CAPITAL DEVELOPMENT BOARD / ILLINOIS DEPARTMENT OF NATURAL RESOURCES

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ILLINOIS BEACH STATE PARK SHORELINE STABILIZATION

1 Lake Front Drive,
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PHYSICAL MODELING SUMMARY

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Executive Summary

A series of 2-dimensional and 3-dimensional modeling tests were performed at the HR Wallingford laboratory in Wallingford, UK during the months of August-December 2020. The goal of these test was to find a beach control structure configuration which cost-effectively stabilized the Illinois Beach State Park shoreline within the three defined project areas.

For each identified area, a separate 3-dimensional physical model was constructed. The modeling methodology consisted of two phases: first, the optimization of the beach control structures and layout to reduce the sediment transport during persistent morphological wave conditions; second, a confirmation series of tests to assess the stability of the rock gradations used for the structures under extreme wave and water level conditions.

Through the physical model testing process, the layouts of the beach control structures in all three areas was updated to fulfil the goals and individual approach to each area. These redesigned layouts were rigorously tested and represent the necessary offshore intervention required to stabilize the shoreline during high water events.

Conceptual & Schematic Design

In 2019, SmithGroup carried out a comprehensive study of the Illinois Beach State Park shoreline, and developed concepts for stabilization while remaining mindful of the park's mission to remain as 'aesthetically natural' as possible.

This study was part of the first project phase, where, in order to determine the causes of the shoreline erosion and design comprehensive solutions, multiple variations of the shoreline structures were developed and analyzed using numerical models. Through joint discussions with the Illinois Department of Natural Resources, the Illinois State Geological Survey, and the U.S. Army Corps of Engineers, accompanied by an extensive technical analysis, three preferred alternatives were chosen that combine nearshore and offshore emergent and submerged rubble breakwaters.

A further description of the first phase of the project and each initial alternative can be found in the "Shoreline Morphology Analysis & Stabilization Options" report prepared on September 6th, 2019 by SmithGroup and Jack C. Cox, PE.

The second phase of this project started in March 2020, with the goal to optimize the original conceptual layouts that resulted from the numerical model by physically modeling the wave environment of Lake Michigan in a coastal 3D laboratory. This process reveals nearshore processes and phenomena that are difficult to replicate in numerical models and allow for creative design solutions.

Enhanced Analysis

Lake Michigan experienced record-high water levels in 2019 and 2020 that led to larger waves impacting and inundating the shoreline. Due to these events, the team reassessed the design criteria to be used to test the structures and a consideration for higher water levels, listed as extreme, was made.

Additionally, a bathymetric survey taken at the project site in early 2020 allowed for some enhancements to the geometry of the structures. The revised layouts that were used as the initial conditions for the laboratory runs are shown in Figures 1-3.

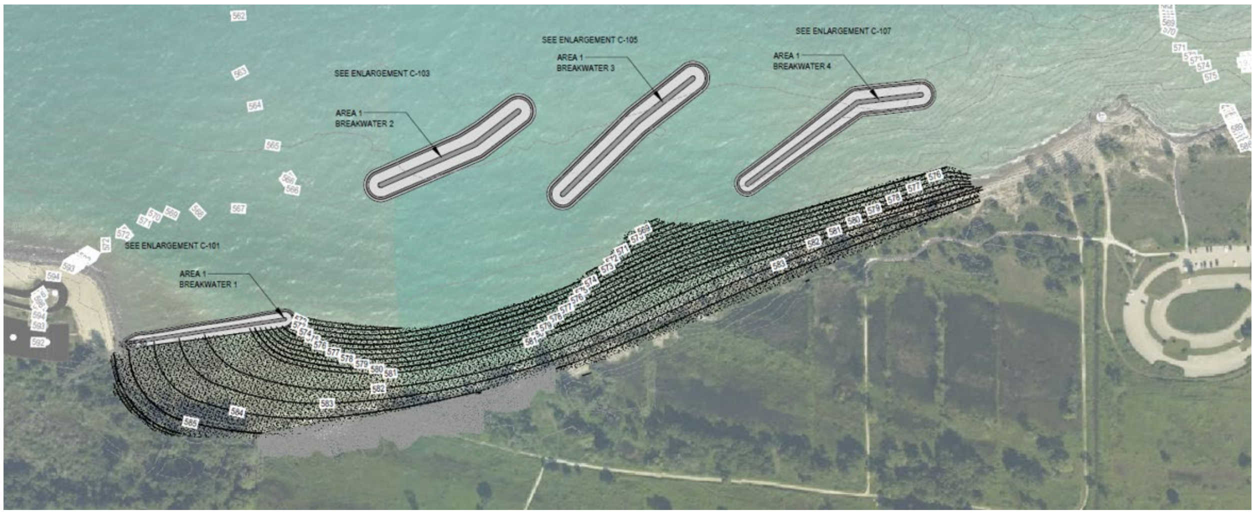

Figure 1: Area 1, Initial Layout for Lab Testing

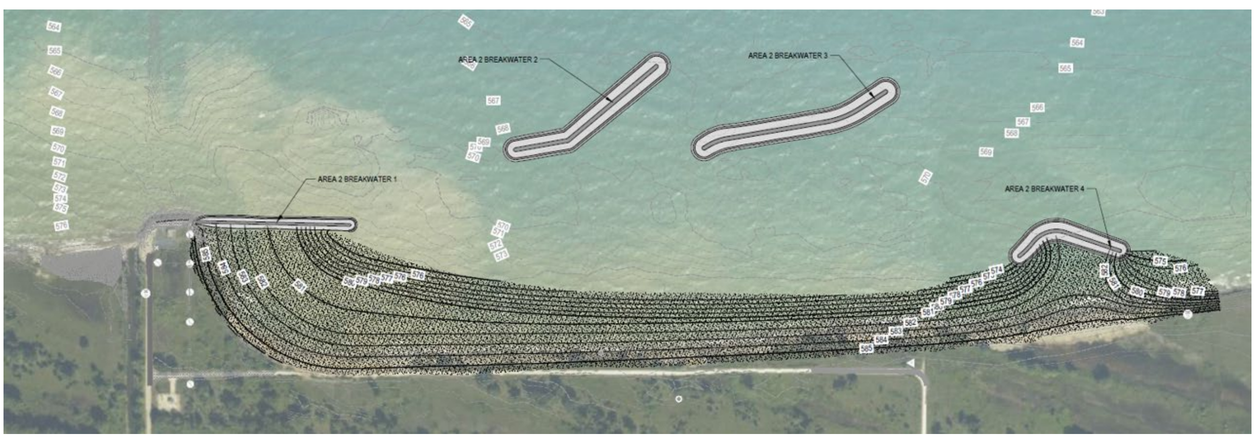

Figure 2: Area 2, Initial Layout for Lab Testing

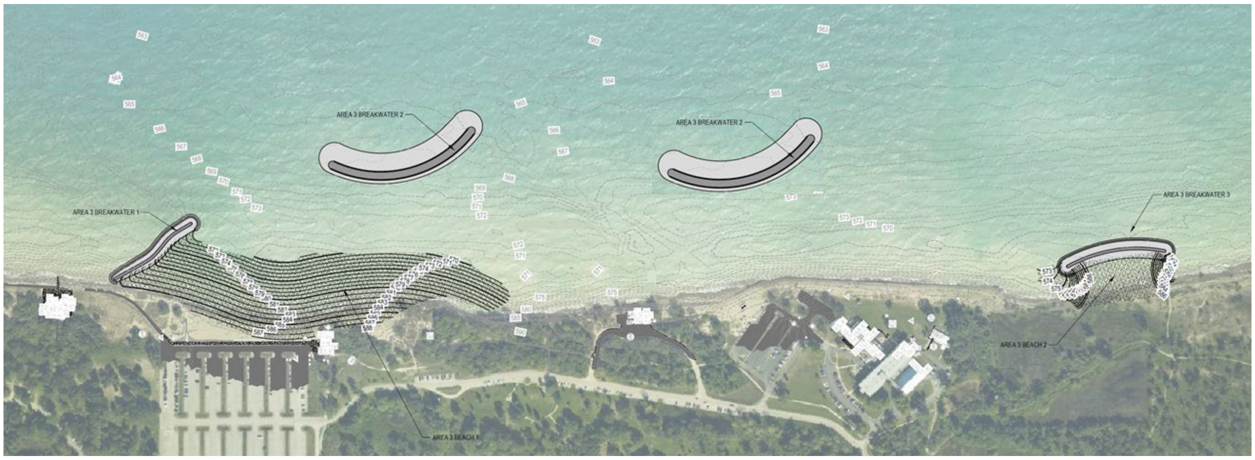

Figure 3: Area 3, Initial Layout for Lab Testing

Lab Facilities

Physical model testing was performed at HR Wallingford's Froude Modeling Laboratory located in Wallingford, UK. The laboratory is home to multiple wave basins and wave flumes allowing some of the tests performed for the IBSP project to run concurrently.

Structure cross section testing was performed in a 2D wave flume. The flumes are equipped with the same state of the art instrumentation and equipment as the larger wave basins. 2D tests are typically where testing starts to optimize and validate cross sections prior to moving into the more complex 3D investigations. The flume used for testing was 131 ft (40m) long by 4 ft (1.2m) wide.

Large 3D wave basins were used to investigate beach morphology and breakwater stability. HR Wallingford's largest wave basin is one of the largest test tanks in the world and allows for developments to be tested without significant scale effects. Various sized basins were used for the three separate basin tests. The basins are equipped with multi-element random wavemakers with active wave absorption.

Live video streams were set up during each test to allow for online viewing. Photographs were taken and laser scanners were used during and after tests to document results.

Physical Modeling Goals

Entering into physical model testing, there were four identified goals:

1. Wave Transmission – A reduction in wave transmission over and through the offshore breakwater structures reduces the energy along the shoreline, thereby reducing littoral transport potential.
2. Beach Morphology – Look at the series of breakwaters as a complete system which works together to develop a stable shoreline within the defined project areas.
3. Structure Stability – Test various cross sections and rock gradations for stability against given design criteria.
4. Habitat – Determine whether habitat features built into the design can enhance transmission and morphology goals.

Methodology

Model Scale

The flume and basin models were constructed as scaled versions of the proposed shoreline stabilization project. For wave models, scale is determined by Froude number scaling. The Froude scaling law is applied to models where gravity is the predominant factor in the fluid motion. The primary concern is to ensure that the main aspects of wave-structure interaction are reproduce at a scale that avoids significant scale effects and can be constructed in an available flume or basin.

Based on these considerations two different model scales were selected.

- For assessment of transmission through and over the various cross-section types in the flume, a 1:30 (model:prototype) scale was used.
- For stability testing of the different beach protection structures as well as morphology testing in the 3D basin, a 1:35 (model:prototype) scale was used.

Rock materials for transmission and stability testing were selected to scale the permeability of the breakwater cross-section correctly and to reproduce the stability of the armor layer under wave attack.

2-Dimensional Flume Testing

Flume testing was utilized to evaluate wave transmission through and over various detached breakwater cross-section types as well as the stability of different armor size and slope combinations for an emergent detached breakwater. The data collected from the 2-D flume testing was utilized to optimize the breakwater cross-sections for 3-D physical model testing.

Transmission testing evaluated four different structure cross-sections: Emergent Breakwaters, Submerged Breakwaters, and two habitat enhancing cross sections: Fish Street/Fish Finger and Habitat/Lee-Side Pond.

- The emergent testing evaluated the transmission through and over an emergent cross-section, with varying crest width and permeability, at various water levels and wave heights.
- Submerged testing evaluated the effect of the seaward slope on the transmission of waves through and over a submerged breakwater cross-section.
- Fish Street/Fish Finger testing evaluated the effect that various lengths of the fish street structures have on the transmission of waves through and over an emergent breakwater cross-section.
- Habitat/Lee-Side Pond testing evaluated the effect that habitat structures constructed in the lee of an emergent breakwater have on the transmission of waves.

Transmission tests were evaluated at various freeboard and wave heights. Freeboard was altered in 1ft increments and wave heights of 6-10 feet were tested with a corresponding 8 second period. This wave environment is considered a storm event at the site and therefore small wave environments will result in higher transmission results.

Stability testing focused on testing the stability of armor on an offshore breakwater where large waves are breaking directly on the front slope. From schematic design, the deepest contour lakeward where a breakwater might be constructed was 565. Water levels corresponding to low, high, extreme 10yr, and extreme 100yr were tested. Depth limited waves up to 12.1 feet breaking on the front slope were tested on an emergent breakwater structure with a crest elevation of 586. Armor stone of the following gradations and front slope were tested: 6-9 tons at 1V:1.5H and 3-6 tons at 1V:2H.

3-Dimensional Model Testing

Physical modeling of the proposed beach control structures and beach was performed to refine the schematic design concepts in each of the three project areas. The main parameters used to evaluate the performance of the beach control structures were beach plan shape and stability of the armor layer of the rock beach control structures. For each Area, the team defined objectives that guided the physical modeling as summarized below:

- Area 1: This area was designed to work as a closed cell due to the lack of sediment supply due to the marina to the north. The structures work to achieve a linear uniform beach width with minimal loss of sediment from the cell.
- Area 2: The structures in this area were designed to reduce the wave energy at the shoreline and reduce the sediment transport rate along the areas of importance. This area remains as an open littoral cell.
- Area 3: The structures in this area were designed to achieve minimum sustainable beach widths along the shoreline with wider pockets at strategic use areas. The goal was to minimize impacts to the viewshed from the conference center and protect the string of wetlands to the south. This area remains as an open littoral cell.

The physical model basin consisted of a mortar bathymetry representing existing seabed conditions up to the 570 contour, a moveable bed sediment to represent the dynamic beach zone above the 570 contour, and the beach control structures. The physical model was constructed as a scaled version of the proposed shoreline stabilization project.

Testing was broken into two sets of tests.

- ‘Design Series’ – Morphological tests using an anthracite beach to optimize the layout of the beach control structures. The beach control structures were represented by sandbags which could be easily moved and manipulated to represent several shapes and lengths. Optimization performance of the beach control structure was assessed by comparing beach plan shape in the lee of the structures and conducting observations using dye tracing, to identify the potential for high currents.
- ‘Confirmation Series’ – Beach response tests using a sand beach followed by stability testing of the beach control structures under storm conditions. The beach control structures were built by hand to model scale using scaled armor stone to assess stability. The structures were not repaired between tests resulting in cumulative damage. Tests were run for a duration of 5hrs prototype and damage was assessed by armor layer displacement.

To document the results of each test and the resulting performance a 3-D laser scanner was used to track the evolution of the Still Water Line (SWL), providing a continuous beach plan shape, and track the position of the beach control structures. Additionally, time-lapse photographs were taken to monitor the movement of SWL and plan shape of the beach. Armor stability was measured by approximating rock displacement based on photographic overlays. Finally, dye tracing observations were made by SmithGroup to observe cross-shore currents and assess the potential for both high currents and sediment transport at key features of the beach control structures and Kellogg Creek.

Different wave and water level conditions were utilized to conduct the morphological and stability tests. For the morphological tests, the conditions had to be energetic enough to mobilize the model sediment and make the beach respond within a reasonable time without causing non-typical responses. Multiple wave heights were tested with the mobile beach present to assess the ability of the wave to mobilize the beach. The following wave conditions and water level were used in the morphological testing:

Table 1 Morphological Test Conditions (at the 565 IGLD85 contour)

Area	Still Water Level (feet IGLD 85)	Wave Height, Hmo (feet)	Wave Period, Tp (seconds)	Nearshore Direction (degrees)
Area 1	582.4	4.9	9	61
Areas 2 & 3	582.4	4.9	9	65

For stability testing of all structures, the wave and water level conditions were varied in 6 cases, looking at 3 water levels and 2 wave directions. The water levels included low, high, and extreme elevations and wave conditions mimicked a 10-year and 100-year return period events from the northeast and southeast. Each case was conducted to simulate a minimum of 30 hours of storm conditions. The test cases are summarized in the Table below.

Table 2 Stability Test Conditions (at the 565 IGLD85 contour)

Case	Return Period	Still Water Level (feet IGLD 85)	Wave Height, Hmo (feet)	Wave Period, Tp (seconds)	Offshore Direction
WC_Low_WL	100 yr	576	6.6	11.5	NNE
WC_High_WL	100 yr	583.3	11.2	11.5	NNE
WC_Ext_WL 01	10 yr	585.2	11.2	10	NNE
WC_Ext_WL 02	100 yr	585.2	12.1	11.5	NNE
WC_Low_WL_SE	100 yr	579.2	6.2	7.5	SE
WC_High_WL_SE	100 yr	582.2	6.2	7.5	SE

For Area 1, optimization assessed 22 different configurations resulting in a total of 10 beach control structures for the final configuration. This final configuration was assessed in stability and morphological testing. Morphological testing showed that the proposed structures can maintain a stable beach with heavily reduced north to south littoral drift. Stability testing in Area 1 informed final decisions regarding allowable damage and armor sizing. It was also noted that under extreme high water, there was a noted landward movement of sediment through overwash of the beach profile.

For Area 2, optimization assessed 17 different configurations of the beach control structures and included the Healthy Ports Futures pilot project structures. The final configuration included seven beach control structures. Morphological testing of the final configuration found that the beach control structures can maintain a stable beach with reduced north to south littoral drift. Stability testing in Area 2 noted many areas of damage, though limited ‘failure’, and informed decisions regarding armor selection.

For Area 3, optimization assessed 34 different configurations resulting in a total of 5 beach control structures for the final configuration. Morphological testing showed that the proposed structures can maintain a stable beach with reduced north to south littoral drift. Stability testing in Area 3 indicated that all structures of the final configuration remain substantially intact. The Stability testing demonstrated the importance of ensuring underlayer and core support for the Armor layer. The rock on the crest of one breakwater was not sufficiently supported by the underlayer rock on the leeward side and overtopping waves pushed the crest rocks over the leeside of the structure.

Select layouts within each area were laser scanned enabling the design team to develop a more refined OPCC.

Findings from the Physical Modeling

2-Dimensional Flume Testing

1. Emergent Breakwater
    a. All variations showed a decrease in transmission as the freeboard increases.
    b. The reduced transmission for a wider crest appears to be independent of the freeboard.
        i. Wider crest is most beneficial when freeboard is at least half of the wave height. This significantly increases the cost of the structure.
    c. The reduced transmission for an impermeable crest appears to be independent of freeboard.
        i. Impermeable crest is always better than permeable crest in overtopping situations.
    d. Recommended Crest Width
        i. Minimum of three rock width
    e. Recommended Crest Height
        i. Permeable Breakwater
            1. For a storm wave height of 11.2' in high water, a freeboard of 35% of the wave height is needed to reduce transmission to desirable levels -> Permeable offshore breakwaters should have a crest elevation of 586 IGLD85
        ii. Impermeable Breakwater
            1. For a storm wave height of 11.2' in high water, a freeboard of 23% of the wave height is needed to reduce transmission to desirable levels -> Impermeable offshore breakwaters should have a crest elevation of 584.5 IGLD85
2. Submerged Breakwater
    a. Transmission decreases as the front slope is made flatter, but there is a diminishing return on a reduction in transmission for slopes flatter than 1:4 (vertical: horizontal).

i. Flattening the front slope exponentially increases cost for marginal transmission reduction.

b. Submerged breakwaters are much less effective at reducing wave transmission. Recommend submerging the crest elevation no more than 20% of the wave height.

c. Recommended Crest Height

i. Based on an 11.2' wave height, the recommended submergence is ~2 ft. Based on a high-water elevation of 582, the designed crest elevation of 'submerged' breakwaters is 580.

3. Fish Street/Fish Fingers

a. There is a reduction in transmission with increased Fish Street/Fish Finger length to 40 feet but increasing further to 60 feet does not result in further transmission reduction.

b. Recommended Crest Height

i. For a permeable breakwater with 40ft fish finger, the crest elevation can be at elevation of the water level to provide the same protection as a permeable emergent breakwater. Based on a high-water elevation of 582, the recommended crest elevation of a permeable breakwater with a series of 40ft fish fingers is 582.

c. Fish fingers need to be added in a series to create the desired habitat and each finger adds significant cost to the breakwater cross section

4. Habitat/Lee-Side Pond

a. Transmission testing indicates that an increase in distance between the crest of the lake-side emergent breakwater and the lee-side low crested breakwater results in a reduction in wave transmission.

b. Depth of space between the two breakwater structures does not significantly influence transmission results.

c. Recommended Crest Height

i. For a lee-side pond breakwater on the order of 90 feet OC from the lakeside main breakwater, the lakeside permeable breakwater should have a recommended height of 582. This height can be reduced to 580.5 if the breakwater is constructed with an impermeable core.

d. While a second breakwater and habitat cobble is extra cost, the reduction in main breakwater height helps offset this cost resulting in a marginal increase.
e. Provided the largest reduction in transmission compared to other options.

Stability testing found that the damage levels for the two weights and slopes on the emergent breakwater structure were low or very low: well below the limits for design given in the Rock Manual (CIRIA, 2007). These gradations became the upper limit of gradations tested within the 3D basin models.

3-Dimensional Model Testing

3D modeling within the model basins revealed several processes and challenges not observed within numerical modeling or 2D flume modeling. While the optimization resulted in numerous beach stabilization structure configurations being tested until, ultimately, a preferred configuration was selected, the 3D modeling revealed additional observations that allowed the design team to make more informed layout decisions for subsequent tests. The main findings that influenced the design are outlined below.

- End diffraction: the full effect of waves diffracting[1] around the structures was not faithfully represented within the numerical model due to its limitations. During the physical runs, this effect became more evident as the wave transformation and energy distribution was affected more clearly by the spacing between the structures than by the crest elevation and subsequent overtopping and transmission.
- Curvature of the structures: different curvatures were analyzed to assess the effects on the shoreline, the convex configuration, Figure 4, proved to be more effective at maintaining a more linear beach.

[1] i.e. wave energy spreads perpendicularly to the direction of propagation when the wave train encounters an obstacle.

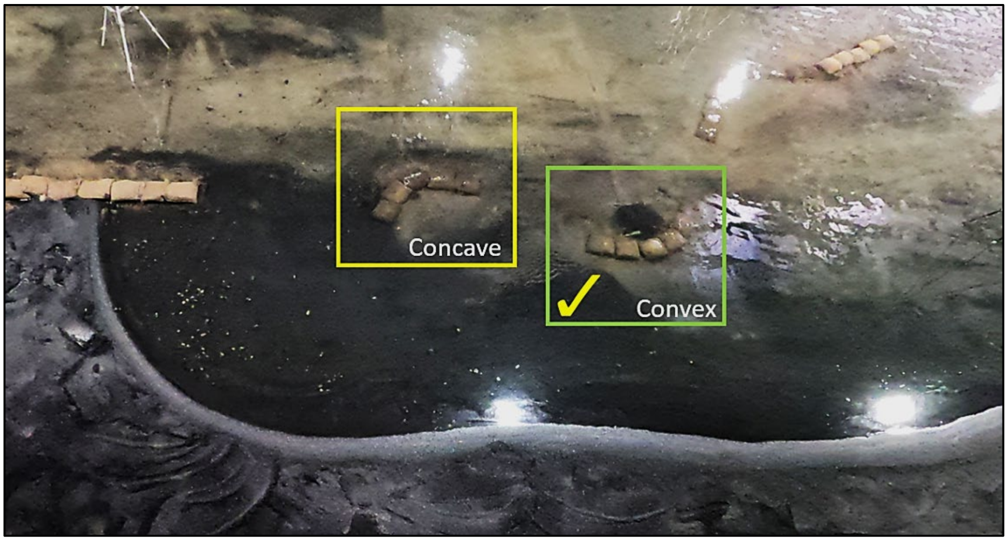

Figure 4: Different Curvatures Tested

- Submerged vs. Emergent: when a structure was performing exceedingly well at blocking the wave energy, and subsequently creating a salient, the team strategically lowered the crest of select segments with the goal of reducing cost while still maintaining functionality (Figure 5).

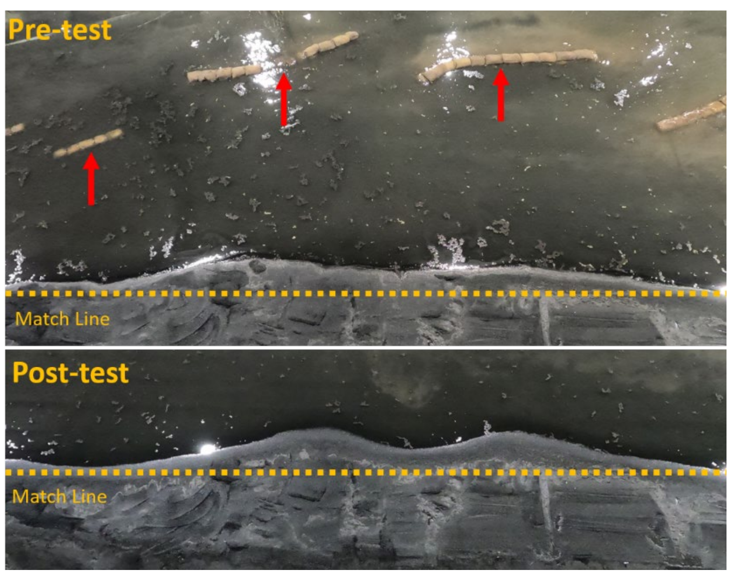

Figure 5: Submerged and Semi Submerged Structures

- Tombolos & Salients: while a salient is desirable in front of a critical area that needs more beach width, the forming of a tombolo is not ideal as it blocks the sediment transport downdrift of the structure, resulting in erosion. For this reason, the distance from the shoreline to the structures was carefully selected to avoid this problem at high water levels.

Figure 6: Example of a Tombolo Forming During the Laboratory Tests

- Addition of 'Fishtails': To enhance the diffraction of the waves behind some structures, an additional submerged segment was added to further re-direct the wave energy. This locally recreates an reverse current which inhibits sediment drift.

Design Development Layouts

Refinement of the design was based on the findings of the testing which resulted in the final layouts shown in Figure 7 through Figure 9

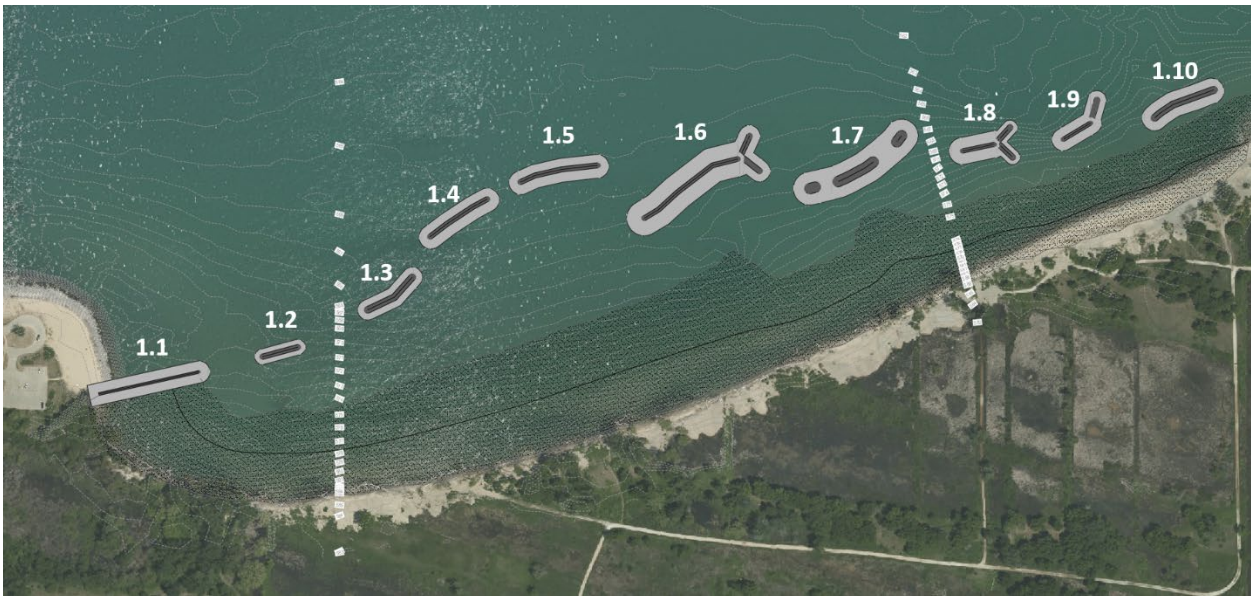

Figure 7:Design Development Layout, Area 1

Figure 8: Design Development Layout, Area 2

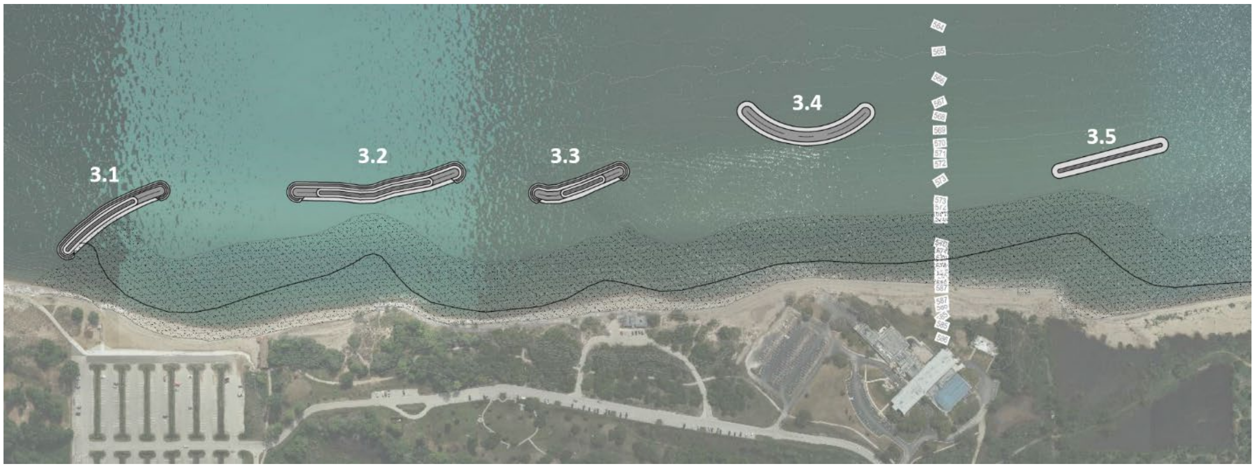

Figure 9: Design Development Layout, Area 3