HYDRAULIC MODEL STUDY
Saint Joseph Harbor
Lighthouse Marina Project Investigation
Report CEE 04-03

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TABLE OF CONTENTS

EXECUTIVE SUMMARY

A 1:60 scale model of a portion of the St. Joseph Harbor was constructed in order to perform scale model tests to determine the response to waves generated on Lake Michigan. The model reproduced the geometry and the bathymetry of a significant portion the harbor including the entrance channel exiting to Lake Michigan. The purpose of the physical hydraulic model study was to determine whether three proposed options for the construction of the boundary between the proposed Lighthouse Marina and the existing navigation channel for the St. Joseph Harbor would result in an unacceptable increase in wave activity that would adversely impact use of the navigation channel. In order to make this determination, the existing harbor configuration was reproduced in the model as well as each of the three proposed options. Wave heights at selected locations were measured in the existing configuration as well as for each of the proposed modifications in order to determine changes in wave climate within the harbor associated with the implementation of the Lighthouse Marina project.

Wave observations within St. Joseph Harbor were made during several storms. These observations consisted of estimating water surface elevation changes at selected locations around the harbor perimeter. The model response for one of the wave configurations was compared to observations within the harbor during a storm on September 19, 2003 and consistent behavior was noted in the model and actual harbors.

Existing information on wave climatology was reviewed to select representative wave conditions to be used in the testing. Locations for measuring wave response were selected in consultation with the Detroit District of the U.S. Army Corps of Engineers. The primary testing was performed for lake levels that are consistent with existing depths in the navigation channel and the harbor response to incident waves with three different periods and two wave directions was measured at up to 14 different discrete locations. Additional, more limited testing was performed for some of the same wave conditions at four foot greater water levels and also for one of the options in conjunction with a proposed modification to the U.S. Coast Guard breakwater location across the navigation channel from the proposed Lighthouse Marina.

Based on several metrics that compare the wave response with the proposed modifications in place to the corresponding existing condition (i.e. at the same water depth, wave period and direction), the following conclusions are obtained:

- The various options tested indicate at most a less than ten percent increase wave activity within the harbor when assessed on the basis of an Overall Harbor Response which is basically a weighted average wave height ratio for all model measurement locations. Over half the individual configurations tested at the low water level condition actually indicate a decrease in wave activity when assessed by this metric;
- Any one of the three options should be acceptable considering the objective of not producing a significant increase in wave activity.

- Among the three options, Options 2 and 3 produced somewhat more favorable wave conditions than Option 1. No significant difference could be detected between Options 2 and 3.
- In terms of individual locations within the harbor, a particular wave configuration may indicate a significant increase in wave height but other wave configurations may then demonstrate a decrease or only nominal increase in wave height. Based on these findings, there is no indication that adverse wave conditions in the navigation channel will result as a consequence of the Lighthouse Marina development/
- Increasing the water level within the harbor by four feet did not produce a detectable increase in wave height ratios.
- A possible modification to the U.S. Coast Guard breakwater across the navigation channel from the Lighthouse Marina development was incorporated into the model and tested in conjunction with the proposed Option 3. The results from the two modifications are not materially different than alternatives that consider the implementation of only one of the alternatives. Therefore, the execution of both projects will not result in an increase in adverse wave conditions with the St. Joseph Harbor. A more limited set of measurements for one wave configuration only indicates that increasing the height of the U.S. Coast Guard breakwater will not adversely increase wave heights in the navigation channel.

INTRODUCTION

A modification to the St. Joseph River channel entrance into Lake Michigan is proposed as part of the proposed Lighthouse Marina development adjacent to the existing Waterfront Marina. The current channel entrance configuration consists of a pair of parallel jetties approximately 1750 feet long projecting into Lake Michigan at about a 280 degree azimuth. There is a change in jetty orientation and narrowing of the channel width roughly at the shoreline. The jetty on the south side of the channel entrance extends another 800 feet with steel sheet pile construction followed by a section that is recessed shoreward from the sheet pile walls. This area is composed of deteriorated structures with more recent armor (both stone and concrete blocks) that have been placed to prevent further erosion Figures 1-3 are images of the current state of this area. This area is where the Lighthouse Marina development is proposed and this area would be replaced by a marina separated from the main channel by a sheet pile wall fronted by armor stone to mitigate local wave activity. There is also a breakwater section on the opposite side of the channel protecting U.S. Coast Guard facilities. Modifications have also been proposed to this breakwater. The satellite photograph in Figure 4 provides an overview of the Saint Joseph Harbor area while a more detailed aerial photograph in Figure 5 indicates the area included in the model study.

Concerns have been raised regarding the possibility that modifications to the harbor will result in an increase in wave activity within the harbor entrance channel, specifically with regards to possible effects on small craft traversing the entrance channel. A physical model of the relevant portion of the harbor was constructed and tested to measure wave heights at selected locations within the harbor subject to incident waves of various periods and directions. Once the response of the existing harbor was quantified, additional testing was performed to examine changes in harbor response for three proposed alternatives associated with the Lighthouse Marina development for the same wave conditions. Finally, testing was conducted for a case in which a proposed alternative for the Lighthouse Marina development along with a proposed modification to the U.S. Coast Guard breakwater were both implemented in the model. This report presents a description of the testing program as well as the measurement results.

Figure 1. View of Lighthouse Marina area looking from about middle of project towards the east.

Figure 2. View of Lighthouse Marina area looking eastward from Corps of Engineers seawall towards the Waterfront Marina entrance.

Figure 3. View of Lighthouse Marina area looking westward towards Corps of Engineers seawall.

Figure 4. . Satellite Image of St. Joseph Harbor area.

Figure 5. Aerial Photograph of St. Joseph Harbor

GENERAL SYSTEM DETAIL

The channel between the two offshore jetties is aligned at an angle of about 280 degrees such that waves from just north of west can enter the harbor directly down the entrance channel. The northern jetty extends further offshore than the southern one, providing some protection from waves coming from a more northerly direction. The harbor entrance channel is approximately 315 ft wide at its offshore end and narrows down to a width of 250 ft onshore of the alignment change (Refer to Figure 5). This narrowing of the entrance channel along with the alignment change will result in a local increase in wave heights, particularly along the north jetty for waves arriving from the west, due to a combination of wave reflection and focusing. The entrance channel width increases to the north beyond the U.S. Coast Guard station, providing a mechanism for reduction in wave amplitudes as the waves diffract into the greater opening beyond that location. The breakwater in front of the Coast Guard station as well as the stone covered area across the entrance channel will also dissipate some wave energy. Further back into the harbor, there are other armor stone emplacements including:

- Armor stone in front of the sheet pile wall forming the boundary between the Waterfront Marina and the main channel;
- Armor stone along the edge of the railroad north of the main channel;
- Short breakwater sections on either side of the entrance to the West Basin Marina.

Each of these sections will serve to provide some level of wave energy dissipation. A general reduction in wave amplitude along the entrance channel is thus expected and will depend on the direction of the incident waves.

The following documents were obtained for defining the existing conditions in the harbor:

- 2002 USACE bathymetric drawings for the St. Joseph River entrance
- USACE design drawings for the USCG Station breakwater
- USACE design drawings for the harbor steel sheet pile sections
- Design drawings for the West Basin marina breakwaters
- Aerial images of the project area

The following were obtained by The Abonmarche Group for the modeling construction and calibration of the model:

- Bathymetric survey of the channel
- Numerous site inspections
- Land survey of proposed marina basin area

The entrance channel is maintained at navigational depths by periodic dredging by the U.S. Army Corps of Engineers. Channel depths on the order of 22 to 25 feet relative to low water datum (577.5 ft, 1985 International Great Lakes Datum) currently exist throughout the navigational channel. Water depths are significantly decreased away from the navigational channel and are on the order of 6-8 feet in the vicinity of the entrances to the Waterfront and the West Basin Marinas. A bathymetric survey was performed by ACI in August 2003, and providing updated information on the existing water depths in the project area.

The St. Joseph River flows through the harbor and out the entrance channel into Lake Michigan. Data on river discharge statistics is available through the U.S. Geological Survey primarily through the gauging station at Niles, Michigan (Station 04101500) with more limited information available at the river mouth (Station 04102533). Discharge data from the second station is only available for a nineteen month period in 1994 and 1995 with a more extensive record available for the first station. The long term average discharge for the Niles gauging station is 3447 cubic feet per second (cfs) as reported in USGS (2002). The ratio between the flows at St. Joseph and Niles for the nineteen month period of common record is 1.30 yielding an estimate for a long term average flow rate through the channel entrance of 4500 cfs. With a channel width of approximately 250 feet and a water depth of 25 ft, this discharge will produce a mean current on the order of 0.7 ft/s, generally much less than wave induced horizontal velocities.

Two different projects in the harbor have been proposed. One involves completing a condominium/small craft marina project on the south side of the entrance channel immediately to the east of the current Corps of Engineers sheet pile wall. This project basically calls for placing a marina in the area where the current dilapidated structure and armor stone is located in Figs. 1-3. This would involve an extension of the sheet piling from the current wall eastwards to the entrance to the Waterfront Marina. The second proposed project involves some modification to the U.S. Coast Guard breakwater in an attempt to reduce wave activity in the area of their berthing slips. In general, an extension of the existing breakwater is the alternative being considered.

WIND AND WAVE CLIMATOLOGY

In order to accurately characterize the nearshore climatology, long-term statistics were reviewed for defining representative wind and wave data at the project location. Summarized wind and wave data for Lake Michigan from NOAA buoy 45007, known or recorded observations within the harbor, and the Benton Harbor airport weather station have provided the basis for characterizing the site climatology.

The USACE Coastal Engineering Research Center Wave Information Study for the Great Lakes (Hubertz et al., 1991) provides the most reliable and up-to-date summary of wave statistics for Lake Michigan. This data is based on a 32-year hind-cast of conditions analyzed from meteorological observations at various sites around Lake Michigan. The annual statistics provide the mean wave height and period, maximum wave height, and percent frequency of occurrence. The information is presented in two formats, one with wave statistics through forty-five degree angle sectors and a second with the same data resolved into 22.5 degree sectors. Table 1 summarizes the 32-year hind-cast conditions for 22.5 degree sectors for the St. Joseph area (corresponding to Station 59 in the study). Figure 6 presents the wave direction percentage of occurrence and the mean significant wave height direction for 45 degree sectors. The percent of occurrence of events for each direction is represented on the outside of the circle in the small "pie" pieces. The concentric circles indicate the percentile of wave occurrence. Based on Figure 6, the majority of the waves come from the southwest at 225 degrees, the occurrence frequency of waves coming from that direction is 22% (through the 45 degree sector centered on 225 degrees) , distributed as shown in Table 2. The larger events represent only ten percent of waves, with heights equal to or larger than 5 ft. In addition, from all directions, the mean wave height will be 2.6 ft. and the mean peak period will be 3.9 s. The highest waves will occur during the months of November through February with a mean height of 3.28 ft, with possible monthly highest waves of 12 ft and periods associated as large as 10s. The harbor will not normally be in use by small craft during those time periods.

Table 1. Wave Statistics for St. Joseph Area

Wave Direction (degrees)	Mean Wave Significant Height (m)	Largest Mean Wave Significant Height (m)	Mean Peak Wave Period (s)	Frequency of Occurrence (%)
0	0.8	4.6	4.4	6.01
22.5	0.6	2.7	4	3.34
45	0.6	2.1	3.8	3.98
67.5	0.5	2	3.4	3.53
90	0.4	1.6	3.1	4.58
112.5	0.5	1.8	3	3.93
135	0.5	1.5	3.1	3.65
157.5	0.5	1.5	3.1	3.07
180	0.4	1.9	3	3.71
202.5	0.6	2.8	3.3	7.80
225	0.8	4.2	4	14.16
247.5	0.9	4.1	4.2	8.98
270	0.9	4.9	4.3	8.42
292.5	1	4.9	4.4	9.27
315	1.2	6.3	4.8	9.51
337.5	1	6.1	4.7	6.03
Largest Wave Period (s)	**Most Frequent Direction (Deg.)**	**All Directions Mean Wave Significant Height (m)**	**Largest Wave Significant Height (m)**	**All Directions Mean Peak Wave Period (s)**
10	225	0.8	6.3	3.9

Table 2. Wave height frequency distributions for waves propagating from 45 degree sector centered on 225 degrees. Station M59 Lake Michigan.

Percent of occurrence	Wave Height (ft)
35	0 - 1.3
35	1.6 – 3.0
20	3.26 - 4.60
8	4.92 - 6.23
2	6.56 - greater

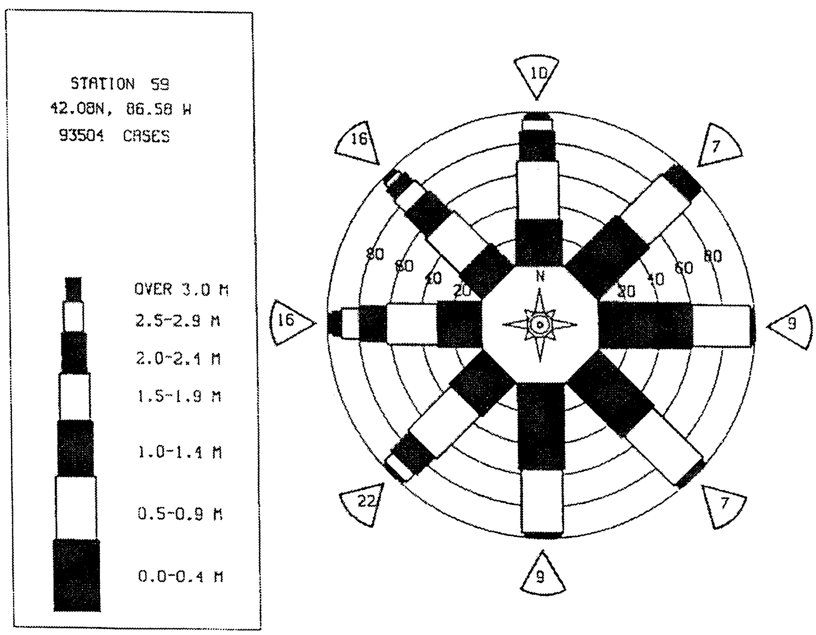

Figure 6. Wave Rose Diagram for Station M59, Saint Joseph Harbor vicinity (from WIS, 1991).

As can be seen from the table data and Figure 6, the most frequent direction of the waves is from the south and west quadrants, between 180 and 270 degrees (approximately 35 % of the total). The largest frequency of occurrence is recorded for the 225 degree direction (accounting for approximately 13% of the total if resolved into 22.5 degree sectors). The larger wave heights are produced by the waves from the north and northwest quadrants, between 270 and 360 degrees (approximately 30% of the total), as the fetch is the largest from these quadrants.

The current orientation of the harbor protects the boats inside the harbor against some of the destructive wave action for waves arriving from the northwest. However, it is expected that waves arriving from the southwest to west directions may result in a significant amount of wave energy entering the harbor.

The NOAA buoy station 45007 located in Lake Michigan provided both wind and wave data for the study. This buoy has long-term recorded data for the wind direction and speed, wave characteristics, and other parameters including air and water temperature. It was determined that the wind speed is greatest during the months of November and December, see Figure 7. The average annual wind speed is 10.3 knots, while the recorded annual speed for the Benton Harbor airport weather station is 10.15 knots. The maximum recorded wind speed is 41.2 knots, with a maximum peak gust of 53.8 knots. The monthly wind averages have a predominant direction from the south and west quadrants. The water waves propagate to the shoreline from these directions approximately 43% of the time, so the correspondence between frequency of wave and wind directions is fairly good.

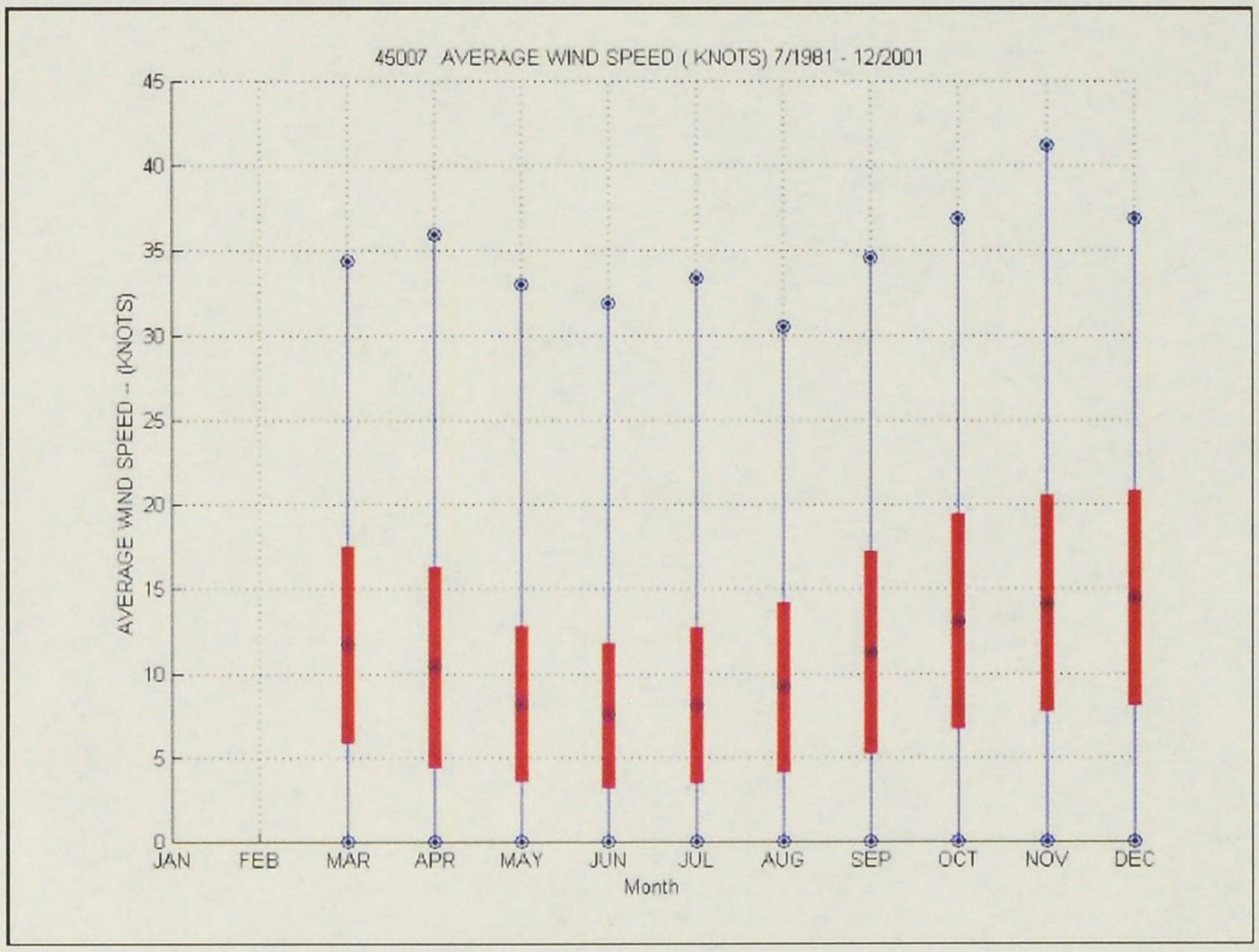

Figure 7. Long-Term Monthly Wind Speed (Buoy 45007)

Three significant storm events were observed in the harbor area by Dan Veriotti of Abonmarche: September 19, October 15, and November 13, 2003. The wave heights at various locations of the harbor were estimated based on visual observations, photographs taken and videotaping performed during the storm events. The NOAA buoy data provided the deep-water significant wave heights and dominant periods, wind direction and speed. The Benton Harbor airport weather station data recording wind direction and speed were also checked for accuracy of the buoy data.

Table 3 presents a summary of the recorded storm events data. Observations are summarized at 7 locations: entrance channel (1), mid-section between entrance channel and USCG Station (2), in the vicinity of the USCG Station (3), at the proposed marina location (4), at the West Basin marina entrance (5), at the Waterfront marina entrance (6), and at the railroad crossing portion of the channel (7). The wave heights reported are representative of the majority of the wave heights propagating in the channel at the specified locations. Where possible, scaled markings (lines) were painted on vertical surfaces such as steel sheet piling prior to the storms for a better scaling of the wave heights. The general trend is for wave heights to decrease from heights on the order of 6-8 feet near the offshore entrance to the harbor entrance channel to heights of only 1-2 feet farther back in the harbor.

Table 3. Recorded Storm Data

Location	Date	Wind Speed (Knts)	Wind Direction	Wind Gust (Knts)	Wave Period (s)	Wave Height (Ft)
Lake Michigan Buoy	9/19/2003	21	W (280)	25	6	5.6
	10/15/2003	27	NW	33	6	8.2
	11/13/2003	31	WNW (300)	37	8	13.8
Benton Harbor Airport	9/19/2003	16	WNW (290)	22	-	-
	9/19/2003	21	-	29	-	-
	11/13/2003	21	-	35	-	-

Storm Event	Area No.	Wave Period (s)	Wave Height (Ft)
(9/19/03)	1	6	(5-6)
	2	6	(6-7)
	3	6	(3-4)
	4	-	(2-3)
	5	-	(1-2)
	6	-	(1-2)
	7	-	(0-1)
(10/15/03)	1	6	(6-7)
	2	6	(7-8)
	3	6	(4-5)
	4	-	(2-3)
	5	-	(1-2)
	6	-	(1-2)
	7	-	(1-2)
(11/13/03)	1	7	(8-9)
	2	7	(8-9)
	3	7	(5-6)
	4	-	(2-3)
	5	-	(2-3)
	6	-	(1-2)
	7	-	(1-2)

MODEL DESCRIPTION

Modeling Criteria

Physical models to examine flow behavior in free surface flow are performed using Froude number similarity, which fixes the relations between model and prototype conditions once the physical model scale has been selected. Dynamic similarity requires keeping all Froude numbers defined by $V/(gL)^{1/2}$ equal in the model and prototype, where V refers to any representative fluid velocity, g the acceleration due to gravity, and L is any system length. The relations between prototype and model parameters are related to the scale ratio $\mathbf{L_r}$ which is the geometric ratio between any length in the model and the corresponding one in the prototype ($\mathbf{L_r}$ = $\text{Length}_{\text{model}}$ / $\text{Length}_{\text{prototype}}$). For a Froude scaled model, assuming the same fluid in model and prototype, the following relations must hold in which the ratio $\mathbf{Q_r}$, for example, represents the ratio of the model flow rate to the corresponding prototype flow rate:

PARAMETER		**RATIO**
Length	$\mathbf{L_r}$	$\mathbf{L_r}$
Velocity	$\mathbf{V_r}$	$\mathbf{L_r}^{1/2}$
Discharge	$\mathbf{Q_r}$	$\mathbf{L_r}^{5/2}$
Time	$\mathbf{T_r}$	$\mathbf{L_r}^{1/2}$

The critical factors with respect to model testing facilities are the model size and discharge. If the scale ratio is too small, both viscous effects and surface tension may become too great in the model. This consideration generally fixes the minimum model size required to avoid distortion of the model flow. An additional consideration with small scale wave phenomena is the ability to accurately measure wave heights. With these considerations in mind, a decision was made to construct the model at a 1:60 scale of the prototype harbor. In order to construct this model scale within the space constraints of the existing wave basin, it was possible to model approximately 1800 feet of the entrance channel extending from the location where the railroad crosses the channel to offshore of the location where the jetties change their alignment. Figure 8 indicates the model project outline. In this configuration, the entrance to the Waterfront marina, the areas where the proposed construction alternatives are to be implemented and the wave effects associated with the change in channel width and alignment were reproduced in the model. The entire Waterfront Marina was reproduced in the model while only a portion of the West Basin Marina was reproduced. Since only small waves were to be expected near the entrance to the West Basin Marina, and those entering into the marina would be scattered in the larger area within, it was not felt to be necessary to reproduce all detail within this marina. In addition, the model is truncated at the railroad crossing whereas the actual channel continues further upstream from there. Although the model was truncated, wave dissipating material (in the form of several rows of swimming pool lane divider floats) were placed at this location of the model simulating the effect of the actual waves continuing up the channel by eliminating a reflection off the back wall of the wave basin.

Figure 8. Model Project Outline.

The current associated with the St. Joseph River discharge was created by means of a recirculating pump that withdrew water from a location in the wave basin outside of the harbor and discharged into the area behind the wave dissipating material at the back of the harbor. The flow through the wave dissipating material straightened the flow and made it relatively uniform across the back of the model. This flow then proceeded out the channel entrance and recirculated back to the pump intake. Some preliminary experiments were performed without the river flow being simulated, but since it was not inconvenient to operate the model in this mode, all subsequent experiments were performed with the river flow even though the experimental results indicate that the effect of the current on wave heights is negligible. A comparison between a set of wave height measurements with the river flow and without the river flow is presented in Table 4. The averaged wave height ratios over the different measurement sections show no significant difference between the two sets of data in terms of the measured wave height. It should also be noted that these measurements were made early in the testing phase while experimental procedures were still being developed and individual comparisons may not be as accurate as subsequent measurements.

Table 4. Effect of River Flow on wave height measurements

EFFECT OF RIVER FLOW ON MEASUREMENTS
RATIO = WITH RIVER FLOW/ WITH OUT RIVER FLOW

PROBES		LOCATION		
	LEFT	CENTER	RIGHT	MEAN
1	0.98	1.01	1.21	1.05
2	1.23	0.99	0.79	0.99
3	1.14	1.06	0.92	1.04
4	1.00			1.00

Instrumentation

Waves are generated in the model with a plunger type of wave generator. The height of the plunger is adjustable which allows for flexibility in regulating the wave height. The wave generator produces monochromatic waves with a period that can be continuously adjusted. It is useful to continuously vary the incident wave periods over ranges that are of practical interest (assumed to be on the order of 4-7 seconds for this application) to identify particular periods that create undesirable wave conditions in the area of interest. These periods are then subjected to more detailed testing.

A wave probe system was used to measure wave heights in selected locations. These measurement locations were established in consultation with the U.S. Army Corps of Engineers, Detroit District and are indicated in Figure 9. Four wave probes were used in the study, necessitating moving the wave probes between measurements to cover all these locations. These capacitance type wave probes are used to measure the water surface displacement at a sampling frequency of 30 samples per second per probe. The

Figure 9. Locations of individual wave probe measurements.

probes are interfaced via an analog-to-digital conversion board to data analysis software that computes the mean water level as well as standard deviations of the water surface for a specified sampling period.

The probes were calibrated prior to the testing and the gain on the individual pre-amplifiers was adjusted to maximize the sensitivity of the probe response for the range of wave heights expected to occur at the various sampling locations. Calibration is performed by recording probe outputs at various water levels within the tank and fitting a linear relationship to the output voltage versus water depth data. Calibration curves for each of the probes are presented in Appendix A.

For simple harmonic waves, the standard deviation of the water surface variation is directly proportional to local wave heights (the wave height as traditionally defined would be 2.83 times the standard deviation for a pure sinusoidal wave) but for more complex wave forms that may be created due to interactions with reflected waves, the standard deviation provides a more accurate and simple way to estimate wave energy. Carpenter (2001) compared the standard deviation of the water surface to the total energy content in measured wave spectra for Lake Michigan waves and found a good correlation between the two. A typical measurement is performed by starting the wave generator and letting it run until the waves have propagated completely down the entrance channel and any possible reflections off the energy dissipation material at the back of the model have propagated back through the model. Under these conditions, a fairly constant wave height is observed at a given location over time in the model as indicated in a typical water surface displacement profile displayed in Figure 10. These are presentations of the same wave record, Figure 10a displaying the complete record while 10b provides more detail on an intermediate portion of the record. Although these particular wave records are fairly clean with the waves arriving straight down the channel, others that are collected at locations where wave reflections create an interaction between two wave trains present more complex water surface profiles. Also, in some experiments (particularly at the larger water depths), a beat pattern was sometimes observed at some locations due to the creation of a partial standing wave system. The beat period of these waves was on the order of about 10-15 seconds. After some initial experimentation, it was decided to sample for a 45 second interval (20-25 individual waves) with sampling starting approximately 30 seconds after switching on the wave generator. The data acquisition software reports the mean water level as well as the amplitude for the run. There is often a problem with airborne particulates accumulating on the water surface and this dirt impacts the wave measurement, appearing as an increase in reported mean water level and a decrease in wave amplitude. Wave results that exhibited this behavior were discarded and the probes were cleaned prior to continuing the measurements. The averages of 4 or 5 individual runs were used to provide average wave amplitudes at each measurement location. These in turn were scaled up to wave heights by multiplying the average wave amplitude by the product of the scale ratio (60) and the sinusoidal wave amplitude-to-wave height conversion factor (2.83) to obtain the final reported wave heights given in the subsequent presentation of experimental results.

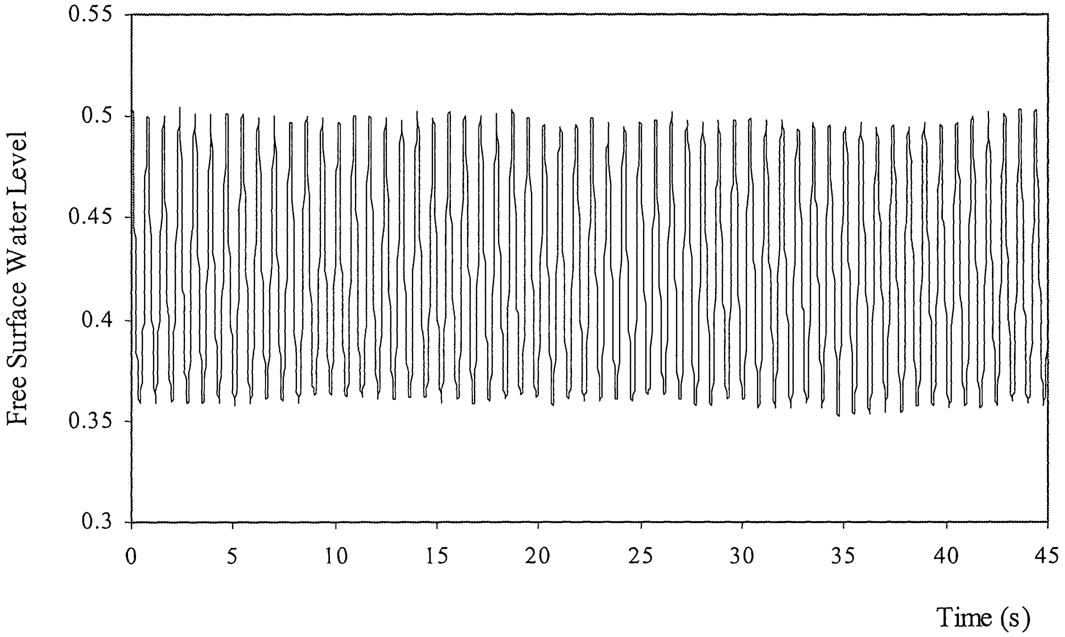

Figure 10a. Typical Water Surface Displacement Record. (T = 6 s, 280°)

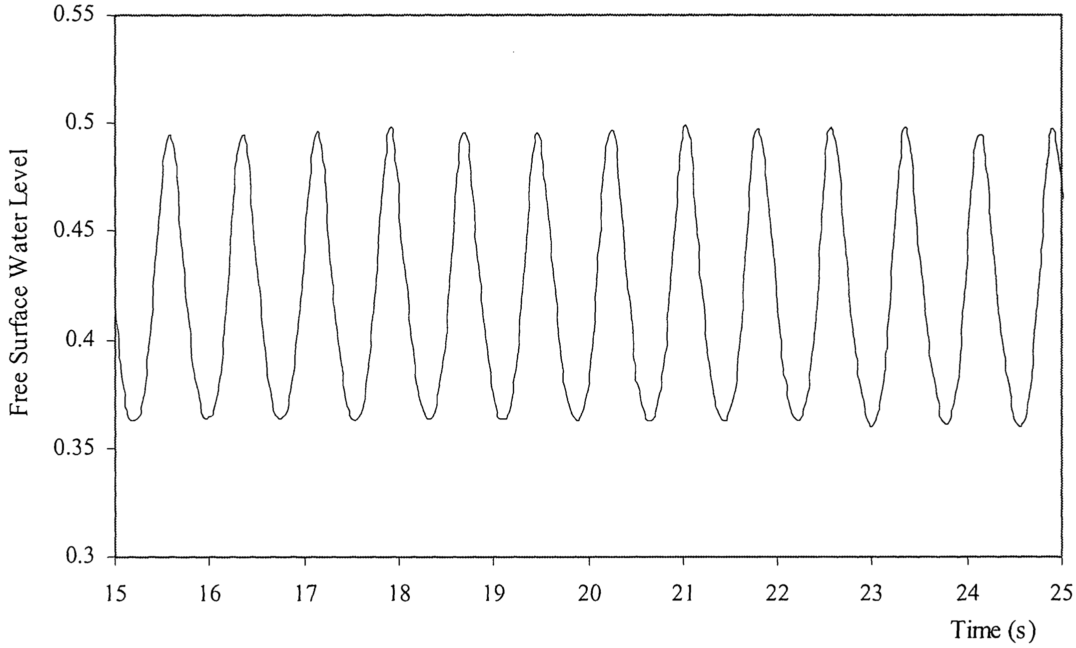

Figure 10b. Sample of a Water Surface Displacement Record. (T = 6 s, 280°)

MODEL CONSTRUCTION

As previously indicated, the physical model of the Saint Joseph Harbor was constructed at a 1:60 scale of the prototype. The back of the model was consistent with the location of the railway bridge across the river channel (refer to the project outline in Figure 8), while the offshore end extended beyond the width and alignment transition in the channel. The actual section modeled varied depending on the alignment of the wave generator. For the waves aligned with the offshore section of the channel (these are referred to below as 280 degree waves) the south jetty wall extended approximately 550 feet offshore beyond the change in alignment while the north jetty wall length was approximately 315 feet beyond the alignment change of that wall. Altering the wave generator direction to represent waves arriving from the southwest created a change in jetty wall lengths so that modeled length of the south wall became 470 feet while the north wall represented was 390 feet. These walls were constructed of masonry blocks over the portion of the entrance channel that was constructed from steel sheet piling. No attempt was made to reproduce the actual height of the steel sheet pile walls and they were constructed at a sufficient height to prevent wave overtopping in the model. Wave overtopping is to be expected in the prototype under certain combinations of water surface elevation and incident wave height, but wave overtopping will only serve to reduce wave heights further back in the harbor.

Bathymetric charts were used to estimate water depths within the harbor area. Depths in the main navigation channel are indicated to be on the order of 24-25 feet relative to low water datum and probably vary over time due to sediment transport through the harbor. Outside of the navigation channel, water depths are much reduced. The navigation channel was reproduced in the model as a constant depth with the floor of the wave basin representing the navigation channel bottom. The lake floor was built up from that using a combination of flat masonry blocks and sand. The masonry blocks were used to build up the general bottom contours while the sand was added over them to provide a continuous sloping bottom to reproduce the provided bathymetry. Figure 11 indicates the model prior to filling with water showing the sand placement within the harbor area.

Figure 11. Harbor bathymetry reproduced in the model with concrete blocks and sand, looking towards Waterfront Marina.

Primary testing was performed using a water depth that is consistent with current Great Lakes elevations. Although water depths vary in a consistent pattern over the course of a year (Monthly Bulletin of Lake Levels for the Great Lakes, Detroit District, Corps of Engineers), current water levels are on the order of or slightly below the low water datum and a water depth was selected in the model to be consistent with these levels. Once the wave basin was filled to the appropriate water depth, a point gauge was installed in a corner of the wave basin and set so that the gauge just touched the water surface. Prior to testing for any day, the water level in the basin was filled to the level indicated by the point gauge. A second point gauge was used for a more limited set of tests at "high" water levels. This corresponded to a water depth four feet greater than the low level reading, corresponding to a water surface elevation that is near the all time high water levels recorded for the Lake Michigan-Huron system.

Other portions of the harbor area in which stone is present were constructed from crushed limestone of a size that is physically scaled from the estimated size of the prototype stone. This includes the breakwater stone in front of the U.S. Coast Guard Station. In the initial phase of the testing, details on the breakwater cross-section were not available so it was constructed from a single stone size. Further into the project, information was received indicating that the breakwater core was constructed with a core consisting of 50-300 pound stone and with a primary armor layer of 3-6 ton stone. Furthermore, the top of the core stone is approximately 3-4 feet above the low water datum. Based on that information, the breakwater was reconstructed to match those specifications. This led to what is essentially a set of two "existing" harbor test measurements, one with the Coast Guard breakwater constructed with uniform stone in the model and a later set with a more realistic representation of the actual breakwater cross-section. In the results presented below, only the data involving simultaneous modifications at both the Lighthouse Marina location and the U.S. Coast Guard station were tested with this latter configuration. When specific results are compared to an existing condition, the one that is appropriate to the breakwater section that was in place for the actual experiments is used in the results presented below.

No attempt was made to reproduce floating docks or other similar detail within the harbor area. The model thus consisted of an outline in masonry blocks of the areas bounded by steel sheet piling with internal stone placed to represent existing stone structures or proposed ones that are part of a proposed harbor modification alternative. The area of the proposed Lighthouse Marina was constructed from a series of different stone sizes with an attempt to reproduce the general conditions depicted in a series of photographs provided for this area of the harbor. This construction is a rather complicated region with armor stone, large concrete blocks, the remnants of old wood piling, etc. (see Figs. 1-3) and minute detail of all this could not realistically be reproduced in the model, but an attempt was made to recreate general features including armor unit size and depth. Figure 12 depicts this specific area of the model with the Waterfront Marina in the background.

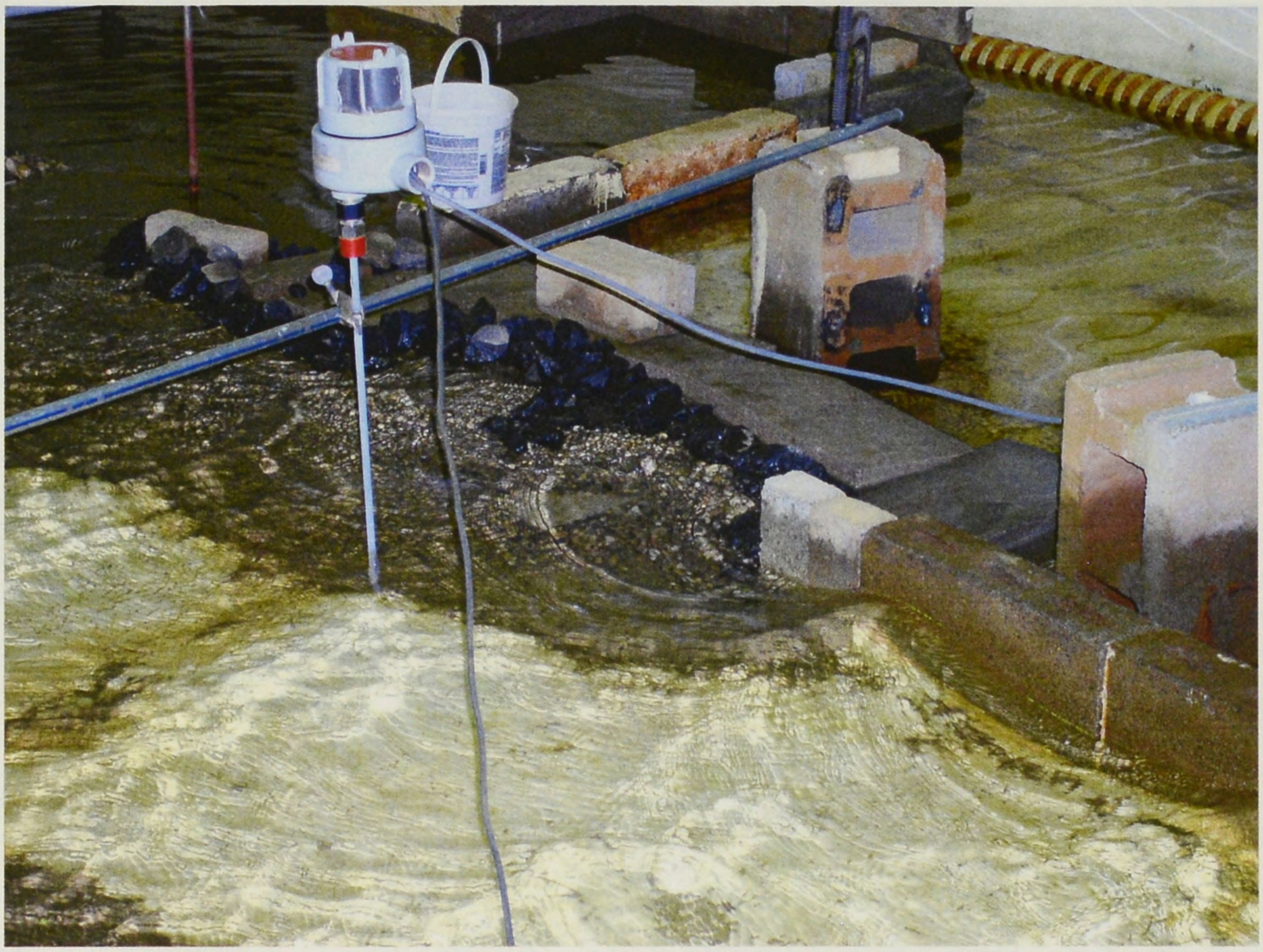

Figure 12a. Model reproduction of existing area where Lighthouse Marina development is proposed.

Figure 12b. More general detail of model construction near Waterfront Marina.

MODEL TESTING PROCEDURE

The experimental protocol involves comparing wave conditions within the harbor for some configuration of interest to the existing configuration in order to measure a change in wave activity at selected locations. These measurements were made for a variety of wave conditions. Variables that were considered to be potentially important in influencing the wave activity within the harbor include:

- Wave period
- Wave direction
- Water depth

Testing was performed over a range of each of these variables as discussed in this section. Since all of these variables are continuous ones, there are an infinite number of combinations that could be studied. Practicality constraints dictate that only some limited combinations of these variables can be selected for investigation. Selection of appropriate test conditions depends on the particular objectives of the study. In this particular case, it is understood that the primary issue is associated with the potential impact of increased wave activity on small craft use of the harbor. Given this objective, it is reasonable to select wave conditions that would be associated with high wave activity but not necessarily extreme conditions during which use of the harbor would not be expected. This consideration was used to select the range of test conditions.

Wave Period – There is a strong correlation of wave height and period such that, in general, the larger wave heights will be associated with the longest periods. Using the hindcast wave height information from the WIS report for Lake Michigan establishes a feasible range of wave periods for the closest station, M59. Considering waves that approach from the west, the mean peak period is 4.3 seconds. Of the same waves, one percent of the waves have a period exceeding about 7.3 seconds; these would be the extreme waves normally during wintertime conditions. Given these statistics, it was decided to investigate waves with periods between about 4.5 and 7 seconds. The wave generator was continuously adjusted over that range to examine for wave conditions that may be relatively more severe towards the back of the harbor due to internal interactions. Since the harbor has considerable internal damping due to the presence of stone and interconnected basins, resonant interactions were not a major issue and no periods with extremely large wave heights were identified. Based on the examination, it was decided to test wave periods of 5.0, 6.0 and 7.0 seconds as being representative of those for which larger incident wave heights may be expected but for which use of the harbor is still possible during the wave event. For convenience, a particular wave height was not associated with each period, but rather a common wave plunger displacement was used for experiments with all of these periods. The selected model condition was to produce a wave that was just under breaking amplitude so that, while the results were not complicated by wave breaking within the harbor, the waves to be measured would be as large as possible. The model study compares the wave climate associated with a particular proposed modification to the existing conditions and if these are compared with a similar incident wave, the effect of the proposed modification can be assessed.

Wave Direction - Based on studies by Carpenter (2001), the largest wave amplitudes towards the back of a harbor entrance channel that exhibits some wave dissipation are observed when the waves propagate straight down the entrance channel. Therefore, one wave direction investigated in this study was associated with waves propagating along the axis of the outer portion of the entrance channel. Discussions with Corps of Engineers personnel also indicated a desire to examine a condition with oblique waves. The decision with respect to the particular oblique wave direction was guided by the geometry of the offshore end of the jetty walls. The WIS report indicates that the largest waves arrive from the sector from 315 degrees with highest frequency (see the wave rose in Figure 6). However, since the north jetty projects further offshore than the southern one, these waves would be prevented from directly impacting on the entrance channel and waves from the northwest will enter the harbor only by diffraction around the offshore end of the north jetty. The diffraction process will reduce those wave amplitudes. On the other hand, waves from the 225-degree sector are more frequent and also will impact on the exposed end of the north jetty and be reflected down the entrance channel. On this basis, it was decided to select an additional wave direction that is effectively from a direction of 260 degrees or 20 degrees off the channel axis alignment. Note that the waves are actually generated at that alignment and since the offshore ends of the jetty walls are not included in the model, the interaction with the jetty entrance is not included in the model. Considering the diffraction at the channel entrance, it was not considered to be reasonable to study steeper wave angles than about 20 degrees so a wave direction of 260 degrees was selected for modeling. A limited initial set of experiments at a wave angle of 15 degrees off the channel axis was also studied and found not to be materially different than the 20-degree waves and measurements with this third direction was not carried through the entire testing program.

Water Depth – The bulk of the testing was performed for water depths representing current Great Lakes levels, essentially at low water datum. However, since lake levels are subject to a fair amount of variation, it was considered prudent to also investigate the system performance at higher water levels that may reasonably be expected in the future. Therefore a more limited set of testing was performed with water depths that correspond to a water level four feet above the low water datum. This is on the order of the extreme high lake levels that were observed in 1986/87 but the monthly mean lake levels vary over the course of a year due to seasonal hydrological influences and this level is only approximately equivalent to high water conditions.

One additional factor in the testing was the choice of St. Joseph River discharge. Although this factor is believed to have a negligible effect on the wave interactions in the harbor, the effect was included in the model. However, only a single river discharge condition was studied; this was for a prototype flow rate of approximately 5000 ft^3/s. The basis for this flow rate, as discussed above, is that it is somewhat above the estimated long term average flow rate for the St. Joseph River at its discharge to Lake Michigan.

A total of three options for possible implementation of the Lighthouse Marina construction project were tested in the model. The first of these was the original permitted option while two additional modifications that were considered to be alternatives that may result in more effective local energy dissipation were also studied. All three options involved a steel sheet pile wall aligned parallel to the navigational channel with armor stone placed on a 1:2 (Vertical:Horizontal) slope in front of it. Following is a discussion of the specifics of the three options.

Figures 13-15 are drawing showing plan and horizontal views of the construction options for the boundary between the Lighthouse Marina and the navigational channel. These options are referred to in the report below as Option 1 – 3. Figures 16-18 are images of each option as implemented in the model.

Option 1 is depicted in Figure 13 and includes a steel sheet pile wall that is roughly an extension of the existing Corps of Engineers wall to the west. The existing water depth in the vicinity of this wall is approximately seven feet. The intention of this option is to place armor stone on a 1:2 (V:H) slope in front of the well laid on the existing bottom and extending ten feet towards the navigation channel. Figure 16 indicates the implementation of this option in the model. An aluminum sheet of the correct length was placed at the appropriate position and crushed limestone of a diameter assumed to be consistent with likely stone size to be used in the project was placed in front of it. The geometry of the proposed Lighthouse Marina consistent with this option was created behind the aluminum sheet.

Option 2 involves several modifications to Option 1. Much more stone is used in this option with 25 feet of stone extending towards the navigational channel but also laid on a 1:2 slope. This requires the sheet pile wall to be set back relative to the existing Corps of Engineers wall. In addition to the armor stone in front of the steel sheet pile wall, this option consists of a series of steel sheet pile baffle walls with a top elevation of 586.5 that extend perpendicular to the main sheet pile wall and 25 feet towards the navigation channel. The baffle wall spacing studied was 50 feet, resulting in essentially a series of cells bounded on the sides with sheet pile baffle walls and filled with stone laid on a slope. The details of this option are indicated in Figure 14 while Figure 17 depicts the laboratory implementation of this option. The baffle walls in the model were constructed from acrylic sheet that was screwed to the aluminum plate serving to represent the back sheet pile wall.

Option 3 was conceptually similar to Option 2 and is indicated in Figures 15 and 18. The major difference between the two is that a smaller amount of stone was used, more or less consistent with Option 1. The stone and baffle walls extended only 10 feet towards the navigational channel and the baffles had a smaller spacing of 25 feet as opposed to the 50 ft spacing in Option 2.

The model was constructed in such a way that any of the three options described above could be installed in the model. The geometry of the existing configuration is substantially different and initial testing was completed for it prior to studying the various options. Towards the end of the testing, it was necessary to re-create the existing configuration and this was accomplished with the same materials with attention paid to reproducing the original setup as accurately as possible.

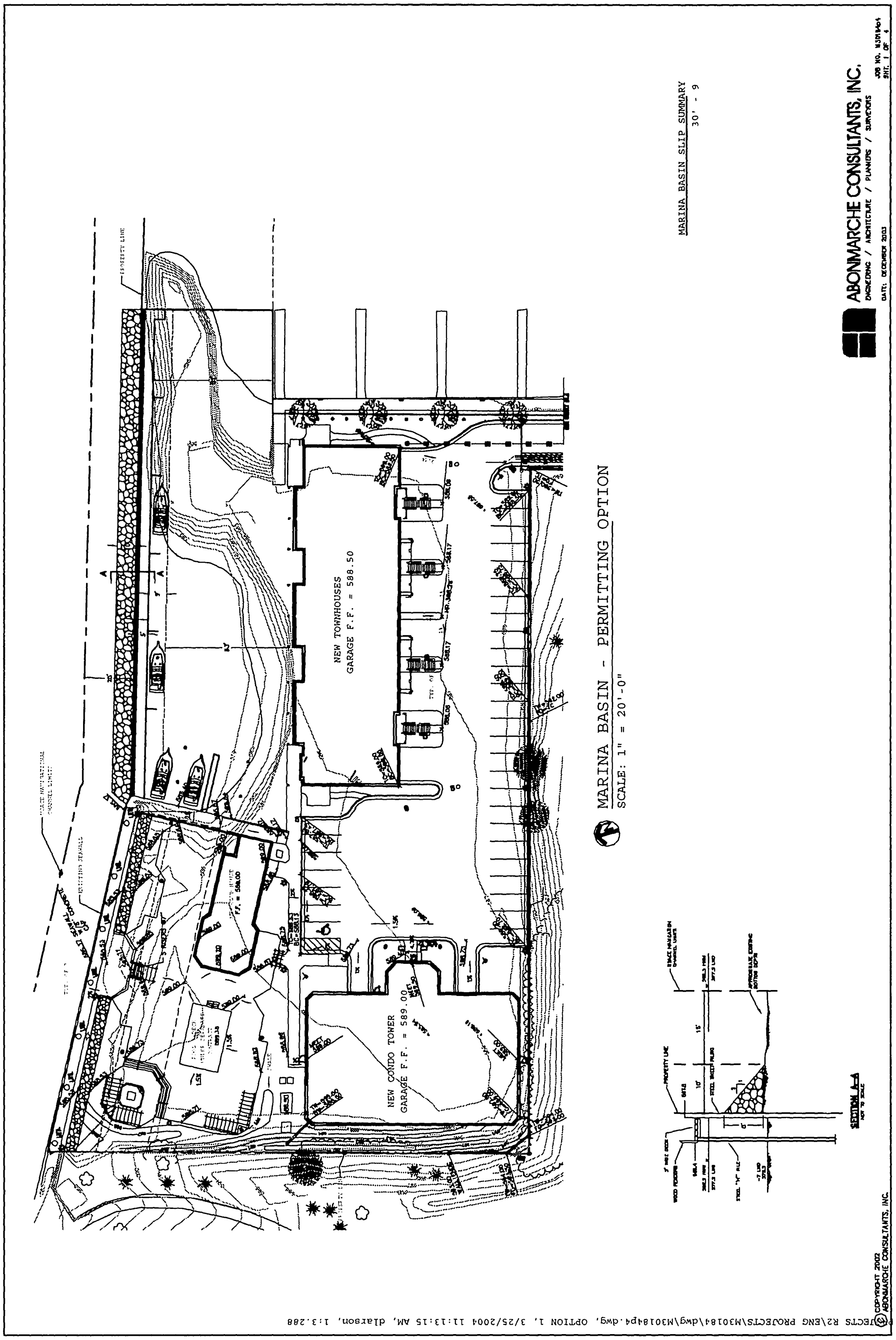

Figure 13. Details of permitted Option 1

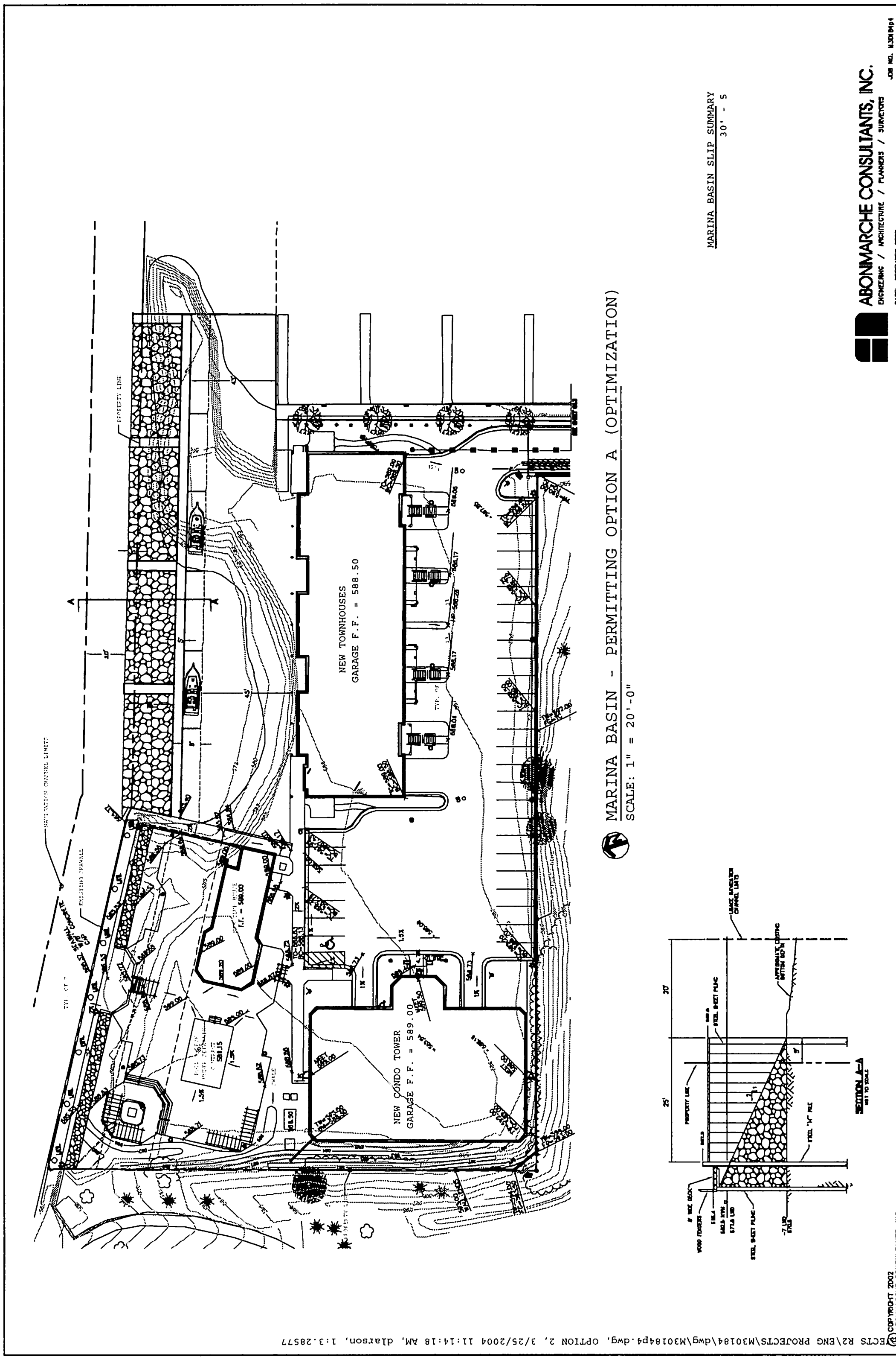

Figure 14. Option 2 variation of permitted construction.

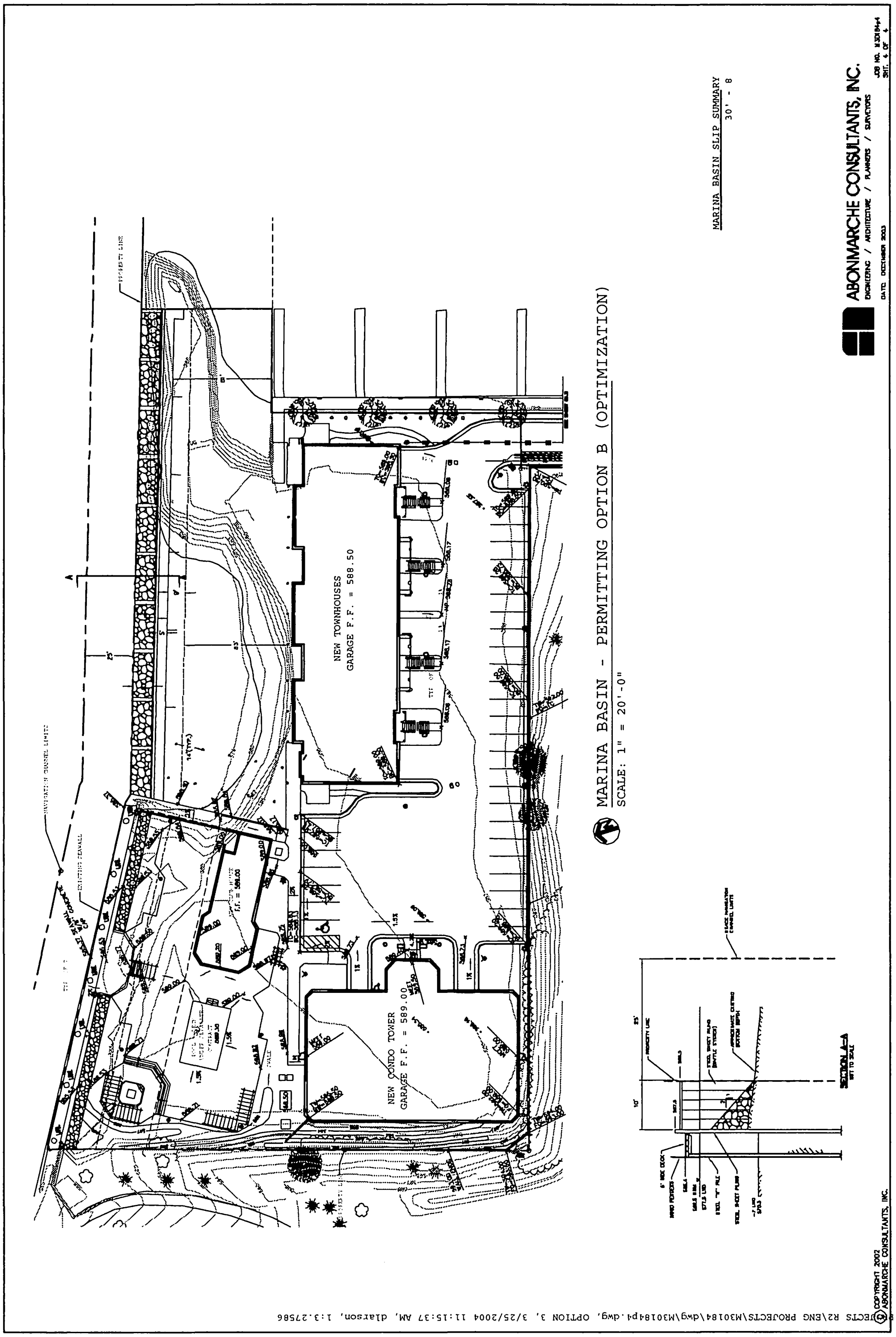

Figure 15. Option 3 variation of permitted construction.

Figure 16. Physical model representation of Option 1.

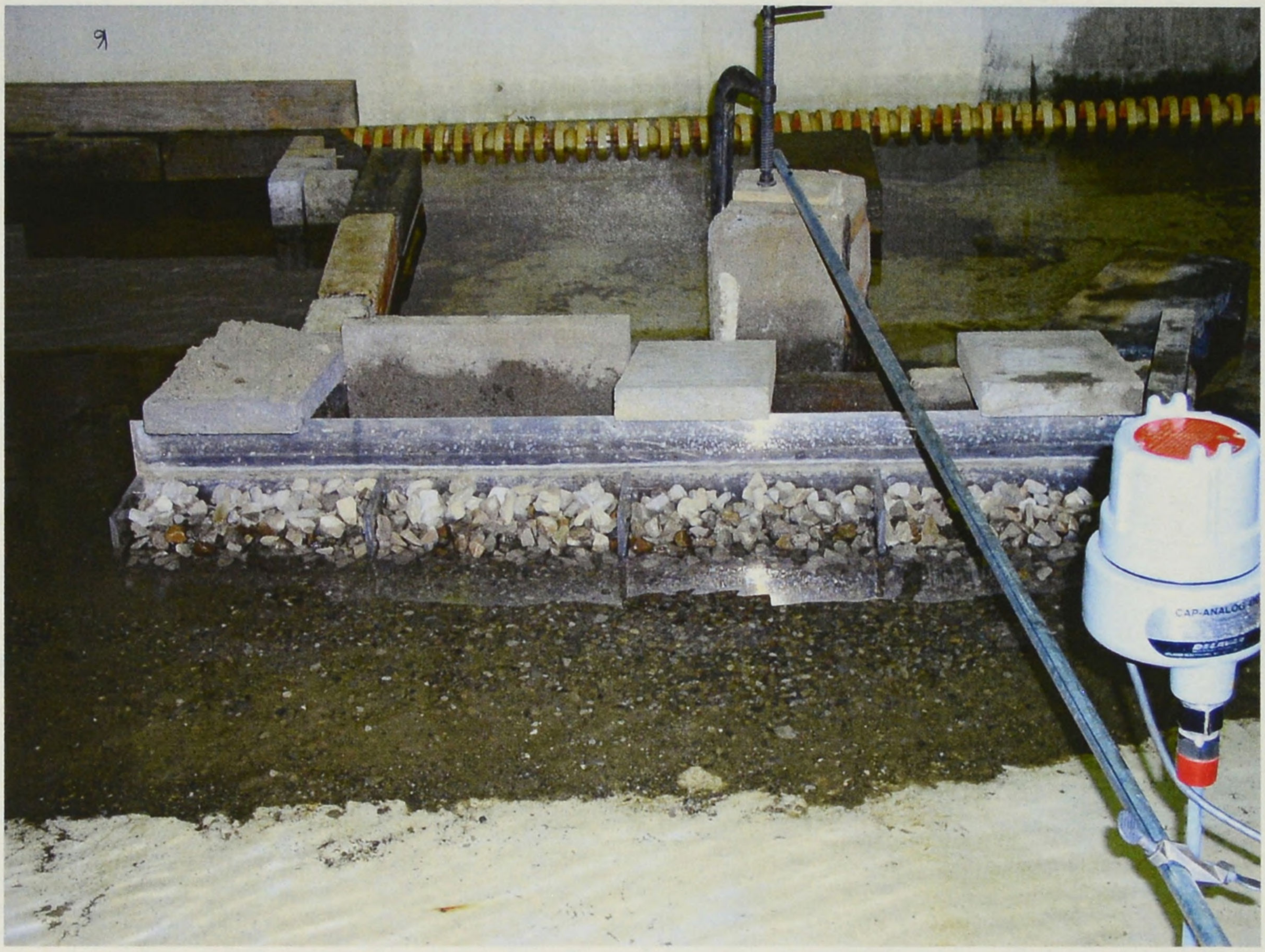

Figure 17. Physical model representation of Option 2.

Figure 18. Physical model representation of Option 3.

RESULTS

Interpretation of Model Results

The objectives of this project were to demonstrate whether or not specific modifications to the St. Joseph Harbor produced more than minor increases in wave activity within the general vicinity of the navigational channel. The most effective way to demonstrate the attainment of the objectives is to perform a comparison of the wave climate under existing conditions within the harbor and that resulting from harbor modifications under the same incident wave conditions. Practicality considerations require that the wave activity be measured at pre-selected points that are intended to be representative of general areas within the harbor. However, there are specific considerations that should be kept in mind as this comparison is made.

The wave height at any specific location within the harbor is the result of the sum of all wave interactions including wave energy dissipation, wave diffraction around barriers and wave reflection primarily off of vertical surfaces. The observed wave height at a particular location may be influenced to a lesser or greater extent by each of these processes depending on the particular harbor geometry. This is particularly true when a system of partial standing waves is set up, in which case a significant variation in wave amplitudes may be observed over distances on the order of one quarter of a wave length (generally on the order of 30-40 feet for the wave periods tested and the depth in the navigation channel). There was little indication of standing wave development in the experimental measurements but the process should still be considered in the interpretation of results. Therefore it is possible that a particular location would indicate an increase in wave activity while other measurement locations in close proximity (a few tens of feet) may indicate a decrease. This possibility is always present when modifications to the harbor geometry are implemented. The implication is that comparisons of wave activity should be broadly integrated over a number of independent measurements in order to assess general changes in wave activity. In certain of the results presented below, a single point may indicate a significant increase in wave activity for a given incident wave conditions but all other measurement points will not. Under these circumstances, it is reasonable to conclude that the wave activity increased specifically at the measurement point but not necessarily for the entire vicinity of the measurement point.

A second issue is that all experimental measurements are subject to uncertainty due to limitations in the experimental procedure as well as measurement precision. Therefore, each comparison of wave activity should be interpreted in light of this uncertainty. All efforts were made in the experimental design to minimize these sources of uncertainty, but they cannot be eliminated. As a part of the investigation, an effort was made to identify the magnitude of the expected uncertainty and this is discussed below prior to the presentation and interpretation of experimental results.

In light of the above consideration, careful consideration was made as to the most appropriate procedures for interpretation of results. The complete set of experimental results is presented in Appendices B and C but the data have been further processed and interpreted according to the following metrics:

- The most fundamental comparison is simply a ratio of the wave height at a particular location after the installation of a proposed modification to the wave height under existing conditions. As explained previously, the progression of the experiments required the measurement of two different "existing" conditions; the first is with the Coast Guard breakwater constructed from a single uniform stone size and is the basis for almost all comparisons in this study. The second existing condition involved the re-construction of the Coast Guard breakwater once details on the cross-section were provided. There are only a few test results in which this latter set of measurements is the appropriate basis for comparison, specifically the cases where the modification to the US Coast Guard breakwater was studied in addition to the modification at the Lighthouse Marina site;
- Since measurements were generally made for three points across the navigational channel at several different cross-sections, arithmetic averages were made of all three points to obtain a better idea of the energy flux past each station. This approach follows the procedure outlined in Carpenter (2001) in which that approach was found to be consistent for interpretation of pocket wave absorber performance in Great Lakes channel entrances. Figure 9 indicates the measurement sections. Locations 1-4 involve the measurements at three discrete points across the jetty channel (the three points are denoted right, center, and left

according to their relative position looking down the jetty channel from the Lake Michigan end) while Locations 5-8 involve a measurement at a single point. Locations 7 and 8 were only used for the later experiments in which the modified Coast Guard breakwater was implemented in the model. It is expected that wave energy should decrease proceeding from the jetty entrance to the back of the harbor due to wave energy dissipation and systematic changes in the average wave height ratio at the measurement section may be taken as indicative of changes in the energy dissipation process.

- Finally, the various permutations of wave conditions and measurement points lead to a large number of wave height ratios for each modification tested. It is difficult to process the meaning of all the individual data in light of potential measurement uncertainties and the possibility of wave reflection and diffraction influencing the wave height ratio at a specific point. Therefore an appropriate overall metric is a weighted-average wave height ratio for each modification and wave condition tested. This ratio was weighted by the existing wave height at a particular location so that a small increase in the height of initially small waves does not unduly impact this metric. The wave height ratio at a particular location was weighted according to the "existing" wave height at that location divided by the average wave height for all measurement locations and the resulting quantity was averaged over all measurement points, resulting in a single number for each configuration tested (i.e. a particular harbor modification at a given water depth, wave period and direction). A number greater than one indicates an overall increase in wave activity while a number less than one is indicative of a decrease. This metric will be referred to in the following discussion as the *Overall Harbor Response (OHR).*

Each of the above metrics is presented in the results discussed below and in general, they are consistent in providing a relative ranking of alternatives studied.

Measurement Uncertainty

It is essential to understand the basic uncertainty associated with the experimental measurements in order that differences in experimental results that may be attributable to measurement error are not interpreted as representing true differences between individual measurements. Measurement uncertainty is due to several components:

- Electrical noise through the electronic components is a lower bound on the measurement error;
- Ability of the probes to monitor a given wave condition. This can be assessed by examination of the variation of individual repetitions measuring the response to the same wave condition;
- Issues associated with uncertainty in the experimental procedure. These include items such as the ability to align the wave generator to a particular direction, ability to re-create a particular test configuration if modifications are made to the model, etc.

An attempt was made to understand each of these effects. For example, measurements of the water surface displacement were obtained without the wave generator in operation in order to obtain reported wave amplitudes for each probe without there actually being waves present in the model. These reported amplitudes are taken to represent measurement limitations of the instrumentation system. These are generally very minor with the exception of one probe that exhibited a fair amount of electrical noise. The electronic noise converted to equivalent prototype wave heights results in a reported wave height on the order of 0.01 ft for three probes. One probe yielded a reported wave height of approximately 0.15 ft. In order to minimize the impact of that uncertainty on the wave measurement, that probe was used in locations where relatively large wave heights were recorded so that the noise level was less than other measurement uncertainties.

A number of repetitions were made with the wave probes in a particular location and the wave generator in a fixed configuration. Standard deviations of the repetitive measurements from each location were computed. Table 5 summarizes the results for each of the four probes used; the standard deviations vary slightly among the different probes. In particular, Probe 4 has a smaller uncertainty than the others because the amplifier gain was increased with this particular probe to measure wave heights at locations where the smallest wave heights were expected. Note that these values are greater than the electronic noise and are considered to be associated with minor variations in the wave generator output as well as the previously mentioned problem with floating particulates in the wave tank influencing the probe output. When individual measurements were well outside the indicated range, the results were discarded as being influenced by the presence of dirt on the probes but the deviations between individual measurements, when small, was accounted for by averaging the results of on the order of five repetitions of the same experiment. The basic uncertainty in wave height measurements (when reported as a prototype wave height) is thus considered to be on the order of 0.3 ft. The measurement uncertainty when expressed as a percentage of the measured wave height is therefore a function of the actual measured wave height. Some individual measurements recorded wave heights on the order of 1-2 feet and these particular measurements are subject to a significant uncertainty. Since most of the study results are presented as ratios of two wave heights (the modified condition to the existing) the combined uncertainties in the two measurements limits the precision of the reported wave height ratio. The methodology outlined in *ISO 5168-1978 Measurement of fluid flow - Estimation of uncertainty of a flow rate measurement* was used to estimate measurement uncertainty for situations where data from multiple locations was aggregated or where multiple uncertain variables were combined (such as in the computation of the wave height ratio). In reality, the uncertainty in the wave height ratios is probably not that great, since the model results were screened during the testing sequence and individual configurations were repeated if a significant anomaly appeared in the results.

Other contributions to measurement uncertainty including issues such as positioning the wave generator were more difficult to assess but care was taken in the experimental design to minimize these. For example, the wave generator locations for the various wave angles were marked on the wave basin floor prior to commencing the measurement program and these marks were used to re-position the wave generator as required in the sequence of measurements. One test in which the wave generator was deliberately slightly misaligned was conducted and the differences in the measured results could not be distinguished from the equivalent set of measurements with the correct wave generator position. Other sources of significant measurement error could not be identified.

Table 5. Experimental uncertainty of probe wave amplitude measurements.

	Uncertainty			
	Probe 1	Probe 2	Probe 3	Probe 4
Model (in)	0.06	0.068	0.056	0.04
Prototype (ft)	0.30	0.34	0.28	0.2

Existing Conditions

As discussed previously, a complete set of wave height measurements were made with the model in the existing configuration for three different wave periods (5, 6, and 7 seconds) and three different wave generator orientations (280, 265 and 260 degrees, these directions representing the azimuth that the generated wave originated from). The 280 degree wave direction corresponds to a wave crest aligned perpendicular to the outer jetty channel while the other wave directions involve an oblique angle of incidence. The complete set of measured wave heights is included in Appendix B. The wave probe locations are indicated in Figure 9. In general, these data show an expected tendency for wave heights to decrease proceeding back into the harbor for all wave periods and directions (also see Figs. 20-25 below). This expectation is based on the dissipation of wave energy by the armor stone along the harbor perimeter and due to diffraction of the waves into the larger area towards the back of the harbor. There are specific locations that may not follow this trend for all wave configurations; for example, reflection of incident waves off the north jetty wall beyond the change in alignment was observed to locally increase wave heights along the north side of the channel in measurement location 2. The reflection associated with the alignment change also tended to reduce the wave heights along the south jetty wall in the same location. For 5 second waves, for example, the measured wave height closest to the north jetty wall was 8.94 ft for the 260 degree waves and 8.18 feet for the 280 degree waves. Near the south wall, the measured wave heights in the same cross section were 4.80 and 3.51 feet, respectively. Differences between the north and south sides of the channel were not as significant for the 6 and 7 second waves.

A limited comparison of these data with the observations from storms in the harbor is possible. In particular, the storm data from 9/19/03 is compared with the experimental results from the 6 second, 280 degree waves as these appear to be the most equivalent sets of conditions. Table 6 provides a comparison at locations in which observations were made at more or less equivalent locations in the model and in the actual harbor. It should be noted that the wave heights observed in the harbor were estimated from water displacement along sheet pile walls, etc. while the laboratory measurements were made with some clearance from those walls. Also the location 4 observed in the harbor was at the entrance to the West Basin Marina while measurement location 5 in the laboratory was located approximately one hundred feet from the end of the stone jetty protecting the west side of the marina entrance (see Figure 9). Considering that the comparisons are made only within general areas, the model behavior is seen to replicate the behavior of the actual marina with an acceptable degree of precision.

Table 6. Comparison between waves measured in model and observed in harbor during September 19, 2003 storm.

St Joseph Harbor		**Physical Model**	
Location	Wave Height (ft)	Location	Wave Height (ft)
1	5-6	1	5.34
2	6-7	2	7.50
3	3-4	3	4.43
5	2-3	5	3.23

All data that reflect measurements for existing conditions within the harbor are included in Appendix B. These include measurements with the re-constructed Coast Guard breakwater and measurements made at higher water levels. All of these will be compared to equivalent experiments performed with modified harbor conditions in the following section.

Harbor Modifications

This section proceeds from the most general metrics presented above to the more specific ones. The data for individual measurement locations and wave configurations are presented in Appendix C both in terms of measured wave heights (scaled to prototype dimensions) as well as the ratio of the wave height under the modified condition to the existing condition. Note that the 265 degree direction was not measured for most test conditions since the differences between that direction and the 260 degree direction was found to be relatively minor. Figure 19 indicates the average wave heights by measurement section for each of the three wave directions. Although there are minor differences between the 260 and 280 degree directions, the difference between the 260 and 265 are negligible (in this plot, at locations 1 and 2, the data for these two directions plot on top of each other with only minor differences elsewhere). Therefore the data for the 265 degree waves are included in the appendices for completeness but are not discussed further below.

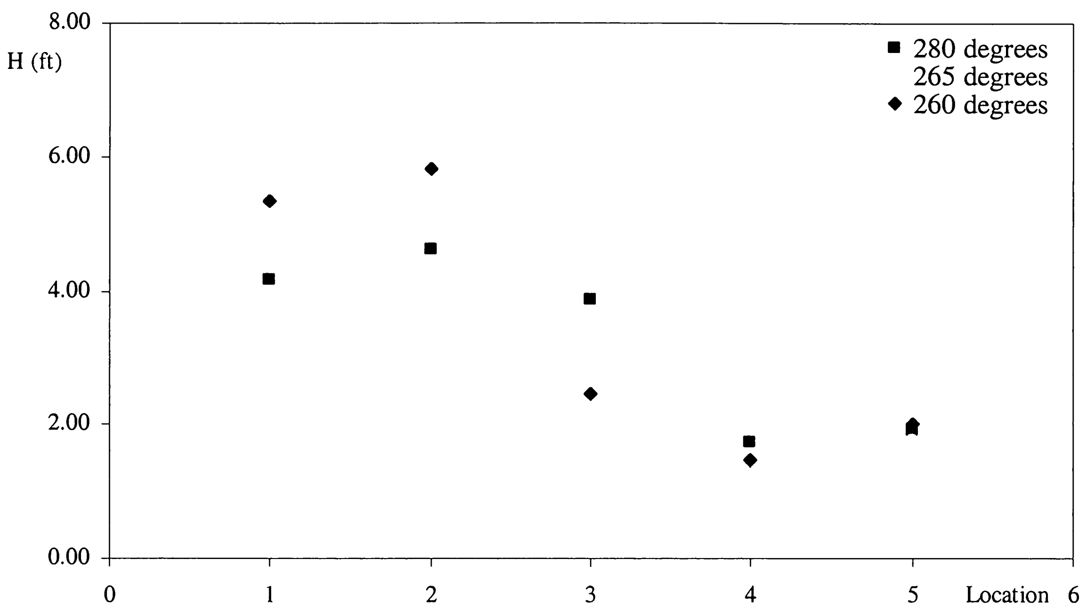

Figure 19. Comparison of measured wave heights for three different incident wave directions.

The weighted average wave height ratio for all locations or OHR was computed for all wave configurations. Again, this number is indicative of a general change in wave activity within the harbor with a number greater than one indicating an increase in wave activity (for a modification compared to the existing condition) and a number less than one indicating a decrease in wave activity. The ratios for all conditions are listed below for the water level associated with existing lake conditions.

280 degree

	5s	6s	7s
OPTION 1	1.01	0.94	1.07
OPTION 2	1.04	0.91	1.08
OPTION 3	1.03	0.95	1.01

260 degree

	5s	6s	7s
OPTION 1	0.98	1.05	1.06
OPTION 2	0.88	0.82	0.84
OPTION 3	0.88	0.84	0.81

An average wave height (averaged over all locations) for each configuration is on the order of 3-4 feet. Considering a basic uncertainty of 0.2-0.3 feet for each individual measurement and considering the effect of random measurement errors offsetting each other when aggregating the individual measurements to form these ratios, an upper bound on the measurement uncertainty in the ratios presented above is on the order of 0.05. Most of the variations among options for a particular configuration are within this uncertainty with the exception of the 6 and 7 second period at a 260 degree incident wave direction for which options 2 and 3 perform better than option 1. A conclusion from examining this aggregate metric is that all three options produce at most a minor increase in wave activity with an indication that Options 2 and 3 somewhat reduce the wave activity under some wave conditions.

A more detailed examination of wave height changes associated with the various options was made by comparing the results at the various sections. In the results presented below, the ratios are presented graphically by location. Figure 9 indicates the measurement positions for each location. Figures 20 - 25 indicate the measured wave heights at each of the locations for each of the two wave directions and three wave periods while the wave height ratios are presented in Figures 26 through 31 for the same experiments.

The absolute wave heights for each of the options and any wave configuration show a clear and consistent trend to decrease with distance into the harbor. The error bars presented in Figures 20 – 25 indicate the measurement uncertainty as discussed above; in most cases, the results for the various options are within the estimated measurement uncertainty. For reasons that are not totally clear, a consistent observation is that the 7 second waves showed more variability. There is generally little difference between the first two locations due to the fact that there is no stone or other energy dissipation material in that portion of the jetty channel, only the change in channel alignment. Since location 2 is just beyond the alignment change (and reduction in channel width), there is often a slight increase in wave height compared to location 1; the same effect is noted in Table 3. The 260 degree wave results show more scatter simply because there are more wave reflections and this complicates any consistent measurement of wave energy.

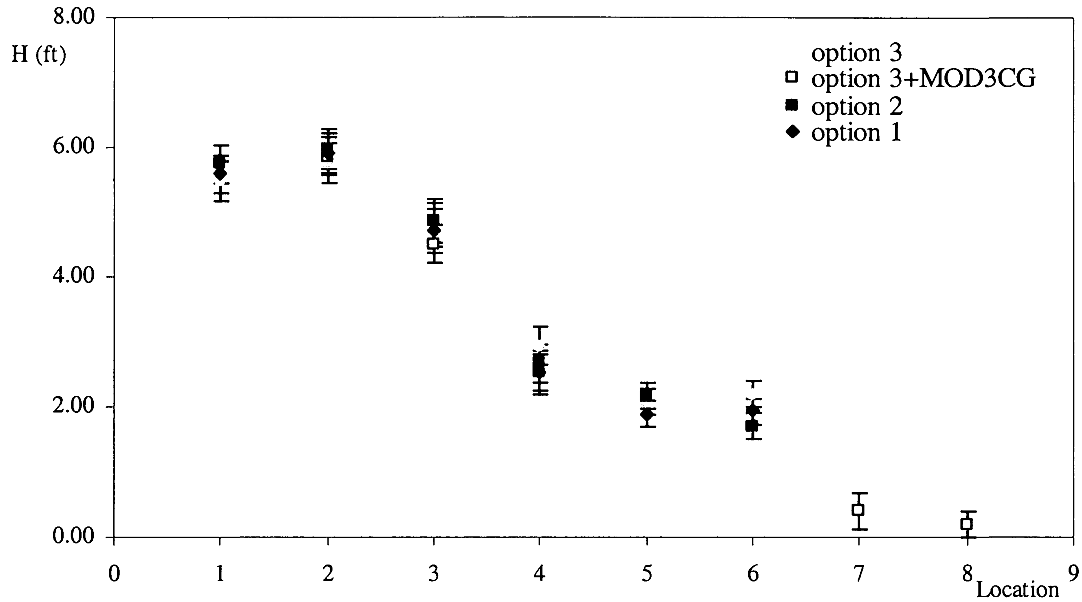

Figure 20. Comparison of the three proposed modifications for a wave period of T = 5s and an incoming wind angle of 280°.

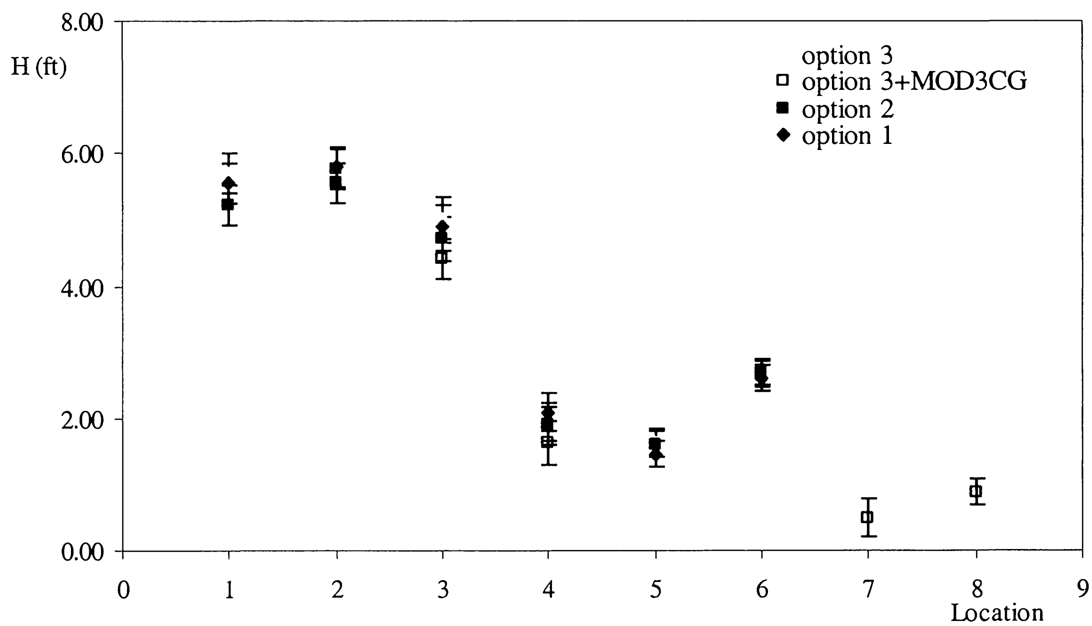

Figure 21. Comparison of the three proposed modifications for a wave period of T = 6s and an incoming wind angle of 280°.

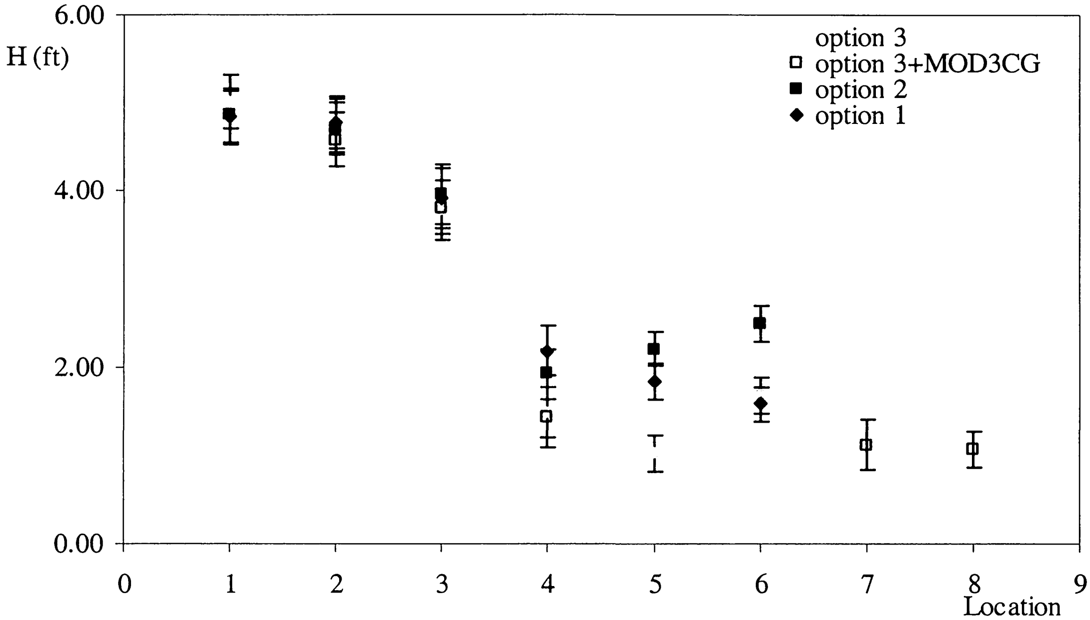

Figure 22. Comparison of the three proposed modifications for a wave period of T = 7s and an incoming wind angle of 280°.

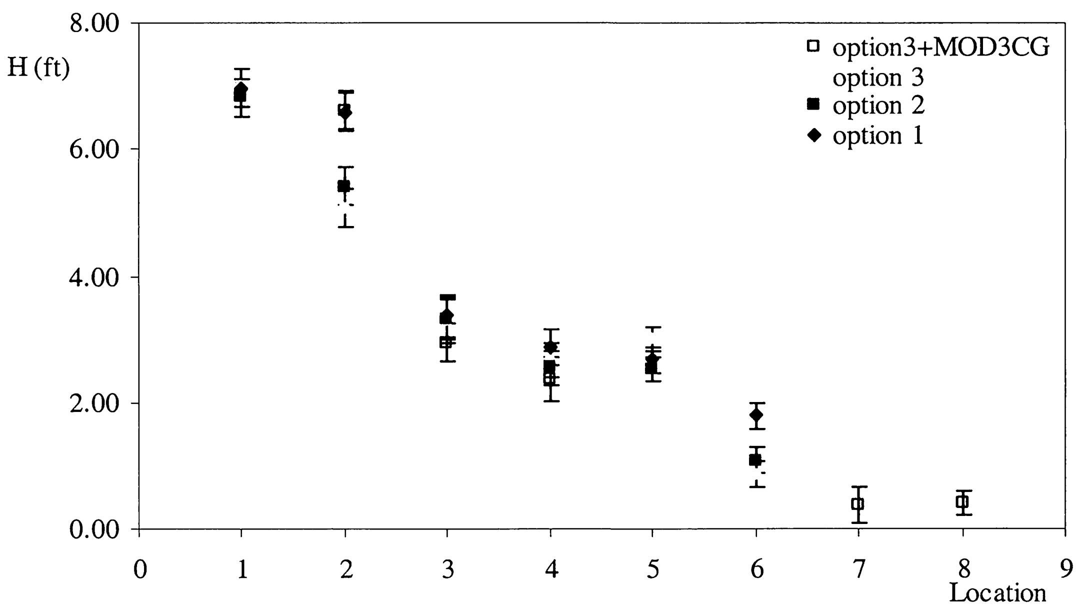

Figure 23. Comparison of the three proposed modifications for a wave period of T = 5s and an incoming wind angle of 260°.

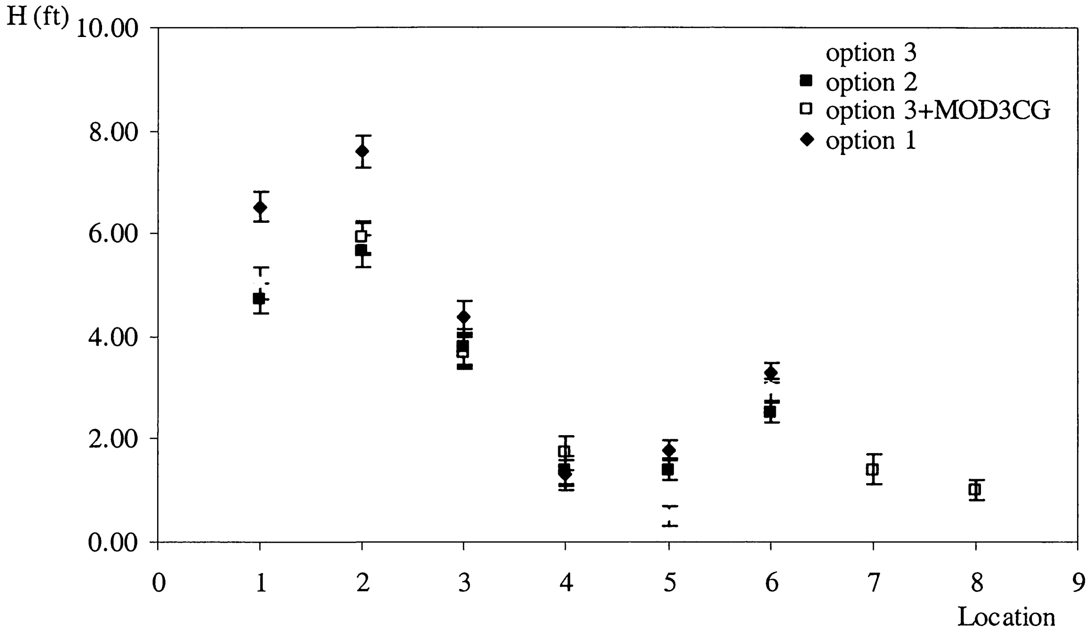

Figure 24. Comparison of the three proposed modifications for a wave period of T = 6s and an incoming wind angle of 260 °.

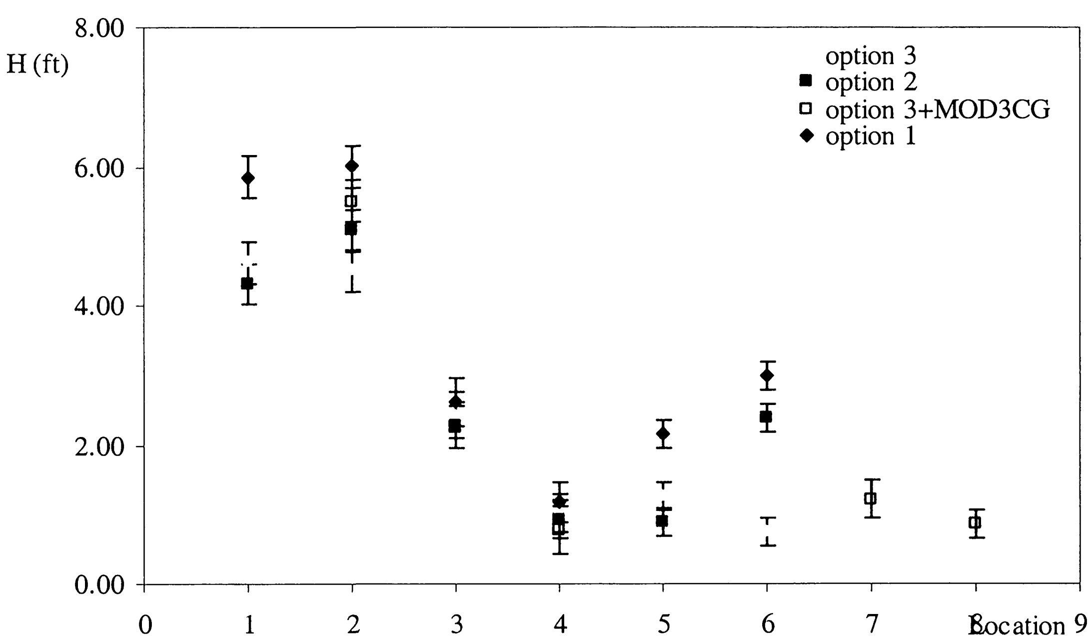

Figure 25. Comparison of the three proposed modifications for a wave period of T = 7s and an incoming wind angle of 260 °.

The wave height ratios are presented in Figures 26 – 31. These ratios are very close to unity at the first two locations for all cases because the incoming waves at these locations have not yet interacted with the harbor modifications or armor stone general. Deviations from unity may be primarily attributable to measurement uncertainty but another factor is that some wave energy is scatted back towards the harbor entrance from the area of the modification. It was possible to verify in a few cases by direct observation that the waves at section 2, for example, were influenced by the particular modification that was being tested. At some particular period, the waves on the north side of the channel near section 2 were observed to break slightly due to the increase in wave height due to the reflection off the north jetty wall. However, this limited wave breaking occurred for only some of the options and did not occur for others, indicating that the wave scattering back towards the entrance varied from option to option. Measurement uncertainty could potentially contribute to a variation in the wave height ratio of approximately ± 0.05 and most of the variations in ratios for a given wave configuration are within that range; a few points are outside that range and are likely due to the differential back-scattering of waves from the various modifications

The variation between options appears to be greater for locations 5 through 8 in the figures. Again part of this variation may be due to experimental uncertainty as the results for these locations involve relatively small wave heights as well as the fact that data from a single location is used to form these ratios (as opposed to the average of three). An estimated uncertainty in the wave height ratios for these locations is about ± 0.15 or three times greater than the locations closer to the harbor entrance. Variations among the options are often more than this range (particularly for the angled waves) indicating that the different options do impact the wave heights at these particular locations. However the results are not consistent from one wave configuration to another and it is difficult to draw definite conclusions. Qualitatively examining all the options for these two locations only, the option 3 appears to produce several situations where significant wave height reduction occurs and option 1 often appears to be the worst. One issue to be kept in mind is that a different selection of measurement locations would possibly have produced different conclusions. To demonstrate this issue, consider the results presented in Figures 26 through 31. The three measurement locations 6 through 8 are within about 150 feet (prototype) of each other. In Figure 29, for example, all three locations yield essentially the same wave height ratios. However, the variation among the three locations for in Figures 30 and 31 is considerably more than can be attributed to experimental uncertainty, indicating that different location performance is observed. The primary reason for this issue is that these measurements are made in locations where both incident and partially reflected waves are interacting and geometrical effects can be very important under these circumstances.

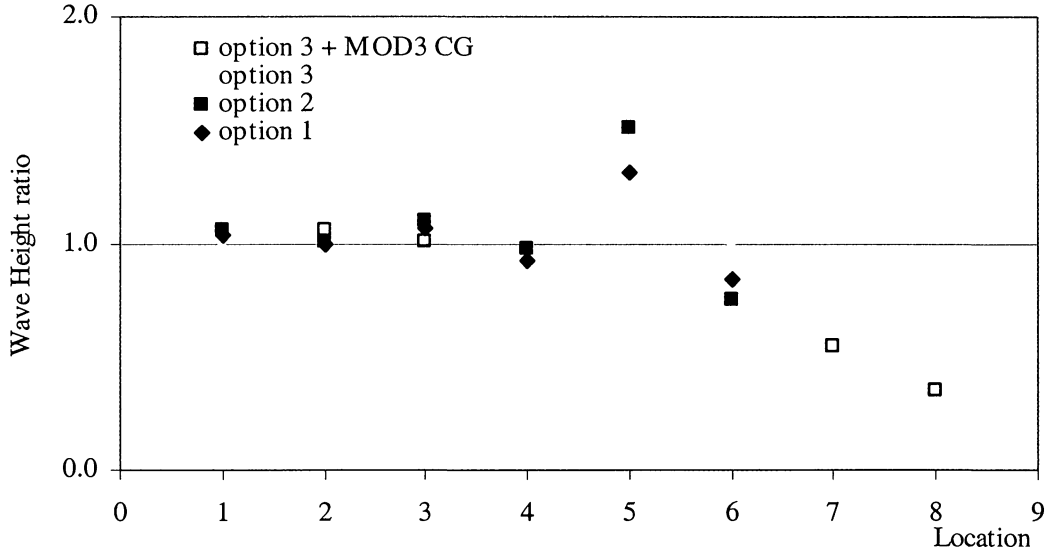

Figure 26. Wave Height Ratio comparison for a wave period of T = 5s and an incoming wave angle of 280°.

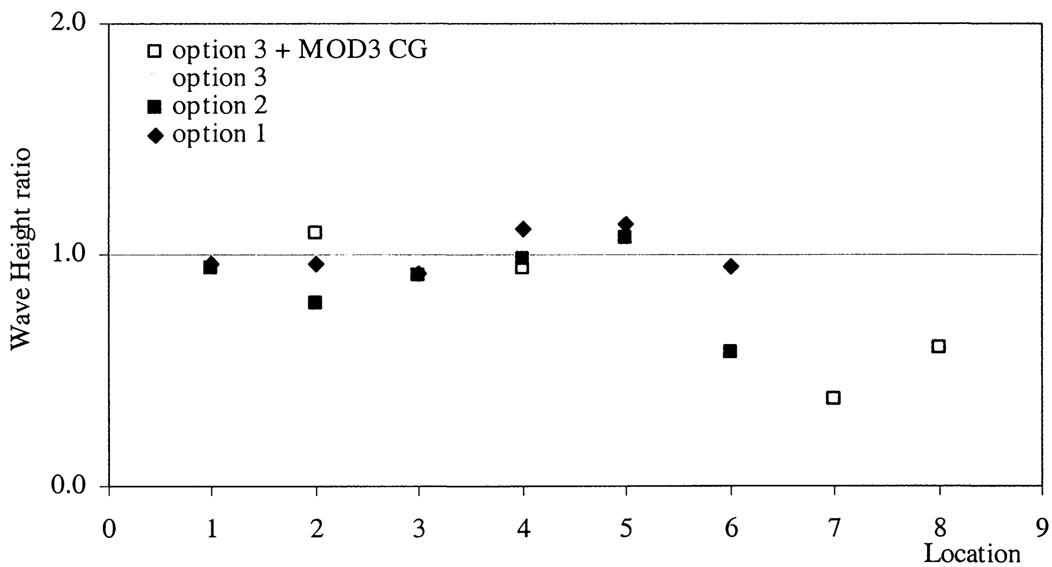

Figure 27. Wave Height Ratio comparison for a wave period of T = 5s and an incoming wave angle of 260°.

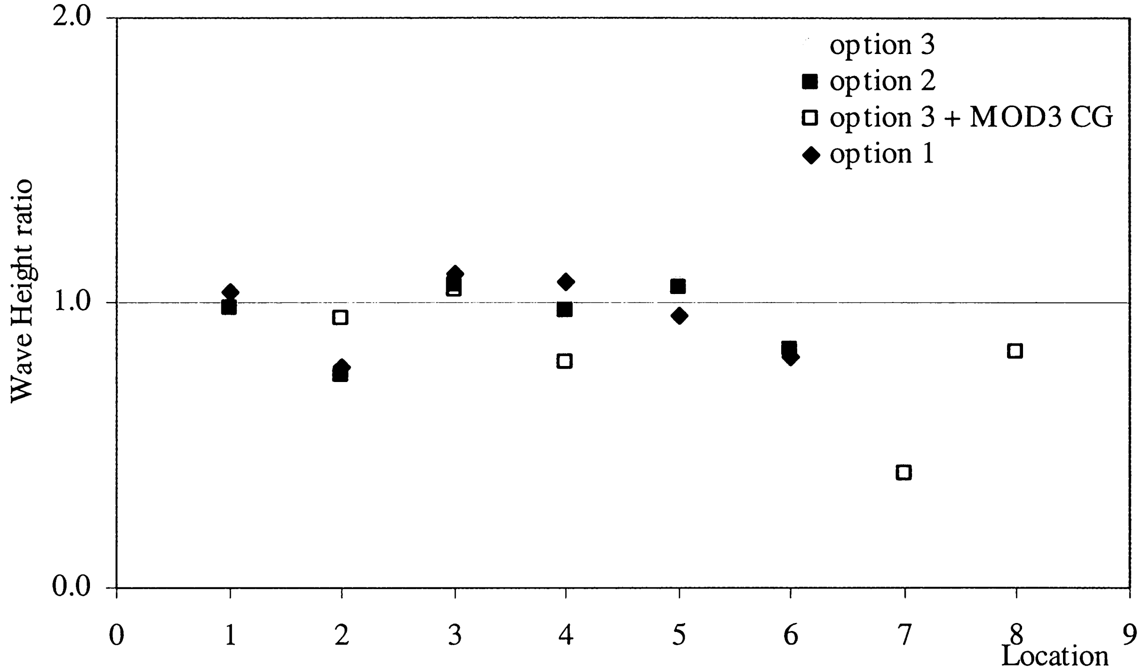

Figure 28. Wave Height Ratio comparison for a wave period of T = 6s and an incoming wave angle of 280°.

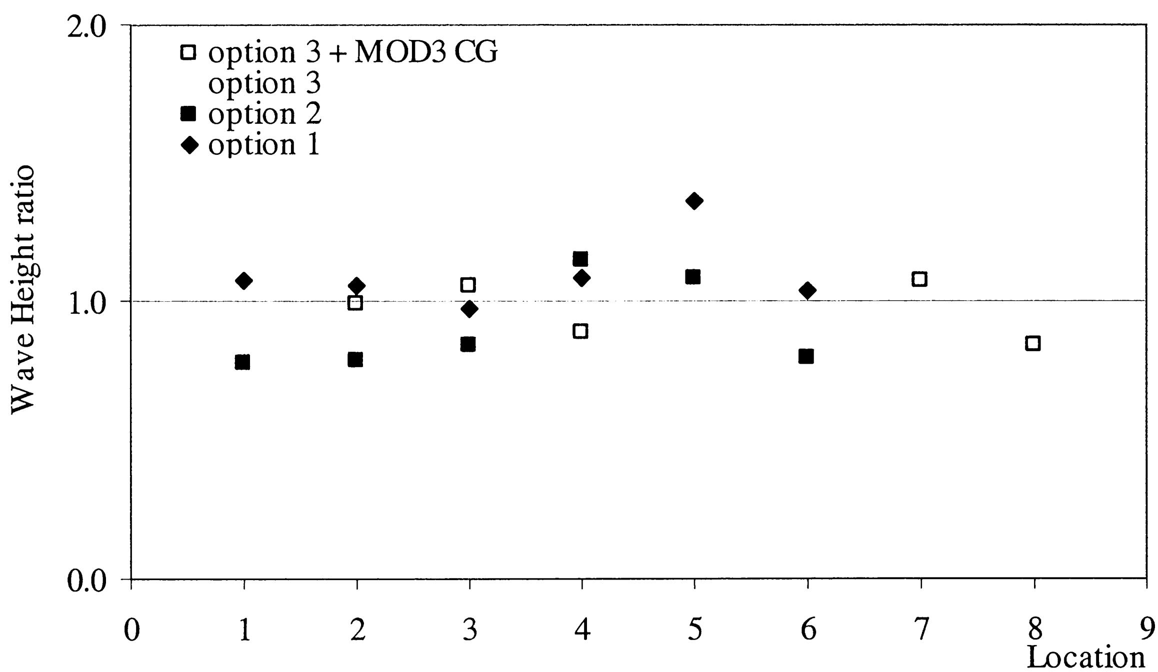

Figure 29. Wave Height Ratio comparison for a wave period of T = 6s and an incoming wave angle of 260°.

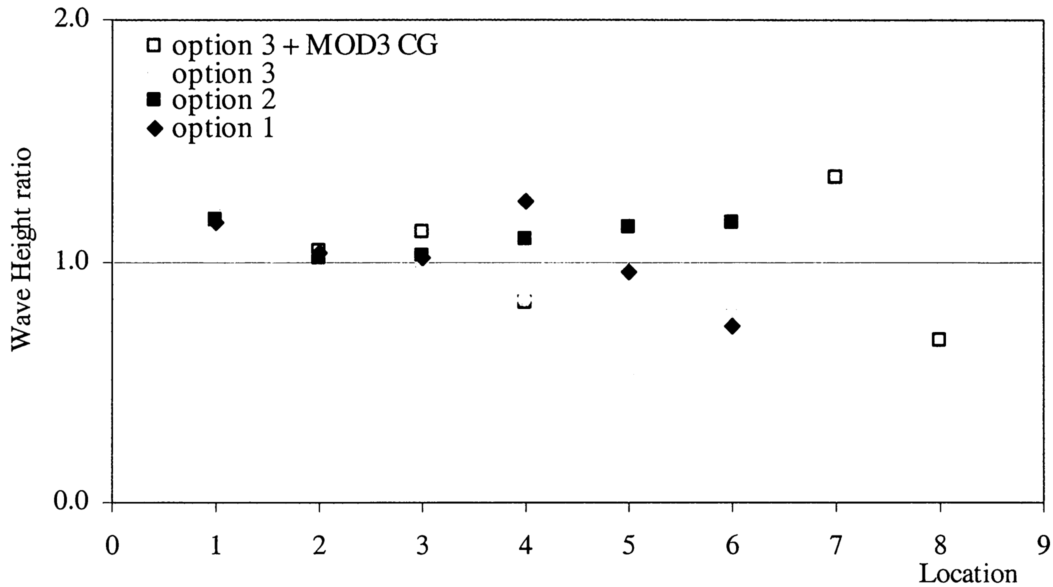

Figure 30. Wave Height Ratio comparison for a wave period of T = 7s and an incoming wave angle of 280°.

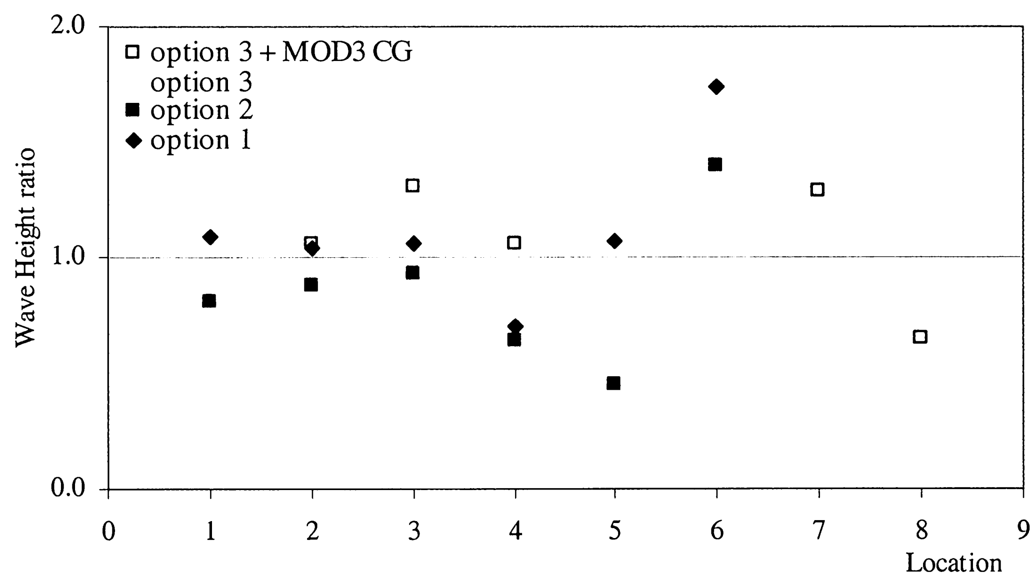

Figure 31. Wave Height Ratio comparison for a wave period of T = 7s and an incoming wave angle of 260°.

More limited measurements were made for the case of a higher water level, primarily to ensure that the general performance at existing water levels could be reasonably extrapolated to higher water level conditions that might be expected to occur in the future. Two wave configurations were studied, namely 6 second waves from a 280 degree angle and 7 second at from a 260 degree angle. The results from these measurements are presented in Figures 32 through 39. The results are presented in two formats; Figures 32 – 37 indicate the measured wave heights at both water levels for each of the options and for both wave configurations while Figures 38 and 39 provide the measured results for the greater water depth presented in terms of wave height ratios. Because of the testing procedure, the wave generator produced different wave heights at the different water levels so it was necessary to measure an "existing" condition for both water levels. The results in Figs. 32 – 37 indicate that while the waves were generally higher within the jetty channel for the higher water level, the interactions with the structures produced waves that were of comparable heights to those measured at lower water levels towards the back of the harbor.

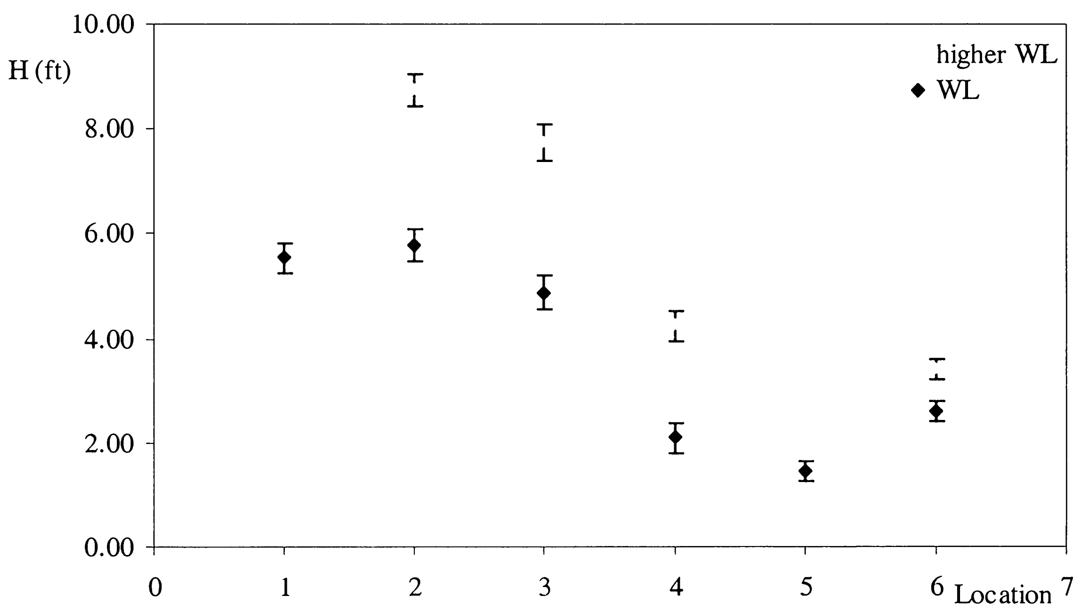

Figure 32. Results for a 4 ft. increase in the free surface water level. Wave period of T = 6s and an incoming wind angle of 280°. Option 1.

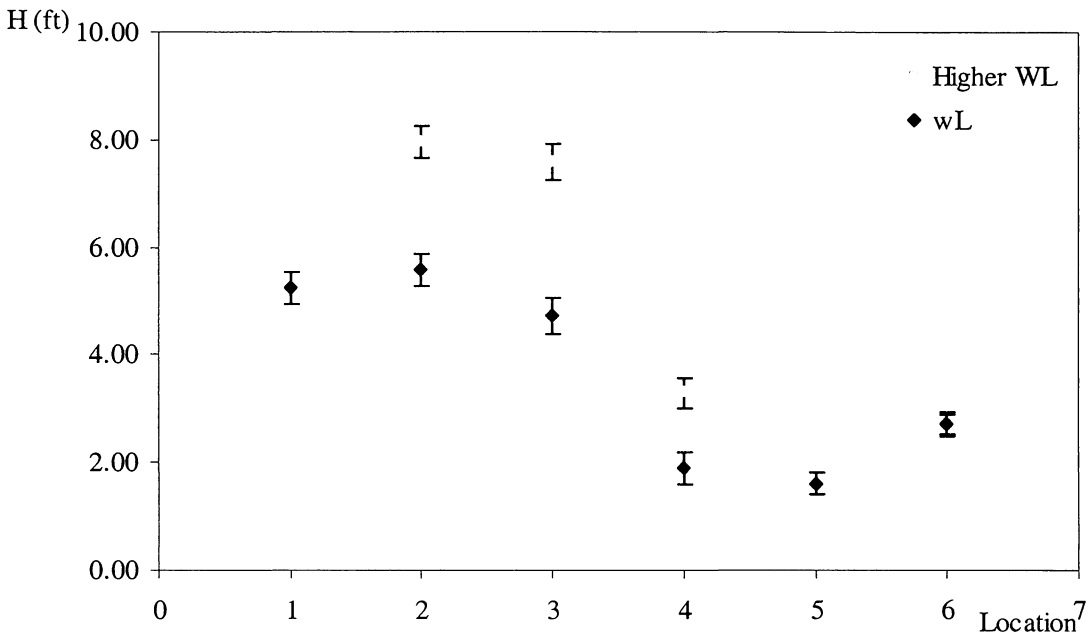

Figure 33. Results for a 4 ft. increase in the free surface water level. Wave period of T = 6s and an incoming wind angle of 280°. Option 2.

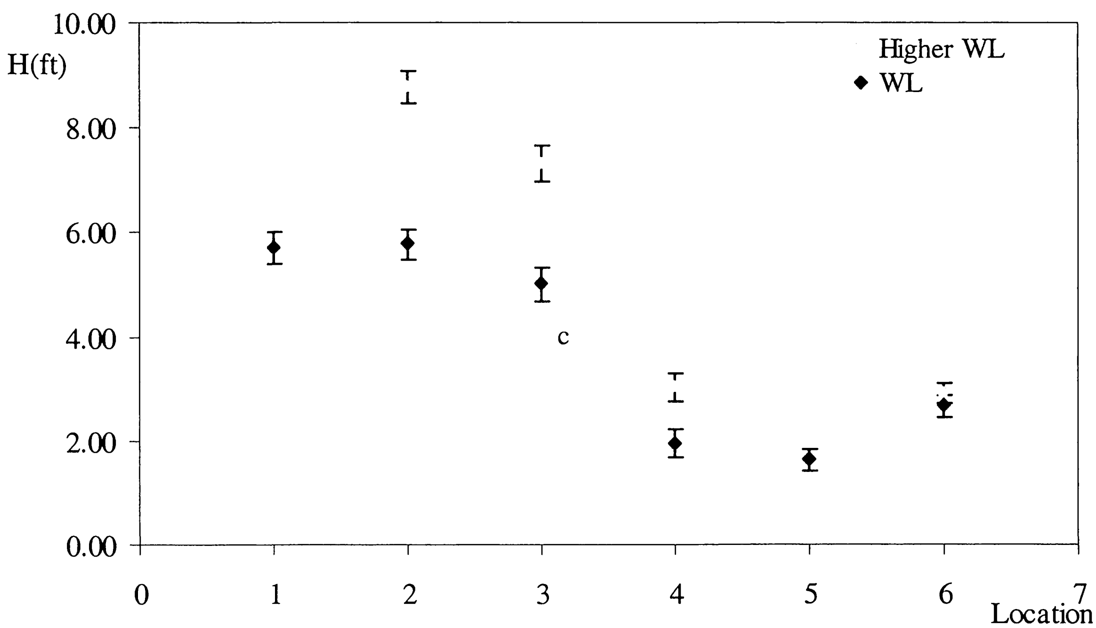

Figure 34. Results for a 4 ft. increase in the free surface water level. Wave period of T = 6s and an incoming wind angle of 280°. Option 3.

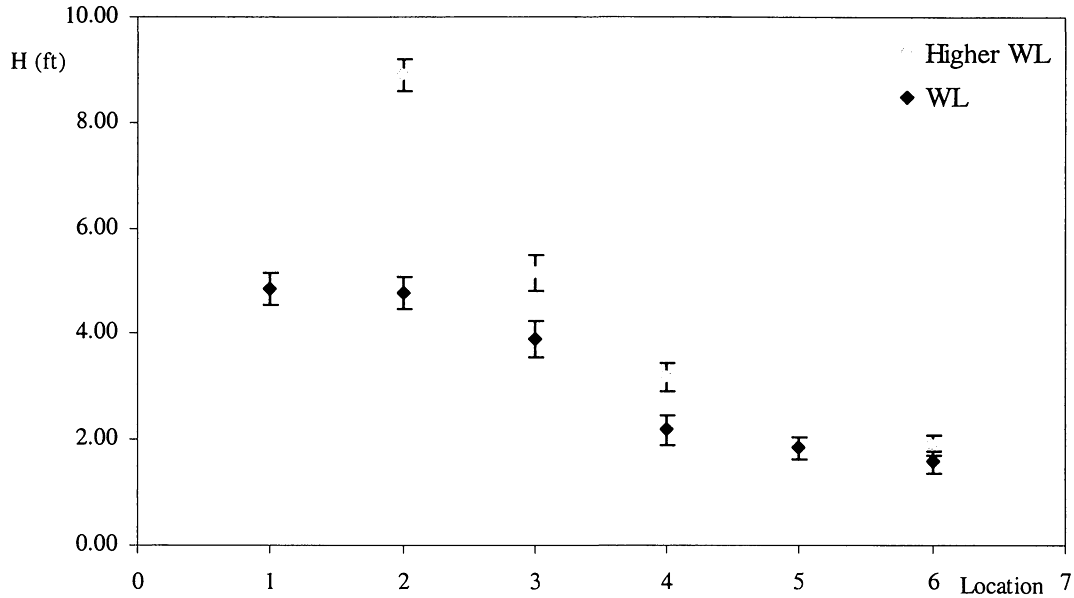

Figure 35. Results for a 4 ft. increase in the free surface water level. Wave period of T = 7s and an incoming wind angle of 260 °. Option 1.

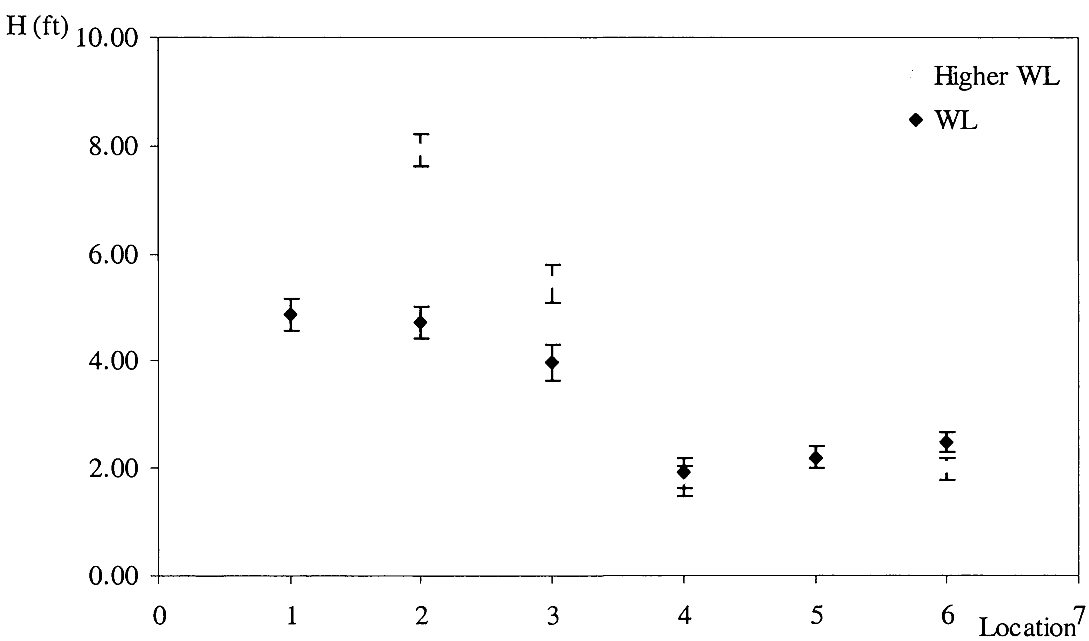

Figure 36. Results for a 4 ft. increase in the free surface water level. Wave period of T = 7s and an incoming wind angle of 260 °. Option 2.

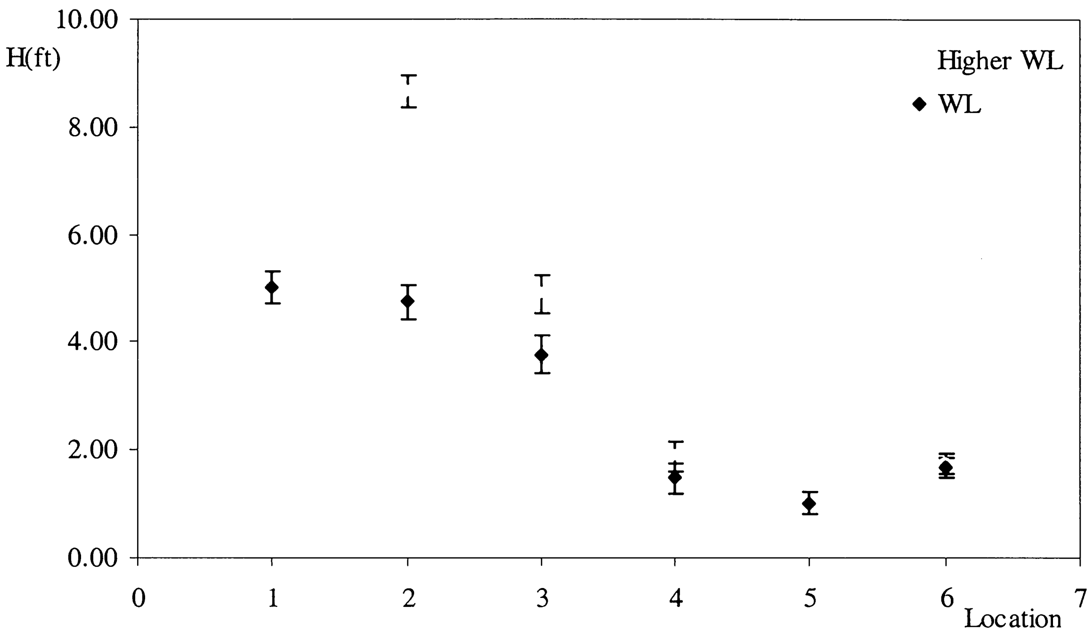

Figure 37. Results for a 4 ft. increase in the free surface water level. Wave period of T = 7s and an incoming wind angle of 260°. Option 3.

Wave height ratios for the higher water level condition are presented in Figures 38 and 39. In general, there is no indication of an increase in wave activity at the conditions tested with the exception of a significant increase in the wave ratio for Option 1 at location 4. A direct comparison of the ratios at the two different water levels leads to equivocal results. For example, raising the water level reduces wave height ratios at locations 4 and 5 for all options with the 6 second wave while the results for the 7 second wave indicates that options 2 and 3 are a little worse at location 4, but option 1 is significantly worse. These conclusions may depend on the specific measurement locations but overall, no trend of a significant deterioration in harbor performance was noted. A better way to assess harbor performance is to compare the OHR for the lower and higher water levels. For example, the OHR for the 6 second wave configuration for both water levels (including only the locations where wave heights were measured for both sets of experiments) is as follows:

6 second, 280 degree waves

	water level	
	Low	high
Option 1	0.95	0.96
Option 2	0.91	0.87
Option 3	0.94	0.88

Although there are minor differences in this quantity between high and low water levels, these are generally within measurement uncertainty but the trends are consistent among the various options.

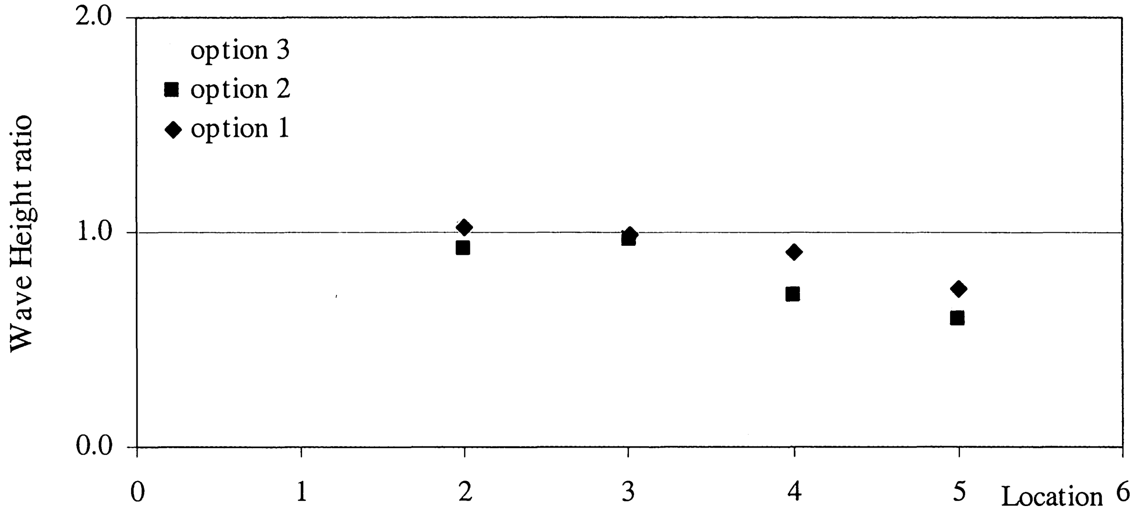

Figure 38. Wave Height ratios for a free surface water level increase of 4 ft. Wave period of T = 6s and an incoming wave angle of 280 °.

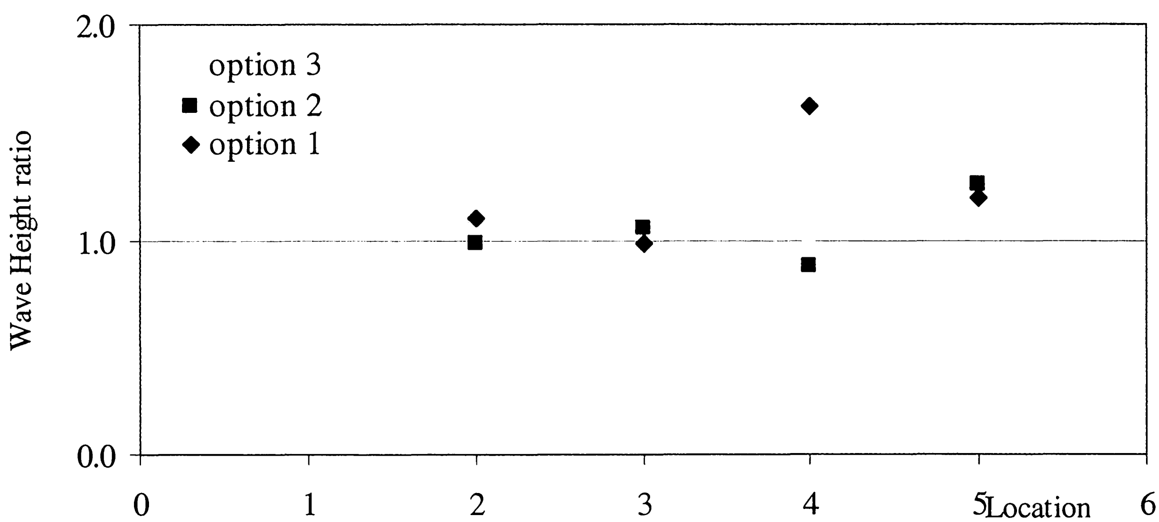

Figure 39. Wave Height ratios for a free surface water level increase of 4 ft. Wave period of T= 7s and an incoming wave angle of 260 °.

A final set of measurements were made in the model while the companion study for the U.S. Coast Guard Station (Wright and Paez, 2004) was being completed. Although there are two independent modifications in close proximity being proposed for the St. Joseph Harbor, there is a strong possibility that both will be implemented. It was decided to test one combination of modifications for both proposed projects simultaneously. The specific choice implemented was Option 3 for the Lighthouse Marina project and Modification 3 for the U.S. Coast Guard project. There were fairly small differences among the measurement results for the various Coast Guard project modifications so the general conclusions should be similar regardless of which modification was actually tested. As mentioned previously, the Coast Guard model study followed this study and required an alteration to the model to better reflect the actual construction details of the Coast Guard breakwater (these were not available at the commencement of this study). In order to ensure consistency in interpretation of model results, a new set of "existing" conditions were measured for the same wave periods and directions. In Figures 40 – 45 where the results are compared to the options without the Coast Guard modification, the wave height ratios are formed from these new existing conditions. Slightly different measurement locations were also used in the Coast Guard study and these locations are indicated in Figure 9. Differences between the initial study with the three Lighthouse Marina options in isolation and the Option 3 with the Coast Guard modification are relatively minor and confirm the expected finding that implementation of the two projects will have no significant effect on wave activity within the harbor compared to a situation in which only one of the projects is implemented.

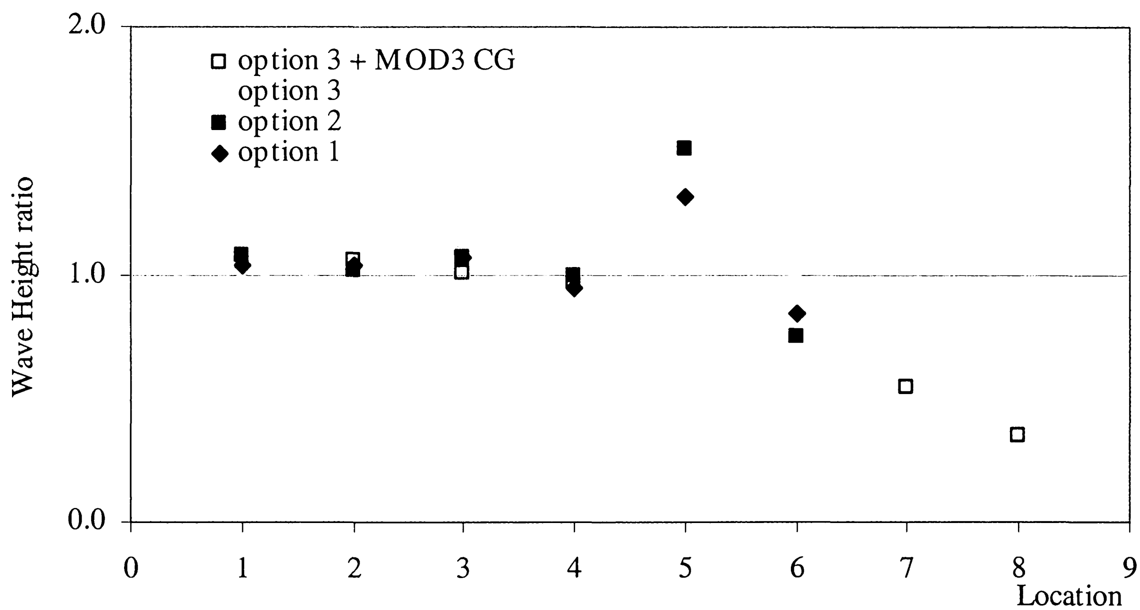

Figure 40. Wave period of T = 5s and an incoming wave angle of 280°.

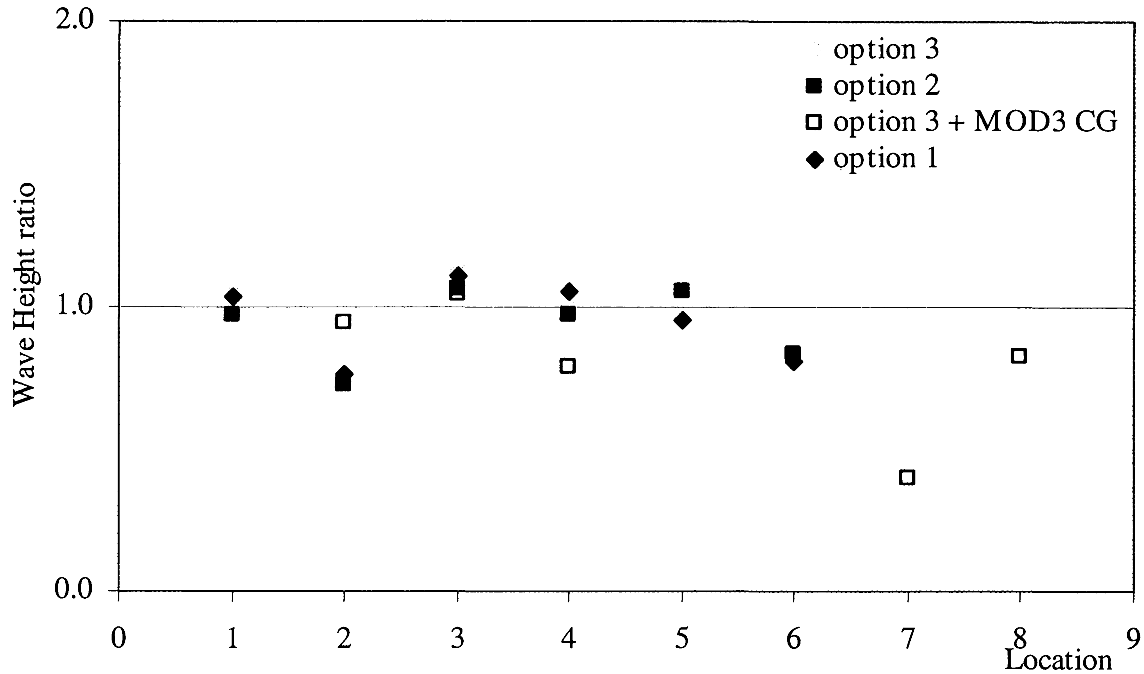

Figure 41. Wave period of T = 6s and an incoming wave angle of 280°.

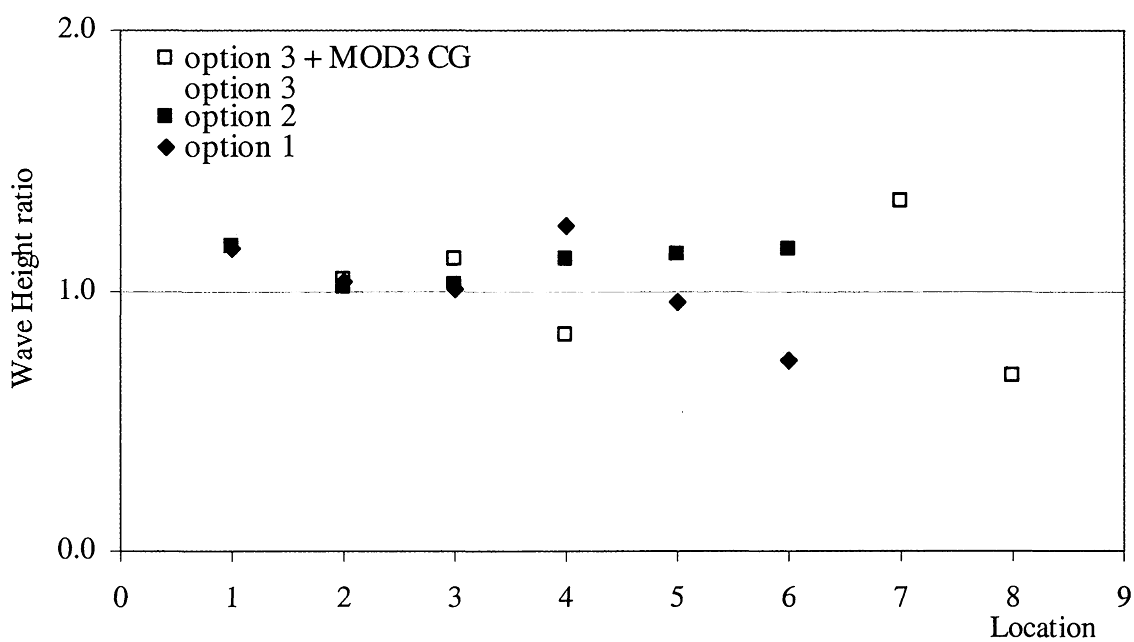

Figure 42. Wave period of T = 7s and an incoming wave angle of 280°.

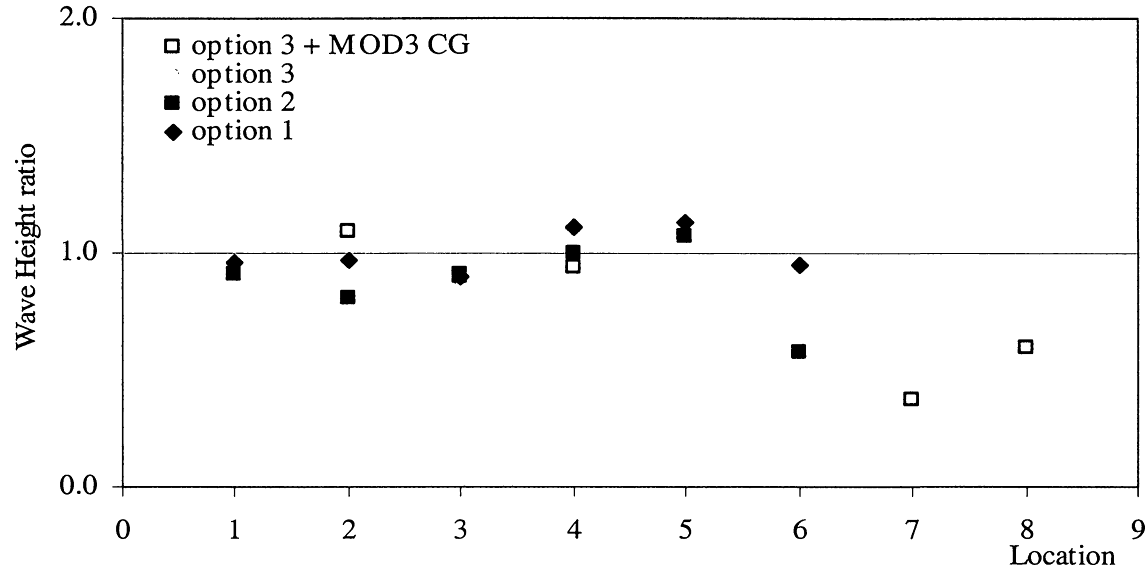

Figure 42. Wave period of T = 5s and an incoming wave angle of 260°.

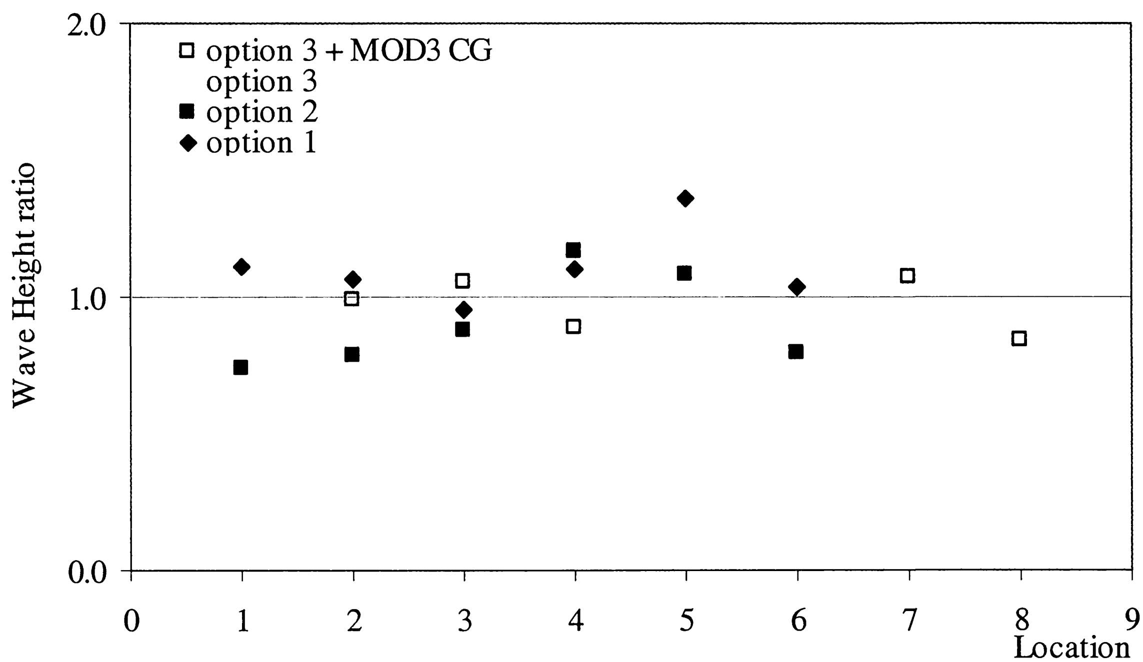

Figure 43. Wave period of T = 6s and an incoming wave angle of 260°.

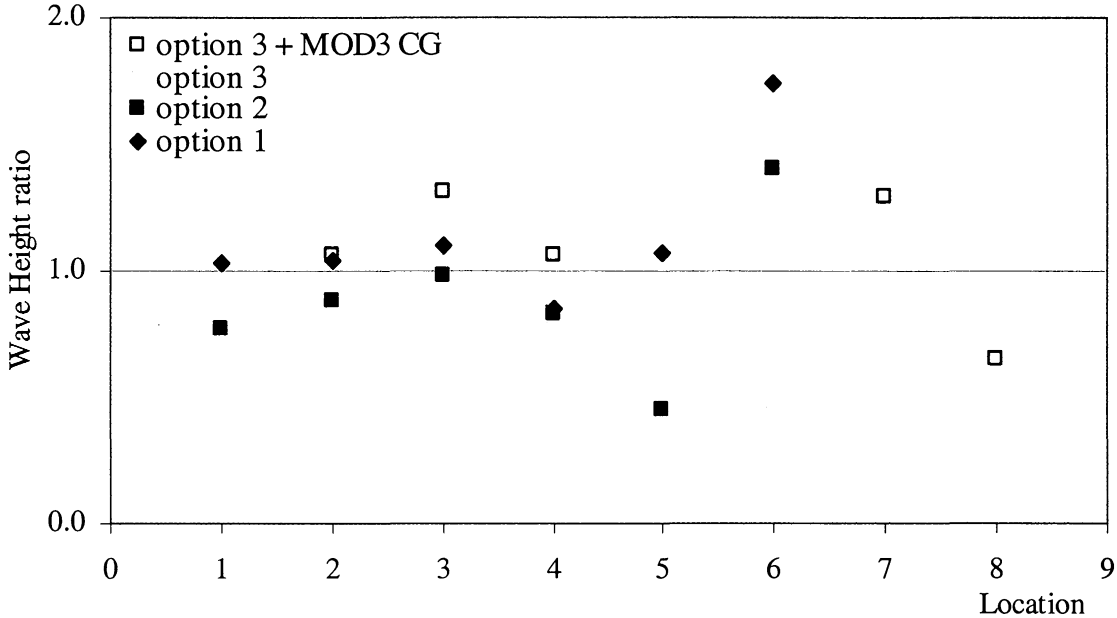

Figure 44. Wave period of T = 7s and an incoming wave angle of 260°.

Late in the testing program, the possibility was raised of increasing the height of the Coast Guard breakwater by a few feet. One final set of experiments was performed for a single wave configuration in which the modification 3 proposed for the Coast Guard breakwater was implemented at a three foot greater crest height. The results for this experiment are presented in Table 7 in terms of a wave height ratio for each measured point. The wave height ratio in Table 7 is the measured wave height at the three foot greater breakwater height compared to the same model conditions except with the existing breakwater height. The results in Table 7 indicate that very small changes in wave height are experienced except on the left and center sides of the channel at location 4 which is where the wave will propagate to after passing the breakwater. It was originally considered that these measurement points might experience greater wave heights due to reflection off a higher breakwater, but the experimental results indicate a decrease in wave heights there. Therefore, it appears that increasing the height of the Coast Guard breakwater will not adversely affect the wave climate within the navigation channel.

Table 7. Experimental results from testing with U.S. Coast Guard breakwater height increased by three feet. Ratio is wave height observed with greater breakwater height compared to wave height with existing breakwater height.

	LEFT	CENTER	RIGHT	Mean
2	0.99	1.07	0.88	0.98
3	0.91	0.74	1.10	0.92
4	0.58	0.58	0.96	0.71
7	0.90			0.90
8	0.81			0.81

CONCLUSIONS

The purpose of this physical hydraulic model study was to determine whether three proposed options for the construction of the boundary between the proposed Lighthouse Marina and the existing navigation channel for the St. Joseph Harbor would result in an unacceptable increase in wave activity that would adversely impact use of the navigation channel. In order to make this determination, the existing harbor configuration was reproduced in the model as well as each of the three proposed options. The primary testing was performed for lake levels that are consistent with existing depths in the navigation channel and the harbor response to incident waves with three different periods and two wave directions was measured at up to 14 different discrete locations. Additionally, more limited testing was performed for some of the same wave conditions at four foot greater water levels and also for one of the options in conjunction with a proposed modification to the U.S. Coast Guard breakwater location across the navigation channel from the proposed Lighthouse Marina. The model response for one of the wave configurations was compared to observations within the harbor during a storm on September 19, 2003 and consistent behavior was noted in the model and actual harbors.

Based on several metrics that compare the wave response with the proposed modification in place and the corresponding existing condition (i.e. at the same water depth, wave period and direction), the following conclusions are obtained:

- The various options tested indicate at most a less than ten percent increase wave activity within the harbor when assessed on the basis of an Overall Harbor Response which is basically a weighted average wave height ratio for all model measurement locations. Over half the individual configurations tested at the low water level condition actually indicate a decrease in wave activity when assessed by this metric;
- Any one of the three options should be acceptable considering the objective of not producing a significant increase in wave activity.
- Among the three options, Options 2 and 3 produced somewhat more favorable wave conditions than Option 1. No significant difference could be detected between Options 2 and 3.
- In terms of individual locations within the harbor, a particular wave configuration may indicate a significant increase in wave height but other wave configurations

may then demonstrate a decrease or only nominal increase in wave height. Based on these findings, there is no indication that adverse wave conditions in the navigation channel will result as a consequence of the Lighthouse Marina development/

- Increasing the water level within the harbor by four feet did not produce a detectable increase in wave height ratios.
- A possible modification to the U.S. Coast Guard breakwater across the navigation channel from the Lighthouse Marina development was incorporated into the model and tested in conjunction with the proposed Option 3. The results from the two modifications are not materially different than alternatives that consider the implementation of only one of the alternatives. Therefore, the execution of both projects will not result in an increase in adverse wave conditions with the St. Joseph Harbor. A more limited set of measurements for one wave configuration only indicates that increasing the height of the U.S. Coast Guard breakwater will not adversely increase wave heights in the navigation channel.

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Hubertz, J. M. D. B. Driver, and R.D. Reinhard, (1991) "*Hindcast Wave Information for the Great Lakes: Lake Michigan*" WIS Report 24, Coastal Engineering Research Center, Vicksburg, Mississippi.

USGS, (2002) *"Michigan Water Resources Data for Water Year 2002"* U.S. Geological Survey Water-Data Report MI-02-1.

Wright, S.J. and D. Paez, (2004) *"Hydraulic Model Study, Saint Joseph Harbor, U.S. Coast Guard Project Investigation"* Report CEE 04-04, Department of Civil and Environmental Engineering, The University of Michigan, Ann Arbor, Michigan.

APPENDIX A

PROBE CALIBRATIONS

Specific Probes were associated with the following measurement locations

Probe	Measurement Locations
A	1 &2
B	3
C	4
D	5 – 8

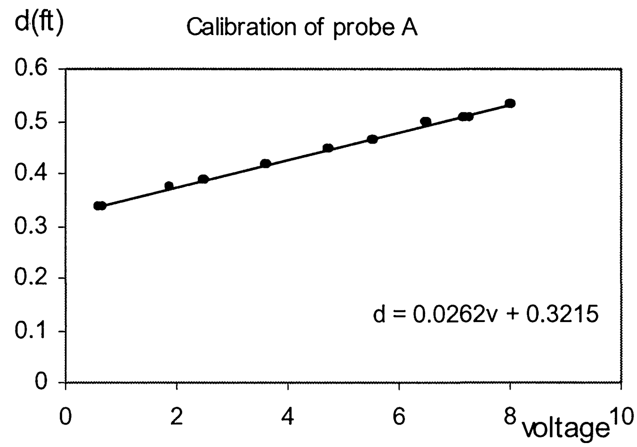

Figure A-1. Calibration Curve for Probe A

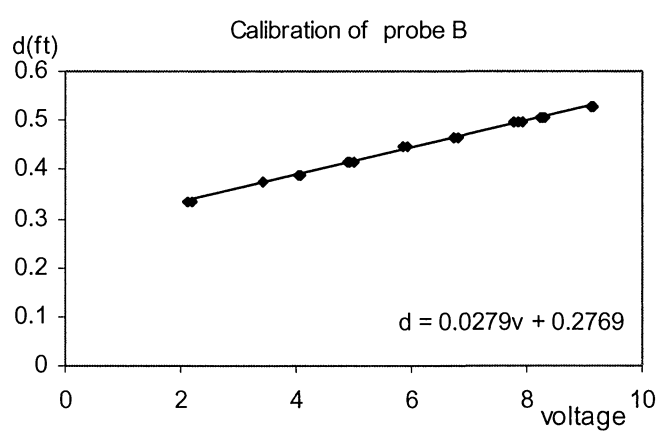

Figure A-2. Calibration Curve for Probe B

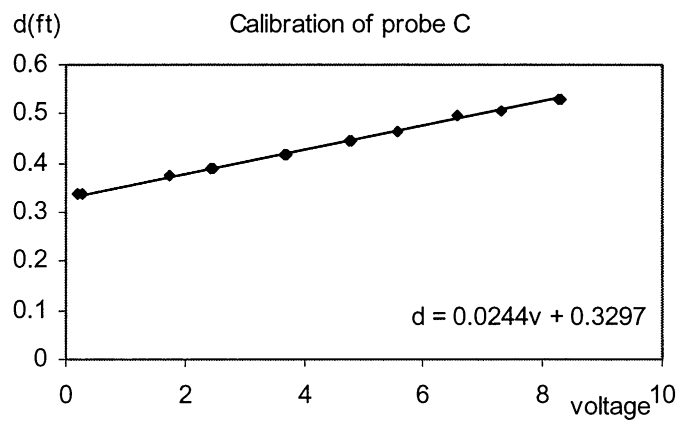

Figure A-2. Calibration Curve for Probe C

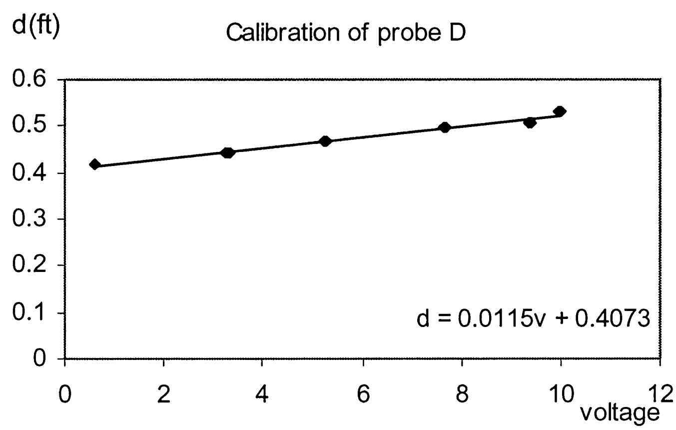

Figure A-3. Calibration Curve for Probe D

APPENDIX B

TABLES OF WAVE HEIGHT MEASUREMENTS FOR ALL EXISTING HARBOR CONFIGURATIONS

Prior to U.S. Coast Guard Breakwater Re-construction

ORIGINAL CONFIGURATION

Incident wave direction: ~280 NW

t = 5 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
1	5.18	4.82	6.23	5.41
2	8.18	6.08	3.51	5.92
3	3.02	5.87	4.34	4.41
4	1.68	3.72	2.83	2.74
5	1.43			1.43
6	2.28			2.28

t = 6 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
1	4.96	6.25	4.82	5.34
2	8.09	7.49	6.92	7.50
3	4.12	4.33	4.84	4.43
4	1.64	2.09	2.12	1.95
5	1.53			1.53
6	3.23			3.23

t = 7 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
1	4.34	4.36	3.78	4.16
2	4.97	5.55	3.34	4.62
3	2.9	4.36	4.31	3.86
4	1.19	2.14	1.93	1.75
5	1.93			1.93
6	2.15			2.15

Incident wave direction: ~265 NW

t = 5 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
1	9.67	8.52	4.58	7.59
2	8.49	2.75	5.87	5.70
3	3.34	6.24	3.97	4.52
4	2.68	4.85	2.38	3.30
5	2.57			2.57
6	1.00			1.00

t = 6 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
1	9.61	6.87	3.76	6.75
2	9.93	6.15	5.96	7.35
3	5.88	2.80	5.15	4.61
4	1.22	1.33	1.29	1.28
5	1.46			1.46
6	3.53			3.53

t = 7 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
1	7.81	4.65	3.51	5.32
2	6.90	5.85	4.69	5.81
3	2.93	3.43	2.58	2.98
4	1.03	1.71	1.07	1.27
5	1.78			1.78
6	3.41			3.41

Incident wave direction: ~260 NW

t = 5 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
1	8.88	7.86	5.13	7.29
2	8.94	6.82	4.8	6.85
3	2.48	5.55	2.99	3.67
4	3.26	3.09	1.43	2.59
5	2.36			2.36
6	1.9			1.90

t = 6 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
1	9.64	5.65	2.97	6.09
2	6.77	7.30	7.57	7.21
3	5.80	3.17	4.50	4.49
4	1.36	1.36	0.89	1.20
5	1.30			1.30
6	3.17			3.17

t = 7 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
1	7.81	4.65	3.56	5.34
2	5.23	6.13	6.04	5.80
3	2.34	3.11	1.95	2.47
4	0.80	2.68	0.90	1.46
5	2.02			2.02
6	1.71			1.71

HIGHER WATER LEVEL (+4ft.)
ORIGINAL CONFIGURATION
Incident wave direction: ~280 NW

t = 6 s.
Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
2	10.42	8.69	6.50	8.54
3	8.49	7.76	7.37	7.87
4	5.36	3.92	5.29	4.86
6	4.62			4.62

Incident wave direction: ~260 NW

t = 7 s.
Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
2	9.22	8.16	6.91	8.09
3	5.02	5.79	4.68	5.16
4	2.58	1.87	1.68	2.04
6	1.60			1.60

Following U.S. Coast Guard Breakwater Re-construction

ORIGINAL CONFIGURATION
COAST GUARD STUDY
Incident wave direction: ~280 NW

t = 5 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
2	7.91	4.51	4.16	5.53
3	2.68	4.87	5.79	4.45
4	2.09	2.94	2.78	2.60
7	0.71			0.71
8	0.54			0.54

t = 6 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
2	9.08	4.45	4.73	6.09
3	3.09	6.13	3.97	4.40
4	1.54	2.17	2.75	2.16
7	1.22			1.22
8	1.07			1.07

t = 7 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
2	5.13	5.48	2.68	4.43
3	2.46	3.00	4.55	3.34
4	1.48	1.90	1.80	1.73
7	0.83			0.83
8	1.60			1.60

Incident wave direction: ~260 NW

t = 5 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
2	7.14	6.62	4.72	6.16
3	1.63	4.41	3.16	3.07
4	2.80	3.14	1.75	2.56
7	1.00			1.00
8	0.68			0.68

t = 6 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
2	5.72	5.06	7.37	6.05
3	3.09	3.43	4.12	3.55
4	2.12	1.82	1.88	1.94
7	1.31			1.31
8	1.19			1.19

t = 7 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
2	4.68	5.19	5.74	5.20
3	1.71	1.65	1.78	1.71
4	0.90	0.61	0.71	0.74
7	0.95			0.95
8	1.32			1.32

APPENDIX C

MEASUREMENT RESULTS FOR ALL MODIFICATIONS TO EXISTING MARINA

Includes both wave heights and wave height ratios (relative to relevant existing condition)

OPTION 1

Incident wave direction: ~280 NW

t = 5 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
1	5.35	5.45	5.97	5.59
2	7.28	6.25	4.17	5.90
3	3.02	5.74	5.33	4.69
4	1.89	3.69	2.02	2.53
5	1.88			
6	1.93			

t = 6 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
1	5.08	6.56	4.98	5.54
2	8.87	4.01	4.50	5.79
3	4.76	5.11	4.78	4.88
4	1.34	2.10	2.85	2.09
5	1.46			
6	2.60			

t = 7 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
1	3.86	5.93	4.71	4.84
2	5.32	5.69	3.32	4.78
3	2.68	4.03	5.02	3.91
4	1.46	2.25	2.82	2.18
5	1.84			
6	1.57			

Incident wave direction: ~265 NW

t = 5 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
1	8.26	7.88	4.37	6.84
2	9.17	4.48	5.50	6.38
3	3.80	7.01	2.10	4.30
4	3.89	3.99	1.72	3.20
5	2.13			
6	1.64			

t = 6 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
1	8.88	6.92	3.68	6.49
2	9.06	6.54	4.39	6.66
3	5.96	2.14	4.83	4.31
4	1.59	1.18	1.72	1.50
5	1.61			
6	3.01			

t = 7 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
1	8.00	4.48	3.41	5.30
2	6.75	6.35	4.60	5.90
3	3.05	2.95	3.01	3.00
4	0.90	1.73	1.76	1.47
5	1.86			
6	3.14			

Incident wave direction: ~260 NW

t = 5 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
1	7.86	8.09	4.96	6.97
2	8.36	6.65	4.73	6.58
3	2.21	5.41	2.51	3.37
4	3.78	3.21	1.63	2.87
5	2.68			
6	1.79			

t = 6 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
1	9.38	6.82	3.39	6.53
2	9.96	7.00	5.84	7.60
3	5.87	2.70	4.52	4.36
4	1.56	1.22	1.12	1.30
5	1.77			
6	3.28			

t = 7 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
1	10.10	4.53	2.91	5.85
2	5.54	6.30	6.19	6.01
3	2.77	2.60	2.49	2.62
4	0.83	1.31	1.40	1.18
5	2.17			
6	2.98			

OPTION 2

Incident wave direction: ~280 NW

t = 5 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
1	6.21	5.58	5.43	5.74
2	8.47	5.57	3.85	5.96
3	2.58	7.04	4.94	4.85
4	2.07	4.09	1.83	2.66
5	2.16			
6	1.70			

t = 6 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
1	4.87	6.58	4.24	5.23
2	8.45	4.06	4.19	5.57
3	4.50	4.75	4.89	4.71
4	1.51	2.16	2.02	1.89
5	1.61			
6	2.70			

t = 7 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
1	4.17	6.08	4.33	4.86
2	5.72	5.29	3.11	4.71
3	2.85	3.97	5.07	3.97
4	1.51	2.00	2.24	1.92
5	2.21			
6	2.49			

Incident wave direction: ~260 NW

t = 5 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
1	9.33	7.03	4.09	6.82
2	7.06	4.72	4.46	5.41
3	2.24	5.26	2.50	3.33
4	2.75	3.36	1.53	2.55
5	2.53			
6	1.09			

t = 6 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
1	7.84	4.62	1.76	4.74
2	5.99	4.92	6.04	5.65
3	3.77	3.05	4.58	3.80
4	1.34	1.65	1.15	1.38
5	1.41			
6	2.51			

t = 7 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
1	7.04	3.90	2.00	4.32
2	4.90	4.92	5.43	5.09
3	1.75	2.19	2.92	2.29
4	0.85	1.00	0.95	0.93
5	0.90			
6	2.39			

OPTION 3

Incident wave direction: ~280 NW

t = 5 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
1	5.04	5.28	6.11	5.48
2	7.69	5.46	4.12	5.76
3	3.41	6.50	4.48	4.80
4	1.95	4.07	2.78	2.94
5	2.07			
6	2.19			

t = 6 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
1	5.58	6.60	4.96	5.71
2	9.11	4.26	3.94	5.77
3	4.94	4.99	5.07	5.00
4	1.73	2.14	1.98	1.95
5	1.63			
6	2.66			

t = 7 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
1	3.92	6.62	4.50	5.01
2	5.67	5.94	2.61	4.74
3	3.17	3.50	4.65	3.77
4	1.32	1.31	1.80	1.48
5	1.02			
6	1.67			

Incident wave direction: ~260 NW

t = 5 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
1	9.33	7.21	4.38	6.97
2	7.20	3.98	4.02	5.07
3	2.53	5.29	2.04	3.29
4	2.75	3.36	1.90	2.67
5	3.00			
6	0.87			

t = 6 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
1	8.18	4.77	2.16	5.03
2	6.74	4.80	6.16	5.90
3	3.89	2.77	4.51	3.72
4	1.56	1.39	1.24	1.40
5	0.51			
6	2.97			

t = 7 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
1	6.92	4.06	2.90	4.63
2	5.40	4.28	3.87	4.51
3	2.72	1.97	2.60	2.43
4	0.70	1.54	0.81	1.02
5	1.27			
6	0.75			

OPTION 3 AT THE MARINA SIDE AND OPTION 3 AT THE COAST GUARD SIDE

Incident wave direction: ~280 NW

t = 5 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
2	8.30	6.11	3.17	5.86
3	2.43	5.96	5.13	4.50
4	2.24	2.75	2.56	2.52
7	0.39			0.39
8	0.19			0.19

t = 6 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
2	8.62	4.34	4.33	5.76
3	3.65	4.99	4.58	4.41
4	1.58	1.46	1.87	1.63
7	0.49			0.49
8	0.88			0.88

t = 7 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
2	5.58	5.28	2.88	4.58
3	2.39	3.51	5.53	3.81
4	1.14	1.56	1.61	1.44
7	1.12			1.12
8	1.07			1.07

Incident wave direction: ~260 NW

t = 5 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
2	7.42	6.62	5.82	6.62
3	1.43	5.31	2.09	2.94
4	2.97	2.48	1.68	2.38
7	0.37			0.37
8	0.41			0.41

t = 6 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
2	6.06	5.18	6.55	5.93
3	3.48	3.80	3.82	3.70
4	1.85	1.68	1.66	1.73
7	1.41			1.41
8	1.00			1.00

t = 7 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
2	4.43	6.77	5.33	5.51
3	2.19	1.99	2.60	2.26
4	0.71	0.68	0.92	0.77
7	1.22			1.22
8	0.87			0.87

Increase Height of Coast Guard breakwater by 3 ft

OPTION 3 AT THE MARINA SIDE AND OPTION 3 AT THE COAST GUARD SID
3FT INCREASE IN BREAKWATER HEIGHT
Incident wave direction: ~280 NW
t = 6 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
2	8.50	4.67	3.82	5.66
3	3.33	3.72	5.02	4.02
4	0.92	0.85	1.80	1.19
7	0.44			0.44
8	0.71			0.71

HIGHER WATER LEVEL (+4ft.)

Incident wave direction: ~280 NW

OPTION 1

t = 6 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
2	10.78	8.94	6.52	8.75
3	8.40	6.77	8.03	7.73
4	2.85	4.92	4.94	4.24
6	3.41			3.41

OPTION 2

t = 6 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
2	11.03	6.81	6.01	7.95
3	8.30	6.94	7.48	7.57
4	2.32	3.77	3.82	3.30
6	2.73			2.73

OPTION 3

t = 6 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
2	11.15	8.62	6.58	8.79
3	7.23	7.06	7.69	7.33
4	2.75	3.48	2.83	3.02
6	2.90			2.90

Incident wave direction: ~260 NW

t = 7 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
2	9.79	9.16	7.73	8.89
3	5.23	6.65	3.61	5.16
4	2.90	3.43	3.21	3.18
6	1.90			1.90

t = 7 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
2	8.89	8.13	6.72	7.91
3	5.65	5.67	5.02	5.45
4	2.29	1.54	1.51	1.78
6	2.00			2.00

t = 7 s.

Wave heights (ft)

	LEFT	CENTER	RIGHT	Mean
2	9.28	8.96	7.79	8.68
3	5.43	5.53	3.72	4.89
4	1.63	1.93	2.07	1.88
6	1.75			1.75

Wave Height Ratios

OPTION 1

Incident wave direction: ~280 NW

t = 5 s.

RATIOS

	LEFT	CENTER	RIGHT	Mean
1	1.03	1.13	0.96	1.03
2	0.89	1.03	1.19	1.00
3	1.00	0.98	1.23	1.06
4	1.12	0.99	0.71	0.92
5	1.32			1.32
6	0.85			0.85

t = 6 s.

RATIOS

	LEFT	CENTER	RIGHT	Mean
1	1.02	1.05	1.03	1.04
2	1.10	0.53	0.65	0.77
3	1.16	1.18	0.99	1.10
4	0.81	1.00	1.34	1.07
5	0.95			0.95
6	0.81			0.81

t = 7 s.

RATIOS

	LEFT	CENTER	RIGHT	Mean
1	0.89	1.36	1.25	1.16
2	1.07	1.02	0.99	1.03
3	0.92	0.92	1.16	1.01
4	1.23	1.05	1.46	1.24
5	0.95			0.95
6	0.73			0.73

Incident wave direction: ~260 NW

t = 5 s.

RATIOS

	LEFT	CENTER	RIGHT	Mean
1	0.88	1.03	0.97	0.96
2	0.93	0.97	0.99	0.96
3	0.89	0.97	0.84	0.92
4	1.16	1.04	1.14	1.11
5	1.14			1.14
6	0.94			0.94

t = 6 s.

RATIOS

	LEFT	CENTER	RIGHT	Mean
1	0.97	1.21	1.14	1.07
2	1.47	0.96	0.77	1.05
3	1.01	0.85	1.01	0.97
4	1.15	0.90	1.26	1.08
5	1.36			1.36
6	1.04			1.04

t = 7 s.

RATIOS

	LEFT	CENTER	RIGHT	Mean
1	1.29	0.97	0.82	1.09
2	1.06	1.03	1.03	1.04
3	1.18	0.84	1.28	1.06
4	1.04	0.49	1.03	0.70
5	1.07			1.07
6	1.74			1.74

OPTION 2

Incident wave direction: ~280 NW

t = 5 s.

RATIOS

	LEFT	CENTER	RIGHT	Mean
1	1.20	1.16	0.87	1.06
2	1.04	0.92	1.10	1.01
3	0.85	1.20	1.14	1.10
4	1.23	1.10	0.65	0.97
5	1.51			1.51
6	0.74			0.74

t = 6 s.

RATIOS

	LEFT	CENTER	RIGHT	Mean
1	0.98	1.05	0.88	0.98
2	1.04	0.54	0.61	0.74
3	1.09	1.10	1.01	1.06
4	0.92	1.03	0.95	0.97
5	1.05			1.05
6	0.84			0.84

t = 7 s.

RATIOS

	LEFT	CENTER	RIGHT	Mean
1	0.96	1.39	1.14	1.17
2	1.15	0.95	0.93	1.02
3	0.98	0.91	1.18	1.03
4	1.27	0.94	1.16	1.09
5	1.14			1.14
6	1.16			1.16

Incident wave direction: ~260 NW

t = 5 s.

RATIOS

	LEFT	CENTER	RIGHT	Mean
1	1.05	0.89	0.80	0.94
2	0.79	0.69	0.93	0.79
3	0.90	0.95	0.84	0.91
4	0.84	1.09	1.07	0.98
5	1.07			1.07
6	0.57			0.57

t = 6 s.

RATIOS

	LEFT	CENTER	RIGHT	Mean
1	0.81	0.82	0.59	0.78
2	0.88	0.67	0.80	0.78
3	0.65	0.96	1.02	0.85
4	0.99	1.21	1.30	1.15
5	1.08			1.08
6	0.79			0.79

t = 7 s.

RATIOS

	LEFT	CENTER	RIGHT	Mean
1	0.90	0.84	0.56	0.81
2	0.94	0.80	0.90	0.88
3	0.75	0.70	1.50	0.93
4	1.06	0.37	1.06	0.64
5	0.45			0.45
6	1.40			1.40

OPTION 3

Incident wave direction: ~280 NW

t = 5 s.

RATIOS

	LEFT	CENTER	RIGHT	Mean
1	0.97	1.09	0.98	1.01
2	0.94	0.90	1.17	0.97
3	1.13	1.11	1.03	1.09
4	1.16	1.09	0.98	1.07
5	1.45			1.45
6	0.96			0.96

t = 6 s.

RATIOS

	LEFT	CENTER	RIGHT	Mean
1	1.13	1.06	1.03	1.07
2	1.13	0.57	0.57	0.77
3	1.20	1.15	1.05	1.13
4	1.06	1.02	0.93	1.00
5	1.06			1.06
6	0.82			0.82

t = 7 s.

RATIOS

	LEFT	CENTER	RIGHT	Mean
1	0.90	1.52	1.19	1.20
2	1.14	1.07	0.78	1.03
3	1.09	0.80	1.08	0.98
4	1.11	0.61	0.93	0.84
5	0.53			0.53
6	0.78			0.78

Incident wave direction: ~260 NW

t = 5 s.

RATIOS

	LEFT	CENTER	RIGHT	Mean
1	1.05	0.92	0.85	0.96
2	0.80	0.58	0.84	0.74
3	1.02	0.95	0.68	0.89
4	0.84	1.09	1.33	1.03
5	1.27			1.27
6	0.46			0.46

t = 6 s.

RATIOS

	LEFT	CENTER	RIGHT	Mean
1	0.85	0.84	0.73	0.83
2	1.00	0.66	0.81	0.82
3	0.67	0.87	1.00	0.83
4	1.15	1.02	1.40	1.16
5	0.39			0.39
6	0.94			0.94

t = 7 s.

RATIOS

	LEFT	CENTER	RIGHT	Mean
1	0.89	0.87	0.82	0.87
2	1.03	0.70	0.64	0.78
3	1.16	0.63	1.33	0.98
4	0.87	0.58	0.91	0.70
5	0.63			0.63
6	0.44			0.44

HIGHER WATER LEVEL (+4ft.)
Incident wave direction: ~280 NW

OPTION 1
t = 6 s.
RATIOS

	LEFT	CENTER	RIGHT	Mean
2	1.03	1.03	1.00	1.02
3	0.99	0.87	1.09	0.98
4	0.53	1.26	0.93	0.91
6	0.74			0.74

OPTION 2
RATIOS

	LEFT	CENTER	RIGHT	Mean
2	1.06	0.78	0.92	0.92
3	0.98	0.89	1.02	0.96
4	0.43	0.96	0.72	0.71
6	0.59			0.59

OPTION 3
RATIOS

	LEFT	CENTER	RIGHT	Mean
2	1.07	0.99	1.01	1.03
3	0.85	0.91	1.04	0.94
4	0.51	0.89	0.54	0.65
6	0.63			0.63

HIGHER WATER LEVEL (+4ft.)
Incident wave direction: ~260 NW

OPTION 1
t = 7 s.
RATIOS

	LEFT	CENTER	RIGHT	Mean
2	1.06	1.12	1.12	1.10
3	1.04	1.15	0.77	0.99
4	1.13	1.84	1.91	1.62
6	1.19			1.19

OPTION 2
RATIOS

	LEFT	CENTER	RIGHT	Mean
2	0.97	1.00	0.97	0.98
3	1.13	0.98	1.07	1.06
4	0.89	0.83	0.90	0.87
6	1.26			1.26

OPTION 3
RATIOS

	LEFT	CENTER	RIGHT	Mean
2	1.01	1.10	1.13	1.08
3	1.08	0.96	0.79	0.94
4	0.63	1.04	1.23	0.97
6	1.10			1.10

OPTION 3 AT THE MARINA SIDE AND OPTION 3 AT THE COAST GUARD SIDE

Incident wave direction: ~280 NW

t = 5 s.

RATIOS

	LEFT	CENTER	RIGHT	Mean
2	1.05	1.35	0.76	1.06
3	0.91	1.22	0.89	1.00
4	1.07	0.94	0.92	0.98
7	0.55			0.55
8	0.34			0.34

t = 6 s.

RATIOS

	LEFT	CENTER	RIGHT	Mean
2	0.95	0.98	0.91	0.95
3	1.18	0.81	1.15	1.05
4	1.02	0.67	0.68	0.79
7	0.40			0.40
8	0.83			0.83

t = 7 s.

RATIOS

	LEFT	CENTER	RIGHT	Mean
2	1.09	0.96	1.08	1.04
3	0.97	1.17	1.22	1.12
4	0.77	0.82	0.90	0.83
7	1.35			1.35
8	0.67			0.67

Incident wave direction: ~260 NW

t = 5 s.

RATIOS

	LEFT	CENTER	RIGHT	Mean
2	1.04	1.00	1.23	1.09
3	0.88	1.20	0.66	0.91
4	1.06	0.79	0.96	0.94
7	0.37			0.37
8	0.60			0.60

t = 6 s.

RATIOS

	LEFT	CENTER	RIGHT	Mean
2	1.06	1.02	0.89	0.99
3	1.13	1.11	0.93	1.05
4	0.87	0.93	0.88	0.89
7	1.08			1.08
8	0.84			0.84

t = 7 s.

RATIOS

	LEFT	CENTER	RIGHT	Mean
2	0.95	1.30	0.93	1.06
3	1.28	1.21	1.46	1.31
4	0.79	1.11	1.29	1.06
7	1.29			1.29
8	0.65			0.65

Increase Height of Coast Guard breakwater by 3 ft

Wave height ratio compared to existing conditions

OPTION 3 AT THE MARINA SIDE AND OPTION 3 AT THE COAST GUARD SIDE.
3FT INCREASE IN BREAKWATER HEIGHT
Incident wave direction: ~280 NW
t = 6 s.

	LEFT	CENTER	RIGHT	MEAN
2	0.94	1.05	0.81	0.93
3	1.08	0.61	1.26	0.98
4	0.59	0.39	0.65	0.55
7	0.36			0.36
8	0.67			0.67

Wave height ratio compared to results with lower breakwater height

OPTION 3 AT THE MARINA SIDE AND OPTION 3 AT THE COAST GUARD SIDE.
3FT INCREASE IN BREAKWATER HEIGHT
Incident wave direction: ~280 NW
t = 6 s.

	LEFT	CENTER	RIGHT	Mean
2	0.99	1.07	0.88	0.98
3	0.91	0.74	1.10	0.92
4	0.58	0.58	0.96	0.71
7	0.90			0.90
8	0.81			0.81

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AIIM SCANNER TEST CHART # 2

Spectra

4 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
6 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
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Times Roman

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8 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
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Century Schoolbook Bold

4 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
6 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
8 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
10 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789

News Gothic Bold Reversed

4 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
6 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
8 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
10 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789

Bodoni Italic

4 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
6 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
8 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
10 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789

Greek and Math Symbols

4 PT ΑΒΓΔΕΞΘΗΙΚΛΜΝΟΠΦΡΣΤΥΩΧΨΖαβγδεξθηικλμνοπφρστυωχψζ≧∓",./≦±=≠°><≯≮>⩽≡
6 PT ΑΒΓΔΕΞΘΗΙΚΛΜΝΟΠΦΡΣΤΥΩΧΨΖαβγδεξθηικλμνοπφρστυωχψζ≧∓",./≦±=≠°><≯≮>⩽≡
8 PT ΑΒΓΔΕΞΘΗΙΚΛΜΝΟΠΦΡΣΤΥΩΧΨΖαβγδεξθηικλμνοπφρστυωχψζ≧∓",./≦±=≠°><≯≮>⩽≡
10 PT ΑΒΓΔΕΞΘΗΙΚΛΜΝΟΠΦΡΣΤΥΩΧΨΖαβγδεξθηικλμνοπφρστυωχψζ≧∓",./≦±=≠°><≯≮>⩽≡

White

Black

Isolated Characters

e	m	1	2	3	a
4	5	6	7	o	•
8	9	0	h	I	B

0123456 65432

A4 Page 6543210

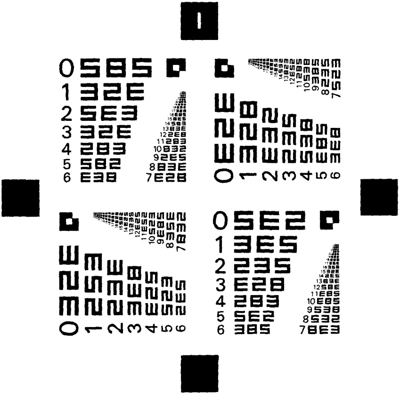
MESH	HALFTONE WEDGES
65	
85	
100	
110	
133	
150	

A4 Page 6543210

0123456 65432

654321 654321 654321

RIT ALPHANUMERIC RESOLUTION TEST OBJECT, RT-1-71



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