HIKA BAY HARBOR OF REFUGE

PRELIMINARY COASTAL ANALYSIS PREPARED BY

FRESHWATER ENGINEERING

Prepared for:
MSA Professional Services
Madison, WI
March 2018

Funding by:

WISCONSIN COASTAL MANAGEMENT PROGRAM

FRESHWATER ENGINEERING

March 19, 2017

Bruce Lunde
MSA Professional Services
2901 International Lane
Suite 300
Madison, Wisconsin 53704

RE: Hika Bay Harbor of Refuge Preliminary Coastal Analysis

Dear Mr. Lunde,

FreshWater Engineering is pleased to submit this preliminary coastal analysis for the Hika Bay Harbor of Refuge in Cleveland, WI. FreshWater understands that the findings of this analysis are intended to inform the next phases of the project and facilitate communication about the project.

We look forward to further discussing the project findings with you and your staff. Feel free to contact me should you have any questions or require additional information.

Sincerely,

Laura Rozumalski, P.E.
President and Principal Engineer
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1 Introduction

FreshWater Engineering was retained by MSA Professional Services to provide a preliminary coastal analysis for the proposed Hika Bay Harbor of Refuge in Cleveland, Wisconsin. The objective of the analysis is to provide information on coastal processes at the project site to aid the design team in conceptual design and communications.

The scope of this study is limited to the following:

- Review, synthesize, and summarize environmental data available for Centerville Creek
- Analyze USACE Wave Information Studies data to determine the offshore wave climate at the site
- Assess nearshore wave climate using offshore waves transformed with linear wave theory
- Estimate littoral sediment transport regime at the site using available data and bulk sediment transport models
- Analyze quantitative historical shoreline evolution at the site and surrounding area using available historical aerial photos and GIS tools

2 Physical Setting

2.1 Location

The project site is located on the western shore of Lake Michigan in Cleveland, Wisconsin in southern Manitowoc County. Hika Park is located near the center of Hika Bay, a broad, shallow bay along the coast of Sheboygan and Manitowoc Counties. Centerville Creek enters Lake Michigan at the northern boundary of the park. The existing boat launch is located at the southern end of the park. The park shoreline is approximately 250 ft long. A location map is shown in Figure 1.

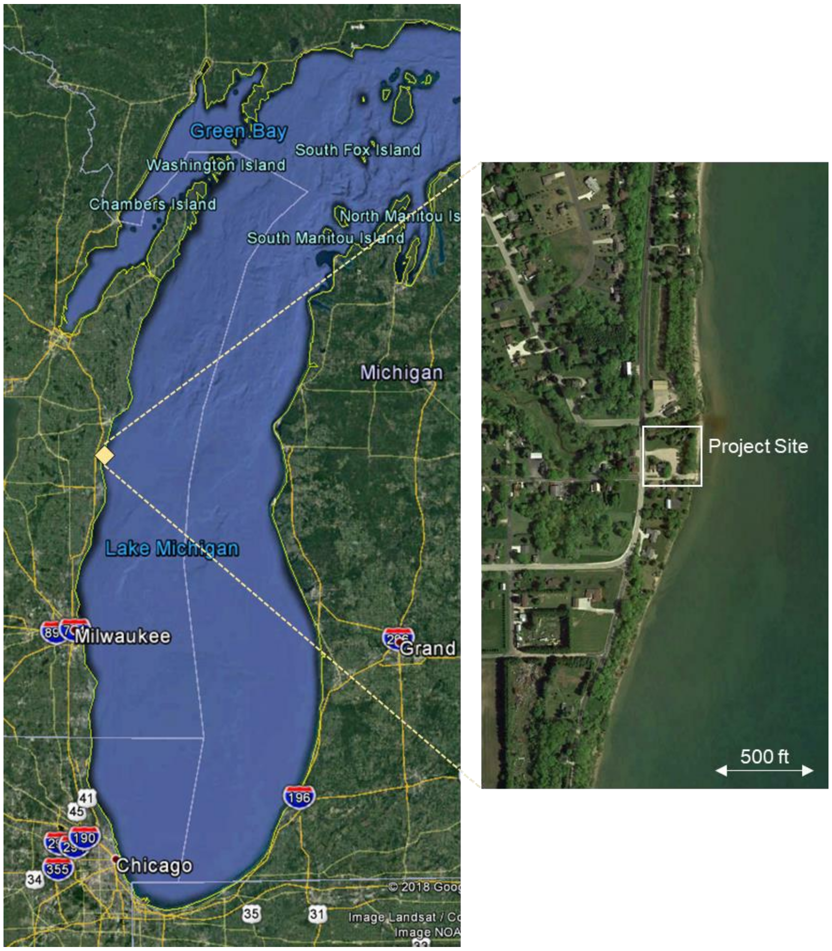

Figure 1. Location map of the project site.

2.2 Site Geology

A shoreline inventory in Sheboygan and Manitowoc Counties was conducted in 1977 by Hadley et al. The area surrounding Centerville Creek was found to be a low-lying coast with an average elevation of about 3 feet over about 4,000 feet of shoreline. The survey found beaches to be 20 to 35 feet wide and composed of sand and cobble. Water depths 50 feet offshore were 1 to 3 feet, with sandy nearshore sediments. To the south of the town of Cleveland, steep bluffs 50 to 60 feet high are present and in 1977 were found to be unstable and eroding at rates of about 0.7 feet/year over the long term. Bluffs north of Cleveland are 40-50 feet tall and were also found to be unstable in 1977.

2.3 Site History

Oblique aerial photos along the shore of Lake Michigan have been taken irregularly since 1976. These photos are available on the Wisconsin Shoreline Inventory and Oblique Photo Viewer (http://floodatlas.org/asfpm/oblique_viewer/). Photos are available for the Hika Park project site from 1976, 2007, 2012, and 2017 and are shown in Figures 2 through 5.

Generally, the site has not shown significant change since 1976. The beach widens to the north of the boat launch to Centerville Creek. Photos taken during periods of high water levels (1976, 2017) show much narrower beaches and a wider mouth. During lower water level years (2007, 2012), beaches are relatively wide and the mouth of Centerville Creek is narrower.

Figure 2. 1976 oblique aerial photo of the project site.

Figure 3. 2007 oblique aerial photo of the project site.

Figure 4. 2012 oblique aerial photo of the project site.

Figure 5. 2017 oblique aerial photo of the project site, split between two photos. There is no photo coverage immediately north of the boat launch.

2.4 Centerville Creek

The outlet of Centerville Creek enters Lake Michigan at the north end of Hika Park. A historic millpond was drained by dam removal in 1998, and riparian habitat areas were restored and stabilized to a point about 300 feet west of the creek outlet. Physical data on the stream is provided from design reports on the restoration and sporadic stream monitoring efforts. The restored stream channel has depths ranging from 1.7 to 4 feet, an average slope of 0.0056, and a bankfull width of about 15 feet (Interfluve, 2011). Channel bed sediments consist mainly of gravel and cobble. Measurements of stream velocity taken in 2013 show that surface flow velocities during average conditions are about 0.2 feet/second, which corresponds to stream flow of 1 to 3 cubic feet per second (cfs) (Poling, 2013).

3 Coastal Analysis

3.1 Lake Michigan Water Levels

Lake Michigan water level data is obtained from NOAA/NOS/CO-OPS gage station 9087057, located in Milwaukee Harbor. Daily average water levels are available from 1970-2018, and annual average lake-wide water levels are available from 1918-2017.

The average annual water level on Lake Michigan is 578.82 feet above the International Great Lakes Datum 1985 (IGLD85). Lake Michigan's water level fluctuates over periods of hours, days, and years as a result of water balance in the lake basin; summarized in Figure 6 (Gronewold et al., 2013). The highest annual average water level of 581.66 feet occurred in 1986, and the lowest annual average water level of 576.38 feet occurred in 1964.

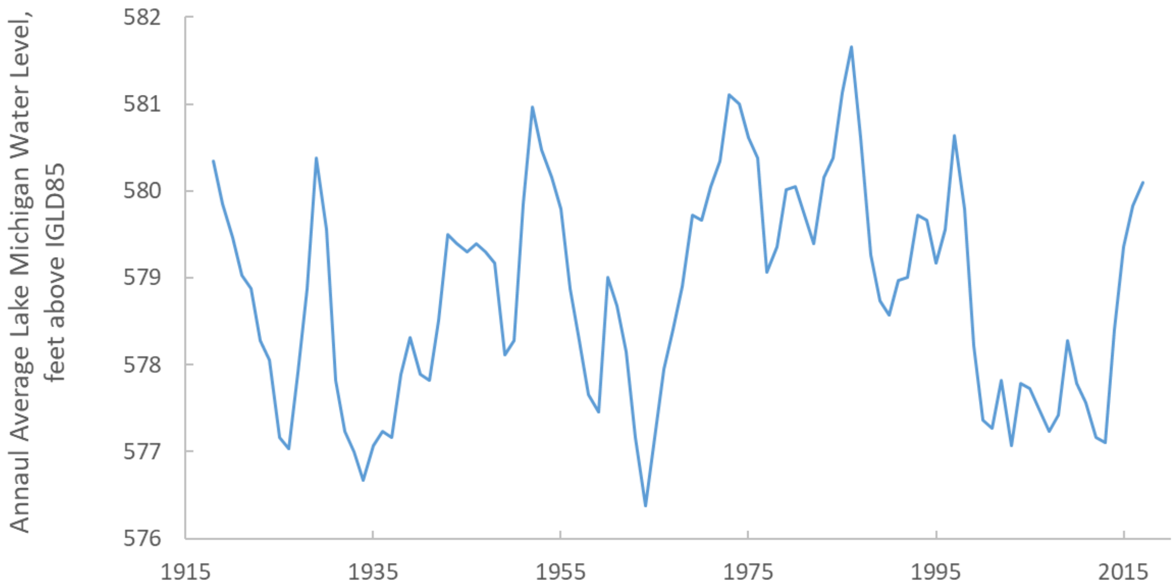

Figure 6. Timeseries of annual average Lake Michigan water level, 1918-2017.

The United States Army Corps of Engineers (USACE) has determined lake-wide high water and low water datums for Lake Michigan. The lake-wide low water datum (LWD) is 577.5 feet, and the ordinary high-water mark (OHWM) is 581.5 feet. The OHWM is determined from land surveys of physical and biological indicators which approximate the contour of the upper extent of lake water. In this sense, the OHWM accounts for short-period water level fluctuations such as seiches, and hydrodynamic factors including wave runup and wind setup.

Analysis of water level gage data was performed to determine the extremes in elevations of still water level at the project site. Daily averages of water levels from 1970-2017 were selected to reduce the influence of short-term fluctuations. Figure 7 shows an empirical cumulative distribution function of daily average water levels. The 95th percentile water level (i.e., the water level greater than all but 5% of water level records) is 580.97 feet, and the 5th percentile water level is 577.07 feet. These values can be considered analogous to mean highest high water (MHHW) and mean lowest low water (MLLW) on oceanic coasts. When compared to OHWM and LWD values, it can be seen that OHWM slightly exceeds the 95th percentile, while the LWD slightly exceeds the value of the 5th percentile water level. The OHWM value of 581.5 feet slightly exceeds the 95th percentile value and would correspond to the 96th percentile water level. The LWD exceeds the 5th percentile value and corresponds to the 14th percentile value (Table 1).

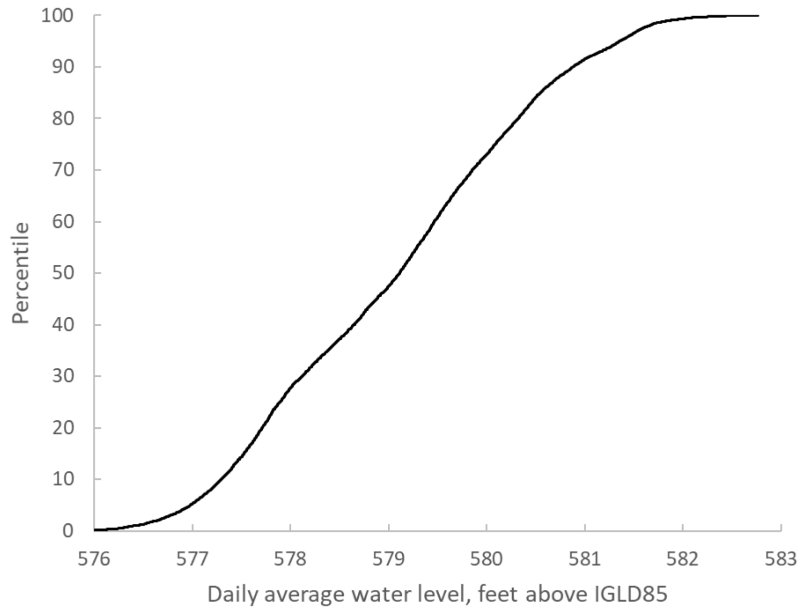

Figure 7. Ranked distribution of daily average water level from 1970-2017.

Table 1. Summary of average and extreme water levels in Lake Michigan.

Long-term Annual Average	OHWM (USACE)	LWD (USACE)	95th Percentile (gage)	5th Percentile (gage)
578.82 ft	581.5 ft	577.5 ft	580.97 ft	577.07 ft

3.2 Shoreline Change Analysis

Recent shoreline evolution in the reach immediately surrounding the project site was measured using historical aerial orthophotographs from the years 1992, 2005, 2010, and 2015. The focus of shoreline change analysis is to investigate the roles Centerville Creek and the existing boat ramp structure play on shoreline evolution in the area, and to determine recent erosional or accretional trends on the shoreline.

3.2.1 Methods

Historic shoreline changes were digitized from orthorectified aerial photos in GIS software. Photos were imported into the GIS system and shorelines were manually digitized, a common method used for geomorphic change analysis (e.g., Zuzek et al., 2003). Shorelines were saved in a shapefile format. The USGS software Digital Shoreline Analysis System was then used to measure positional change between historical shorelines at 10-meter intervals along transects cast from a baseline parallel to the shore.

Shoreline change was analyzed using two methods: the end point rate, in which the distance between two shoreline points is divided by the time difference; and the linear regression rate, in which a linear regression is used to find a best-fit trend of shoreline movement along a transect line.

3.2.2 Data Sources

Shorelines were delineated on digital aerial orthophotos obtained from public data sources. Table 2 provides a summary of the photo sources and water levels at the time of each photo.

Table 2. Summary of photo sources used in shoreline change analysis.

Date	Type	Resolution	Source	Daily mean water level (feet)
Spring 1992	County-wide photo mosaic	2 m	WI DNR	579.03
7/25/2005	Digital ortho quarter quad	1 m	NAIP	578.08
7/3/2010	Digital ortho quarter quad	1 m	NAIP 578.28	9/22/2015
	Digital ortho quarter quad	1 m	NAIP	579.79

3.2.3 Shoreline Change Analysis

Photos of the shoreline in the project area show that, in general, changes to the shoreline have been minor between 1992 and 2015 (Figure 8). A relatively wide beach is present north and immediately south of Centerville Creek in all photos, though the beach is submerged in 2015 due to the higher water levels. South of the boat launch, beaches are narrow and not present in 2015. Low water levels that persisted from 2000-2014 explain why beaches appear wider in the 2005 and 2010 photos. A sharp rise in water level after 2014 explains the apparent shoreline retreat in 2015.

Figure 9 shows the location of the shorelines used in this analysis superimposed on the 2010 aerial photo. North of the boat launch, the 1992 and 2015 shoreline positions appear to closely match, while the 2005 and 2010 shoreline positions are also similar. South of the boat launch, shoreline differences are much smaller. This is likely due to steeper foreshore slopes in the area where a low bluff is present.

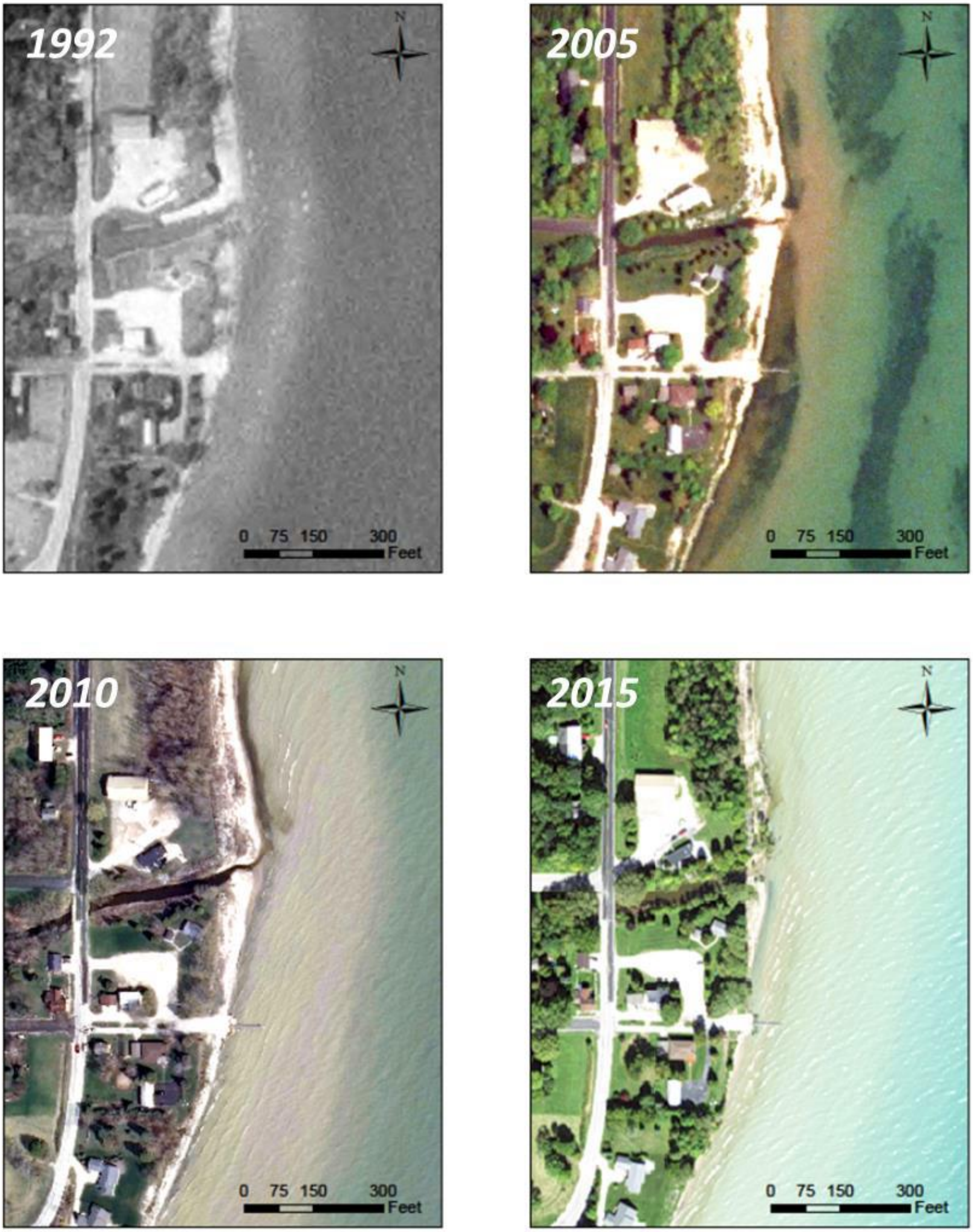

Figure 8. Orthophotos used for analysis of recent shoreline change.

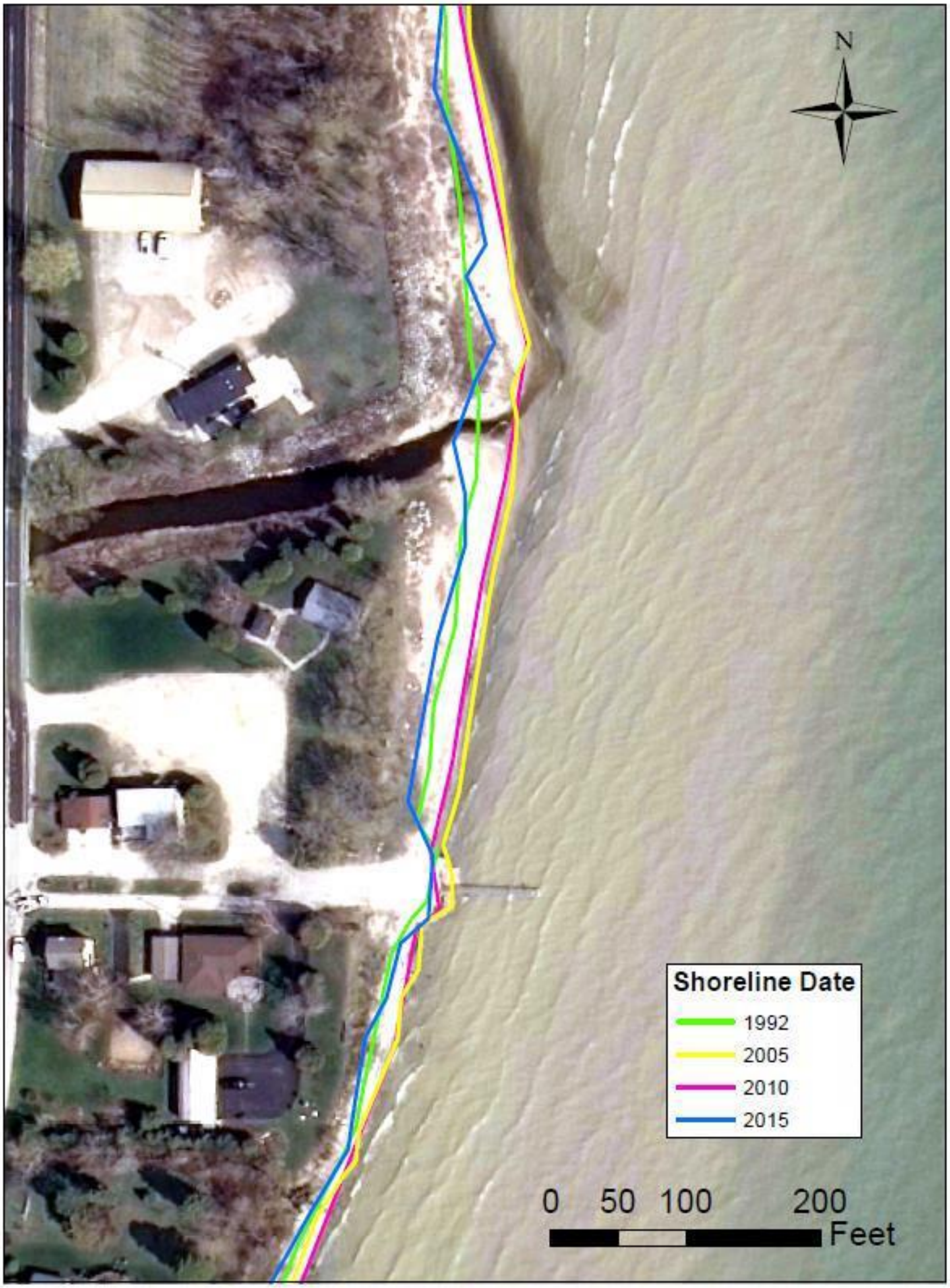

Figure 9. Historical shorelines near the project site shown on the 2010 aerial photo.

Shoreline change in the reach extending 500 meters to the north and south of the boat launch is highly dependent on the local shoreline characteristics. To the north of the launch and in Hika Bay Park, shoreline position on the wide sandy beaches is highly sensitive to water level fluctuations, as evidenced by the -3.86 ft/year retreat rate between 2010 and 2015 (Table 3). South of the launch shoreline position is less sensitive to water level changes, suggesting a steeper foreshore slope where a low bluff is present. The linear regression rate for the reach is 0.02 feet/year between 1992 and 2015, indicating that the shoreline is stable, neither eroding nor accreting between periods of high water (Figure 10).

It is possible that this analysis underestimates erosion rates since the analysis principally considered a period of low water levels. The current sustained period of high water levels could cause shoreline erosion at much higher rates as waves are allowed to break closer to shore. Further investigations that include beach slope measurements would be able to determine if shoreline erosion has taken place using the 'Bruun's Rule' principle.

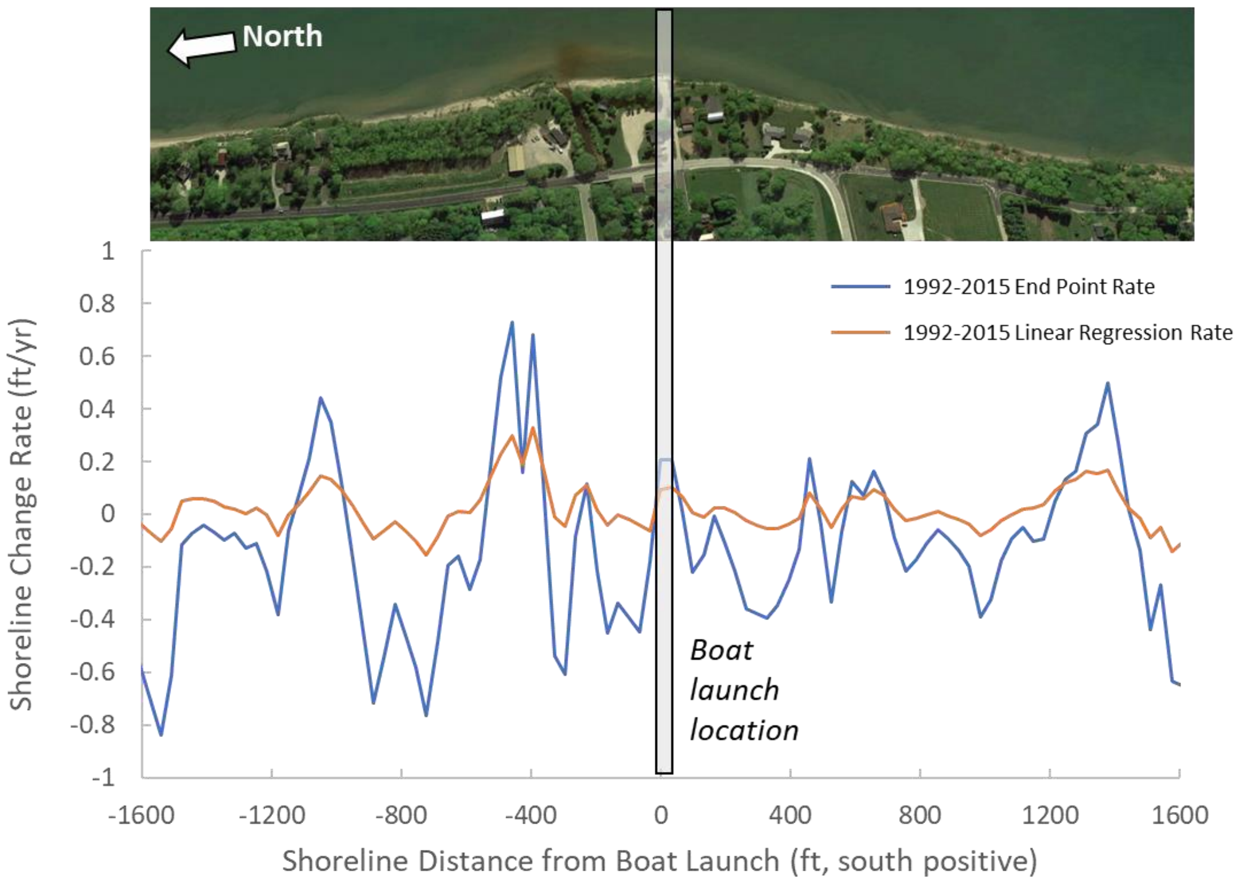

Figure 10. Comparison of EPR and LRR methods of shoreline change measurement in the area surrounding the project site.

Table 3. Summary table of shoreline changes for the 1 km reach surrounding Hika Park.

Location	End Point Rate 1992-2005	End Point Rate 2005-2010	End Point Rate 2010-2015	End Point Rate 1992-2015	Linear Regression Rate 1992-2015
Entire Reach	1.30	-0.76	-3.28	-0.14	0.02
North 500 m	1.59	-0.57	-3.86	-0.22	0.01
South 500 m	0.95	-0.66	-2.34	-0.12	0.02
Hika Park	1.75	-1.54	-3.75	-0.16	0.01

3.3 Offshore Wave Climate

The offshore wave climate information was obtained from USACE Wave Information Studies (WIS) hindcast model data. WIS uses third-generation spectral wave models to hindcast (i.e., model past conditions) wind and wave conditions at offshore locations on US coastlines. These data are well validated (Jensen, 1994) and provide valuable information where buoy data is unavailable.

The nearest WIS station to the project site is WIS Station 94076, located east of the project site at a water depth of 112 feet (34 m). The data series reports hourly wind and wave conditions between 1979 and 2014.

Wind data recorded at WIS station 94076 are summarized in the wind rose in Figure 11. It is observed that wind direction and speed are well distributed between the northerly, westerly, and southerly directions. The average wind speed is 14.0 miles per hour (mph), and the maximum wind speed occurred on November 11th, 1998, from the southwest at 53.24 mph.

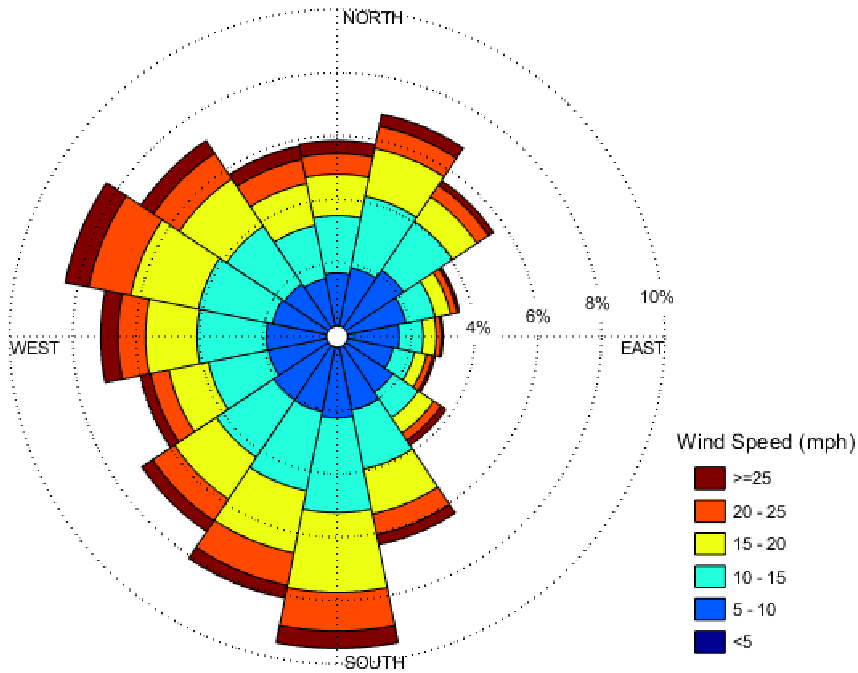

Figure 11. Wind rose at the offshore location at WIS Station 94076.

WIS wave hindcast model results are summarized in the wave height rose in Figure 12. The wave rose includes hourly significant wave height from 1979-2014, except during times when surface ice was present at the model. The largest waves (up to 16 feet) approach the project site from the northeast and south-southeast directions. These directions account for 18% and 22% of total waves, respectively. Large waves greater than 8 feet in height make up less than 0.006% of the total wave record, and generally approach from the northeast or southeast.

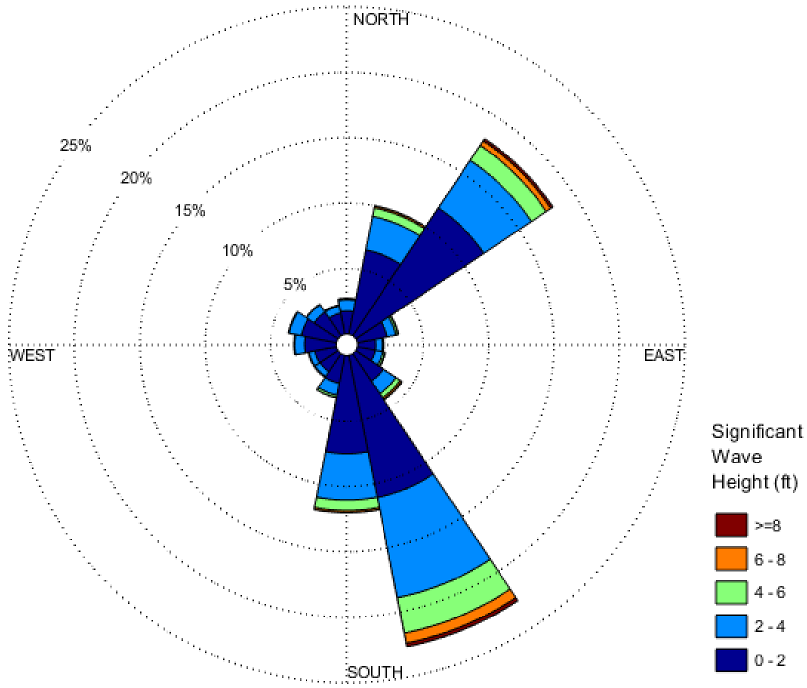

Figure 12. Wave rose for offshore location at WIS Station 94076.

Table 5 summarizes the top ten storm events in the WIS data record from 1979-2014 and storm data records from 1960-1978. Seven of these storm waves approach from the northeast and three approach from the south-southeast. The largest modeled wave occurred on February 10, 1960 with a significant wave height of 16.14 feet and a period of 8.69 seconds.

Table 4. Summary of the ten largest storms recorded by WIS Station 94076.

Event	Date	Max. Sig. Wave Ht. (ft)	Peak Period (s)	Direction (deg. from N)
1	February 10, 1960	16.14	8.69	49
2	December 4, 1990	15.84	10.38	38
3	January 4, 1982	15.71	9.53	36
4	December 12, 2012	15.58	9.42	37
5	November 11, 2011	15.32	9.51	176
6	December 23, 1988	14.73	8.68	158
7	January 18, 1996	14.30	8.46	162
8	December 15, 1987	13.97	9.93	37
9	February 8, 1987	13.48	10.32	33
10	April 1, 1993	13.45	9.15	43

The WIS model record from 1979-2014 was analyzed to determine the wave height exceedance frequency characteristics for offshore waves. Storm events were selected using the peaks-over-threshold method and selected the highest significant wave height in data clusters separated by at least 48 hours. Significant wave heights corresponding to certain recurrence intervals were determined using the Generalized Pareto Distribution using the methods of Anderson et al. (2015). The findings of the wave height-frequency analysis are summarized in Table 6. The '1-year' wave height is 11.2 feet, and the '50-year' wave height is 16.3 feet.

Table 5. Offshore wave height/frequency analysis results.

Recurrence Interval (years)	**1**	**5**	**10**	**25**	**50**	**100**
Wave Height (feet)	11.2	12.2	14.4	15.5	16.3	17.1

3.4 Nearshore Wave Climate

Offshore waves obtained from WIS Station 94076 were transformed to the assumed offshore limit of the beach profile for nearshore wave analysis and sediment transport estimate. The offshore profile boundary is the depth at which the beach profile is assumed not to change and can be approximated as $H = 1.56H_e$, where H_e is the effective wave height with a 0.137% chance of exceedance in a given year (Dean and Dalrymple, 2002). At the project location, this depth is approximately 19 feet.

Waves were transformed from the offshore location to a water depth of 19 feet using linear wave theory and a desktop method. The wave height rose in Figure 13 indicates that waves at the nearshore location are bi-modal, approaching from the northeast and southeast directions.

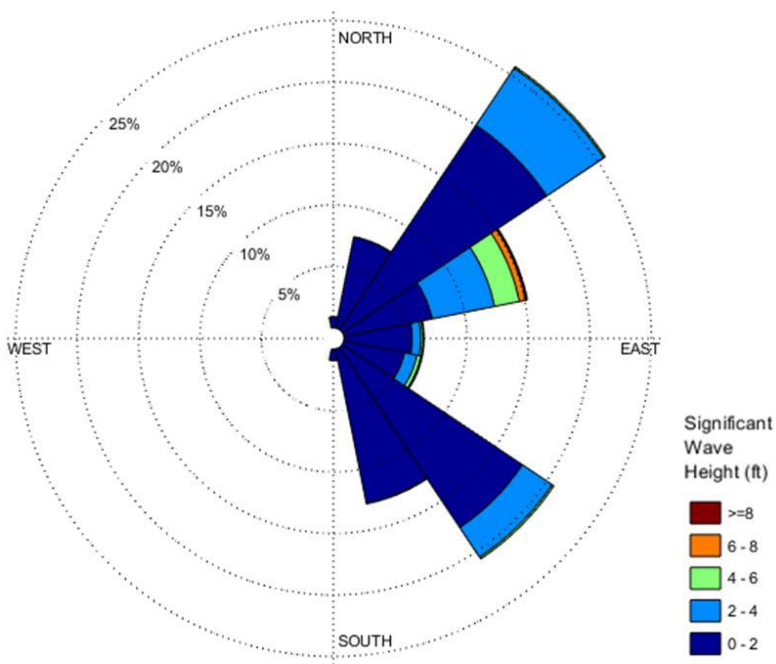

Figure 13. Wave height rose for the nearshore location.

The nearshore transformed wave results were analyzed for wave height exceedance probability using the same methods as the offshore wave analysis. Table 7 summarizes the results of this analysis. The expected '1-year' significant wave height at the nearshore location is 9.4 feet and the '50-year' significant wave height is 14.2 feet.

Table 6. Nearshore wave height/frequency analysis results.

Recurrence Interval (years)	**1**	**5**	**10**	**25**	**50**	**100**
Wave Height (feet)	9.4	11.6	12.4	13.5	14.2	14.9

3.5 Sediment Transport Estimate

Sediment transport estimates for the proposed project location were obtained from formulas using the transformed nearshore wave data. Given the lack of beach profile and sediment information, only bulk longshore sediment transport (LST) rates could be estimated.

The so-called 'CERC' (Coastal Engineering Research Center) equation was used to calculate LST rates at the location because of the lack of beach profile and sediment information (CEM, 2004). The CERC method provides an estimate of potential transport assuming sand covers the entire beach profile bottom. Actual LST rates may differ from these estimates depending on antecedent conditions, beach conditions, actual bathymetry, and wave breaking characteristics. It is also assumed that no sediment transport occurs during times when the water surface is covered with ice.

Annual LST rates from 1979-2014 are shown in Figure 14. Transport rates are divided into northerly and southerly components, with transport to the south noted as positive and transport to the north as negative. The net transport is the balance between the northerly and southerly components. Generally, net transport is directed to the north at an average rate of about 200,000 yd^3/year. The maximum net transport rate occurred in 1988, when about -570,000 cubic yards was transported to the north. Four years (1983, 1987, 1993, and 2011) experienced net transport either near zero or slightly to the south. The maximum net transport to the south was approximately 90,000 yd^3/year in 1987. This reversal in transport direction in 1987 is attributable to several large storms that year which generated large waves approaching from the northeast.

The sediment transport regime present at the project site also includes cross-shore transport across the nearshore profile and fluvial sediment transport in Centerville Creek. Cross-shore transport cannot be reliably estimated without more detailed beach profile and composition information, but in the Great Lakes is generally thought to be balanced between offshore and onshore transport (Colman and Foster, 1994). Sediment transport to the lake from Centerville Creek is not estimated here, but likely occurs at low rates compared to LST rates.

Designs featuring shore-perpendicular protrusions into the lake would be expected to trap littoral sediments to some extent. In general, sand trapping would be expected to the south, though the bi-modal nature of the wave climate means that some beach growth could be expected on the north side of a structure. Further investigation of the area is needed to determine the geomorphic consequences of these structures.

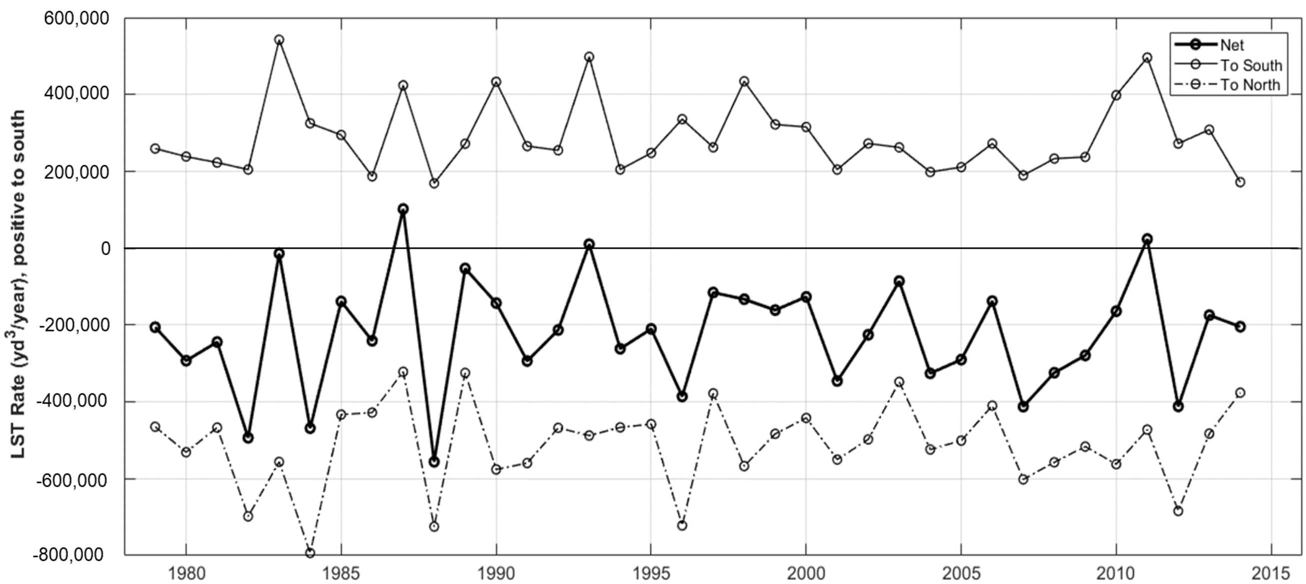

Figure 14. Estimate of annual LST at the project site. Negative rates signify transport to the north and positive rates signify transport to the south.

4 Next Steps

This preliminary investigation of coastal processes at Hika Park is meant to inform the design team of general coastal conditions for the proposed project. Based on the findings of this preliminary analysis, FreshWater recommends the following:

- Further coastal analysis should be performed as a component of design alternative development, including consideration of storms and water level fluctuations.
- Design alternatives, including a 'do-nothing' alternative, should be assessed considering economic, social, ecological, and geomorphic factors.
- Detailed site, beach, and nearshore surveys be performed to inform design and analysis. Survey data may be compared to 2012 topo-bathymetric lidar data to measure recent beach changes following several years of high lake levels.
- Flow measurements and surveys of Centerville Creek should be performed as part of future site investigation. Hydrologic and hydraulic analysis of the creek should be performed for each design alternative.
- Further investigations of the site should include an inventory of nearby coastal infrastructure/resources, measurements of beach and nearshore sediment analyses and measurements of shoreline and backshore erosion since 2015.
- Estimations made in this study indicate that net LST is typically directed to the north. Given the assumptions and uncertainty associated with this estimate, further investigation and modeling is needed to better understand the nature of sediment transport at the site. Design alternatives should consider impacts to sediment transport processes through detailed modeling and 'sediment budget' analyses.

5 References

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