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Port Bay Barrier Bar REDI Project – Concept Plan Report

Wayne County Soil and Water Conservation District, Wayne County, NY



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Table of Contents

Table of Contents.....	i
List of Figures.....	ii
List of Tables	iii
Appendices.....	iii
1.0 Project Background.....	1
2.0 West Barrier Bar.....	1
2.1 Rock Revetment.....	1
2.2 Pier.....	1
2.2.1 Structural Inspection.....	1
2.2.2 Coastal Engineering Considerations.....	2
2.2.3 Dredging Considerations.....	5
2.2.4 Proposed Concept.....	6
2.3 West Channel Wall.....	6
2.3.1 Structural Inspection.....	6
2.3.2 Structural Analysis.....	7
2.3.3 Geotechnical Considerations.....	8
2.3.4 Coastal Considerations.....	9
2.3.5 Dredging Considerations.....	9
2.3.6 Proposed Concept.....	10
2.4 “Gap” area.....	10
2.4.1 Coastal Considerations.....	11
2.4.2 Structural Considerations.....	12
2.4.3 Proposed Concept.....	12
2.4.4 Additional Considerations for Design and Modeling.....	12
2.5 Bay Side of West Barrier Bar.....	12
2.5.1 Bay Side of West Bar Access Road.....	12
2.5.2 South End of Channel Sheet Pile Wall.....	13
3.0 Eastern Barrier Bar.....	15
3.1 Background.....	15
3.1.1 Historic Shorelines.....	15
3.1.2 Sediment Budget.....	22
3.1.3 Design Water Levels.....	22
3.1.4 Bathymetry.....	23
3.1.5 Deep water Waves.....	23
3.1.6 Determination of Incident Waves.....	25
3.1.7 Summary.....	25
3.2 Nature-Based Berm.....	25
3.3 Headland Breakwaters & Beach Fill.....	25
3.4 East Bar Access	26
3.5 East Bar Holistic Shoreline Protection Approach.....	26
4.0 Maintenance and Monitoring	27
5.0 Anticipated Construction Measures.....	27



5.0	Anticipated Construction Measures.....	27
5.1	Sheet Pile Wall and Pier	27
5.2	Rock Revetment – “Gap” and East Bar Access	27
5.3	HeadLand Breakwaters.....	27
6.0	Conclusions.....	28
7.0	References.....	29

List of Figures

Figure 1 – Existing rock revetment along west barrier bar	1
Figure 2 – Condition of sheet pile wall along west side of pier.....	2
Figure 3 – 2020 Lakebed Contours at West Pier.....	3
Figure 4 – Beaver Creek Inlet (filled in with sediment).....	3
Figure 5 – West pier head and existing deflector wall (2 DEC 2020).....	4
Figure 6 – Overwash of cobble and gravel in the navigation channel ~ looking east (2 DEC 2020).....	5
Figure 7 – Condition of sheet pile wall along channel (03 MAR 2021).....	7
Figure 8 – Condition of sheet pile wall along channel (03 MAR 2021).....	7
Figure 9 – Existing sheet pile wall cross section (assumed configuration).....	8
Figure 10 – Dredging photo (03 MAR 2021).....	9
Figure 11 – “Gap” area between rock revetment and pier (looking west) 10AUG2021.....	10
Figure 12 – Current PBIA dredging permit and spoil placement areas (yellow indicates “gap area” location)	11
Figure 13 – Excerpt from Envirolok brochure.....	13
Figure 14 – Area of erosion south of existing sheet pile wall (03 MAY 2021).....	14
Figure 15 – Area of exposed tree roots ~ west bar bayside (03 MAY 2021).....	14
Figure 16 – Port Bay East Barrier Bar Historic Shoreline Change 2002 – 2021.....	17
Figure 17 – Port Bay East Barrier Bar Shoreline Change 2010 -2002 (8 years)	18
Figure 18 – Port Bay East Barrier Bar Shoreline Change 2021 -2010 (11 years).....	19
Figure 19 – Port Bay East Barrier Bar Shoreline Change 1965 & 2021.....	20
Figure 20 – Port Bay East Barrier Bar Estimated Future Shoreline Change 10 YRS, 20 YRS and 30 YRS.....	21
Figure 21 – Recent conditions (sediment budget) at Port Bay (CY per year).....	22
Figure 22 – WIS station location map	23
Figure 23 – Deepwater wave class angle bands for Port Bay REDI-project	24
Figure 24 – Low water and high water east bar access comparison.....	26
Figure 25 – Staging area, headland breakwaters and proposed causeway	28



List of Tables

Table 1 – Design lake levels (Oswego Harbor, NOAA Station 9052030)..... 22

Table 2 – Deep water waves (WIS 91054)..... 24

Appendices

A “Effects of structurally-engineered beached on coastal processes and shores of the Great Lakes”
 by Charles W. Shabica, Michael C. Mohr, and Stefanie Nagelbach.

1.0 Project Background

A SEQR package was originally submitted for the project on June 4, 2021 with the knowledge that further investigation of the proposed concept would be completed by the Bergmann team during the SEQR review and comment period. The plans have been revised in accordance with additional modeling efforts and comments received during initial review. A detailed description of the revised concept plans is included in this report.

2.0 West Barrier Bar

2.1 ROCK REVETMENT

The proposed plan for the west barrier bar in the area of the existing revetment remains unchanged from the June 4 submission. The revetment is stable and performing as expected (see Figure 1). Drainage improvements and the use of a cellular grid to retain the roadway aggregate are proposed to make the roadway more resilient to wave events and surface runoff. Underdrain will be installed to capture surface runoff, which will be directed toward the bay side of the barrier bar.

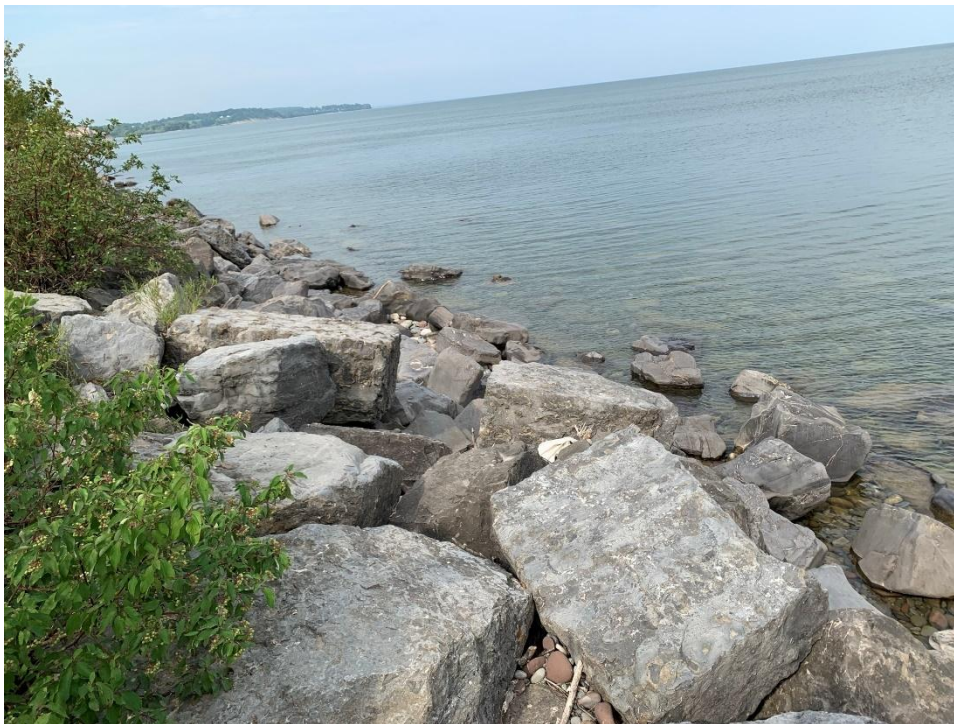


Figure 1 – Existing rock revetment along west barrier bar

2.2 PIER

2.2.1 Structural Inspection

A visual inspection of the pier was performed on December 2, 2020 by David Guetta, PE, and Joshua Repp, PE, from Bergmann to observe the condition of the existing sheet pile wall along the western side of the outlet channel and along the pier.

The sheet pile wall at the pier, which acts to line the exterior of the concrete structure, closely resembled a 3/8" thick PMA profile type sheeting and is in poor condition and is heavily deteriorated along the water

line. The sheeting is completely severed & scoured at the water line at several areas, including the along the eastern and western walls of the pier. The northern most wall of the pier could not be observed due to poor access and unsafe conditions at the time of the inspection but is expected to exhibit similar conditions.

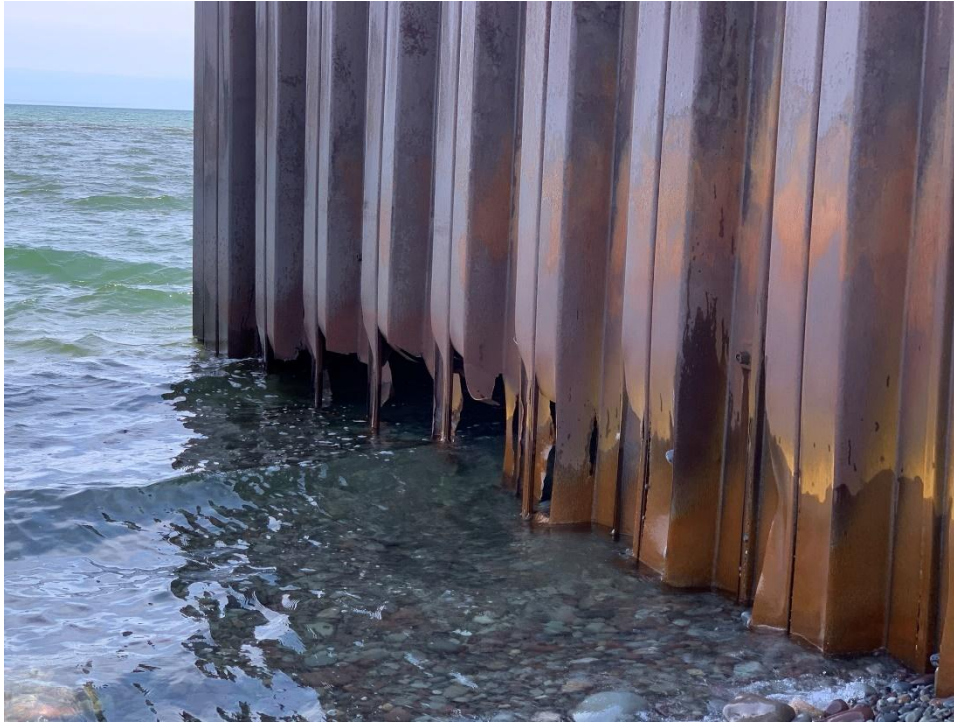


Figure 2 – Condition of sheet pile wall along west side of pier

2.2.2 Coastal Engineering Considerations

The west pier was built in 1971 and provides a harbor-like setting for recreational boating. The west pier also mimics a groin, trapping a portion of the longshore sediment transport. Based on the US Army Corps of Engineers (USACE) 2019 Lake Ontario sediment budget study, approximately 7,600 CY of littoral drift sediment is moving from west to east annually at Port Bay. The annual dredging at the Port Bay inlet varies between 1,000 CY to 3,000 CY annually. The percent of trapped sand by the West Pier varies between 13 - 39 percent of the net littoral drift.

To maintain navigation between the lake and the bay, dredging is inevitable. The West pier assists in pushing the lakebed contours lakeward and preventing sediment accretion inside the Port Bay inlet.

Figure 3 illustrates how the presence of the pier changes lakebed contours. Imagine a sand particle following the 240 ft NAVD 88 contour moving west to east. The particle will move along this contour and then deposit a distance further from the shoreline (roughly 120 feet from the East barrier bar shoreline). Conversely, if the pier were removed, the 240 ft contour would be closer to the natural shoreline (approximately 65 ft from the East Barrier bar shoreline). An example of this phenomenon is shown in Figure 4 at Beaver Creek at Willow Lake Park. Without the shelter of a jetty or groin, the inlet at Beaver Creek has filled in with sediment. An unprotected channel opening becomes a sediment sink and fills in rapidly.

In addition, without the shelter of the west pier, lake waves approaching from the west and northwest will make it difficult and at times unsafe for boats to access the lake and bay. If the west pier was extended out into the lake it that would result in the trapping of more littoral sediment and dredging would require marine plant based equipment.

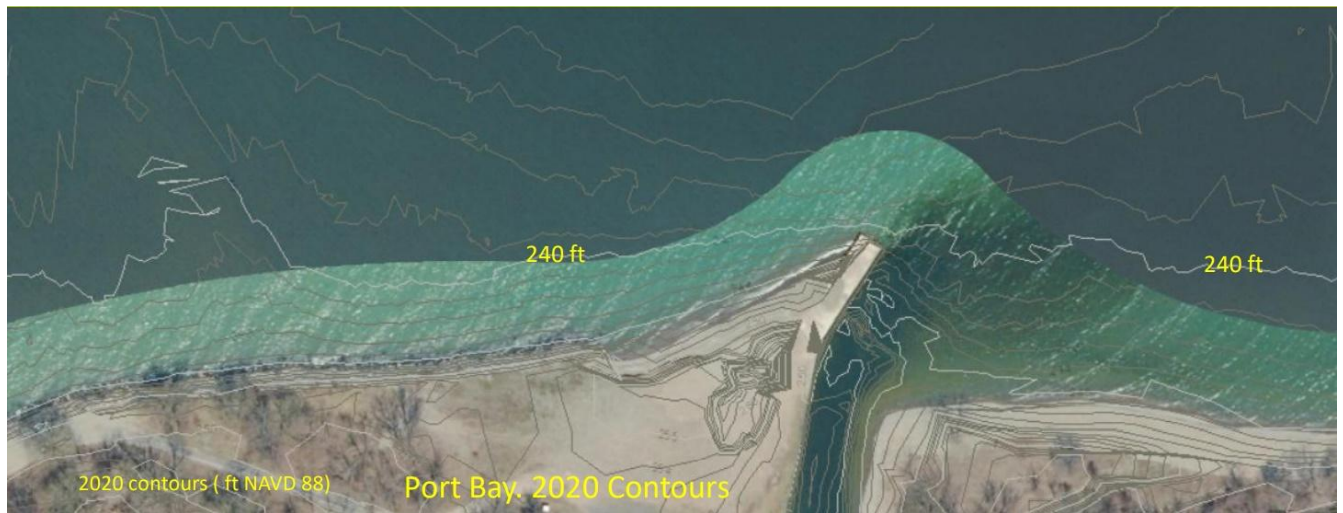


Figure 3 – 2020 Lakebed Contours at West Pier



Figure 4 – Beaver Creek Inlet (filled in with sediment)

The revised concept plan includes keeping the footprint of the west pier as-is. Although the west pier impacts an average of 26 percent of the littoral drift annually, it still allows for safe navigation.

The plan includes encapsulating the west pier in steel sheet pile, adding a new concrete deck and preserving the 3-ft-tall concrete parapet wall (deflector wall). Presently gravel and cobble material is washed up and thrown over the lower elevation portion of the west pier and fills the navigation channel. Based on input from the PBIA during an August 18, 2021 meeting, a temporary (or removable) deflector wall (crest elev. 254 ft & approx. 65 feet in length) is proposed and will extend back towards the shoreline during the navigation season (May to September). By placing the deflector wall at this location, the sediment will now migrate to the head of the west pier and deposit off the northern tip. Land-based equipment will have access at the tip of the west pier for maintenance dredging operations. The removable deflector wall will consist of a pre-cast concrete block system and will be removed prior to the winter season.



Figure 5 – West pier head and existing deflector wall (2 DEC 2020)



Figure 6 – Overwash of cobble and gravel in the navigation channel ~ looking east (2 DEC 2020)

2.2.3 Dredging Considerations

The contractor hired by the PBIA drives his equipment on the existing pier in order dredge the channel adjacent to the pier that cannot be reached from the eastern barrier bar. To maintain the land-based means of dredging currently funded by the PBIA, the proposed pier must be traversable by equipment. For this reason, a rock breakwater/pier was not considered.

Based on input from the PBIA, installing a removable deflector wall along the western edge of the west pier is preferred. The proposed removable deflector wall (64.8 linear feet) would be in place during navigation season and removed prior to winter months. The deflector wall design will be determined during the next project phase and may consist of a removable pre-cast concrete block system or something similar. When the deflector wall is in place, littoral sediments will accrete near the head of the west pier therefore reducing the over washing the west pier significantly. In the winter months, the deflector wall will be removed, and sediment will resume over wash of the west pier and deposit directly in the navigation channel where it can be easily removed during the first significant dredging event at the start of the season (typically the end of March).

Future water levels on Lake Ontario are uncertain. However, it is important to the resiliency of the project to consider both low and high-water conditions and how it impacts dredging at Port Bay. During high water years, the navigation channel at Port Bay will still require routine maintenance but should not see an increase in dredging frequency. However, the volume of sediment in the channel may increase if there has been several major storms in the region. This increase in sediment supply is due to more erosion occurring under high water conditions in the nearshore zone, specifically at unarmored shorelines. Under low water conditions, the navigation channel at Port Bay will fill in more quickly, requiring frequent maintenance dredging.



2.2.4 Proposed Concept

The June 4 submission included an extension of the pier layout to the east. Upon further investigation, it was determined that the changes to the footprint would pose a significant challenge to navigation. Sight lines would be negatively impacted due to the angle of approach and the elevation of the western end of the east barrier bar, which would impact boater safety. In addition, the layout would likely increase the trapping of sediment between the east end of the pier and the shoreline resulting in increased dredging.

The revised concept plan shows the encasement of the pier with new marine-grade sheet pile wall while maintaining the existing pier footprint. The elevation of the platform of the pier will be increased from 250.5 ft (NAVD 88) to 252.5 ft and a temporary deflector wall will be extended (in place during navigation season) to minimize the throw of material into the channel. The length of the new deflector wall is approximately 64.8 ft. The temporary deflector wall will be removed during winter months.

2.3 WEST CHANNEL WALL

2.3.1 Structural Inspection

A visual inspection of the west channel wall was performed on December 2, 2020 by David Guetta, PE, and Joshua Repp, PE, from Bergmann to observe the condition of the existing sheet pile wall. The sheeting located south of the pier structure, for a distance of approximately 145 ft, is 3/8" thick profiled type sheeting and closely resembles the details indicated within the 1971 permit application. The majority of the 145 ft length is in fair condition. However, there is a small length of sheeting, located at the transition point between the south end of the pier and the beginning of the anchored sheet pile retaining wall, that has displaced and rotated significantly towards the outlet channel. No tie-backs were observed where the wall had displaced. The remainder of the wall, where tie-backs were present, appeared plumb.

The remainder of the sheet pile wall, approximately 255 ft in length that continues to line the western side of the outlet channel, consisted of flat web 3/8" thick sheeting and is in fair condition with little deterioration observed along the waterline. The alignment of the sheeting appeared plumb, with no signs of instability or distress. The top of the sheeting was deformed locally, presumably from the initial driving of the sheet piles during original construction. At the very south end of the sheet pile wall the ground behind the wall has eroded away, exposing some of the anchor tie-backs, and has undermined the south end of the concrete slab landing that abuts the wall. A metal pipe railing is present along the top of the sheet pile wall along the majority of the wall length and is loose. The railing can be displaced several inches when exerted upon due to a loose connection to the sheet pile wall. The railing is in otherwise good condition.

The capacity of the anchored wall is developed by concrete deadman anchorages buried beneath the ground, which could not be observed at the time of the inspection. The only element of the tie-back anchor system observable during the inspection was the head of the anchor tie-back protruding through the face of the sheet pile wall. Various anchor head types were encountered along the site, with some consisting of a hooked reinforcing bar bearing against the sheeting face and, in other areas, a rod welded to a small bearing plate. The anchor heads appeared to be in poor to fair condition, with little uniformity in design at each anchor location, calling into question their reliability and quality of construction. Figure 7 and Figure 8 show the condition of the sheet pile wall along the channel.



Figure 7 – Condition of sheet pile wall along channel (03 MAR 2021)



Figure 8 – Condition of sheet pile wall along channel (03 MAR 2021)

2.3.2 Structural Analysis

A structural analysis of the existing anchored wall was performed to determine the wall's ability to safely retain the fill if the outlet channel were dredged to the permitted dredge depth elevation (236.8 ft, NAVD88). Without any as-built record documentation, the wall was analyzed assuming the construction configuration depicted in the approved 1971 USACE permit in Figure 9.

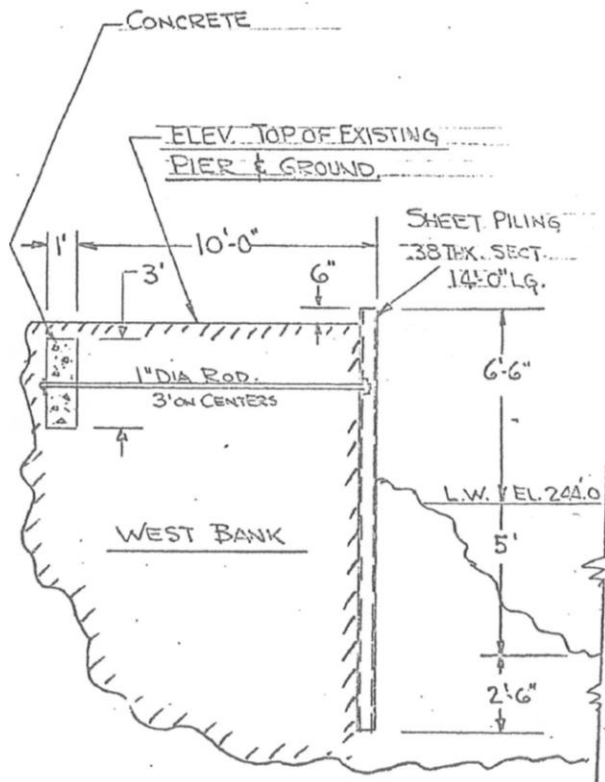


Figure 9 – Existing sheet pile wall cross section (assumed configuration)

The results of the analysis indicated a minimum factor of safety on passive pressures to be 0.50 for the normal load case, which is well below the recommended value of 1.50 for a permanent sheet pile wall. Therefore, despite the historical performance to date, the sheet pile wall is not adequate to retain fill during a completely dredged condition. Complete results of the sheet pile wall analysis will be included in the 60% design package submitted with the permit application.

The Bergmann Team investigated potentially replacing the last 200 linear feet of the west channel wall (southern end and to PBIA signage) with a stone revetment instead of steel sheet pile. Due to the proximity to the navigation channel and channel depths – the revetment would have a wide footprint (greater than 40 feet wide) and result in encroachment in the park area. This would also prevent direct access for public and fisherman to the water's edge. Therefore, the stone revetment was no longer considered as a potential project measure.

2.3.3 Geotechnical Considerations

In October 2020, Earth Dimension, Inc. (EDI) advanced five borings behind the sheet pile wall along the western edge of the channel. The borings were advanced to a depth of approximately 25 ft below the ground surface elevation. Samples were sent to Geotechnics for laboratory testing, including moisture content, Atterberg limits, and grain size distribution.

McMahon & Mann Consulting Engineering and Geology, P.C., (MMCE) completed an analysis considering both cantilevered and anchored sheet pile wall systems. The analysis results show that a cantilevered wall would require an embedment of at least 20 ft and an anchored wall would require an embedment of at least 10 ft. Soil conditions indicate that driving sheet piles may be difficult (i.e., high SPT N-values and gravelly soils). Therefore, MMCE recommends reducing the embedment length as much as possible through the use of an anchored sheet pile wall system. An anchored wall system is also recommended to

limit deflection. MMCE also recommends that the existing stockpile of dredged material adjacent to the wall be moved such that it will not increase the loading on the new wall or influence the capacity of a tieback system.

MMCE's geotechnical memorandum will be included in the 60% design package for the permit submission.

2.3.4 Coastal Considerations

The west channel wall will be encapsulated with new steel sheet pile. During the design phase of the project, wave crest and trough forces acting on a vertical wall will be determined. Wave trough force is important in this case as it has a pulling effect on the vertical wall. This will be further investigated during project design and the results will be included in the 60% design package for the permit submission.

2.3.5 Dredging Considerations

Dredging activities for the western side of the channel are performed by an excavator adjacent to the channel. The vertical sheet pile wall allows the excavator to access the channel. Dredging is primarily confined to the area of the channel north of the Port Bay welcome sign. According to the current dredging permit, dredging spoils have been stockpiled adjacent to the wall.

To access the east bar, the contractor currently uses dredged spoils to construct a land bridge across the channel. He then drives his equipment over the sheet pile wall and across the land bridge in order to reach the east barrier bar (see Figure 10). From the east barrier bar, the contractor can dredge the east side of the channel. The driving of equipment over the sheet pile wall is likely the cause of the existing wall's poor condition in this area.

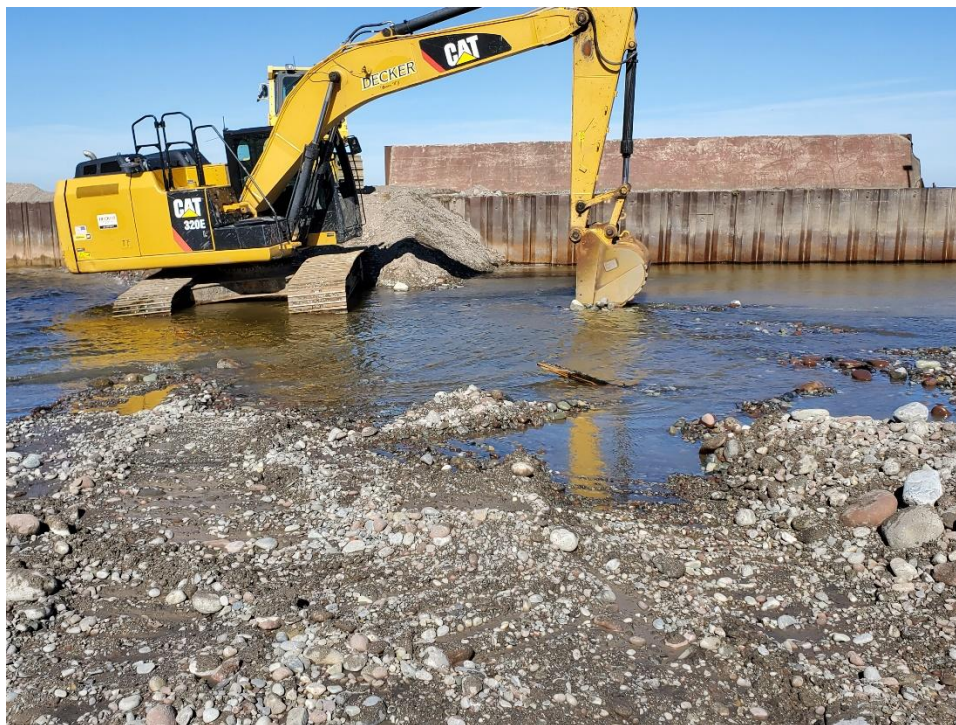


Figure 10 – Dredging photo (03 MAR 2021)

Future water levels on Lake Ontario are uncertain. However, it is important to the resiliency of the project to consider both low and high-water conditions and how it impacts dredging at Port Bay. During high

water years, the navigation channel at Port Bay will still require routine maintenance but should not see an increase in dredging frequency. However, the volume of sediment in the channel may increase if there has been several major storms in the region. This increase in sediment supply is due to more erosion occurring under high water conditions in the nearshore zone, specifically at un-armored shorelines. Under low water conditions, the navigation channel at Port Bay will fill in more quickly, requiring frequent maintenance dredging.

2.3.6 Proposed Concept

The Bergmann team recommends keeping the old sheet pile wall intact and constructing a new anchored sheet pile wall in front of it. The thickness of the marine grade sheet pile wall itself is approximately 12" to 18". A gap of less than 2 ft is required between the new and old wall to provide driving clearance and room for an interior whaler to anchor the new wall. This will allow dredging activities adjacent to this part of the wall to continue as before. The wall will be designed to handle the load of heavy equipment adjacent to the wall.

The June 4 concept plan recommended new sheet pile wall along the entire length of the channel, which would be offset 3 to 5 ft from the existing wall. The team has further refined the offset to 3 ft, to account for the thickness of the sheet piling, waler, and a minimum distance for constructability. The waler will be installed between the old wall and new wall to keep it from being a hazard to navigation. Section 3.0 discusses proposed changes to the east bar that include improvements to access the east bar to end the contractor's current practice of driving over the sheet pile wall.

2.4 "GAP" AREA

The area between the end of the existing rock revetment and pier is known as the "gap" area for the purposes of this project (see Figure 11). The length of shoreline is approximately 250 linear feet. The water level at time of inspection on 10AUG2021 was 245.4 ft (NAVD 88) based on the NOAA gage at Oswego Harbor.



Figure 11 – "Gap" area between rock revetment and pier (looking west) 10AUG2021

2.4.1 Coastal Considerations

The rate of erosion at the “gap” area varies with water levels and storm events. Some of the sediment is carried offshore and a portion is carried to the downdrift shoreline. This reach will be incorporated in the wave numerical model to determine if shoreline armoring is warranted. It is the team's intent to keep this area as natural as possible while also protecting the tie-backs for the new steel sheet pile wall. Presently, the PBIA places dredged material from the navigation channel at this location in accordance with their permit (see Figure 12).

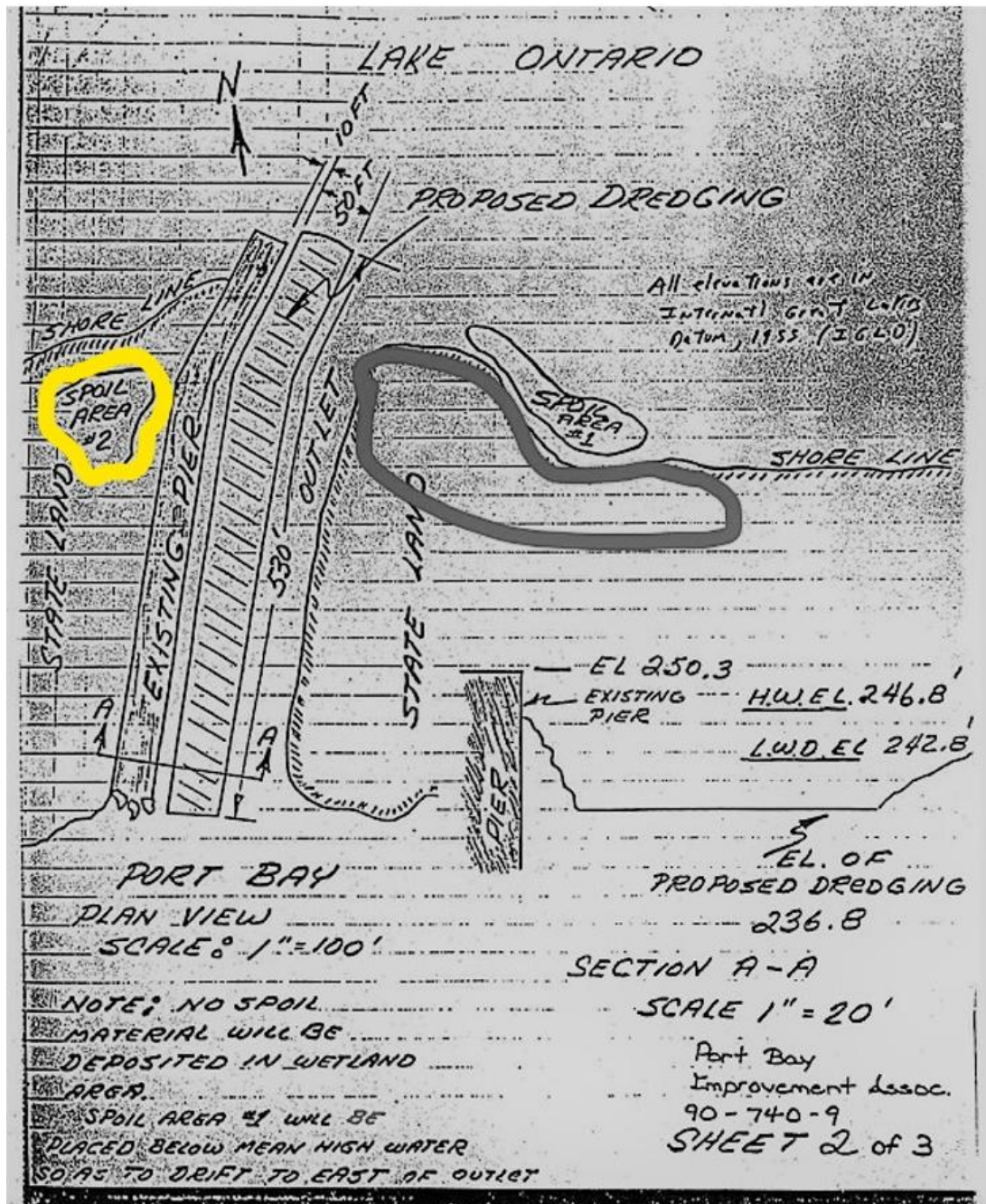


Figure 12 – Current PBIA dredging permit and spoil placement areas (yellow indicates “gap area” location)



2.4.2 Structural Considerations

The proposed channel sheet pile walls are anchored with deadman tie-backs that will be located near the surface of the west barrier bar. Further coastal engineering investigation and modeling will determine whether the “gap” area requires armoring to protect the new tie-back system. The modeling of the shoreline will determine if it is necessary to protect the wall anchorages from erosion that may occur to the gap area. The erosion could undermine the anchorages and negatively impact the stability of the channel wall.

2.4.3 Proposed Concept

The concept plans show the stone revetment being extended to the pier to provide protection to the wall anchorages, approximately 250 linear feet. However, this area will be kept in its natural state if further analysis shows that the anchorages will not be impacted or if an alternate scheme for protecting the anchorages is feasible. The concept plans show the worst-case scenario for impacts in this area. The team's intent is to keep this area natural if possible.

2.4.4 Additional Considerations for Design and Modeling

As a result of preliminary modeling and analysis work, the team has identified that further investigation in this area is required. These items include:

- The rock revetment in the “gap” area and the deflector will attenuate wave energy at the west bar. Modeling will clarify the impacts of each measure to determine which feature, if not both, is best for long-term project resiliency.
- At the PBIA's request, a temporary length of deflector wall will be considered to change the way in which sediment is deposited into the channel during navigation season.

2.5 BAY SIDE OF WEST BARRIER BAR

2.5.1 Bay Side of West Bar Access Road

The bay side of the west bar access road is subject to erosive forces when water levels are high and wave events overtop the western bar. Envirolok bags are currently being investigated as a potential stabilizing solution. Envirolock bags provide stability and allow vegetation growth, which eventually masks the bags and takes on the appearance of a vegetated slope. An excerpt from the Envirolok brochure is shown in Figure 13. Further investigation is needed to determine the most appropriate green solution for this area. The desire is to protect the bayside shoreline and provide additional opportunities for aquatic habitat.



Figure 13 – Excerpt from Envirolok brochure

2.5.2 South End of Channel Sheet Pile Wall

At the terminus of the channel sheet pile wall, a stone revetment (approximately 75 feet in length) is proposed. The stone revetment will prevent scour from occurring behind the newly encased steel sheet pile structure. The stone revetment reach will extend from the terminus of the steel sheet pile for 75 +/- feet and tie-into the existing grade. Existing erosion in this area is shown in Figure 14.

In addition, there is a group of trees along the bayside on the west barrier bar with roots exposed (Figure 15). During high water, this tree area/ shoreline is inundated and threatens the stability of the trees. The Bergmann team will investigate bank stabilization measures to ensure tree and bank stability in the next phase of the project.



Figure 14 – Area of erosion south of existing sheet pile wall (03 MAY 2021)



Figure 15 – Area of exposed tree roots ~ west bar bayside (03 MAY 2021)



3.0 Eastern Barrier Bar

3.1 BACKGROUND

To properly evaluate the existing conditions of the eastern barrier bar historic shorelines and a preliminary wave analysis were completed.

3.1.1 Historic Shorelines

It is common coastal engineering practice to geo-reference historic aerial photos of a shoreline to estimate annual shoreline erosion rates. For Port Bay, orthophotos from 2002, 2005, 2010, 2015, 2018 and 2021 were readily available for digitizing the east barrier bar shoreline. It is critical to shift the shoreline to the same datum (i.e., +2 ft LWD or 245.3 ft IGLD 1985) for all years – to make an accurate comparison of bar morphology change over a given time period. Low Water Datum (LWD) is equal to 243.3 ft IGLD 1985 for Lake Ontario. The dates for each aerial photograph were available – the average daily water level for that date was taken from the nearby Oswego Harbor NOAA station. All shorelines were shown as +2ft LWD or 245.3 ft IGLD 1985 and assuming a nearshore slope of 10H:1V. Below is a list of historic shoreline figures developed for the East Bar and key observations:

- Figure 16 – Port Bay East Barrier Bar Historic Shoreline Change 2002 – 2021

As shown in the figure, the west end of the east bar has remained relatively stable over the past 19 years. This is due to annual placement of dredged material at this location (called spoil area #1 from Figure 12). However, as you move east along the east bar, there is a consistent recession of the lakeside shoreline until it meets the stone revetment at the private property.
- Figure 17 – Port Bay East Barrier Bar Shoreline Change 2010 -2002 (8 years)

The net loss and gain is presented in this figure. Over the 8-year time period there has been loss of 0.42 acres. This shoreline retreat pattern indicates a high wave energy environment and warrants a breakwater to stabilize the shoreline.
- Figure 18 – Port Bay East Barrier Bar Shoreline Change 2021 -2010 (11 years)

Historic high lake levels occurred in 2017 and 2019 on Lake Ontario, which led to increased erosion and periodic breaching of the east bar. In response, the east bar was nourished with sand/gravel and cobble in 2014, 2016, 2018, 2019 and 2020. Due to the placement of additional beach fill – there is a net gain at the east bar of 0.53 acres after the 11-year period. This can be seen in the figure by the fanning of material into the bay.

Without this emergency nourishment, and under natural coastal conditions the east bar would show a net loss instead of a net gain. The east bar is sediment starved due to the overall depletion of littoral sediment along the southern shoreline of Lake Ontario over the past 100 years.

- Figure 19 – Port Bay East Barrier Bar Shoreline Change 1965 & 2021 (56 year)

A 1965 aerial image of Port Bay was geo-referenced in ArcGIS. An overlay of the proposed headland breakwaters, the West Pier, and the 2021 shoreline of the East Bar were also added as layers to the figure. The East Bar has retreated approximately 250 ft (center of the bar) over the past 56 years.



- Figure 20 – Port Bay East Barrier Bar Estimated Future Shoreline Change 10 YRS, 20 YRS and 30 YRS

Future shorelines were estimated/ projected for the next 10 yr, 20 yr, and 30 yrs based on the results from Figure 18. The future shoreline scenarios assume no change in shoreline protection and periodic nourishment of the bar continues as in the recent past. As presented in Figure 20, the east bar will continue to migrate back into Port Bay and will eventually breach. With a breach in the east bar, lake waves will enter the Bay increasing erosion and wave energy in the bay. Increased wave energy will negatively impact the establishment of submerged aquatic vegetation (SAV) and fish spawning areas. This future shoreline retreat pattern and predicted breaching of the east bar indicates a high wave energy environment. In turn, a rubblemound breakwater is warranted to stabilize the shoreline.



Figure 16 – Port Bay East Barrier Bar Historic Shoreline Change 2002 – 2021

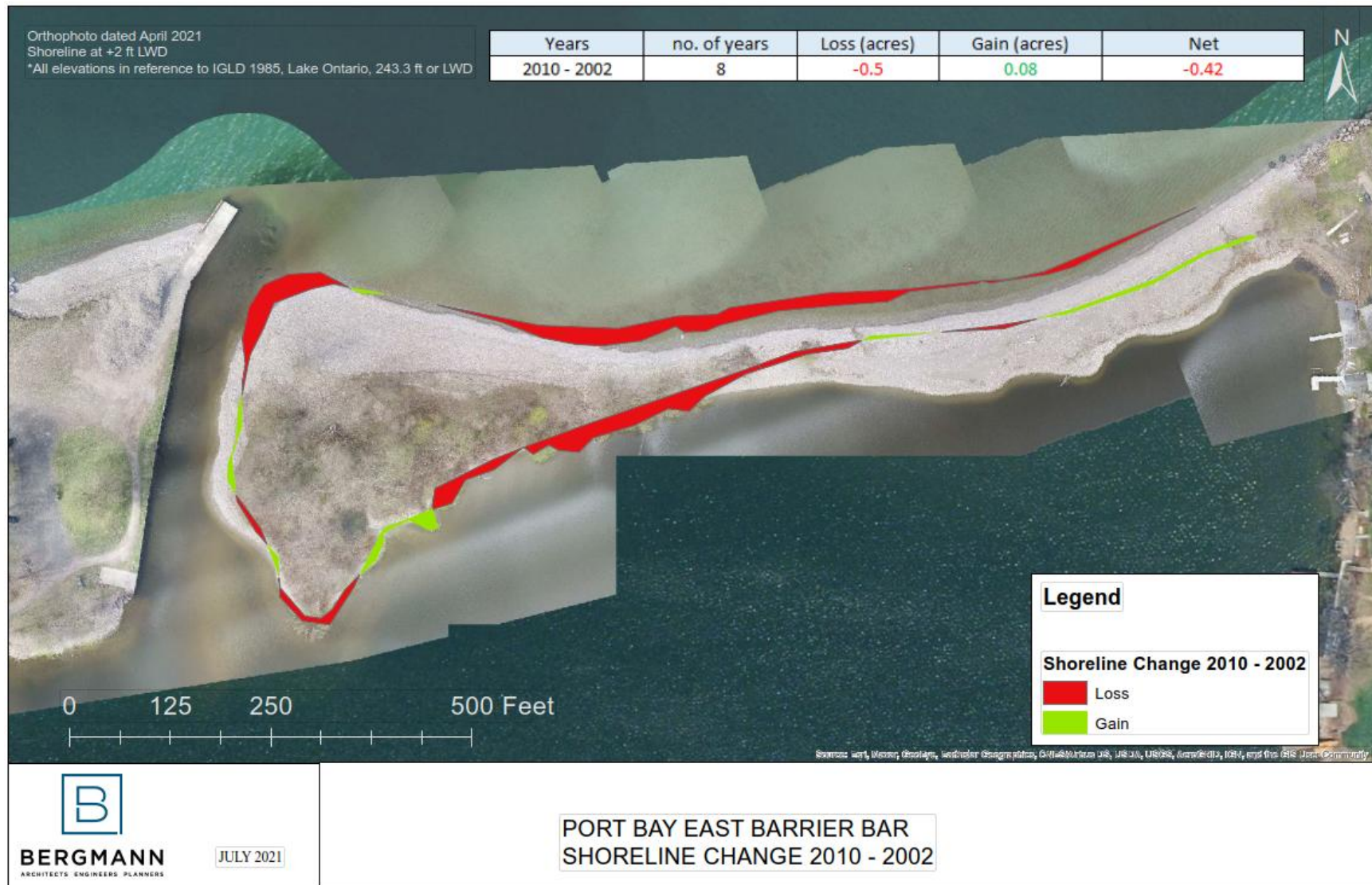


Figure 17 – Port Bay East Barrier Bar Shoreline Change 2010 -2002 (8 years)

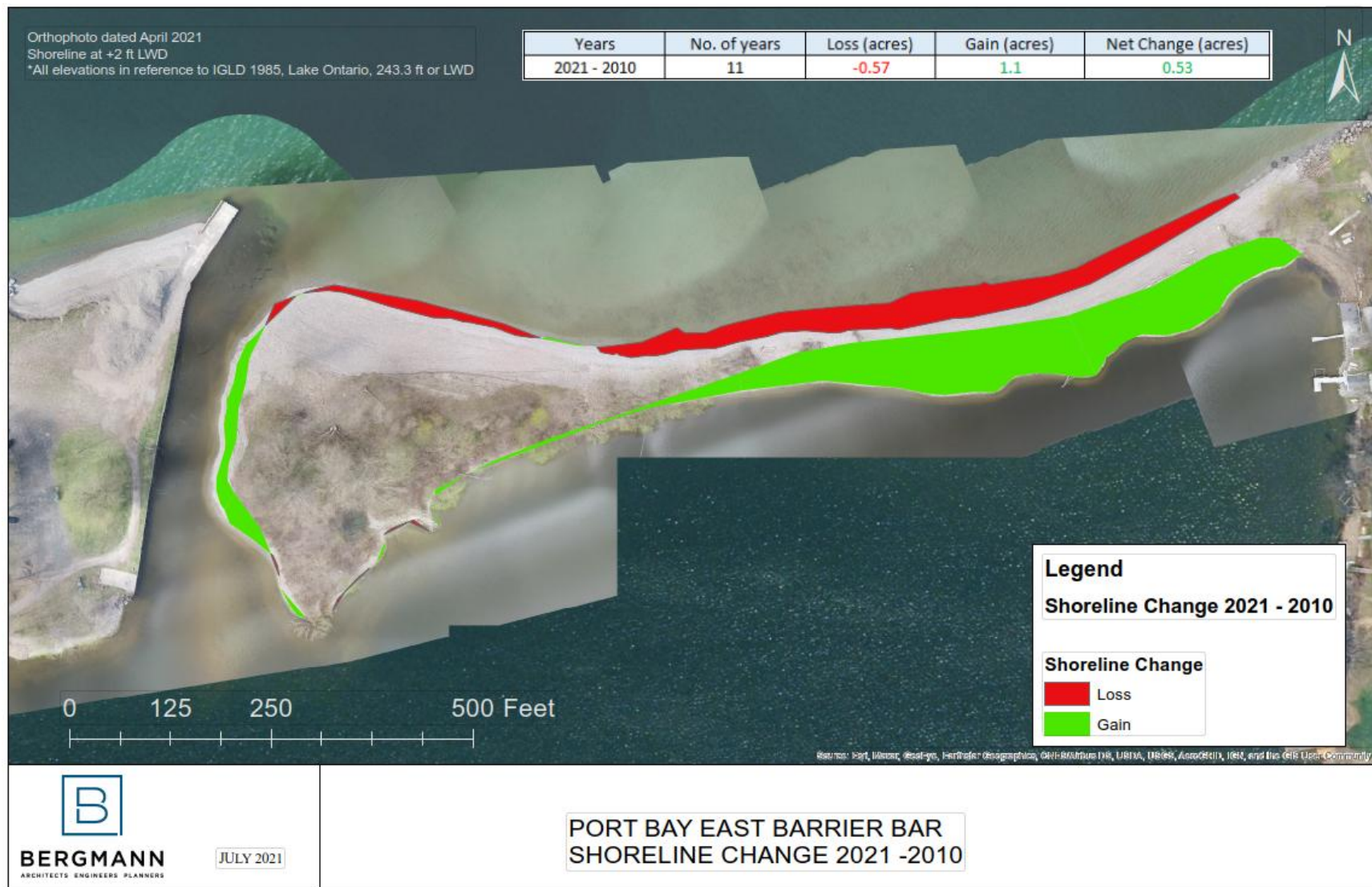


Figure 18 – Port Bay East Barrier Bar Shoreline Change 2021 -2010 (11 years)

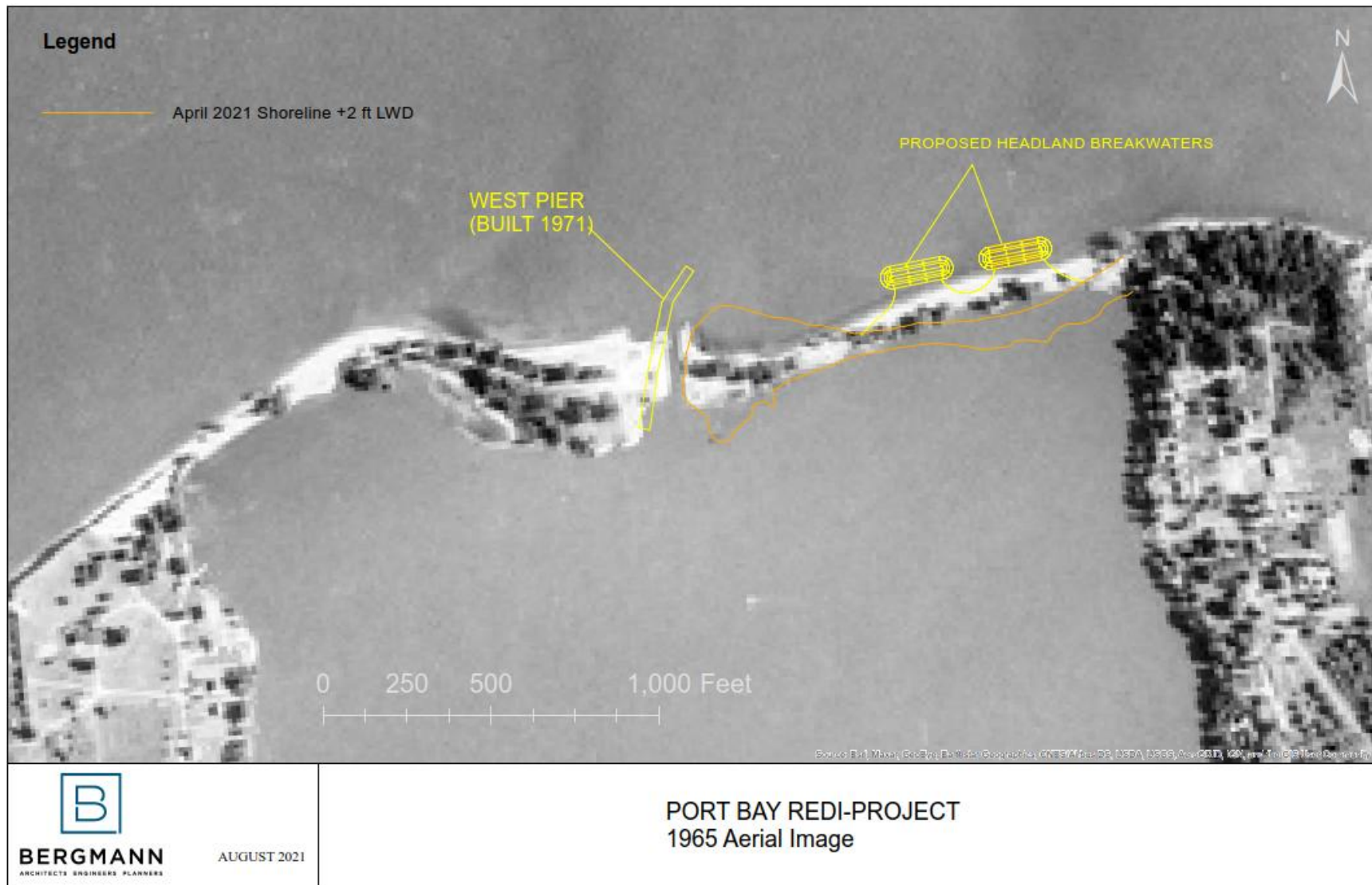


Figure 19 – Port Bay East Barrier Bar Shoreline Change 1965 & 2021

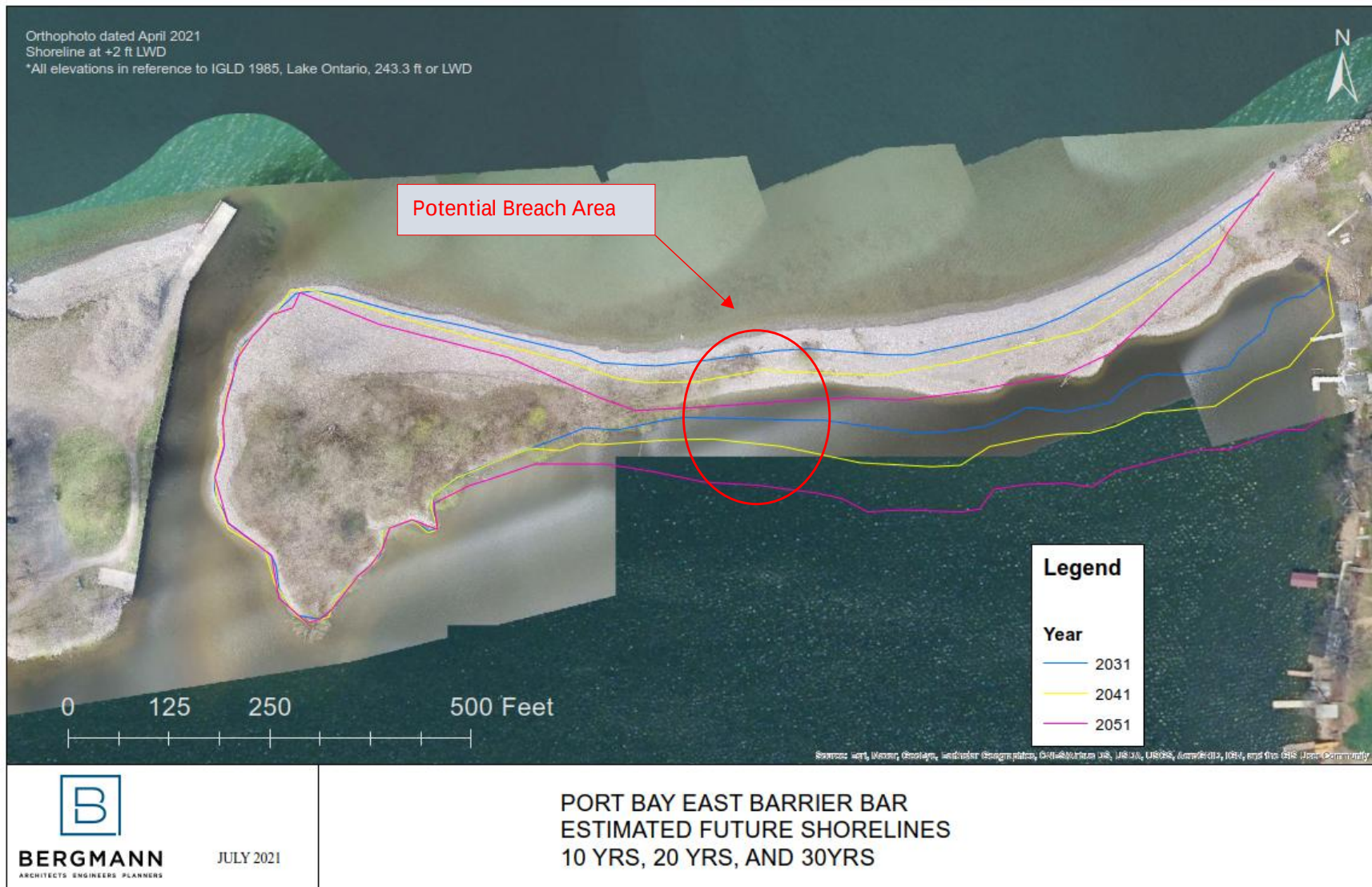


Figure 20 – Port Bay East Barrier Bar Estimated Future Shoreline Change 10 YRS, 20 YRS and 30 YRS

3.1.2 Sediment Budget

The US Army Corps of Engineers Buffalo District completed a sediment budget for the southern shoreline of Lake Ontario in 2019. Figure 21 presents the sediment budget cells near the Port Bay Barrier Bar. The purple arrow denotes the net littoral drift moving from east to west along the nearshore. The net littoral drift is estimated at 7,600 CY/ year moving east to west.

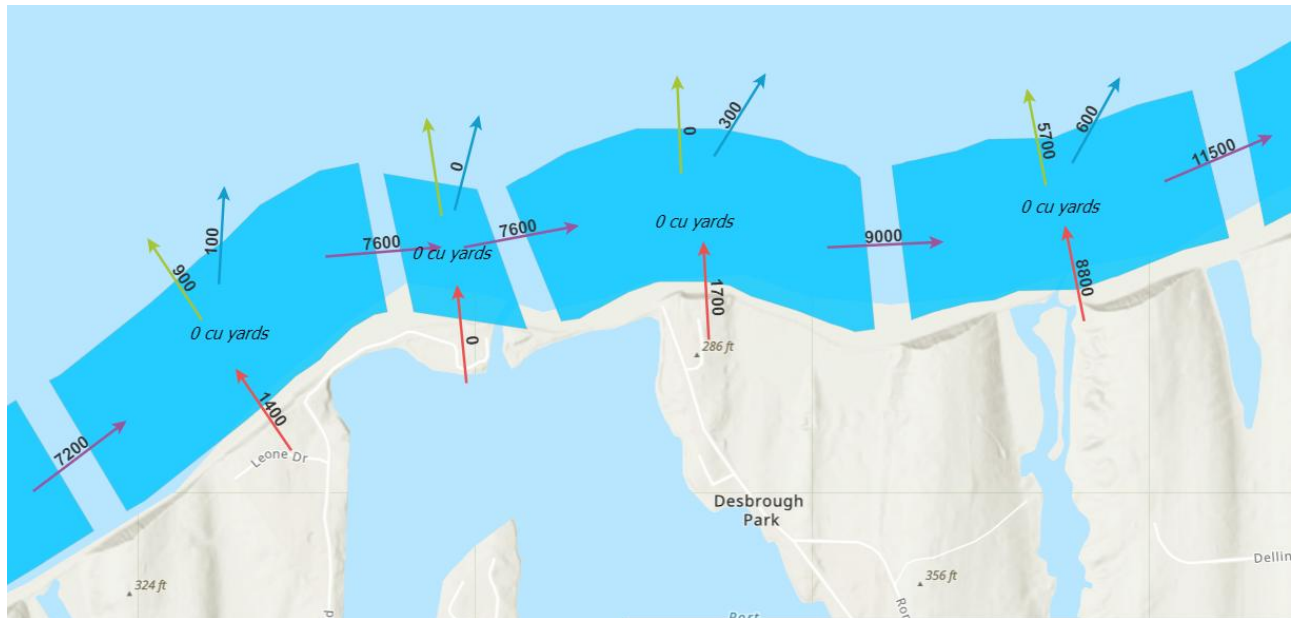


Figure 21 – Recent conditions (sediment budget) at Port Bay (CY per year)

The west pier, built in 1971, mimics a groin structure. A portion of the longshore sediment transport is trapped by the west pier and deposits around the tip of the pier, in the navigation channel and washes over washes the pier. Annual dredging at Port Bay inlet varies between 1,000 CY to 3,000 CY annually per conversation with the Wayne County Soil and Water Conservation District. Exact dredging records were not available. Based on the annual dredging volumes, the west pier traps between 13 - 39 percent of the net littoral drift or an average of 26 percent annually. Because the west pier is critical in maintaining a harbor-like setting for boats to access the lake and the bay, our revised concept plan includes keeping the footprint of the west pier as-is.

3.1.3 Design Water Levels

The annual maximum water levels for Port Bay were obtained by using the NOAA gage at Oswego Harbor, NY (see Table 1).

Table 1 – Design lake levels (Oswego Harbor, NOAA Station 9052030)

RI (yr)	(ft) IGLD 1985
2	246.9
5	247.3
10	247.7
20	248.3



Ordinary High-Water Mark (OHWM) is 247.3 ft IGLD 1985. The long-term average water level for Oswego Harbor is 245.3 ft IGLD 1985. The conversion from IGLD 85 to NAVD 88, is 0.09 ft.

3.1.4 Bathymetry

The bathymetry at the eastern bar consists of shore parallel contours with no significant complexity from shoals or reefs. The offshore slope of the lake bottom varies between 100H:1V and 50H:1V from El. 241 ft (NAVD 88) and lakeward. The nearshore slope varies between 20H:1V and 8H:1V from El. 241 ft to 247 ft. Due to the presence of shore parallel contours without complex reefs, it is reasonable to use Goda's algorithm for estimating incident wave heights at the east bar in an excel spreadsheet. Goda's algorithm results will be used to check the accuracy of the wave model.

3.1.5 Deep Water Waves

Deep water waves were obtained from the nearest Wave Information Study (WIS) station 91054. The WIS station is approximately 4 miles offshore from the project area (Figure 22). Deep water waves were divided into three wave angle classes 1-3 relative to the shoreline normal. Figure 23 presents the wave angle bands and shoreline orientation. For this project class angle 2 and 3 (west to north) waves are critical for design of shoreline protection. Table 2 presents the deep-water wave heights, periods and wave direction.

The yellow highlighted peak wave period for class angle 3 denotes long wave periods between 10 and 11.2 seconds. Peak wave period is critical in design of shore protection. Longer wave periods result in greater wave runup on a beach and more overtopping of structures.

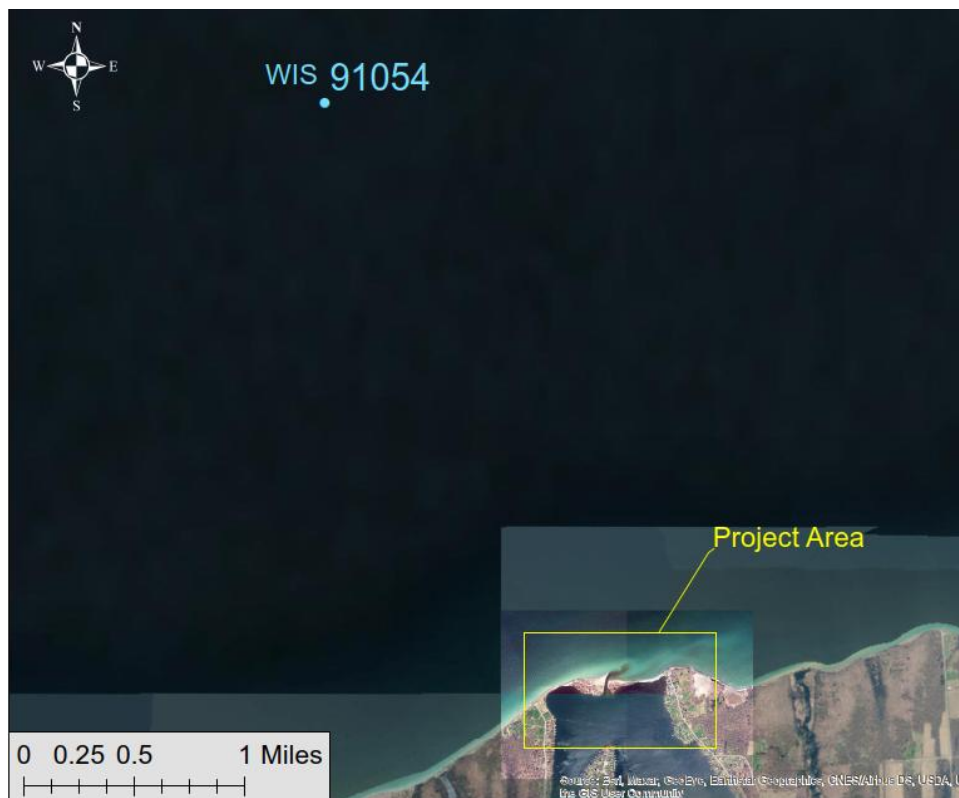


Figure 22 – WIS station location map

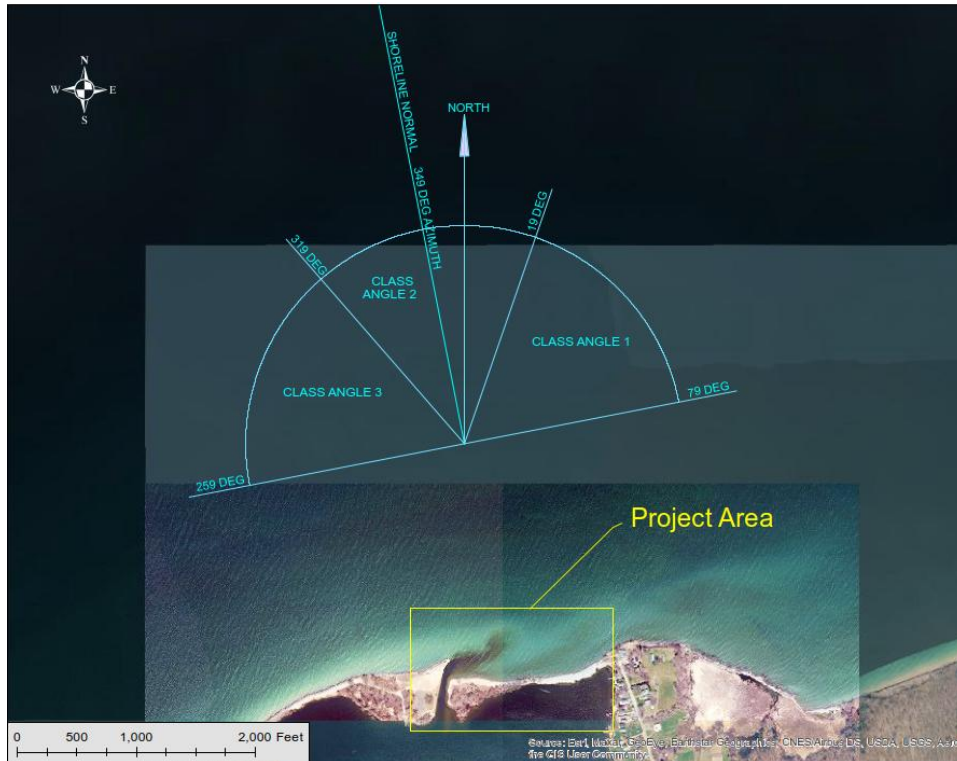


Figure 23 – Deepwater wave class angle bands for Port Bay REDI-project

Table 2 – Deep water waves (WIS 91054)

Determination of Deepwater Waves			
Wave Class Angle 3			
Storm RI	Hmo (ft)	Peak Period, Tp (s)	Mean Wave Direction (DEG)
2	16.6	10.1	319
5	20.1	11.1	319
10	20.9	11.2	319
20	23.3	11.2	319
Wave Class Angle 2			
Storm RI	Hmo (ft)	Peak Period, Tp (s)	Mean Wave Direction (DEG)
2	9.3	7	349
5	11.1	7.5	349
10	13	8.3	349
20	17	9.2	349
Wave Class Angle 1			
Storm RI	Hmo (ft)	Peak Period, Tp (s)	Mean Wave Direction (DEG)
2	8	6.7	19
5	9.8	7.3	19
10	11.5	7.8	19
20	17.3	9.2	19
	Long Wave Periods (10-11 s) from class angle 3 - wave period is always constant.		



3.1.6 Determination of Incident Waves

Goda's algorithm was used for estimating incident waves at the east barrier bar. A detailed discussion of incident waves will be provided in the 60% design package submitted with the permit. In general, the incident waves are breaking for the selected wave cases. Waves propagating from the north west have the long wave periods during storm events, varying between 10 and 11.2 seconds.

Armor stone sizing for the headland breakwaters was determined using the Hudson Equation:
Average armor stone diameter = 3.4 feet (S.G.=2.64)

3.1.7 Summary

The incident waves at the East Bar are breaking for the wave cases presented in Table 2. Also, waves propagating from the west and north west (class angle 3) have long peak wave periods: 10 to 11.2 seconds. Long wave periods result in greater wave runup on a beach and more overtopping of shoreline structures. Goda's algorithm was used as a preliminary estimate of the incident waves at the east bar.

A numerical wave, hydrodynamic and sediment transport model will be completed in the next project phase to determine existing conditions, incident waves, orbital velocities, and evaluate the project impact on littoral drift.

3.2 NATURE-BASED BERM

The June 4 submission included a nature-based berm along the east barrier bar. The historic shoreline analysis that has since been completed shows that the wave energy from Lake Ontario will cause shoreline migration to continue into the bay unless additional measures are put in place to reduce wave energy and provide additional protection for the bar.

The revised plan includes a nature-based berm along the east barrier bar behind two headland breakwaters. The berm will have an interlocking tree stump and log core with gravel and cobble fill material planted with vegetation, including willow stakes on the lakeside and bayside. The slope of the berm on the lakeside and bay side have been revised to 3H:1V. The crest width of the berm is 12 ft and the crest elevation is 252 ft NAVD 88.

3.3 HEADLAND BREAKWATERS & BEACH FILL

"In contrast to bulkheads and revetments, breakwaters are placed out in the water, rather than directly on shore, to intercept energy of approaching waves and form a low-energy-shadow zone on their landward side" (USACE, 1980). The project includes construction of two headland breakwaters (each 150 ft in length and 70 ft gap) parallel to the shoreline orientation at Port Bay to reduce wave energy at the shore. The crest elevation of the breakwater is 251 ft NAVD 88 and three armor stones wide (10.2 ft). The concept plans show the project footprint and typical breakwater cross sections.

The headland breakwaters are placed such that they do not "stick out" farther into the lake than the head of the West Pier. Beach fill will be placed between the headland breakwaters and the east bar to a planform equilibrium. The beach fill will be similar in grain size to what is already native to the east bar. Beach fill is critical so the headlands do not trap littoral drift and cause negative downdrift impacts. A numerical wave model will be set up and run (the models include CMS-Wave and CMS-FLOW) to show the headlands will not negatively impact littoral drift.

Headland breakwaters are designed to mimic natural rocky headlands with sandy beaches. The beach behind the headlands is pre-filled to a fully equilibrated planform shape. Headland breakwaters and pre-filled beaches have been successfully used at Port Union, Ontario, Canada and Braddock Bay (both located on Lake Ontario) with minimal downdrift impacts.

3.4 EAST BAR ACCESS

The June 4 submission showed impacts to Lake Ontario to provide the dredging contractor access to the east bar during both high water and low water events. The extent of the impact and rock revetment has been refined for the revised submission. Access to the east bar will be provided by the crest of the nature-based berm (12-ft wide). The access will begin at the east end of the property and traverse across to the west end of the east bar and then tie into existing grade. Figure 24 illustrates that the additional fill and rock revetment is necessary to provide access to the east bar without impacting the adjacent private property.

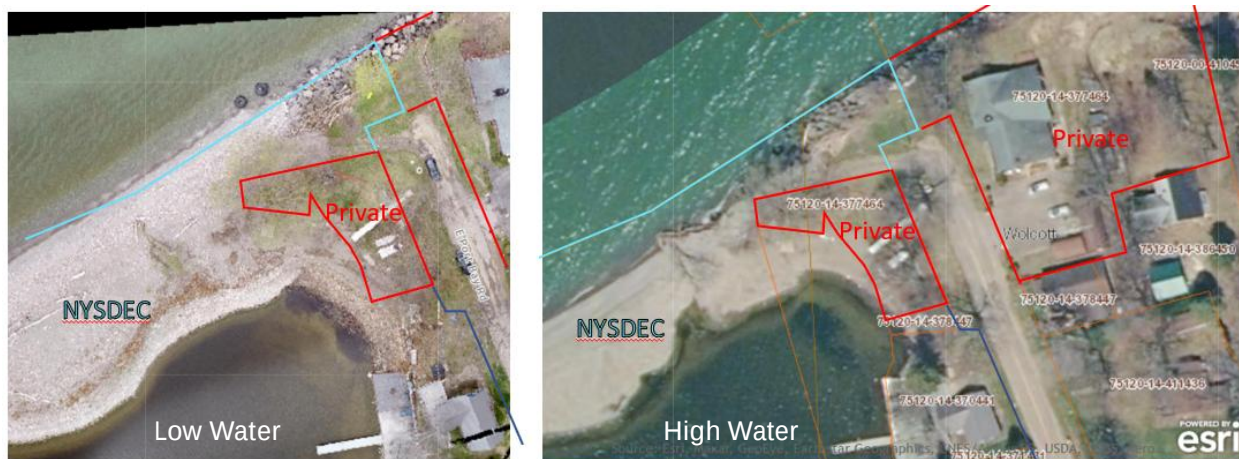


Figure 24 – Low water and high water east bar access comparison

During low water, equipment may access the east bar easily. During high water, it is impossible to access the east bar without driving through private property. Access during both high and low water conditions is necessary for dredging purposes. With permanent access, tracked equipment can access the east bar and perform channel dredging operations during a full range of lake level conditions. The equipment will no longer be driven over the channel sheet pile wall and over a land bridge created through the channel to access the east bar.

The proposed east access will also ensure that construction vehicles entering the bar area will be directed away from any potential turtle habitat on the bay side of the bar.

3.5 EAST BAR HOLISTIC SHORELINE PROTECTION APPROACH

The Bergmann team created a solution that incorporates both nature-based, 'soft' and structural 'hard' shoreline protection measures. The East barrier bar measures, including, beach nourishment (from dredge spoils) placed at the west end of the east bar, nature-based berm, stone revetment, access, headland breakwaters and beach fill all work together to provide holistic shoreline protection solution.

To maintain the shoreline from erosion at the west end of the east bar – continued placement of dredged spoils is recommended. Access to the east bar will be provided during high and low water events via a nature-based berm. The berm will allow dredging equipment to access the navigation channel for maintenance dredging as well as provide an additional layer of shoreline erosion protection for wave runup. A portion of the nature-based berm will have a stone revetment along the lakeside, specifically east of the headland breakwaters. A stone revetment is warranted here due to the direct exposure to lake waves and ice. To prevent migration and breaching of the East Bar a headland breakwater system, including two headland breakwaters and beach fill (between the breakwaters and barrier bar) is recommended. The headland breakwaters are positioned roughly 400 ft east of the navigation channel



to avoid trapping of littoral sediments at the navigation channel from the east. The headland breakwaters will dissipate wave energy approaching the east bar – making the east barrier bar more resilient to storms and fluctuating water levels.

4.0 Maintenance and Monitoring

An adaptive management plan for the west and east barrier bars will be developed by the Wayne County Soil & Water Conservation District and DEC that considers both low and high water extremes on Lake Ontario.

Key elements to be included in the plan are:

- Healthy Port Futures will assist with monitoring as long as the placement of the wedge of dredged material along the east bar continues (natural wave action will reintroduce the dredged material into the littoral zone).
- The west end of the east barrier must continue to be nourished with dredged spoils whenever dredging operations are completed. Nourishment must be provided from the area adjacent to the channel to the tie-in of the headland beach area with the barrier bar. Excess material can be placed as a wedge and reintroduced into the littoral zone.
- Monitor the nature-based berm and used dredge spoil material to replenish eroded areas to maintain east bar access when dredging operations are performed.
- Spoils from dredging activities accessed from the west bar will no longer be stockpiled in close proximity to the sheet pile wall. Designated areas for spoils will be identified along the western bar.

5.0 Anticipated Construction Measures

5.1 SHEET PILE WALL AND PIER

New sheet pile can be driven in wet conditions. A turbidity curtain will be used to retain fine grained material that may be disturbed during the driving process. The wall will be backfilled with on-site material along the channel and concrete at the pier. Sediment filter logs will be placed as needed near excavation areas for the anchor blocks and tie-backs.

5.2 ROCK REVETMENT – “GAP” AND EAST BAR ACCESS

The rock revetment toe berm will be keyed into the lakebed at least 1 ft. Excavated material will be side-casted into the lake and into the littoral system. Quarry stone, such as limestone, will be used for the rock revetment.

5.3 HEADLAND BREAKWATERS

A staging area is required for the construction of the headland breakwaters and stock piling of quarry stone. The staging area would be at the west pier/bar area and roughly 1.0 acre. It is recommended that the headland breakwaters be built first, before the repair of the west pier and west wall. The breakwaters will be built by land-based construction methods and may include building a temporary causeway across the Port Bay inlet for truck access (see Figure 25). Breakwater construction will be coordinated with PBIA in order to minimize temporary navigation impacts.

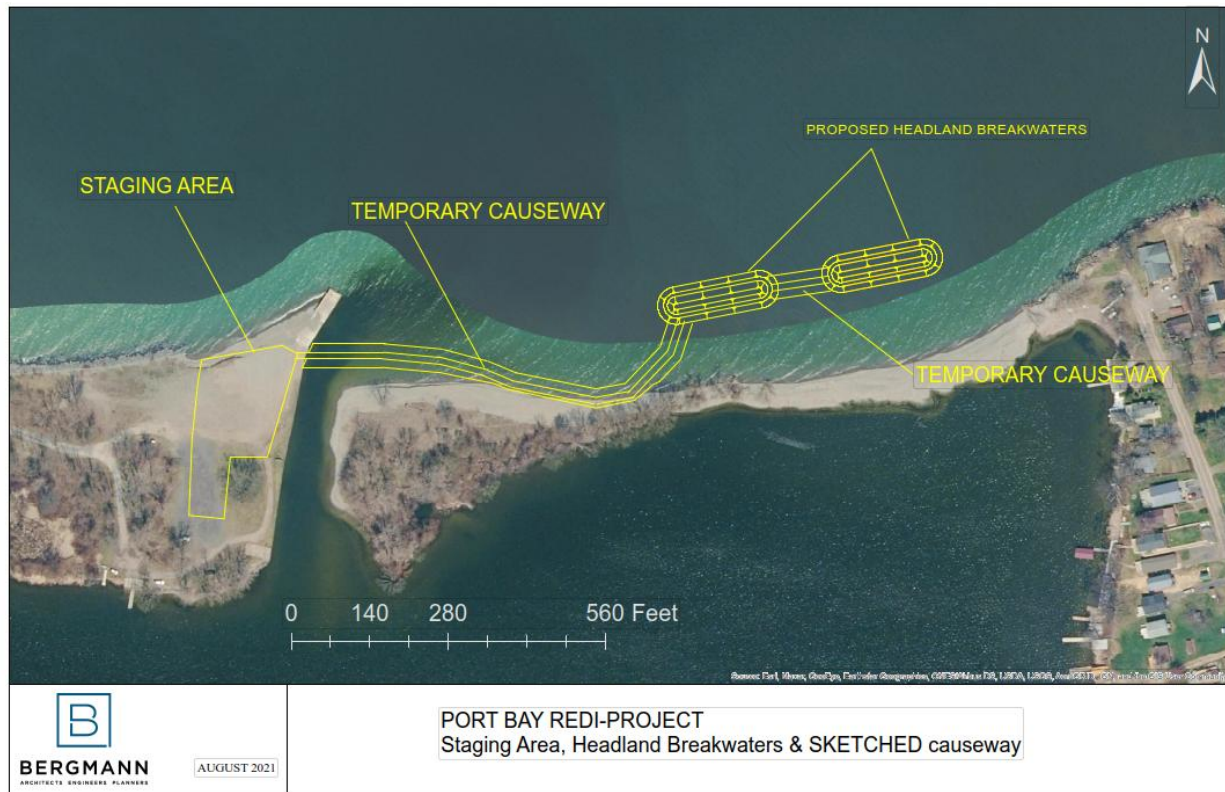


Figure 25 – Staging area, headland breakwaters and proposed causeway

6.0 Conclusions

Based on results of the historic shoreline change and the preliminary wave analysis, a reduction in wave energy at the east barrier bar is warranted to prevent breaching & further migration of the east bar into the Bay. The Bergmann Team is recommending a holistic approach of both soft and hard shoreline management measures to protect the east bar while maintaining navigation at Port Bay. The measures are inter-connected and intended to work together to achieve all project goals. Measures are listed below:

1. Dredging & bypassing: Continue maintenance dredging of navigation channel and placement of sediment on shore at the west end of the east bar (soft measure) Spoil area #2. Placement of material here has proven to stabilize the west end of the east bar with minimal shoreline erosion. Also, this will placement of material will protect the shore face that is not protected by the headland breakwaters.
2. Nature-based berm: consisting of cobble, gravel, root wads and tree logs along the eat barrier bar (soft measure). This soft structure will help reduce wave runoff and block debris from depositing in the bay.
3. Rock revetment at east access: revetment will protect access and reduce wave runoff (hard measure).
4. Rock revetment at “gap” area: hard structure will prevent further erosion at Gap area.
5. Headland breakwaters (hard) & beach fill (soft): A breakwater system, such as headland breakwaters & beach fill is warranted to ensure a 30-year project life. The proposed measure will stabilize the East bar from further erosion. Headland Breakwaters and pre-filled beaches have been successfully used at Port Union, Ontario, Canada and Braddock Bay (both located on Lake



Ontario) with minimal downdrift impacts. Refer to the Shabica et al 2010 for more background on headland breakwaters and engineered beaches.

6. West pier & west channel wall: this “hard” measure includes encapsulating the west pier and channel wall with new steel sheet pile and adding a concrete cap. The footprint of the pier will not change. Land based dredging may access the West Pier as well. The West pier is essential to navigation, as it creates a harbor like setting for boats to access the lake and bay safely.

7.0 References

Shabica, Charles, Mohr, Nagel, 2010, Effects of Structurally engineered beaches on coastal processes and shores of the Great Lakes (included in Appendix A).

US Army Corps of Engineers, 1976 May 4, Guidance for the Use of WES Technical Report H-76-1; Design Wave Information for the Great Lakes

US Army Corps of Engineers, 1981 Low-Cost Shore Protection: A Property Owner's Guide

US Army Corps of Engineers, 2019, Sediment Budget for Lake Ontario. [Sediment Budgets for Lake Erie and Lake Ontario \(arcgis.com\)](#)



Appendix A

Effects of structurally-engineered beaches on coastal processes and shores of the Great Lakes

By

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ABSTRACT

Impacts of coastal structures on America's shores and beaches represent a scientific, economic and engineering challenge to assure that our beaches are protected and sustained. Coastal professionals go to great lengths to properly design, build, monitor, and maintain engineered beaches. However, the beach has a powerful and visceral connection to human nature that makes us all personally and emotionally sensitive to possible threats, proposed changes, and, ultimately, loss of this important component of human well-being. Inadvertent sand starvation and loss of beaches downdrift of 19th- and 20th-century harbor structures has left many people with a natural aversion to "engineered structures." Despite that, a common solution to eroding beaches downdrift of harbors was construction of groins and groin fields that functioned like smaller versions of the harbor breakwaters trapping sand on their updrift sides. Sand bypassing and nourishment of downdrift beaches, introduced in most cases more than 50 years after harbor construction, was generally "too little and too late" to keep groin fields filled with sand.

Toward the end of the 20th century, coastal scientists and engineers, recognizing the reduced effectiveness of groins on sediment-starved coasts, began designing and constructing nearshore stone breakwaters and headlands that are better able to hold sand. These projects were typically filled (premitigated)

with sand to minimize downdrift problems. Breakwater projects designed to protect beaches in the Great Lakes, including the first designed by the U.S. Army Corps of Engineers are reviewed, and several are examined in detail. These include public and private beaches on the south shore of Lake Erie, the north shore of Lake Ontario near Toronto, and the west shore of Lake Michigan north of Chicago. The largest is a 55-breakwater system designed to protect the state park at Presque Isle in Lake Erie (58 were originally designed and approved).

Monitoring and examination of historic air photos has shown that attached and detached breakwaters and armored headlands, if built well within the surf zone, have minimal impacts on downdrift beaches and shores. Exceptions are temporary interruption of the littoral drift when structures are built without being adequately filled (nourished) with sand, or interruptions to planned nourishment due to lack of funding. In Illinois, state regulators adopted a requirement that any structure that may trap sand be pre-mitigated with the anticipated sand fill quantity plus a 20% overfill of new sand to assure no negative impact.

Importantly, access to Google Earth on the Internet has allowed citizens and scientists to view and monitor the coast in a historical context, unbiased by a lack or misunderstanding of scale or perceived bias of coastal "experts."

The shores of the Great Lakes are fertile grounds for innovation in coastal design and engineering. The variety of shorelines left by the glaciers is grist for the creative mill of coastal planners, scientists and engineers. The Great Lakes, in addition to being the largest body of freshwater in North America, has more than 9,400 miles of shoreline. The shores include rocky headlands and natural pocket beaches primarily in the upper Great Lakes (notably Lake Superior and the northern ends of Lake Michigan and Lake Huron), and

ADDITIONAL KEYWORDS:

Breakwaters, headlands, pocket beach, beach erosion, coastal surveys, groins, sand, littoral drift.

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eroding glacial deposits of cohesive or sandy material with narrow sand beaches in the lower Great Lakes (Pope *et al.* 1999). Although most of the Great Lakes

coasts are still rural, intense urbanization has developed along the shores adjacent to river mouths and natural harbors.

The earliest and most disruptive human-made structures affecting Great Lakes beaches are harbor entrance breakwaters and jetties. When many of these were built in the mid- to late-19th century in the Great Lakes, sand was often considered a nuisance, as harbors and channels filled with sand and had to be removed, typically by dredging. Up until the late 20th century, dredged sand

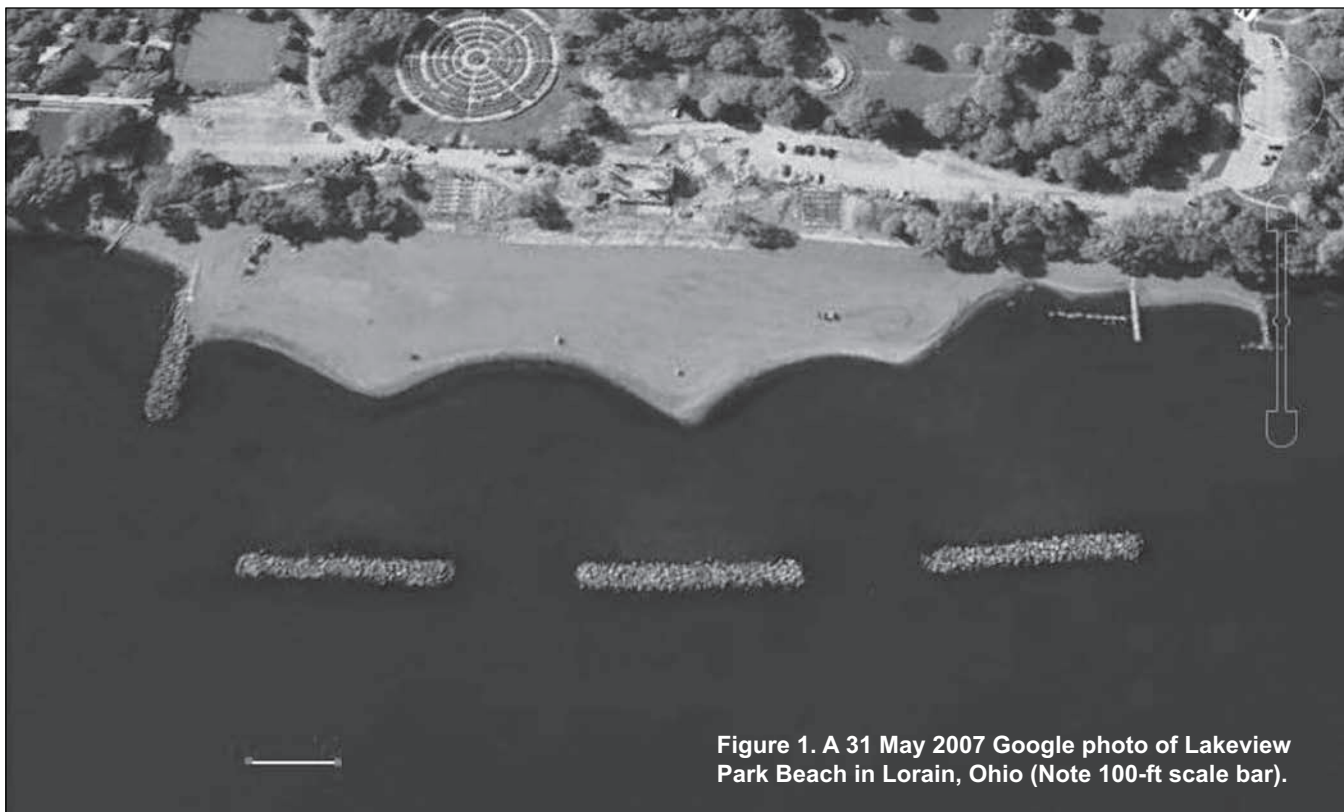


Figure 1. A 31 May 2007 Google photo of Lakeview Park Beach in Lorain, Ohio (Note 100-ft scale bar).

was removed from the harbor and was either dumped in deep water, used in lake fills, or mined for the construction industry. The lake fills, sometimes extending over a mile into the lake, are complete barriers to littoral drift sand transport if they extend beyond the surf zone. The result is often a sediment-starved system downdrift of the littoral barriers, where natural beaches are ephemeral and most wide beaches (exceeding 100 ft) are trapped updrift of the lake fills and harbors. Sediment-starved coasts in the Great Lakes include the east and west shores of southern Lake Michigan, the south shore of Lake Erie and the north shore of Lake Ontario.

The sources of beach sand in the Great Lakes include sediment from rivers and streams, as well as storm wave erosion of the lakeshores and nearshore lakebed. While fine silts and clays are typically carried offshore by waves and currents, coarser material is left close to shore as sand bars and beaches. This coarse material is in a dynamic equilibrium, moving alongshore and on and off the beach, driven by wind, storm waves, and alongshore currents. The larger the waves and more frequent the storm, the greater the rate of sand transport within a coastal "littoral cell." Littoral cells are bounded by structures that extend beyond the surf zone, such as rocky headlands or harbor

entrance channel jetties, and engineered fills, like the Chicago or Toronto lake fill, where dynamic equilibrium becomes "forced equilibrium."

STRUCTURES THAT PROTECT BEACHES

On urban Great Lakes coasts, impermeable piers, groins and groin fields were constructed as early as the mid- to late-19th century to protect eroding beaches and shores. Like the harbor entrance breakwaters, wide beaches developed on updrift sides of the structures. As a rule-of-thumb, the longer the structure, the wider the trapped beach and the greater the impact on littoral drift sand. Depending on availability of construction material, groins evolved from wood piles and rock-filled cribs to concrete and steel sheetpile in the early 20th century after World War I. In general, the groins worked well until high lake levels of the 1970s, combined with accelerated nearshore lakebed erosion, left groin-held beaches narrow to non-existent in many areas including Lake Michigan (Shabica *et al.* 2004) and Lake Erie (Pope and Rowen 1983).

A structural solution applied to beach preservation in other countries and at Winthrop Beach, Massachusetts (1935) is the segmented, shore parallel, breakwater. The first in the Great Lakes was

built in Lake Erie at Lorain, Ohio, in 1977, followed by others in urban areas of the lower Great Lakes. Constructed of quarystone, these systems, on both public and private beaches, include detached shore-parallel breakwaters, attached breakwaters and armored headlands. They were typically nourished (pre-mitigated) with sand brought in from other locations (inland sources or dredge sites).

LAKEVIEW PARK BEACH, LORAIN, OHIO

Lake Erie breakwater-held beaches, in addition to Presque Isle, Pennsylvania, include three in Lorain, Ohio; one in Euclid, Ohio; one in the Village of Geneva-on-the-Lake, Ohio; and one in North Madison, Ohio. Lakeview Park Beach in Lorain is notable in that it was the first use of segmented nearshore breakwaters constructed by the Corps of Engineers to control beach erosion (Pope and Rowen 1983). Built in 1977, the system was designed using wave diffraction analysis and corresponding littoral drift patterns, to be stable under a variety of wave and lake level conditions. It includes two end-groins and three 250-ft-long breakwaters separated by 160-ft gaps, 450-500 ft offshore in depths of 10-13 ft. Littoral drift sands are intended to pass landward of the structures. The system replaced failing seawalls, a revetment and six groins that,

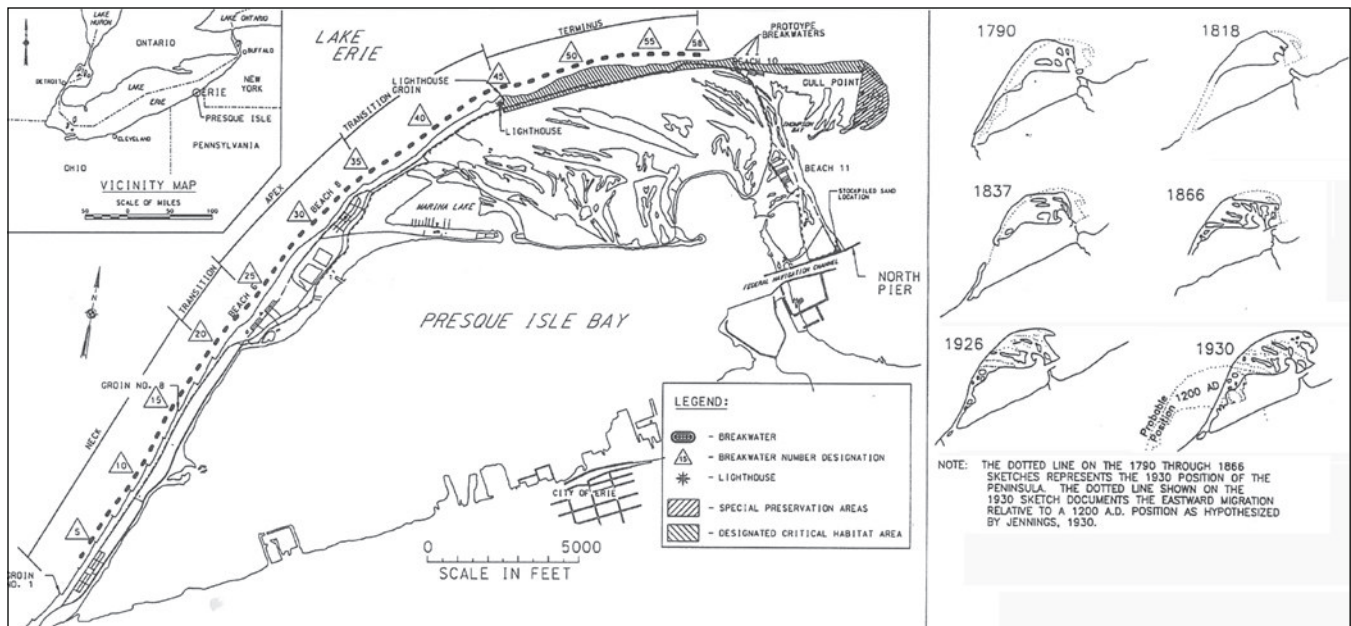


Figure 2. Vicinity and location map of Presque Isle, Erie, Pennsylvania (left). Growth and migration of Presque Isle 1790-1930 (right).

with dwindling littoral drift quantities, were no longer effective at holding sand. Potential littoral drift was estimated to be 20,000 cu yds per year to the east but only 5,000 to 8,000 cu yds per year passed the site and overtopped the landward extension of Lorain Harbor's west breakwater, immediately to the east. Approximately 110,000 cu yds of medium (0.5mm) sand was placed landward of the breakwaters, resulting in a beach averaging 200 ft wide and 1,320 ft long. Annual maintenance of 5,000 cu yds of sand was predicted by the designers. The beach was nourished only twice during the monitoring period with a total of 9,000 cu yds of medium-fine sand placed between 1980 and 1981 at the western end of the beach. Five years of monitoring from 1977 to 1983 showed a net gain in the system of 3,000 cu yds (for more details see Pope and Rowen 1983). A 200-ft-wide band of fine sand accumulated lakeward of the breakwaters over a previously gravel and cobble lakebed between 1977 and 1981. According to a U.S. Army Corps of Engineers report (1996), Lakeview beach received a total of 16,000 cu yds of maintenance sand prior to the project being turned over to the local sponsor. Since then, the city of Lorain has relocated a small amount of sand and placed 4,000 cu yds new sand on the beach (James Reagan, Acting Administrator, city of Lorain Engineering Dept., pers. comm. 2010). Comparison of the 1977 shoreline with a 2007 Google air photo shows a net beach recession of approximately 150 ft. The system was

designed for average beach widths of 150 ft behind the east and central breakwaters, 100 ft behind the west breakwater and 20 ft next to the west groin, based on a lake level of +1.7 ft LWD (Low Water Datum, IGLD 1955). Examination of the 2007 Google air photo (Figure 1) shows that these standards have been maintained.

PRESQUE ISLE STATE PARK, PENNSYLVANIA

Presque Isle State Park, established in 1921, surrounds Presque Isle Bay at the eastern end of Lake Erie and includes recreational beaches, hiking trails, and a marina that hosts more than 4 million visitors per year. Presque Isle (French for "almost an island") is a compound recurved sand spit that projects from the Pennsylvania mainland into Lake Erie and protects the federal commercial harbor at Erie, Pennsylvania (Figure 2). Formed during the Wisconsin glacial clays, sands and gravel, the spit is an anomalous sandy feature on a generally sand-starved coast. Historic maps (Jennings 1930, Gorecki and Pope 1993) suggest that the entire peninsula moved in a northeasterly direction along the shore at an estimated rate of one-half mile per century (Figure 2).

After the conclusion of the War of 1812, the attention of the United States government was directed to Erie Harbor because of the role it had played in Commodore Perry's memorable battle at the west end of Lake Erie. The first beach erosion study of the peninsula by the

U.S. Army Corps of Engineers was done in 1819. The River and Harbor Act of 26 May 1824 authorized improvement of Erie Harbor and protection of the shore at the neck of the peninsula, which by its position forms the federal commercial harbor of Erie. With the construction of harbors and shore protection to the west (updrift) over the last two centuries, sediment supply to the peninsula has been diminished. Numerous (more than six dozen) shore protection works along the peninsula were also constructed during that time. Storm waves readily overtopped the neck that has a low crest elevation of approximately +10 ft LWD. Over the last 200 years, the neck (generally less than 800 ft in width) breached four times (winter 1828-29, winter 1832-33, November 1874, and October 1917). It should be noted that several of these occurred after creation/enlargement of federal harbors to the west, most notably Conneaut Harbor, Ohio (initial jetties built 1827-1832, lengthened 1868-1871, 1894; original breakwaters built 1897-1905, modified and enlarged 1912-1917), approximately 20 mi to the west. The earliest shore protection structures consisted of timber seawalls or vegetative plantings. In the last century, a number of stone revetments and steel sheetpile seawalls were constructed. While these were more durable, they were built in response to immediate threats at specific locations. In 1955, 11 steel sheetpile groins nourished with sand fill were constructed along the neck to stabilize



Figure 3. A 1992 aerial view of Presque Isle.

this vulnerable area. This began the shift to beach augmentation using sand, such that by 1960 periodic nourishment was the main source of shore protection.

The large quantities and cost of beach nourishment needed (160,000 to 172,000 cu yds annually), prompted the Commonwealth of Pennsylvania to request an evaluation of this practice in 1968. Numerous alternatives were considered including no action, nourishment alone, nourishment and sand recycling from the east end of the peninsula, and a variety of structural configurations with nourishment. Section 501(a) of the Water Resources Development Act of 1986 (Public Law 99-662), authorized and funded the construction of 58 offshore segmented rubble-mound breakwaters (55 were built) with initial placement of 373,000 cu yds of sand fill. The design included beach berms with average widths of 75 ft and crest elevations of 10 ft above LWD. Existing shore protection structures were removed prior to construction. The plan also provided for annual nourishment of approximately 38,000 cu yds of sand fill. The breakwaters are 150 ft long with crest elevations of 7.8 ft above LWD and are separated by 350-ft gaps. Construction began in October 1989 and was completed in November 1992 (Figure 3). A more complete description of the project may be found in Mohr (1994).

The project design was the result of extensive technical investigations that included two- and three-dimensional physical model studies, as well as construction of three 125-ft prototype breakwaters in the vicinity of Beach 10 near the east end of the peninsula. Numerous studies

were also conducted to assure that the project was socially, environmentally, and economically viable. These studies were key to the success of the project as they improved public awareness and acceptance of segmented breakwaters for shoreline protection, particularly for a project of this magnitude.

During construction, the need for the breakwaters at the root (western end) of the peninsula was questioned. Aerial photographs from 1955 to 1990 suggested that the area between Groins 1 and 2 has been stable. Nourishment has not been needed in this area since groin construction in 1955. In order to confirm the need for breakwater construction at this location, GENESIS (GENeralized model for SIMulating SHoreline change, Hanson and Kraus 1989) modeling was conducted (Mohr 1992). This led to the decision to defer construction of Breakwaters 1, 2 and 3.

From 1975 until completion of the shoreline erosion control project in 1992, sand was added to the new system on the order of about 160,000-172,000 cu yds per year using a medium coarse sand with a median size of about 1.8 mm. The scheduled annual nourishment of 38,000 cu yds is based upon a sediment budget analysis of the peninsula developed for the General Design Memorandum (GDM) that determined that the breakwaters would reduce sediment transport rates along the shore by 75 percent (USACE 1986). Naturally occurring sediment inputs to the system are relatively small in quantity, estimated at 40,000 cu yds annually, primarily from bluff recession to the west of the peninsula.

Littoral material travels along the Presque Isle peninsula in a predominantly eastward direction. As it reaches the depositional east end, some sediment accumulates at Gull Point, some travels beyond Gull Point to build up offshore bars and a platform off Thompson Bay, and the remainder is transported into the Erie Harbor entrance channel. The natural pre-project subaerial growth rate of Gull Point was estimated at 0.4 acres per year. If it is found that this growth rate is not being maintained, a portion of the scheduled project nourishment is directed to this area.

Since the annual nourishment program began in 1975, the monitoring program has consisted of obtaining complete aerial photo coverage of the peninsula three times per year and biannual visual inspections by walking the beaches. After completion of the breakwaters in 1992, the program was augmented by annual topographic/bathymetric surveys. Funding constraints have precluded obtaining the photos and surveys the last several years. The objectives of the nourishment and monitoring program are as follows:

- 1) Confirm the decision to defer construction of the first three authorized breakwaters using GENESIS (Hanson and Kraus 1989) modeling. Measured shoreline position using aerial photography is compared with GENESIS results.

- 2) Evaluate breakwater settlement. Subsurface conditions below the western breakwaters (7-11) suggested potential settlement. Crest elevation change is examined.

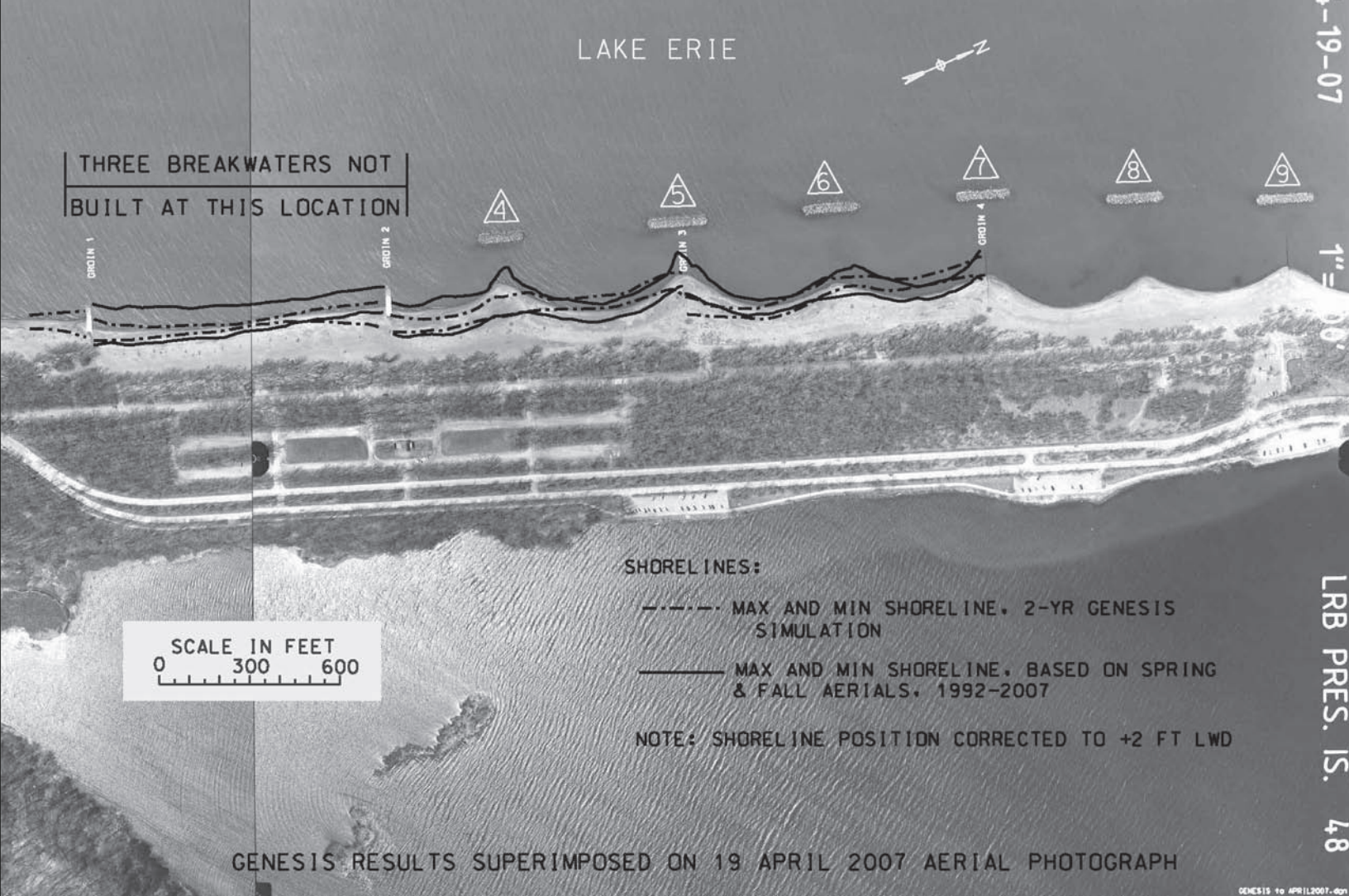


Figure 4. Surveys confirm GENESIS-based decision to defer construction of three breakwaters.

3) Document annual nourishment quantities and locations.

4) Measure shoreline position with respect to the breakwaters. The maximum lakeward extent of the salients formed behind the breakwaters should average approximately 250 ft from the breakwater centerlines.

5) Maintain Gull Point growth.

Time has proven the decision to defer construction of the first three breakwaters to be wise. A comparison of the GENESIS prediction, based upon the two-year simulation run, with measured shorelines taken from the spring and fall aerial photographs for the time period ranging from 1993 to spring 2007 (dated of latest photographs) is shown on Figure 4. The dashed lines represent the maximum and minimum shoreline resulting from the two-year GENESIS simulation. The locus of maximum and minimum shoreline locations (corrected to the +2 ft LWD elevation using a beach slope of 1V:9H) from the spring and fall 1992 to 2007

aerial photographs is also presented. The actual minimum shoreline (most landward retreat) response between Groins 1 and 2 has been similar to that predicted by the GENESIS model. However, the actual maximum shoreline has advanced further lakeward than that predicted by GENESIS. The recommendation to defer construction of the three breakwaters at this location remains valid. This recommendation saved approximately \$1 million in construction costs.

The geotechnical analysis performed during the design of the breakwaters indicated that there was a potential for long-term settlement of about 1.5 ft to 4.5 ft for Breakwaters 7 through 11 with the maximum occurring at Breakwater 8. For this reason, these breakwaters were constructed near the beginning of the contract and additional stone was placed as needed later in the contract to ensure the design crest elevation of +8 ft LWD. In order to determine the extent of any additional settlement, periodic surveys have been taken along the crests of Breakwaters 6

through 13. The first surveys consisted of taking physical measurements. However, this was changed to SHOALS (Scanning Hydrographic Operational Airborne Lidar Survey) surveys in 1997. A 2004 survey revealed some settlement in these breakwaters with Breakwater 7 showing the most. Settlement below +6 ft LWD (a loss of more than 1.8 ft in elevation below design level) has occurred in 33 percent of Breakwater 7's crest. The rest had crest elevations predominantly above +7 ft LWD. Since no adverse beach response has been observed behind the breakwaters, no further action besides continuing monitoring is recommended at this time.

Since the completion of the breakwaters, an average annual nourishment quantity of 32,200 cu yds has been placed (1993 to 2009). Only lake-dredged sand is allowed that has a median particle size of 0.7 mm. While the initial placement was done by hydraulic dredge due to the large quantities, subsequent annual nourishment placement has been by land-based equipment. From 1993 to 2003, the



Figure 5. Built on reclaimed land in Lake Ontario east of Toronto, Bluffer's Park first opened in 1975 and was expanded to its current size in the early 1980s. The park is home to four sailing clubs. Note 100-ft scale bar and armored headland on left (Google air photo 2009).

sand was trucked through the city of Erie into the park and placed on the beaches. The Erie North Pier was strengthened allowing for the overloading of sand from self-unloading vessels into a designated stockpile area. Since that time, all new sand is distributed to the beaches from the replenished stockpile.

The sediment budget calculated for the project (USACE 1986) predicted that with the breakwaters installed, the required annual nourishment amounts would be approximately 25% of pre-project quantities. Comparison of pre- and post-project nourishment quantities reveal that annual nourishment with the project is about 19% of pre-project amounts and is about 86 percent of the GDM estimate with the project. The GDM estimate has not been met due to funding limitations and represents about 99,300 cu yds of sand that has not been added to the system since 1993. The federal government shares equally in funding the nourishment with the Commonwealth of Pennsylvania. It should be noted that in 2005 and 2006, federal funding was substantially reduced, and in 2007 no federal funding for nourishment was available. The shortfall was significantly augmented by funds from

the Commonwealth of Pennsylvania to ensure its success. However, as will be noted in the Gull Point growth discussion, it has been observed that Gull Point has not been expanding at the minimum desired rate. This may be partially attributed to the reduced nourishment program.

Since construction, the sand behind Breakwaters 1 to 19 has been stable with little or no nourishment or excavation needs. At higher number breakwaters, an alternating "hot-spot" erosion and "cold-spot" accretion pattern has developed where new sand is placed annually due to shoreline recession. Sand is excavated from an average of five breakwaters per year where excessive sediment accumulation behind the breakwater causes tombolo development. This results in the shoreline extending out to the breakwater, and erosion immediately downdrift. The hydraulic model study (Seabergh 1983) indicated that with the formation of a tombolo, the sediment movement was diverted lakeward of the breakwaters, moved downcoast parallel to the breakwater and then shoreward around the downcoast tip of the breakwater. Actual experience has shown that downdrift erosion extends several breakwaters until a rhythmic shoreline is restored.

This situation is not desired and thus material is excavated from the zones of unwanted accretion and recycled into areas of erosion.

A goal of the project and the designed breakwater configuration was to develop a stable, sinuous shoreline with the breakwaters averaging approximately 250 ft off the shoreline. In order to determine the shoreline distance from the centerline of the breakwaters, the shorelines were measured using uncorrected-scaled aerial photographs obtained spring 1993 to 2007. Based upon the average water level for the day of the photo and assuming an average slope of 1V:9H for the beach near the waterline, the shoreline position was corrected to represent the shore at an average water level of +2 ft LWD. Since construction, the shoreline annually returns to a planform that varies from project sector to sector. The average distance during the spring between the salients and the breakwaters from 1993 to 2007 was 225 ft, with a standard deviation per breakwater of 62 ft. The occurrence of significant salients and tombolos is primarily limited to east of the lighthouse (Figure 2).

Throughout the neck (Breakwaters 4 through 19), the shoreline has been very



Figure 6. Armored headland and attached breakwaters constructed in 2005-2006 to protect public beaches in Port Union, Ontario. A second system was constructed about a half mile to the north in 2009. Note 100-ft scale bar (Google air photo 2009).

stable with no required renourishment and salients averaging 240 ft (+/- a standard deviation of 30 ft from the breakwater line). Although there are slight oscillating patterns of varying salient widths, they are temporally stable, and the shoreline position is naturally maintained within design tolerances. Through the apex of the peninsula (zone of greatest curvature, Breakwaters 20 through 34), there is more temporal and spatial fluctuation in the shore position with a range of 110 to 340 ft. The shoreline in Sector 2 (transition from the neck to the apex) averages 250 ft from the breakwaters. The average distance of the shoreline from the breakwater line throughout the Apex (Breakwaters 23-34) is 225 ft with a standard deviation of 39 ft. The transition from the apex and into the project terminus displays significant spatial and temporal fluctuation in shoreline position, increasing in amplitude toward Breakwater 58. Although the average position was 220 ft, there is a significant range (425 ft) in the distances from the breakwater to the shore for individual breakwaters and for a single breakwater from year to year. A wave-like pattern of alternating zones of erosion (no salient, shoreline cut back) and accretion (tombolo or near tombolo) passes through the Terminus

(Breakwaters 45-58) as sediment waves move along the shore. These waves have a longshore periodicity of approximately 4 or 5 breakwater lengths.

Shoreline change east of the project area at Gull Point has been computed annually until 2006 (last available fall aerial photograph) to determine if sufficient growth continues to occur. The GDM (USACE 1986) stated that the condition of growth must be maintained if the integrity of Gull Point is to be preserved. The GDM also states that adverse impacts to Gull Point exist if:

- (1) The average annual growth rate of Gull Point falls below the prenourishment rate of 18,400 cu yds per year or 0.4 acres of surface area growth per year, or
- (2) The Gull Point area is in danger of being severed from the main body of the peninsula due to severe erosion immediately downdrift of the breakwater system. Physical contact between Gull Point and the peninsula must be maintained if migrating sediment is to reach Gull Point.

Each year the change in the size of Gull Point is computed using the recent fall aerial photograph and the May 1991 shoreline. The shorelines were corrected

to +2 ft LWD. The May 1991 shoreline is used as the basis of comparison since it was used for the Gull Point environmental study. If it is determined that Gull Point is not growing at the minimum desired rate, a portion of the nourishment material is placed east of Beach 10, updrift of Gull Point. Until recently the goal was met. However, the average annual planform change of Gull Point from 1991 to 2005 was -0.01 acres per year, and from 1991 to 2006 was -0.30 acres per year, which is less than the minimum desired value of 0.4 acres per year. As previously noted, actual annual nourishment has been about 86 percent of the GDM estimate with the project. The GDM estimate has not been met due to funding limitations and represents about 99,300 cu yds (2.6 years of nourishment material) of sand not added to the system since 1993.

In summary, 55 of the authorized 58 breakwaters were completed in 1992. Since then the shoreline response at the location of the three deferred (not built) breakwaters has shown that the decision to not construct those has been appropriate. The project goal to establish a sinuous shoreline behind the breakwaters has been achieved, with areas requiring sand or with excessive sand (tombolo



Figure 7. View of the largest headland-protected beach in Lake Michigan at Forest Park Beach in Lake Forest, Illinois (above). The system lies within the surf zone, extending about 400 ft from the bluff toe into the lake. In comparison, 3.5 miles updrift (north) is Great Lakes Naval Training Center Harbor (below) that was, for nearly a half century, a total littoral barrier. The breakwaters, built in 1923, protrude 2,400 ft into Lake Michigan. Between 1923 and 1976, approximately 2.5 million cu yds of sand was impounded in the harbor and on its updrift side (Chrzastowski and Trask 1995). 2007 Google photos both to same scale.

formation) addressed during the annual nourishment program. However, the predicted annual nourishment requirement of 38,000 cu yds has not been met due to funding limitations, resulting in an actual average annual amount of 32,200 cu yds placed and represents a deficit of 2.6 years of nourishment material over the past 17 years. This is beginning to affect Gull Point downdrift of the breakwaters and is being partially addressed by placing a portion of the nourishment sand downdrift of the project area.

TORONTO REGION, LAKE ONTARIO

Western Lake Ontario has a long history of engineered coastal structures. The Toronto metropolitan area waterfront is characterized by lake fills (Figure 5) and headland-protected beaches. According to the Ontario Ministry of the Environment (Persaud 2003):

Most of the large lake fills for the purpose of land creation have been centered in the western basin of Lake Ontario, especially the area adjacent to the Toronto waterfront. The Toronto Harbour Commission, under Federal charter, has been using this technique to develop the Toronto waterfront since 1911. Since the 1950s, the Commission has been involved in the construction of the Eastern Headland (also known as the Leslie Street Spit) which is the largest lake fill structure in Lake Ontario. During the late 1960s and 1970s, the conservation authorities bordering western Lake Ontario and other government agencies (e.g. municipalities, Government of Ontario) proposed shoreline plans which

included varying degrees of land creation through lake filling.

Creation of new land resources through filling in the littoral (shallow, near-shore) zone can be an appealing concept for several reasons. With lake-front property commanding a premium price, the creation of new land by lake filling is attractive, particularly in the heavily populated western basin of Lake Ontario. In many cases, lake fills provide recreational opportunities that would otherwise not exist and could not be provided through the purchase of existing shoreline properties.

Study of the coastal system dynamics has played an important role in the development of the Toronto lakeshore. For example, modeling of the littoral drift system in the Toronto region, a key component to proper design and sustainability, was developed in the latter part of the 20th century (Greenwood and McGillivray 1978).

Armored headlands and breakwaters are a prominent form of beach protection in Toronto and include two systems constructed in 2005 and 2009 in Port Union, Ontario (Figure 6). Planned and constructed by the Toronto Region Conservation Authority, the beach systems are being monitored for impacts to the littoral system and fisheries (Ontario Ministry of the Environment 2000). As the shorelines are nearly fully engineered and the structures well-designed, impacts are likely to be minimal. More importantly, the government is committed to monitoring and remediation if necessary.

FOREST PARK BEACH, LAKE FOREST AND SUNRISE PARK BEACH, LAKE BLUFF, ILLINOIS

Attached breakwaters at Forest Park Beach in Lake Forest, and Sunrise Park Beach in Lake Bluff, both of which are on the Lake Michigan shore in Illinois, were built in 1987 and 1991 respectively to protect municipal beaches. The beaches are located on Illinois' shoreline north of Chicago, a 24-mile stretch of urban coast that (with the exception of Illinois Beach State Park) is fully engineered. The near-shore is considered sediment starved, with eroding cohesive clay lakebed exposed in many locations or covered with a thin veneer of sand in others (Shabica and Pranschke 1994). Sand mining and construction of total littoral barriers like Great Lakes Naval Training Center Harbor and Waukegan Harbor breakwaters in the early 20th century have exacerbated the loss of littoral sand.

The coastal geology of Illinois' North Shore is primarily eroding glacial clay-till bluffs and lakebed that are composed of about 10 percent sand. Photographs of the North Shore from the 1880s through the 1930s show extensive development of rock-filled wood crib piers that performed like groins, trapping sand on their northern sides. In unprotected areas, bluff retreat rates averaged 8 to 10 inches per year (approximately 0.4 cu yds of sand and gravel lost per linear foot of lakeshore per year) (Jibson *et al.* 1994). After World War I, most of the piers were progressively replaced with steel sheetpile groins. Over the next 50 years the groins would steadily lose effective-

ness. Forest Park Beach and Sunrise Park Beach groins were no exception, with beaches narrowed and bluff toes scoured by storm waves in the 1970s.

In the mid 1980s, the city of Lake Forest hired W.F. Baird and Associates to design and engineer a sustainable beach to replace the failing groins at Forest Park Beach. In addition to numerical analysis, a physical hydraulic model of the system was conducted in order to maximize the probability for success of the project. Designers assumed littoral drift in this coastal cell would be negligible (Anglin *et al.* 1987). This is not surprising as Great Lakes Naval Training Center Harbor, 3.5 miles updrift, was considered a total littoral barrier (Figure 7). Based on results of the model, five attached breakwaters were built in depths of up to -11 ft LWD, 180 ft lakeward of the preconstruction shoreline (Figure 7). The four north beach cells were filled with 200,000 cu yds of 2.8 mm fine gravel "birdseye sand" that the engineers anticipated would be more stable than the native medium sand (Anglin *et al.* 1987). State regulators required as-built and post-construction surveys to assure no negative impacts. Monitoring consultants recommended an additional 20,000 cu yds of sand be added immediately downdrift of the site prior to project construction as insurance that the littoral stream would not be disrupted by the work (Dean and Seymour 1986). Although this was not done, it is noteworthy that the Illinois DNR now recommends a 20% sand overfill for projects, both public and private, that may trap sand.

The Forest Park site, including the shore 800 ft updrift and 1,200 ft downdrift, was surveyed from 1987 to 1989 and included 43 profiles ranging from 100 ft to 620 ft apart. Results of the monitoring showed stable profiles in the beach cells but sand accretion on the updrift side of the project of approximately 10,000 cu yds of sand. To compensate for the accretion, the city of Lake Forest placed 10,000 cu yds of sand downdrift of the site over a three-year period between 1991 and 1993. Because of the unanticipated sand accretion, another five years of monitoring was recommended from 1991 to 1995 with the Illinois State Geological Survey providing independent oversight. This survey included 71 profiles at 50-ft intervals extending up to 800 ft lakeward and 27 profiles at 200-ft intervals extend-

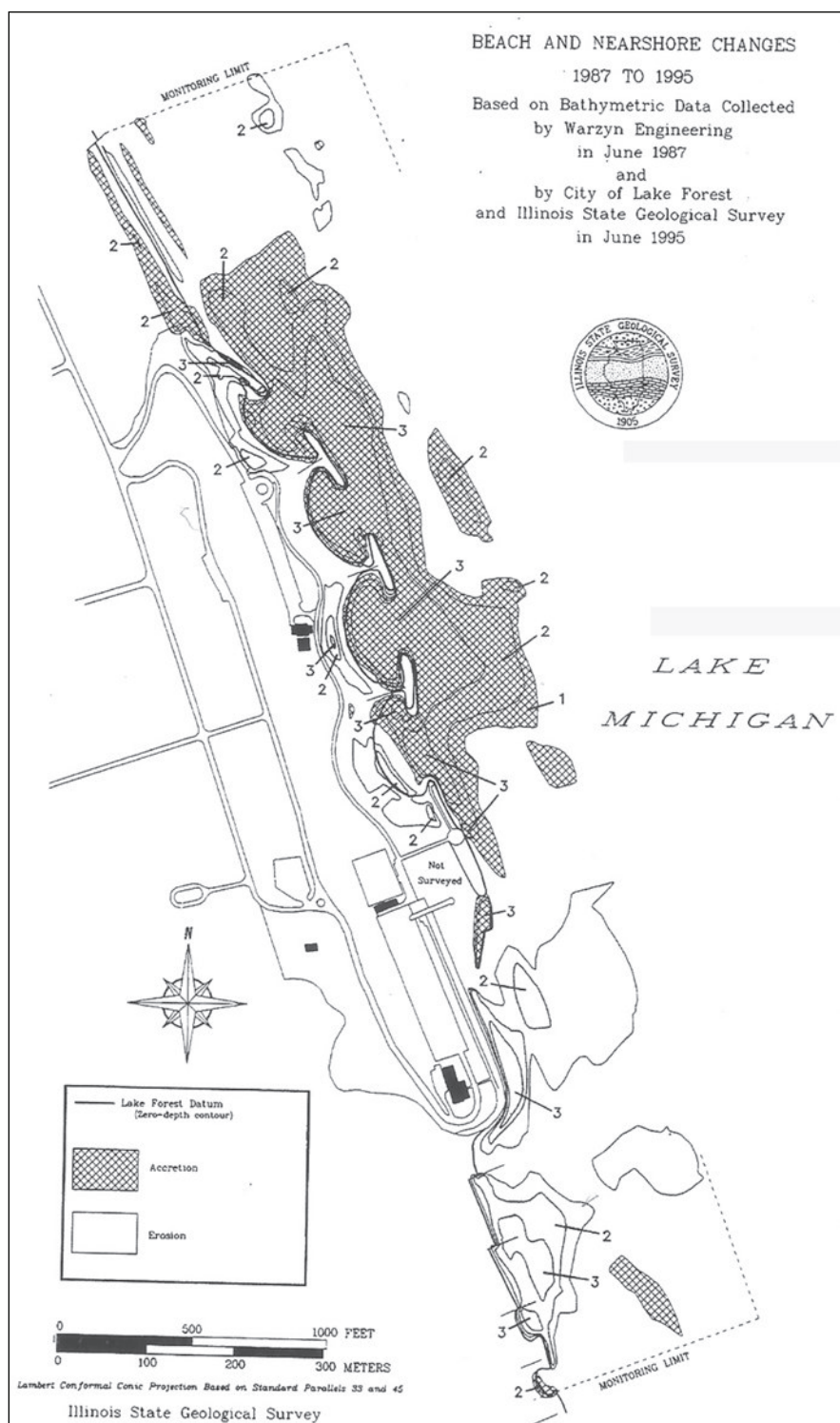


Figure 8. Isopach map showing sand accretion and erosion at Forest Park Beach, Lake Forest. The map is based on a comparison of survey data from 1987 and 1995 (first eight years after construction) and shows sand loss off the dry beach and an accretionary wedge of sand around the structure. Only changes greater than 1 ft are shown (from Chrzastowski and Trask 1996).

ing up to 2,950 ft offshore to depths of 23 ft (beyond the approximate depth of closure of 20 ft). Sand accretion was reported around the structure during rising lake levels between 1992 and 1994 followed by net erosion during falling lake levels between 1994 and 1995.

Results show some sand loss to the dry beach areas and a 3-ft thick accretionary wedge extending from the shore within the beach cells to 300 to 400 ft beyond the breakwaters (Figure 8). The net volume of material removed from the littoral system from 1987 to 1995 was



Beach (Chrastowski and Trask 1996). Future surveys might show whether the accretionary wedge is a permanent feature or simply the result of varying levels of littoral drift sand related to irregular bypassing of sand dredged from Waukegan Harbor (7 miles updrift) and storm activity.

From 1989 to 1995, the boat launch basin at the south end of the project trapped 22,440 cu yds of fine sand that was dredged and then placed in the near-shore (depth less than 10 ft) downdrift (south) of the property (Chrastowski and Shabica 1996). More recently, between 2,500 and 6,000 cu yds per year of fine sand has been dredged from the basin (Chrastowski 2005 and Mary Van Arsdale, director, Lake Forest Park District, pers. comm. 2010). This is evidence that native sand is bypassing the facility. A 2010 inspection of the surface sand in beach cells 2 and 4 show a mixture of native sand and birdseye sand in proportions of about 50/50.

North of Forest Park Beach is Sunrise Park Beach in Lake Bluff, a single-cell breakwater system designed and engineered by Shabica and Associates. The objective was to create a sustainable public beach and protect an actively eroding bluff. With a limited budget and the newly constructed Forest Park Beach as a functioning “prototype,” it was determined that a physical hydraulic model would not be necessary. The site is approximately 2 mi south of Great Lakes Naval Training Center Harbor and is fronted by a lakebed that was “stripped of nearshore sand” due to sand impoundment at the harbor (Chrastowski and Shabica 1996). A stone headland and a spur breakwater were constructed in 1990-91 to protect a single-cell beach at the south end of Sunrise Park that was to be used for sailboat access and recreation. The beach was filled with 2,300 cu yds of granular material as a base and 6,600 cu yds of new medium to coarse sand

Figure 9. Sunrise Park Beach, Lake Bluff, Illinois, Lake Michigan. South Beach (project beach) opposite 100-ft scale bar, with spur breakwater on the north side of the beach cell and armored headland on south side of cell. North Beach (control beach) with new headland breakwaters at top of photo. Google air photo, 11 October 2007.

(Figure 9). State regulators required a five-year monitoring survey program that was voluntarily extended by two years. Sunrise Park Beach (South Beach) and a groin-protected control site (North Beach) located updrift from the site were surveyed from 1992 to 1999. During two periods of rising lake levels, the project beach gained 0-1 cu yd of sand while the control beach lost approximately 0-2.6 cu yds of sand per linear foot of lakeshore per year. During two periods of falling lake levels, the project beach lost 1-4 cu yds of sand while the control beach lost 0-0.5 cu yds sand per linear foot of lakeshore per year. Both beaches showed a net loss of sand over the study period.

The North Beach, already narrow at the beginning of the study was in poor condition at the end of the study and was considered unsuitable for public recreation. In 2003, the decision was made to protect it with headland breakwaters that were installed in 2004 with 2,300 cu yds of new sand fill (Figure 9). At the South Beach, approximately 250 cu yds of new sand are added to the beach every few years to compensate for a net loss of sand from the beach cell. This amounts to about 1.5 percent of the original fill quantity in annual maintenance. No maintenance fill is added to the North Beach. No adverse impacts on downdrift beaches or shore protection structures has been observed. The five property owners immediately south of Sunrise Park Beach either are or were clients of the senior author. These properties have been inspected on an annual basis through 2010. From north to south, the properties are respectively protected by a steel sheetpile seawall and revetment, two beaches held by stone headland breakwaters, and steel groins with quarystone revetments.

In terms of sustainability and environmental impacts, survey data from Forest Park Beach and Sunrise Park Beach show that attached breakwater systems are able to hold sandfill within the beach cells with minimal maintenance. Sand accretion on the nearshore sediment-starved lakebed off Forest Park Beach, while unanticipated by the designers, may help minimize lakebed erosion but more importantly, should improve the quality of the shallow water benthic ecosystem (Meadows *et al.* 2005).

Today, more than 18 engineered beach systems have been permitted and

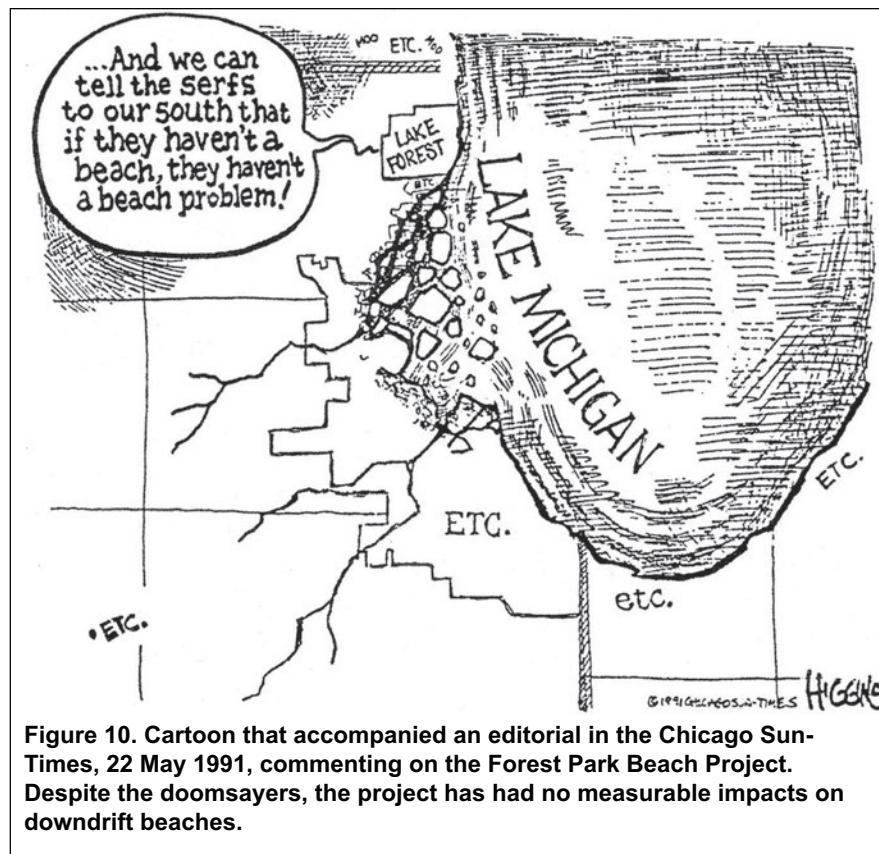


Figure 10. Cartoon that accompanied an editorial in the Chicago Sun-Times, 22 May 1991, commenting on the Forest Park Beach Project. Despite the doomsayers, the project has had no measurable impacts on downdrift beaches.

constructed in Illinois. All but one are attached breakwater-held pocket beaches. Illinois regulators now require a 20% sand overfill for new beach construction. This assures that there is continuity of sand bars around the new structures and no net loss of sand from the littoral drift system.

SUMMARY

Monitoring surveys at Lakeview Park Beach, Sunrise Park Beach, Forest Park Beach, as well as comparison of historic air photos at other Great Lakes sites has shown that beach protection structures including segmented (detached) breakwater systems and attached breakwaters (pocket beaches) and armored headlands, if built well within the surf zone, pre-mitigated with sand fill, and maintained following design standards, have minimal impact on downdrift beaches and shores and require minimal quantities of maintenance sand. In contrast, sand monitoring is crucial to assure sustainability at the more than 4-mile-long detached breakwater system at Presque Isle. Here additional sand is required to assure growth of the downdrift sand spit at Gull Point.

The detached breakwater system at Lakeview Park, by design requires 5,000 cu yds per year sand maintenance. However, after five years of monitoring and

9,000 cu yds of new sand placed, the system showed a net gain of 3,000 cu yds of sand by 1983, or an average of 1,200 cu yd per year of sand nourishment during the study period. The city of Lorain later placed an additional 4,000 cu yds of sand and none has been needed since then. The Presque Isle detached breakwater system by design requires annual surveys with nourishment of 38,000 cu yds of sand per year including removal of tombolos behind breakwaters to assure sand bypassing of the system and growth of Gull Point downdrift. If the breakwaters had not been built, the Presque Isle site would have required beach nourishment quantities of 160,000-172,000 cu yds annually. The beaches at Bluffer's Park and Port Union, Ontario need minimal sand maintenance. The attached breakwater system (pocket beaches) at Forest Park by design requires little or no sand nourishment after the initial 10,000-cu-yd sand gain was compensated for. At Sunrise Park south sailing beach, annual sand nourishment is about 125 cu yds per year to compensate for the wide gap between breakwaters.

LESSONS LEARNED

Structural solutions to protect beaches should be a last resort for unengineered stretches of coast where beach nourish-

Compensation for structural impacts: The sand bank

Many of America's urban beaches today are threatened due to sand loss caused by human activities including harbor dredging and shore armoring. Attempts to mitigate these problems are often too little-too late. Cumulative impacts of activities that reduce the supply of sand to coastal beaches is sometimes difficult to assess but should be considered in any shore management plan. A detailed "sediment budget" that identifies sinks and sources for sand can provide a useful starting point. Sediment budgets are approximations of the yearly balance of sand entering a self-contained coastal system (coastal cell) through rivers and shore erosion, and exiting the system through loss to deep water, entrapment in an embayment and upland loss.

For coastal beaches to be sustainable, sand removed from the system, or prevented from getting into the cell by a human-made structure, should either be replaced or compensated for. A vehicle proposed for assuring fair-play in sand management is a "sand bank" administered by a local or state agency, where a section of coast deprived of sand could benefit from sand paid-for out of a special taxing district fund generated by offending structures or activities.

Structural impacts on a coastal include:

- Impoundment of sand by a new structure
- Diversion of sand out of the littoral system either offshore beyond the surf zone (lake fills and harbor breakwaters) or to upland areas (sand mining)
- Prevention of sand entering the system through rivers and streams (dams, harbor dredging) or prevention of shore erosion (seawalls, revetments, groins, breakwaters)

Mitigation for these impacts can take the form of sand bypassing, nourishment with new sand brought in from outside the system, sand overfills with construction of new structures and/or financial compensation.

While the greatest impacts are from fills and harbors, there is a growing concern that eroding shores that are armored should also contribute to the system. For example, on a fully engineered coast using an estimate of the annual average shore recession rates, eroding sediments retained by structures that might have otherwise nourished the beach can be calculated. According to Jibson and Staude (1994) under natural conditions, annual sand and gravel loss from Illinois' North Shore bluff recession averaged 0.4 cu yds per linear foot of lakeshore. This is based on an average recession rate of 8 to 10 inches per year and bluff soils containing 20% sand and gravel. Thus, a coastal property with a bluff protected by a 100-ft-wide revetment, under the "sand bank" would be taxed for 40 cu yds per year. At a delivered cost of \$36 per yard of new sand, this represents an assessment of \$1,440 per year. In many states (including Illinois), coastal properties are taxed substantially higher than adjacent properties inland, yet rarely is that money is used for coastal sand management. A well-managed sand bank would assure the health and longevity of the beach, one of our most important natural resources.

ment cannot be justified based on cost/benefit analyses. Sediment-starved coasts, such as Presque Isle or Illinois' North Shore, are good candidates for structures when nourishment is no longer an economical solution.

In regions where there are no existing structures to study for effectiveness and impact on the littoral system and adjacent shorelines, it will be prudent to build a prototype like the three-breakwater system at Presque Isle or conduct a physical hydraulic model. This will not guarantee a "perfect solution" as seen at Forest Park but can fine-tune the performance of the structures assuring an economical and effective solution to sustainable beaches.

Great care should be exercised when applying structural solutions to locations where most downdrift beaches are in a natural state. For example, at Presque Isle, the Corps of Engineers has quantified the sand necessary to successfully sustain the Gull Point sand spit downdrift of the breakwaters. The only impediment to Gull Point's health might be insufficient funding for beach nourishment. A "sand bank" would make that problem moot.

A big hurdle in proposing a new type of beach protection system is political or sociological. By nature, humans are resistant to change. Even legitimate engineering or scientific arguments pale in the face of fear-of-change. Planners of the Presque Isle and Lorain Beaches in Lake Erie showed good insight in preparing the public for the realities of a new method of shore protection. On the other hand, the Forest Park Beach project is a good example of the difficulties in dealing with outspoken adversaries that oppose the project regardless of the benefits and costs both economic and environmental. A lack of understanding of "scale" is often at the root of opposition. For example, well-meaning citizens may see no difference between impacts of complete barriers to littoral drift, like Illinois' Great Lakes Naval Training Center Harbor, and a substantially smaller structure like Forest Park Beach that lies well within the surf zone. A 1991 cartoon published in the Chicago Sun-Times is an example of an outspoken adversary stirring up opposition (Figure 10). Despite the doomsayers and an intense campaign to derail the project, the Forest Park Beach was completed, and to date, no measurable negative impacts have been reported. Three years after the

completion of Forest Park Beach, Sunrise Park Beach also met with some public resistance. Fortunately, the fact that breakwaters at Forest Park Beach did not fulfill cataclysmic predictions of downdrift destruction of property helped.

The breakwaters reviewed in this paper fall into two main types: detached with a continuous beach and attached with separate beach cells (pocket beaches) between breakwaters or headlands. Regular transport of littoral drift sand landward of the breakwaters is a key component of detached breakwater systems. Natural irregularities in wave regime and bathymetry can result in formation of tombolos behind some breakwaters and excessive erosion behind others, disrupting sand transport and requiring sand relocation maintenance. In contrast, littoral drift transport at attached breakwater systems, placed closer to shore, occurs primarily lakeward of the structures. Tombolo formation is not an issue and sand transported between adjacent cells

is quantitatively less, resulting in lower maintenance costs.

In conclusion, the most successful sustainable beaches on sediment-starved coasts are nearshore attached-breakwater beaches that are pre-mitigated with sand fill. This must include fill to capacity within the structures, and as we have learned in this study, sand fill should also be placed around the structure to assure an uninterrupted littoral system. The systems should be streamlined at both ends to promote continuity of sand transport around the structure.

It is also important that an annual monitoring plan be developed. However, as we have seen with Forest Park Beach, irregularities in the quantities of sand entering the system, including periodic dredged sands from Waukegan Harbor, can result in misleading survey data. This phenomenon was also observed at Presque Isle where sand moved intermittently alongshore in "slugs." Survey profiles should extend updrift

and downdrift at least twice the length of the new system, and to the offshore depth of closure. Repeatability of survey points is important and the minimum number of profiles should be undertaken to economically show areas of sand gain and loss. Depending on survey results, the builder should be prepared to add approximately 20% new sand to the system (Illinois regulators already require a 20% sand overfill). If a "sand bank" system is instituted, a well-funded regional cell-by-cell sediment budget will assure the sustainability of this valuable coastal resource, the beach.

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