Swift Contract No. 69606

MINNESOTA DEPARTMENT
OF NATURAL RESOURCES

Grand Marais
Municipal Campground
Water Access Project

Wave Numerical Model
Study

Prepared For:

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Minnesota
DEPARTMENT OF
NATURAL RESOURCES

Consultant's Report

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Executive Summary

The boat launch at Grand Marais currently experiences agitation levels in excess of comfortable launching/retrieval operations. Due to the unpredictable nature of Lake Superior, concerns exist over the safety of the boat launch during storm events. As the upland area adjacent to the launch is slated for redevelopment, it is the interest of Minnesota Department of Natural Resources to explore alternatives for decreasing the agitation within the area of the boat launch.

The objective of this study was to develop alternatives which would provide sheltering from waves entering Grand Marais Bay. To do this, SmithGroupJJR developed a local wave hindcast using historical data from an offshore virtual wave gauge station and used tandem advanced numerical models to determine the wave environment at the project site for a given return period event. Boundary conditions including wave reflection and diffraction caused by existing structures and shorelines in and around the project area were considered as part of the numeric modeling effort.

While there are no standards or guidelines for agitation at boat launches (as the risk of use is generally left to the discretion of the owner or user), it can be surmised that due to the size of vessels which utilize the boat launch at Grand Marais and resonance amplification that can cause excessive heave in small vessels, the limit of safe use of the boat launch is an agitation of 0.5 feet. Using this threshold, four design condition events related to the 1 month, 6 month, 1 year, and 20 year storm events were modelled. Typical boater practice would suggest that waves greater than 4 feet at the mouth of Grand Marais Bay would essentially eliminate the use of the boat launch.

Three alternative breakwater designs were reviewed. Using the 0.5 feet agitation limit, the required length of breakwater to reducing incoming wave agitation could be defined. Based on the resulting agitation and the quantity of rubble required to construct the breakwater, an alternative, titled "Extended Breakwater" was chosen. This alternative extended the existing rubble groin to wrap around the boat launch creating a sheltered bay. A conceptual plan and cross section of this breakwater were developed based on land construction practices and is presented herein. The overall volume for this construction is estimated to be 1,250 cubic yards.

In addition to the wave agitation study, the inundation of the upland associated with an extreme 50 year storm event was investigated. Using a smooth particle hydrodynamic model, the upland inundation was predicted to be 5.5 feet above the low water datum which would flood approximately 250 feet of upland. It is therefore suggested that all potable water and sewer systems be placed outside of this area.

A review of sediment load was performed over concerns that a perennial stream outfall located to the north of the boat launch may cause sediment buildup on or in front of the launch. Due to the heavily forested areas within the drainage basin which feeds this stream, it is unlikely that any appreciable amount of sediment reaches the bay. Therefore, the alternative proposed will not alter the current sediment dynamics at the project site.

The purpose of this report is to summarize the technical methods used to develop and test the three breakwater alternatives that were developed in this study. Much of the modelling performed within this analysis is highly technical in nature. The appendices include pictorial results from a number of model runs. These images are intended to document the process which was followed and to provide individuals conducting future studies, models, and engineering easy access to the assumptions and model parameters which were used in this phase of the analysis.

Introduction

This report documents the in-depth study of the agitation levels with the Grand Marais Harbor. This work consisted of reviewing and analyzing the existing local and regional environmental conditions that impact the Grand Marais harbor and water access at the site. Using the determined harbor agitation and recommended levels for safe operation, conceptual alternative designs for extending the existing breakwater were developed and modelled to determine the effects on the agitation at the existing boat launch.

In addition to the boat launch agitation study, the inundation of the shoreline due to extreme storm waves and elevated water levels was analyzed using advanced numerical models. This study will be essential for the placement of upland infrastructure in future development projects.

A creek located near the project site was reviewed for possible sedimentation concerns.

Water Levels

Water level variation can have an important influence on the operations of the boat launch. Low water levels may cause operational issues due to lack of keel clearance and high water levels can cause upland flooding.

NOAA's Great Lakes Environmental Research Laboratory (www.glerl.noaa.gov) has recorded water levels for the great lakes since 1917. Lake wide monthly averages show seasonal variation which is influenced primarily by precipitation and land retention such as in the form of snow and groundwater. Historical data shows that the lake levels are at their lowest between February and April and are at their highest between August and October. Annual water level variation between seasons averages 1 feet (0.3 m). Table 1, shown below, provides the maximum and minimum lake wide monthly average water levels from 1917-2013. This number represents a monthly average and therefore weekly, daily, and hourly averages within these given months would be greater.

Table 1 Historically Recorded Maximum and Minimum Water Levels, Lake Superior (1917-2013)

Identifier	Imperial	Metric
Low Water (Chart) Datum (LWD), IGLD 85	601.1 ft	183.2 m
1917-2013 Highest Lake Wide Monthly Average	603.38 ft (+2.33 ft LWD)	183.91 m (+0.71 m LWD)
1917-2013 Lowest Lake Wide Monthly Average	599.48 ft (-1.57 ft LWD)	182.72 m (-0.48 m LWD)

Storm surges may change the water level locally for a short period of time. Storm surges occur mainly during the winter due to strong winds when the water levels are low. In the case of Lake Superior, surges and wind setup due to storms have less influence on the local water elevation given the very deep waters within the main lake bed.

The report Design Water Level Determination on the Great Lakes (USACE 1993) establishes design water levels for several return periods based on the analysis of 34 depth gauges. The design water levels for Grand Marais are shown in Table 2. These water levels were used within the modeling of the 1, 10, and 50 year extreme return period wave event conditions.

Table 2 Design Water Levels

Water Levels	Value	Reference IGLD-85 Datum
Low Water Datum (LWD)	183.2 m	0 m (0 ft)
High Water Level 1-Year	183.72 m	+0.70 m (+2.3 ft)
High Water Level 10-Year	183.92 m	+0.72 m (+2.4 ft)
High Water Level 50-Year	184.02 m	+0.82 m (+2.7 ft)
High Water Level 100-Year	184.06 m	+0.86 m (+2.8 ft)

Figure 1 below shows the water level trend over the past two years and the projection of water levels for the next six months.

LAKE SUPERIOR WATER LEVELS – JANUARY 2014

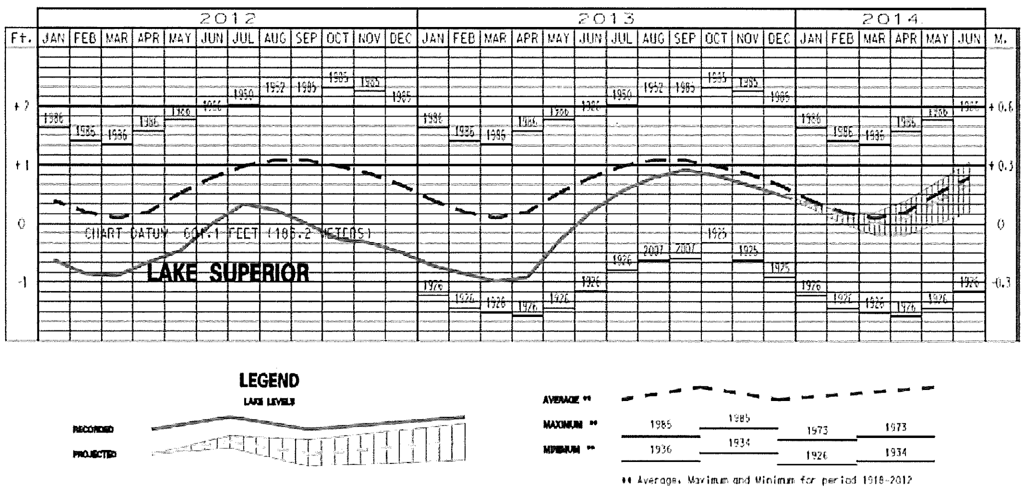


Figure 1 Lake Superior Water Levels and Six-Month Forecast (USACE, January 2014)

Bathymetry

Bathymetric information was obtained from several sources. Large scale bathymetric data was obtained from the National Geophysical Data Center, NOAA. A selected grid from the Great Lakes Bathymetry database at 3-second resolution was extracted. For more refined nearshore bathymetric data, a nautical chart with information on shoals was combined with LIDAR 2008 information which includes highly detailed information of the offshore, the beach, and the upland near the project site.

With the above information, a Digital Terrain Model was created from which two modeling grids were developed. Figure 2 and Figure 3 show the large and small scale models developed for the analysis.

Figure 4 shows a more detail bathymetry of the project location. This model area was used for a higher resolution smooth particle hydrodynamics model. The bathymetric figures are all presented at low water datum.

The boat launch, shown in Figure 4, is sheltered by a small groin which extends approximately 165 feet into the lake basin.

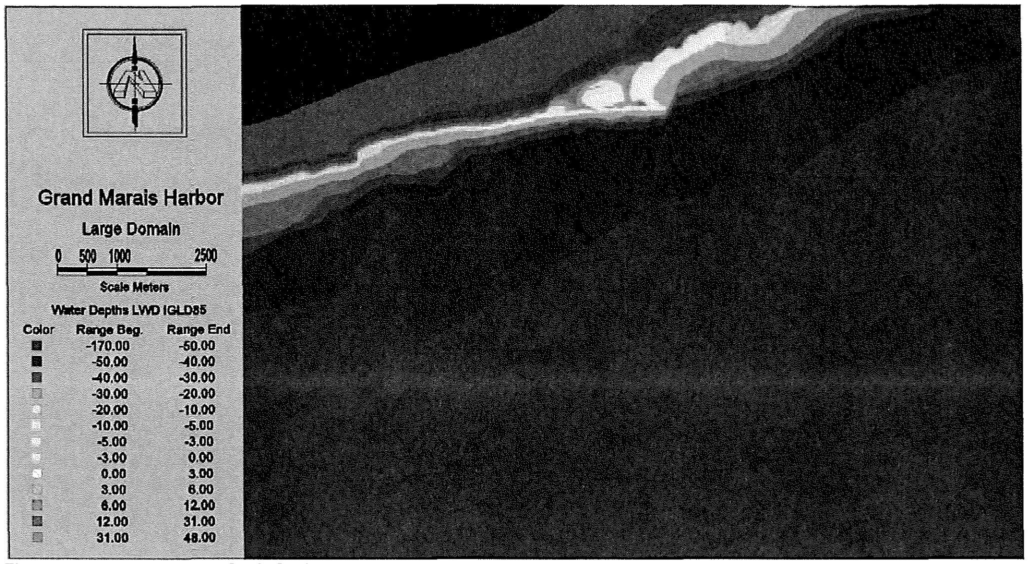


Figure 2 Large Scale Bathymetry

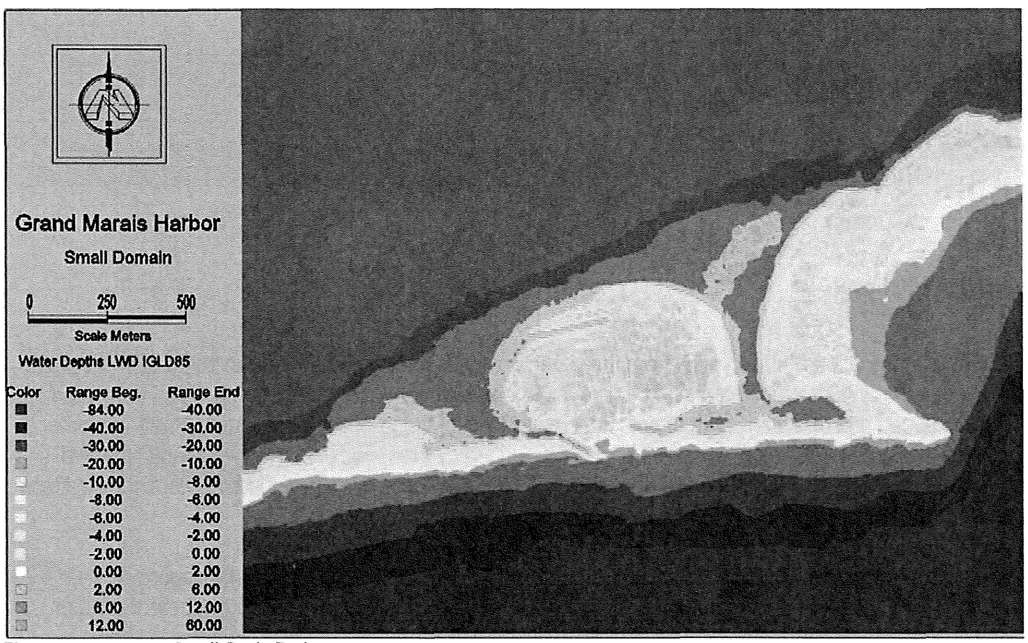


Figure 3 Small Scale Bathymetry

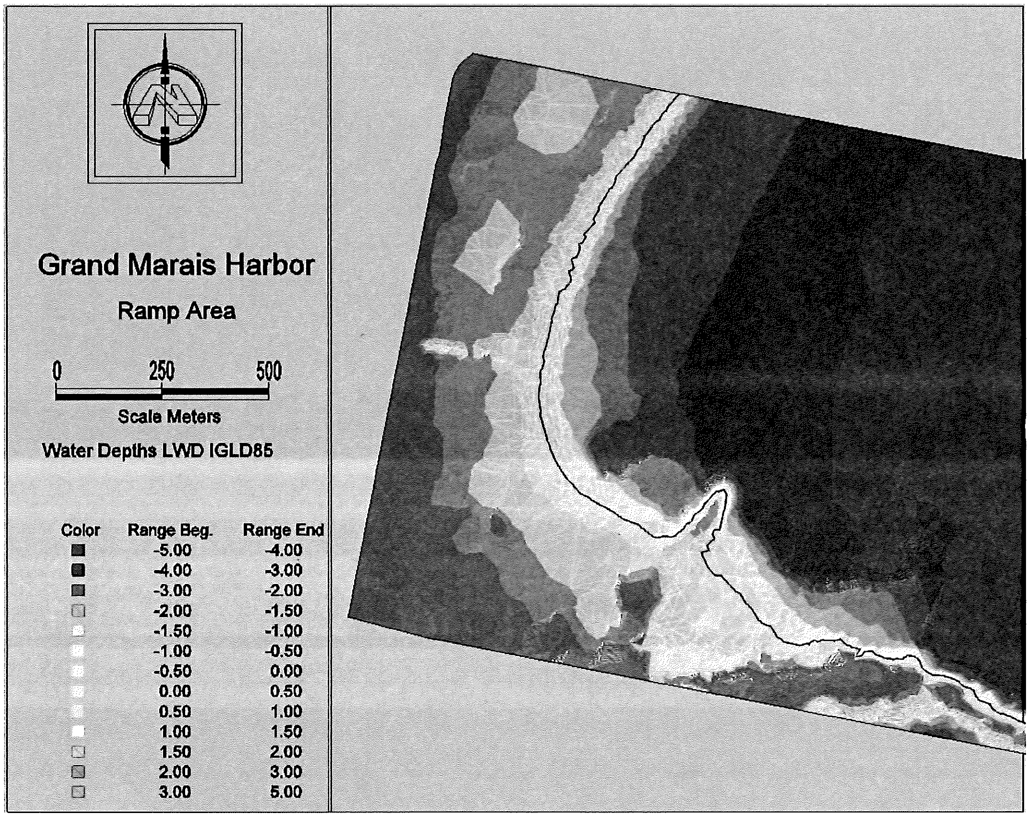


Figure 4 Detailed Boat Launch Area Bathymetry

Wind Analysis

The Wave Information Studies (WIS) from the USACE was used to determine the extreme offshore wave conditions. The WIS is completed using a hindcast model with 34 years of measured wind information including speed and direction. Due to actual measured winds being used in combination with computer model hindcast, the WIS result is generally of higher accuracy than the forecast wave conditions, and is often representative of observed wave conditions. As the WIS uses 34 years of data in its hindcast, there can be confidence in its determination of extreme wave events.

The closest WIS virtual wave gauge station to the project site is Station 95300 which is approximately 4.76 miles (7.7 km) south of the project site at 47.68 N, 90.32 W and it is located at a point where the water depth is 164 m (538 feet). The location of Station 95300 and its relation to the project site is shown in Figure 5.

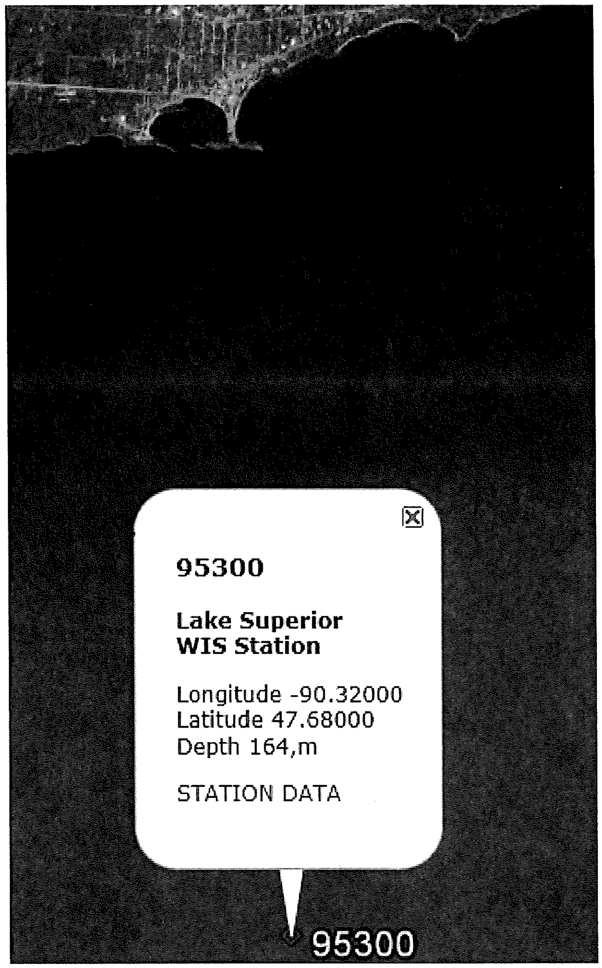


Figure 5 WIS Station Location

The wind rose for this station is shown in Figure 6. As shown, the most frequent winds occur from the NW to SW quadrants. Most of the environmental forces from these directions will have little effect on the project site from a water perspective given the orientation and entrance of the Bay at Grand Marais as well as the location of the boat launch within the bay. The winds which will have the greatest influence on the waves entering the bay and impacting the boat launch will be directed from the East through South quadrants.

Due to the long distance between the project site and the WIS station, the historical wind conditions at the station were included in the large scale wave model as additional energy. Extreme wave conditions will generally coincide with extreme wind events where the storm duration is long enough to generate a fetch limited wave condition.

The extreme winds were analyzed using the 155 highest recorded winds per quadrant. These winds were fitted to a Weibull distribution with k values from 0.75 to 2, and to the Gumbel distribution. The distribution analysis with the best fit was selected for each case. The design return period extreme winds at 10 meter height at the location of the WIS station are shown in Table 3.

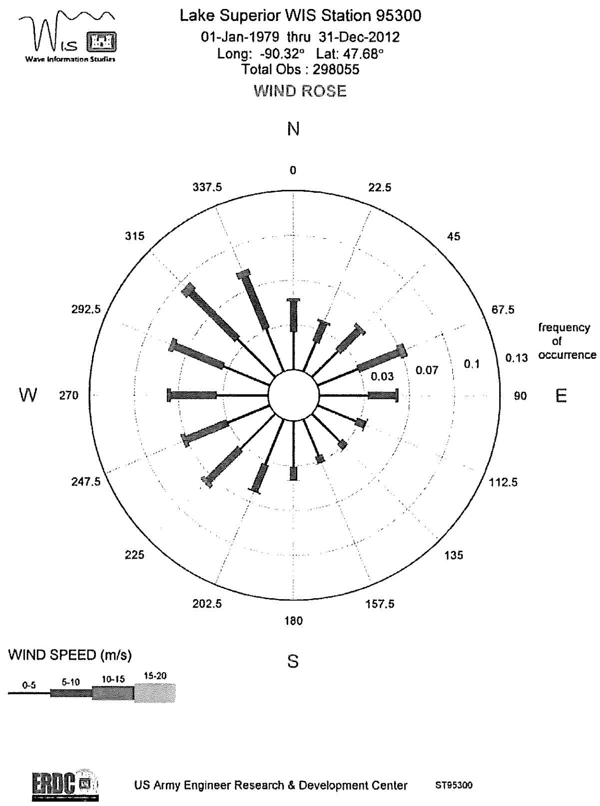


Figure 6 Wind Rose for WIS station 95300

Table 3 Extreme Wind Return Period Events per Direction

	E	SE	S	SW	W	ALL
Return Period (years)	*Wind Speed (knots)*	*Wind Speed (knots)*	*Wind Speed (knots)*	*Wind Speed (knots)*	*Wind Speed (knots)*	*Wind Speed (knots)*
1	30.8	21.6	23.2	29.3	28.9	32.8
10	36.0	25.9	27.9	34.7	32.6	36.2
25	38.1	27.6	29.8	36.9	34.0	37.6
50	39.6	28.9	31.2	38.5	35.1	38.6
100	41.2	30.2	32.7	40.1	36.2	39.7

Wave Conditions

1.1.-Yearly Average Wave Conditions

1.1.1.- *Wave Direction*

Hindcasted wave data at the WIS virtual wave gauge spans a record period from 1979 through 2012. Hourly estimates of wave height (Hmo), peak period (Tp), and direction (dir) due to the wind speed are included in this data. Current WIS data is developed using the wave model WAM. The model is driven by the wind fields derived from the Climate Forecast System Reanalysis (CFSR). The wind fields are spatially interpolated to the wave model grid. The temporal resolution of the CFSR wind fields is also 1-hr. During the winter months daily ice concentration fields are also applied to the wave model simulations using the NOAA Great Lakes Ice Atlas.

It was established that, given the orientation of the Grand Marais Bay, the directions that could affect the project were clockwise from east (90°) to west (270°). The wave occurrence frequency for Station 95300, for each direction, Table 4, shows a high concentration of waves coming from the east; occurring 26.11% of the time throughout the year. This is explained because of the long, unrestricted fetch from this direction. The second most common direction for incoming waves is the southwest. These directions also represent the propagation direction of the highest waves, as shown in the wave rose plot, Figure 7.

Table 4 Wave Direction Occurrence Probability

Direction		Occurrence Frequency
E	90°	26.11%
ESE	112.5°	5.42%
SE	135°	3.19%
SSE	157.5°	3.46%
S	180°	5.21%
SSW	202.5°	9.63%
SW	225°	11.12%
WSW	247.5°	5.89%
W	270°	4.71%

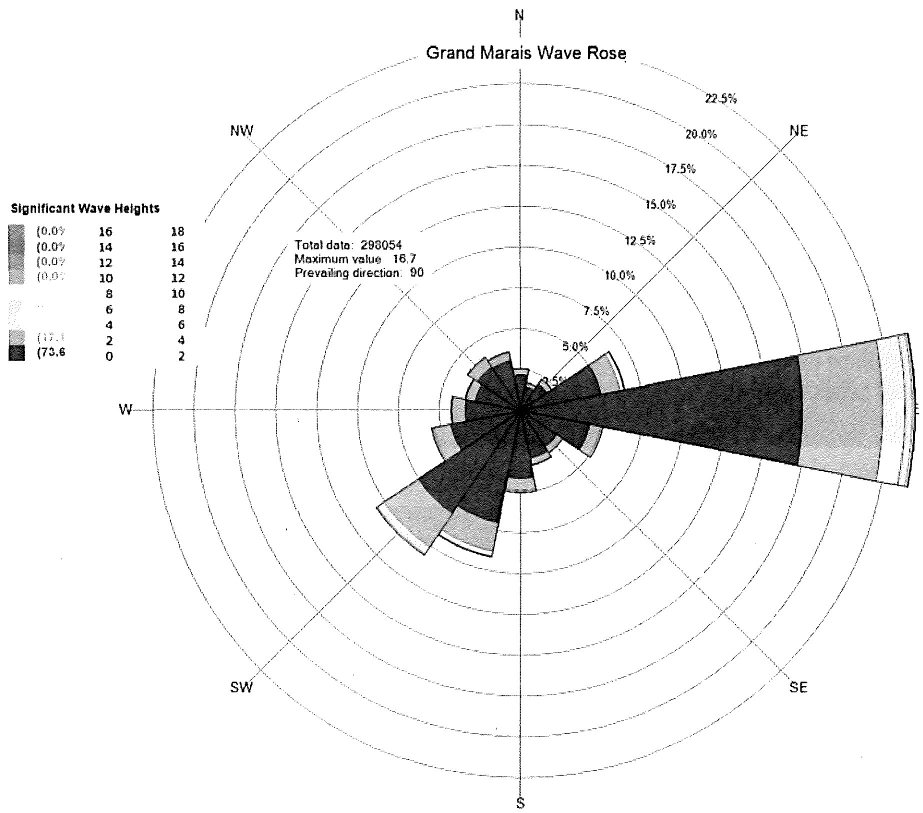


Figure 7 Wave Rose for Station 95300, All Seasons

It was recognized that the boat launch at Grand Marais would only be utilized during the 'boating season' and therefore agitation levels outside of this window could be greater than those recommended for user comfort and safety. For the purposes of this study, the boating season has been defined as being May to October, where the average low temperatures are above freezing, and the non-boating season has been defined as being November to April. The monthly temperature highs and lows throughout the year for the region are shown in Figure 8. As winter storm winds are generally stronger than those in the summer, wave conditions will be different between these two periods of time. The presence of ice would also change the wave conditions at the site significantly as the available fetch for the wind to produce waves would be reduced.

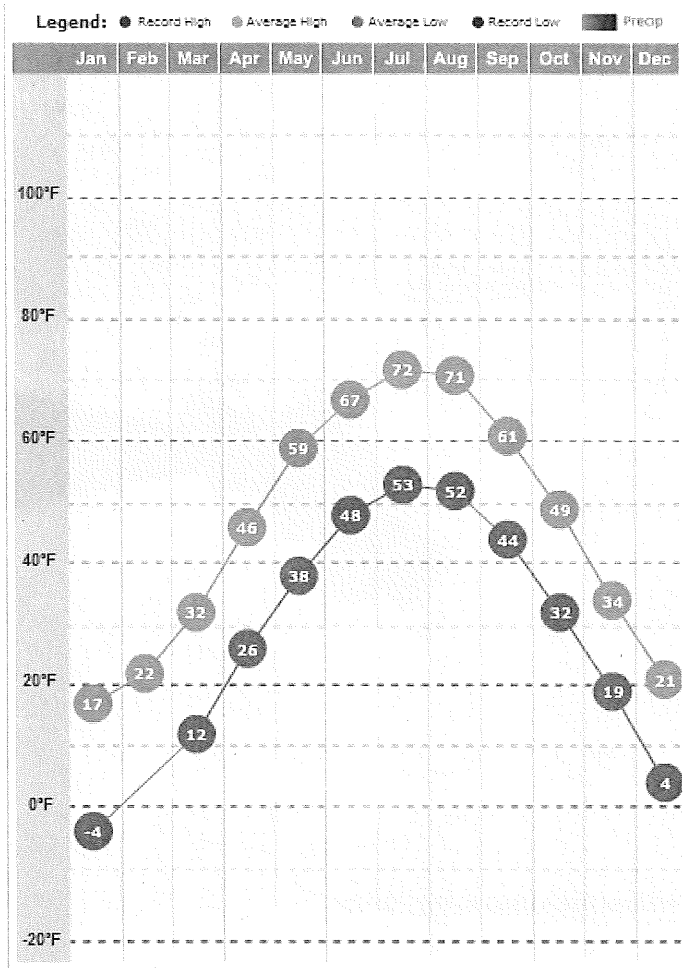


Figure 8 Monthly Average Temperature for Grand Marais Region

The wave rose for both the boating and non-boating seasons are shown in Figure 9 & Figure 10. For both cases the most frequent wave direction, and the direction of the largest waves, is the east. While it is shown that the dominant wave direction throughout the year is from the east, it can be seen that the non-boating season has a higher occurrence of large waves. A review of the largest waves in the hindcast revealed that within the defined boating season, all waves larger than 9.2 feet occurred in October. This study and the conceptual design of protective structures will only concentrate on dampening the waves within the boating season.

Given the orientation of the Bay, it can be argued that east originating waves will not penetrate into the Bay. However, considering that waves from the east direction represent a range of waves with directions from 67.5° to 112.5°, all easterly originating waves were modelled as coming from 112.5°.

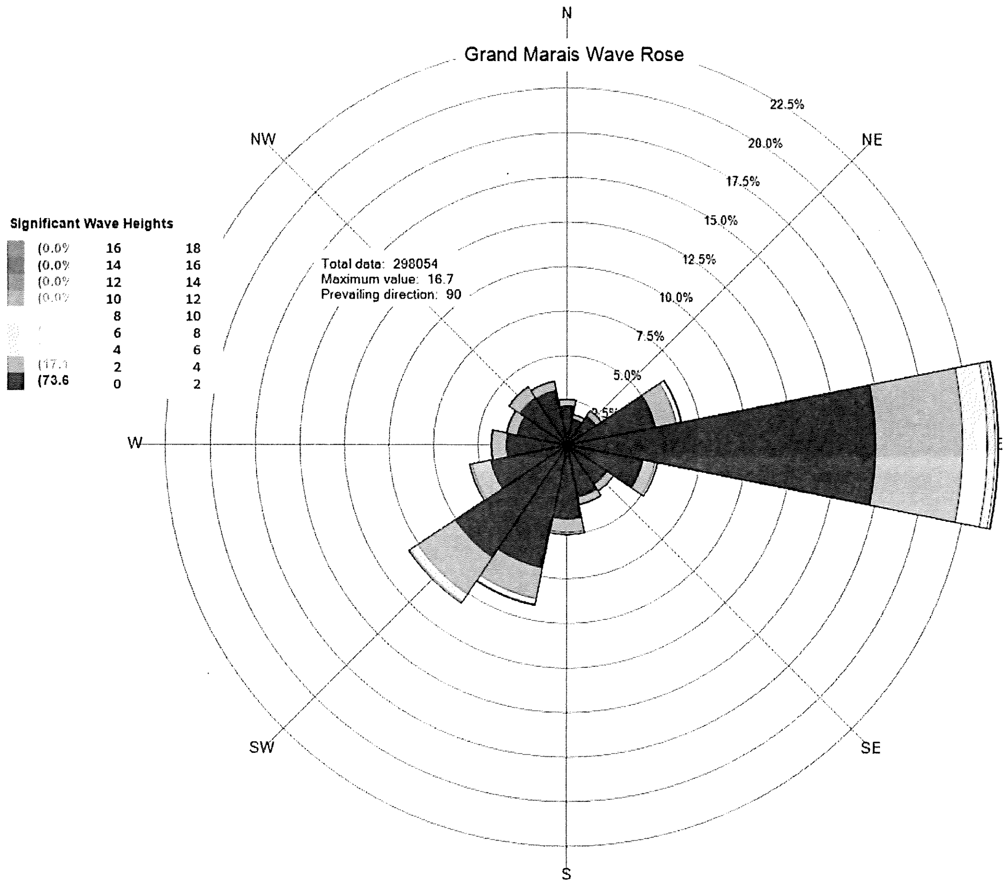


Figure 9 Wave Rose Boating Season, May – October

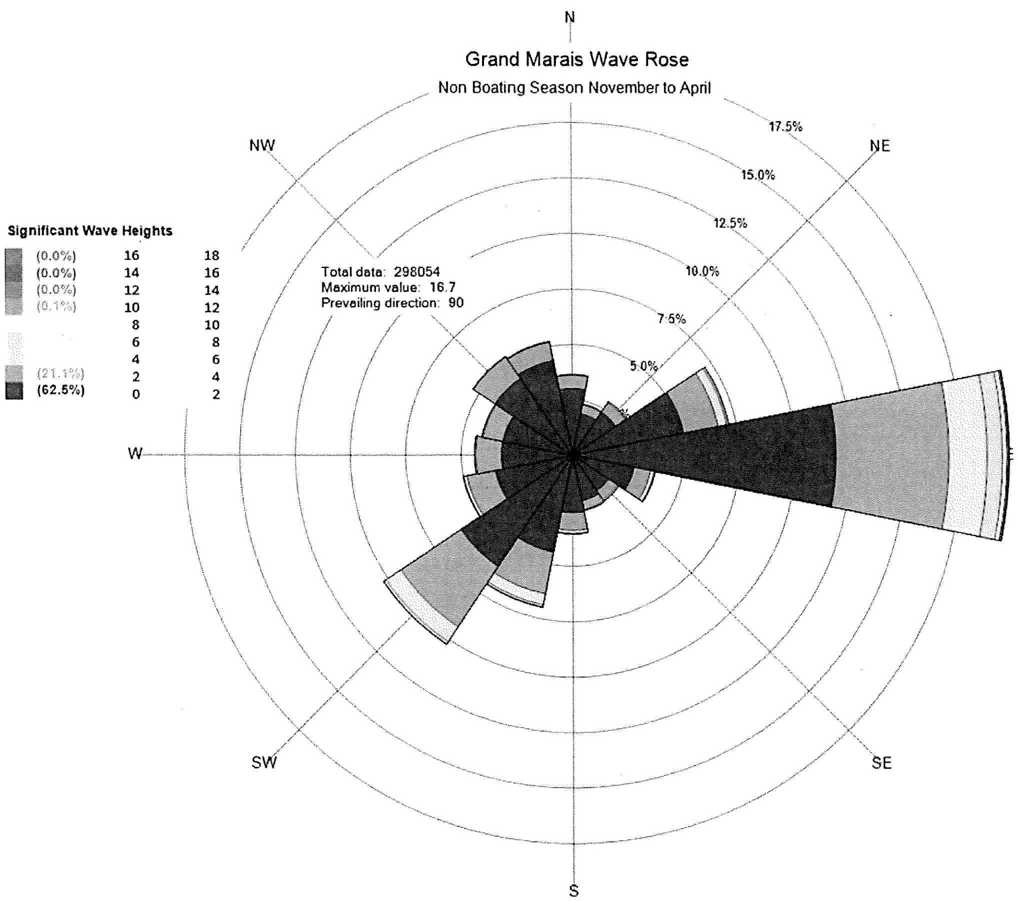


Figure 10 Wave Rose Non-Boating Season, November – April

The exceedance probability, that is the probability of a certain wave height per wave direction to be exceeded was determined for all wind directions of interest and is shown in Figure 11. Waves from the east, as shown, are much higher than those from other directions.

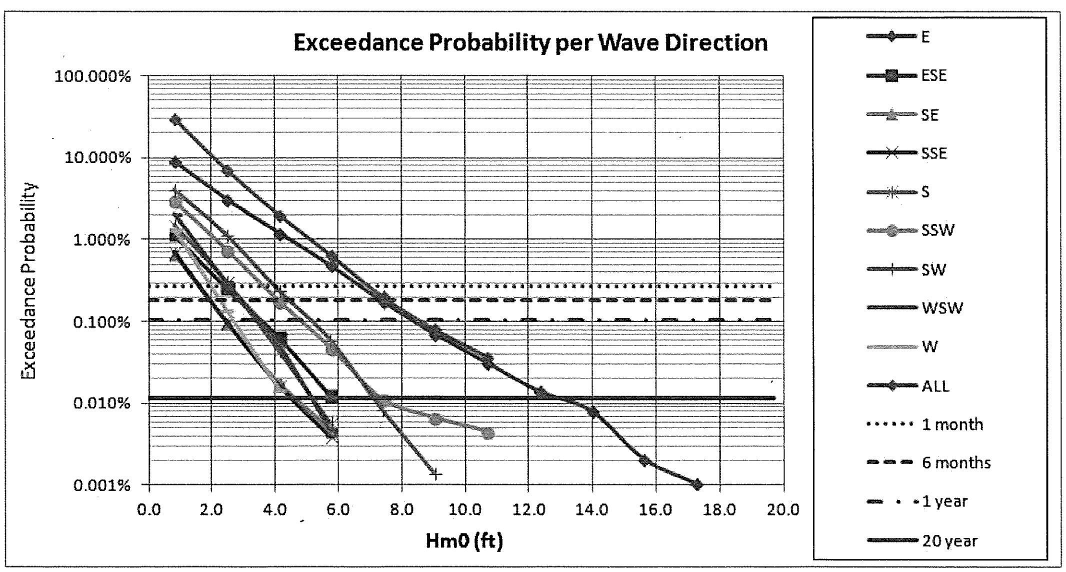


Figure 11 Exceedance Probability per Wave Direction

1.1.2.- Wave Height

The exceedance wave height probability for both boating and non-boating seasons per the direction of largest wave heights, the east, was developed. Table 5 shows the return period wave heights for each season which corresponds to the results shown in Figure 12. It is clear that during the non-boating season more energetic wave conditions are present.

Table 5 Wave Height Seasonal Exceedance

Return Period Event	Exceeded Hmo (feet)	
	Boating	Non-Boating
	E	E
1 Month	3.87	6.20
6 Months	4.80	6.95
1 Year	5.74	8.15
20 Years	8.07	13.02

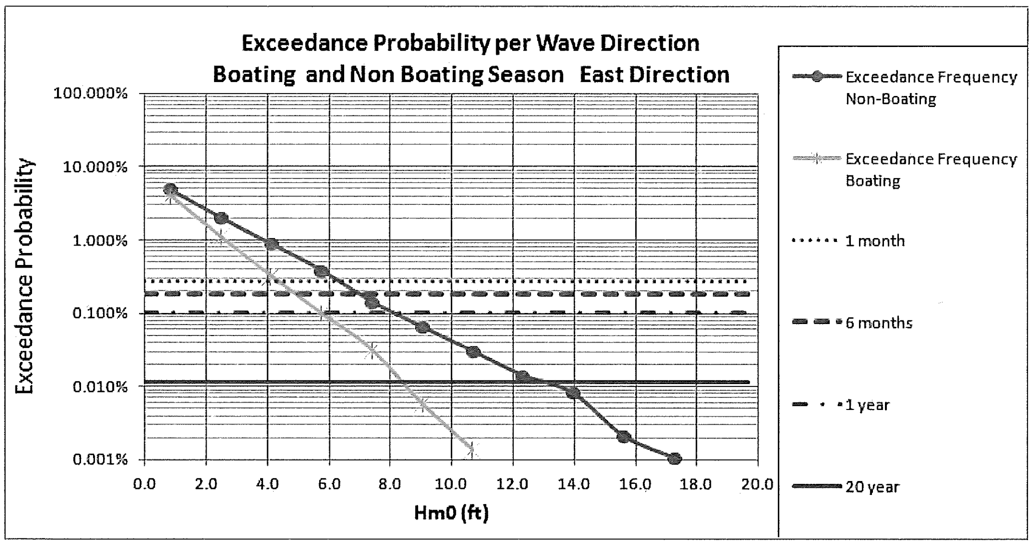


Figure 12 Relationship of Wave Heights from the East Direction for Boating and Non-Boating Seasons

1.1.3.- Wave Period

Wave transformation, refraction, shoaling, reflection, and diffraction are highly affected by the wave period. Waves with larger wave periods will tend to refract and diffract more than shorter period waves. The largest wave periods provided by the WIS hindcast are on the order of 10 s. However, wave periods of this length are not common.

Using the waterfall tables for the closest wave station, S04, in WIS Report 23 "Hindcast Wave Studies for the Great Lakes: Lake Superior" (1992), the most predominant wave period associated with the selected representative wave heights was used within the numerical modelling. The waterfall table for waves from the east is shown below in Figure 13.

STATION S04 47.67N 90.07W AZIMUTH(DEGREES) = 90.0
PERCENT OCCURRENCE(X1000) OF HEIGHT AND PERIOD BY DIRECTION

HEIGHT(METRES)	PEAK PERIOD(SECONDS) <3.0	3.0-3.9	4.0-4.9	5.0-5.9	6.0-6.9	7.0-7.9	8.0-8.9	9.0-9.9	10.0-10.9	11.0-LONGER	TOTAL
0.00-0.49	499	1251	118	14	3	.	.	.	.	.	1885
0.50-0.99	.	1070	3161	160	12	7	.	.	.	.	4410
1.00-1.49	.	.	890	928	64	7	2	.	.	.	1891
1.50-1.99	.	.	37	429	202	38	2	.	.	.	708
2.00-2.49	.	.	.	126	98	96	7	1	.	.	328
2.50-2.99	.	.	.	2	134	50	24	8	.	.	218
3.00-3.49	.	.	.	.	4	90	19	13	1	.	127
3.50-3.99	.	.	.	.	.	56	29	12	8	.	105
4.00-4.49	.	.	.	.	.	6	41	25	13	.	85
4.50-4.99	.	.	.	.	.	.	5	35	12	1	53
5.00-5.49	.	.	.	.	.	.	.	21	17	4	42
5.50-5.99	.	.	.	.	.	.	.	5	14	1	20
6.00-6.49	.	.	.	.	.	.	.	.	13	11	24
6.50-6.99	.	.	.	.	.	.	.	.	4	10	14
7.00+	.	.	.	.	.	.	.	.	1	19	20
TOTAL	499	2321	4206	1659	517	350	129	120	88	46	

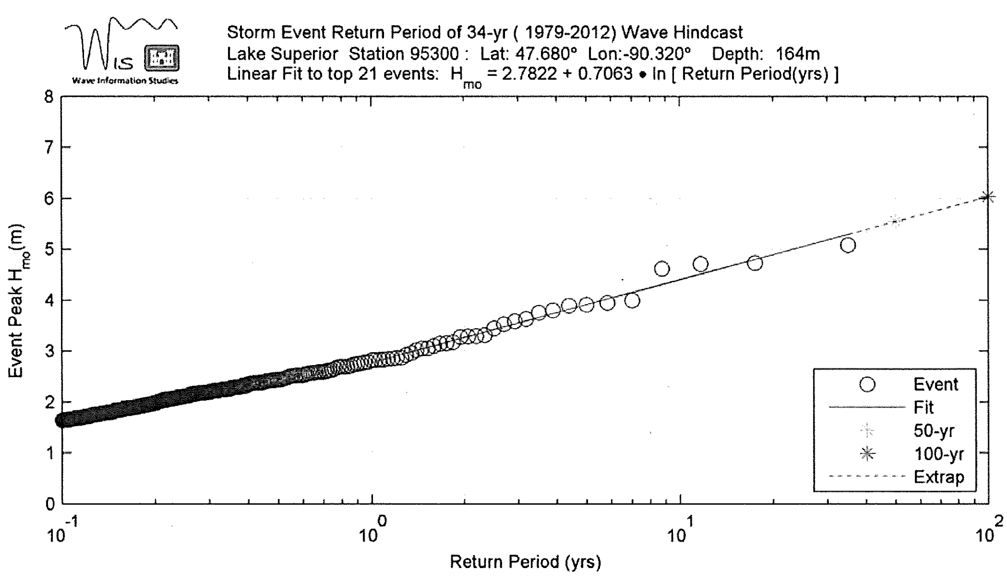
MEAN HS(M) = 1.1 LARGEST HS(M)= 9.9 MEAN TP(SEC)= 4.5 NO. OF CASES= 9311.

Figure 13 Hindcast Wave Information for Lake Superior, Station S04, Direction: East (USACE, 1992)

1.2.-Extreme Wave Analysis

Though extreme waves are not used within the models analyzing the agitation levels at the boat launch, extreme conditions will ultimately be important for the design and construction of the coastal structures.

The WIS station reports an extreme wave distribution for all wave directions and the 10 highest events, as shown in Figure 14. It can be seen that 8 of the 10 highest events occurred from the east direction and 2 from the south direction. Given the exceedance probability differences found for each direction in Figure 11, a more detail analysis was carried out to obtain the extreme wave conditions from the east, southeast, south, and southwest directions.



Top 10 events based on Peak H_{mo}

Event	Date/Time(UTC)	H_{mo}	T_p	θ_{mean}	Event	Date/Time(UTC)	H_{mo}	T_p	θ_{mean}
1	1994/11/28 04:00	5.08	10.18	91.0	6	2012/02/29 21:00	3.95	9.11	93.0
2	2007/03/02 05:00	4.73	10.09	90.0	7	1986/11/08 22:00	3.91	7.88	208.0
3	1998/11/10 19:00	4.71	9.24	89.0	8	1984/10/19 11:00	3.89	8.41	93.0
4	1992/11/02 20:00	4.62	10.05	90.0	9	1999/04/04 11:00	3.80	8.42	91.0
5	2010/10/27 10:00	4.00	8.19	200.0	10	2009/12/25 17:00	3.75	8.65	90.0

An event is defined as any period when H_{mo} > 1.00m θ_{mean} is direction that waves are arriving from

ERDC US Army Engineer Research & Development Center ST95300

Figure 14 Extreme Wave Distribution, WIS Station 95300, USACE

Using the 34 years of available information the extreme wave heights per direction were obtained. The extreme waves with return periods of 1, 10, 25, 50, and100 years were analyzed. Weibull with k values from 0.75 to 1.5 and the Gumbel extreme probabilistic distribution were fitted against the highest 140 events per direction. The regression fit for the east direction can be seen in Figure 15.

Comparing the all direction extreme wave conditions found by USACE to the Weibull distribution performed on all data, a good agreement was found for return periods greater than 25 years. However, the analysis showed poor agreement between the distribution analysis and the annual and ten year return period events. Therefore, these events were not determined by direction and it is recommended that all direction wave characteristics be used for these return periods, which can be considered conservative. The results of this directional analysis are shown in Table 6.

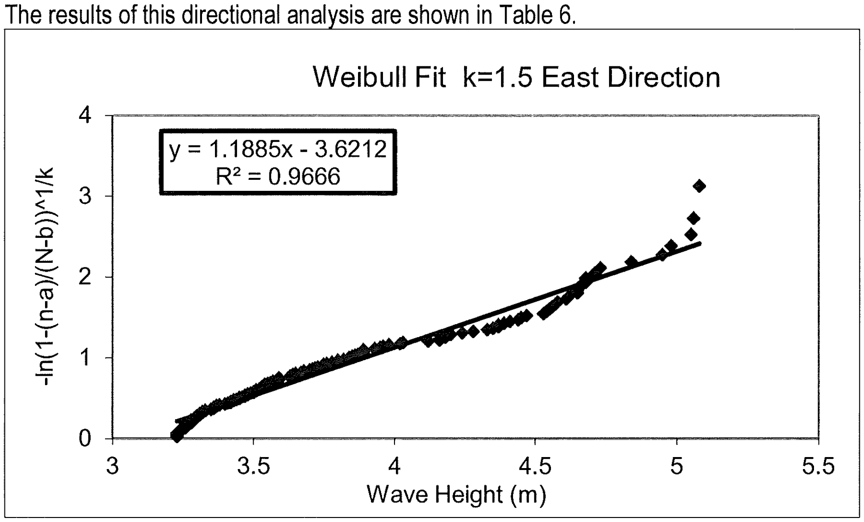


Figure 15 Weibull Extreme Wave Regression, East Direction

Table 6 External Wave Heights by Return Period

	E		SE		S		SW		ALL		ALL USACE	
Return Period (years)	*Hmo (m)*	*Hmo (feet)*	*Hmo (m)*	*Hmo (feet)*	*Hmo (m)*	*Hmo (feet)*	*Hmo (m)*	*Hmo (feet)*	*Hmo (m)*	*Hmo (feet)*	*Hmo (m)*	*Hmo (feet)*
1									4.0	13.0	2.8	9.1
10									5.1	16.8	4.4	14.5
25	5.7	18.6	2.9	9.5	4.02	13.2	3.78	12.4	5.6	18.3	5.1	16.6
50	6.0	19.8	3.1	10.2	4.35	14.3	4.01	13.2	5.9	19.4	5.5	18.2
100	6.4	21.0	3.3	10.9	4.69	15.4	4.24	13.9	6.3	20.6	6.0	19.8

Operation and Modelling Criteria

There are no set standards for operations of a boat launch. Guidelines for agitation do not specifically cover wave heights at the boat launch though it is universally agreed that boat launches should be located well inside a sheltered basin. Due to resonance amplification, small waves can cause a small boat to have excessive heave. Therefore, for safe egress and for the purposes of this study, it is recommended that the wave height at the boat launch be less than 0.5 feet.

Understanding the response of users will help set the design criteria for the boat launch. Lake Superior is known to have sudden storms which rapidly change the marine environment. While local boats may be more accustomed to rough waters, it has been considered, for the purposes of this study, that deepwater wave heights greater than 3.5 feet will effectively stop users from launching vessels at the boat launch. It is assumed that for the majority of boaters already on the water, deepwater waves greater than 4 feet will drive them back to the boat launch. Based on the hindcast analysis presented above, this event has a return period of approximately 1 month during the boating season.

More experienced boaters may choose to be on the lake at greater agitation levels, or less experienced boaters may get caught unaware in a storm. For this reason, the yearly and twenty year events during the boating season were also reviewed. This can be considered a value engineering approach as the conceptual designs presented will attempt only to limit agitation to 'comfortable' levels.

The hydrodynamic numerical modelling consisted of testing four conditions of various storm intensities. These conditions represent the boating season event occurring once per month, once per six months, once per year, and once per 20 years and are labeled as Conditions 1, 2, 3, & 4 respectively. It can be seen in Table 7 below that Condition 1, the once per month event, is similar in scale to the condition in which boaters will stop utilizing the boat launch.

Table 7 Numerical Modelling Conditions Matrix

	Wave Criteria for Modelling	
Condition	Deepwater Wave Height	Peak Wave Period
	Hmo	Tp
Condition 1 (1 month)	3.87 ft	6.5 s
Condition 2 (6 month)	4.80 ft	7.0 s
Condition 3 (1 year)	5.74 ft	7.5 s
Condition 4 (20 year)	8.07 ft	8.8 s

Numerical Modeling Setup

1.3.-Large Scale Model

The deepwater wave conditions analyzed from WIS station 95300 were run through a hydrodynamic model to the project site. Given the location of the WIS station, first a large scale domain was modeled using the STWAVE numerical model from USACE. All the events were modeled with wave interaction and wind effects using a 25x25 meter grid. The deepwater wave boundary conditions applied to the model were given in Table 7. A Bretschneider-Mitsuyase wave spectrum was used with directional spreading as a function of the associated wind speed. Figure 16 shows the large scale wave model results for a 50-year return period condition from the ESE. Due to the deepwater between the WIS station and the shoreline, only small changes in wave characteristics were observed.

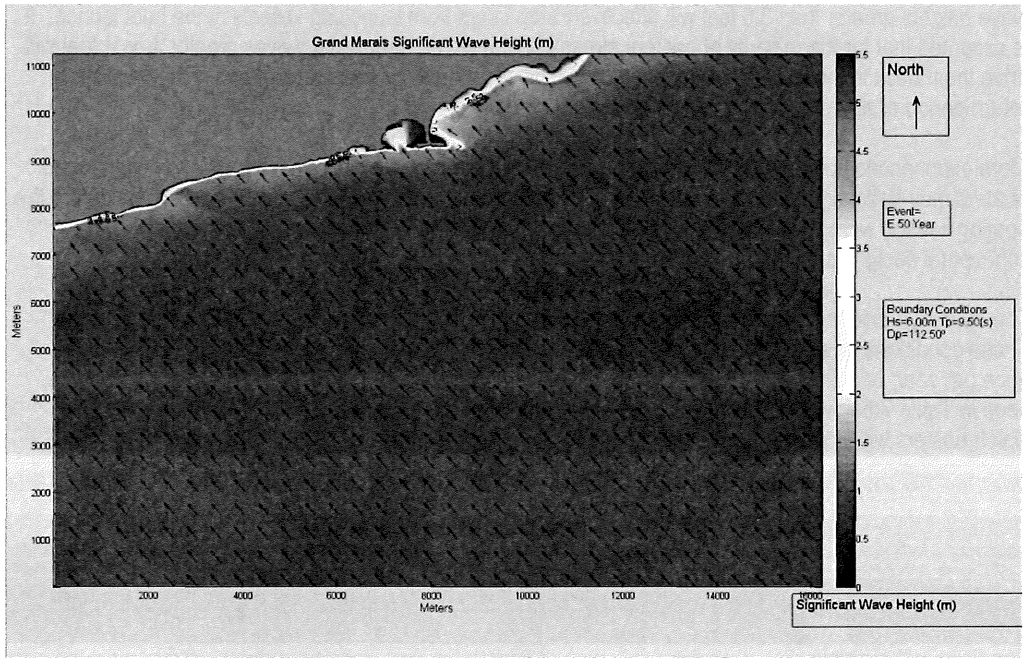


Figure 16 Large Scale Wave Model, ESE Wave Direction

1.4.-Detail Numerical Modeling

In order to determine the wave conditions at the boat launch location, a more refined model of Grand Marais Bay, shown in Figure 17, was created. The nonlinear Boussinesq-type model, NOWT-PARI, was used in this analysis to take into account the effects of waves diffracting into the Bay through the Federal breakwaters and the wave reflection off of existing structures within the bay basin.

Using the offshore extreme wave conditions given in Table 6, waves were modelled through the large scale model and the detailed numerical model to obtain the conditions at the boat launch. The critical wave conditions were found to occur from waves with an east direction. Wave diffraction through the entrance was noticed to play an important role, however waves reflecting from the small breakwater sheltering the inner harbor were critical into causing agitation near the boat launch area where waves diffract through the entrance and later a complex wave pattern from the reflecting and incoming waves is observed. This effect can be seen in Figure 18. A generalized 0.50 reflection coefficient was applied on all sloping structures.

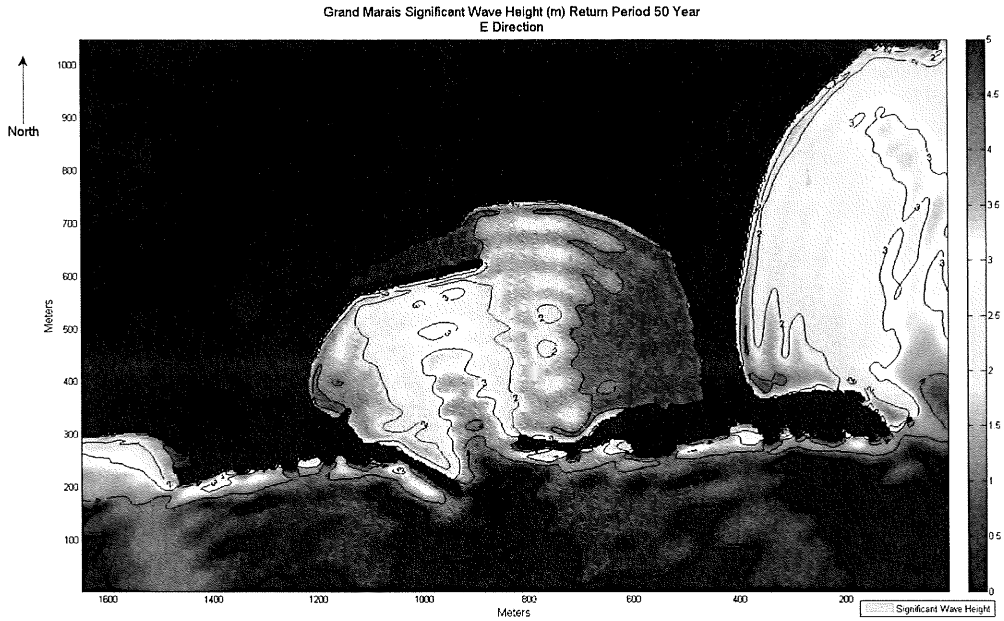


Figure 17 Small Scale Wave Model

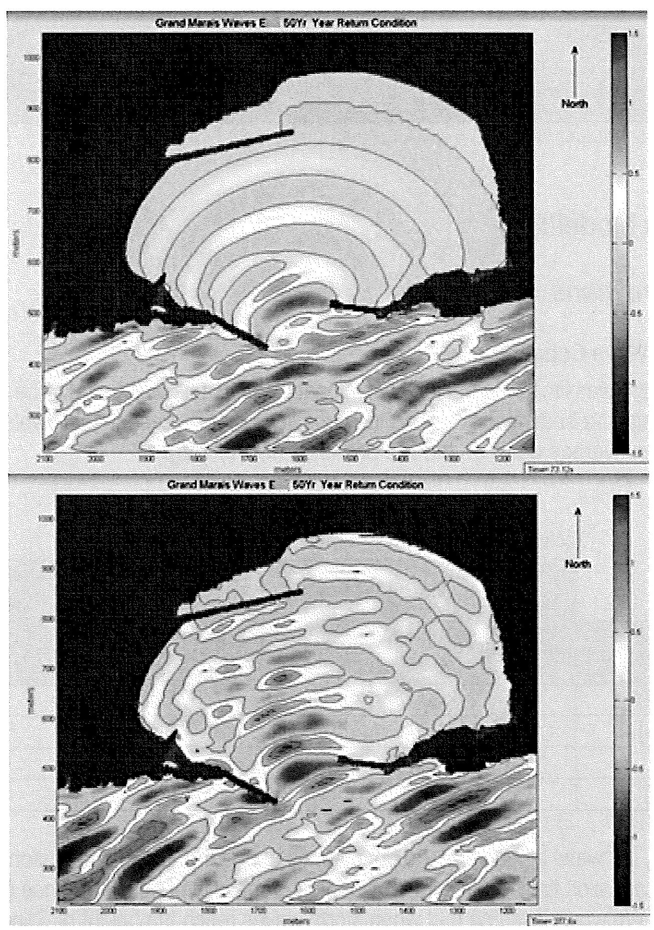


Figure 18 Easterly Waves Entering Bay and Reflecting Off of Interior Breakwater

The wave heights at two different output locations were selected as reference points to represent the wave climate at the boat launch location. The obtained wave characteristics at these points for all the modeled waves were then compared to the recommended wave height criteria. The allowable wave height at the boat launch area, for the purposes of this study, is taken as 0.5 feet.

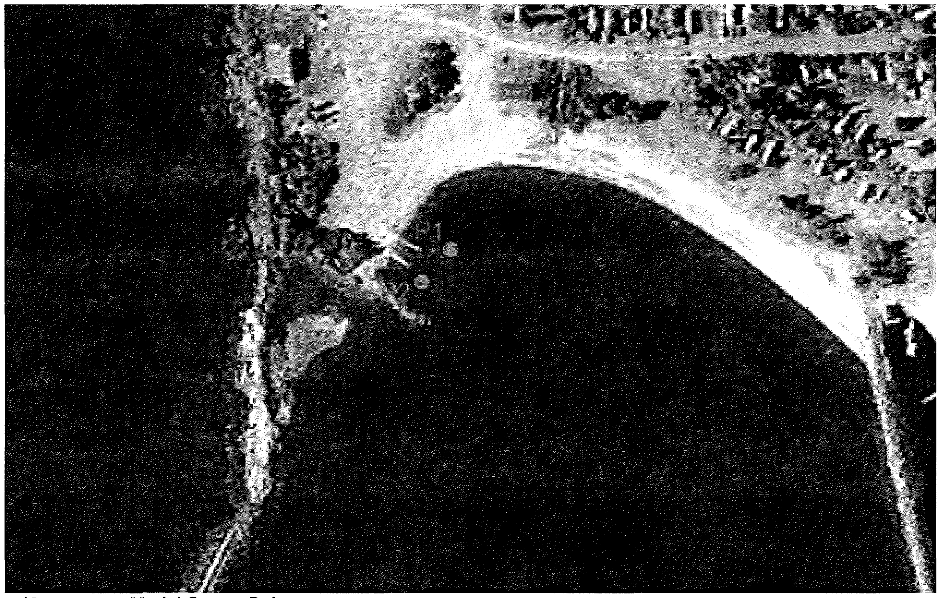


Figure 19 Model Output Points

Wave Agitation Modelling

1.5.-Existing Conditions

1.5.1.- Operational Wave Conditions

The agitation modelling was limited to easterly wave conditions as this was found to be the governing direction for agitation at the boat launch. Using the conditions matrix presented in Table 7, the agitation was determined as an average between the two points shown in Figure 19.

Table 8 Existing Modelled Wave Agitation at Boat Launch

	Wave Criteria for Modelling		Wave Agitation at the Boat Launch
Condition	Deepwater Wave Height	Peak Wave Period	Local Wave Height
	Hmo	Tp	Hs
Condition 1 (1 month)	3.87 ft	6.5 s	0.44 ft
Condition 2 (6 month)	4.80 ft	7.0 s	0.83 ft
Condition 3 (1 year)	5.74 ft	7.5 s	0.99 ft
Condition 4 (20 year)	8.07 ft	8.8 s	1.65 ft

As shown in Table 8, the wave agitation at the boat launch during the 1 month condition is already below the required 0.5 feet agitation limit. This event has a high probability of occurring once a month during the open water season. We would assume that when storm waves reach the height of Condition 1 outside the basin breakwaters, use of the boat launch for launching will decrease and the facility primarily will be used for boat retrieval.

For less frequent storms, such as Conditions 2 through 4, the agitation at the boat launch is above the 0.5 feet threshold which means use of the boat launch will be hindered and possibly even dangerous.

1.5.2.- Extreme Wave Conditions

Assuming no modifications are made to the protective structures around the boat launch, the wave climate at the two model output locations was measured and is presented in Table 9. As the modelled criteria are extreme events and correspond to events outside the boating season, the results will be used in the design phase of the breakwater and should not be considered in the agitation requirements study.

Table 9 Wave Heights at Comparison Points Extreme Wave Conditions

Direction	Return Period (years)	P1 Hmo (m)	P2 Hmo (m)	Mean Hmo (m)	Mean Hmo (feet)
E	1	0.77	0.70	0.74	2.41
	10	0.90	0.86	0.88	2.89
	50	0.92	0.87	0.90	2.94
SE	1	0.44	0.40	0.42	1.38
	10	0.59	0.52	0.56	1.82
	50	0.65	0.59	0.62	2.03
S	1	0.34	0.33	0.34	1.10
	10	0.55	0.52	0.54	1.76
	50	0.57	0.59	0.58	1.90
SW	1	0.31	0.30	0.31	1.00
	10	0.33	0.37	0.35	1.15
	50	0.48	0.46	0.47	1.54
W	1	0.14	0.12	0.13	0.43
	10	0.21	0.15	0.18	0.59
	50	0.19	0.19	0.19	0.62

1.6.-Alternatives to Reduce the Agitation Levels at the Boat Launch Location

In order to reduce the agitation at the boat launch location, three alternatives which included elongating the existing groin breakwater were considered. These alternatives included a singular rubble mound extension, a fishtail extension, and the addition of a bin wall adjacent to the boat launch with a rubble mound extension. These three alternatives are shown in Figure 20 thru Figure 22 below.

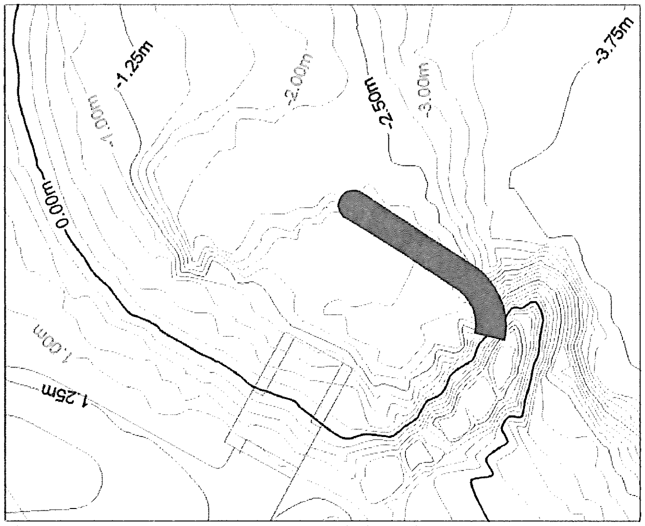


Figure 20 Alternative 1: Extended Breakwater

The breakwater alternatives proposed, Alternatives 1 & 2, are both connected to the existing breakwater groin and would be constructed of local rubble similar to that of the existing structure. This would create a visually continuous breakwater. By extending the existing breakwater structure around the boat launch, an enclosed, sheltered basin is created. The location of the breakwater extension was specifically chosen to limit construction away from the deeper lakebed contours while creating an adequate basin size beyond the toe of the boat launch for boater staging.

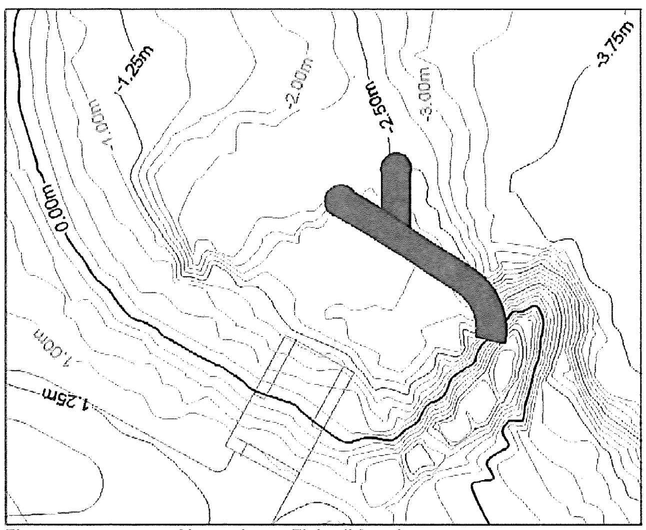


Figure 21 Alternative 2: Fishtail Breakwater

Fishtail breakwaters, as shown in Figure 21, assist in breaking down the energy of an incoming wave by forcing the wave to diffract twice around the breakwater tip. This effectively aids in reducing the amount of wave energy, and therefore wave height, which enters the sheltered basin. Since the distance between the tips of the fishtail are important, this type of structure is more efficient for short period waves. The additional rubble contained in the second breakwater spur results in a higher construction cost.

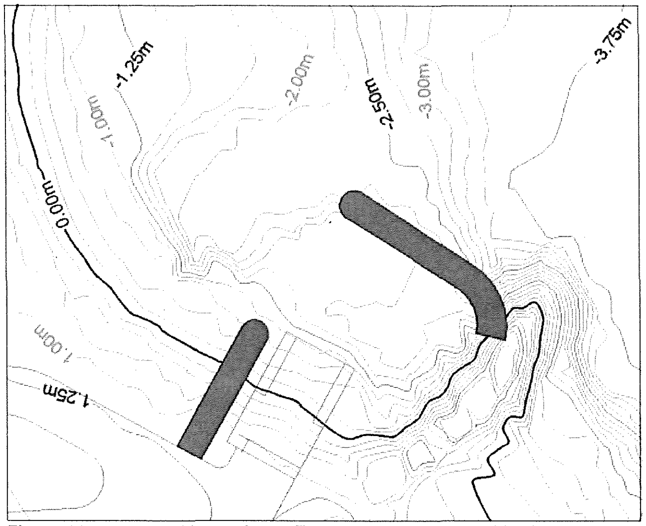


Figure 22 Alternative 3: Extended Breakwater with Bin Wall

As noted previously, waves off of the northern breakwater within Grand Marais Bay are reflected into the boat launch basin. In an effort to block these reflected waves, an alternative which adds a bin wall parallel to the boat launch in addition to the extended breakwater, as shown in Figure 22, was reviewed. Bin walls, which are vertical faced, can provide additional boater staging. However, because waves can be easily reflected off of them, they can cause additional agitation in and outside of the enclosed basin.

1.7.-Reduction in Agitation

Each of the breakwater alternatives were modelled with the operational conditions listed in Table 7 to determine the reduction in agitation at the boat launch using the average of the two identified output points. The modelled agitation at the boat launch for each condition for the existing layout was given in Table 8. The goal in each modelling case was to reduce the wave agitation to an acceptable 0.5 feet wave height during the event. Since Condition 1, the 1 Month event, already produced agitation below 0.5 feet, this condition was not further analyzed. The results for the remaining conditions per alternative are shown in Table 10. The table shows the significant wave height (Hs) at the boat launch, the modelled breakwater length (L), and the modelled bin wall length (L_{Bin}).

Table 10 Resulting Reduction in Agitation due to Breakwater Modifications

Condition		Deepwater Wave Height, Hmo (ft)	Existing Boat Launch Hs (ft)	Alternative 1		Alternative 2		Alternative 3		
				Hs (ft)	L (ft)	Hs (ft)	L (ft)	Hs (ft)	L (ft)	L_{Bin} (ft)
1	1 Month	3.87	0.44	Existing is below 0.5 ft, & was not modelled						
2	6 Months	4.80	0.83	0.43	105	0.36	138	0.47	105	40
3	1 Year	5.74	0.99	0.46	105	0.40	138	0.50	105	40
4	20 Years	8.07	1.65	1.27	164	1.25	196	1.49	164	40

As shown, the 20 year event, represented by Condition 4, was not reduced to the desired 0.5 feet agitation at the boat launch. After a number of model iterations, it was determined that the required breakwater length would be prohibitive and that during an event of 8 foot waves in deepwater, the launch would not be utilized. As storms generally build over time, it is theorized that educated boaters will exit the water prior to the peak of such an event. Therefore, it is recommended that the 1 year event be considered the "design event."

In addition to the numerical modelling of waves generated outside of Grand Marais Bay, locally generated waves created within the bay were also examined. These waves would not be impeded before entering the boat launch basin as it currently exists. Due to the restricted fetch within the basin, the locally generated waves, such as from the northeast, would not grow larger than 0.7 feet. Once a breakwater is constructed, this energy will be damped below the desired 0.5 feet and is therefore not further considered.

Recommended Layout and Cross Section

Reviewing the performance of each breakwater alternative as given in Table 10, it is apparent that a fishtail breakwater provides the best wave reduction. However, the difference in agitation as predicted by the model, which is less than an inch, is not great enough to warrant the additional expense created by the added rubblemound construction. Therefore, Alternative 1, the extended breakwater, is the recommended layout.

To allow land based construction, modifications will be required to the existing breakwater. This will require removal of the larger armor stone material and placment of a bed of coarse stone which will act as a roadway for the construction equipment. It is recommended that the 'roadway' be a minimum of 12 feet wide to accommodate smaller construction equipment. Following construction, rubble will be replaced on top of the existing jetty to cover the newly placed coarse stone which will act as a filter. This may, depending on the cross section of the existing breakwater, widen the structure. It is recommended that the changes to this breakwater be engineered in tandem with the new breakwater extension.

As discussed previously, the new breakwater extension will be designed to survive a more severe event than those analyzed in this study. The service life of a structure is typically less than the design event it is engineered for. This is due to the probability of a design event happening within a projects service life. The service life is defined as the amount of time a project is expected to function with only a low level of maintenance. Projects will generally continue to function well past their service life though it is advisable that they be thoroughly inspected and updated as needed.

A return period event has an approximate 64% probability of occurring within its given time frame. The relationship of return period design to the life of the project is shown in Table 11. A 64% probability of occurrence is generally too high when it comes to risk of damage to the structure and therefore it is typical to reduce damage risk by increasing the return period event for which a structure is designed. An owner must understand that there is savings in capital cost by reducing the return period event for which a project is designed but that the project may require more maintenance throughout its lifetime thereby negating the initial savings. For the purposes of this conceptual design, it is recommended that the design return period event for this breakwater be a 50 year event with no damage to the structure.

Percent Chance of Exceedance	Return Period (years)	Life of Project (years)									
		5	10	15	25	30	35	40	50	75	100
0.02	5000	0%	0%	0%	1%	1%	1%	1%	1%	2%	2%
0.1	1000	1%	1%	2%	3%	3%	3%	4%	5%	7%	10%
0.2	500	1%	2%	3%	5%	6%	7%	8%	10%	14%	18%
0.5	200	3%	5%	7%	12%	14%	16%	18%	22%	31%	39%
1	100	5%	10%	14%	22%	26%	30%	33%	40%	53%	63%
2	50	10%	18%	26%	40%	46%	51%	55%	64%	78%	87%
2.5	40	12%	22%	32%	47%	53%	59%	64%	72%	85%	92%
4	25	19%	34%	46%	64%	71%	76%	81%	87%	95%	98%
10	10	41%	65%	79%	93%	96%	98%	99%	100%	100%	100%
20	5	67%	89%	97%	100%	100%	100%	100%	100%	100%	100%
50	2	97%	100%	100%	100%	100%	100%	100%	100%	100%	100%
99	1	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%

Table 11 Probability of a Given Return Period within a Project Life

Reviewing the extreme wave events shown in Table 9, it can be seen that the 50 year event from the east results in the highest wave heights near the boat launch (i.e. 2.94 feet). Using this wave height, the breakwater height and the armor stone size can be determined.

The breakwater height should be at least one wave height above the waterline to limit overtopping. Therefore, assuming a design life of 50 years, the 50 year water level given in Table 2, 184.02 m (603.6 feet), with the added wave height suggests the crest height should be a minimum of 606.5 feet.

Using the Hudson equation to determine the median rock size (D_{50}) and the Coastal Engineering Manual's recommendations for breakwater cross sections and rock sizes, the following characteristics, shown in Table 12, are determined. It should be noted that the numbers provided should be accepted as conceptual design only and further engineering should be performed to develop a cross section for construction.

Table 12 Breakwater Stone Sizes

Layer	D_{50}	Size Range	Layer Thickness
Armor Stone	1.3 ft	1 – 1.6 ft	2.6 ft
Filter Stone	7 in	5 – 9 in	2.0 ft
Core	2.5 in	1 – 4.5 in	-

In order to construct the breakwater from the land, an adequate 'roadway' constructed of core material is required. This suggests that the core material have a width of at least 12 feet above the waterline. For the purposes of a conceptual design, the crest of this roadway was placed 1 foot above the current waterline. This may be adjusted later depending on actual construction equipment used and their associated weights.

A plan and cross section of the described breakwater is shown in Figure 23 and Figure 24. The estimated volume for this construction is 1,250 cubic yards though additional material will be required for alterations to the existing groin.

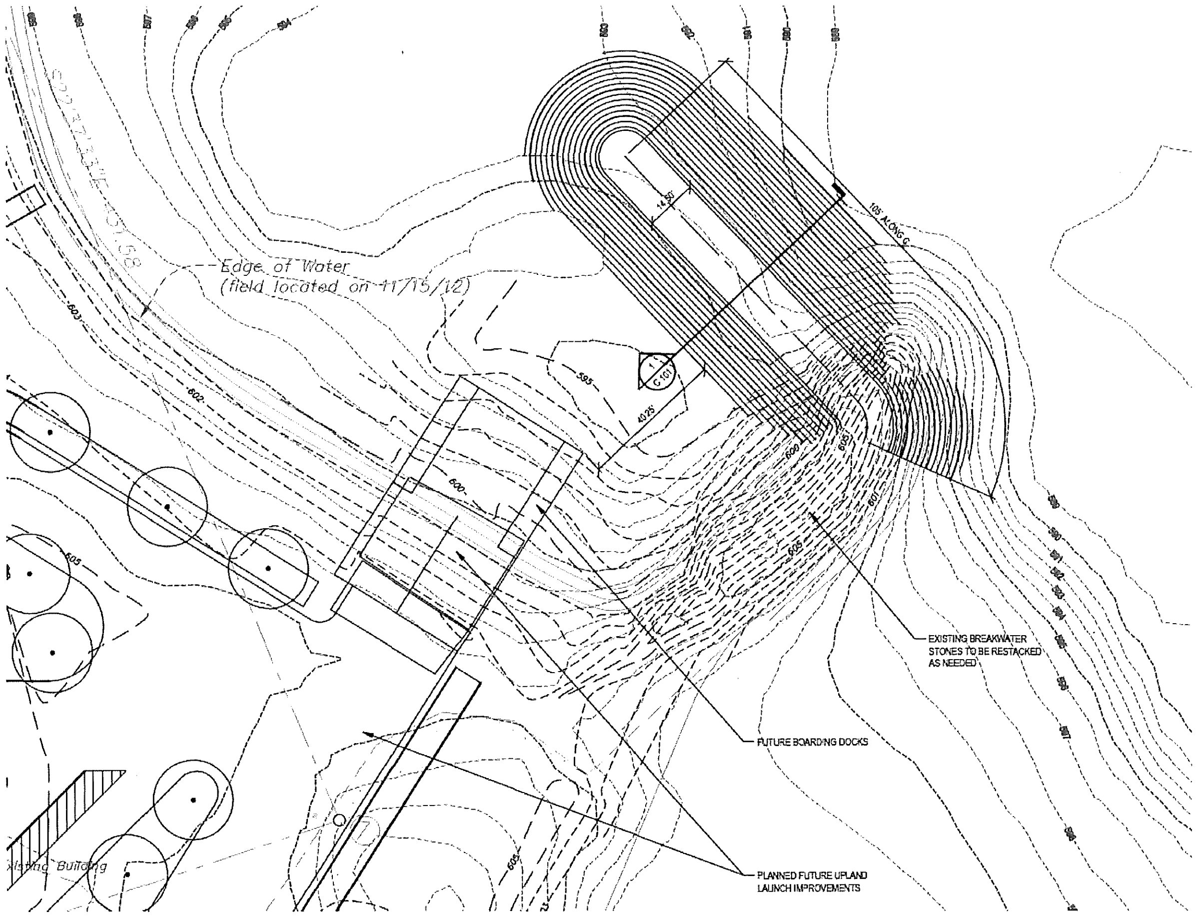


Figure 23 Plan View of Breakwater Extension

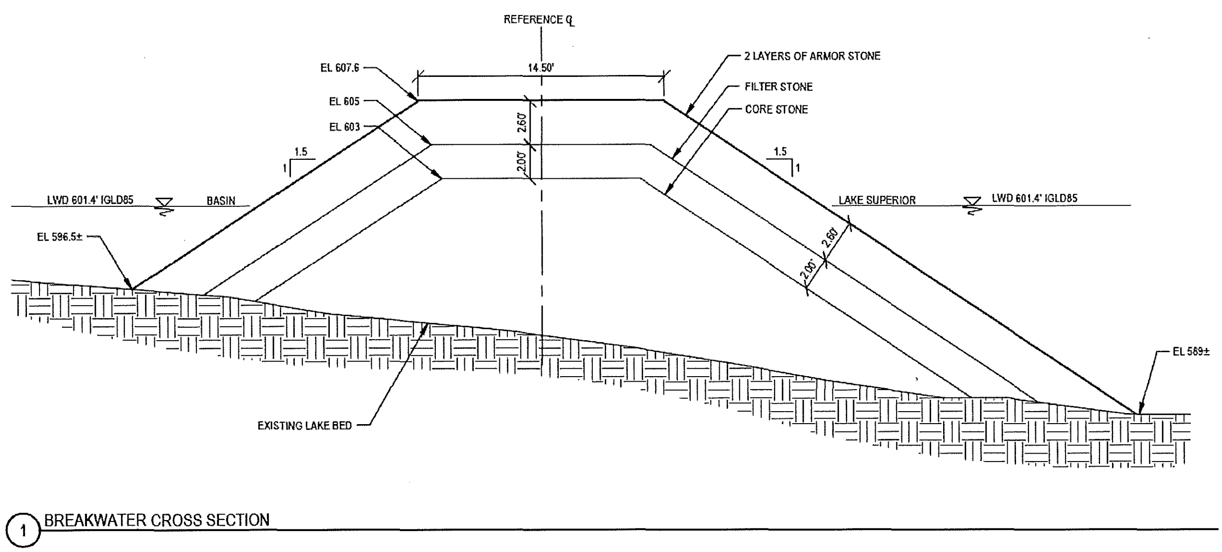


Figure 24 Cross Section of Breakwater Extension

Levels of Upland Inundation/Surge Protection

The levels of upland inundation were calculated using a smooth particle hydrodynamic model over the small domain, in a virtual 170x230m wave tank. The 50 year return period wave event and 50 year surge were used as boundary conditions for the model. The maximum incoming wave was used in the model to estimate the maximum inundation level. Additionally, the reflected waves from the harbor's inner breakwater that affect the project site were also included in the model. Two serpent type flat wave generators were used at both incoming wave boundaries, based on the Biesel transfer function.

The maximum water level was determined to be 5.5 feet above LWD at 606.5 foot level for the 50 year return period event. Figure 26 shows the maximum inundation level estimated for the 50 year return period event in relation to the proposed upland facilities.

Figure 25 SPH Inundation Model

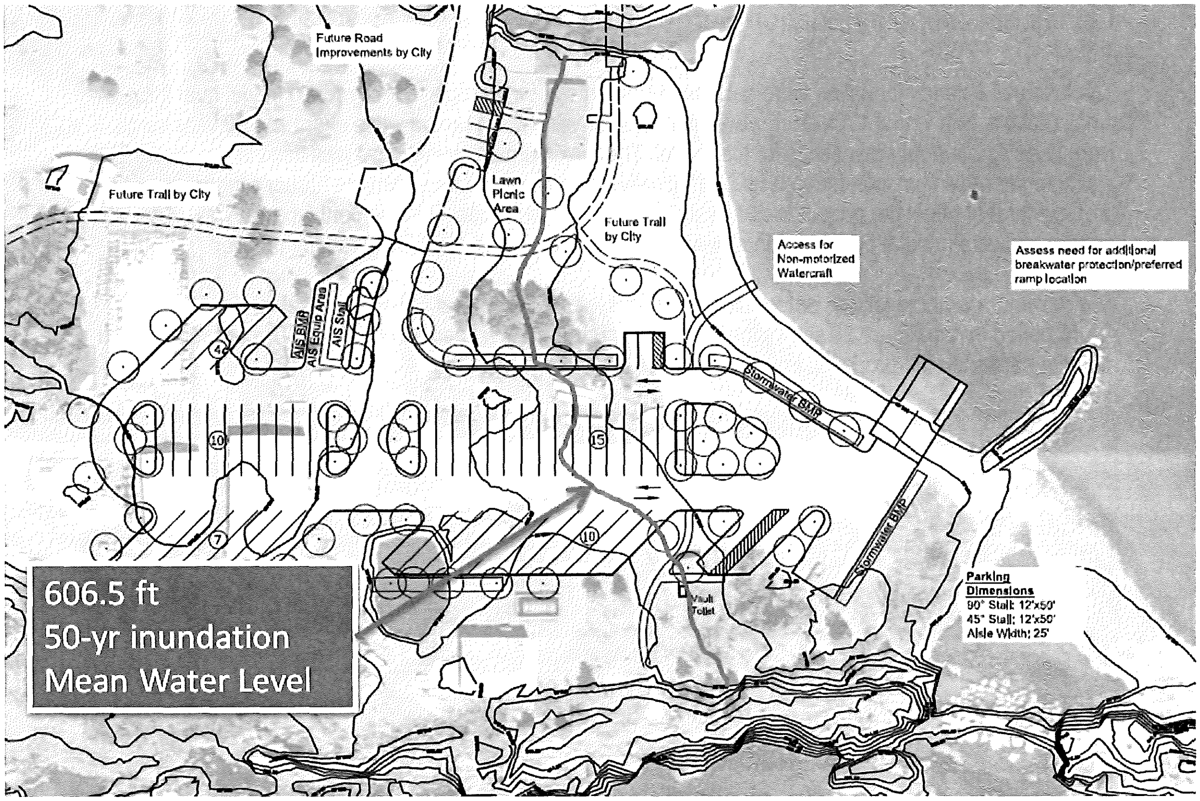


Figure 26 Maximum Inundation Level for 50-Year Return Period Event

Analysis of Perennial Stream

A perennial stream draining an area of approximately 540 acres discharges to Lake Superior approximately 300 feet north of the boat launch area. The stream drains generally in a northerly to southerly direction over moderately to steeply sloping terrain. Watershed slopes typically range from greater than 15% in upland areas to approximately 3% in lower areas near Lake Superior. Land use consists of mostly forested areas interspersed with scattered homes. Primary areas of watershed development include small clusters of development along Highway 61 and the campground itself. Soils are believed to consist primarily of sandy loams with clay and rocks present. Bedrock is present at shallow depths, particularly in upland areas.

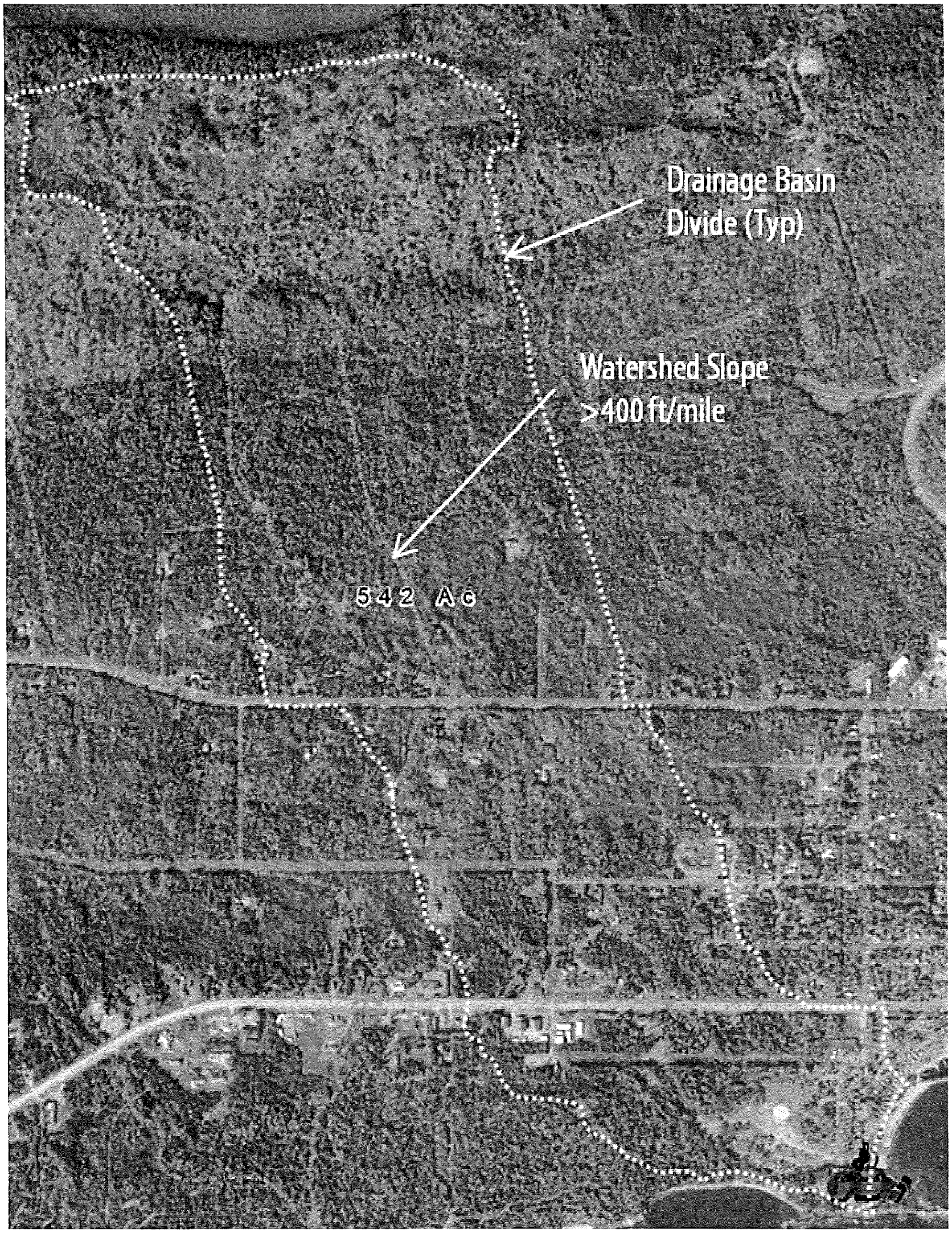


Figure 27 Perennial Stream Drainage Basin

While detailed sediment transport analysis of the stream was not completed for this study, it is believed that the sediment load is relatively low based upon previous water sampling of the stream. This is due to the dense vegetative cover of the tributary and floodplain areas. This dense cover should limit the amount of soil erosion and sediment transported to the site by the stream. The primary potential source of sediment loading in the stream is likely stream bank erosion in urbanized portions of the watershed – primarily the campground itself. Another potential source of loading could be leaf fall from deciduous trees during the autumn months.

Modeling and review of aerial photography indicates a pattern of littoral drift from north to south in the nearshore zone with a slight accumulation of sediment near the existing boat launch. There is no evidence to suggest an increase in sediment loading near the mouth of the stream. While the construction of a new coastal structure may change the drift pattern, visual evidence of increasing sediment accumulation attributable to the stream location should be closely monitored. However, documentation reviewed for preparation of this report provides no evidence to suggest that large scale remedial measures such as stream relocation or construction of an upland sediment trap are necessary to enhance operation of the boat launch.

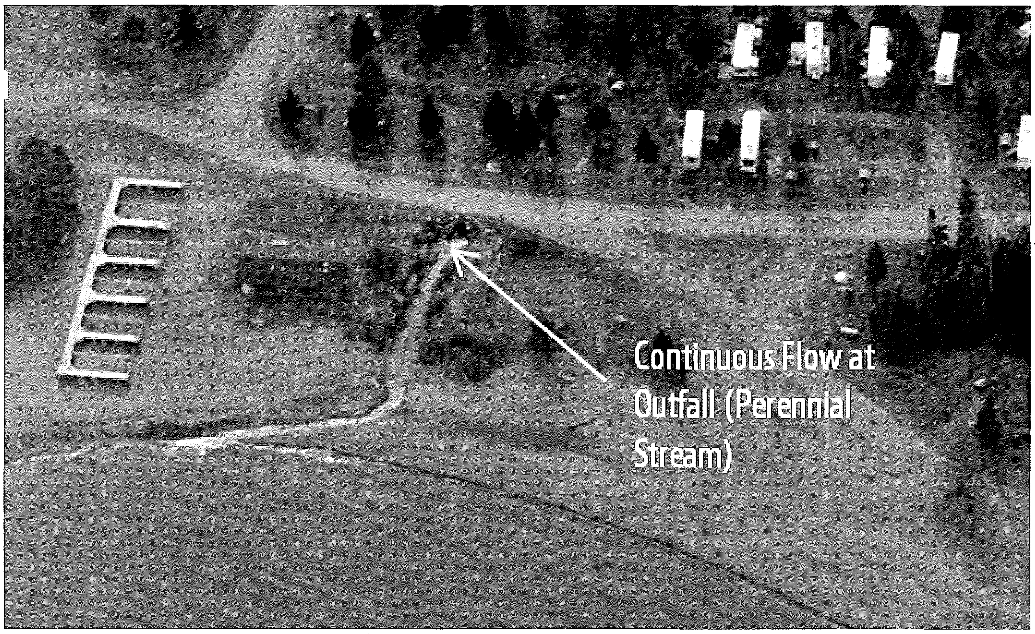
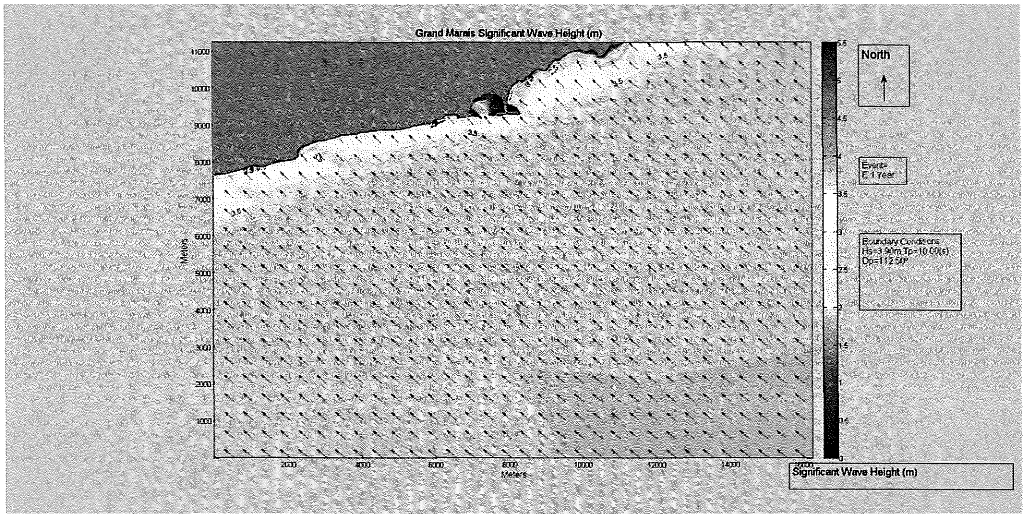
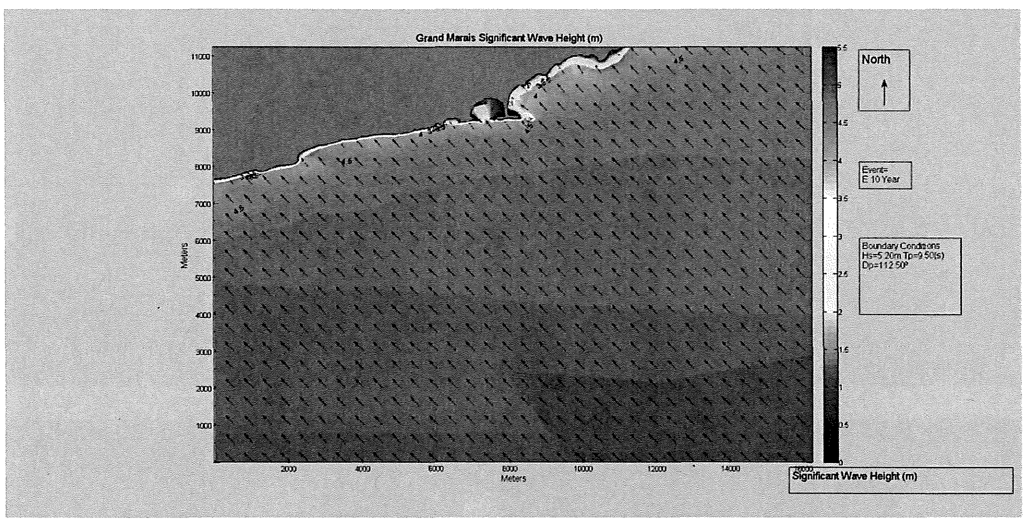


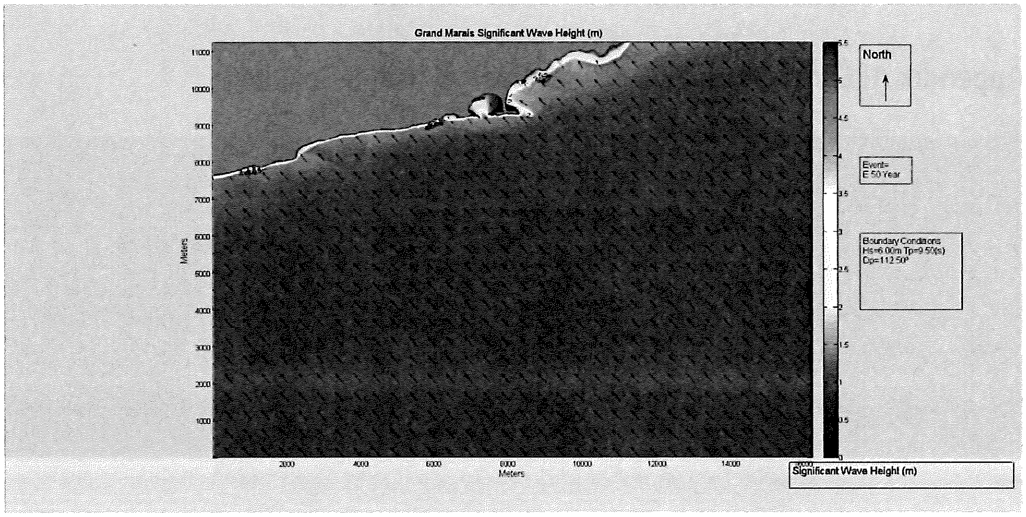
Figure 28 Perennial Steam Outfall

Sediment transport from upland areas to the mouth of the stream could increase significantly if watershed development or clearing of forested areas accelerates in the future. An effective stormwater management program maintained by the City of Grand Marais will help control sediment discharge from the stream to the Lake and boat launch area should this occur. This program, at a minimum, should require and enforce erosion and sediment controls for upland construction sites, stormwater management for new development, and provide a leaf and yard waste collection program. In addition, the condition of the stream banks should be inspected periodically and remedial actions taken to stabilize eroding banks.

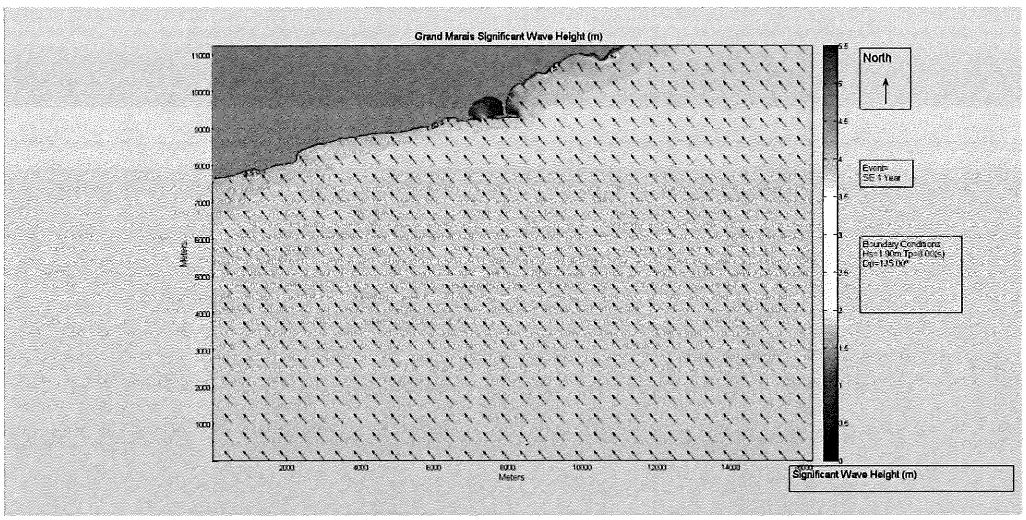
Appendix A: Large Scale STWAVE Numerical Model Results



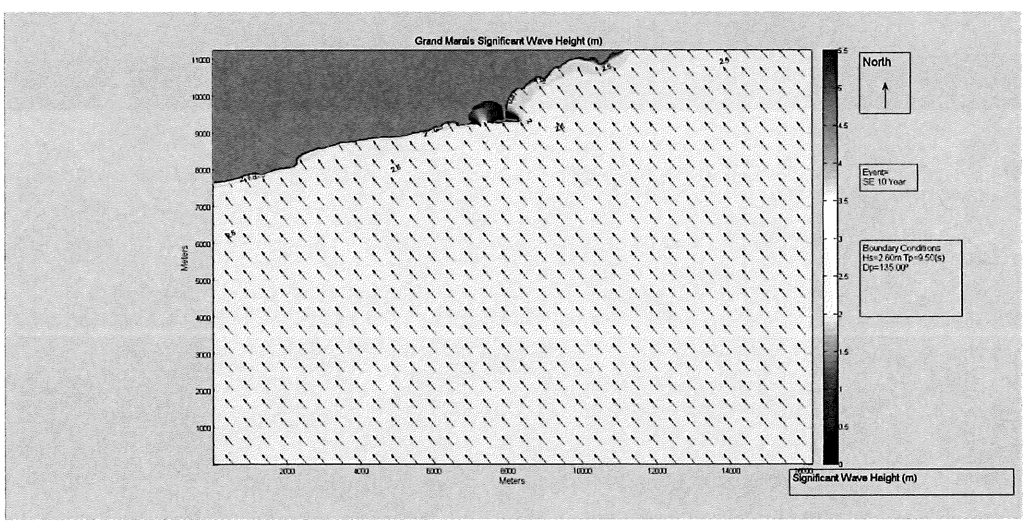




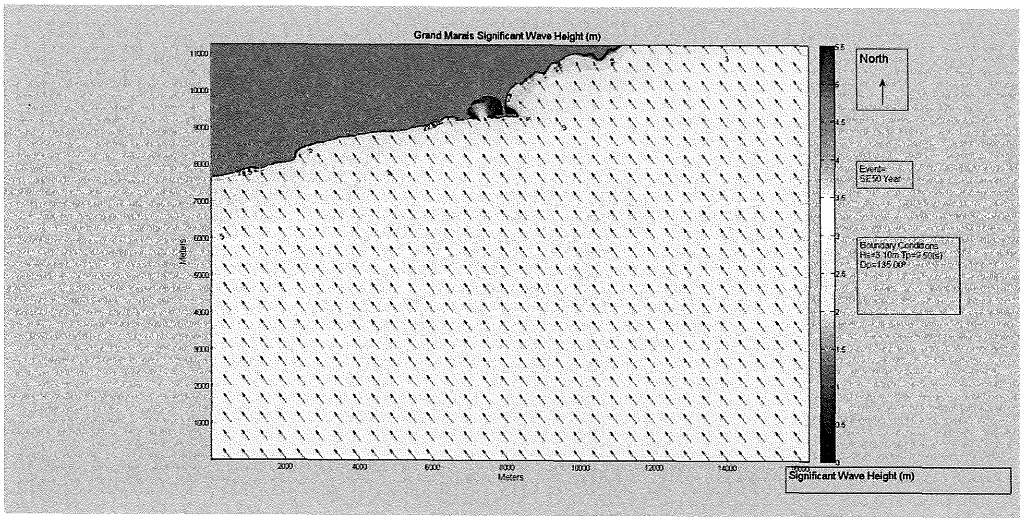
Grand Marais Significant Wave Height (m)
North
Event= E 50 Year
Boundary Conditions
Hs=6.00m Tp=9.50(s)
Dp=112.50°
Meters
Significant Wave Height (m)

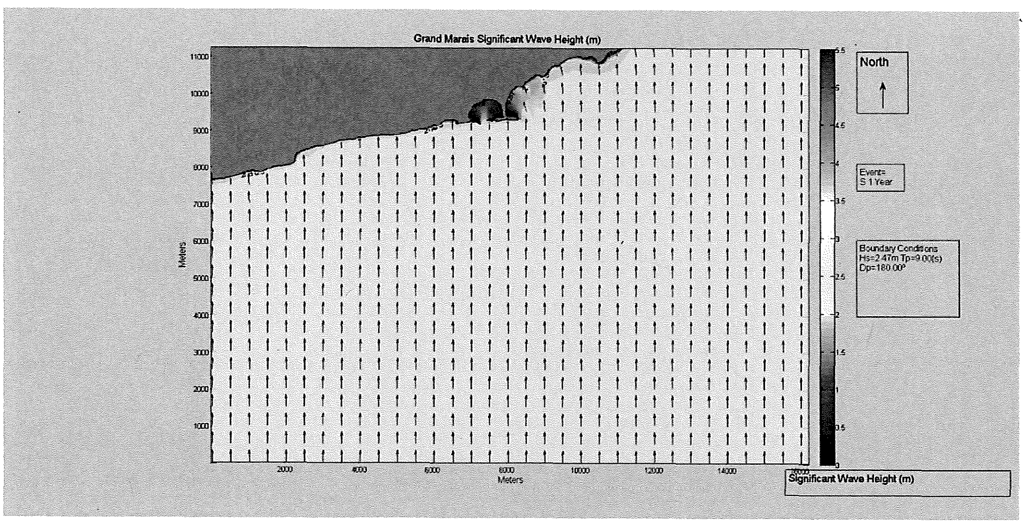


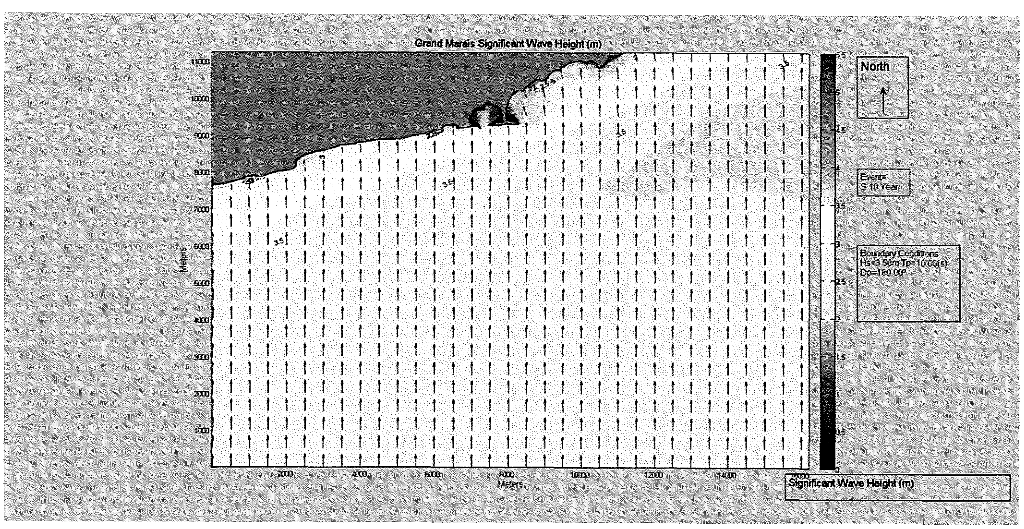
Grand Marais Significant Wave Height (m)
North
Event= SE 1 Year
Boundary Conditions
Hs=1.90m Tp=8.00(s)
Dp=135.00°
Meters
Significant Wave Height (m)

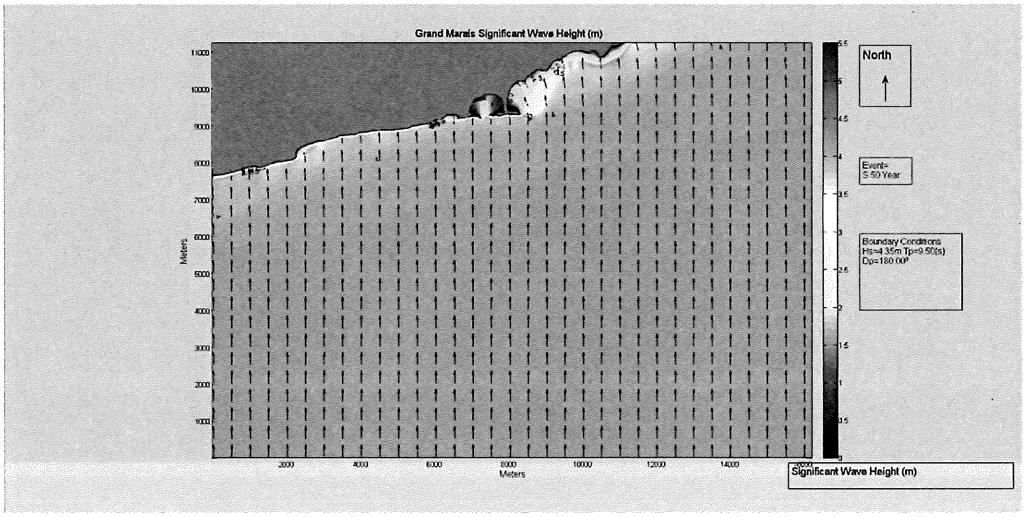


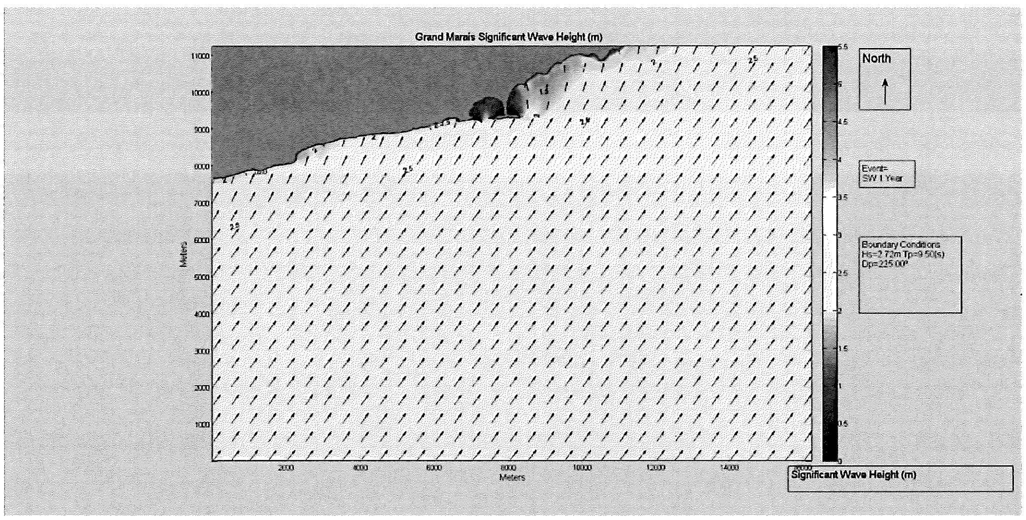
Grand Marais Significant Wave Height (m)
North
Event= SE 10 Year
Boundary Conditions
Hs=2.60m Tp=9.50(s)
Dp=135.00°
Meters
Significant Wave Height (m)

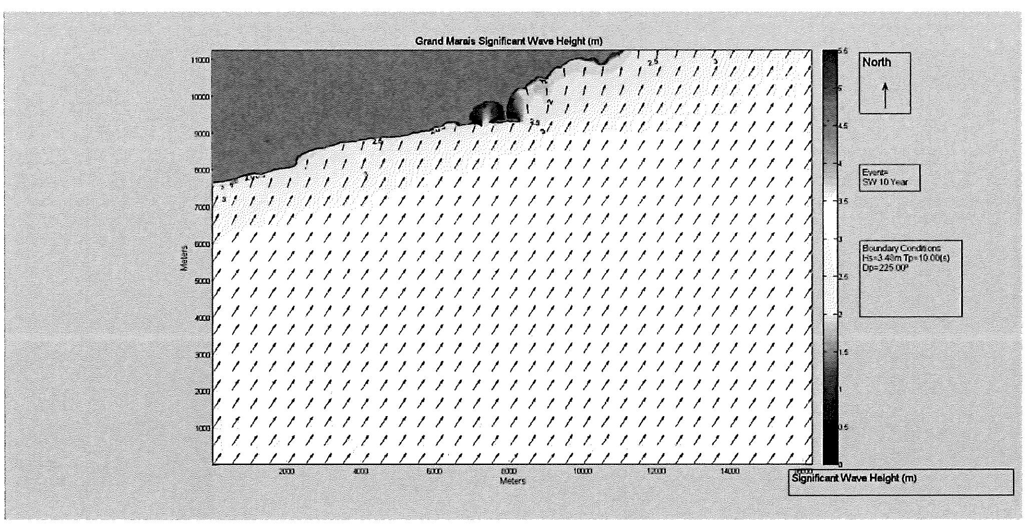

Grand Marais Significant Wave Height (m)
North
Event=
SE50 Year
Boundary Conditions
Hs=3.10m Tp=9.50(s)
Dp=135.00P
Meters
Meters
Significant Wave Height (m)

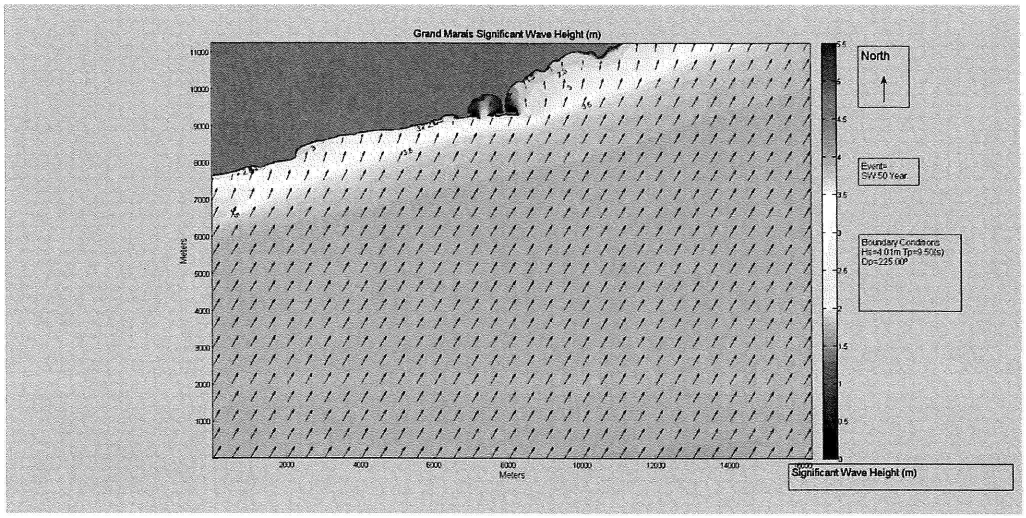

Grand Marais Significant Wave Height (m)
North
Event=
S 1 Year
Boundary Conditions
Hs=2.47m Tp=9.00(s)
Dp=180.00P
Meters
Meters
Significant Wave Height (m)

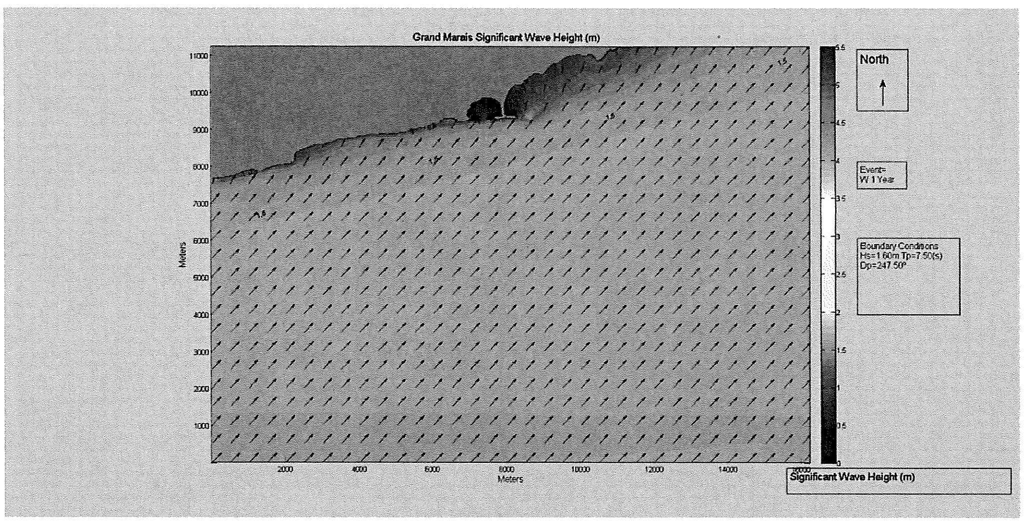

Grand Marais Significant Wave Height (m)
North
Event=
S 10 Year
Boundary Conditions
Hs=3.58m Tp=10.00(s)
Dp=180.00P
Meters
Meters
Significant Wave Height (m)

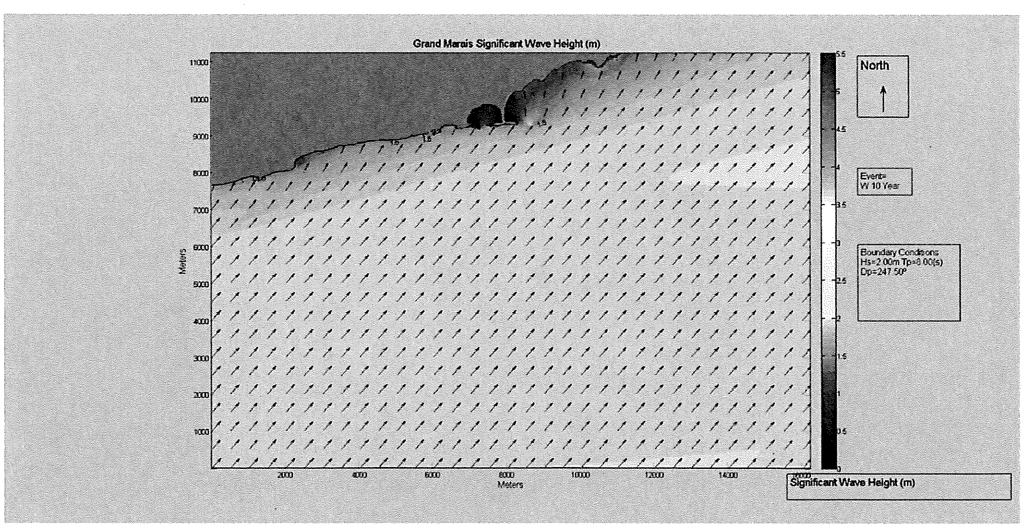

Grand Marais Significant Wave Height (m)
Meters
Meters
North
Event=
S 50 Year
Boundary Conditions
Hs=4.35m Tp=9.50(s)
Dp=180.00°
Significant Wave Height (m)

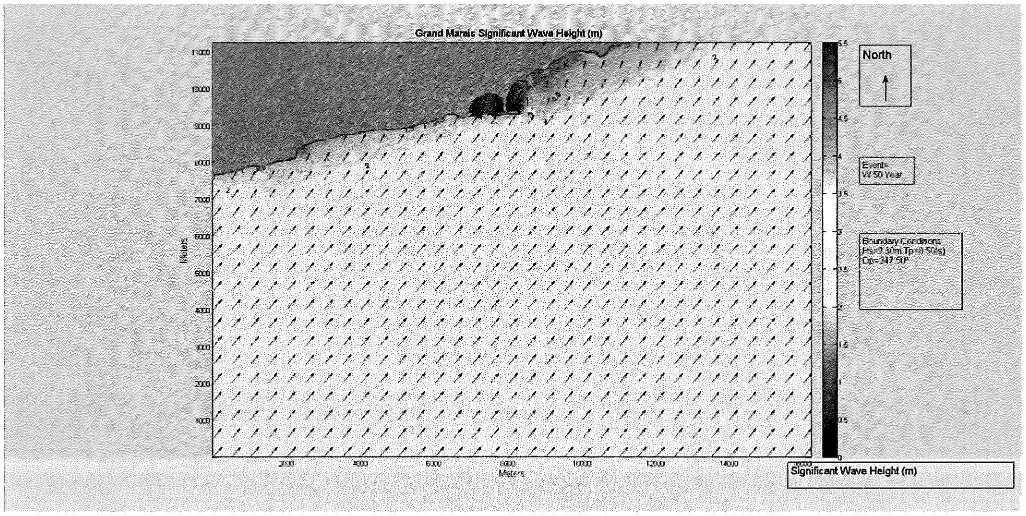

Grand Marais Significant Wave Height (m)
Meters
Meters
North
Event=
SW 1 Year
Boundary Conditions
Hs=2.72m Tp=9.50(s)
Dp=225.00°
Significant Wave Height (m)


Grand Marais Significant Wave Height (m)
Meters
Meters
North
Event=
SW 10 Year
Boundary Conditions
Hs=3.48m Tp=10.00(s)
Dp=225.00°
Significant Wave Height (m)


Grand Marais Significant Wave Height (m)
Meters
Meters
North
Event=
SW 50 Year
Boundary Conditions
Hs=4.01m Tp=9.50(s)
Dp=225.00º
Significant Wave Height (m)


Grand Marais Significant Wave Height (m)
Meters
Meters
North
Event=
W 1 Year
Boundary Conditions
Hs=1.60m Tp=7.50(s)
Dp=247.50º
Significant Wave Height (m)

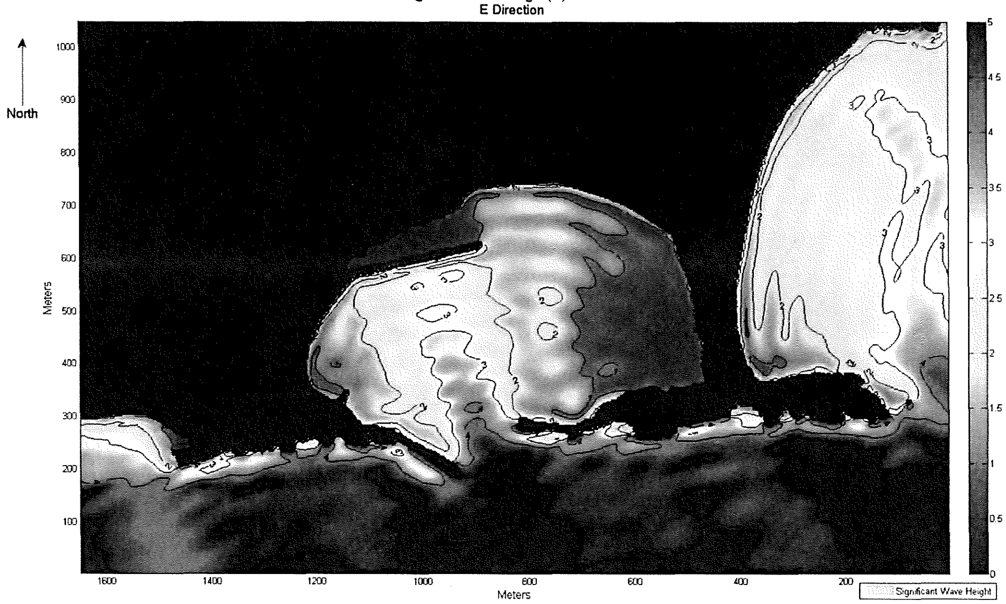

Grand Marais Significant Wave Height (m)
Meters
Meters
North
Event=
W 10 Year
Boundary Conditions
Hs=2.00m Tp=8.00(s)
Dp=247.50º
Significant Wave Height (m)



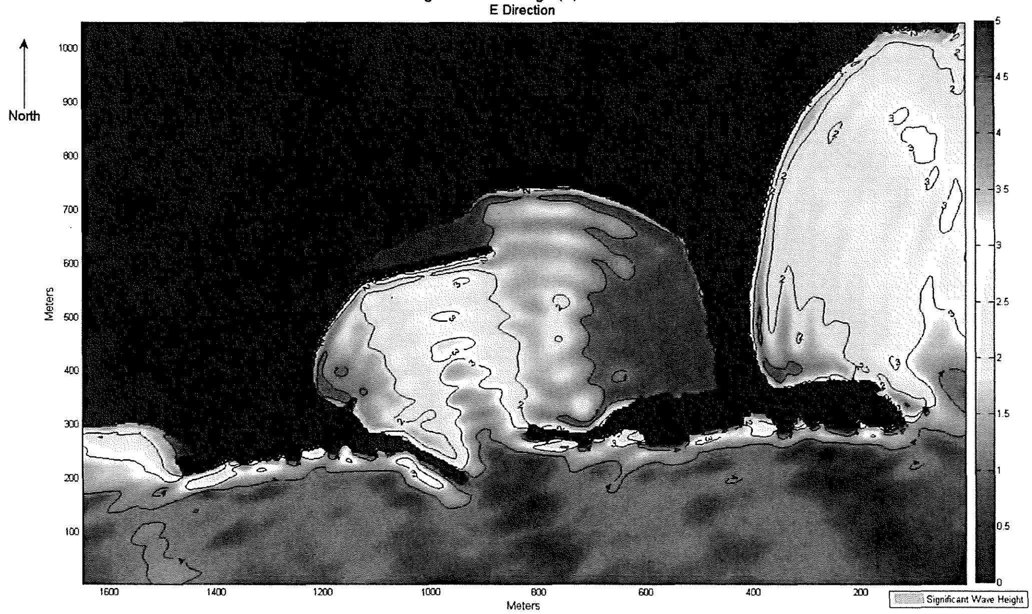
Grand Marais Significant Wave Height (m)
Meters
Meters
North
Event=
W 50 Year
Boundary Conditions
Hs=2.30m Tp=8.50(s)
Dp=247.50°
Significant Wave Height (m)

Appendix B: Small Scale Boussinesq Numerical Model Results Extreme Events

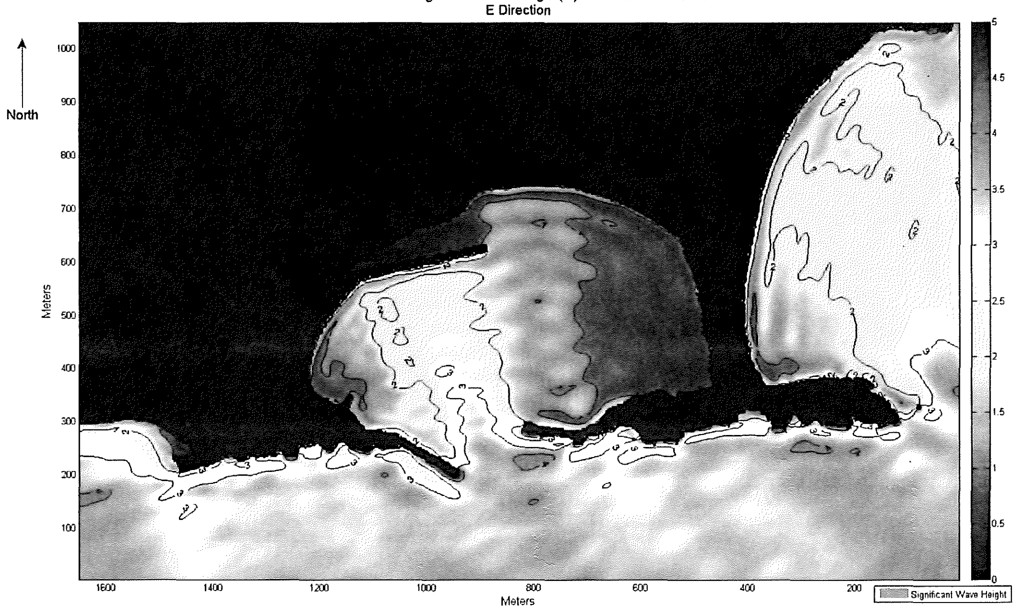
Grand Marais Significant Wave Height (m) Return Period 50 Year
E Direction



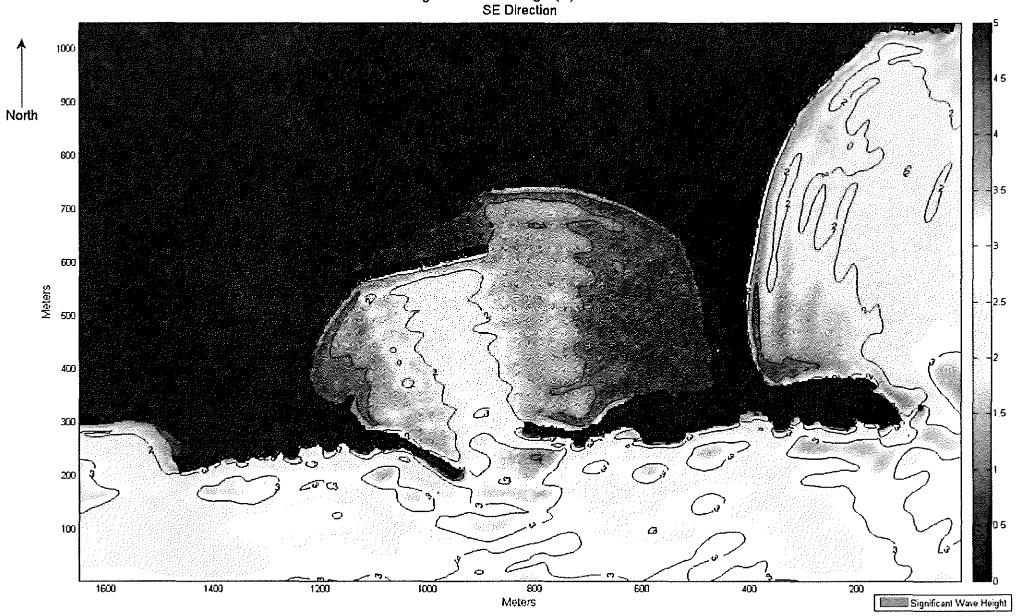
Grand Marais Significant Wave Height (m) Return Period 10 Year
E Direction



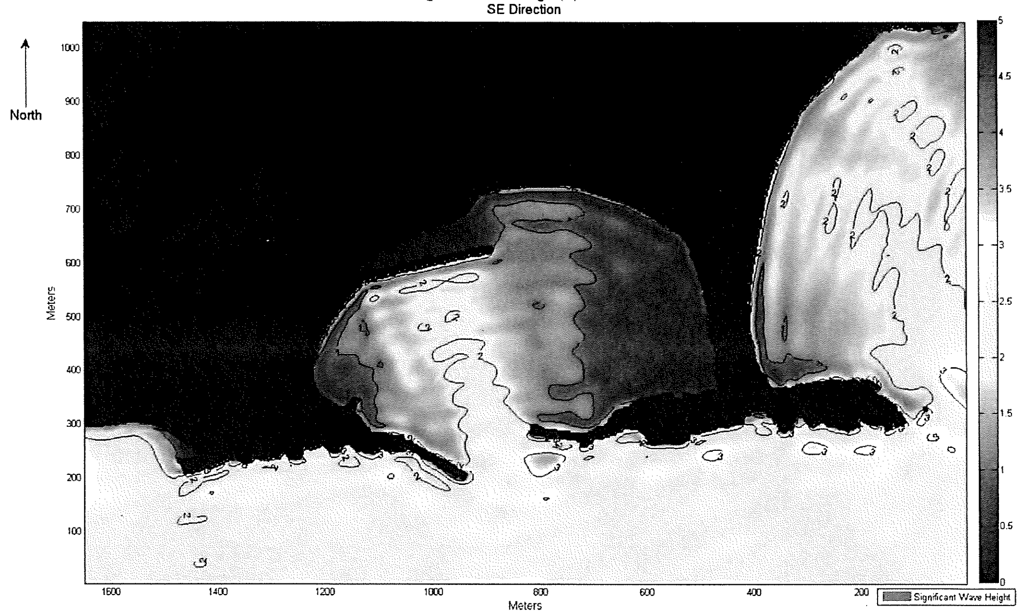
Grand Marais Significant Wave Height (m) Return Period 1 Year
E Direction



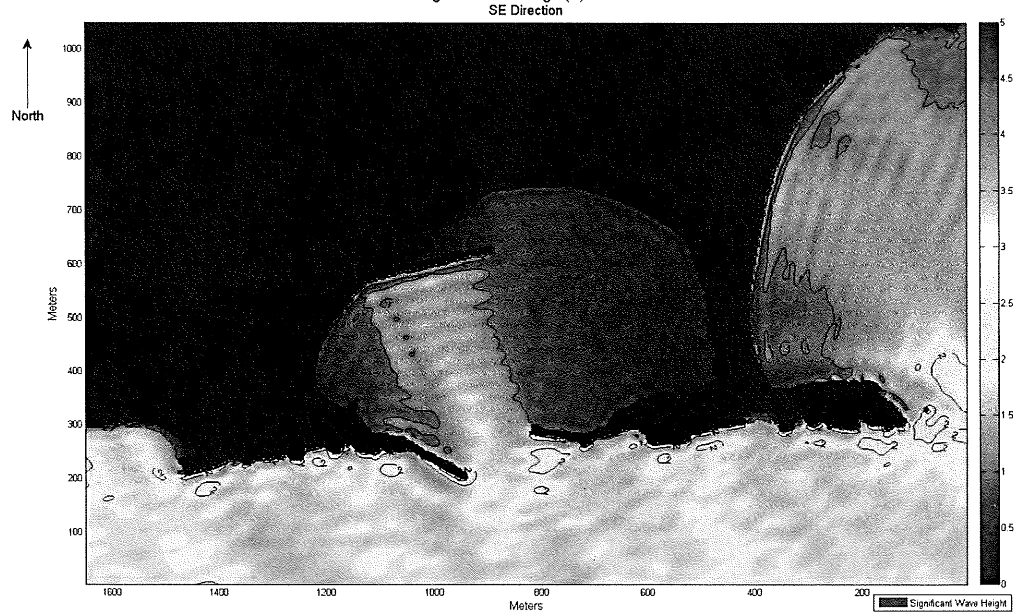
Grand Marais Significant Wave Height (m) Return Period 50 Year
SE Direction



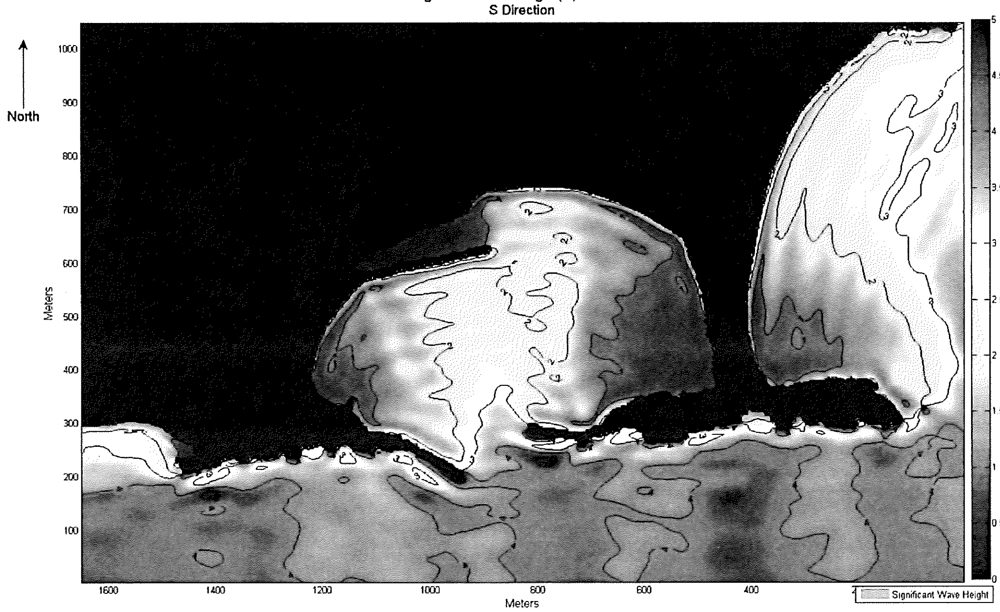
Grand Marais Significant Wave Height (m) Return Period 10 Year
SE Direction



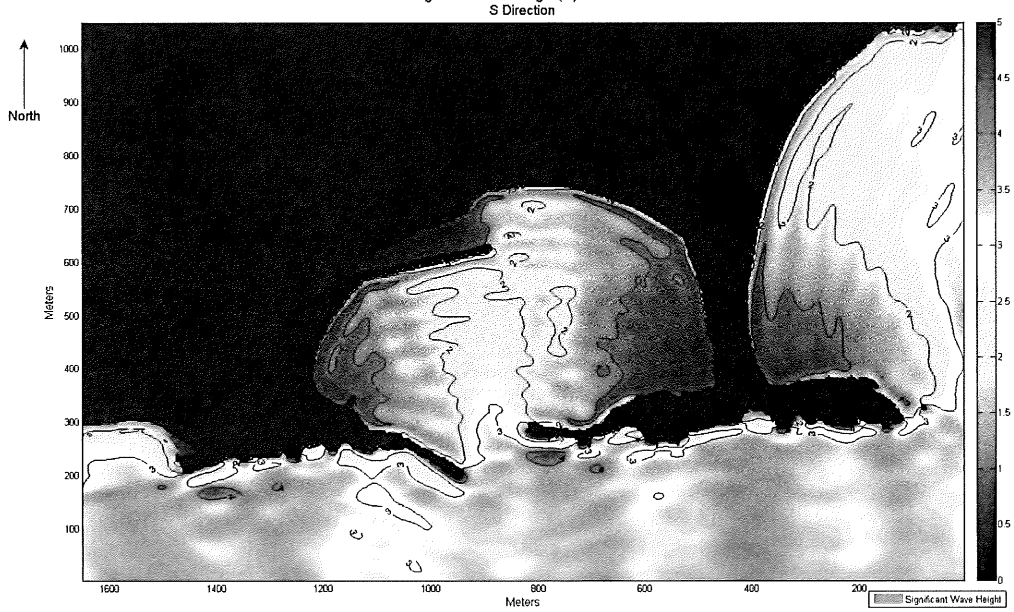
Grand Marais Significant Wave Height (m) Return Period 1 Year
SE Direction



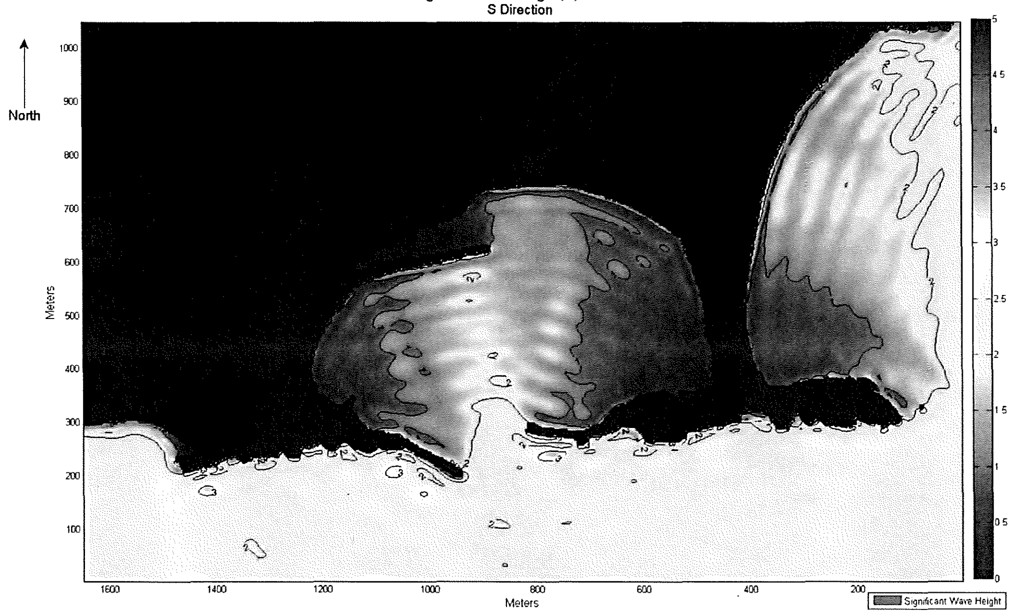
Grand Marais Significant Wave Height (m) Return Period 50 Year
S Direction



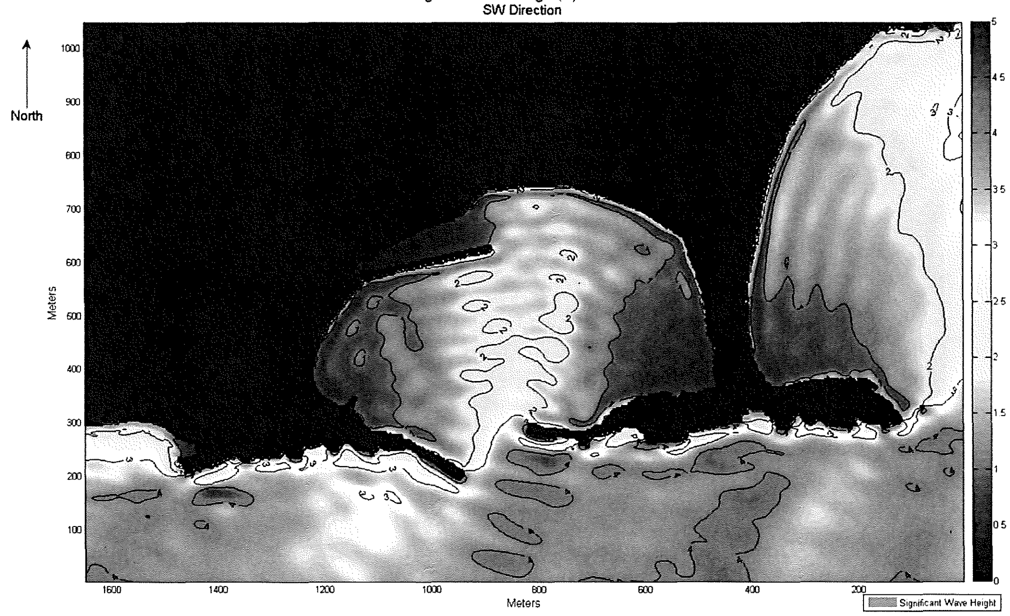
Grand Marais Significant Wave Height (m) Return Period 10 Year
S Direction



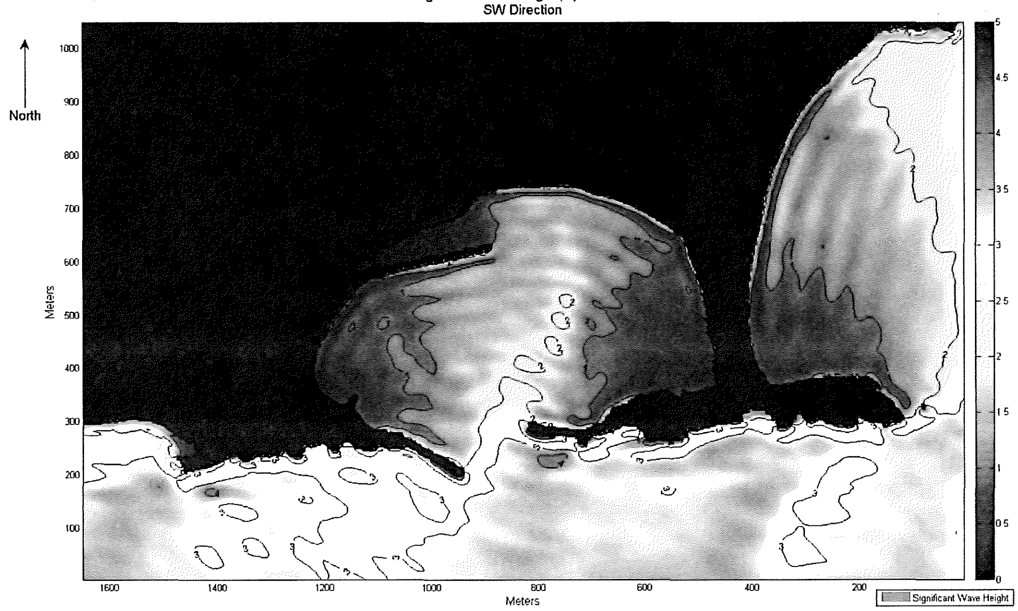
Grand Marais Significant Wave Height (m) Return Period 1 Year
S Direction



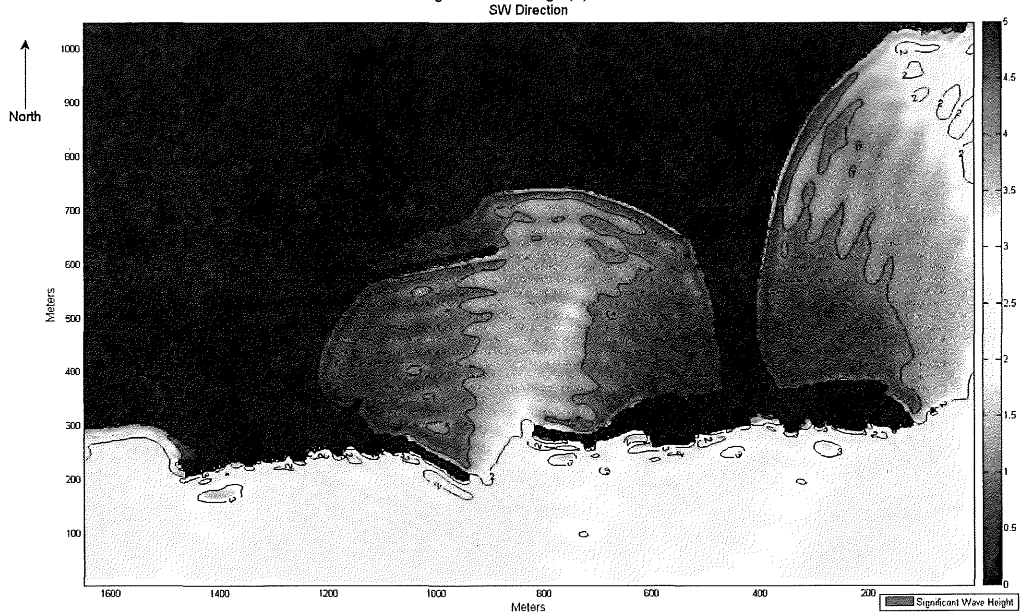
Grand Marais Significant Wave Height (m) Return Period 50 Year
SW Direction



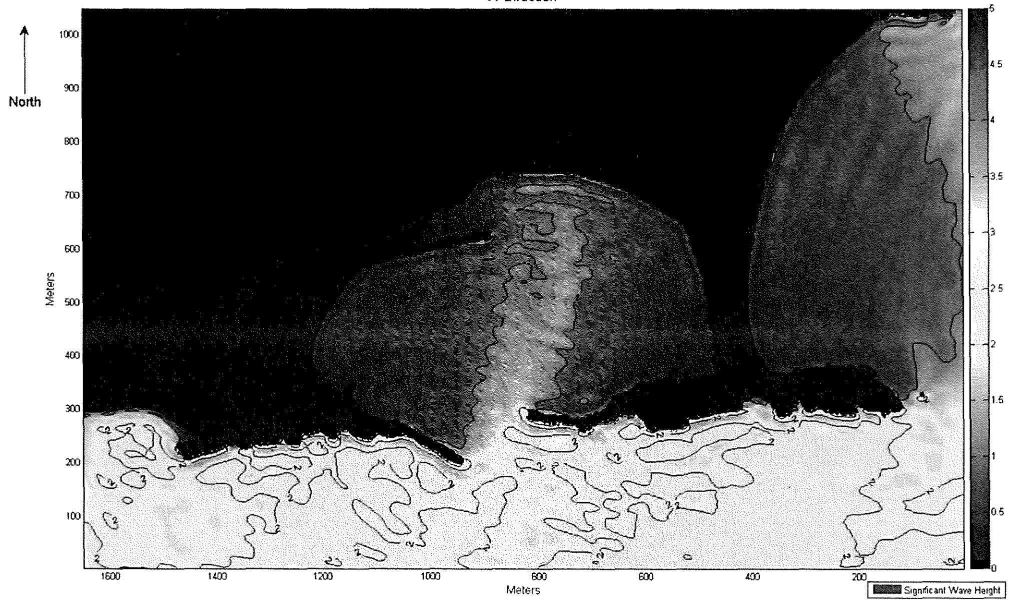
Grand Marais Significant Wave Height (m) Return Period 10 Year
SW Direction



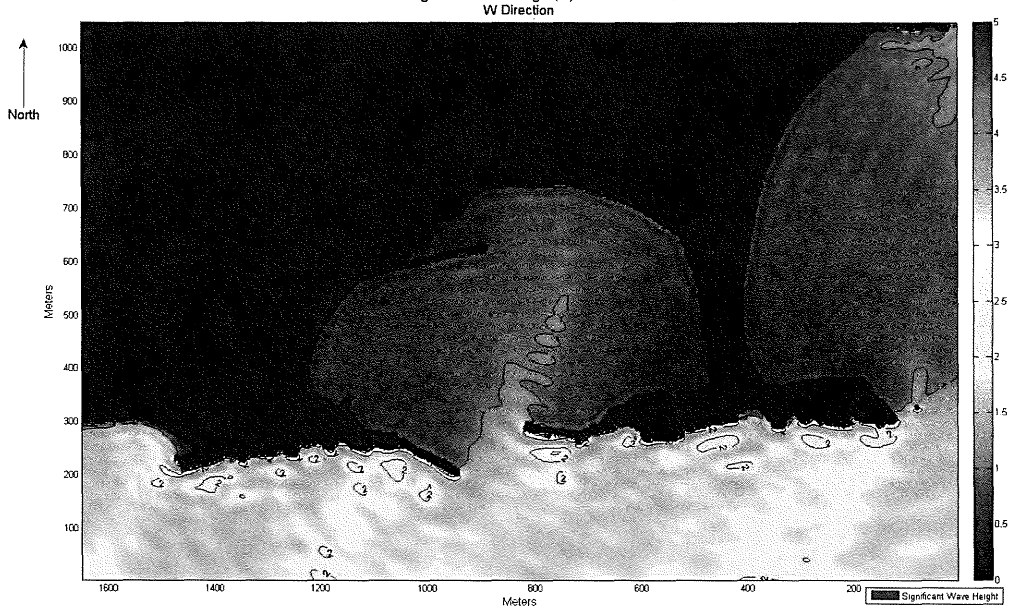
Grand Marais Significant Wave Height (m) Return Period 1 Year
SW Direction



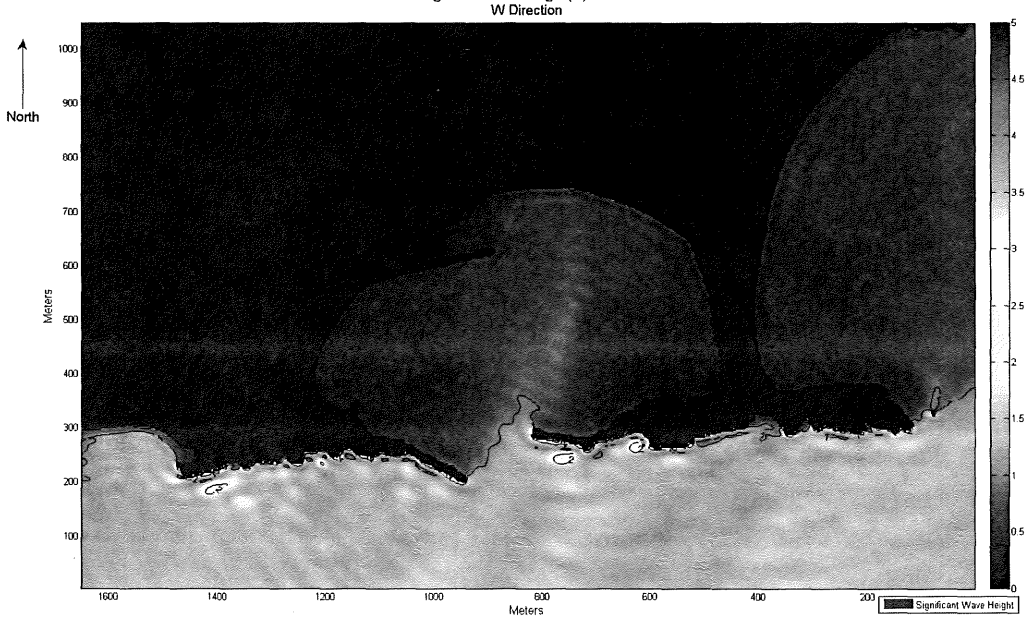
Grand Marais Significant Wave Height (m) Return Period 50 Year
W Direction



Grand Marais Significant Wave Height (m) Return Period 10 Year
W Direction



Grand Marais Significant Wave Height (m) Return Period 1 Year
W Direction



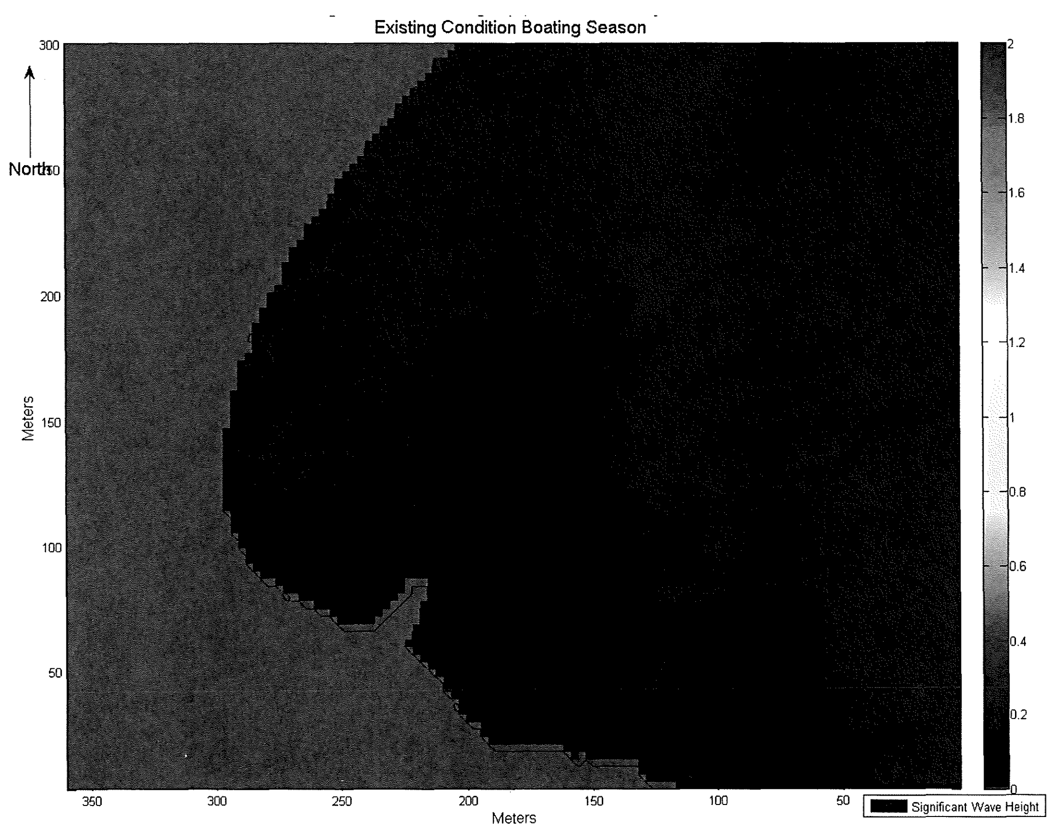
Appendix C: Small Scale Boussinesq Numerical Model Results Conditions Matrix

Wave Criteria for Modelling (all easterly direction)

Wave Climate Condition Boating Season	Deepwater Wave Height Hmo	Peak Wave Period Tp
Condition 1 (1 month)	3.9 ft (1.2 m)	6.5 s
Condition 2 (6 month)	4.8 ft (1.5 m)	7.0 s
Condition 3 (1 year)	5.7 ft (1.8 m)	7.5 s
Condition 4 (20 year)	8.0 ft (2.5 m)	8.8 s

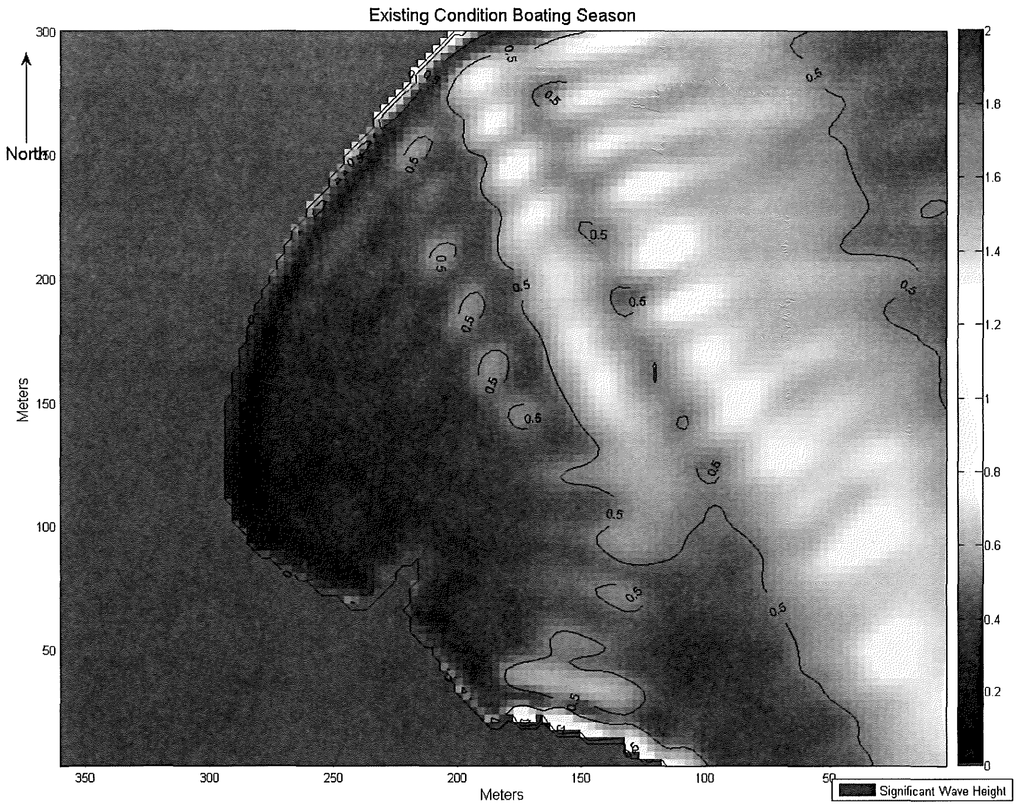
- Condition 1:Hmo=3.9ft Tp=6.5s

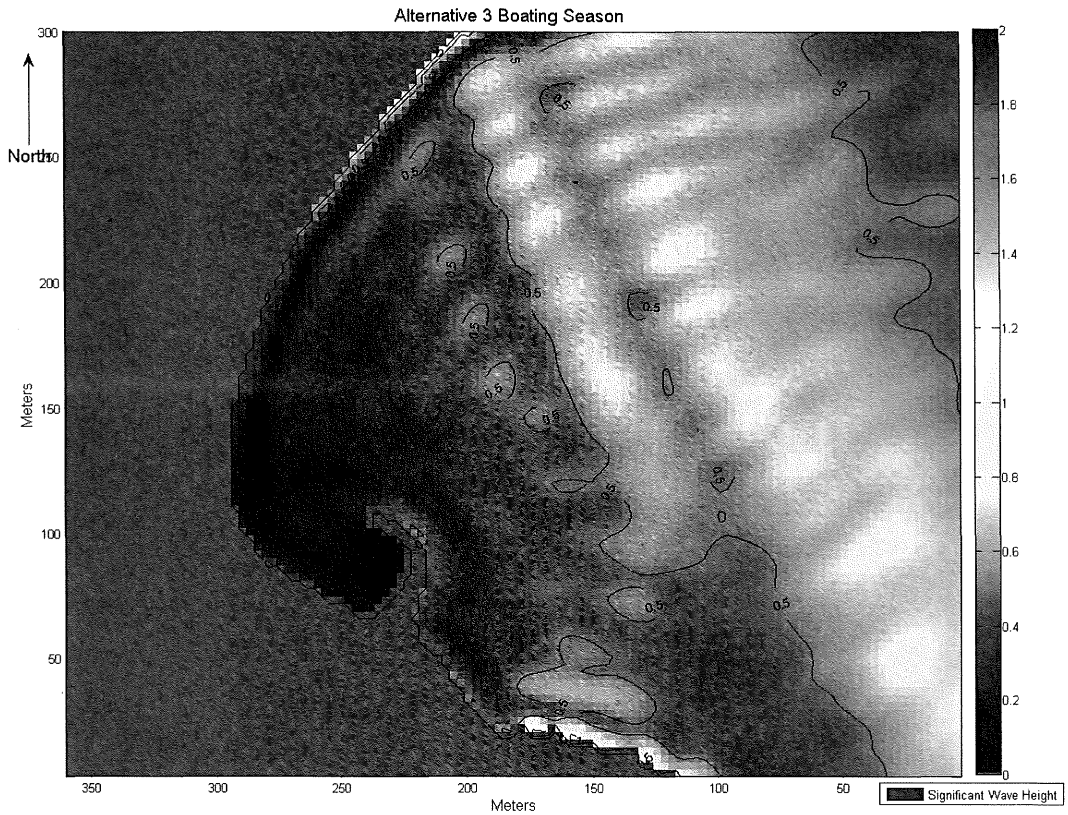
Alternative	Mean Hmo (ft)
Existing condition	0.44



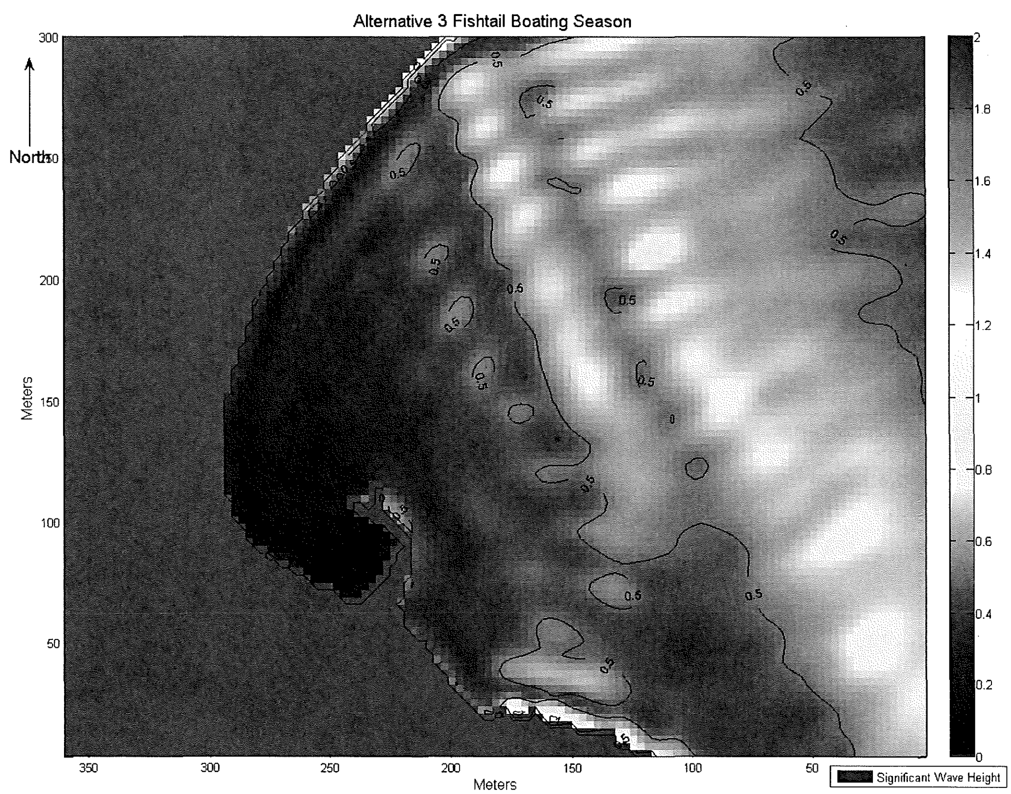
- Condition 2: Hmo=4.92ft Tp=7.0s

Alternative	Mean Hmo (ft)	Relative Improvement
Existing condition	**0.83**	**0.00%**
Extended breakwater	**0.43**	**48.36%**
Extended breakwater Binwall	**0.47**	**43.07%**
Extended breakwater Fishtail	**0.36**	**56.80%**

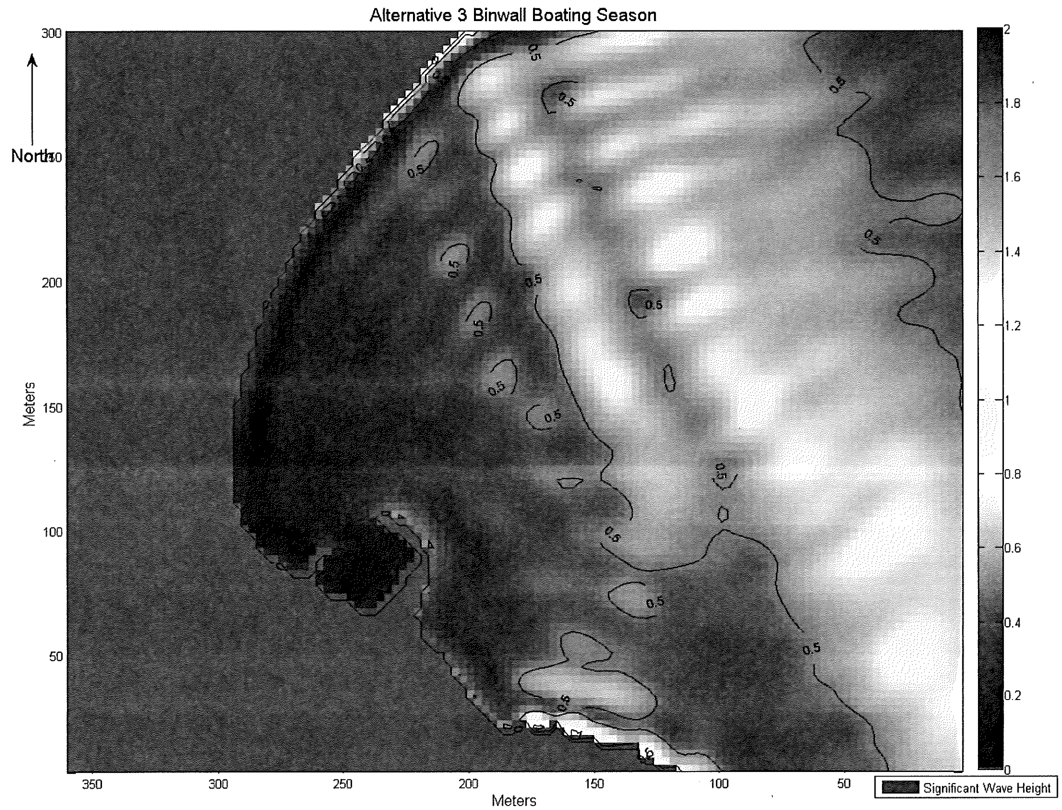




Alternative 3 Boating Season
North
Meters
Significant Wave Height

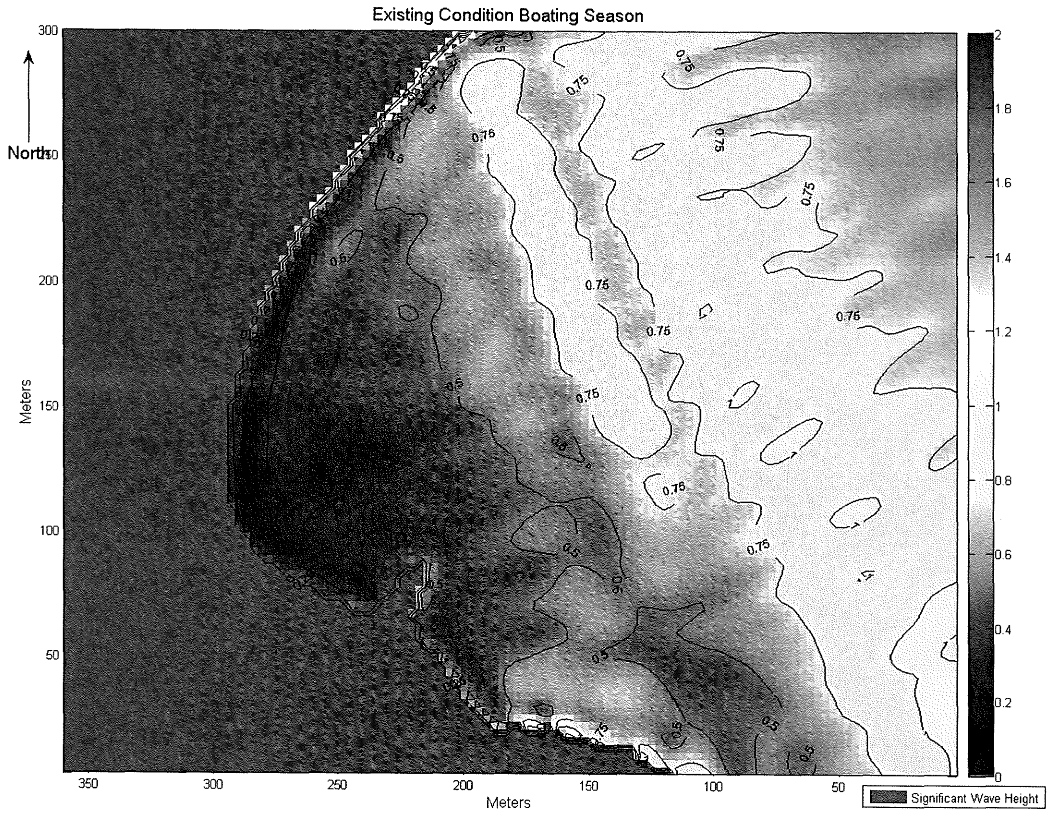


Alternative 3 Fishtail Boating Season
North
Meters
Significant Wave Height

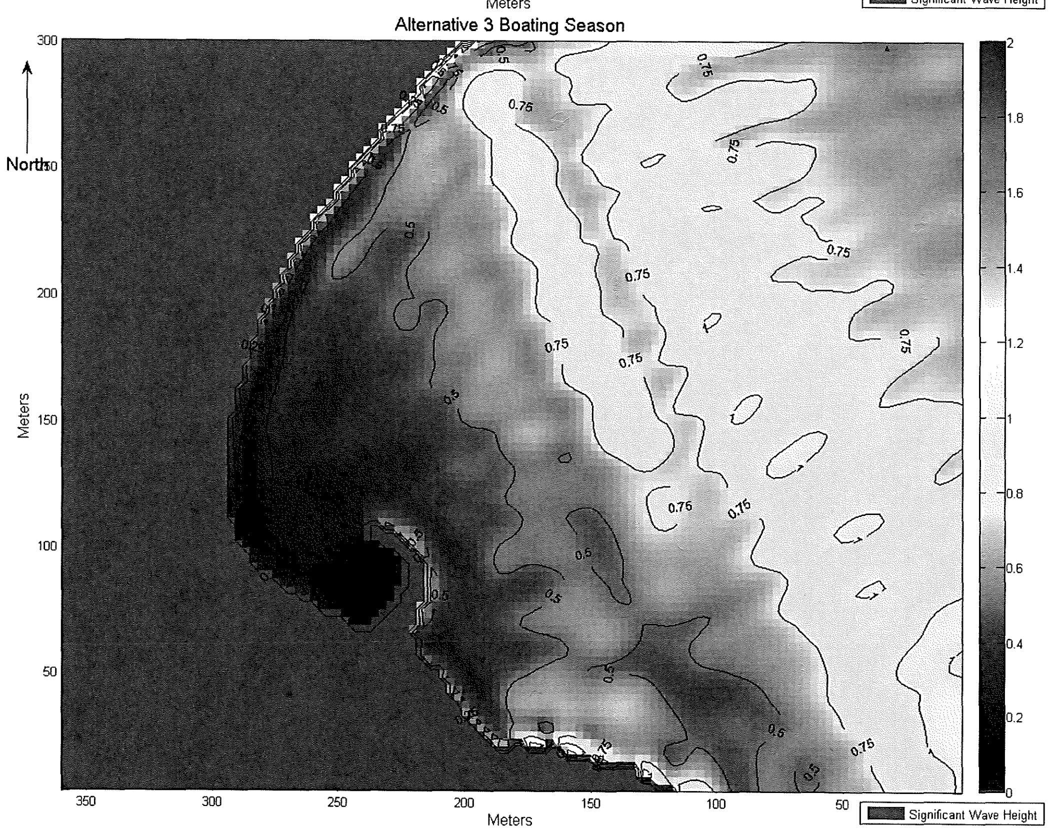


- Condition 3: Hmo=5.7ft Tp=7.5s

Alternative	Mean Hmo (ft)	Relative Improvement
Existing condition	**0.99**	**0.00%**
Extended breakwater	**0.46**	**53.67%**
Extended breakwater Binwall	**0.50**	**49.99%**
Extended breakwater Fishtail	**0.40**	**59.99%**

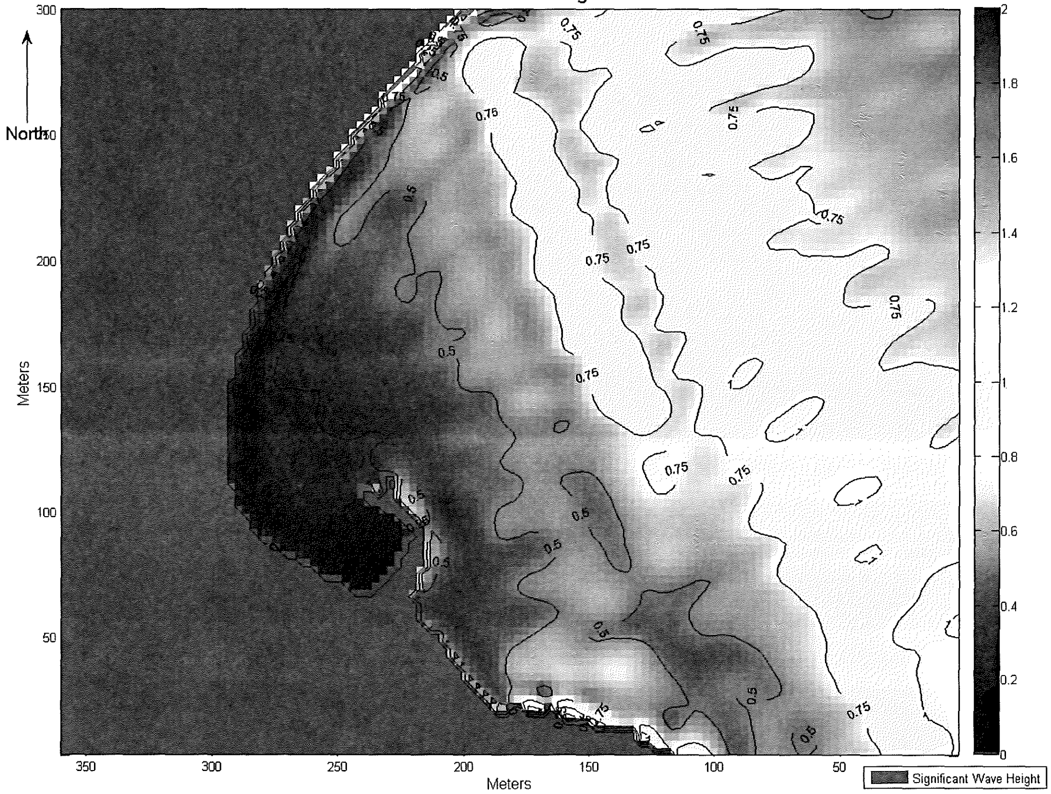


Existing Condition Boating Season
North
Meters
Meters
Significant Wave Height

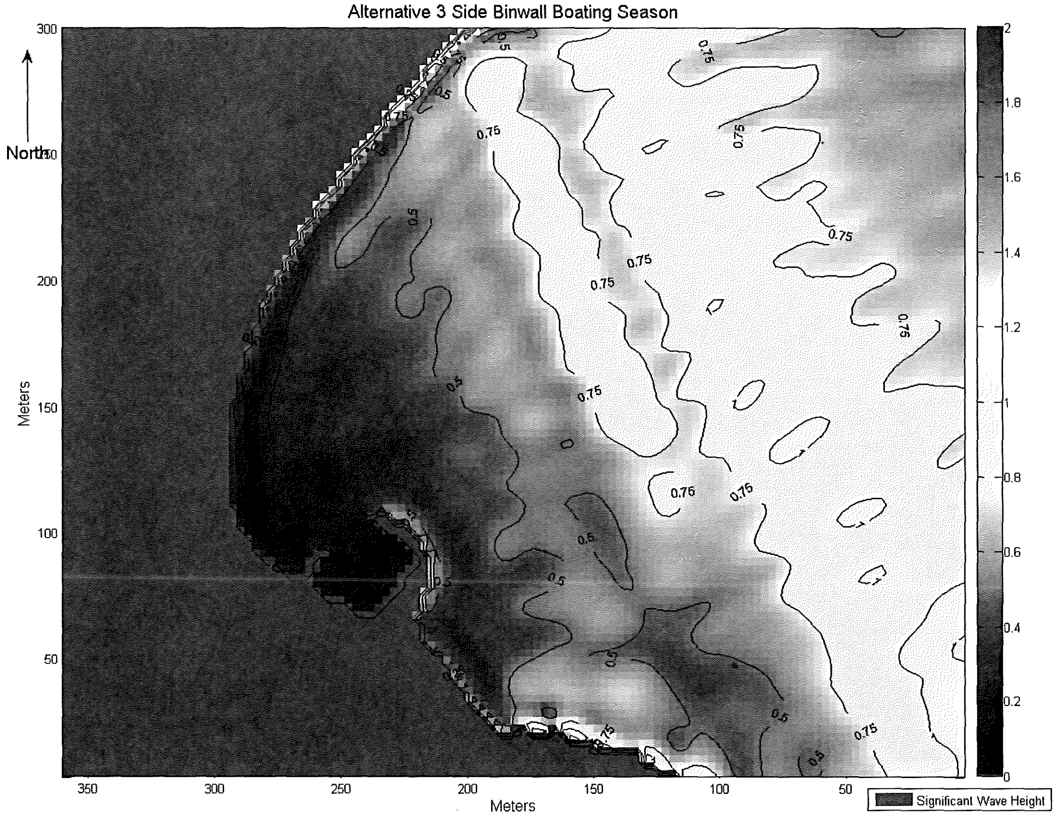


Alternative 3 Boating Season
North
Meters
Meters
Significant Wave Height

Alternative 3 Fishtail Boating Season

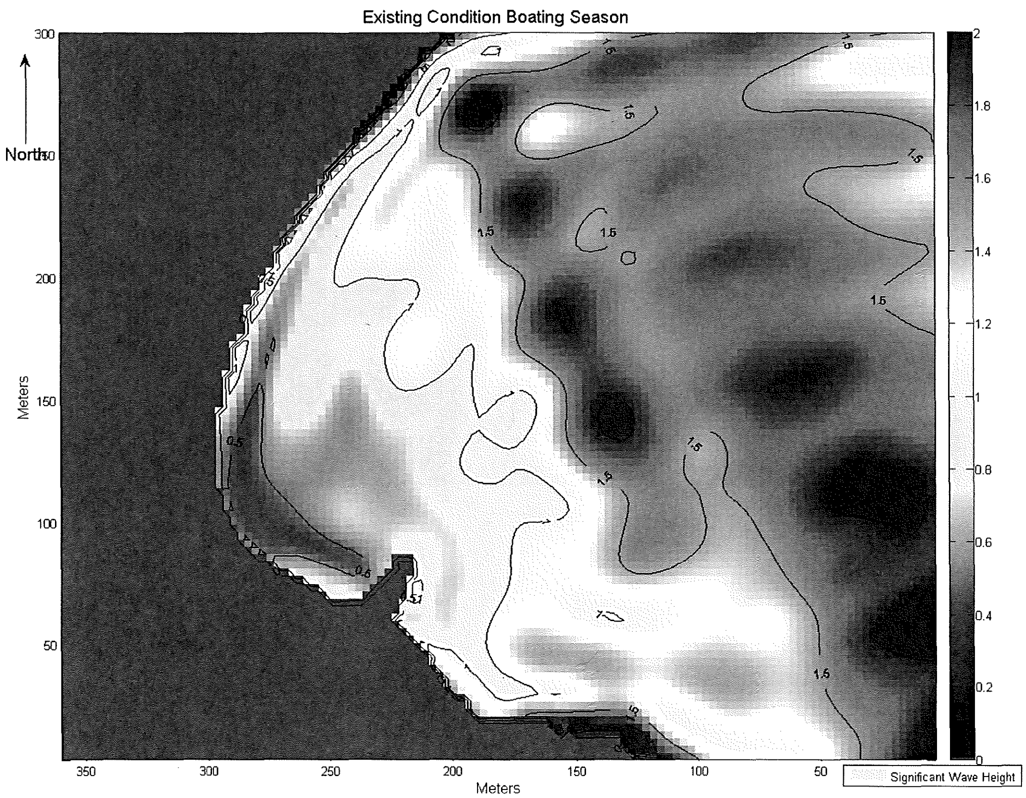


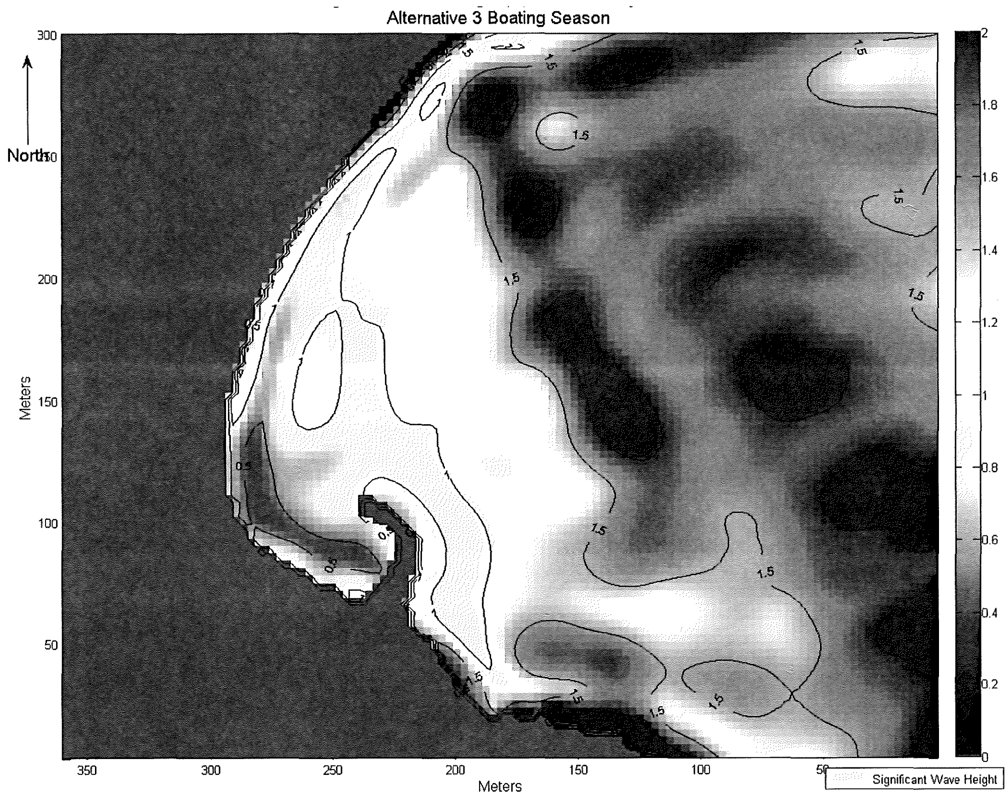
Alternative 3 Side Binwall Boating Season



- Condition 4: Hmo=8.0ft Tp=8.8s

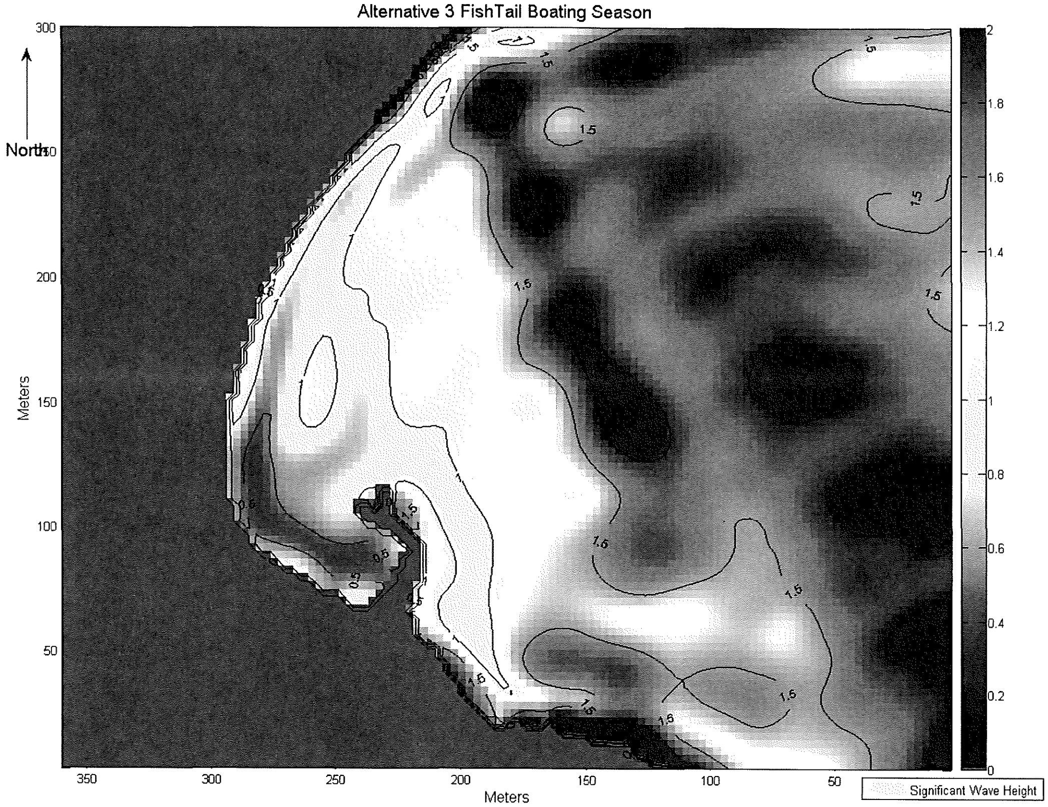
Alternative	Mean Hmo (ft)	Relative Improvement
Existing condition	1.65	0.00%
Extended breakwater	1.27	23.30%
Extended breakwater Binwall	1.49	9.61%
Extended breakwater Fishtail	1.25	24.38%





Alternative 3 Boating Season
North
Meters
Meters
Significant Wave Height
300
250
200
150
100
50
350
300
250
200
150
100
50
2
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
1.5
0.5

Alternative 3 FishTail Boating Season



Alternative 3 Binwall Boating Season