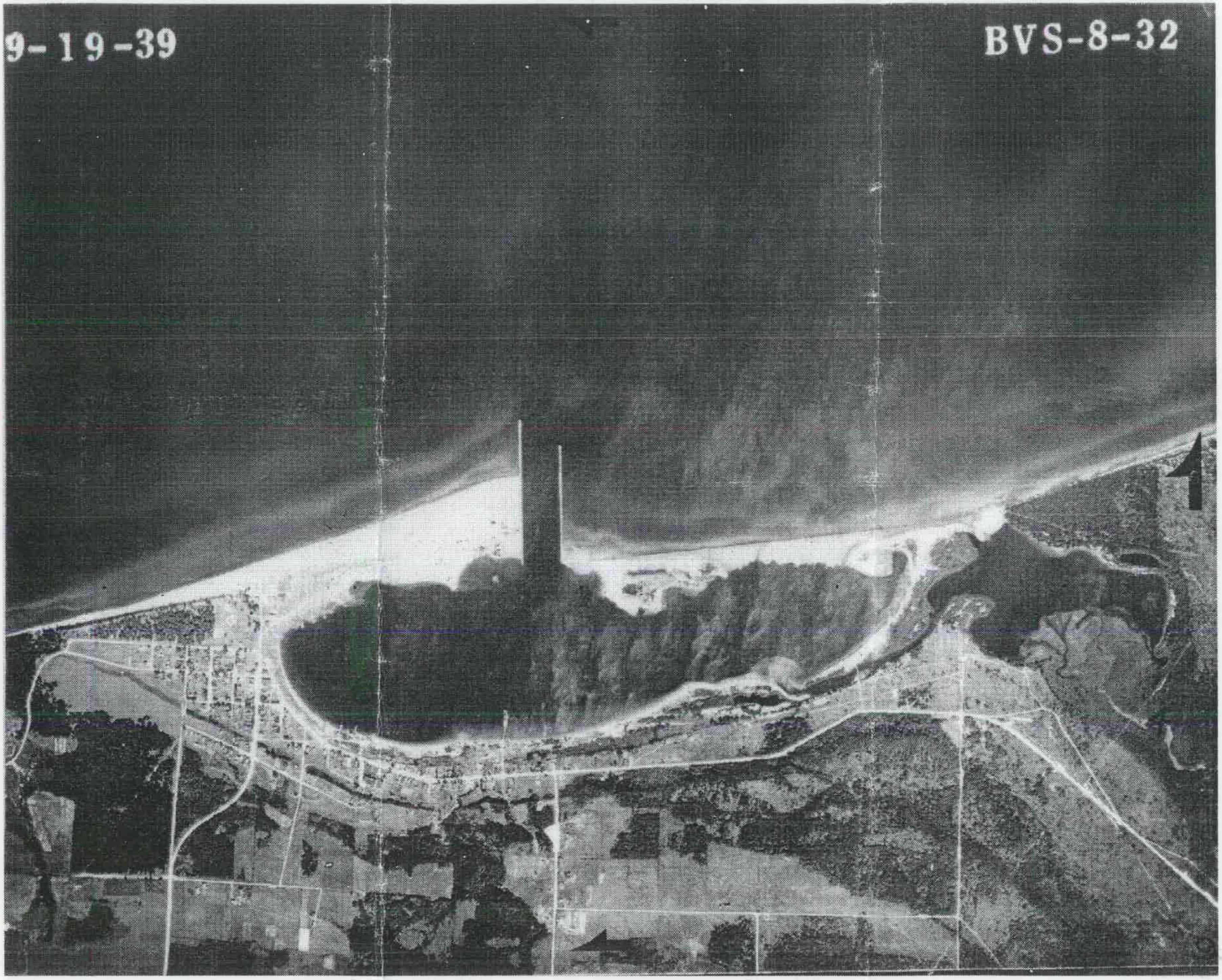
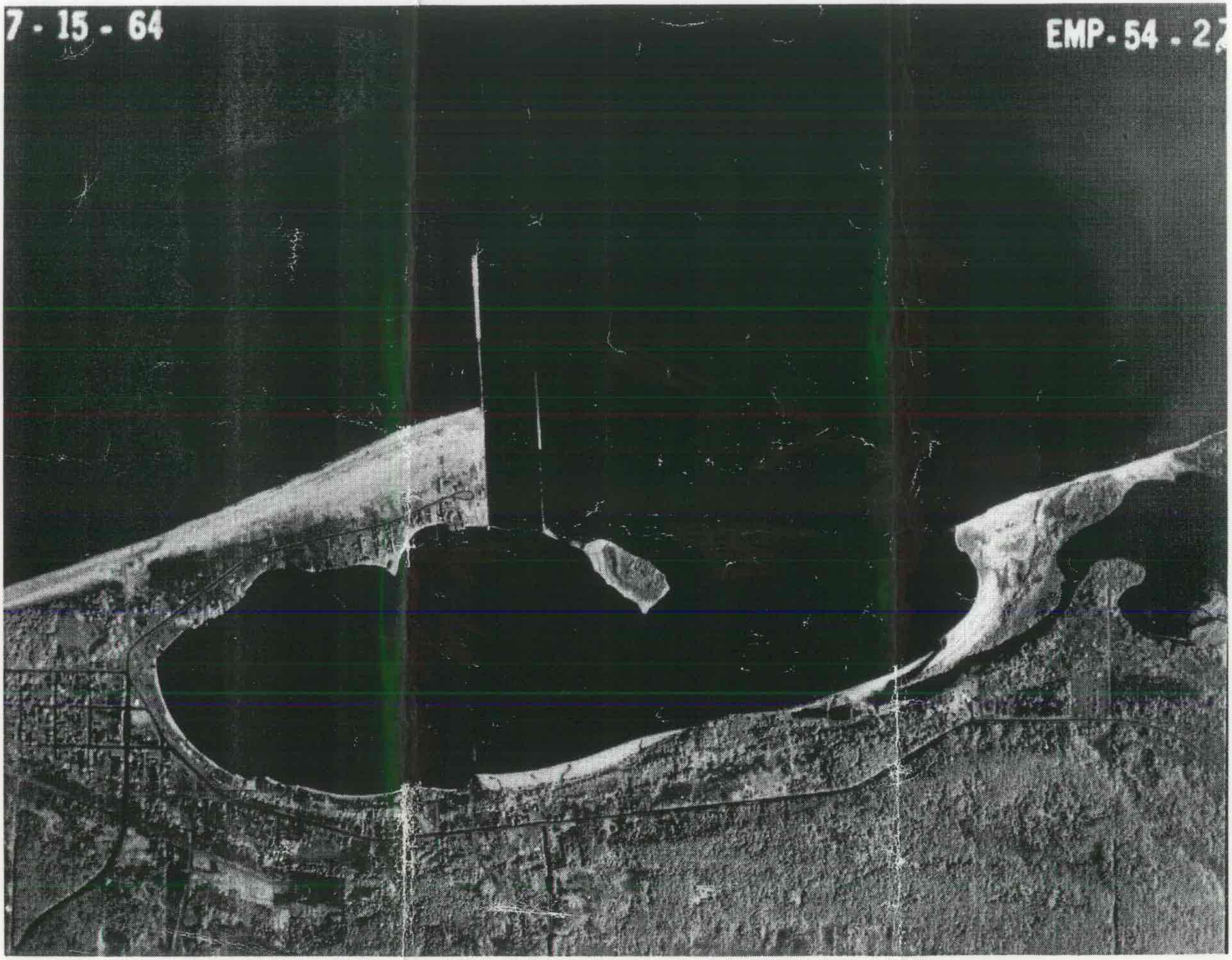
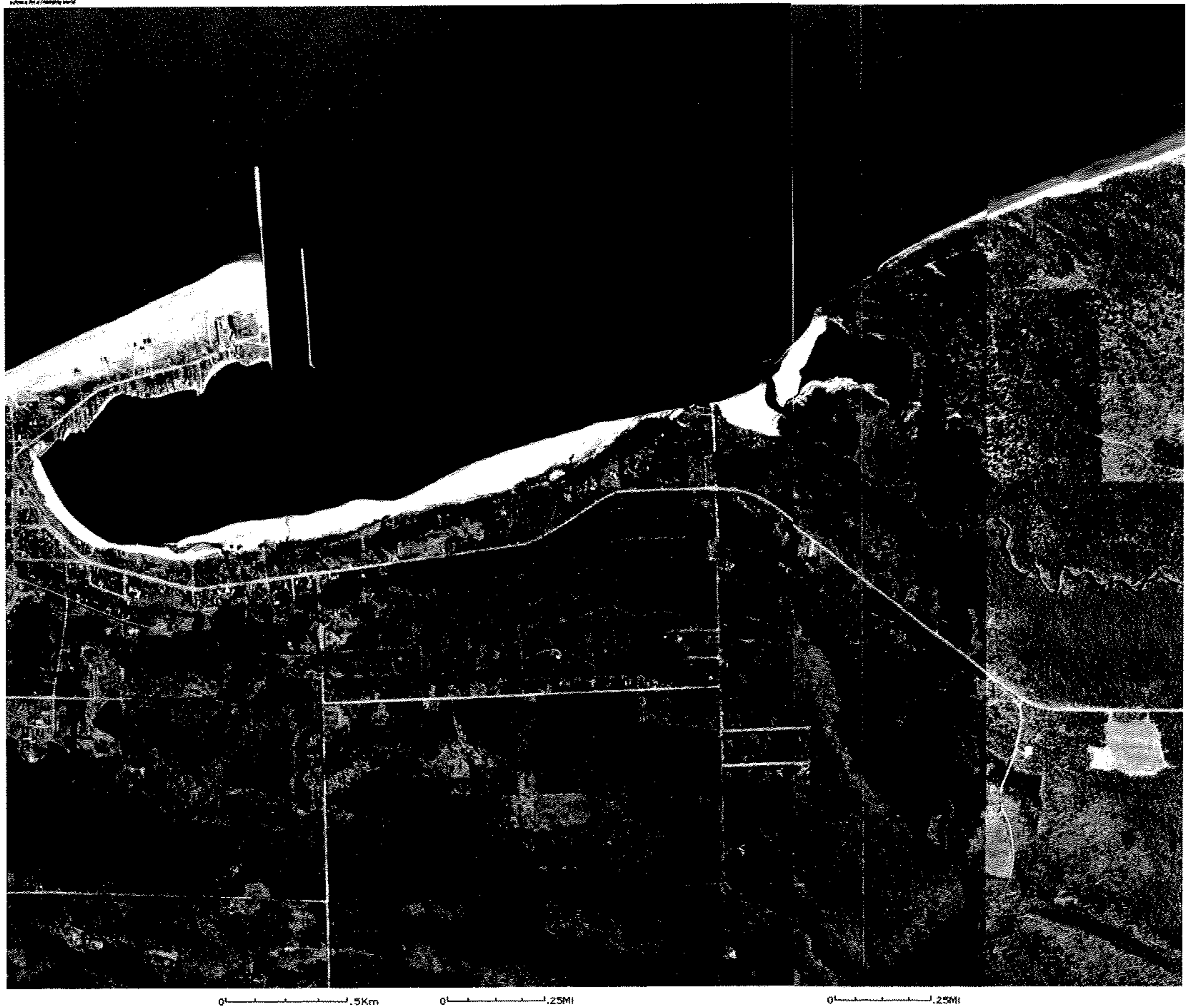
46°40'
50"
40"
30"
20"
10"


9-19-39
BVS-8-32

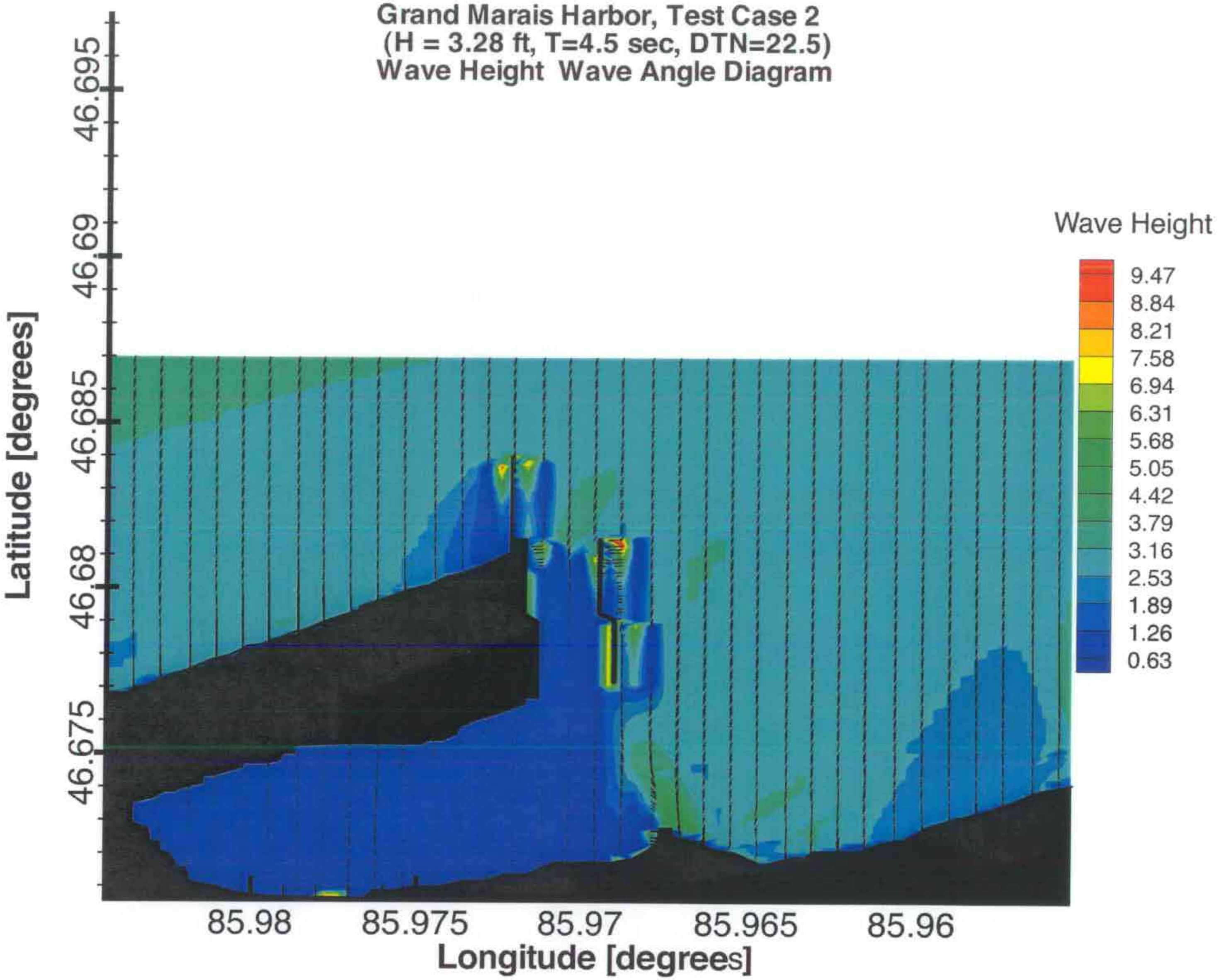

7 - 15 - 64
EMP- 54 . 2

1 km NE of Grand Marais, Michigan, United States 06 May 1993



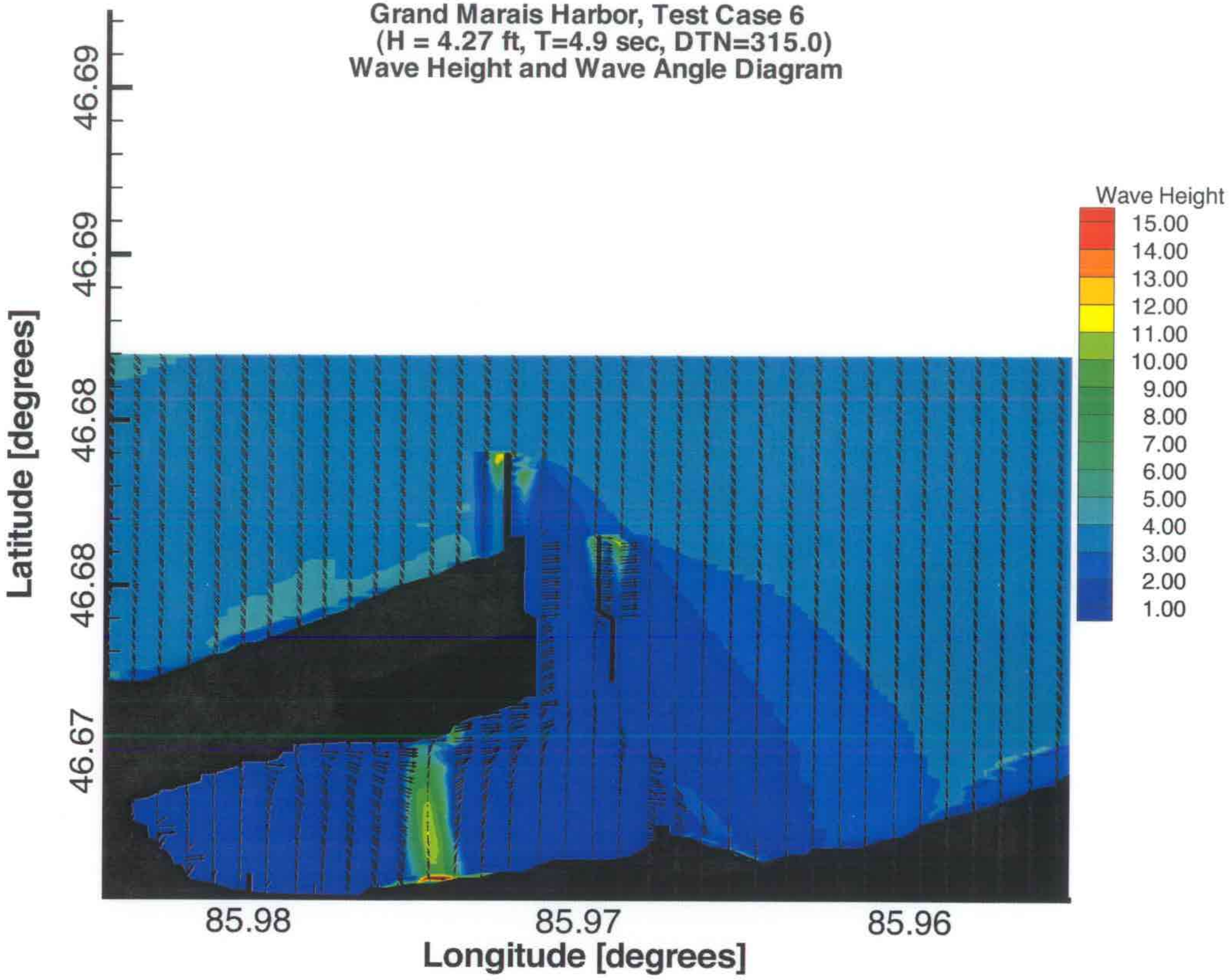
1975

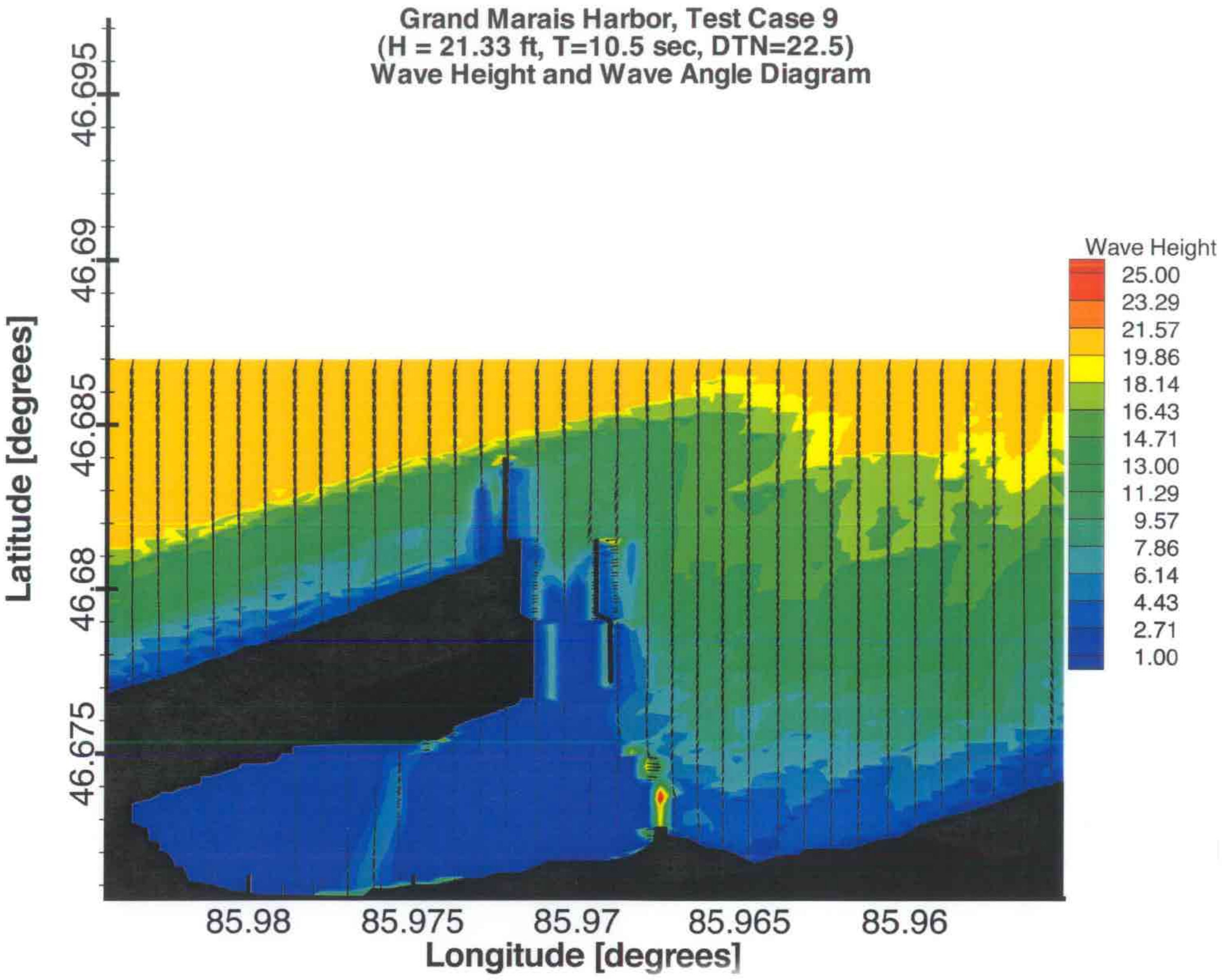
1975



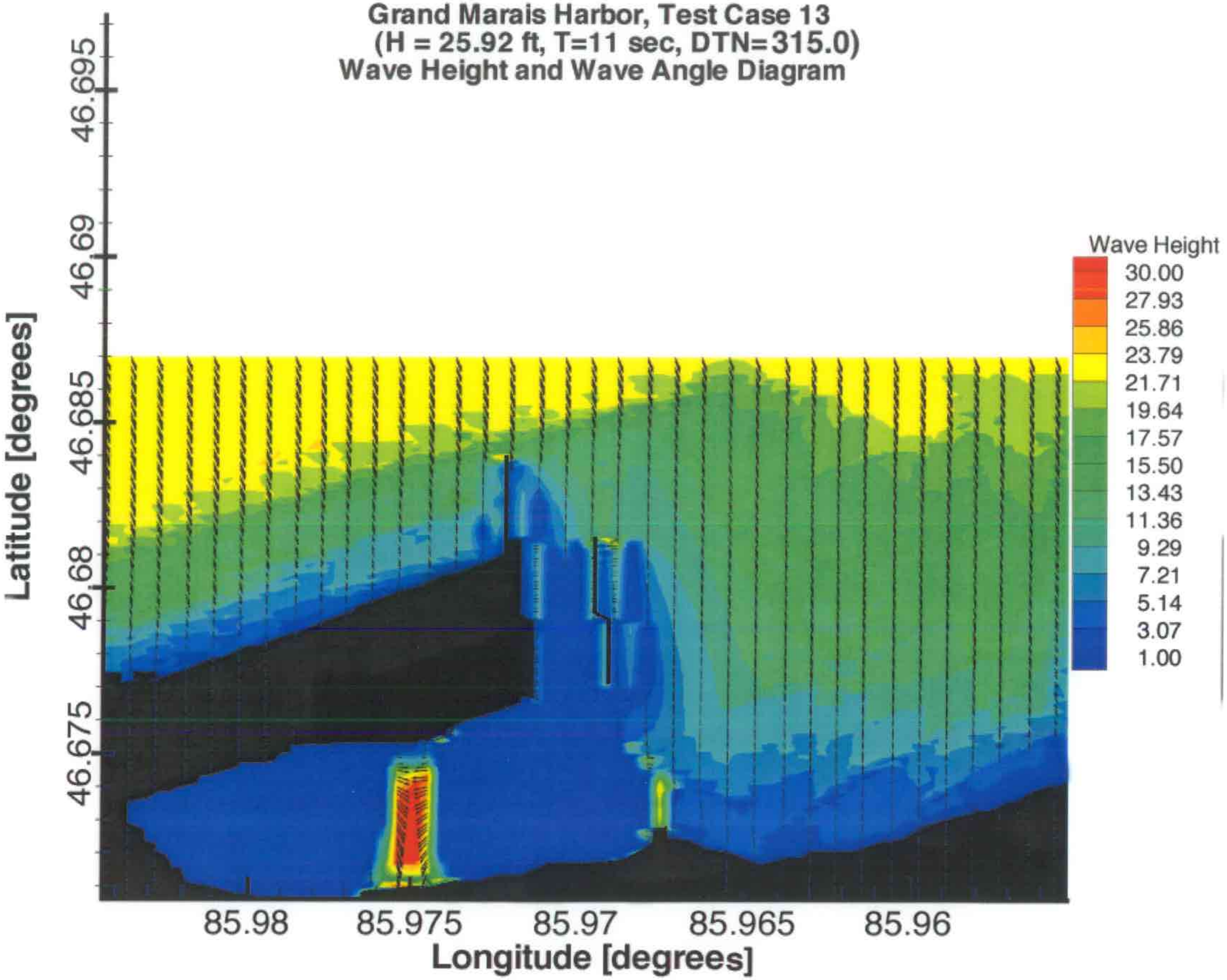
Grand Marais Harbor, Test Case 2
(H = 3.28 ft, T=4.5 sec, DTN=22.5)
Wave Height Wave Angle Diagram

Grand Marais Harbor, Test Case 6
(H = 4.27 ft, T=4.9 sec, DTN=315.0)
Wave Height and Wave Angle Diagram





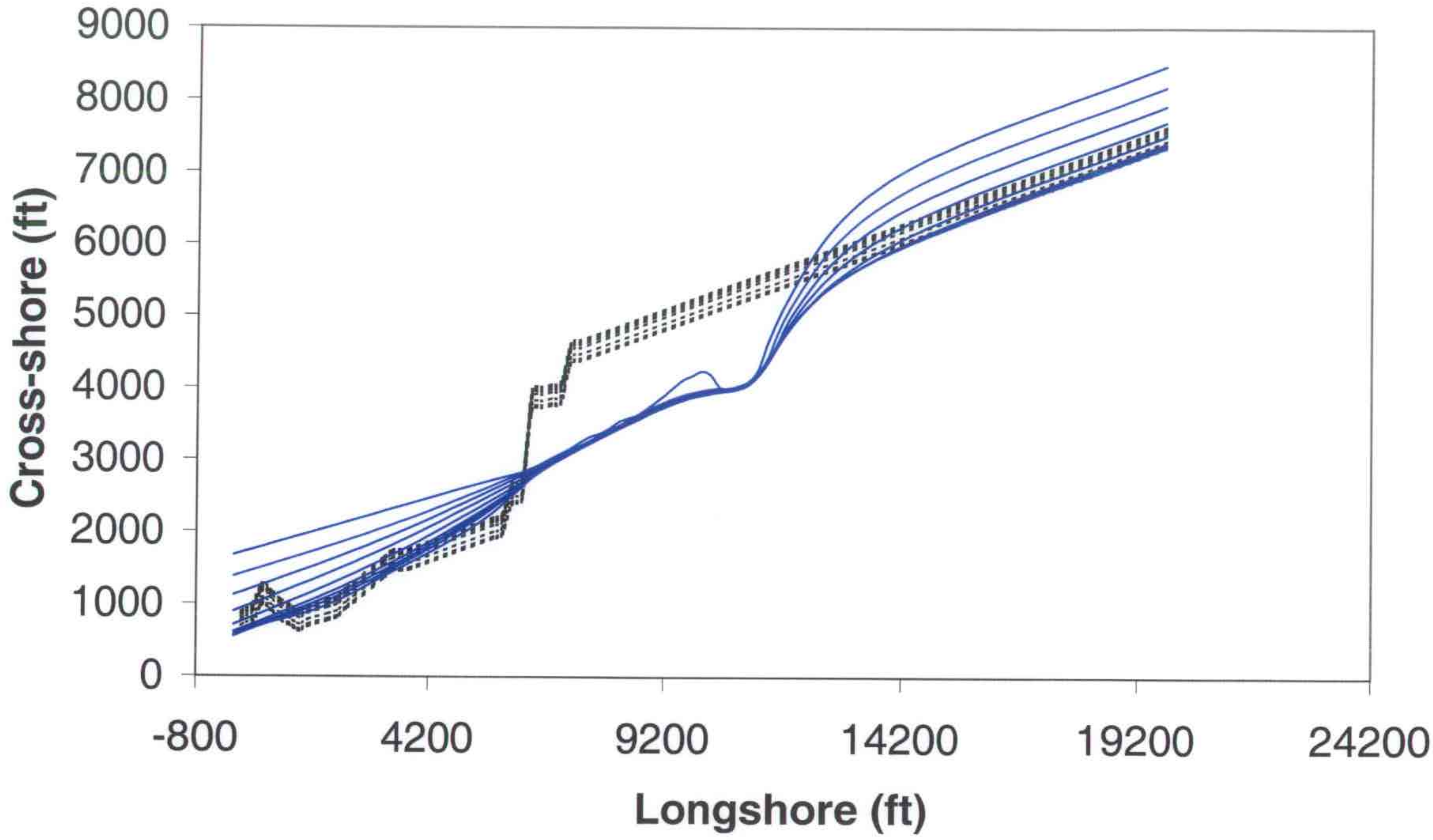
Grand Marais Harbor, Test Case 9
(H = 21.33 ft, T=10.5 sec, DTN=22.5)
Wave Height and Wave Angle Diagram



Grand Marais Harbor, Test Case 13
(H = 25.92 ft, T=11 sec, DTN=315.0)
Wave Height and Wave Angle Diagram

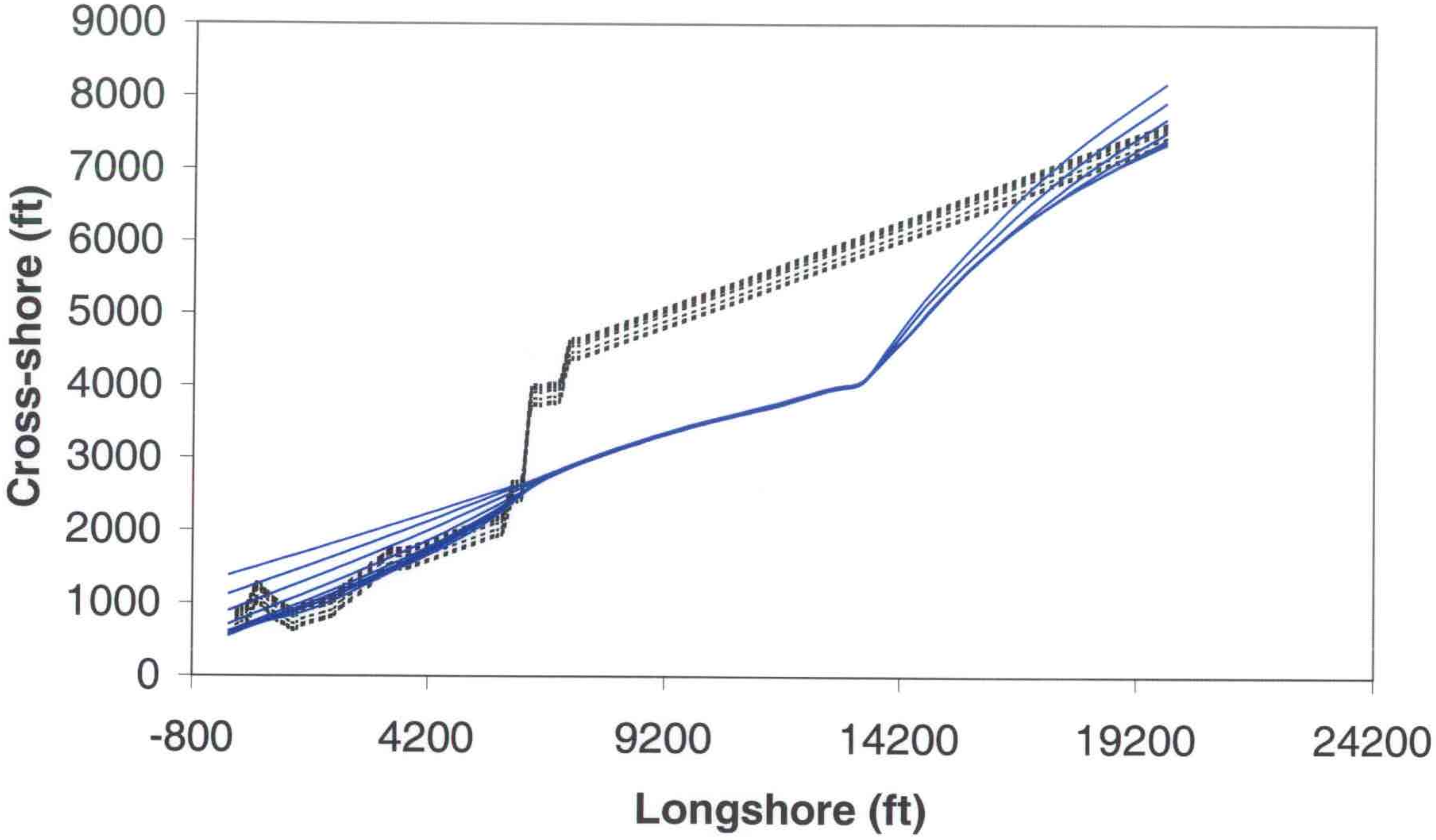
Appendix D:

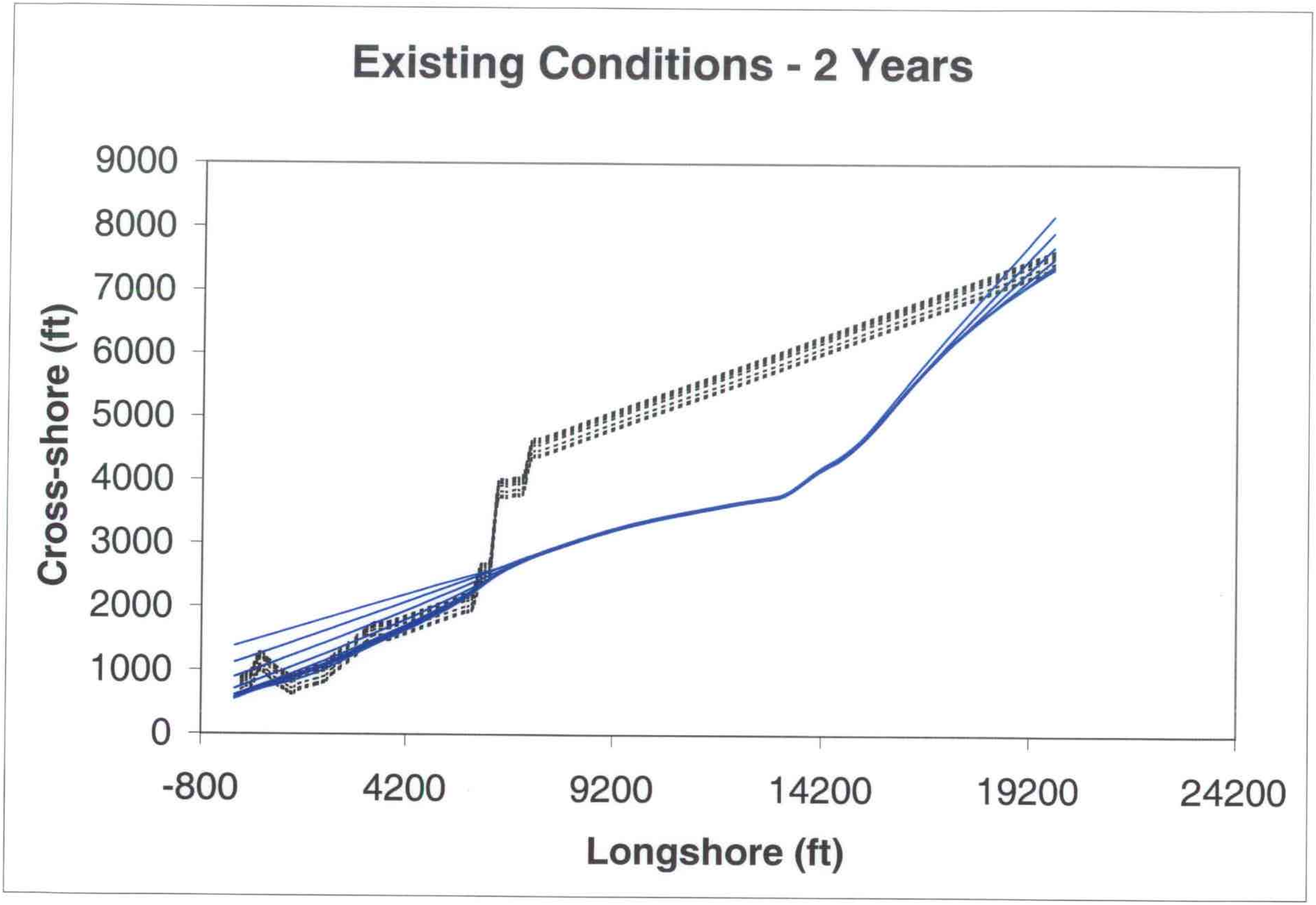
N-Line Output



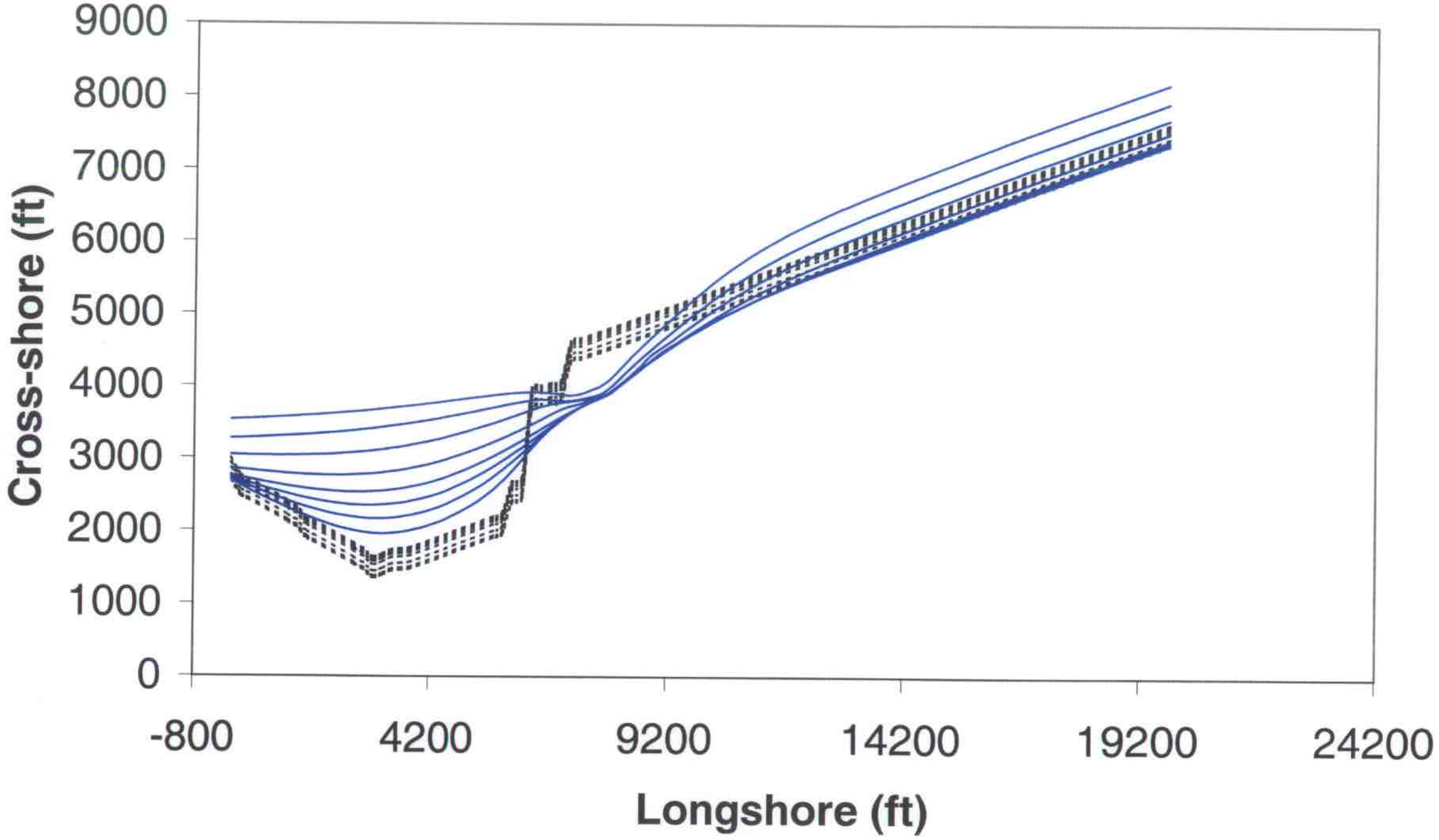
Existing Conditions - 1 Month

Existing Conditions - 1 Year

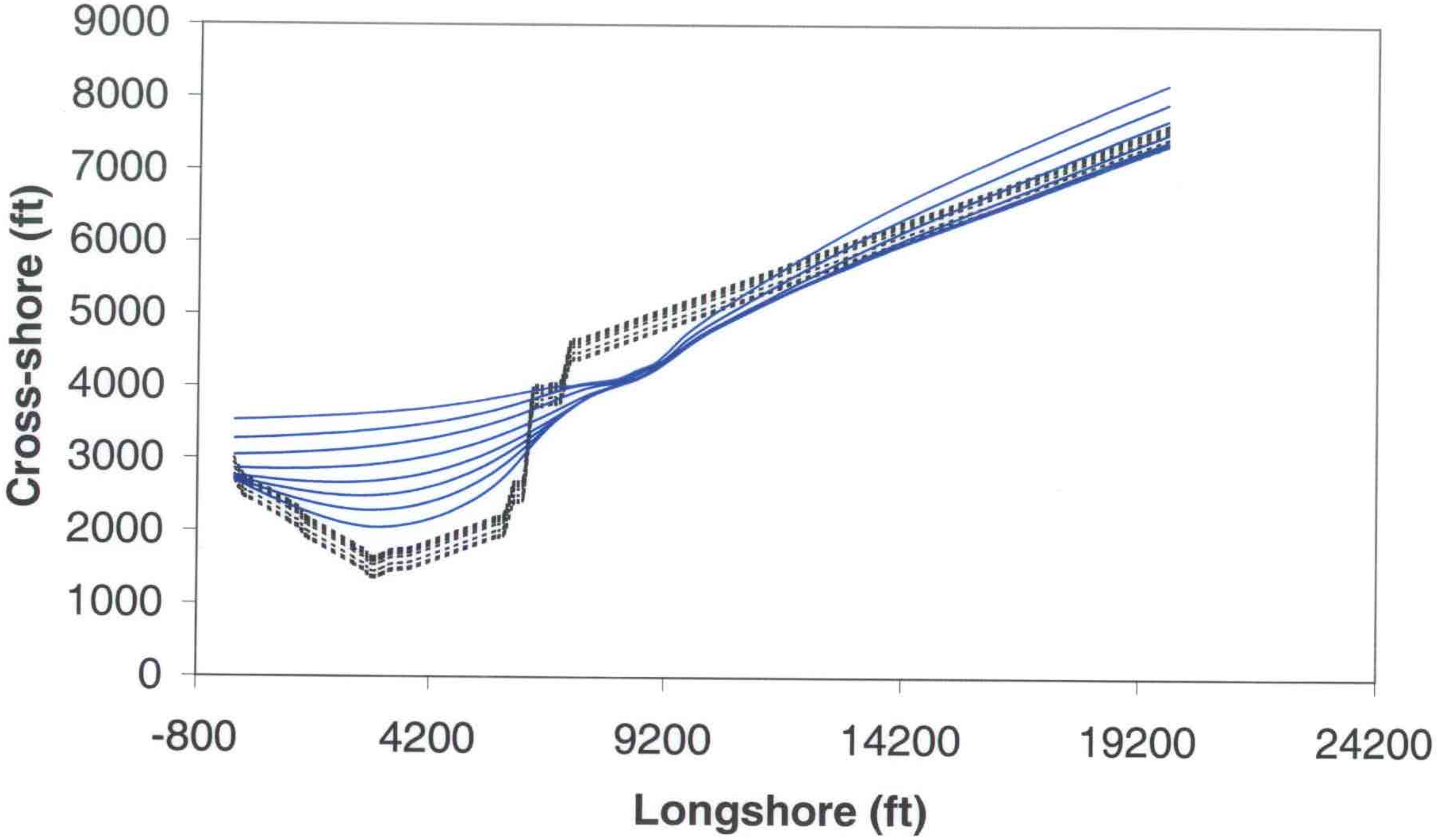


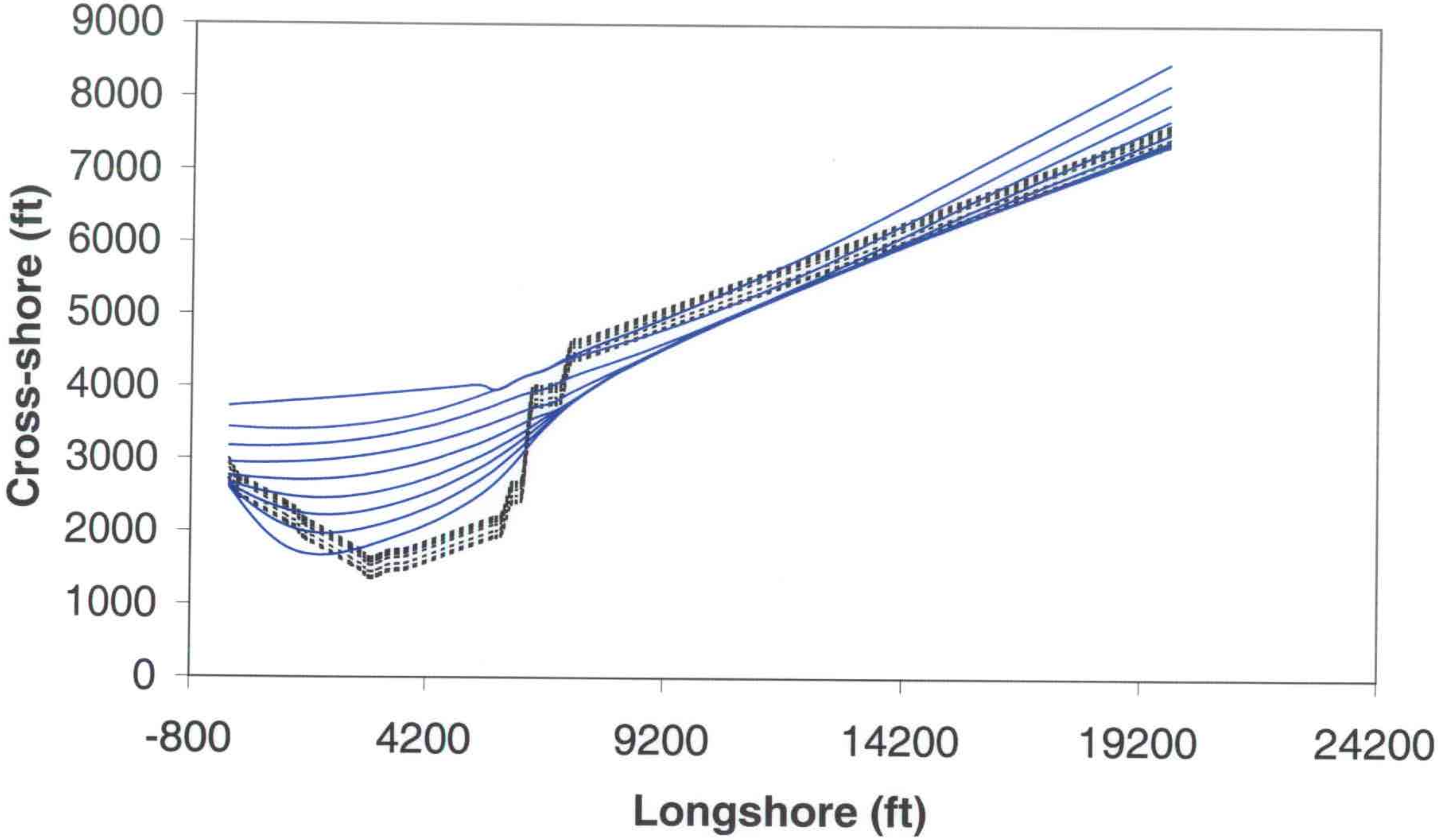

Existing Conditions - 2 Years
Cross-shore (ft)
9000
8000
7000
6000
5000
4000
3000
2000
1000
0
-800
4200
9200
14200
19200
24200
Longshore (ft)

55° Breakwater - 1 Month

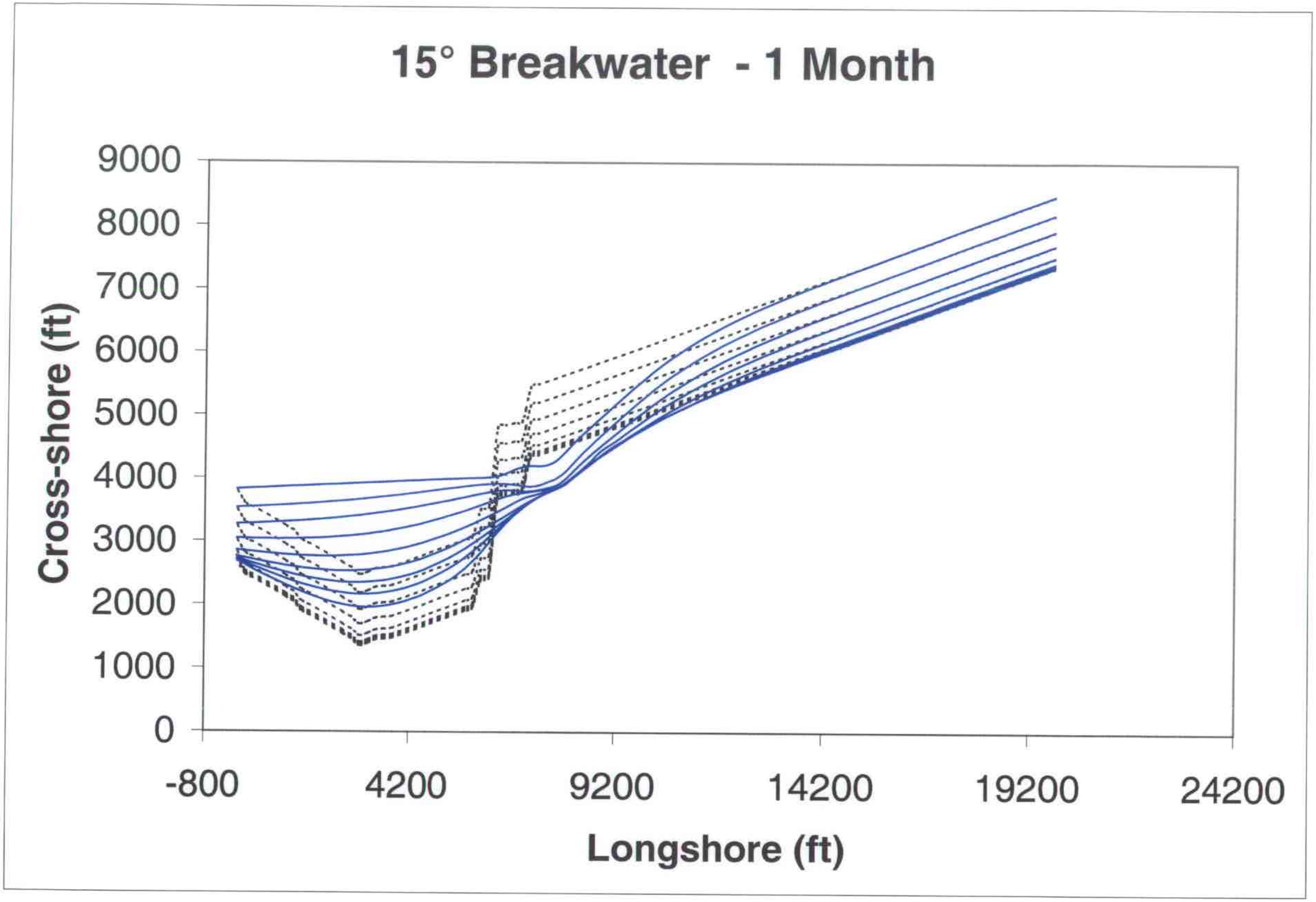


55° Breakwater - 1 Year

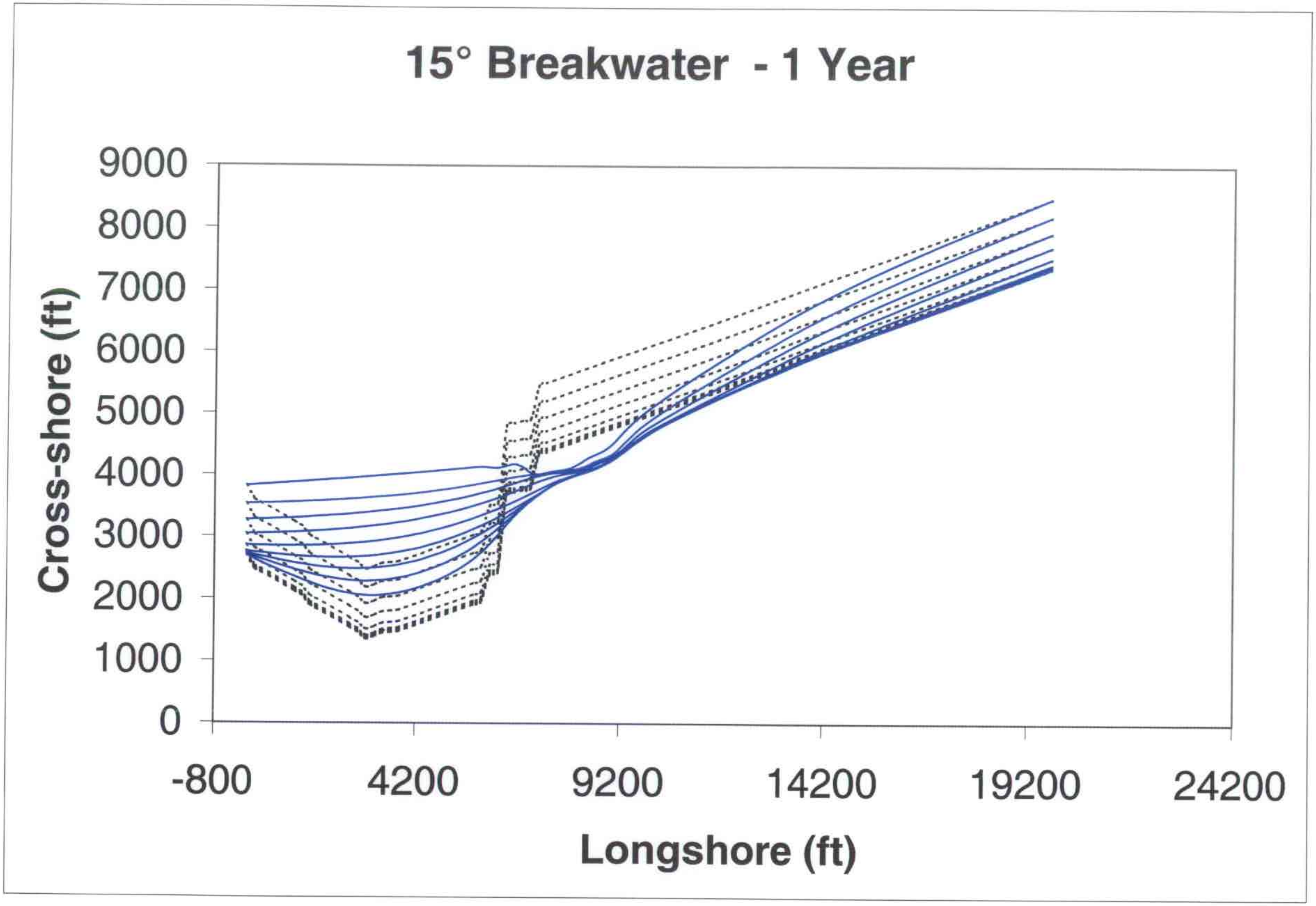


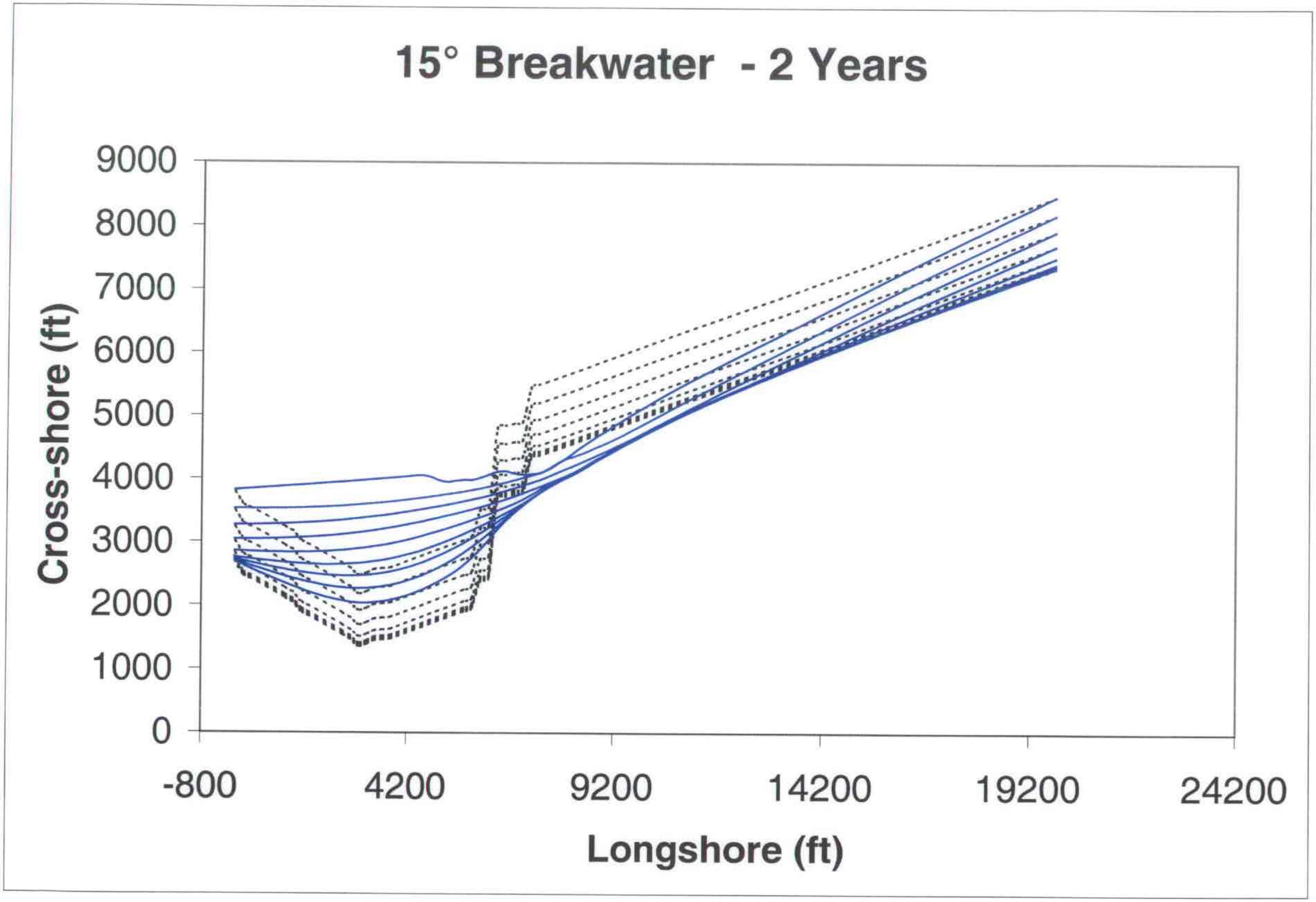


55° Breakwater - 2 Years

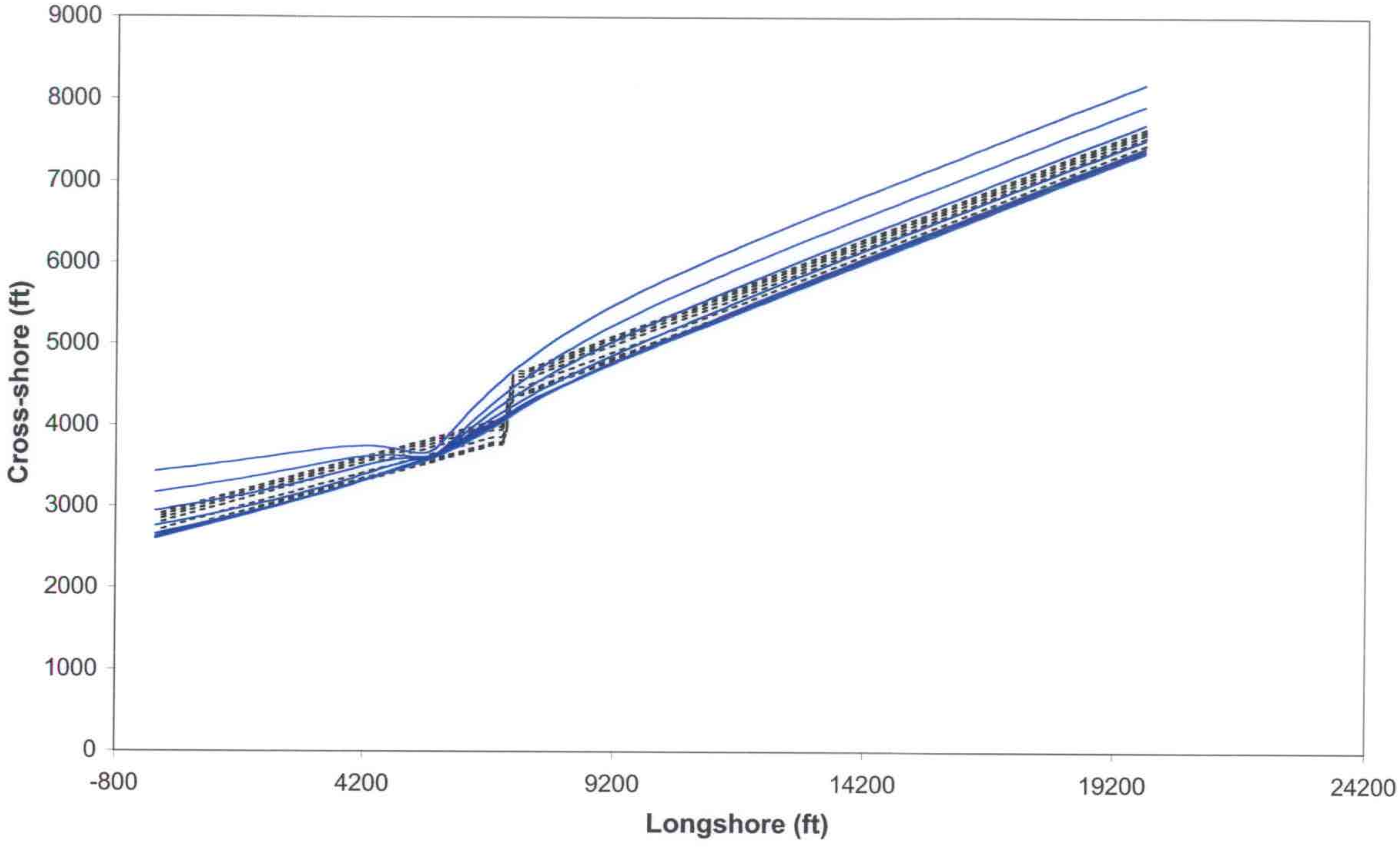


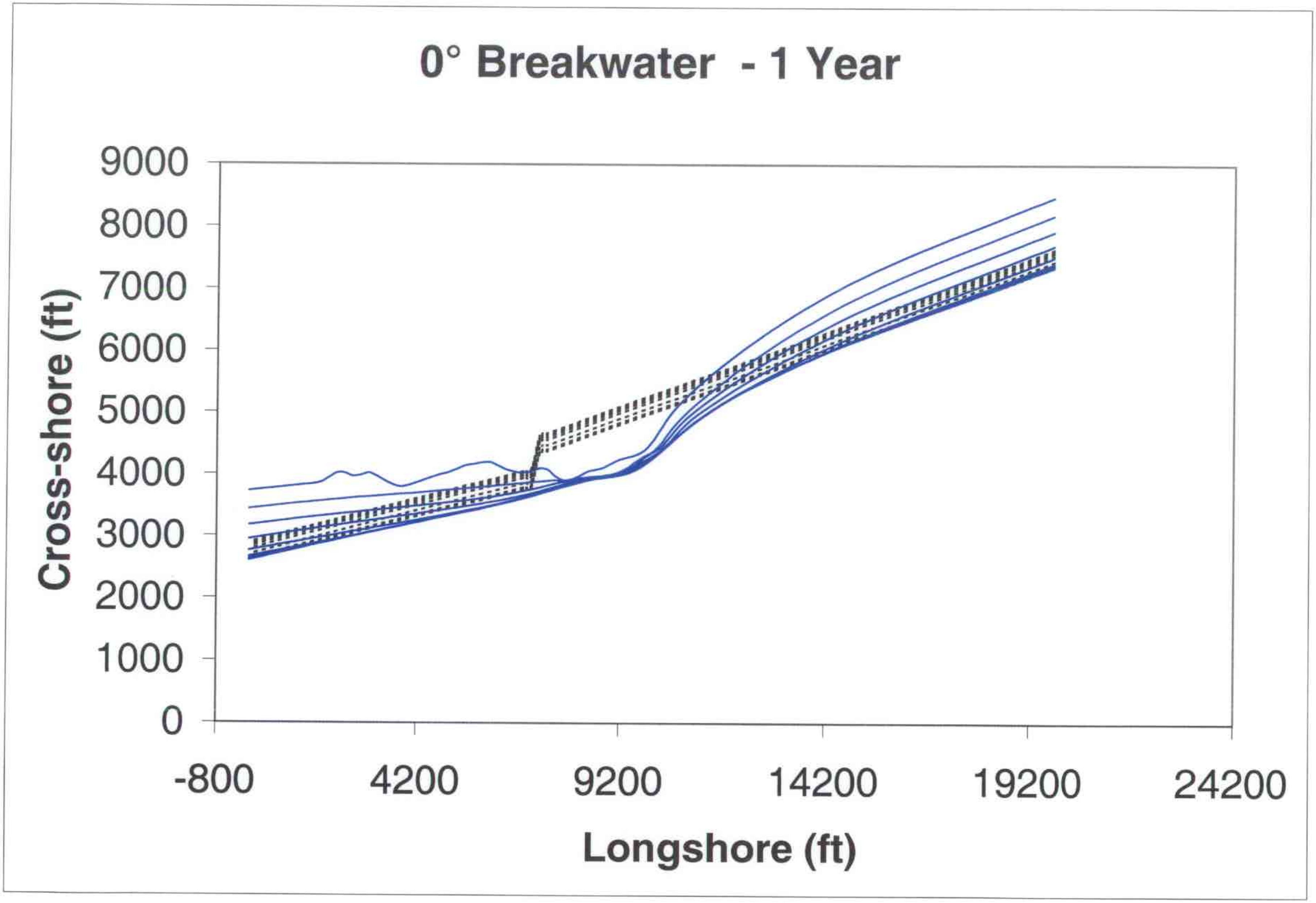
15° Breakwater - 1 Month

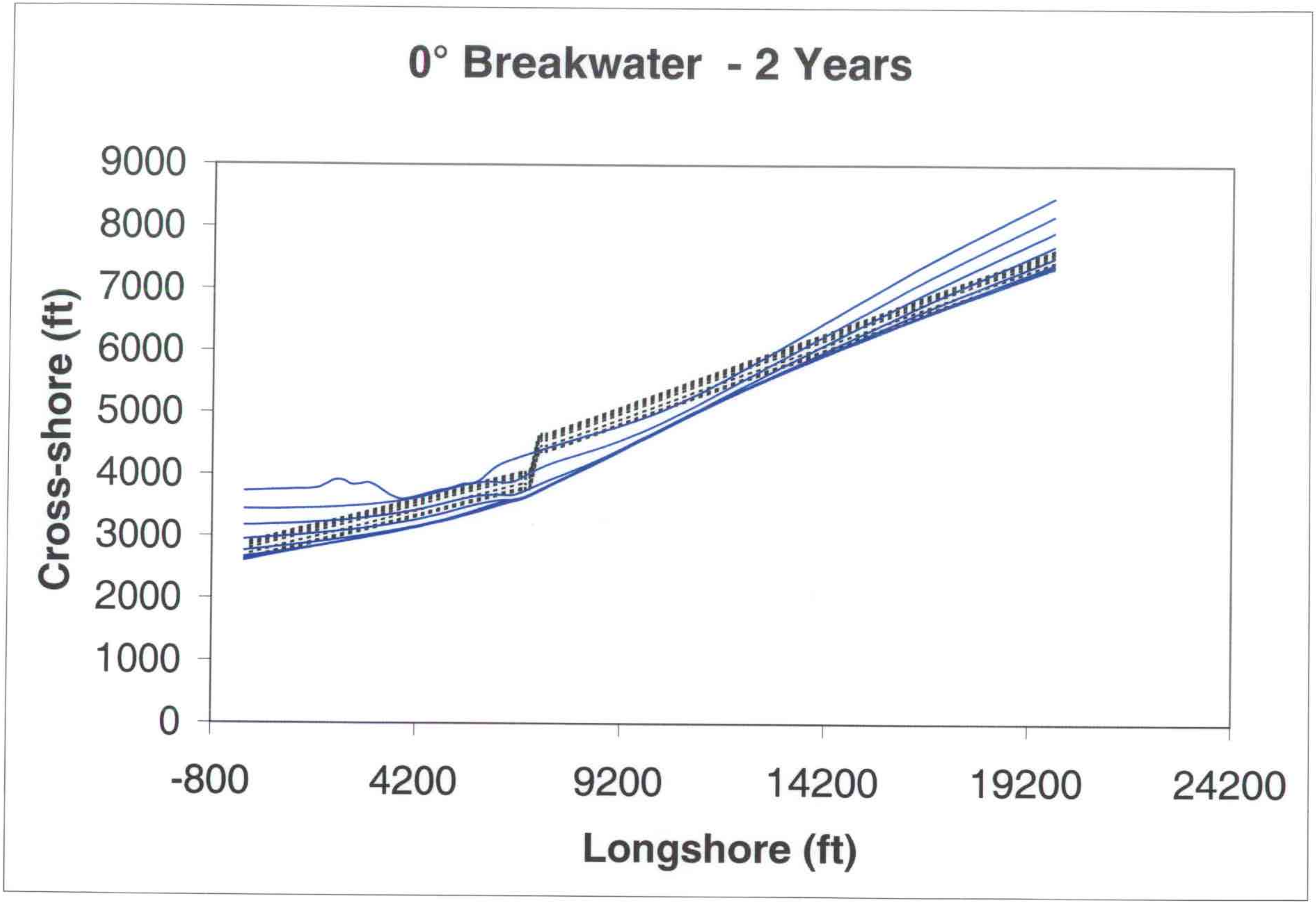

15° Breakwater - 1 Year
Cross-shore (ft)
9000
8000
7000
6000
5000
4000
3000
2000
1000
0
-800
4200
9200
14200
19200
24200
Longshore (ft)


15° Breakwater - 2 Years
Cross-shore (ft)
9000
8000
7000
6000
5000
4000
3000
2000
1000
0
-800
4200
9200
14200
19200
24200
Longshore (ft)

0° Breakwater - 1 Month

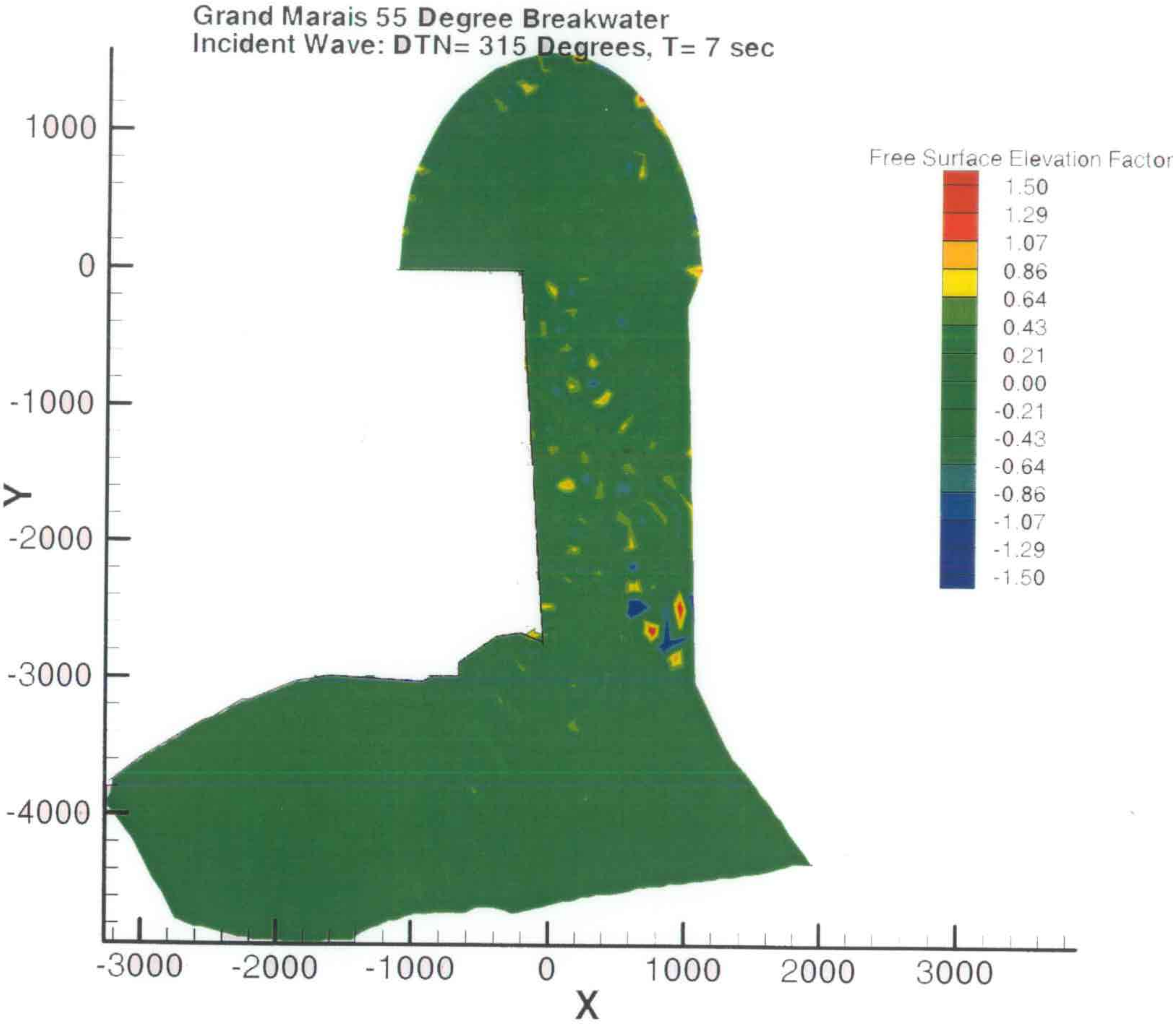



0° Breakwater - 1 Year
Cross-shore (ft)
9000
8000
7000
6000
5000
4000
3000
2000
1000
0
-800
4200
9200
14200
19200
24200
Longshore (ft)


0° Breakwater - 2 Years
Cross-shore (ft)
9000
8000
7000
6000
5000
4000
3000
2000
1000
0
-800
4200
9200
14200
19200
24200
Longshore (ft)

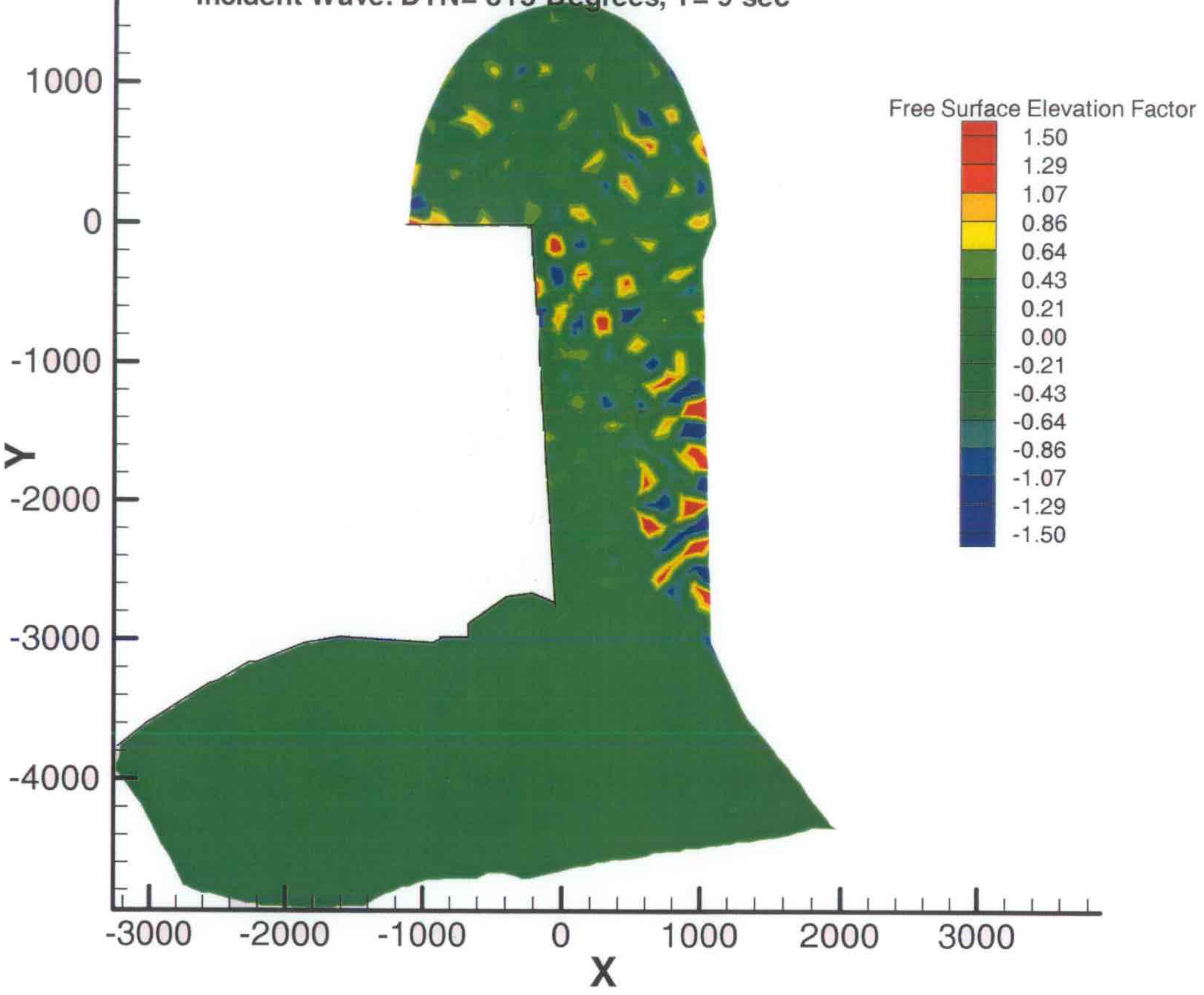
Appendix E:

Selected HARBD Output

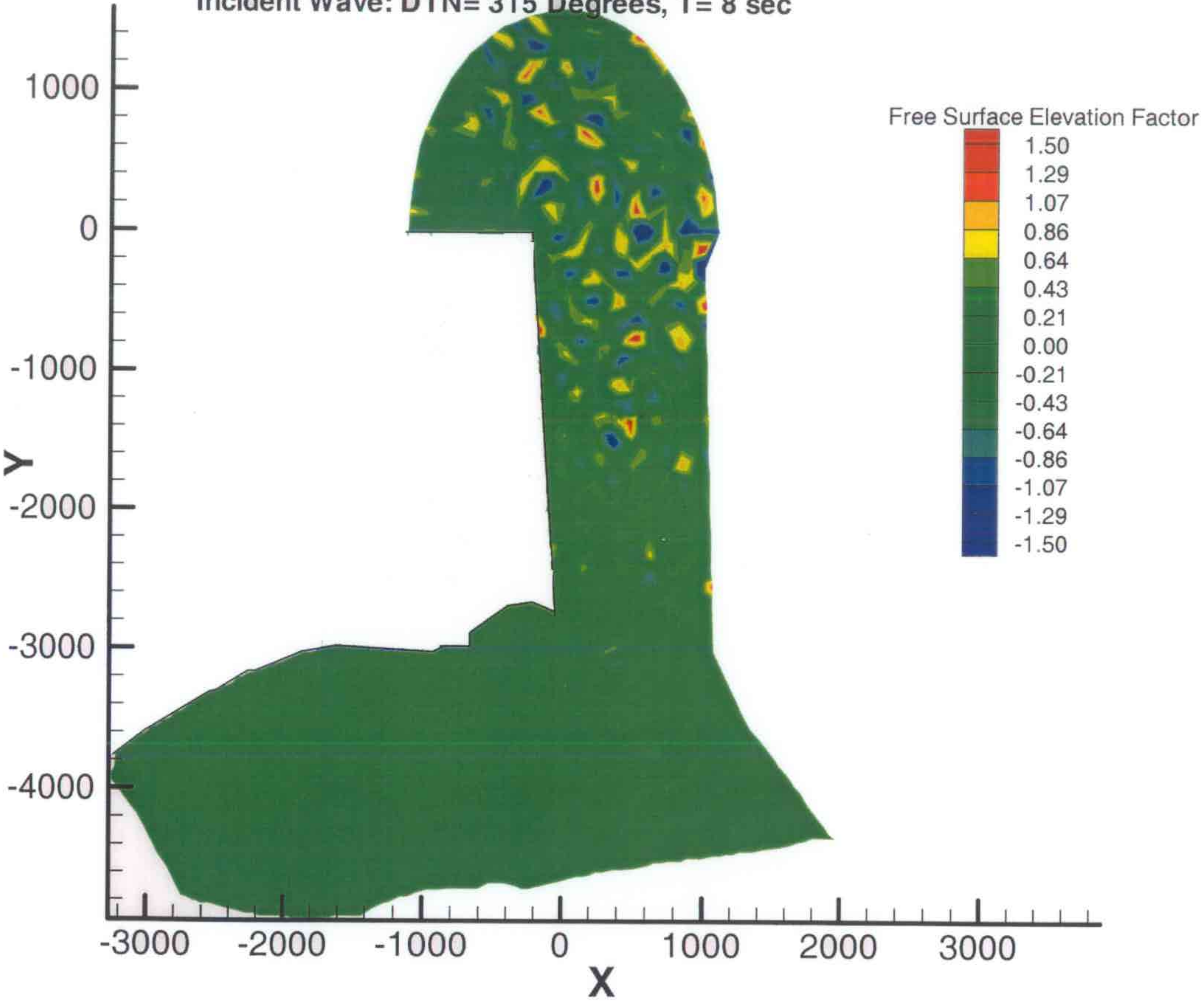


Grand Marais 55 Degree Breakwater
Incident Wave: DTN= 315 Degrees, T= 7 sec

Grand Marais 55 Degree Breakwater
Incident Wave: DTN= 315 Degrees, T= 9 sec

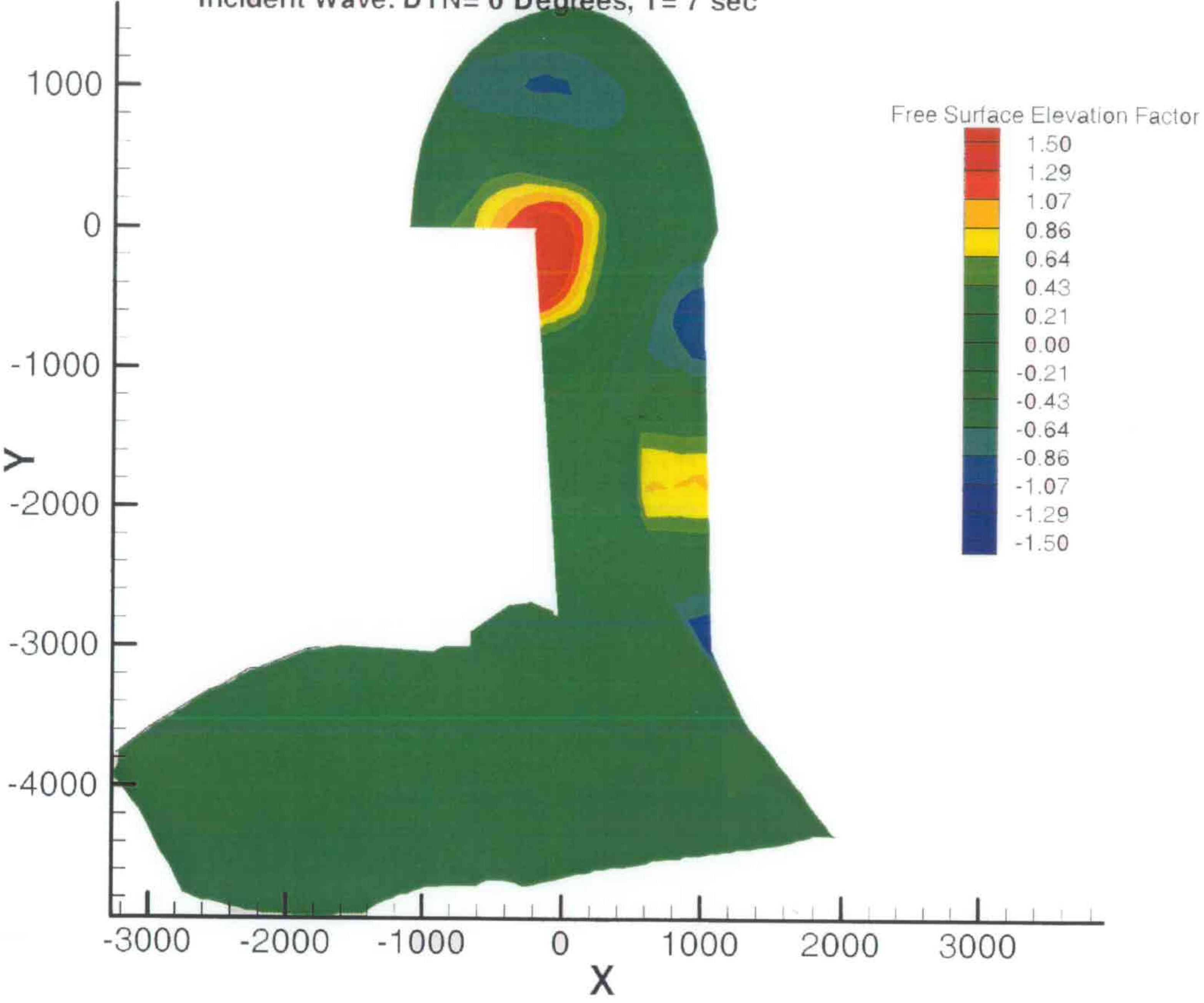


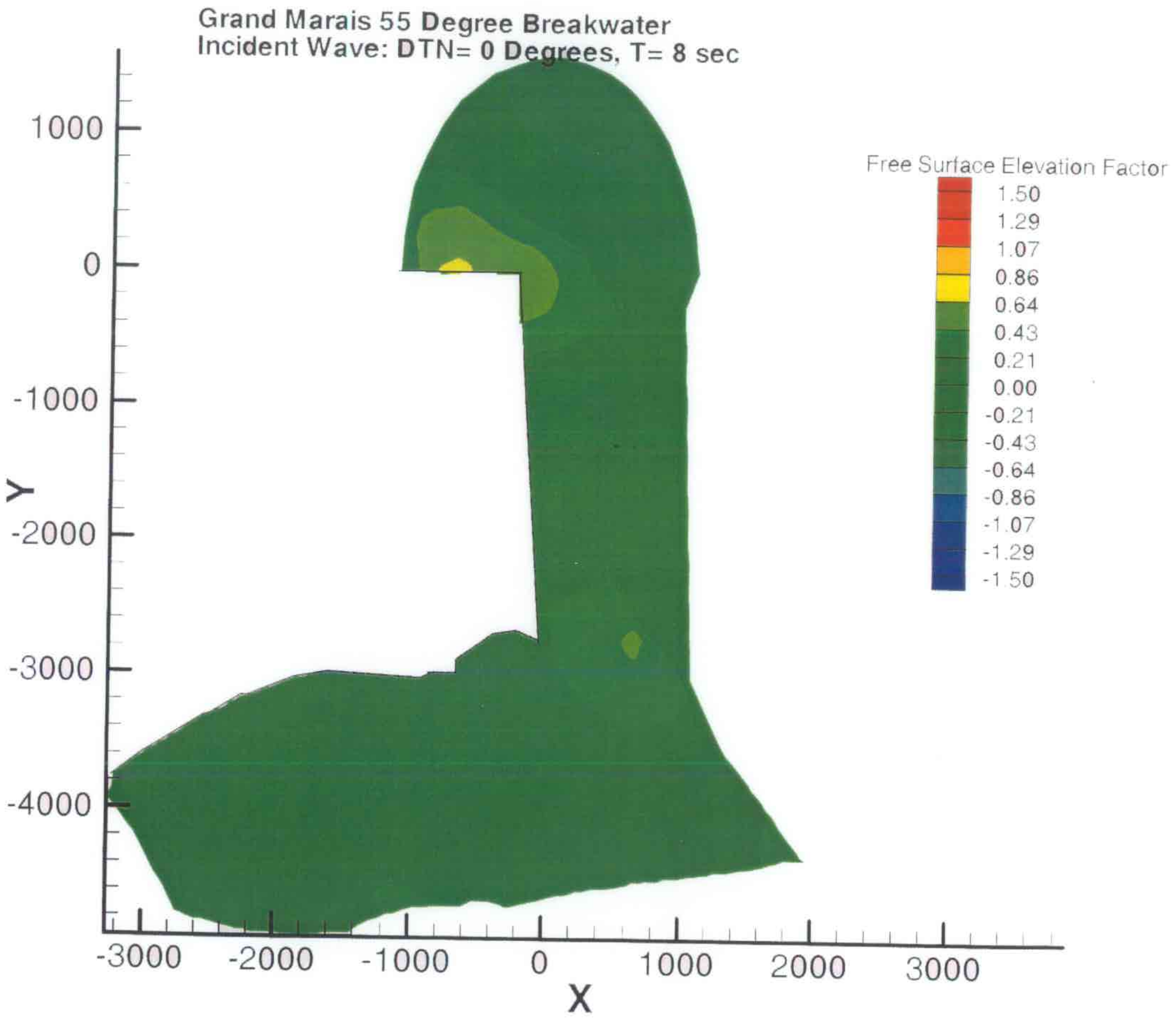
Grand Marais 55 Degree Breakwater
Incident Wave: DTN= 315 Degrees, T= 8 sec

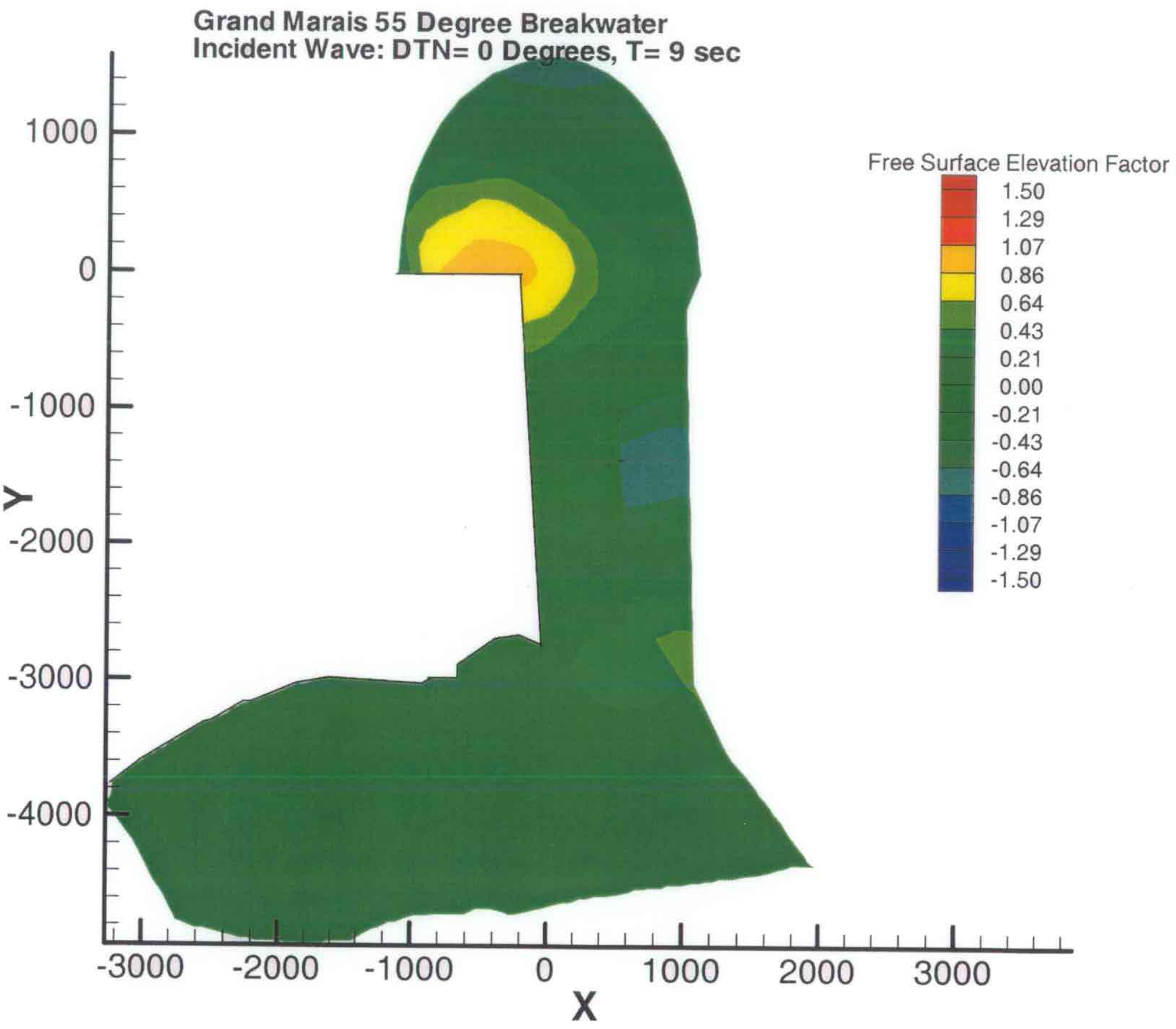


Frame 001 | 15 Sep 2000 | Converted Excel Data

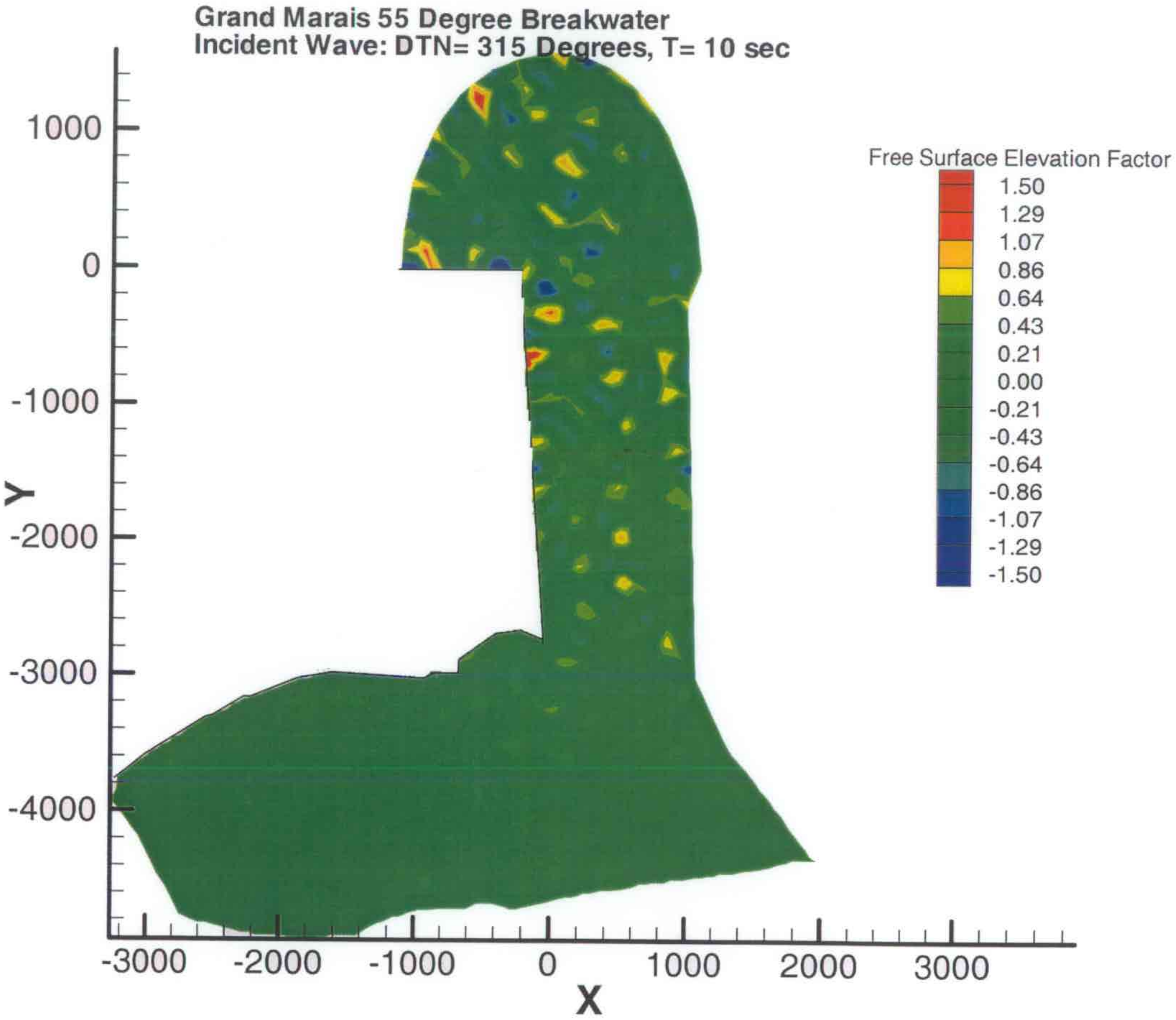
Grand Marais 55 Degree Breakwater
Incident Wave: DTN= 0 Degrees, T= 7 sec




Grand Marais 55 Degree Breakwater
Incident Wave: DTN= 0 Degrees, T= 8 sec
Free Surface Elevation Factor
1.50
1.29
1.07
0.86
0.64
0.43
0.21
0.00
-0.21
-0.43
-0.64
-0.86
-1.07
-1.29
-1.50
1000
0
-1000
-2000
-3000
-4000
Y
-3000
-2000
-1000
0
1000
2000
3000
X

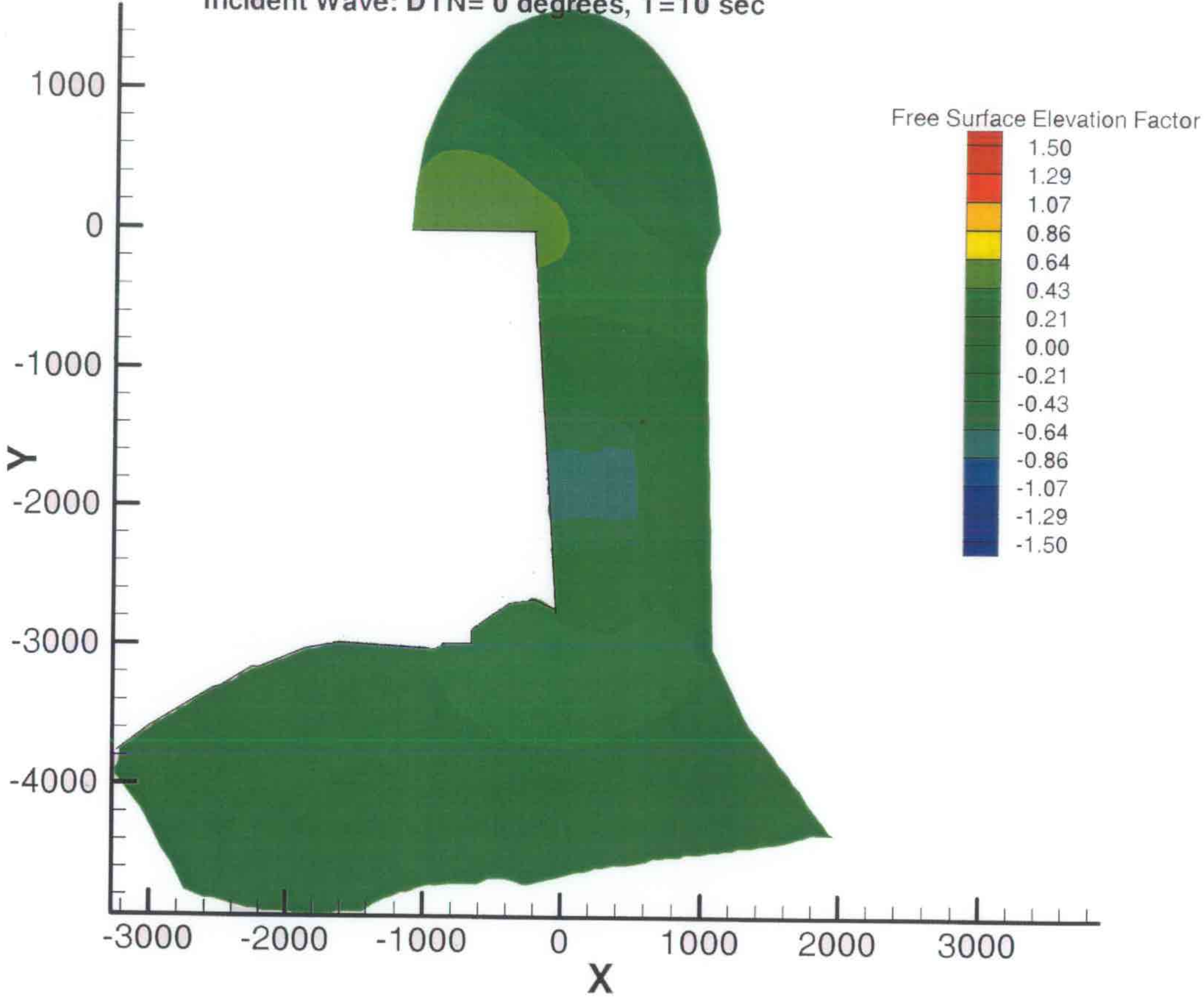


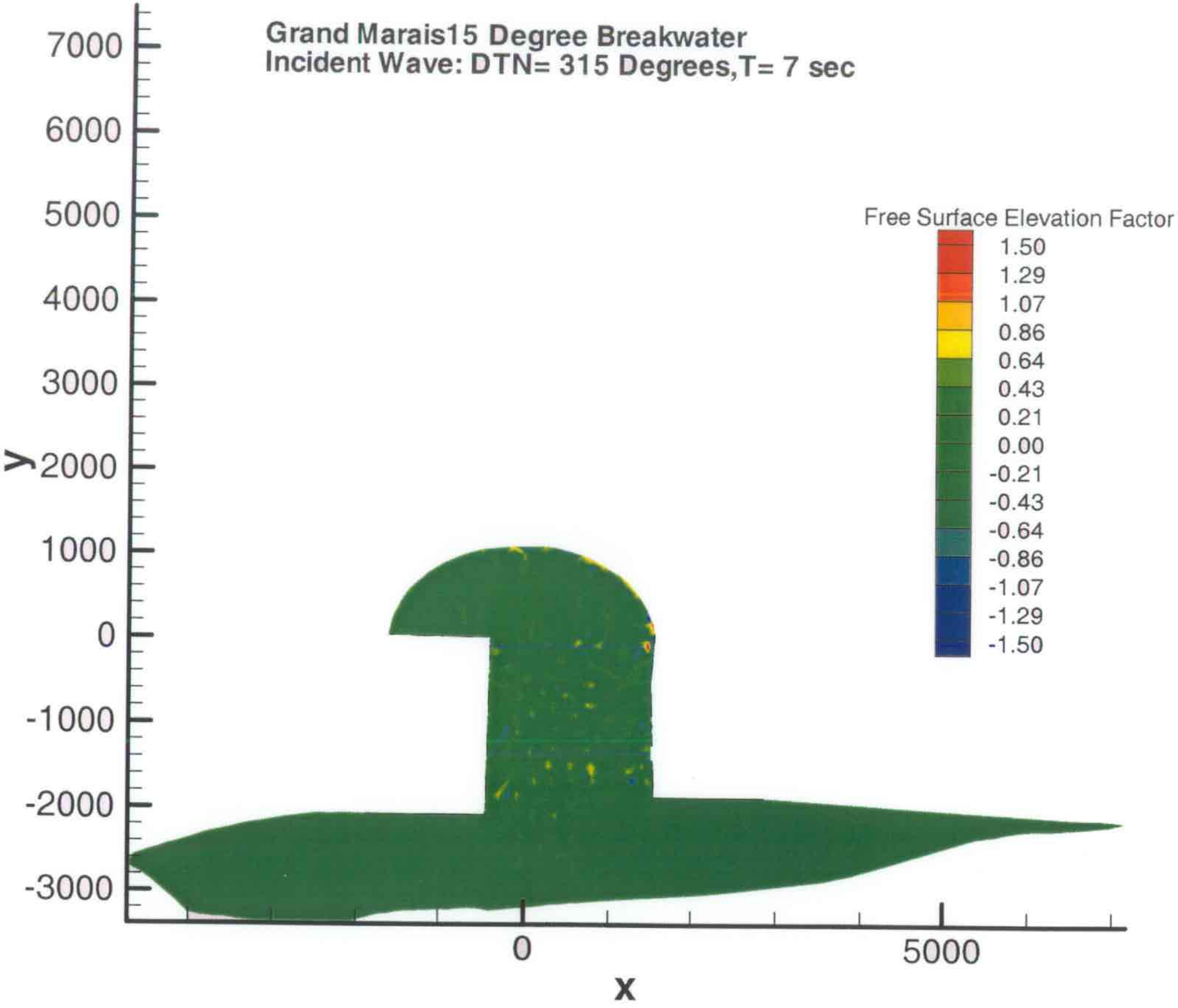
Grand Marais 55 Degree Breakwater
Incident Wave: DTN= 0 Degrees, T= 9 sec



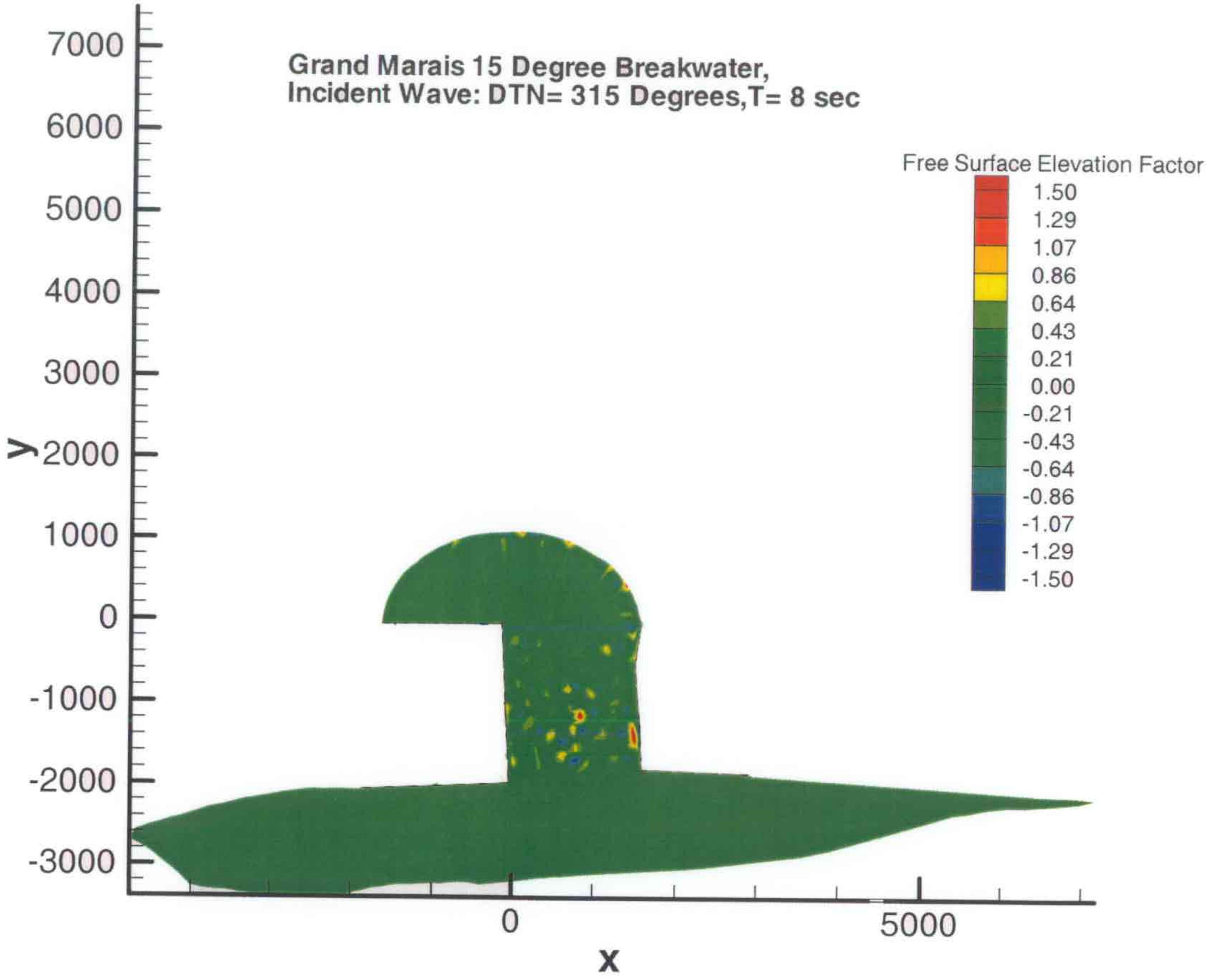
Grand Marais 55 Degree Breakwater
Incident Wave: DTN= 315 Degrees, T= 10 sec

Grand Marais 55 Degree Breakwater
Incident Wave: DTN= 0 degrees, T=10 sec

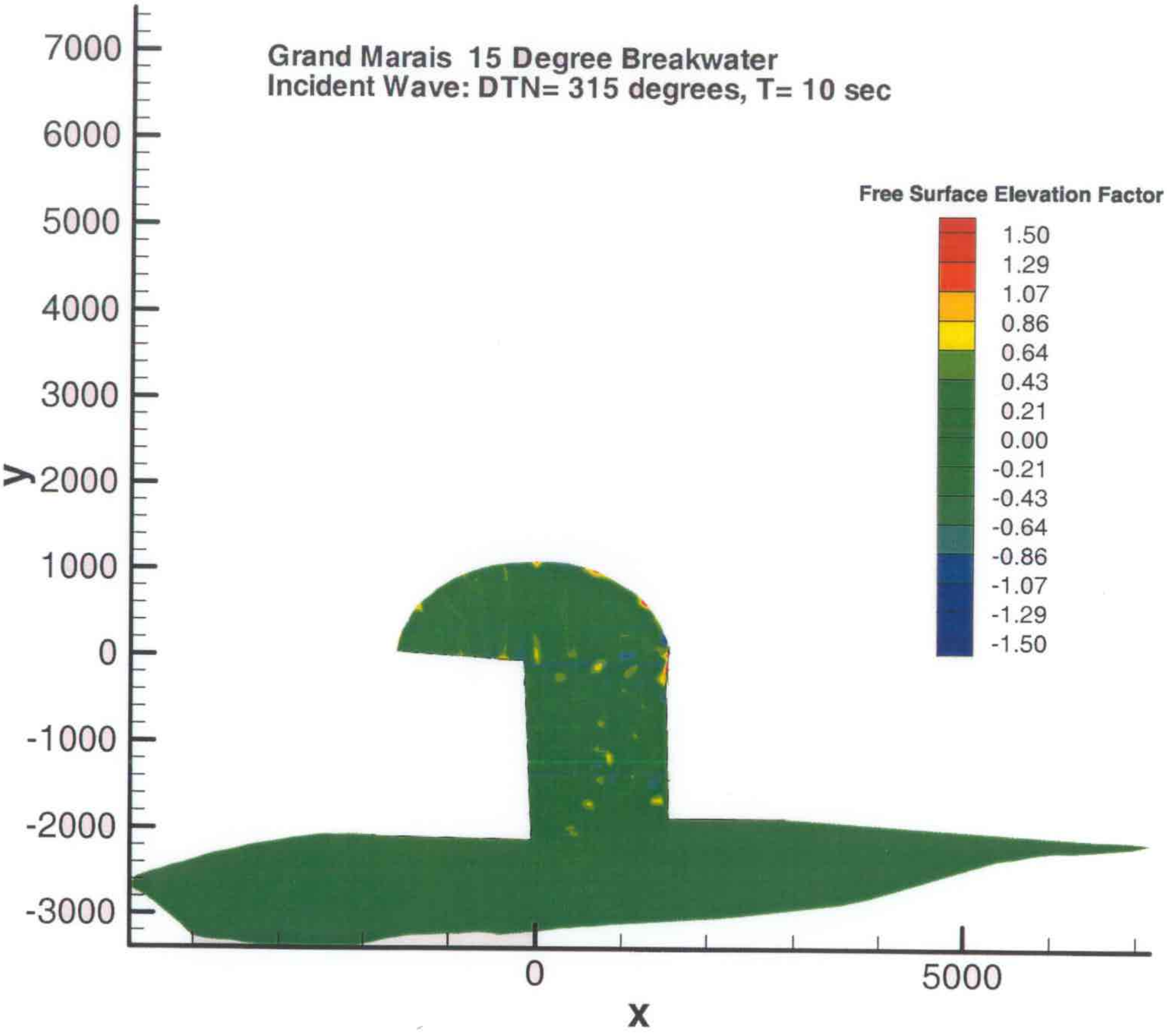


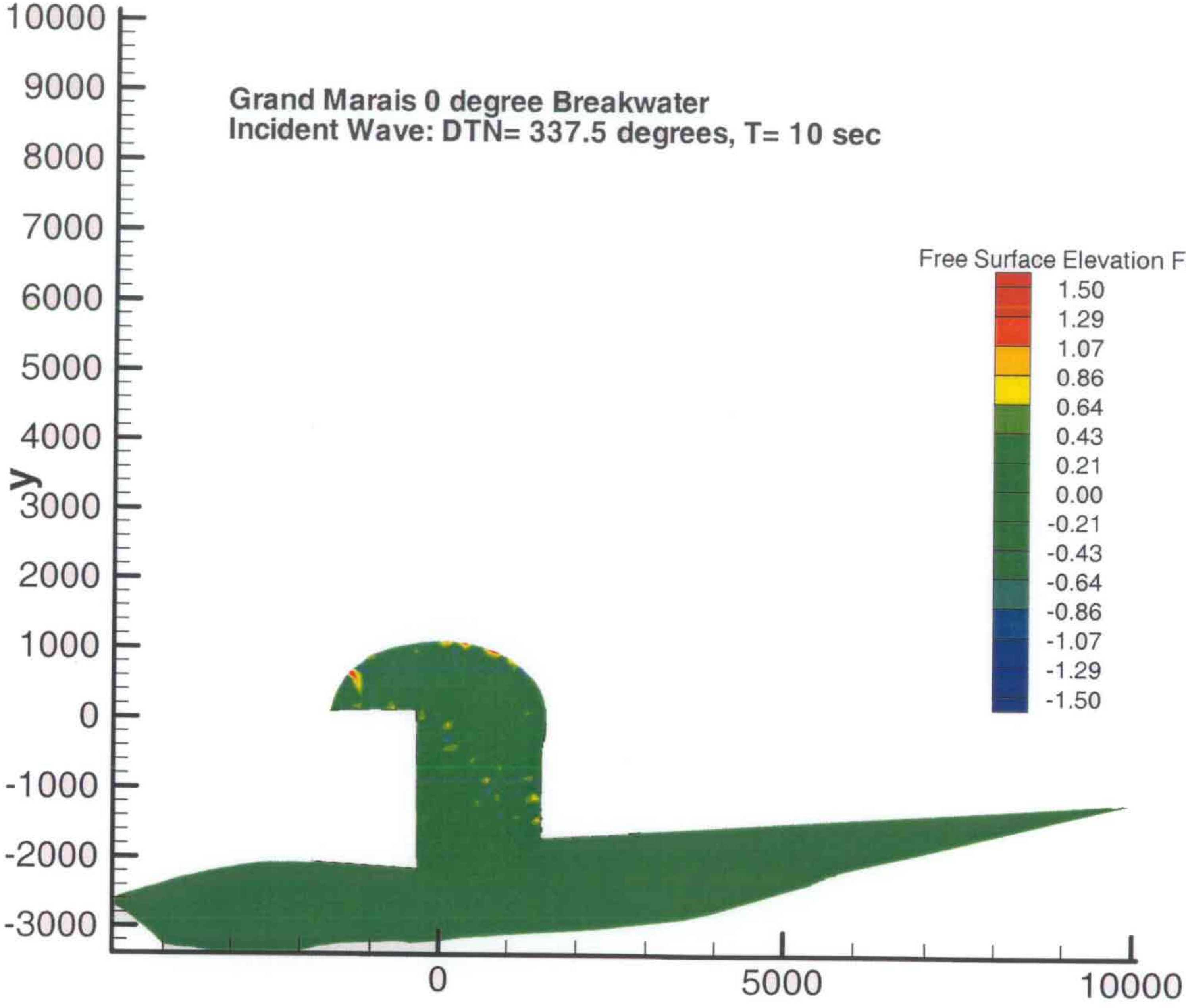


Grand Marais15 Degree Breakwater
Incident Wave: DTN= 315 Degrees,T= 7 sec

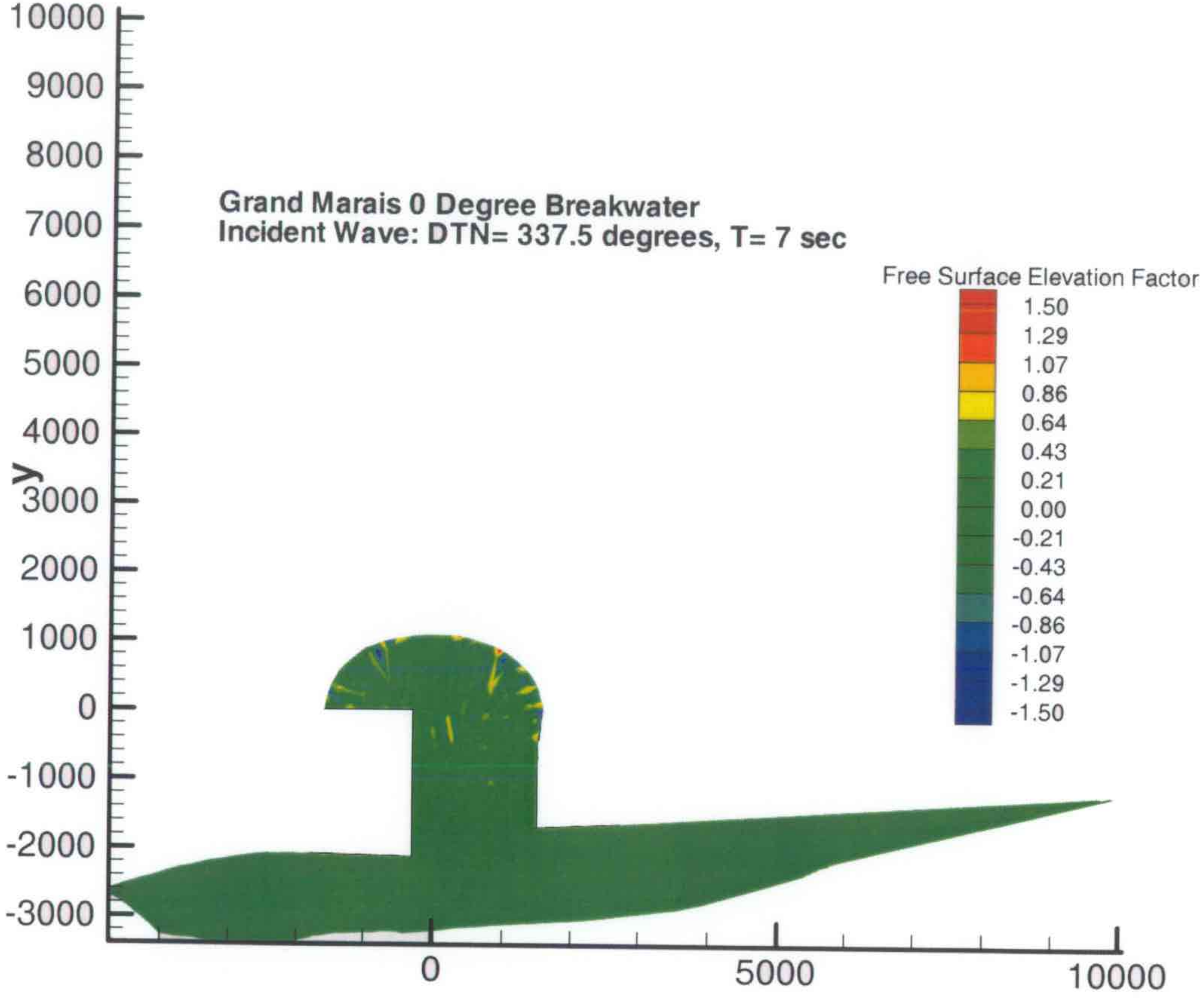


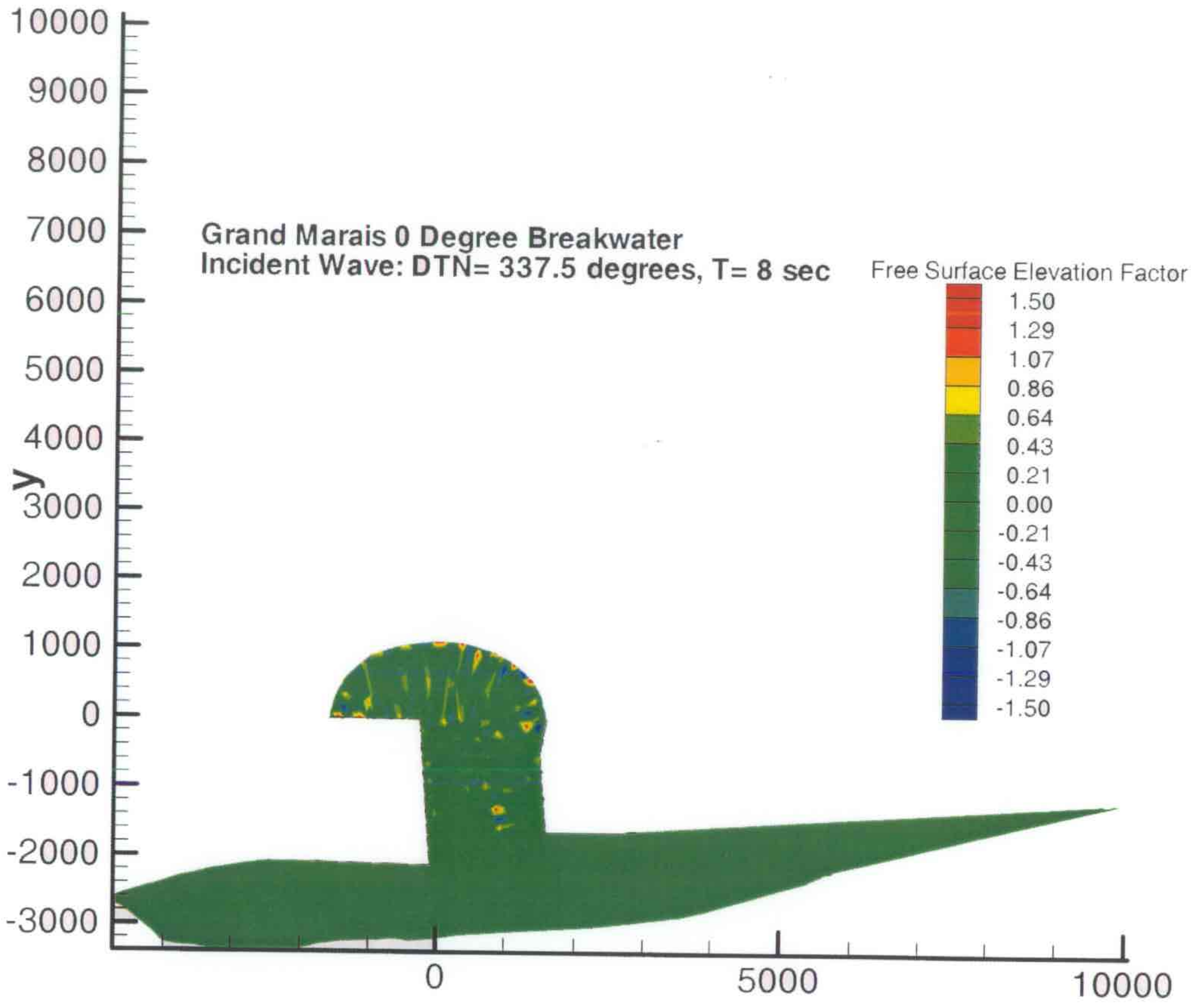
Grand Marais 15 Degree Breakwater,
Incident Wave: DTN= 315 Degrees,T= 8 sec

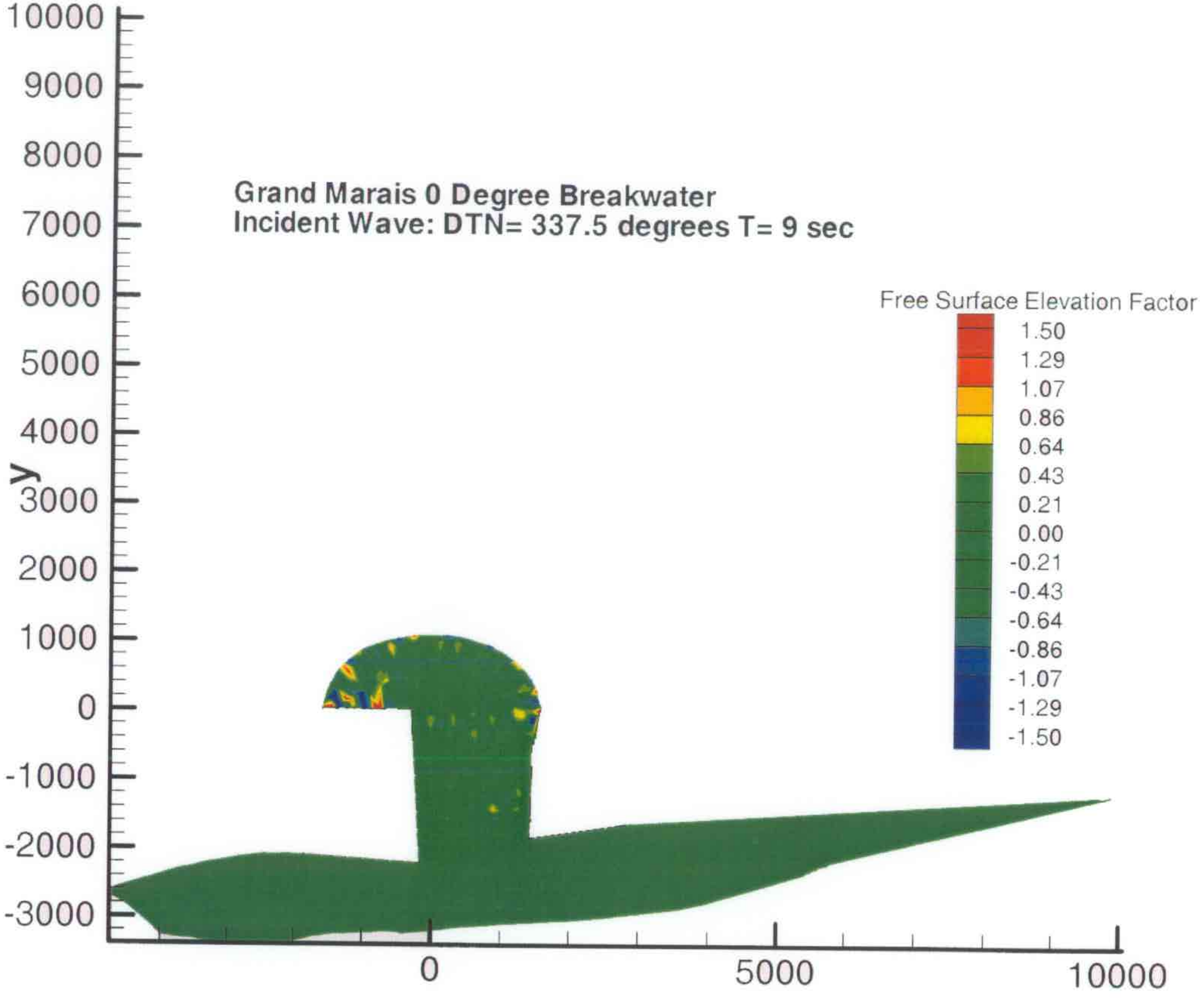

Grand Marais 15 Degree Breakwater
Incident Wave: DTN= 315 degrees, T= 10 sec
Free Surface Elevation Factor
1.50
1.29
1.07
0.86
0.64
0.43
0.21
0.00
-0.21
-0.43
-0.64
-0.86
-1.07
-1.29
-1.50
y
7000
6000
5000
4000
3000
2000
1000
0
-1000
-2000
-3000
x
0
5000

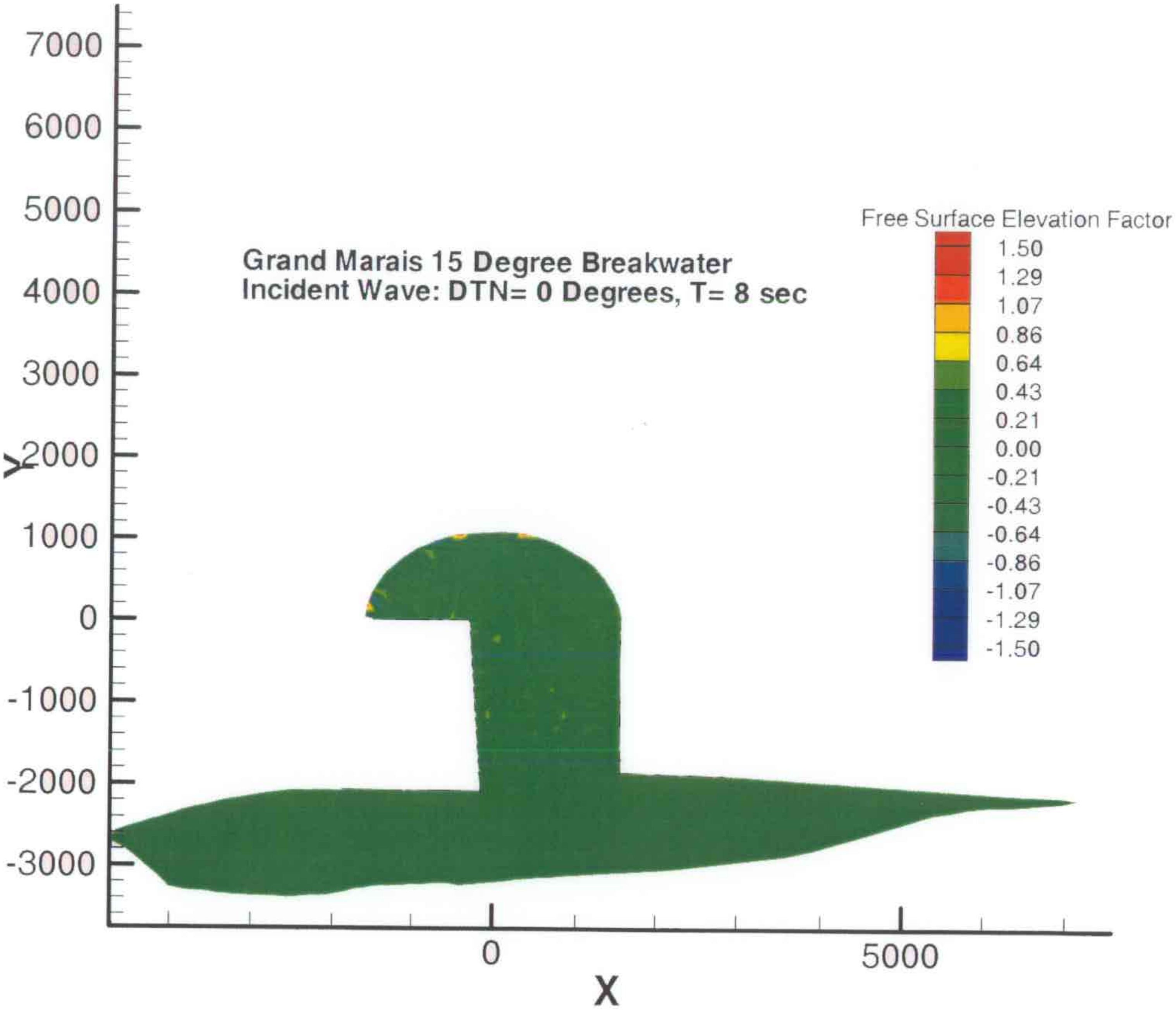


Grand Marais 0 degree Breakwater
Incident Wave: DTN= 337.5 degrees, T= 10 sec

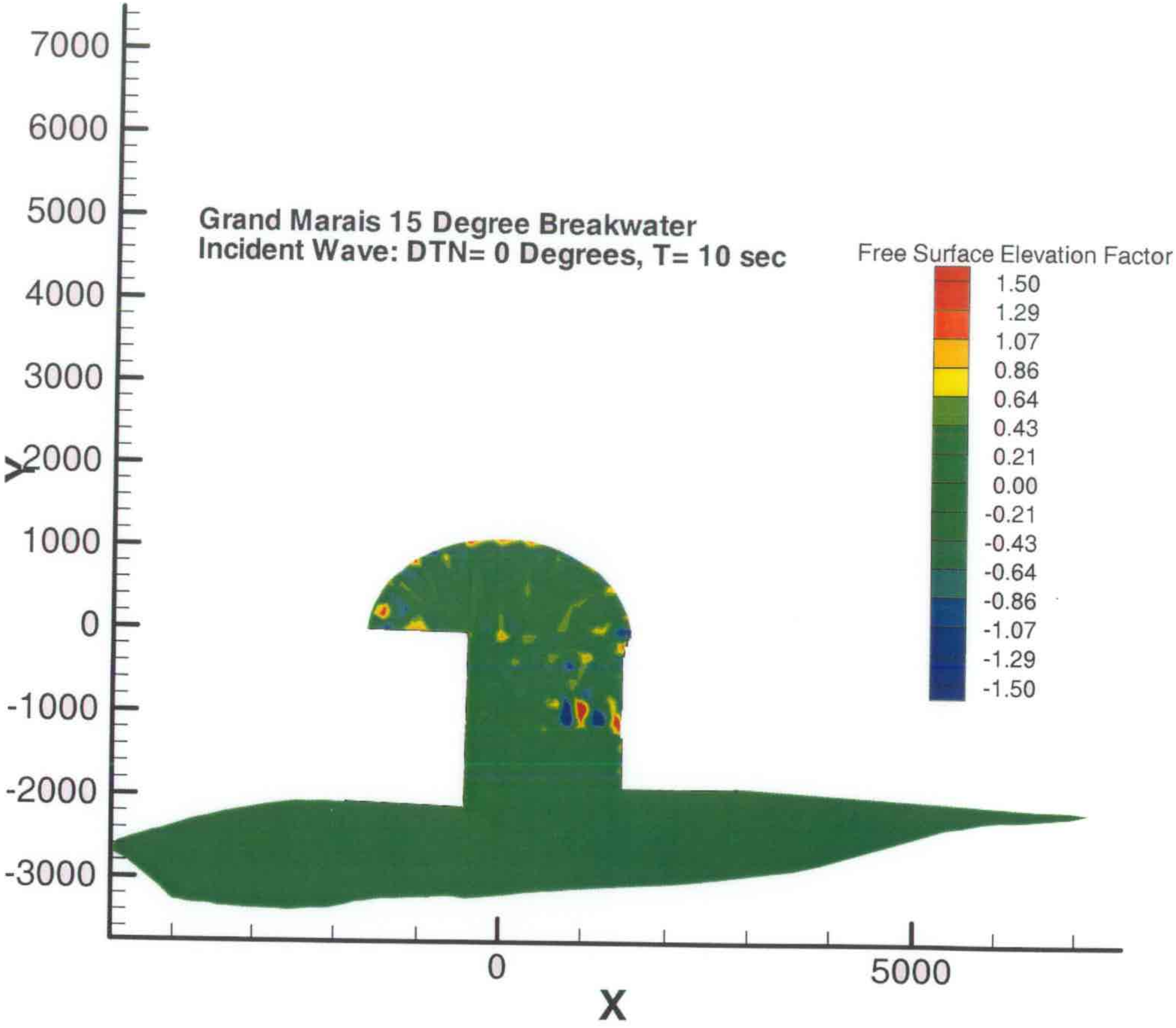

Grand Marais 0 Degree Breakwater
Incident Wave: DTN= 337.5 degrees, T= 7 sec
Free Surface Elevation Factor
1.50
1.29
1.07
0.86
0.64
0.43
0.21
0.00
-0.21
-0.43
-0.64
-0.86
-1.07
-1.29
-1.50
y
10000
9000
8000
7000
6000
5000
4000
3000
2000
1000
0
-1000
-2000
-3000
0
5000
10000

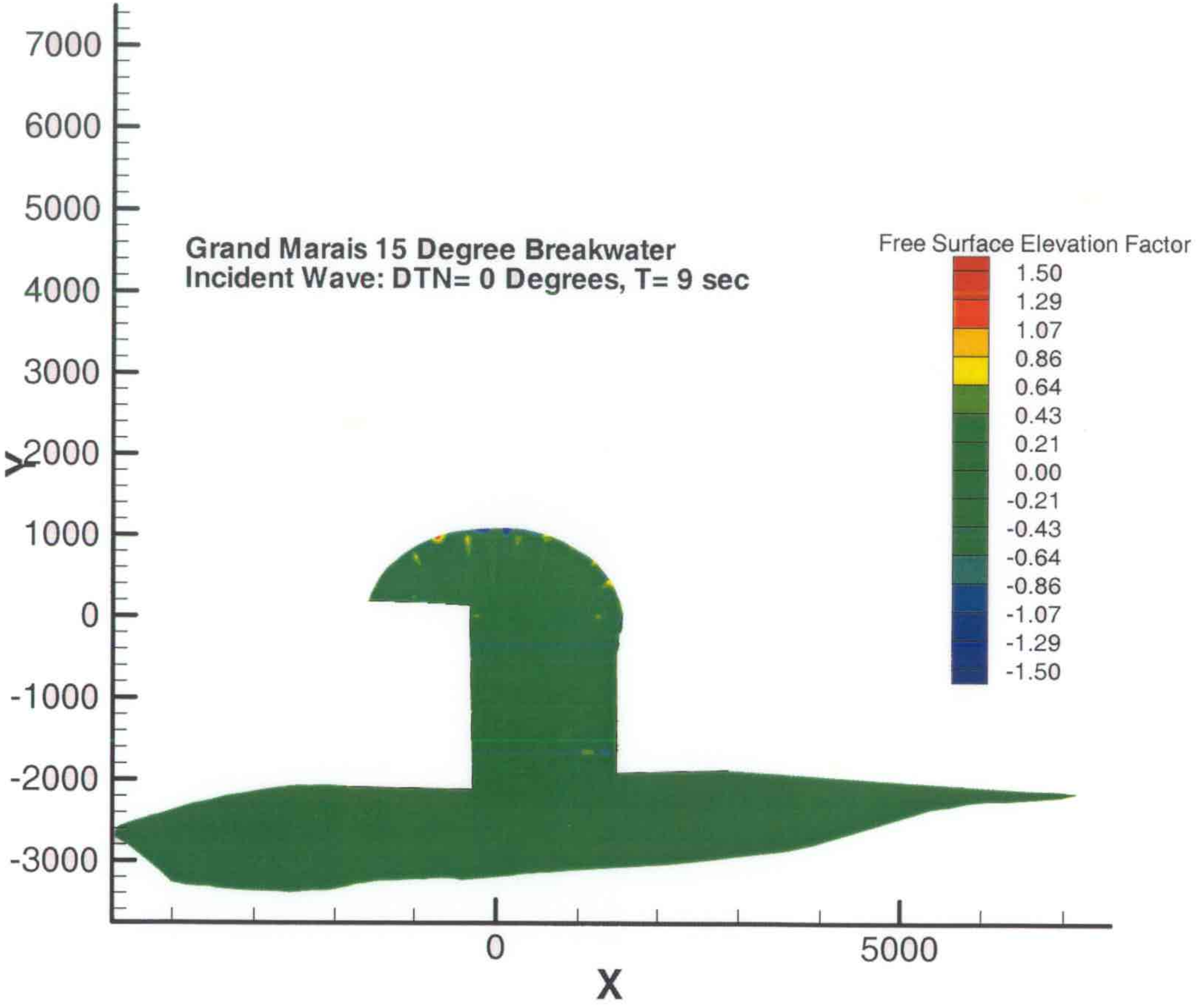

Grand Marais 0 Degree Breakwater
Incident Wave: DTN= 337.5 degrees, T= 8 sec
Free Surface Elevation Factor
1.50
1.29
1.07
0.86
0.64
0.43
0.21
0.00
-0.21
-0.43
-0.64
-0.86
-1.07
-1.29
-1.50
y
10000
9000
8000
7000
6000
5000
4000
3000
2000
1000
0
-1000
-2000
-3000
0
5000
10000


Grand Marais 0 Degree Breakwater
Incident Wave: DTN= 337.5 degrees T= 9 sec
Free Surface Elevation Factor
1.50
1.29
1.07
0.86
0.64
0.43
0.21
0.00
-0.21
-0.43
-0.64
-0.86
-1.07
-1.29
-1.50
y
10000
9000
8000
7000
6000
5000
4000
3000
2000
1000
0
-1000
-2000
-3000
0
5000
10000

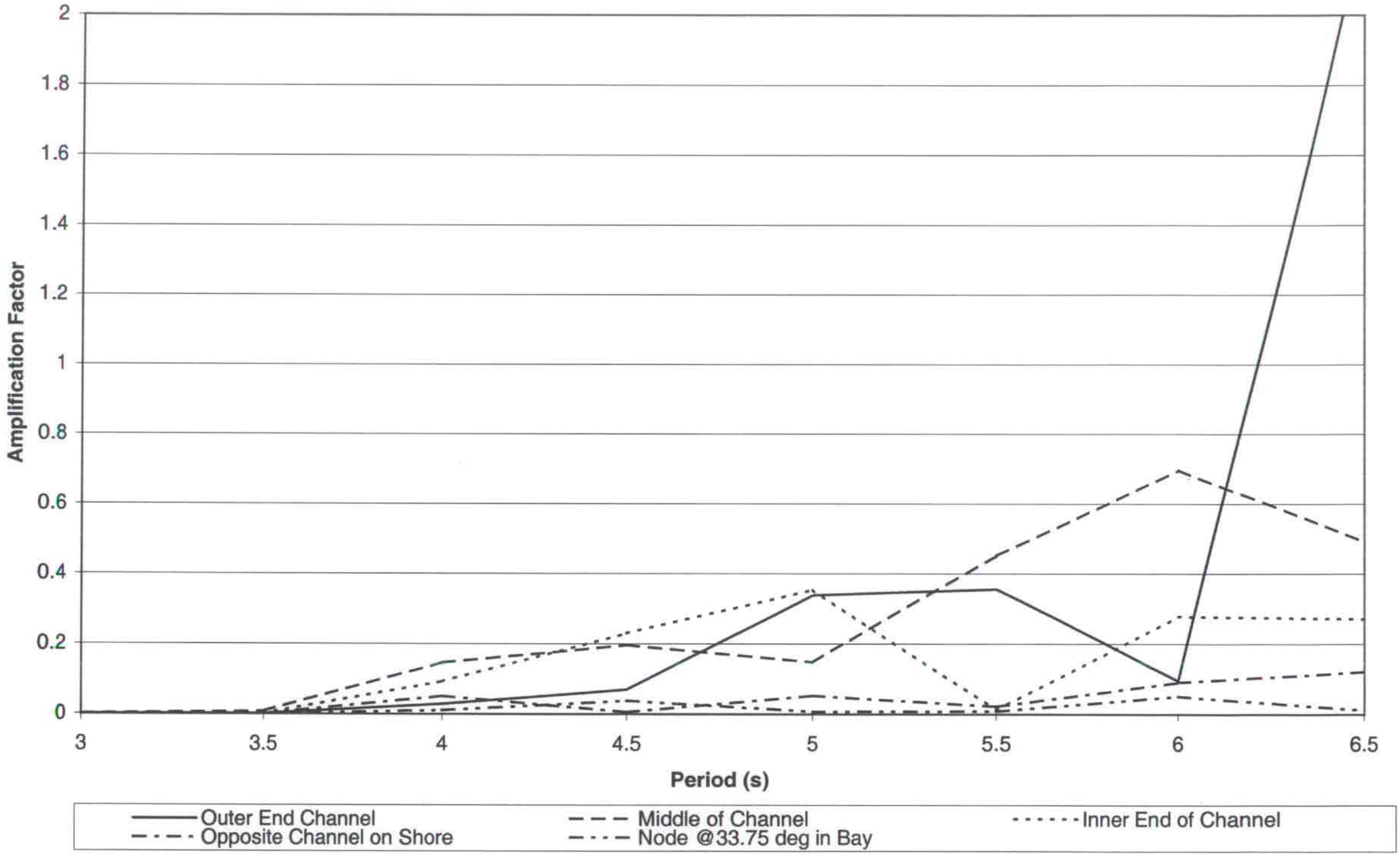


Grand Marais 15 Degree Breakwater
Incident Wave: DTN= 0 Degrees, T= 8 sec

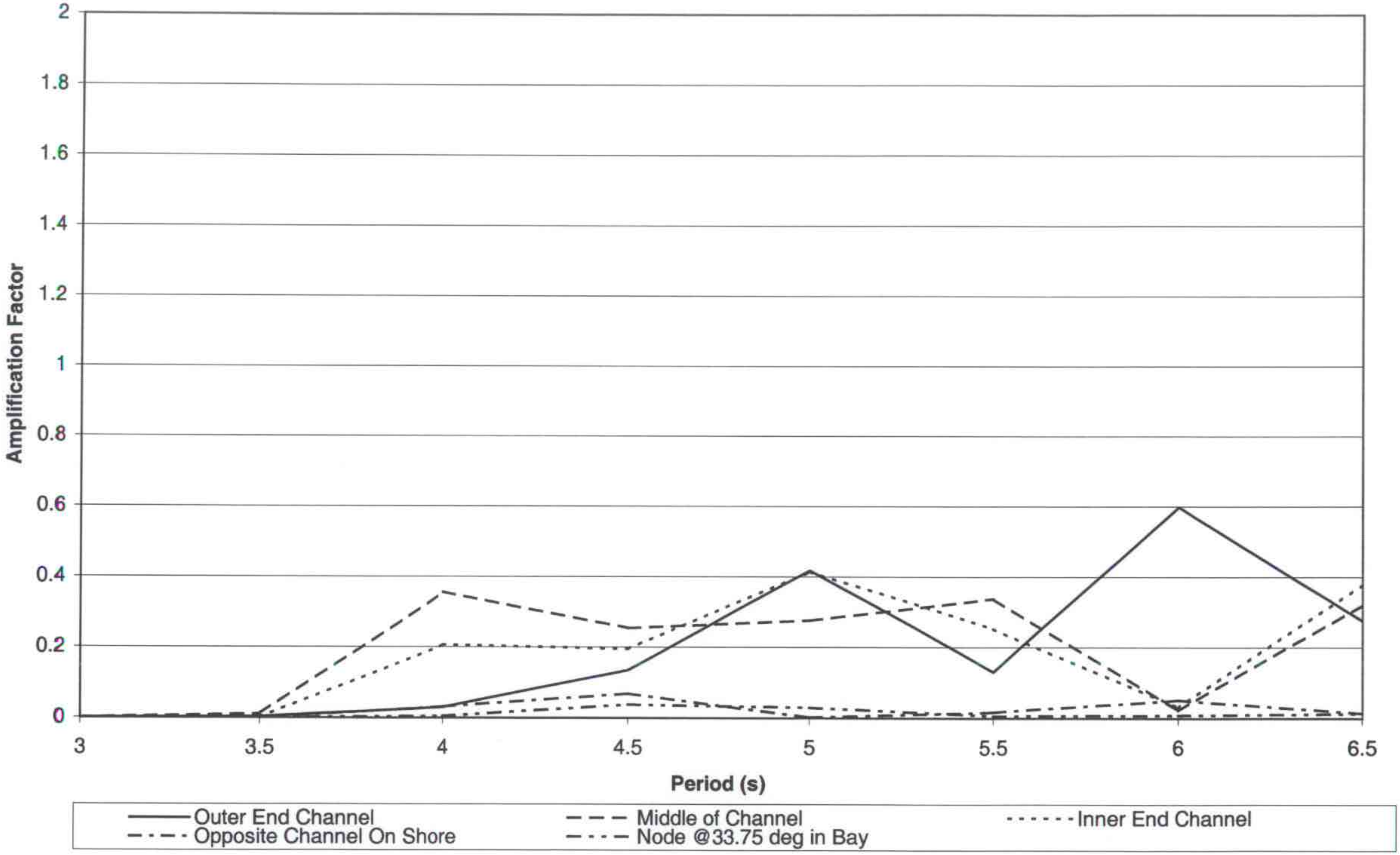

Grand Marais 15 Degree Breakwater
Incident Wave: DTN= 0 Degrees, T= 10 sec
Free Surface Elevation Factor
1.50
1.29
1.07
0.86
0.64
0.43
0.21
0.00
-0.21
-0.43
-0.64
-0.86
-1.07
-1.29
-1.50
Y
7000
6000
5000
4000
3000
2000
1000
0
-1000
-2000
-3000
X
0
5000


Grand Marais 15 Degree Breakwater
Incident Wave: DTN= 0 Degrees, T= 9 sec
Free Surface Elevation Factor
1.50
1.29
1.07
0.86
0.64
0.43
0.21
0.00
-0.21
-0.43
-0.64
-0.86
-1.07
-1.29
-1.50
7000
6000
5000
4000
3000
2000
1000
0
-1000
-2000
-3000
Y
0
5000
X

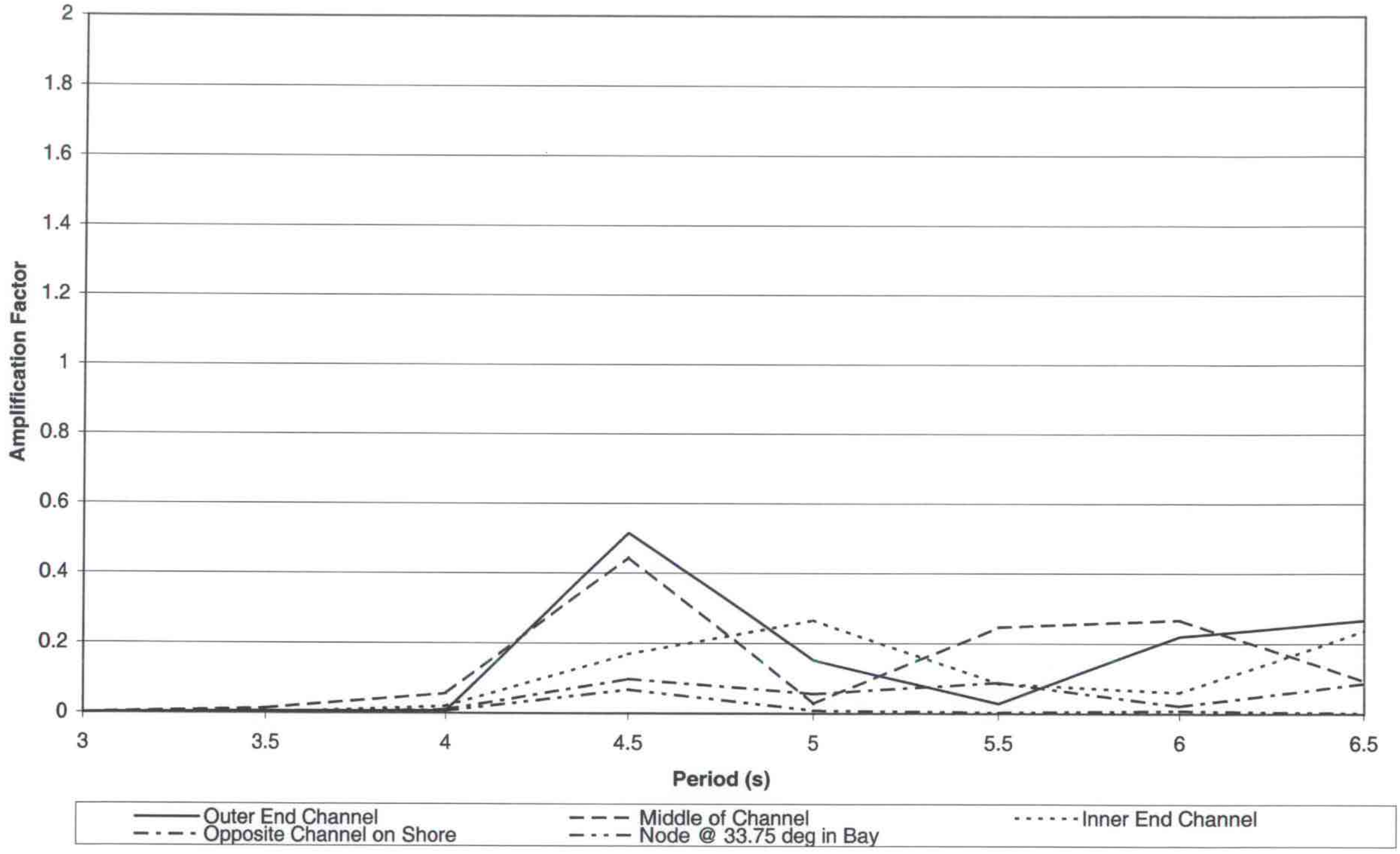
0 Deg breakwall - n67 Deg incident

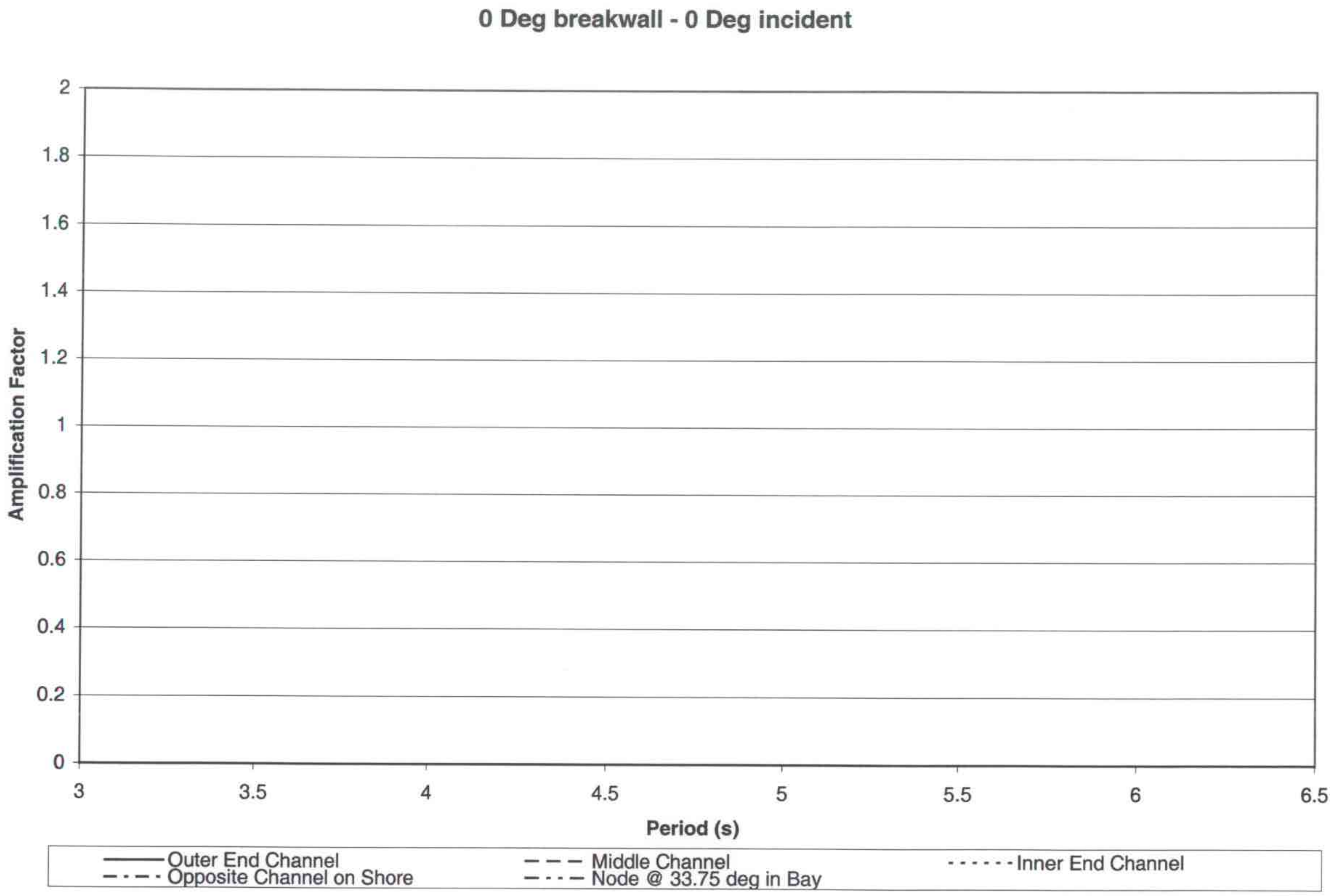


0 Deg breakwall - n45 Deg incident

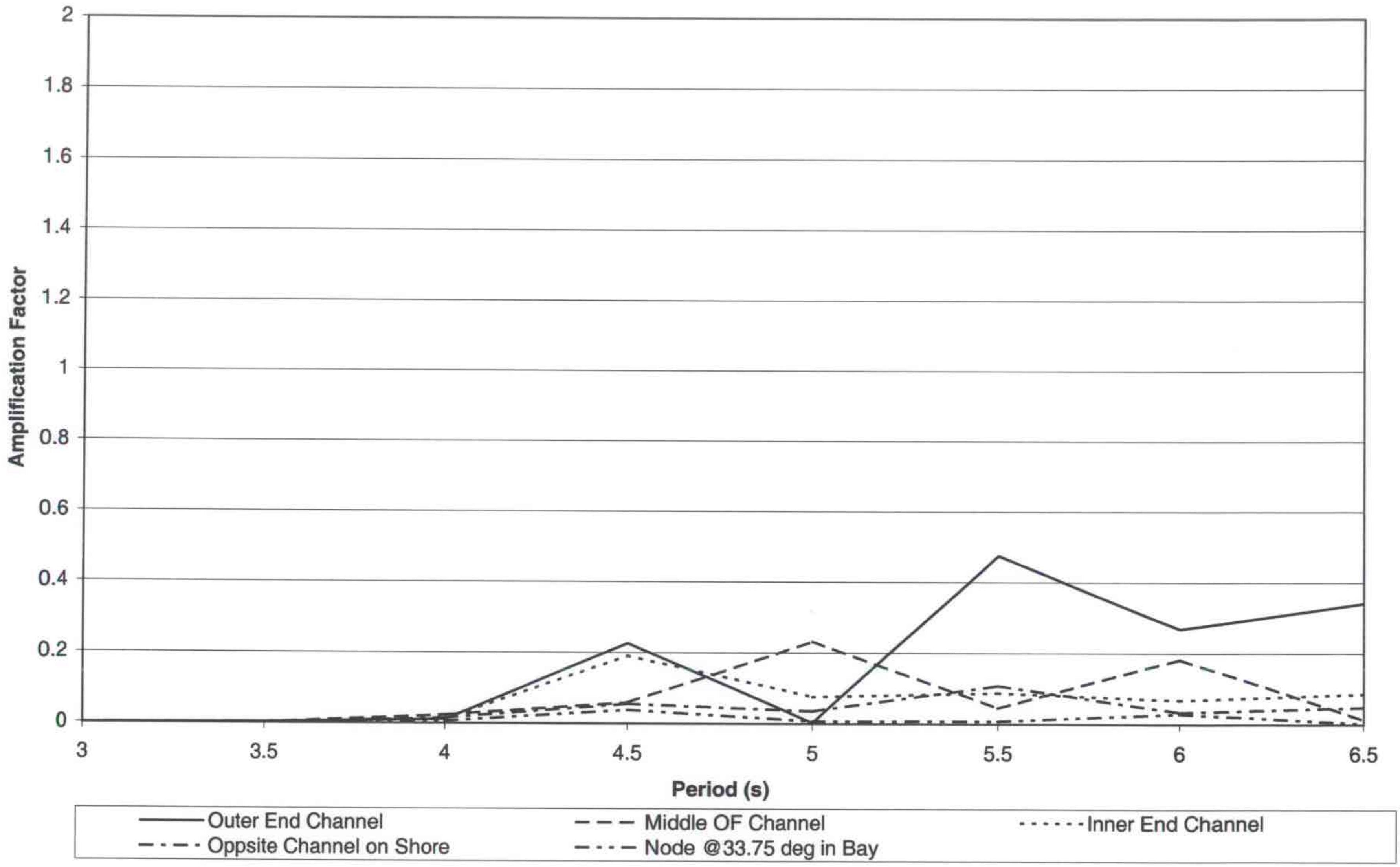


0 Deg breakwall - n22 Deg incident

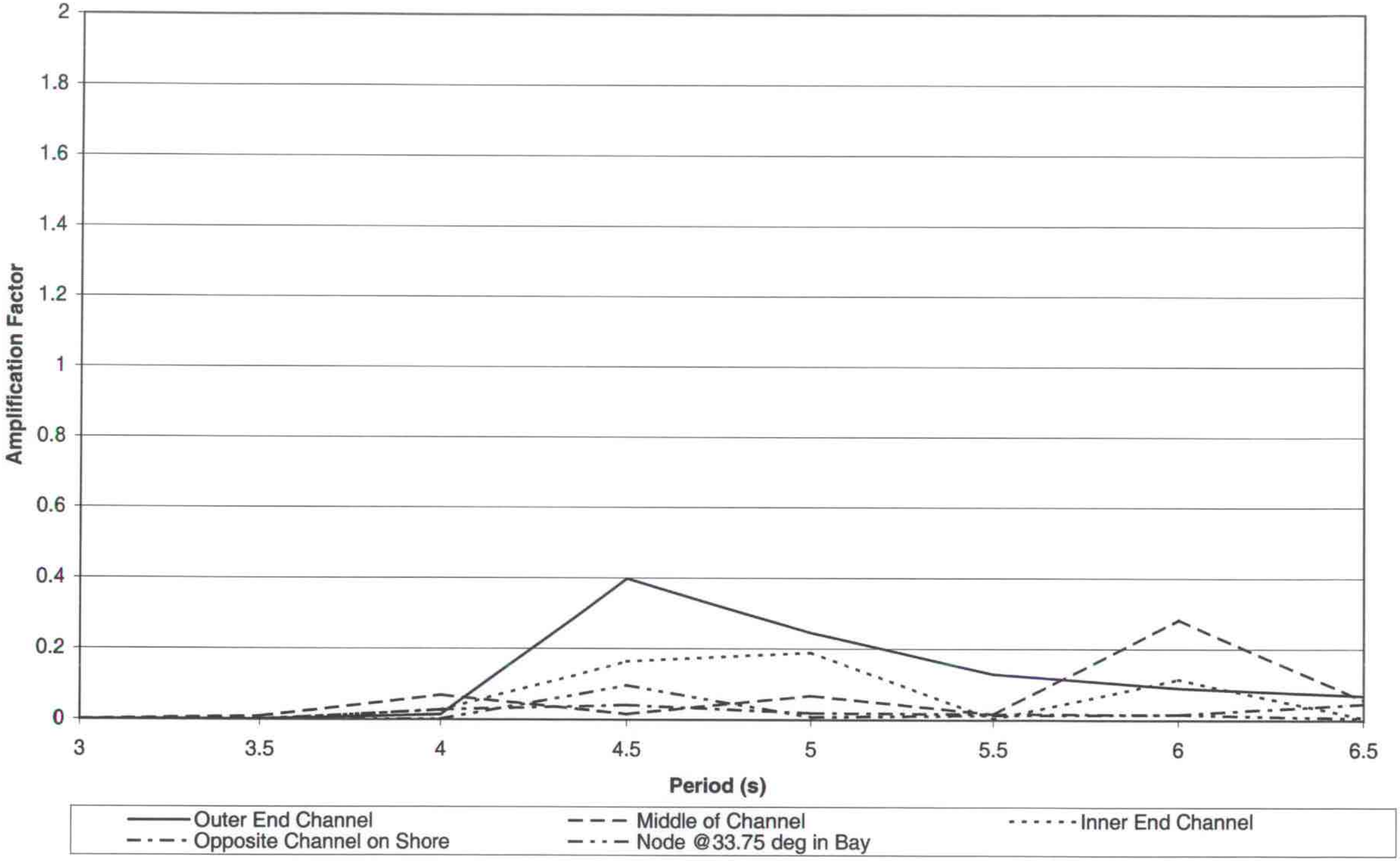



0 Deg breakwall - 0 Deg incident
Amplification Factor
2
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
3
3.5
4
4.5
5
5.5
6
6.5
Period (s)
Outer End Channel
Opposite Channel on Shore
Middle Channel
Node @ 33.75 deg in Bay
Inner End Channel

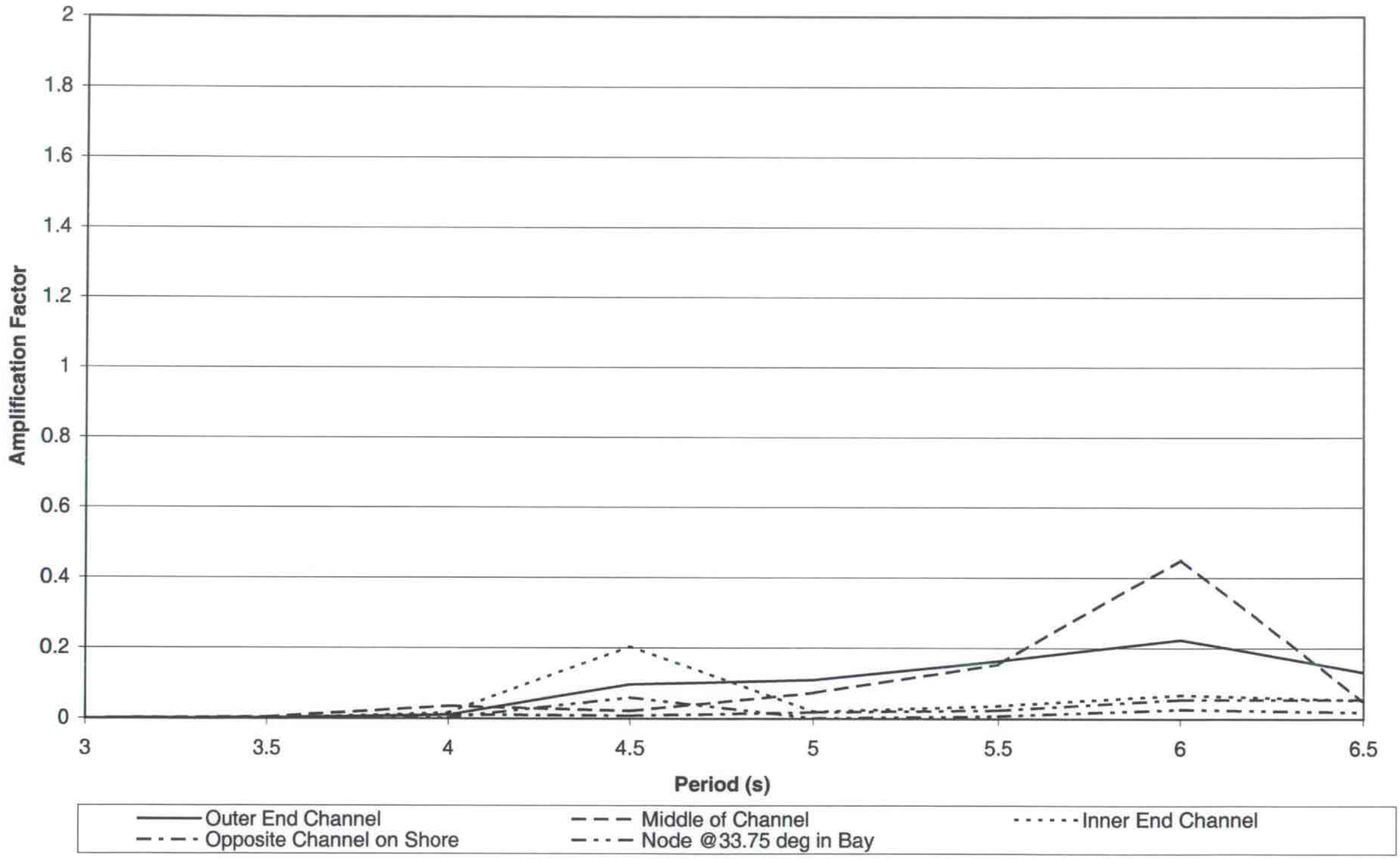
0 Deg breakwall - p22 Deg incident



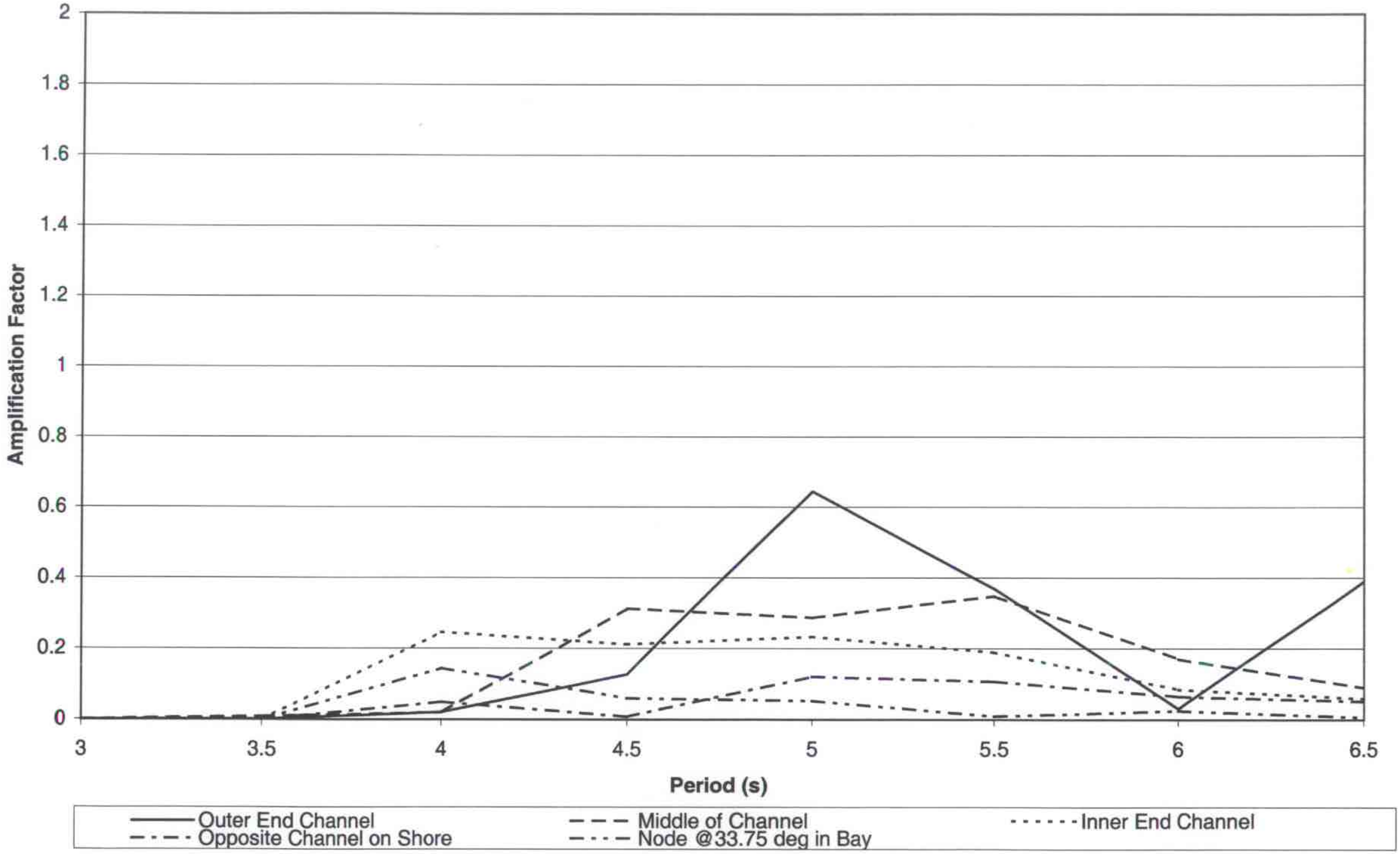
0 Deg Breakwall - p45 Deg Incident



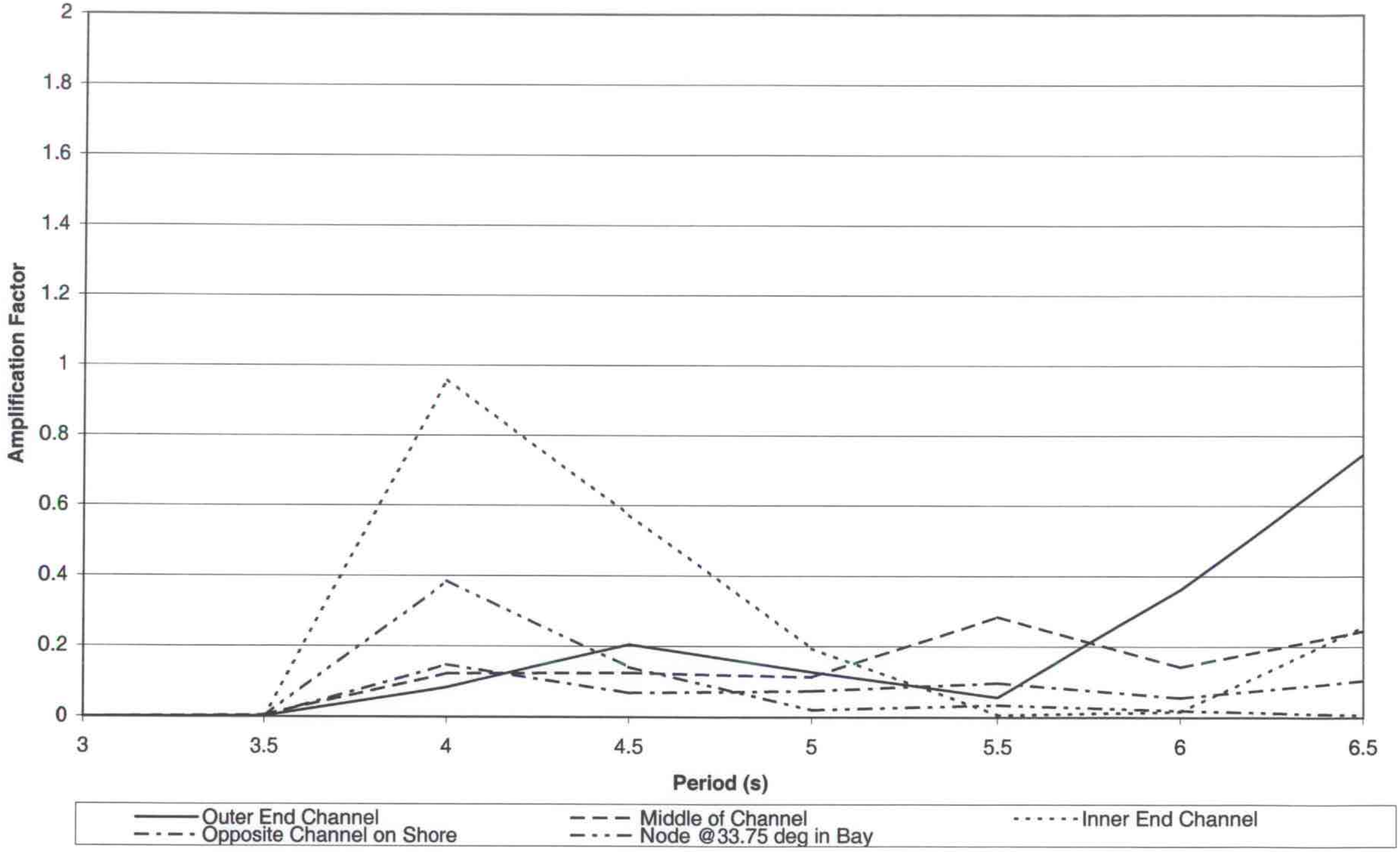
0 Deg Breakwall - p67 Deg Incident



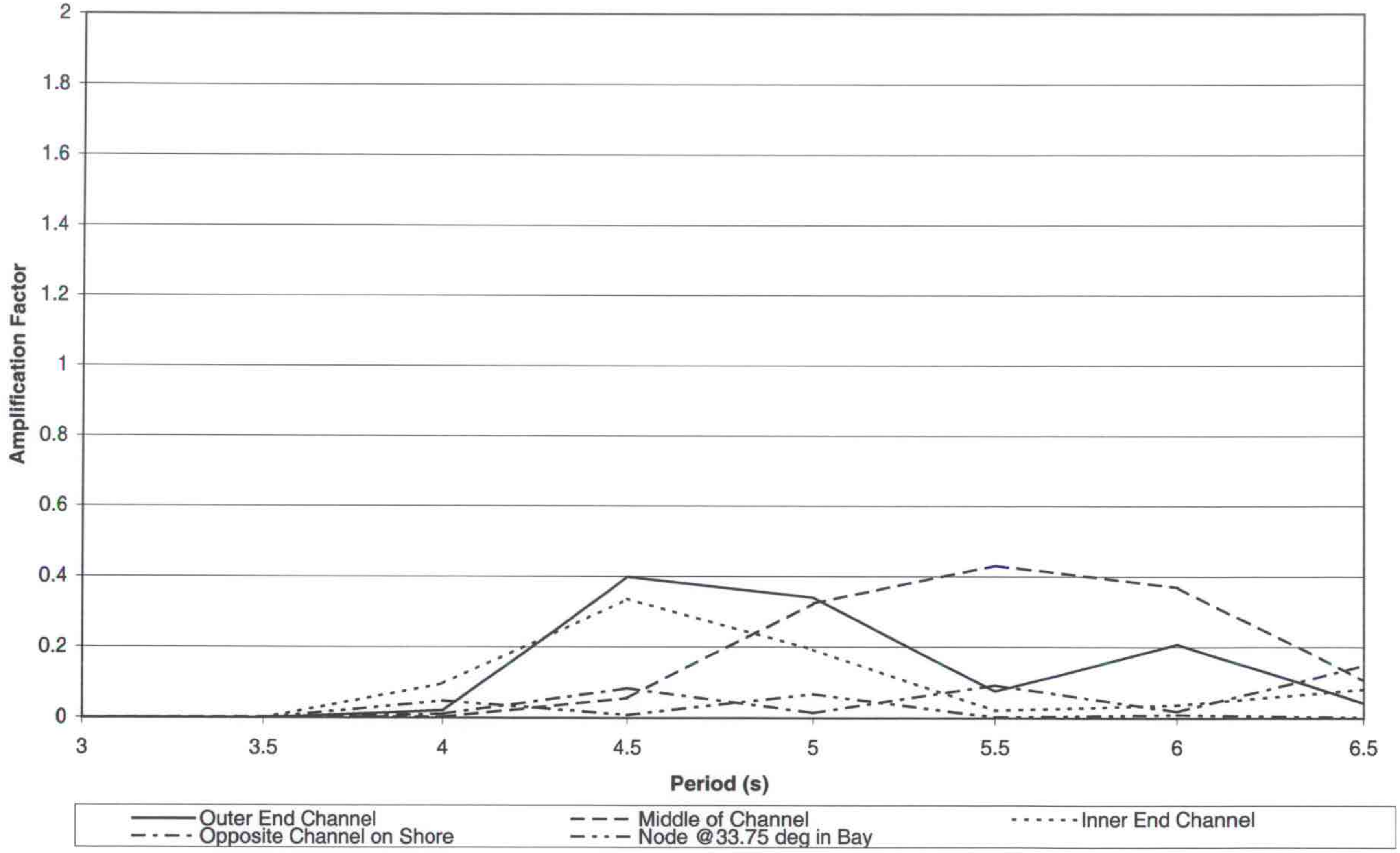
15 Deg Breakwall - n67 Deg Incident

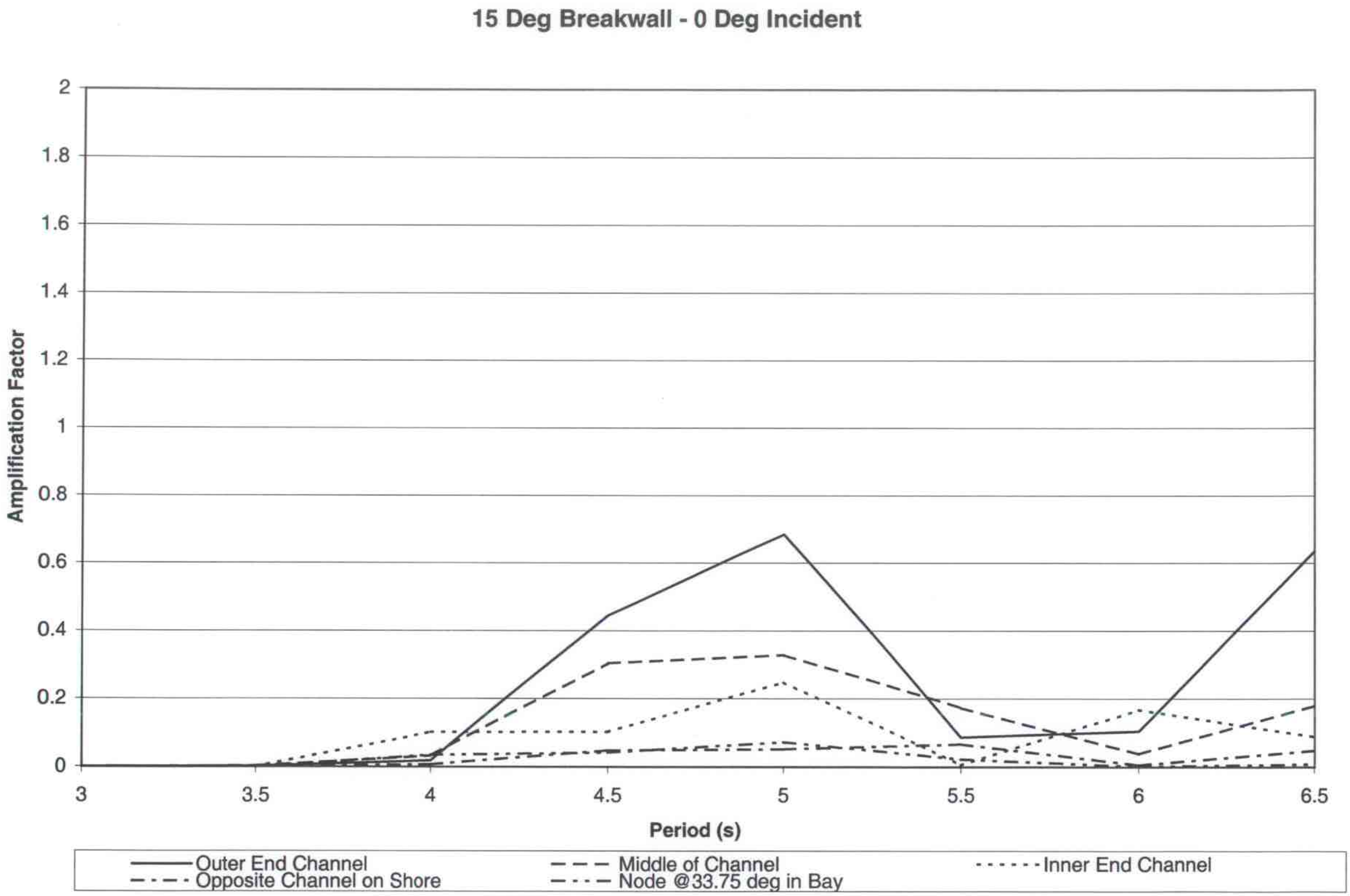


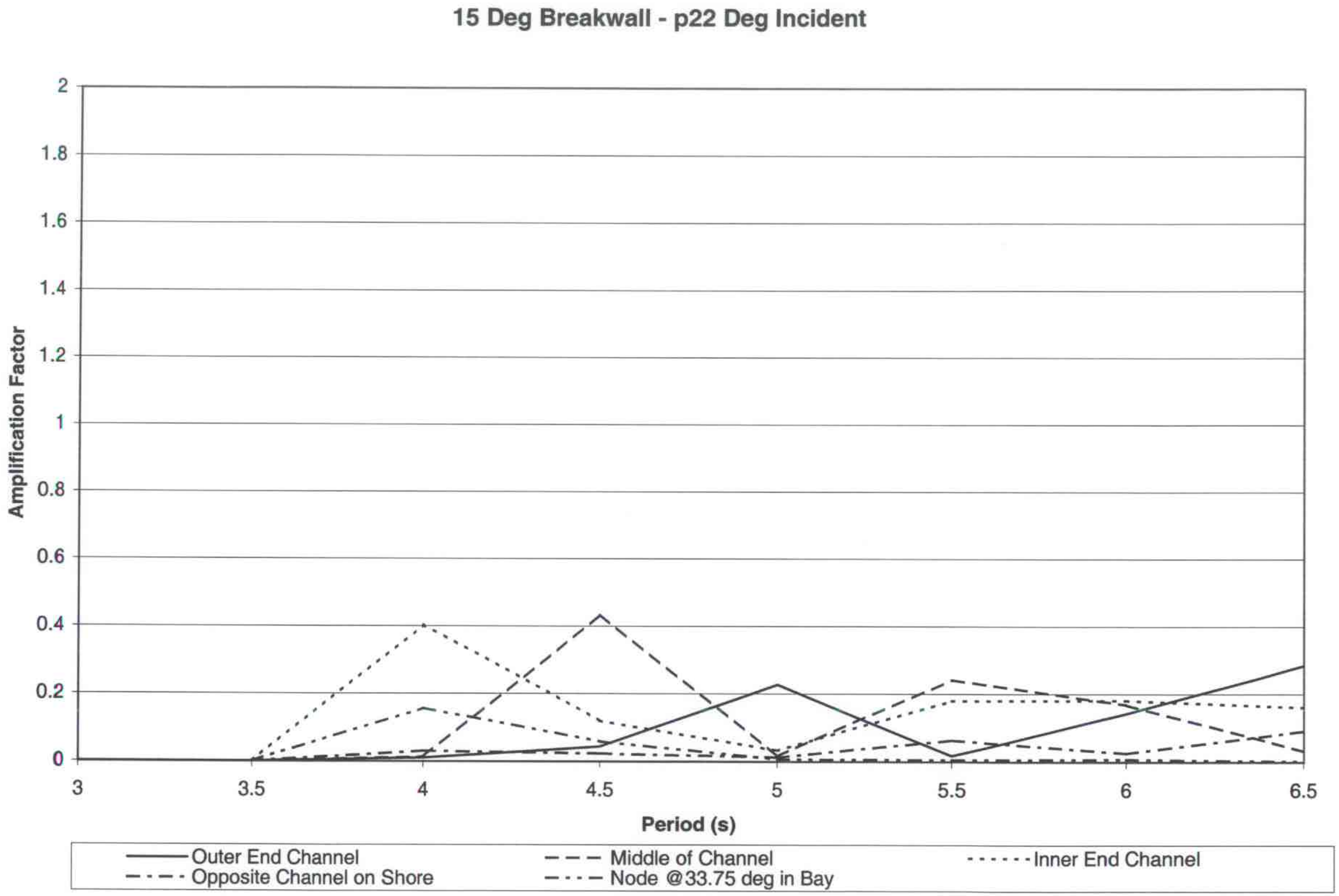
15 Deg Breakwall - n45 Deg Incident

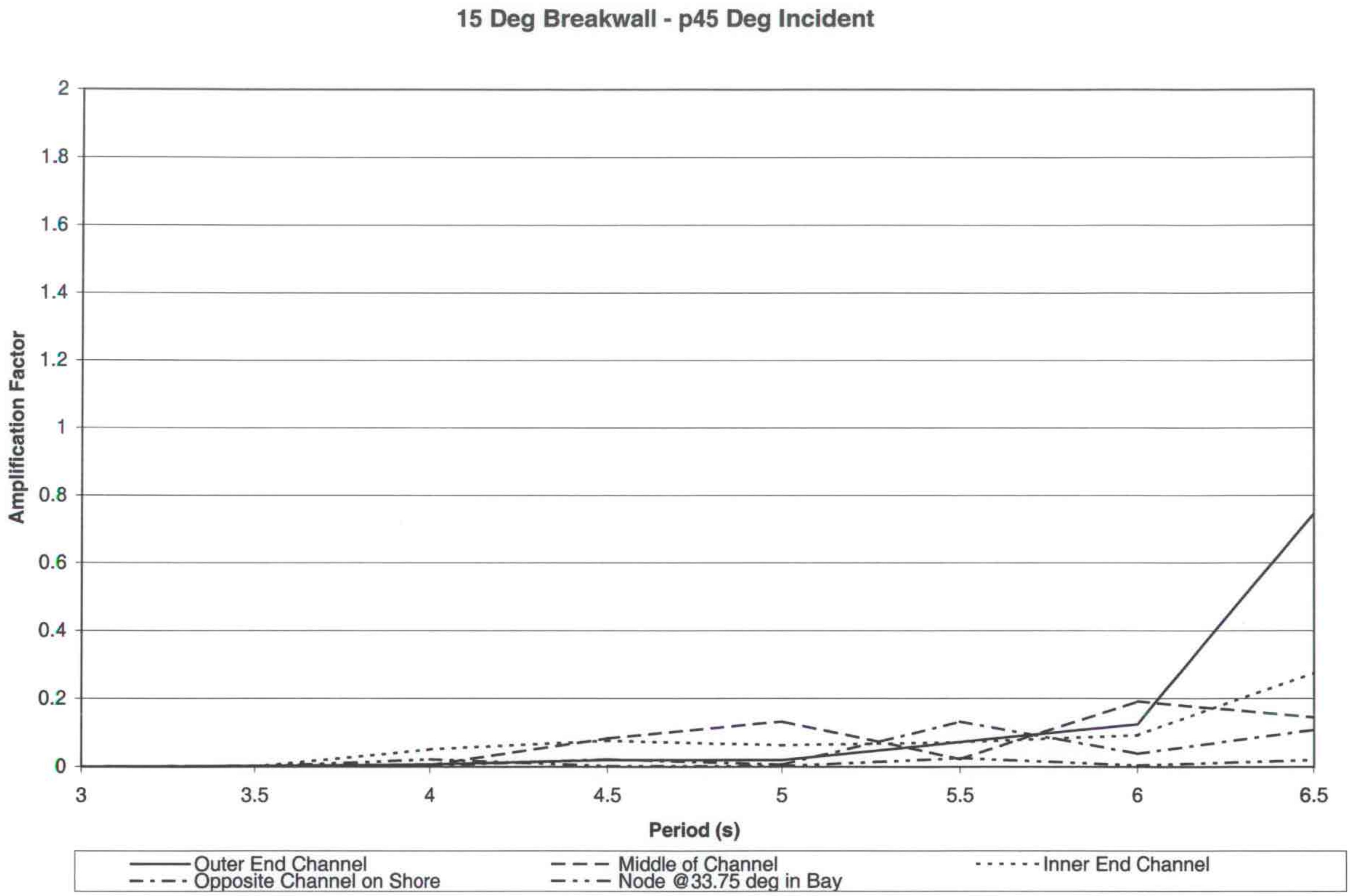


15 Deg Breakwall - n22 Deg Incident

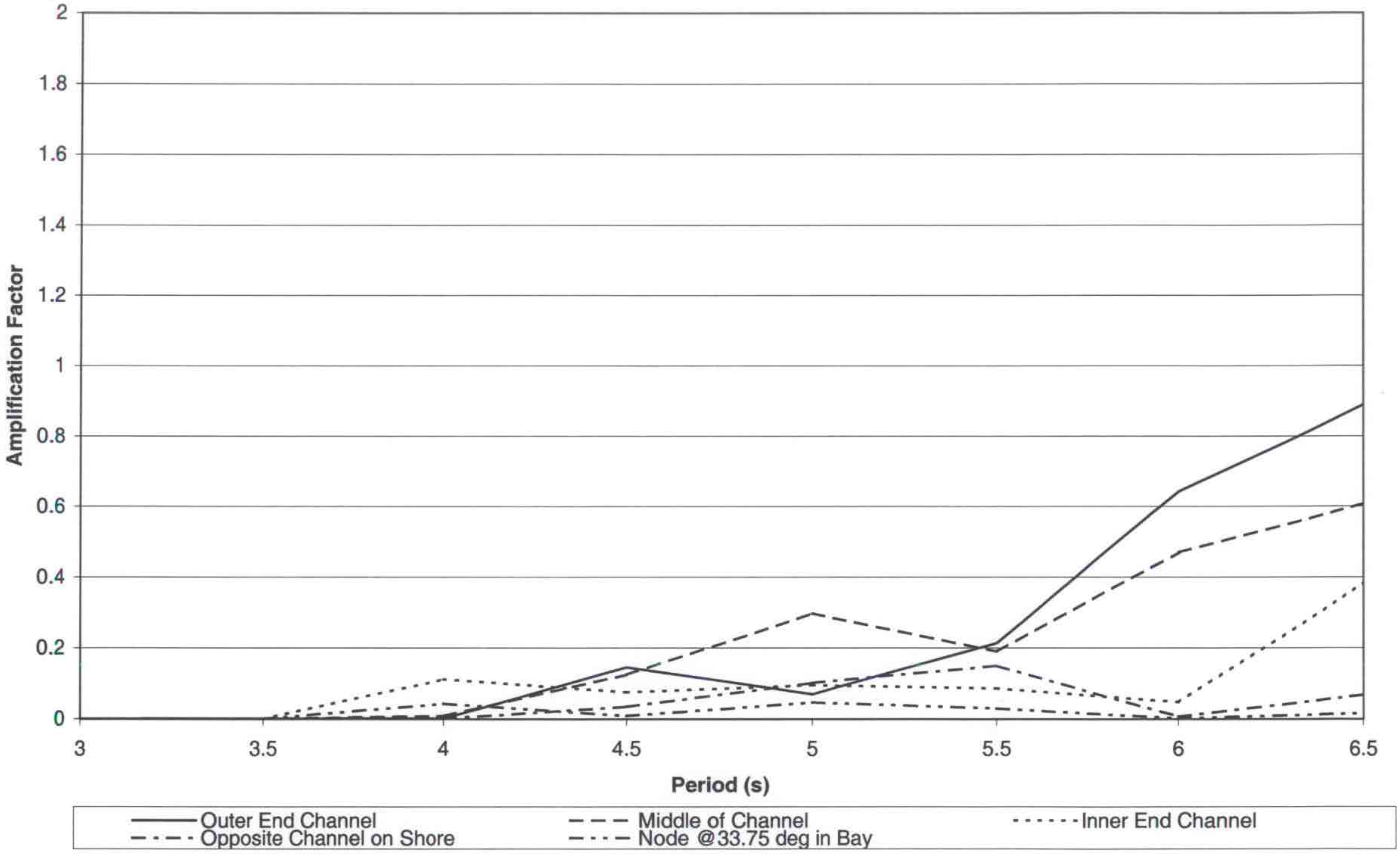



15 Deg Breakwall - 0 Deg Incident
Amplification Factor
2
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
3
3.5
4
4.5
5
5.5
6
6.5
Period (s)
Outer End Channel
Middle of Channel
Inner End Channel
Opposite Channel on Shore
Node @33.75 deg in Bay

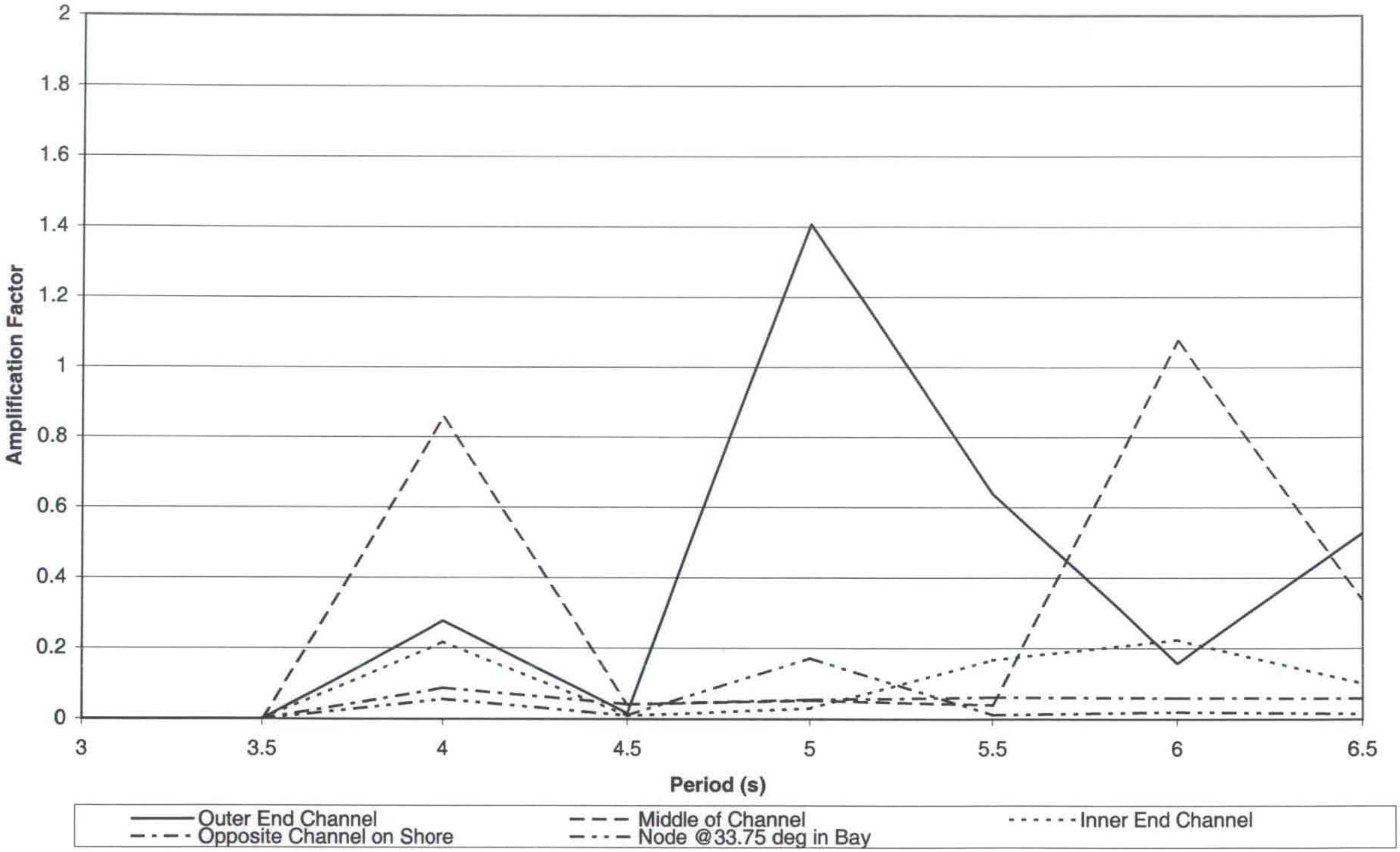

15 Deg Breakwall - p22 Deg Incident
Amplification Factor
2
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
3
3.5
4
4.5
5
5.5
6
6.5
Period (s)
Outer End Channel
Middle of Channel
Inner End Channel
Opposite Channel on Shore
Node @33.75 deg in Bay

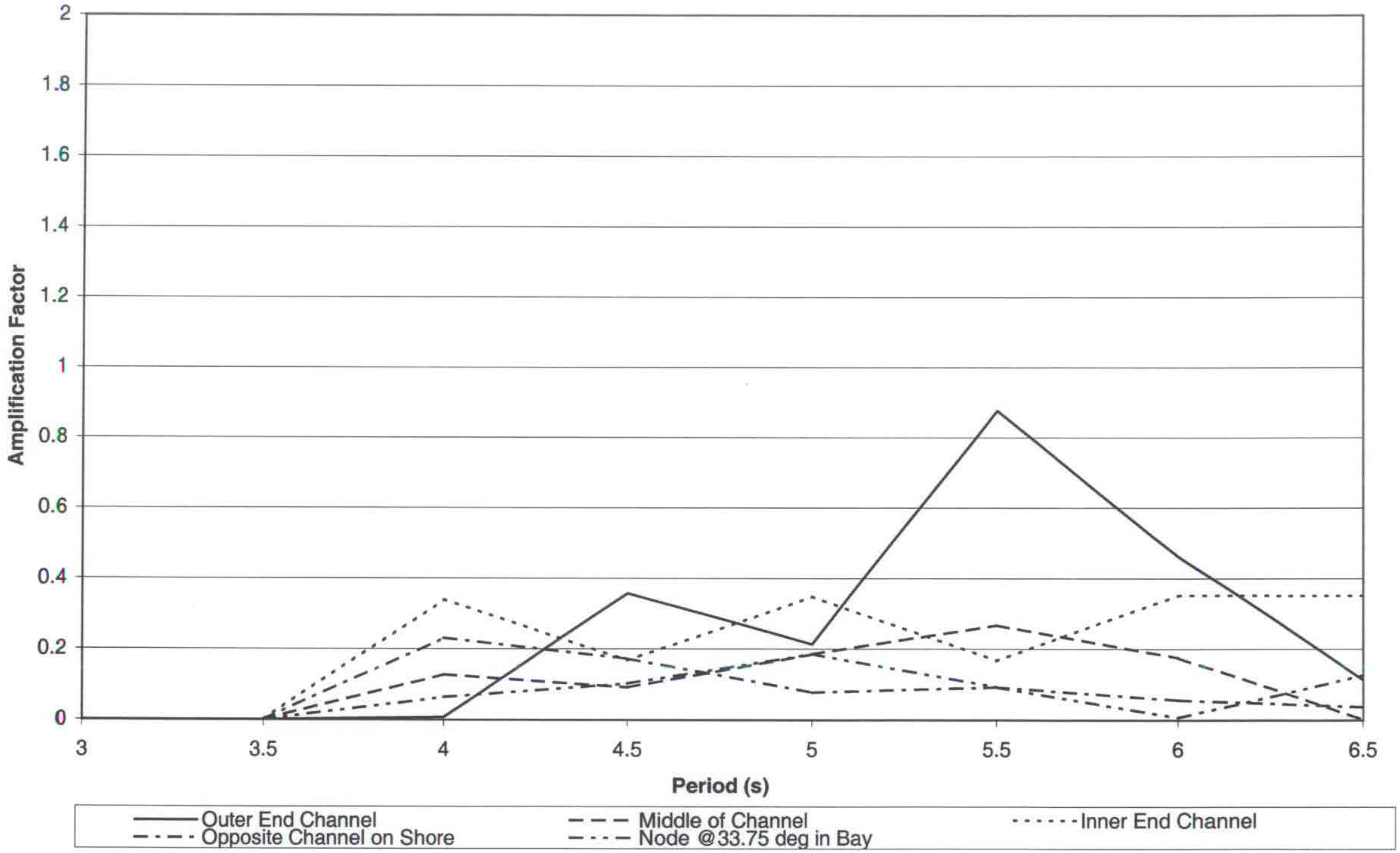

15 Deg Breakwall - p45 Deg Incident

15 Deg Breakwall - p67 Deg Incident

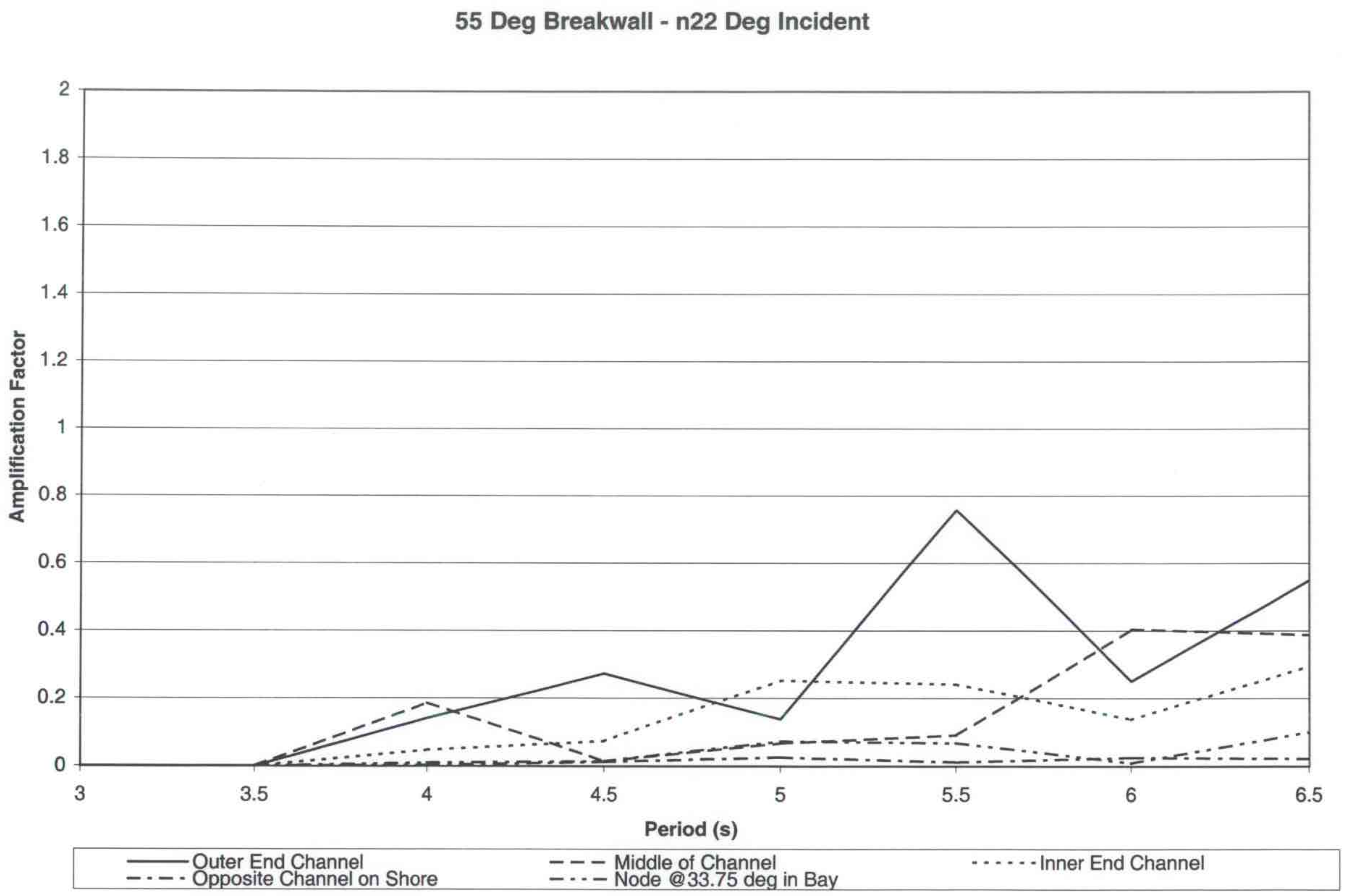


55 Deg Breakwall - n67 Deg Incident

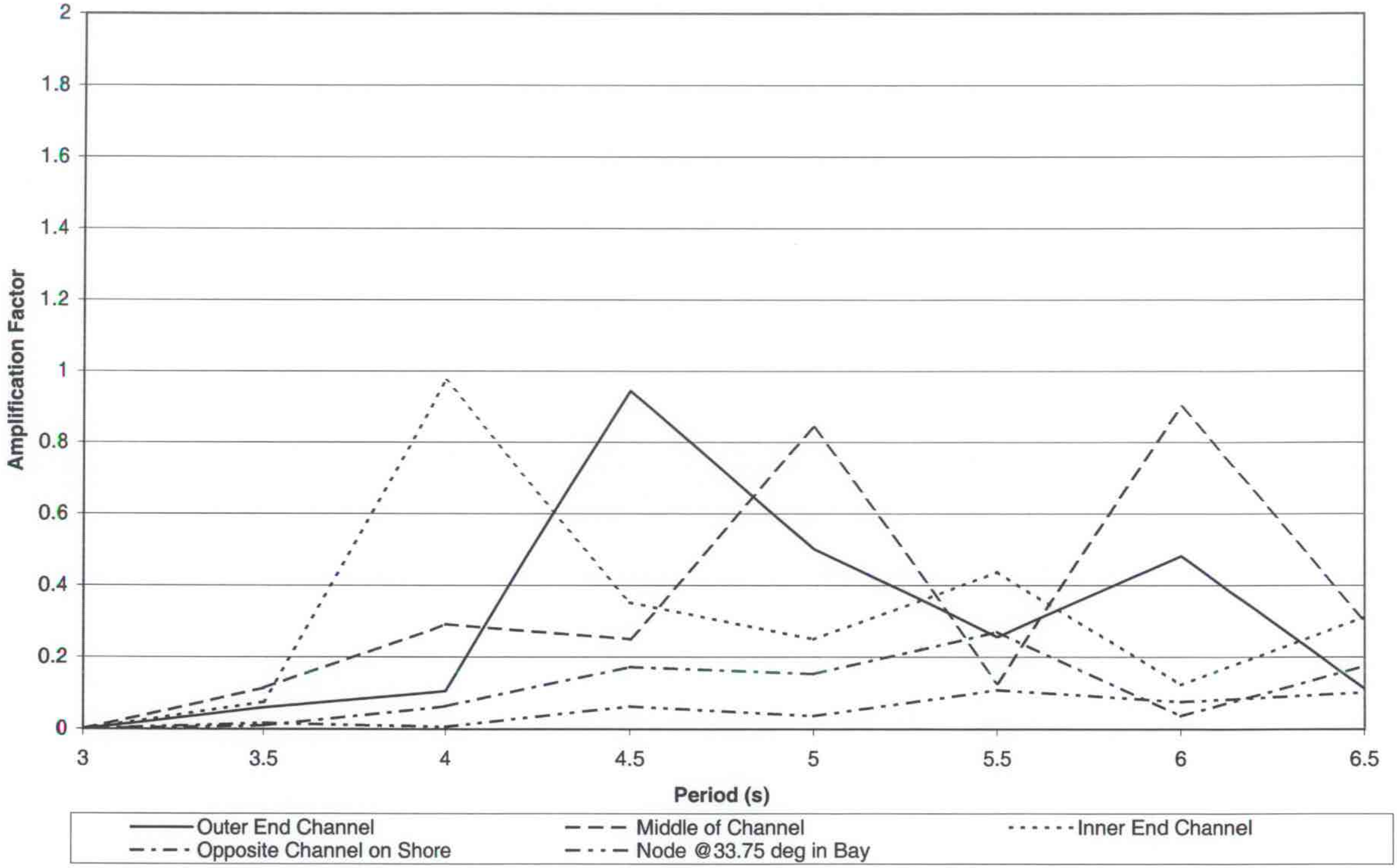


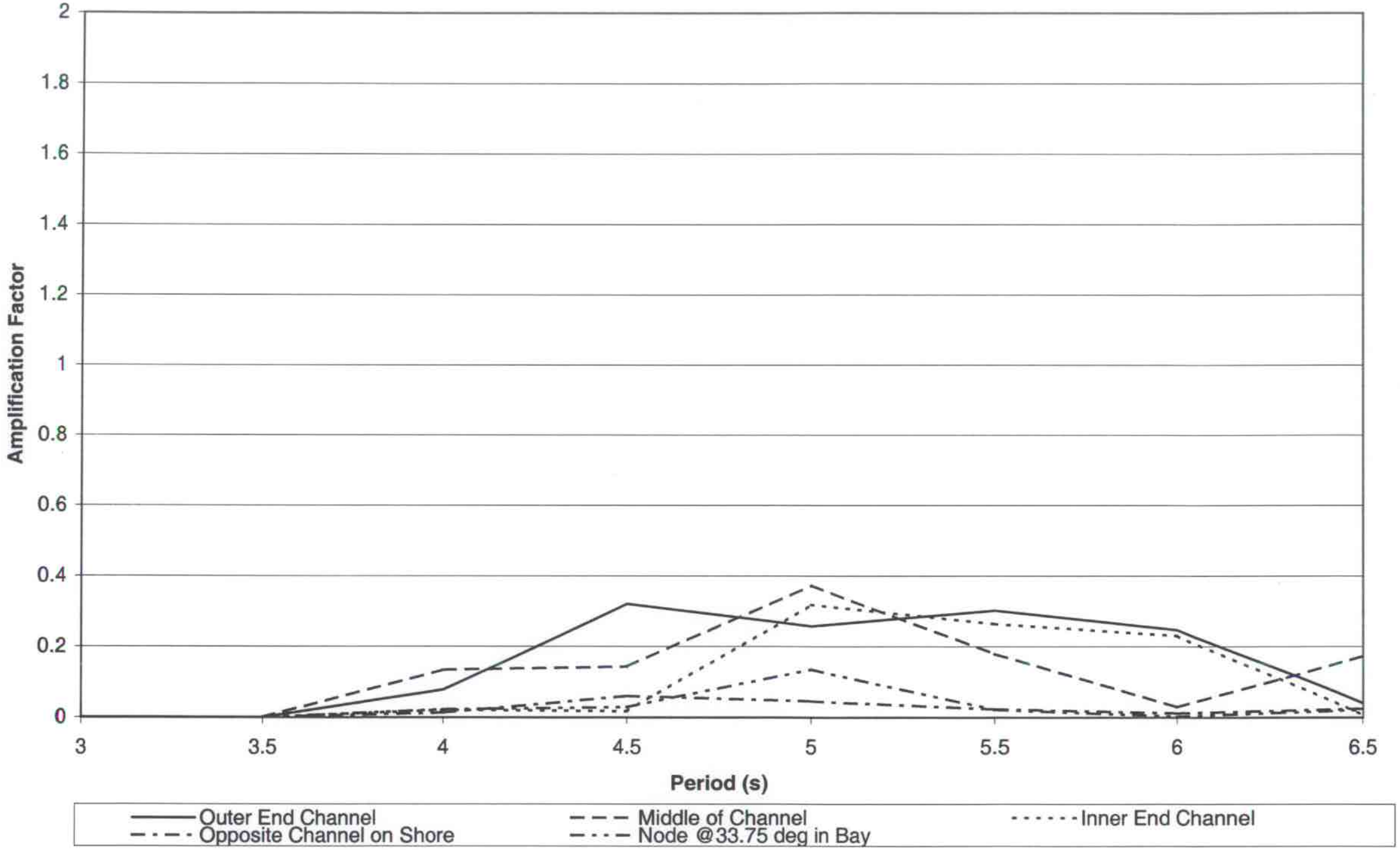


55 Deg Breakwall - n45 Deg Incident


55 Deg Breakwall - n22 Deg Incident
Amplification Factor
2
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
3
3.5
4
4.5
5
5.5
6
6.5
Period (s)
Outer End Channel
Middle of Channel
Inner End Channel
Opposite Channel on Shore
Node @33.75 deg in Bay

55 Deg breakwall - 0 Deg Incident





55 Deg Breakwall - p45 Deg Incident

55 Deg Breakwall - p67 Deg Incident