Two Rivers Harbor Wave Mitigation Study

Two Rivers, Wisconsin

May 2017

Prepared by: US Army Corps of Engineers – Detroit District
Section 22, Planning Assistance to States

United States Army Corps of Engineers
Great Lakes Hydraulics and Hydrology Office
477 Michigan Ave. – 6th Floor
Detroit, MI 48226
## Contents

1. Background .................................................................................................................. 6
2. Introduction .................................................................................................................... 7
   2.1 Location ..................................................................................................................... 7
3. Data Description and Discussion .................................................................................. 11
   3.1 Wave Gage Deployment ........................................................................................... 11
   3.1.1 Outer Wave Gage ............................................................................................... 12
   3.1.2 Inner Wave Gage ............................................................................................... 13
   3.1.3 Wave Gage Data ............................................................................................... 13
   3.2 Water Level Data ..................................................................................................... 16
   3.3 Wave Discussion ...................................................................................................... 16
4. Wave Modeling .............................................................................................................. 20
   4.1 Software .................................................................................................................... 20
   4.2 Bathymetry ............................................................................................................... 20
   4.3 Boundary Conditions .............................................................................................. 20
   4.4 Modeled Storm Conditions ................................................................................... 20
   4.4.1 Storm One ......................................................................................................... 21
   4.4.2 Storm Two ......................................................................................................... 23
   4.4.3 Storm Three ...................................................................................................... 24
   4.5 Model vs Wave Gage Comparison ......................................................................... 25
5. Alternatives ................................................................................................................... 27
   5.1 Offshore Breakwater ............................................................................................... 27
   5.2 Dog-leg Extension ................................................................................................. 27
   5.3 Interior Stub Breakwater ...................................................................................... 29
   5.4 Replacement of Steel Sheetpile with Rock ......................................................... 29
6. Model Results ................................................................................................................. 31
   6.1 Maximum Percent Wave Height Reduction at Four Points ................................... 31
   6.2 Wave Height Reductions during Storm Events ..................................................... 31
   6.2.1 October 27th – 29th 2015 Storm One ............................................................... 32
   6.2.2 October 31st – November 1st, 2015 Storm Two ............................................. 38
   6.2.3 November 17th – 20th, 2015 Storm Three .................................................... 45
7. Modeling Results .......................................................................................................... 52
8. Shoaling Discussion ...................................................................................................... 56
9. Qualitative Comparison of Alternatives ....................................................................... 58
List of Figures

Figure 1: Location of Two Rivers Harbor ......................................................... 8
Figure 2: Harbor Alignment and Wave Rose ...................................................... 9
Figure 3: Site Map ......................................................................................... 10
Figure 4: Wave Gage Location ................................................................. 11
Figure 5: Outer Wave Gage with RDI Workhorse ADCP ............................. 12
Figure 6: Inner Wave Gage with RDI Sentinel ADCP ................................. 13
Figure 7: Time Series of Significant Wave Height ....................................... 14
Figure 8: Time series of hourly peak wave heights .................................... 15
Figure 9: $H_{max}$ for Fall Waves at Both Gages ........................................... 15
Figure 10: Lake Michigan Water Levels at the Kewaunee Gauge NOAA 9087068 ................................................................. 16
Figure 11: Reflective and Wave Absorbing Surfaces in the Outer Harbor .... 17
Figure 12: Wave Spectra for Peak of Storm Three ................................. 18
Figure 13: Wave Rose for Fall Deployment Showing the Reflection Off the Back Harbor Wall ............................................. 19
Figure 14. Winds Associated with Storm One .............................................. 21
Figure 15. Wind direction for Storm One .................................................... 22
Figure 16: Wave Rose Storm One ............................................................... 22
Figure 17. Winds Associated with Storm Two .............................................. 23
Figure 18: Wave Rose Storm Two ............................................................... 23
Figure 19. Wind Rose Associated with Storm Three ................................... 24
Figure 20. Wind Direction for Storm Three ............................................... 24
Figure 21: Wave Rose Storm Three ........................................................... 25
Figure 22. Modeled Wave Heights (Blue) versus Wave Gage Measurements (Orange) from October 5, 2015 to December 2, 2015 ......................................................... 26
Figure 23. Location of Proposed Offshore Breakwater ................................ 27
Figure 24. Proposed Dog-leg Extension - North Jetty ................................. 28
Figure 25. Proposed Dog-leg Extension - South Jetty ................................ 28
Figure 26. Location of Proposed Interior Stub Breakwater ......................... 29
Figure 27. Location of Rock Along Back of the Harbor ............................... 30
Figure 28. Location of Rock Along South Side of the Harbor ..................... 30
Figure 29. Location Map of the Four Observation Points ............................ 31
Figure 30. Wave Height Change at the East Twin Observation Point for Storm One ......................................................... 32
Figure 31. Wave Height Change at the West Twin Observation Point for Storm One ................................................................. 33
Figure 32. Wave Height Change at the Back Harbor Observation Point for Storm One ................................................................. 33
Figure 33. Wave Height Change at the Gage Observation Point for Storm One ................................................................. 34
Figure 34. Percent Wave Height Change for Offshore Breakwater – Above Low Water Datum, Storm One ................................. 35
Figure 35. Percent Wave Height Change for Offshore Breakwater – Submerged, Storm One ................................................................. 35
Figure 36. Percent Wave Height Change for Dog-leg Extension - South Jetty, Storm One ................................................................. 36
Figure 37. Percent Wave Height Change for Rock Along Back of the Harbor, Storm One ................................................................. 36
Figure 38. Percent Wave Height Change of Interior Stub Breakwater, Storm One ................................................................. 37
Figure 39. Detail of Percent Change for Interior Stub Breakwater, Storm One ................................................................. 37
Figure 40. Wave Height Change at the East Twin Observation Point for Storm Two ................................................................. 39
Figure 41. Wave Height Change at the West Twin Observation Point for Storm Two ................................................................. 39
Figure 42. Wave Height Change at the Back Harbor Observation Point for Storm Two ................................................................. 40
Figure 43. Wave Height Change at the Gage Observation Point for Storm Two ................................................................. 40
Figure 44. Percent Wave Height Change for Offshore Breakwater - Above Low Water Datum, Storm Two ................................................................. 41
Figure 45. Percent Wave Height Change for Offshore Breakwater - Submerged, Storm Two ................................................................. 42
Figure 46. Percent Wave Height Change for Dog-leg Extension - South Jetty, Storm Two ................................................................. 42
Figure 47. Percent Wave Height Change for Rock Along Back of the Harbor, Storm Two ................................................................. 43
Figure 48. Percent Wave Height Change for Interior Stub Breakwater, Storm Two ................................................................. 43
Figure 49. Detail of Percent Change for Interior Stub Breakwater, Storm Two ................................................................. 44
Figure 50. Wave Height Change at the East Twin Observation Point for Storm Three ................................................................. 45
List of Tables

Table 1: Maximum Percent Change Associated with Storm One for Each Alternative at Each of the Four Monitoring Sites. 38
Table 2: Maximum Percent Change Associated with Storm Two for Each Alternative at Each of the Four Monitoring Sites. 44
Table 3: Maximum Percent Change Associated with Storm Three for Each Alternative at Each of the Four Monitoring Sites. 50
Table 4: Alternative Matrix 58
1 Background

The Two Rivers Harbor federal navigation project was originally authorized by the Rivers and Harbors Act of March 3, 1871 and additionally by the River and Harbors Acts of March 2, 1907, August 30, 1935 and July 3, 1958. These authorizations provided for two piers and a north revetment, an entrance channel and an 18 feet deep inner basin. They also provided for a small stilling basin beyond the shoreline on the east side of the channel and a channel 10 feet deep in the East Twin River from the inner basin upstream to the 22nd Channel Street bridge.

Waves created on Lake Michigan have been observed to propagate into the harbor entrance and move upstream from the outer harbor into the inner harbor. It is reported that these waves have impacted both the Coast Guard (from launching vessels at their station) and commercial fishing vessels (operating on East Twin River), in addition to disruption in the operation of Seagull Marina. Historically, considerable effort has been expended to reduce the wave energy reaching the inner harbor. While these efforts have been effective at other harbors on Lake Michigan (Pentwater, Charlevoix, Saugatuck, White Lake and Portage Lake), adverse wave conditions continue to be reported at Two Rivers Harbor.

This study was conducted under the US Army Corps of Engineers’ (USACE or Corps) Planning Assistance to States (PAS) Program, or Section 22 of the Water Resources Development Act of 1974. The objective of this study is to measure the wave climate over a field season in order to better understand the wave conditions that are causing the problems at Two Rivers and then development of a numerical model to simulate wave transformations in the harbor and evaluate a number of wave mitigation alternatives.
2 Introduction

Although considerable efforts have been made to reduce the wave energy entering Two Rivers Harbor the harbor is still experiencing large waves. It has been reported that too large of waves have prevented the Coast Guard from launching vessels from their station at Two Rivers and impacted a commercial fishing operation on the East Twin River. Furthermore, the City of Two Rivers is considering the addition of boat slips along East Street and is concerned that waves may impact this site. Since no quantification of the wave climate was made before or after previous improvement efforts, it is difficult to assess their effectiveness.

Significant support for this PAS study was provided by the Corps’ Great Lakes Hydraulics and Hydrology (H&H) Office. H&H examined possible solutions aimed at reducing the wave energy within the harbor and entrance channel. Since some of the traditional wave absorbing approaches have not adequately reduced the wave climate thus far, it is important to first understand the wave conditions that are causing the problem before prescribing a solution. As such, this study was divided into two phases, a wave and current data collection phase and a modeling phase. This PAS study was funded under a cost share agreement, dated 02 March 2015, between the Corps and the City of Two Rivers, WI.

2.1 Location

The study site is located on the western shore of Lake Michigan at Two Rivers, WI, which is approximately 80 miles north of Milwaukee, Figure 1. The harbor mouth faces to the southeast, which exposes it to long fetches from the southeast. Figure 2 shows a wave rose from hindcast data near Two Rivers.
Figure 1: Location of Two Rivers Harbor
This wave hindcast was simulated over a 36 year period (1979-2014). The wave statistics were calculated from data generated every three hours over this time period. Over 300,000 wave calculations are summarized in the wave rose. It is important to note that these are offshore, deep-water waves from a hindcast, not the measured waves that will be discussed later. The depth of Lake Michigan at this location is 68 feet. At this depth, the waves have not undergone any appreciable transformation due to bottom effects. The waves in figure 2 are bimodally distributed, with the majority of waves coming out of the northeasterly direction and from the south-southeasterly direction. Waves with a northerly component will not enter the harbor with any appreciable energy and are not expected to cause navigation issues within the harbor. The majority of the remaining waves are approximately parallel to the harbor alignment. These are likely the waves that are causing navigation problems within the harbor and entrance channel.

The three main users of the harbor are the U.S. Coast Guard, Susy Q Fish Market and Seagull marina, Figure 3. Other users include Rogers Street Fishing Village, Twin City Marine. Vietnam Veterans Memorial Park is also considered an additional point of interest.
Figure 3: Site Map
3 Data Description and Discussion

3.1 Wave Gage Deployment

Data was collected at Two Rivers between May and December of 2015. Of which, consisted of four data retrieval trips during the field season of 2015. The gages were initially deployed during the week of 4 May. The gages were retrieved for maintenance and to download data in September and October and redeployed until their final retrieval during the week of 1 December 2015. The location of the two gages is shown on Figure 4. The location of the outer gage was selected to represent waves near the harbor mouth that have not yet been affected by transformation processes due to the harbor structure, such as reflection and diffraction. These offshore waves can also be used as a boundary condition to drive the numerical wave model. The location of the inner wave gage was selected since it represented the approximate location of the problematic waves described by users of the harbor. In siting this gage, consideration had to be given to avoid the pathway of anglers and to otherwise not interfere with navigation at this popular harbor. As such, the center of the channel was avoided, even though it would have provided a better measure of the problematic waves.

Figure 4: Wave Gage Location
3.1.1 Outer Wave Gage
The outer wave gage was located at N 44° 08.447', W 87° 33.570' and was in approximately 19 feet of water. This gage consisted of an uplooking acoustic Doppler current profiler (ADCP), Figure 5 and was initially deployed on 5 May 2015. The gage was lowered to the lakebed from the Research/Vessel (R/V) Lee. Its proper placement and orientation was confirmed with underwater video examination. The gage had a drag line attached to it which was attached to a cluster weight and a buoy with an acoustic release. Details of the field configuration can be found in Appendix 1. An attempt to retrieve this gage was made in September; however, the acoustic release malfunctioned and would not release the buoy. Multiple passes with a grappling hook were unsuccessful at snagging the drag line. This gage was later retrieved on 4 October by the Coast Guard after the buoy’s backup release was activated. This gage was redeployed the following week after maintenance and a data download. The final retrieval of the gage was on 1 December 2015 using a drag line and grappling hook with the assistance from the Sault Ste. Marie Area Office.

This gage consisted of an RDI Workhorse collecting data at 1200 kHz and with 4 acoustic beams angled at 20 degrees from zenith. This gage turned itself on every hour during deployment and collected wave and water level data for 20 minutes before turning itself off to conserve battery life. Data was collected at 2 Hz, resulting in 2400 measurements of waves and water levels each hour. During the 7 month deployment, this gage collected over 12 million data points that have been processed and statistically analyzed.

Figure 5: Outer Wave Gage with RDI Workhorse ADCP
3.1.2 Inner Wave Gage
The inner wave gage is located at N 44° 08.778', W 87° 33.844'. This gage, Figure 6, was an uplooking ADCP manufactured by RDI and mounted to a sea spider. This inner wave gage is a Sentinel V and operating at 1000 kHz and used 4 acoustic beams angled 25 degrees from zenith. A fifth vertical beam measured the distance to the surface of the water and measured changes in water surface elevation. Similar to the outer gage, this device turned itself on for the same 20 minutes each hour and measured waves and water levels at a sampling frequency of 2 Hz.

This gage was initially deployed on 6 May 2015 from the R/V Lee. It was retrieved in September with the intent of redeploying the next day; however, the gage had malfunctioned and had to be sent back to the manufacturer for service. Upon repair, the gage was redeployed on 6 October and ultimately retrieved on 1 December 2015. The malfunction caused the loss of data from 9 August to 6 October.

Figure 6: Inner Wave Gage with RDI Sentinel ADCP

3.1.3 Wave Gage Data
During each hour, 2400 wave measurements are taken. These measurements have been processed and statistically analyzed to calculate a variety of wave parameters. Figure 7 shows a time series of significant wave height at both the inner and outer gage.
Significant wave height (Hs) is an average of the largest 1/3 of the waves and is the most common way to characterize a time series of waves. Note that there is a gap in the data from the inner harbor gage from 9 Aug to 6 Oct while it was being repaired. During this time period, however, the outer gage did not measure any noteworthy events, so the missing inner harbor data is not a significant loss. Two observations stand out when looking at Figure 7. First, the waves inside the harbor are significantly smaller than those at the outer gage, suggesting that a significant amount of wave energy is lost in the entrance channel. This is likely due to reflection, diffraction and breaking along the channel. The sand pocket beach shown on Figure 3 is expected to absorb a significant amount of wave energy that diffracts into it. The second observation is that the waves at both gages in the spring and summer are significantly less energetic than those in the fall.

![Figure 7: Time Series of Significant Wave Height](image)

In calculating the significant wave height, the larger waves can be overlooked in the averaging process. As such, a plot of maximum wave height is shown in Figure 8. These wave roses are for the data at both the inner and outer gage and show not only direction and maximum wave height, but also the frequency with which these waves occur. From this graph, waves as high as 4 ½ ft in the inner harbor were measured during this 7 month deployment. It is also interesting to note how well the direction of the wave energy lines up with the orientation of this harbor. This fact is likely related to the wave problems experienced in the inner harbor.
Figure 8: Time series of hourly peak wave heights

Figure 9: $H_{\text{max}}$ for Fall Waves at Both Gages
Further investigation of the inner harbor gage show relatively large waves with north westerly direction.

### 3.2 Water Level Data

Water levels are recorded by National Oceanic and Atmospheric Administration (NOAA) at several gages around Lake Michigan. The nearest gage to Two Rivers is at Kewaunee, WI. A plot of this data for the deployment period is shown in Figure 10. The expected seasonal rise during the summer is seen in this data. Superimposed on top of this seasonal trend are short duration water level fluctuations. The magnitude of these fluctuations increase in the fall and winter. It is suspected that these are storm surges, which would be expected to increase during the more energetic fall and winter. A comparison of water levels at the NOAA gage with that collected by the wave gages will be developed later, in hopes of observing the presence or absence of infragravity waves.

![Lake Michigan Water Levels at the Kewaunee Gauge NOAA 9087068](image)

Figure 10: Lake Michigan Water Levels at the Kewaunee Gauge NOAA 9087068

### 3.3 Wave Discussion

The addition of large armor stone to an otherwise reflective breakwater is expected to absorb waves and reduce the likelihood of large waves propagating upstream and impairing navigation (Figure 11). Since this has not completely mitigated the wave action concerns at Two Rivers, other factors must be looked at more closely to understand and solve this problem.
Two additional processes need to be discussed as one or both of them may be influencing the wave conditions at Two Rivers. These are the presence of infragravity waves and the development of resonance in the inner harbor.

Infragravity waves are long-period waves with a period of 30 – 300 seconds. Most of the regular, wind-driven waves on the Great Lakes are under 12 seconds in period. Infragravity waves, like tides, are not affected by the presence of marine engineering works, such as wave absorbers, stone linings or even breakwaters, and pass by with little change. The presence of infragravity waves at Two Rivers could explain the energetic conditions seen in the inner harbor and the inability to preclude them with conventional approaches. Because of their long wavelength, infragravity waves never break. They do, however, slow down in shallow water, resulting in water piling up or an increase in wave amplitude.

Infragravity waves can be caused by a number of processes acting in the Great Lakes. The passage of a fast moving squall line can create a long period wave if the squall line is moving at about the same speed as the waves propagating out from it. These are sometimes called meteotsunamis and have been observed in the Great Lakes.

Infragravity waves can also be created as waves break over a reef and recombine into a longer period wave. There does not appear to be the bathymetry or geomorphology near Two Rivers to create a long period wave using this process.
Lastly, as a storm passes over the lake and waves are created, the largest ones will move away from the storm at a faster rate than the smaller ones and will result in the waves being sorted into groups of similar size. The mean lake level can oscillate with a wavelength that is equal to the length of the group, thus creating a long period, or infragravity, wave.

Due to their size, the Great Lakes typically have short period waves. Ten second periods are considered long waves. The wave gages were configured to measure for 20 minute intervals. Infragravity waves can have periods of approximately 30 to 300 seconds. If infragravity waves were present during the measurement phase of this study they would have been sampled at least four times during the 20 minute sampling interval of the wave gage. If infragravity waves were measured, a large peak would occur at a frequency associated with waves having 30 to 300 second period, 0.03 Hz to 0.003 Hz in Figure 12. The largest spike in energy shown in Figure 12 is associated with a frequency of approximately 0.14 Hz or 6.7 seconds, so the wave contributing the most energy into the harbor have a period of 6.7 seconds and are not infragravity waves. Similar results for the other storms were also observed.

![Figure 12: Wave Spectra for Peak of Storm Three](image)

The other, and more likely cause of large waves in the inner harbor is due to resonance. The walls in the inner harbor are mostly composed of highly reflective steel sheet pile. Waves striking this material will reflect with almost no energy loss. If the incoming wave is in phase with a reflected wave, their heights are additive. As this situation persists, more wave energy is pumped into these resonating waves. Resonance can cause a small incident wave train to grow significantly and could be responsible for the problematic waves at Two Rivers. Resonance problems can be solved by replacing the reflective surfaces with wave absorbing stone.

Measured waves from inside Two Rivers Harbor over the fall deployment show approximately 10 percent of the waves inside the harbor originate from the northwest. These waves are reflections off the back harbor wall and have a magnitude nearly as
large as the incident wave, Figure 13. This is consistent with wave reflecting off a vertical steel or concrete wall.

Figure 13: Wave Rose for Fall Deployment Showing the Reflection Off the Back Harbor Wall
4 Wave Modeling

4.1 Software
The Coastal Modeling System is an integrated suite of numerical models for simulating flows, waves, sediment transport and morphology in coastal environments. It was used to analyze existing and proposed geometry for a given set of storms. CMS-Wave is a spectral wave transformation model that solves the wave-action balance equation using a forward marching finite difference method. It includes physical processes such as wave shoaling, refraction, diffraction, reflection, wave-current interaction, wave breaking, wind wave generation, white capping of waves, and the influence of coastal structures. CMS is the appropriate software for screening level analyses of many alternatives due to the physics represented in the model, the speed of computation and the level of detail required for proposed geometries.

4.2 Bathymetry
The model was developed from three sources of bathymetric data. These data were combined to form one continuous bathymetric surface for use in the model. The main source of data within the harbor into the East Twin River is from a single beam survey collected by the USACE Detroit District in 2014. LIDAR covers the land, nearshore and some of the West Twin River. These data were collected by the USACE through the Joint Airborne Lidar Center of Expertise coastal mapping mission. They are available from the NOAA Office of Coastal Management through their Digital Coast Data Access Viewer. Deeper parts of the model domain were obtained from NOAA National Center for Environmental Information.

The three sets of data were viewed together and in areas just offshore of the end of the jetties the overlapping data was edited to give priority to the single beam bathymetry collected by the USACE by removing some points from the remotely sensed data sets. Additionally, there were some data gaps found between the three data sets. These gaps were analyzed and smoothed to best represent actual conditions.

4.3 Boundary Conditions
The time series of wave spectra used to force the model were obtained from the Great Lakes Coastal Forecast System. These data were reviewed and corrected to account for missing data points and to add the correction factor for the water level above/below low water datum (the zero point for the bathymetry). Hourly water levels from the NOAA gage at Kewaunee, WI (9087068), Figure 10, were used to adjust the water surface elevation above low water datum of 577.4 ft., IGLD85 on Lake Michigan.

4.4 Modeled Storm Conditions
Computational effort is an important consideration for any modeling exercise. Given the number of alternatives under investigation, the three largest storms based on data from the outer harbor wave gage were modeled for this analysis. All three of these events came from the fall deployment.
4.4.1 Storm One
Storm one occurred from 27-29 October 2015 with the peak occurring at 10:00am 28 October 2015. Sustained wind speeds up to 25 miles per hour were reported at the Manitowac County Airport. Winds for this event had a bimodal distribution, Figure 14. Winds started from the northeast and had a rapid shift to the south, southwest, Figure 15. Wave direction and magnitude associated with Storm One are shown in Figure 16. The dominant wave direction associated with Storm One is from the southeast with seven to eight foot waves. On the Great Lakes waves generally follow the predominant wind direction. However, it is not unusual for land based wind stations such as the Manitowac County Airport to vary from open water based measurements. Open water wind estimates are not available for the modeling period so land based wind stations were used in the model.

Figure 14. Winds Associated with Storm One
Figure 15. Wind direction for Storm One

Figure 16: Wave Rose Storm One
4.4.2 Storm Two

Storm Two occurred from 31 October 2015 through 1 November 2015, with the peak hour occurring at 8:00pm on 31 October. This storm had winds from a single, southeast direction with sustained winds up to 20 miles per hour (Figure 17) and wave heights up to seven feet (Figure 18). Wind and wave directions are more closely aligned for Storm Two when compared to Storm One.

**Wind Rose (MPH)**

31Oct2015 to 1Nov2015

![Wind Rose Diagram](71x363 to 351x638)

*Figure 17. Winds Associated with Storm Two*

**Wave Rose (feet)**

Great Lakes Coastal Forecast System

![Wave Rose Diagram](72x88 to 341x339)

*Figure 18: Wave Rose Storm Two*
4.4.3 Storm Three
Storm Three occurred 17-20 November 2015 with the peak occurring at 6:00 pm on 19 November. This storm also has a bimodal distribution of wind directions starting from the southeast and shifting to the south, southwest, Figure 19 and Figure 20. This storm had sustained winds of up to 30 miles per hour (Figure 19) and wave heights up to seven feet largely from the east (Figure 21).

Figure 19. Wind Rose Associated with Storm Three

Figure 20. Wind Direction for Storm Three
4.5 Model vs Wave Gage Comparison

Since this study included a wave data collection phase completed in December 2015, changes were made to the standard model parameters to more accurately reflect the conditions at the Two Rivers Harbor. These included changes to the backward and forward reflection coefficients to produce modeled wave heights that more accurately match the gage data. Figure 22 shows the model versus gage data at the inner harbor for the time period of October 5, 2015 to December 2, 2015 when the three largest storms occurred. The model follows the trends well but consistently under predicts the peaks on the order of 25-percent. Larger percent differences occur but generally apply to smaller waves, the model was tuned to best capture the large waves while maintaining reflection coefficients within accepted values in literature.
Figure 22. Modeled Wave Heights (Blue) versus Wave Gage Measurements (Orange) from October 5, 2015 to December 2, 2015.
5 Alternatives
Seven alternatives were modeled to determine which would result in the largest decrease in wave height throughout the harbor. The alternatives are described below.

5.1 Offshore Breakwater
Two versions of the offshore breakwater were modeled, one with the breakwater extending above the water level for each modeled storm, actual height would be determined during design level modeling. Another offshore breakwater was investigated and placed 3 feet below the low water datum elevation of 577.5 feet, IGLD85. Both breakwaters were modeled at approximately 250 feet southeast from the ends of the jetties (Figure 23). Additional model runs could be performed to optimize the distance between the offshore breakwall and the existing jetties. Optimizing the angle of the offshore breakwall and the distance from the jetties may or may not improve the efficacy of the breakwall, however, the general pattern of 50% or better reduction in wave energy is expected with an offshore breakwall.

Figure 23. Location of Proposed Offshore Breakwater

5.2 Dog-leg Extension
Two versions of the dog-leg extension to the existing jetty were modeled, one with the dog-leg extension on the north jetty and one with the extension on the south jetty (Figure 24 and Figure 25, respectively).
Figure 24. Proposed Dog-leg Extension - North Jetty

Figure 25. Proposed Dog-leg Extension - South Jetty
5.3 Interior Stub Breakwater
This alternative consists of building a small breakwater inside the harbor on the southeast wall (Figure 26).

Figure 26. Location of Proposed Interior Stub Breakwater

5.4 Replacement of Steel Sheetpile with Rock
Two alternatives modeled included replacing steel sheetpile with rock within the harbor and channel inlet. One of those alternatives replaced approximately 400 feet of steel sheetpile along the back of the harbor where the channel inlet connected (Figure 26), while another alternative replaced the approximately 900 foot steel sheetpile along the south jetty near the harbor – channel inlet connection (Figure 28).
Figure 27. Location of Rock Along Back of the Harbor

Figure 28. Location of Rock Along South Side of the Harbor
6 Model Results

The seven alternatives were modeled with storm data collected from the gages deployed during the study (May to December 2015). The model results were reviewed at a four points and transects throughout the harbor with special emphasis on the area to the northeast of the confluence of the East Twin River with the West Twin River due to development interests in this area.

6.1 Maximum Percent Wave Height Reduction at Four Points

Four points were chosen to assess each alternative’s effectiveness in reducing wave height during each storm simulation. One point was examined on the East Twin River, one on the West Twin River and one at the confluence of the two at the back of the harbor. A fourth point, located near the Coast Guard Station and approximately at the location of the USACE wave gage, was also examined. Figure 29 shows the location of the points. Each location is examined for maximum percent reduction for each storm and for each alternative.

![Figure 29. Location Map of the Four Observation Points](image)

6.2 Wave Height Reductions during Storm Events

Three storm events were reviewed to further examine each alternative’s effectiveness in reducing wave heights. The same four points were examined for each alternative and each storm.
6.2.1 October 27th – 29th 2015 Storm One
The first storm event reviewed was from October 27th to October 29th. Winds associated with this storm have a bimodal distribution with the largest winds coming from the east northeast and the south, southwest as shown in Figure 14. Figure 30 through Figure 33 show the percent reduction wave heights compared to the existing conditions of the harbor for seven alternatives at the four observation points.

The figures show that the largest reductions in wave heights on the East Twin, at the back of the harbor and at the wave gage location was achieved by the detached breakwater or a dog-leg extension to the south jetty. For the West Twin observation point these alternatives were also effective but the placement of rock along the back of the harbor also showed large percent reductions in wave heights. These graphs show how reductions in wave heights change over the course of the storm. These results however, are a function of where the results are extracted. Figure 34 through Figure 38 show wave height change through the entire harbor in response to each alternative but only show the change for the peak of the storm.

Figure 30. Wave Height Change at the East Twin Observation Point for Storm One
Figure 31. Wave Height Change at the West Twin Observation Point for Storm One

Figure 32. Wave Height Change at the Back Harbor Observation Point for Storm One
Percent reduction in wave heights throughout the harbor were also compared to the existing geometry during peak storm conditions and are shown for select alternatives in Figure 34 through Figure 38. Wave height reduction graphics were not prepared for the Dog-Leg Extension – North Jetty due to the fact the Dog-leg Extension – South Jetty almost always out performs the Dog-leg Extension – North Jetty. Similarly, Rock Along South Side of the Harbor generally does not perform as well as other alternatives and were left out of the peak wave reduction figure series. The peak of Storm One occurred at 10:00 am on 28 October. Figure 34 and Figure 35 contrast the differences between the Offshore Breakwater – Above Low Water Datum and the Offshore Breakwater - - Submerged.. The Offshore Breakwater – Above Low Water Datum, Figure 34, reduces wave heights throughout the harbor approximately 50 to 60 percent while wave heights within the harbor associated with the Offshore Breakwater - Submerged, Figure 35, are reduced 5 to 20 percent. With the addition of a Dog-leg Extension – South Jetty an approximate 40 to 50 percent reduction in wave heights at the junction of the West and East Twin Rivers is expected, Figure 36. Figure 37 shows the wave height reduction during the peak of the storm achieved with Rock Along Back of the Harbor. Twenty to 40 percent wave height reductions are expected at the river junction for this alternative. The harbor entrance is relatively unchanged. An Interior Stub Breakwater placed at the harbor junction with the East Twin River offers localized reductions in wave height by 20 to 40 percent on the inner harbor side of the Interior Stub Breakwater but the outer harbor side will see increased wave heights for approaching waves and additionally waves reflected off the break wall.
Figure 34. Percent Wave Height Change for Offshore Breakwater – Above Low Water Datum, Storm One

Figure 35. Percent Wave Height Change for Offshore Breakwater – Submerged, Storm One
Figure 36. Percent Wave Height Change for Dog-leg Extension - South Jetty, Storm One

Figure 37. Percent Wave Height Change for Rock Along Back of the Harbor, Storm One
Figure 31 shows results at the West Twin River for the Interior Stub Breakwater alternative. This time series shows little change compared to existing geometry; however, as seen in Figure 39, the point where model results are extracted lie near the nodal point between no change and approximately 25 percent reduction.
Table 1 shows the maximum percent change for each alternative at each of the observation points. These are the maximum changes, Figure 30 through Figure 33 capture the entire range of change occurring over Storm One. Positive values indicate increased wave heights when compared to the existing geometry. Based on the chosen orientation of the Interior Stub Breakwater, increased wave heights occur near the Coast Guard Station.

Table 1: Maximum Percent Change Associated with Storm One for Each Alternative at Each of the Four Monitoring Sites.

<table>
<thead>
<tr>
<th>Alternative</th>
<th>Percent Wave Height Reduction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>East Twin</td>
</tr>
<tr>
<td>Offshore Breakwater</td>
<td></td>
</tr>
<tr>
<td>Above Low Water Datum</td>
<td>-66%</td>
</tr>
<tr>
<td>Submerged</td>
<td>-22%</td>
</tr>
<tr>
<td>Dog-leg J Extension</td>
<td></td>
</tr>
<tr>
<td>North Jetty</td>
<td>-59%</td>
</tr>
<tr>
<td>South Jetty</td>
<td>-69%</td>
</tr>
<tr>
<td>Interior Stub Breakwater</td>
<td>-6%</td>
</tr>
<tr>
<td>Rock Along Back of Harbor</td>
<td>-28%</td>
</tr>
<tr>
<td>Rock Along South Side of the Harbor</td>
<td>-21%</td>
</tr>
</tbody>
</table>

6.2.2 October 31st – November 1st, 2015 Storm Two

Figure 40 through Figure 43 show the calculated wave height reductions compared to the existing conditions of the harbor for the seven alternatives at the four observation points. They show the largest reductions in wave heights on the East Twin, at the back of the harbor and at the wave gage location was achieved by the Offshore Breakwater or the Dog-leg Extension - South Jetty. For the West Twin observation point these alternatives were also effective but the Rock Along Back of the Harbor alternative also showed large reductions in wave heights as well.
Figure 40. Wave Height Change at the East Twin Observation Point for Storm Two

Figure 41. Wave Height Change at the West Twin Observation Point for Storm Two
Figure 42. Wave Height Change at the Back Harbor Observation Point for Storm Two

Figure 43. Wave Height Change at the Gage Observation Point for Storm Two
Wave height reductions were computed throughout the harbor for the peak of Storm Two for several alternatives. Similar to Storm One, the Offshore Breakwater – Above Low Water Datum reduced wave heights approximately 50 to 60 percent throughout the interior of the harbor; the Offshore Breakwater - Submerged is significantly less effective at reducing wave heights accomplishing 5 to 20 percent wave height reductions for the same storm, Figure 44 and Figure 45. The Dog-leg Extension - South Jetty generates similar wave height reductions as the Offshore Breakwater - Above Low Water Datum, Figure 46. The Rock Along Back of the Harbor reduces wave heights at the rear of the harbor without increasing wave heights compared to existing geometry, Figure 47. This alternative also provides significant benefit to the Seagull Marina. The Interior Stub Breakwater again provided localized wave reductions but also created pockets of increased wave heights when compared to existing geometry,

![Percent Wave Height Change](image)

Figure 44. Percent Wave Height Change for Offshore Breakwater - Above Low Water Datum, Storm Two
Figure 45. Percent Wave Height Change for Offshore Breakwater - Submerged, Storm Two

Figure 46. Percent Wave Height Change for Dog-leg Extension - South Jetty, Storm Two
Figure 47. Percent Wave Height Change for Rock Along Back of the Harbor, Storm Two

Figure 48. Percent Wave Height Change for Interior Stub Breakwater, Storm Two
Table 2 shows the maximum percent change for each alternative at each of the observation points. These are the maximum changes. Figure 40 through Figure 43 contain the entire range of change over Storm Two. Positive values indicate increased wave heights when compared to the existing geometry. Based on the chosen orientation of the Interior Stub Breakwater increased wave heights occur near the Coast Guard Station.

Table 2: Maximum Percent Change Associated with Storm Two for Each Alternative at Each of the Four Monitoring Sites.

<table>
<thead>
<tr>
<th>Alternative</th>
<th>Percent Wave Height Reduction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>East Twin</td>
</tr>
<tr>
<td>Offshore Breakwater</td>
<td></td>
</tr>
<tr>
<td>Above Low Water Datum</td>
<td>-63%</td>
</tr>
<tr>
<td>Submerged</td>
<td>-19%</td>
</tr>
<tr>
<td>Dog-leg Extension</td>
<td></td>
</tr>
<tr>
<td>North Jetty</td>
<td>-42%</td>
</tr>
<tr>
<td>South Jetty</td>
<td>-67%</td>
</tr>
<tr>
<td>Interior Stub Breakwater</td>
<td>-4%</td>
</tr>
<tr>
<td>Rock Along Back of the Harbor</td>
<td>-28%</td>
</tr>
<tr>
<td>Rock Along South Side of the Harbor</td>
<td>-10%</td>
</tr>
</tbody>
</table>
6.2.3 November 17th – 20th, 2015 Storm Three

The third storm event reviewed was from November 17th to the 20th. Figure 50 through Figure 53 show the percent reduction of wave heights compared to the existing conditions of the harbor at seven alternatives at the four observation points.

The figures show that the largest reductions in wave heights on the East Twin, at the back of the harbor and at the wave gage location was achieved by the Offshore Breakwater – Above Low Water Datum or the Dog-leg Extension - South Jetty. For the West Twin observation point these alternatives were also effective but the placement of rock along the back of the harbor also showed large reductions in wave heights.

![Figure 50. Wave Height Change at the East Twin Observation Point for Storm Three](image-url)
Figure 51. Wave Height Change at the West Twin Observation Point for Storm Three

Figure 52. Wave Height Change at the Back Harbor Observation Point for Storm Three
Figure 53. Wave Height Change at the Gage Observation Point for Storm Three

Figure 54. Percent Wave Height Change for Offshore Breakwater – Above Low Water Datum, Storm Three
Figure 55. Percent Wave Height Change for Offshore Breakwater - Submerged, Storm Three

Figure 56. Percent Wave Height Change for Dog-leg Extension - South Jetty, Storm Three
Figure 57. Percent Wave Height Change for Rock Along Back of the Harbor, Storm Three

Figure 58. Percent Wave Height Change for Interior Stub Breakwater, Storm Three
Table 3 shows the maximum percent change for each alternative at each of the observation points. These are maximum changes. Figure 50 though Figure 53 show the entire range of change. Positive values indicate increased wave heights when compared to the existing geometry. Based on the chosen orientation of the Interior Stub Breakwater increased wave heights occur near the Coast Guard Station.

Table 3: Maximum Percent Change Associated with Storm Three for Each Alternative at Each of the Four Monitoring Sites.

<table>
<thead>
<tr>
<th>Alternative</th>
<th>Percent Wave Height Reduction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>East Twin</td>
</tr>
<tr>
<td>Offshore Breakwater</td>
<td></td>
</tr>
<tr>
<td>Above Low Water Datum</td>
<td>-70%</td>
</tr>
<tr>
<td>Submerged</td>
<td>-32%</td>
</tr>
<tr>
<td>Dog-leg Extension</td>
<td></td>
</tr>
<tr>
<td>North Jetty</td>
<td>-50%</td>
</tr>
<tr>
<td>South Jetty</td>
<td>-68%</td>
</tr>
<tr>
<td>Location</td>
<td>-6%</td>
</tr>
<tr>
<td>----------------------------------------------</td>
<td>-----</td>
</tr>
<tr>
<td>Interior Stub Breakwater</td>
<td>-28%</td>
</tr>
<tr>
<td>Rock Along Back of the Harbor</td>
<td>-10%</td>
</tr>
</tbody>
</table>
7 Modeling Results

The modeled alternatives in this study were selected from a broad range of possibilities. The intention was to gain an understanding of viable alternatives for further analysis required for design and cost estimation. These results are best used to identify trends in wave height reduction in response to various storms and wave climates. Isolated values of percent change especially along harbor walls should garner extra scrutiny. A slight change in storm direction could lead to one alternative slightly outperforming another. This is expected as a single alternative cannot be the best for every storm. Through an optimization process performed prior to design level modeling, selected alternatives that reduce wave heights for the broadest range of conditions will be identified.

This modeling was successful in removing some alternatives from consideration. There are many alternatives that appear to be good candidates for further investigation and these will benefit from some sort of optimization analysis. For example, should the Dog-leg Extension be something the City of Two Rivers would like to pursue, several iterations adjusting the angle, length and location are suggested prior to design level modeling. While some of these options may be technically viable, political or financial motivations may make it an undesirable alternative. These require input from the City of Two Rivers and other stakeholders for further analysis.

Generalized model results suggest the largest reductions in wave heights are associated with above water structures exterior to the harbor. When considered as a stand-alone alternative, the submerged Offshore Breakwater proved rather inconsistent at achieving reductions in wave height when compared to other alternatives. The Offshore Breakwater – Submerged is most effective during times of larger waves as seen in Figure 60 and Figure 61. However, for both Storm One and Storm Two, percent reductions return to near zero after the peak of the storm and then percent reductions increase again for smaller wave heights, perhaps in response to a directional shift shown in Figure 62. The effectiveness of the proposed Offshore Breakwater Submerged was further investigated for a directional dependency. A monochromatic wave, 7.5 feet high and with a shore normal direction was modeled. Results were compared to similar simulations with the same monochromatic wave height but the direction was shifted +/- 15 degrees. Generally, none of the interior monitoring locations showed signs of a directional dependency. If interest in this alternative exist, additional optimizations are recommended.
Figure 60: Submerged Offshore Breakwater Details, Percent Reduction, Storm One

Figure 61: Submerged Offshore Breakwater Details, Percent Reduction, Storm Two
Figure 62: Submerged Offshore Breakwater Details, Directional Comparison, Storm One

Depending on the storm direction, the Offshore Breakwater seems to provide the greatest reduction in wave heights. The two Dog-leg Extensions (North and South Jetties) are the two next best options to reduce wave heights. The Dog-Leg Extension – South Jetty out performs the Dog-leg Extension – North Jetty at every location for each modeled storm. This is likely due to the fact that the predominant wave energy has a southern component.

Figure 63. Two Rivers Bathymetry
The Rock Along Back of the Harbor provides approximately 25 to 40 percent reductions in wave height at the harbor junction compared to existing geometry for each modeled storm; larger reductions are realized along the south wall of the West Twin River. Harbor resonance occurs when the incident waves are in phase with the reflected waves. Solving harbor resonance problems relies on one of two approaches. Placing additional stone in the harbor helps remove resonance by decreasing the energy associated with the reflected waves. The other alternative is a major realignment of the harbor to alter the phase of the reflected waves. Placing rock at the harbor junction is expected to reduce the energy of the reflected wave, and help with harbor resonance.

The Interior Stub Breakwater provides localized reduction in wave heights in the vicinity of the proposed developments with increases in wave heights when compared to the existing geometry near the Coast Guard station. Should this alternative prove viable, it will greatly benefit from optimization scenarios to ideally determine the angle and length of placement.

None of these alternatives were examined in conjunction with any other alternative. There is a potential for multiple alternatives to be combined into a single solution. When the City of Two Rivers is ready to move forward with design level modeling, it is recommended that any potential combination of alternatives be modeled as a single treatment.
8 Shoaling Discussion

While sediment transport modeling was not an explicit part of this study, there was interest in how these alternatives would affect shoaling at a cursory level. As such, the Detroit District’s opinion as to how each of the alternatives will affect shoaling is discussed. Since the accretion and erosion effects have not been quantified, some of the opinions expressed may appear to contradict each other. For example, an alternative may be expected to intuitively cause shoaling due to a wave reduction but also increase erosion due to altered hydraulics. Quantifying these processes with a model or calculations is required to determine whether the dominant process is erosion or accretion.

Offshore Breakwaters – Both the submerged and above water breakwater will create a wave ‘shadow zone’ landward of the proposed structure which would be expected to increase shoaling. However, it is unlikely that any adverse effects would be seen since the jetties already exist in this ‘shadow’. The proposed structure is expected to cause shoaling in the same area in which sediment is currently accumulating (the flair in the federal navigation channel). As long as the shoaled material is placed (after dredging) on the downdrift side, there will be no interruption to the longshore transport due to the structure.

Dog-leg Extension – North Jetty – This alternative is expected to affect shoaling in two ways. First, it is effectively increasing the length of the breakwater which in turn creates space for further accommodation within the accretion fillet on the north and south side. If the fillets are currently full and bypassing sand (and we don’t know the status of this question), then this alternative will slow or stop the bypassing while the fillets refill to capacity. This is likely to result in a temporary reduction in the need to dredge but at the expense of reducing the sand supply to the shoreline to the north and south of the harbor and possibly increasing erosion of the shoreline. Secondly, material from the south would have a more difficult time bypassing the harbor and would likely get caught in the navigation channel of the proposed dog-leg since it opens to the south. If this alternative is selected, the accretion fillets should be prefilled to eliminate any interruption to the sand supply to the shoreline north and south of the harbor. This interruption could also be mitigated with beach nourishment.

Dog-leg Extension – South Jetty – This alternative is expected to have the same effect as the north dog-leg alternative with one exception. Since the dog-leg opens to the north and since it appears that the dominant direction of longshore transport (based on bathymetry and geomorphology) is to the north, this alternative is less likely to trap material trying to bypass from the south. Strictly from a shoaling perspective, this alternative is preferred to the north dog-leg alternative.

Interior Stub Breakwater – Shoaling in the East and West Twins upstream of the proposed structure may increase if the structure creates backwater up into these tributaries. While this alternative is not changing the size of the cross section, the
backwater will likely decrease the stream’s slope and its ability to transport sediment. Accumulation of sediment and debris is expected on the lakeward side of the stub as this will be an ineffective flow area and will become depositional. The shape of the stub may be altered to reduce accretion.

*Rock Along Back of the Harbor* – This alternative is not expected to increase shoaling but may play a minor role in where sediment deposits. The existing steel sheet pile wall is more reflective than a stone wall, so one would expect the reflected waves to carry sediment more to the middle of the channel. The alternative would likely result in deposition closer to the banks, where it is less likely to interfere with navigation. This change, however, is likely to be small.

*Rock Along the South Side of the Harbor* – Lining the south jetty with armor stone will not affect the amount of sediment or wave energy that enters the harbor; however, it may influence where along the channel it will deposit. Without the stone, the more energetic wave climate may be able to transport sediment further up the channel, while the alternative with stone may reduce the ability of the wave to transport sediment up the channel, resulting in shoaling more landward than without the stone. This difference, however, is likely to be small.
9. Qualitative Comparison of Alternatives

A qualitative matrix was developed in order to provide decision makers with a frame of reference as to how each alternative might influence the existing harbor conditions. Implementing any of the proposed alternatives could change the existing conditions within the harbor. Table 4 below provides a qualitative assessment of how each alternative might alter shoaling patterns, affect safety/navigability, reduce wave heights, and cost to construct.

Table 4: Alternative Matrix

<table>
<thead>
<tr>
<th>Alternatives</th>
<th>Shoaling</th>
<th>Risk to Safety/Navigability</th>
<th>Reduction to Wave Height</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Offshore Breakwater – Above Low Water Datum</td>
<td>≈</td>
<td>M</td>
<td>H</td>
<td>$$$</td>
</tr>
<tr>
<td>Offshore Breakwater – Submerged</td>
<td>≈</td>
<td>H</td>
<td>L</td>
<td>$$$</td>
</tr>
<tr>
<td>Dog-leg Extension – North Jetty</td>
<td>Increase</td>
<td>L</td>
<td>H</td>
<td>$$</td>
</tr>
<tr>
<td>Dog-leg Extension – South Jetty</td>
<td>Decrease</td>
<td>L</td>
<td>H</td>
<td>$$</td>
</tr>
<tr>
<td>Interior Stub Breakwater</td>
<td>Increase</td>
<td>H</td>
<td>L</td>
<td>$</td>
</tr>
<tr>
<td>Rock Along Back of the Harbor</td>
<td>≈</td>
<td>L</td>
<td>M</td>
<td>$$</td>
</tr>
<tr>
<td>Rock Along South Side of the Harbor</td>
<td>≈</td>
<td>H</td>
<td>L</td>
<td>$$</td>
</tr>
</tbody>
</table>

*Note: Description of the matrix ratings are discussed in each subsection.

9.1 Shoaling

As seen above, only the Dog-leg Extension – South Jetty alternative would most likely decrease the severity of shoaling and reduce the frequency of dredging. Conversely, the Dog-leg Extension – North Jetty is likely to trap sediment moving northward and the Interior Stub Breakwater will also trap sediment on the lakeward side of the structure. The four remaining alternatives – Offshore Breakwater – Submerged, Offshore Breakwater – Above Low Water Datum, Rock Along the Back of the Harbor and Rock Along the South Side of the Harbor – are expected to have little to no impact to the existing shoaling patterns. This negligible impact has been denoted in the matrix using the approximately equal to symbol, or ≈.

9.2 Risk to Safety/Navigability

All of the alternatives were qualitatively assessed in terms of navigability and safety. It should be noted that most vessels approach the harbor either head on or from a northerly direction. The largest vessel utilizing the harbor is 65 feet in length and approximately 18 feet in width. This vessel is owned and operated by the Suzie-Q Fish Company. Also important, a Coast Guard station is located within the harbor. The largest vessel they operate is 50 feet in length and 14 feet wide.

For comparative purposes, a qualitative rating was developed for each alternative to highlight any potential safety and navigability risks that might arise. The letter “H” (High risk) signifies potentially unsafe navigation conditions. Whereas the letter “M” (Moderate risk)
risk) indicate that navigability and safety should not be significantly impacted, but would increase the risk of the existing conditions. Notably, the letter “L” is used to show a Low risk to existing safety and navigation conditions.

The Offshore Breakwater – Above Low Water Datum would not pose any serious problems to navigation. However, wave action may make it difficult or challenging for vessels entering and exiting the harbor. Notably, vessels would be required to make two 90 degree turns to enter or exit the harbor if this alternative was constructed. In addition, navigation lights should be placed at both ends of the structure to warn of this hazard during nighttime navigation. The Offshore Breakwater – Above Low Water Datum has been rated a moderate risk for these reasons. In contrast, the Offshore Breakwater – Submerged is a continuous navigation hazard since this structure would not be visible to mariners at any time and is given a high risk rating. Consequently, the Detroit District does not support this alternative or recommend that the city pursue its implementation.

Both of the Dog-leg Extension alternatives (North and South Jetty) do not pose any serious hazards to navigation. Similar to the Offshore Breakwater – Above Low Water Datum alternative, navigation lights should be placed at the end of the structure to aide in nighttime navigation. As previously mentioned, the majority of vessels entering/existing the harbor are traveling in a northerly direction or directly out into the lake. Thus, the Dog-leg Extension – North Jetty would require vessels to pass the structure and then make a 120-degree turn in order to access to harbor. Navigating the Dog-leg Extension – South Jetty would provide the most accessible route in and out of the harbor. Thus, both of the Dog-leg Extension alternatives (North and South Jetty) have been given a low risk rating.

The Interior Stub Breakwater alternative has been given a high risk rating because it creates a significant hazard to navigation and safety. This alternative involves the placement of a vertical wall within the inbound portion of the Federal channel. Thus, vessels entering the inner harbor would have to maneuver around the structure, into oncoming traffic (or the left hand side of the channel). In short, this alternative would create a blind intersection and navigation obstacle for incoming vessels. Thus, the Detroit District does not support this alternative or recommend that the city pursue it.

As seen in the above matrix, Rock Along the Back of the Harbor is expected to have a low risk impact to vessel navigation, maneuverability or safety. This is primarily due to the fact that the bank would need to be cut back and rock would be placed along the river’s edge as a means of absorbing wave energy. Because the bank is being cut back, dimensional rock would not be placed within the footprint of the navigation channel. However, the alternative Rock Along the South Side of the Harbor would require dimensional rock to be placed along the edge of the navigation channel which may pose a hazard to outbound vessels and is therefore rated a high risk.

9.3 Impact on Wave Mitigation
The purpose of this study was to identify potential structural solutions for mitigating wave energy in the Two Rivers Harbor. For comparative purposes, the matrix above
presents a qualitative rating for each alternative showing how well the alternative may reduce wave heights. To develop the matrix rating, qualitative ratings (low = <25%, medium=25-49% and high = >50%) were applied to the percent wave reduction reported for each alternative’s four observation points. The observation point ratings were then aggregated for each alternative under the three storm events in order to develop the above matrix rating. Notably, the qualitative rating indicates whether one might expect a low, medium or high reduction in wave height if the alternative were constructed.

Both of the Dog-leg Extensions are rated high in terms of their ability to reduce wave height in the harbor. The Offshore Breakwater – Above Low Water Datum also scores high in terms of reducing harbor wave heights. Rock Along the Back of the Harbor was the only alternative to be assigned a medium rating. The three remaining alternatives – Rock Along South Side of the Harbor, Interior Stub Breakwater and Offshore Breakwater – Submerged – received low ratings for reducing wave height.

9.4 Generic Cost Discussion
The matrix, above, provides qualitative information about the potential level of cost to implement the study alternatives. Only conceptual level alternatives were modeled for this study. As a result, the cost information presented in the matrix should only be used as a reference and is not based on any in-depth engineering analysis. Dollar signs are used to signify the potential level of cost associated with each alternative. For example, an alternative with one dollar sign would most likely be lower in cost in comparison to an alternative with two dollar signs.

The Offshore Breakwater alternatives (both Submerged and Above Low Water Datum) would most likely be the most costly to construct given their size and the depth of the water where it would be constructed. Notably, these alternatives would require marine-based construction equipment which is more costly than the land-based equipment. Both of the Dog-leg Extension alternatives would also require significant amounts of stone to be placed in deep water; however, the depth of the water is several feet shallower than the Offshore Breakwater alternative. In addition, the Dog-leg Extension alternatives may be able to utilize land-based equipment during construction instead of marine equipment. Thus, the Dog-leg Extension alternatives were rated less costly than the Offshore Breakwater alternatives.

As seen in the matrix, the stub breakwater would most likely be the most cost effective of the proposed alternatives. Its lower cost is mostly attributed to its smaller size and proximity to land. The Rock Along Back of the Harbor and Rock Along South Side of the Harbor would most likely be comparable to the Dog-leg Extension given the amount of rock needed for placement. However, the cost of the Rock Along Back of the Harbor could likely become much higher depending on the level of soil contamination.
10. Permitting and De-Authorization

Any alterations to Federal navigation projects by others than the Corps require a Section 408 permit. Under this process (authorized by Section 14 of the Rivers and Harbors Act of 1899 and codified in 33 USC 408), the USACE will review any proposed alteration or occupation or use of a USACE civil works project to determine if the proposed activity will not be injurious to the public interest and will not impair the usefulness of the project.

All of the alternatives evaluated in this study would, at a minimum, require a formal Section 408 permit prior to construction. Details regarding the Section 408 process can be found on the District’s website (http://www.lre.usace.army.mil/Missions/Civil-Works/Engineering-and-Construction/Section-408/). The city should contact the Detroit District’s Section 408 Program Coordinator (LRE408Coordinator@usace.army.mil) if there are any questions regarding the Section 408 process.

Several of the alternatives will likely require deauthorizing a specific portion of the Federal navigation channel. Given that the alternatives are only conceptual and no detailed plans/specifications are available, a determination as to whether deauthorization will be needed cannot be made at this time. The Detroit District recommends that the city engage both the Section 408 Coordinator and the Chief of Programs early on in the city’s design phase.