ILLINOIS DEPARTMENT OF NATURAL RESOURCES

ILLINOIS BEACH STATE PARK

Shoreline Morphology Analysis & Stabilization Options

SMITHGROUP

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APPENDIX A

Illinois Beach State Park
Site Inventory Map

SITE INVENTORY & ANALYSIS: NORTH UNIT

SITE INVENTORY & ANALYSIS: SOUTH UNIT

APPENDIX B

Metocean Analysis

1. Metocean Analysis

It is essential that a good understanding of the lake climate at the area of study be developed. The morphological changes of the shoreline are caused directly by Lake Michigan and the yearly storm events that impact this area. Sediment transport along the western side of Lake Michigan is a normal, well-documented event yet the rate of transport potential varies. As development along the shoreline continues, this rate is also impacted by removal or addition of sediment to the littoral system.

1.1. Water Levels

NOAA maintained water level stations are located in Milwaukee, WI (Station ID: 9087057) and Calumet Harbor, IL (Station ID: 9087044). The project site is approximately midway between these two locations and therefore an interpretation between the two facilities has been approximated. Data was downloaded from https://tidesandcurrents.noaa.gov/map/ on June 27th, 2018.

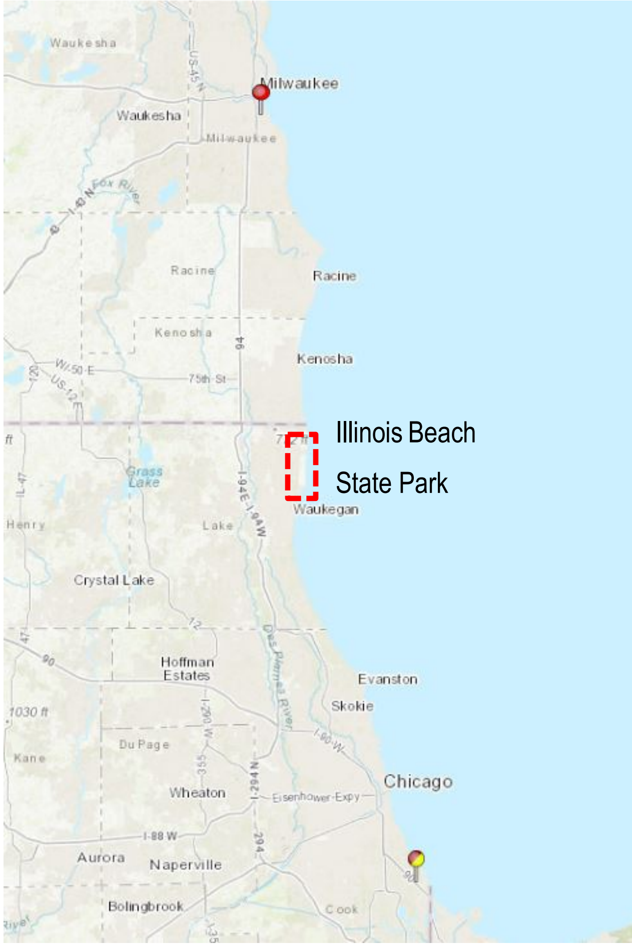


Figure B-1 NOAA Water Level Stations

Return period forecasts, statistical probabilities, for surge and flood events were developed through a Weibull distribution analysis using historical monthly average water levels and 6-minute peak water levels within a given month.

Table B-1 Water Level Analysis

	Milwaukee 9087057	Calumet Harbor 9087044	Illinois Beach (approximate)
Surge Return Periods (ft)**			
5 yr.	1.31	2.20	1.70
10 yr.	1.40	2.46	1.87
50 yr.	1.60	3.11	2.26
100 yr.	1.68	3.40	2.44
500 yr.	1.85	4.12	2.85
Yearly Monthly Peak Flood MSL Return Periods, IGLD85*			
5 yr.	581.31	580.91	581.13
10 yr.	581.67	581.28	581.50
50 yr.	582.34	582.02	582.20
100 yr.	582.59	582.30	582.46
500 yr.	583.10	582.89	583.01
Monthly MSL Water Level, IGLD85*			
Lowest Recorded	576.02	575.96	575.99
5%	577.10	576.74	576.94
15%	577.62	577.41	577.53
25%	578.09	577.83	577.97
50%	579.29	578.86	579.10
75%	580.18	579.72	579.98
85%	580.53	580.18	580.37
95%	581.25	580.85	581.07
Max Recorded	582.40	582.35	582.38

*All elevations reference International Great Lakes Datum, 1985

**"Surge" refers to changes in water level that are on a shorter time duration than one month. Surges on Lake Michigan range from 20 minutes to a few hours.

1.2. Winds

Historical recorded wind data was taken from the Wave Information Study (WIS) Station 94033 located offshore, approximately 4 miles east of the project site. This data includes roughly 36 years of data ranging from 1979 – 2014. The wind data was run through a Weibull distribution analysis to determine storm winds from 16 compass directions. The wind rose and return period storm winds in miles/hour are given below.

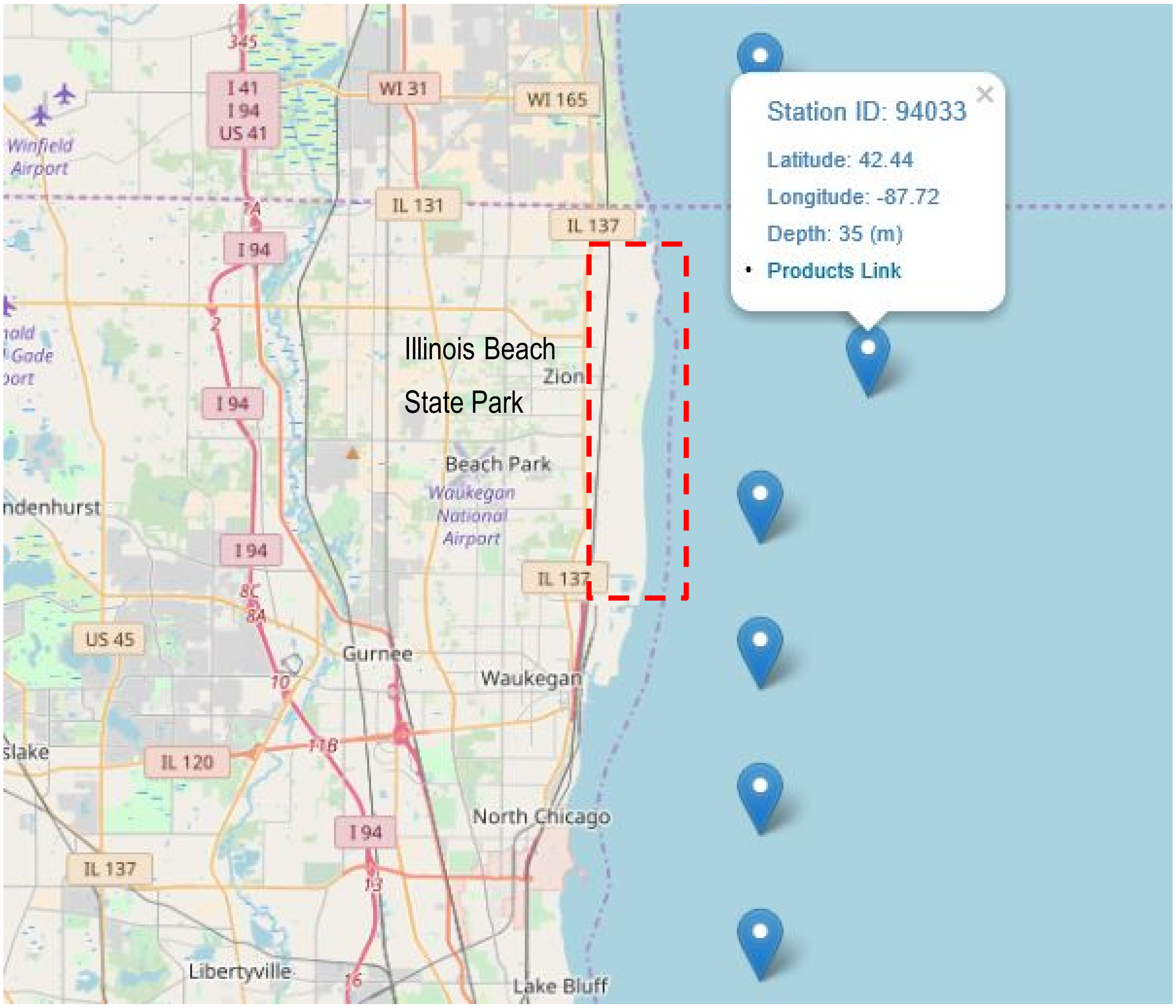


Figure B-2 WIS Station Used in Analysis

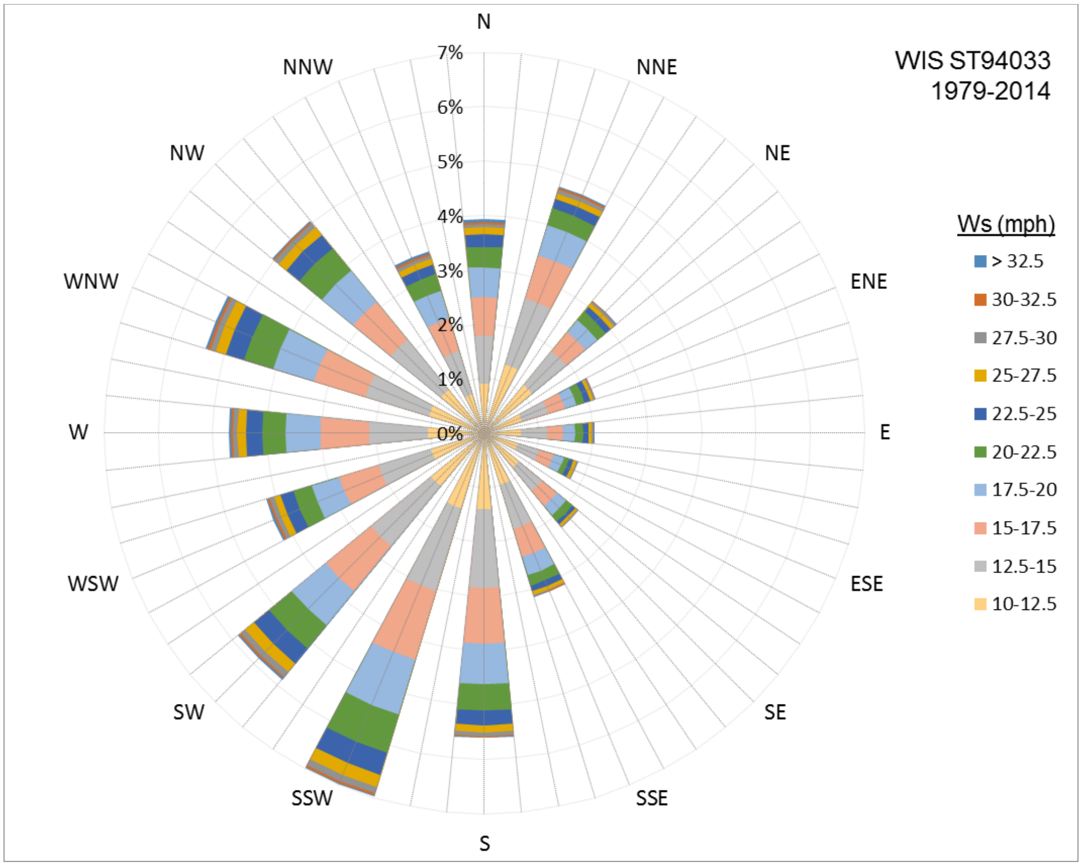


Figure B-3 Wind Rose

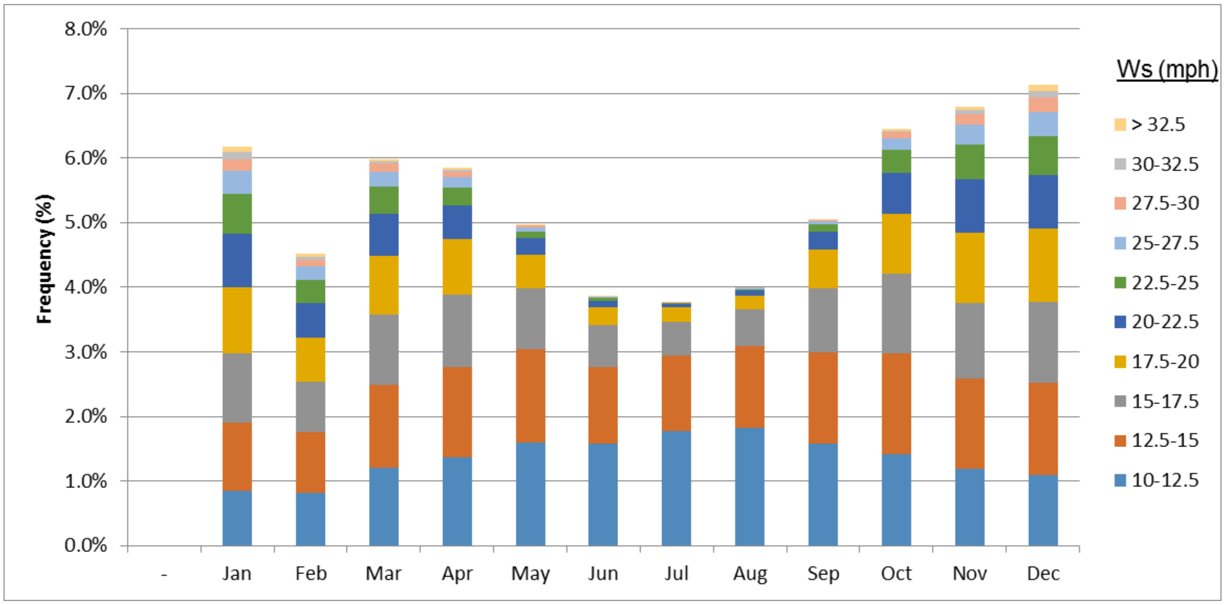


Figure B-4 Wind Speed Frequency by Month

Table B-2 Return Period Wind Speeds by Direction (mph)

Return Periods	N	NNE	NE	ENE	E	ESE	SE	SSE
1 yr.	31.60	30.74	28.02	27.39	26.70	26.72	27.09	28.45
10 yr.	40.45	37.35	36.65	35.32	36.67	33.96	33.39	33.87
25 yr.	43.44	40.45	39.59	38.90	40.37	36.17	35.25	35.88
50 yr.	45.62	42.89	41.75	41.68	43.13	37.75	36.59	37.38
100 yr.	47.76	45.40	43.87	44.53	45.87	39.28	37.87	38.86

Return Periods	S	SSW	SW	WSW	W	WNW	NW	NNW
1 yr.	30.52	31.65	31.01	31.16	30.46	31.99	30.04	31.65
10 yr.	35.18	35.72	39.28	38.53	39.03	37.68	38.46	40.39
25 yr.	36.41	37.13	41.65	39.86	41.24	40.84	40.83	44.08
50 yr.	37.26	38.16	43.33	40.74	42.78	43.43	42.51	46.92
100 yr.	38.07	39.17	44.93	41.55	44.24	46.16	44.11	49.78

1.3. Waves

Offshore wave conditions for the site were collected from two sources: USACE's Wave Information Studies (WIS) and Great Lakes Observing System (GLOS). Both of these data sources are based on numerical modeling results for various points throughout the lake. Real-time data is collected from established buoys anchored in each of the Great Lakes and used to drive the numerical models. Each model has gone through an extensive calibration process by the USACE and NOAA respectively.

WIS

Information for WIS Station 94033 is provided in the preceding section. The wave data was run through a Weibull Distribution Analysis to determine storm winds from 8 compass directions that can impact the site. The wave rose and return period storm wave characteristics in feet are given below.

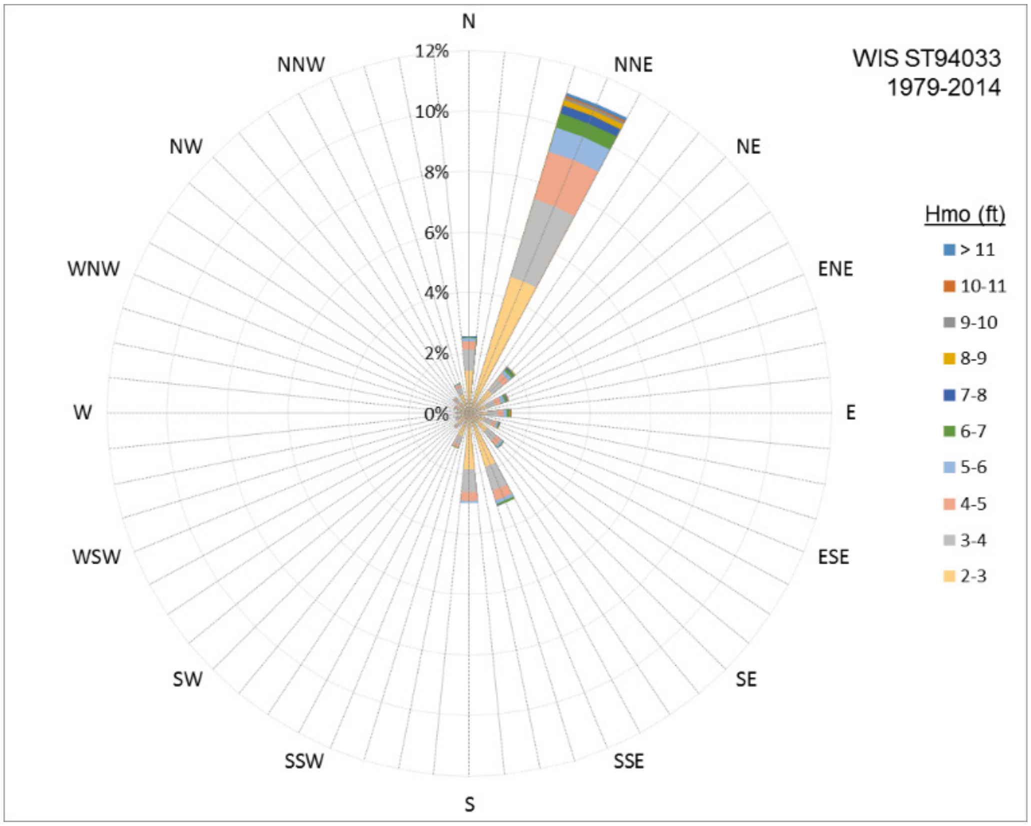


Figure B-5 Wave Rose, WIS

Table B-3 Return Period Wave Height by Direction (feet), WIS

Return Periods	N	NNE	NE	ENE	E	ESE	SE	SSE
1 yr	6.70	10.05	7.50	6.74	6.68	5.82	5.69	6.21
10 yr	10.17	16.11	10.68	10.72	11.31	9.12	7.96	7.82
25 yr	11.62	17.93	12.92	12.73	12.98	10.24	8.51	8.31
50 yr	12.73	19.23	14.89	14.33	14.21	11.07	8.89	8.66
100 yr	13.84	20.47	17.07	16.00	15.43	11.88	9.25	9.00

*Highlighting represents return period used in numerical modeling analysis

Table B-4 Wave Period Frequency by Wave Height, WIS

Hmo (ft) \ Tp (s)	0-5	5-6	6-7	7-8	8-9	9-10	10-11	11-12	12-13
<= 6	78.658%	12.351%	5.771%	0.957%	0.324%	0.053%	0.002%	0.000%	0.000%
6-7	0.006%	0.154%	0.448%	0.159%	0.103%	0.035%	0.004%	0.000%	0.000%
7-8	0.000%	0.015%	0.185%	0.108%	0.062%	0.032%	0.010%	0.000%	0.000%
8-9	0.000%	0.003%	0.078%	0.074%	0.057%	0.035%	0.005%	0.001%	0.000%
9-10	0.000%	0.000%	0.027%	0.032%	0.033%	0.020%	0.005%	0.002%	0.000%
10-11	0.000%	0.000%	0.005%	0.015%	0.021%	0.018%	0.010%	0.002%	0.000%
11-12	0.000%	0.000%	0.001%	0.006%	0.008%	0.018%	0.006%	0.002%	0.000%
12-13	0.000%	0.000%	0.000%	0.001%	0.006%	0.014%	0.009%	0.003%	0.000%
13-14	0.000%	0.000%	0.000%	0.000%	0.003%	0.005%	0.005%	0.002%	0.000%
14-15	0.000%	0.000%	0.000%	0.000%	0.002%	0.002%	0.003%	0.003%	0.000%
> 15	0.000%	0.000%	0.000%	0.000%	0.002%	0.003%	0.011%	0.005%	0.000%

*Shading indicates higher percentages of entire data set. Boxed cells indicate highest percentage bin per wave height.

GLOS

Historical recorded wave data was taken from GLOS Point 42.4407 N, -87.7206 W located offshore, approximately 4.5 miles northeast of the project site. This data includes roughly 10 years of data ranging from 2008 – 2018. The wave data was run through a Weibull Distribution Analysis to determine storm winds from 8 compass directions that can impact the site. The wave rose and return period storm wave characteristics in feet are given below.

Figure B-6 GLOS Point Chosen for Site

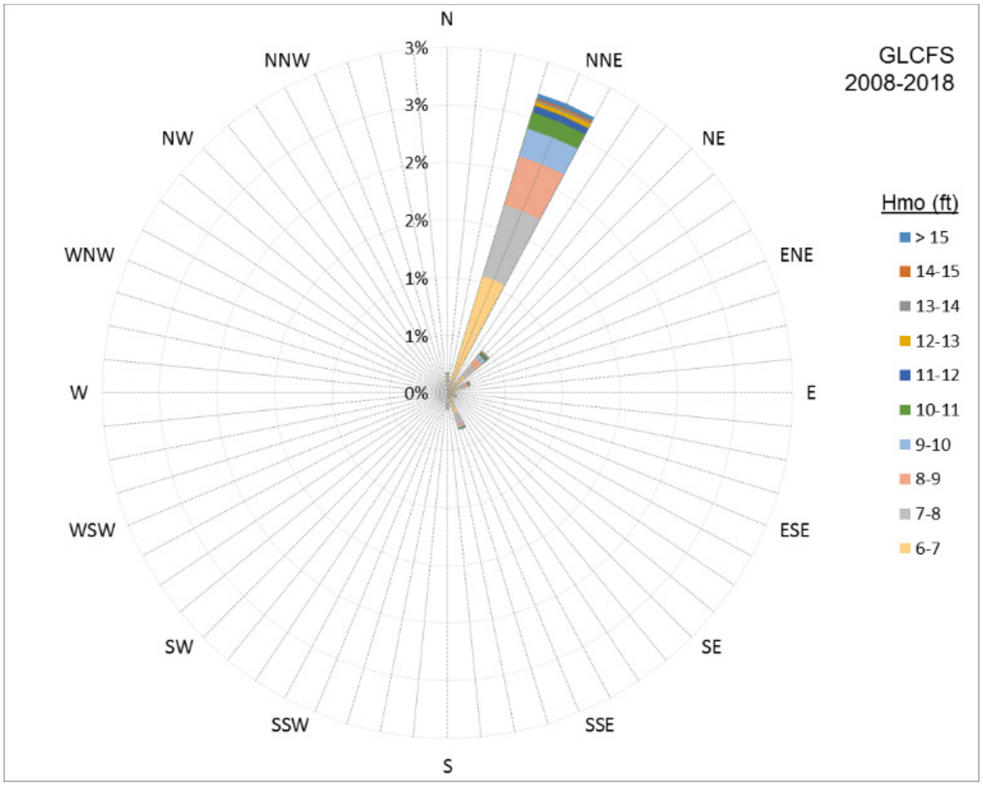


Figure B-7 Wave Rose, GLOS

Table B-5 Return Period Wave Height by Direction (feet), GLOS

Return Periods	N	NNE	NE	ENE	E	ESE	SE	SSE
1 yr	7.81	10.40	9.81	6.88	4.74	5.63	6.32	7.47
10 yr	11.33	18.09	14.51	12.34	7.76	10.00	9.06	10.93
25 yr	12.51	19.52	18.75	14.74	8.00	11.52	10.74	11.79
50 yr	13.37	20.47	22.82	16.59	8.15	12.64	12.16	12.38
100 yr	14.21	21.33	27.61	18.48	8.28	13.74	13.69	12.93

*Highlighting represents return period used in numerical modeling analysis

Table B-6 Wave Period Frequency by Wave Height, GLOS

Hmo (ft) \ Tp (s)	0-5	5-6	6-7	7-8	8-9	9-10	10-11	11-12	12-13
<= 6	88.391%	6.825%	0.460%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%
6-7	0.000%	1.224%	0.678%	0.015%	0.000%	0.000%	0.000%	0.000%	0.000%
7-8	0.000%	0.132%	0.879%	0.062%	0.000%	0.000%	0.000%	0.000%	0.000%
8-9	0.000%	0.001%	0.507%	0.096%	0.000%	0.000%	0.000%	0.000%	0.000%
9-10	0.000%	0.000%	0.201%	0.115%	0.003%	0.000%	0.000%	0.000%	0.000%
10-11	0.000%	0.000%	0.021%	0.154%	0.015%	0.000%	0.000%	0.000%	0.000%
11-12	0.000%	0.000%	0.000%	0.060%	0.019%	0.003%	0.000%	0.000%	0.000%
12-13	0.000%	0.000%	0.000%	0.024%	0.026%	0.003%	0.000%	0.000%	0.000%
13-14	0.000%	0.000%	0.000%	0.004%	0.015%	0.001%	0.000%	0.000%	0.000%
14-15	0.000%	0.000%	0.000%	0.000%	0.015%	0.001%	0.000%	0.000%	0.000%
> 15	0.000%	0.000%	0.000%	0.000%	0.018%	0.029%	0.000%	0.000%	0.000%

*Shading indicates higher percentages of entire data set. Boxed cells indicate highest percentage bin per wave height.

1.4. Ice

During the winter months, ice forms along the shorelines and extends out into the lake. The thickness of the ice is a function of many factors including temperature, sunlight, snow insulation, cracking & refreezing, water movement, etc. As these variables change year to year, it is impossible to accurately estimate ice thickness without obtaining core samples. In lieu of samples, the USACE recommends using the Stefan Equation to estimate ice thickness. This simple equation uses accumulated freezing degree days (AFDD) and a coefficient based on ice cover condition to estimate thickness. This method can be calibrated to known data, if available, to further refine the estimation.

Daily average temperature was collected from Waukegan National Airport, located approximately 3 miles southwest of the project site. Temperature data includes roughly 29 years of data ranging from 1989 – 2018. The coldest winter on record during this time occurred in 2013-2014. Based on the top 5 coldest winters on record, it is recommended that ice thickness of 28 inches be used for design.

APPENDIX C

Modeled Alternatives

1. Modeled Alternatives

This section includes the various modeled alternatives explored and their resulting shorelines after 5 representative years of wave conditions and water levels by using the coastal evolution model.

1.1. Area 1

Two hard points north and south of this area hold the shoreline at a fixed position, resulting in a non-equilibrium pocket beach. If allowed to erode, the shoreline would result in additional retreat of approximately 150 ft.

Figure C-1: Five year shoreline projection without mitigation, Area 1 (map data: Google, USDA Farm Service Agency)

1.1.1. Alternative 1.1

Alternative 1.1 consists of a north L-shaped groin that would move that wave-diffraction point farther off-shore, a detached breakwater oriented to block the prevalent incoming waves and create a shadow zone behind it, and a small spur to retain the sediment that may moves out of the cell. Since this is already a starved beach and little sediment comes from the north, this solution also includes pre-filling of sand that will be re-distributed until the shoreline reaches its equilibrium position.

Results of this alternative (Figure C-2) show that there is accumulation on the south half of the beach, however, the opening between the L-head groin and the offshore breakwater is wide enough so that the equilibrium position is further back than desired.

Figure C-2: Alternative 1.1 with shoreline recession after five years

1.1.2. Alternative 1.2

Alternative 1.2 consists of an array of detached offshore breakwaters oriented towards the NE that would block the prevalent incoming waves, sheltering the shoreline immediately behind it. These diffracted waves have a reduced wave height and change direction locally. Pre-filling of the nearshore is required and will be redistributed until the shoreline reaches equilibrium.

The results of this alternative (Figure C-3) are showing that the structures hold the coastline position reasonably well with sediment accumulation south of the area.

Figure C-3: Alternative 1.2 with shoreline recession after five years

1.1.3. Alternative 1.3

Alternative 1.3 explores the addition of a north submerged breakwater that would diffract the waves, modifying their direction and affecting the final shape of the beach. A T-head groin was proposed at the southern limits to extend the second diffraction point offshore and, together with an L-head groin, form a recreational beach between stations 44+00 and 50+00. The public parking area is adjacent to this beach providing an additional recreational opportunity.

Alternative 1.3 shows that by moving the diffracting point south and leaving a large opening before the next structure only changes the localized erosion point, therefore more structures are needed along this stretch. On the other hand, the small opening between the T-head groin and the L-groin are holding the new proposed beach in place.

Figure C-4: Alternative 1.3 with shoreline recession after five years

1.1.4. Alternative 1.4

Alternative 1.4 was developed after reviewing the results of alternative 1.2 with the goal of reducing the littoral drift by 50%. Since the sediment accumulation at the south area was not required, one of the structures was eliminated. The result shows that the structures better than alternative 1.3 at holding the coastline, however, there's still recession south of the marina breakwater.

Figure C-5: Alternative 1.4 with shoreline recession after five years

1.1.5. Alternative 1.5

The fifth structure in alternative 1.4 created a surplus of sediment in the south side of Area 1, therefore the length of the structure was reduced, which also results in a less costly alternative.

Figure C-6: Alternative 1.5 with shoreline recession after ten and twenty years

1.1.6. Alternative 1.6

Since alternative 1.5 was still resulting in some erosion immediately south of the marina, the team decided to move the first offshore breakwater and attach it to the existing marina breakwater, the remaining offshore breakwaters were redistributed and the final model runs show that this configuration was the most efficient at holding coastline in place.

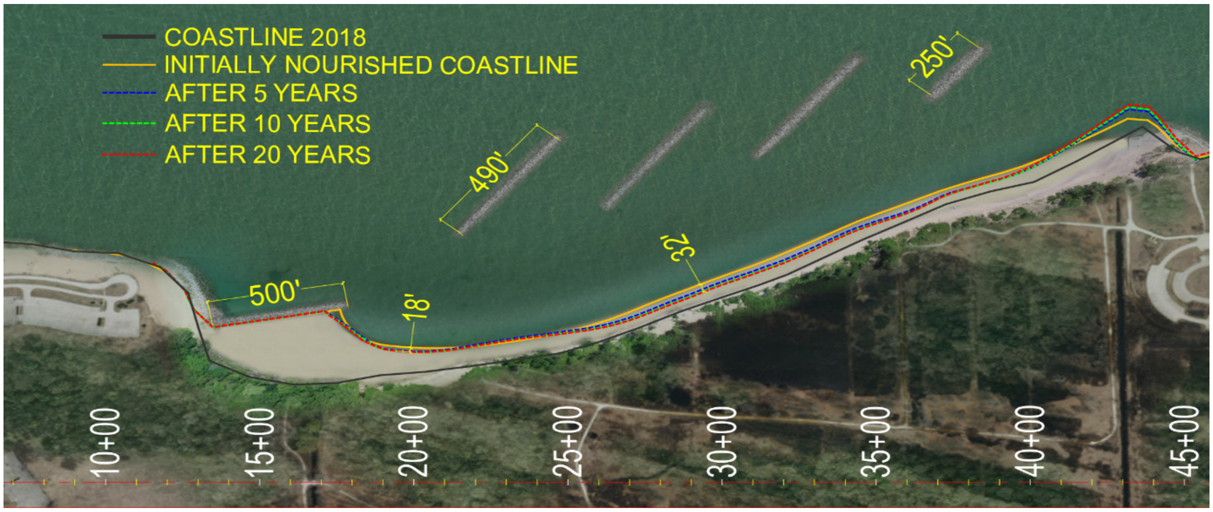


Figure C-7: Alternative 1.6 with shoreline recession after ten and twenty years

1.2. Area 2

The alternatives for this area are designed to protect the shoreline adjacent to the Lake County Public Water District Intake (between stations 70+00 and 95+00) which has a hardened revetment edge. The shoreline to the south is natural and unprotected. The coastline retreat in this area has intensified in the last few years due to the high-water level conditions.

Figure C-8: Five year shoreline projection without mitigation, Area 2 (map data: Google, USDA Farm Service Agency with Drone Overlay)

1.2.1. Alternative 2.1

Alternative 2.1 proposes a breakwater that would extend from the existing revetment at the intake building. This breakwater would be approximately 650 ft long and would create a protected area behind it. Pre-filling with a coarse grained sand is proposed as well. The more coarse the pre-fill, the less likely it is to be transported by wave action. A small L-shaped groin would also be constructed to anchor the southern end of the pre-fill.

While Alternative 2.1 indicates that this alternative holds the coastline in position between the two structures, the south groin is causing a negative effect downdrift that is not acceptable since this area is considered part of a nature preserve.

Figure C-9: Alternative 2.1 with shoreline recession after five years

1.2.2. Alternative 2.2

Alternative 2.2, reduces the length of the breakwater and includes the addition of three detached offshore breakwaters. The breakwaters are oriented perpendicular to the NE wave direction creating a shadow area behind them. Pre-filling of the nearshore would also be required to increase the beach width. This nourishment would reshape over time until an equilibrium profile is obtained

Results for this alternative show that the offshore breakwaters are effective in protecting the shoreline behind them. However, the area adjacent to the north revetment is still experiencing erosion, indicating its length is not adequate to protect this area.

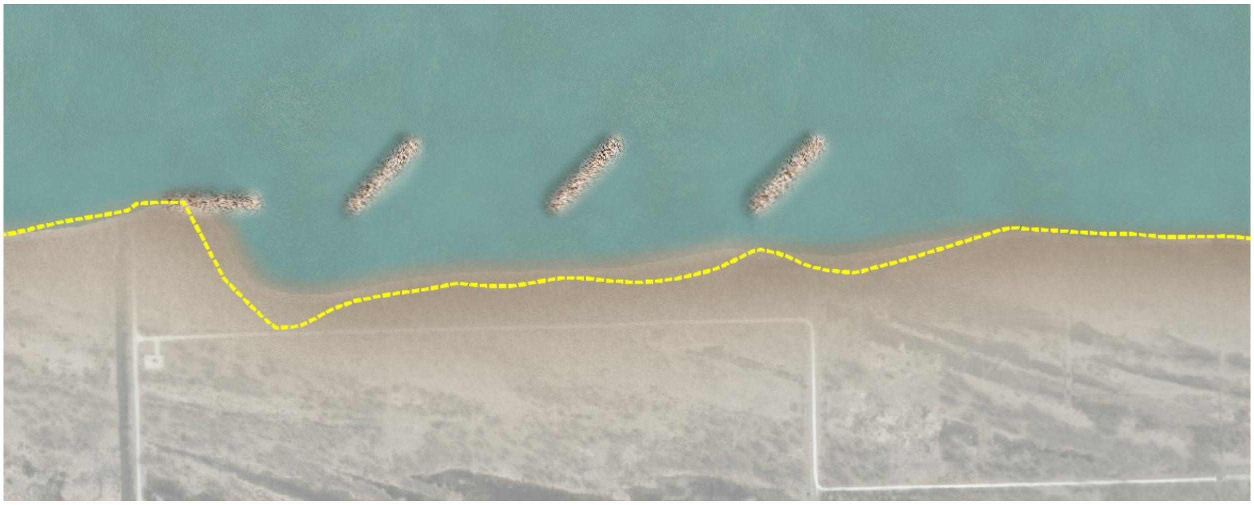


Figure C-10: Alternative 2.2 with shoreline recession after five years

1.2.3. Alternative 2.3

After the coastal evolution model results were completed (Section 9), a third alternative was proposed that includes the 650 long breakwater and reduces the number of offshore breakwaters to two. Pre-filling of the nearshore would still be required.

This third iteration for Area 2 shows that by keeping the length of the north revetment and the two offshore structures, they work well at holding the coastline position while not showing the same effect downdrift as a shore-perpendicular structure.

Figure C-11: Alternative 2.3 with shoreline recession after five years

1.2.4. Alternative 2.4

Alternative 2.4 explores the idea of adding an offshore breakwater parallel to the coastline. Model results show that sediment starts to accumulate behind the structure and the coastline retreats south of the third offshore breakwater is minimal. Even so, this option was discarded after a meeting with the stakeholders that expressed concern for potential damage to the existing Lake County Water District water intake pipe.

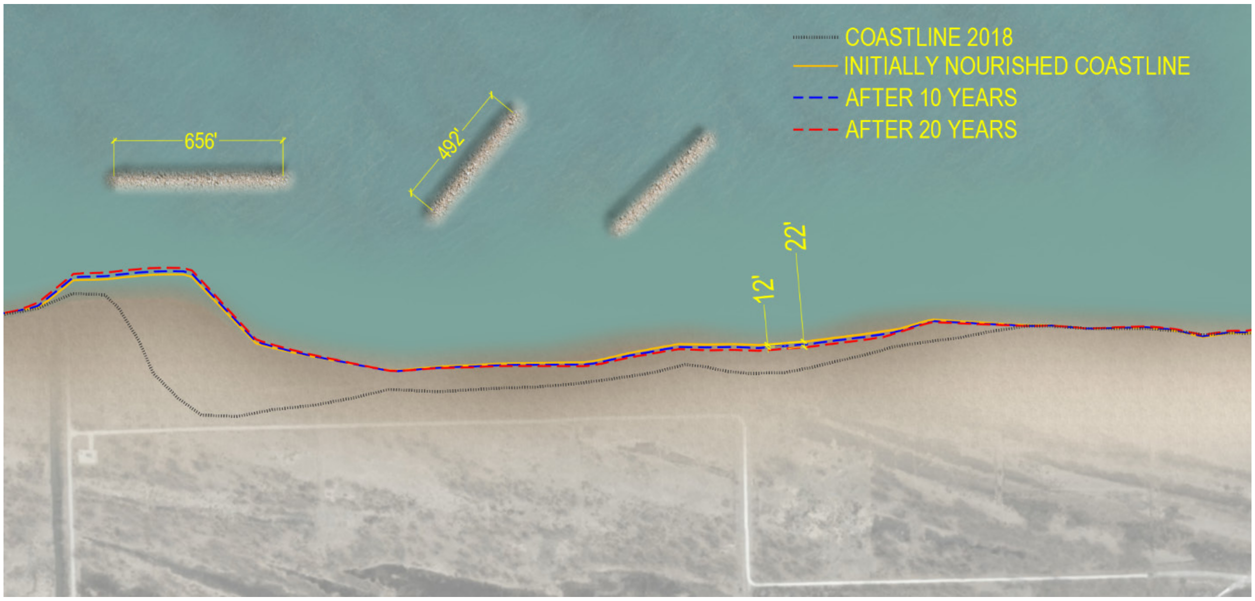


Figure C-12: Alternative 2.4 with shoreline recession after five years

1.2.5. Alternatives 2.5 – 2.7

Revisiting alternative 2.3 with offshore breakwater iterations, the team sought to assess the coastline evolution using large cobble as the nourished material instead of sand. These alternatives were discarded since cobble is not a desirable material for this area.

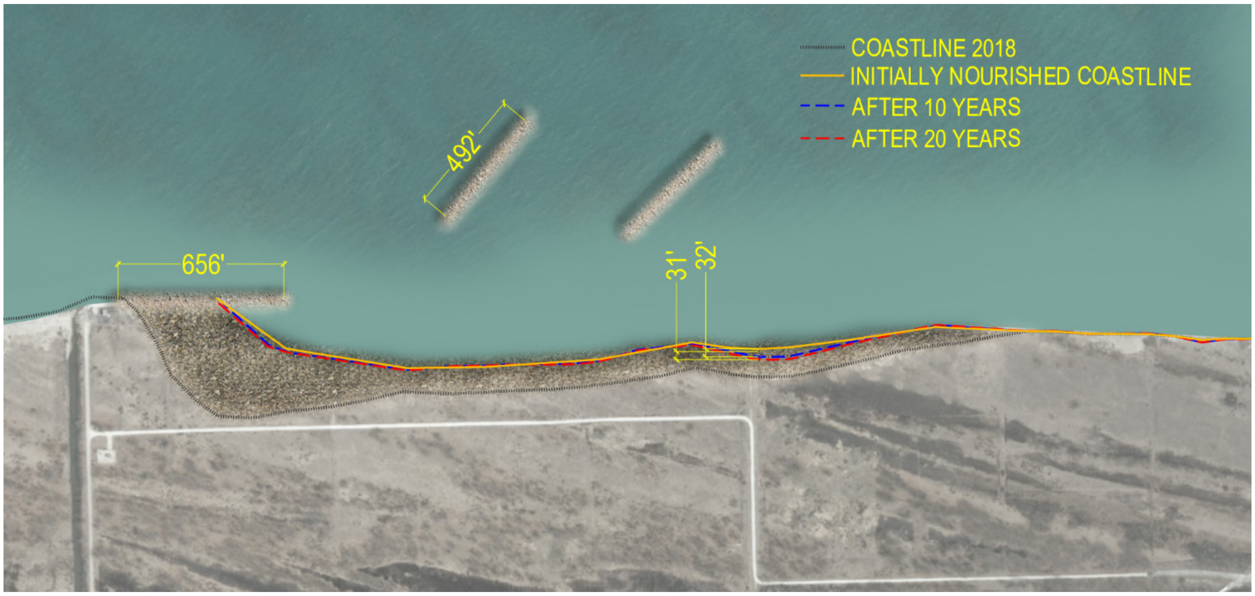


Figure C-13: Alternative 2.5 with shoreline recession after ten and twenty years

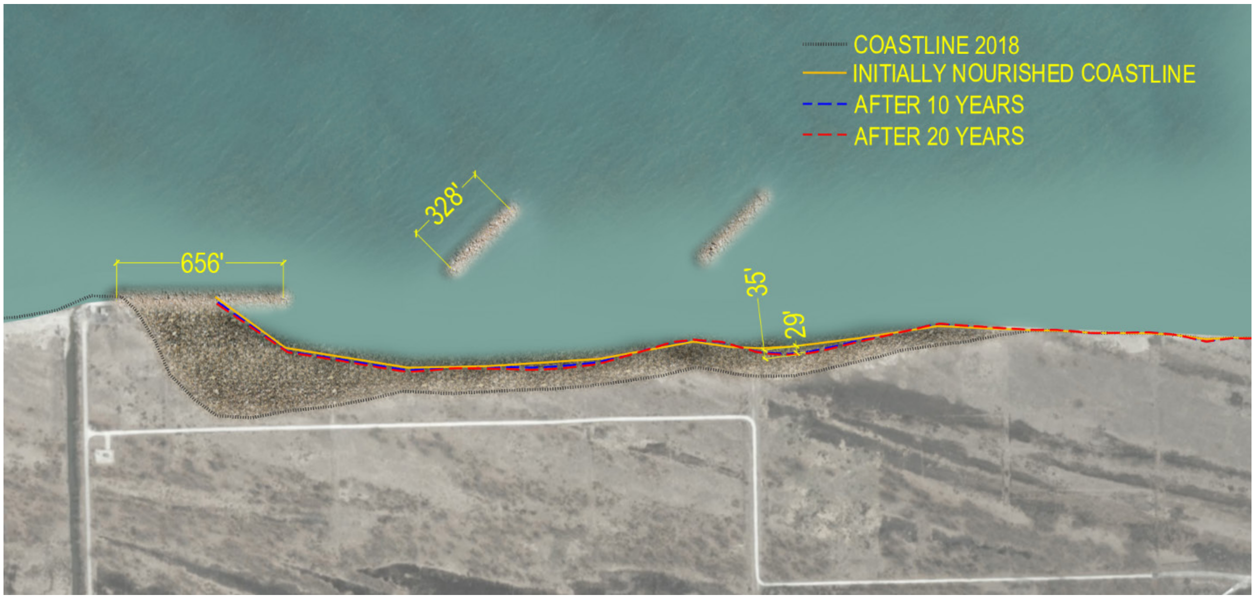


Figure C-14: Alternative 2.6 with shoreline recession after ten and twenty years

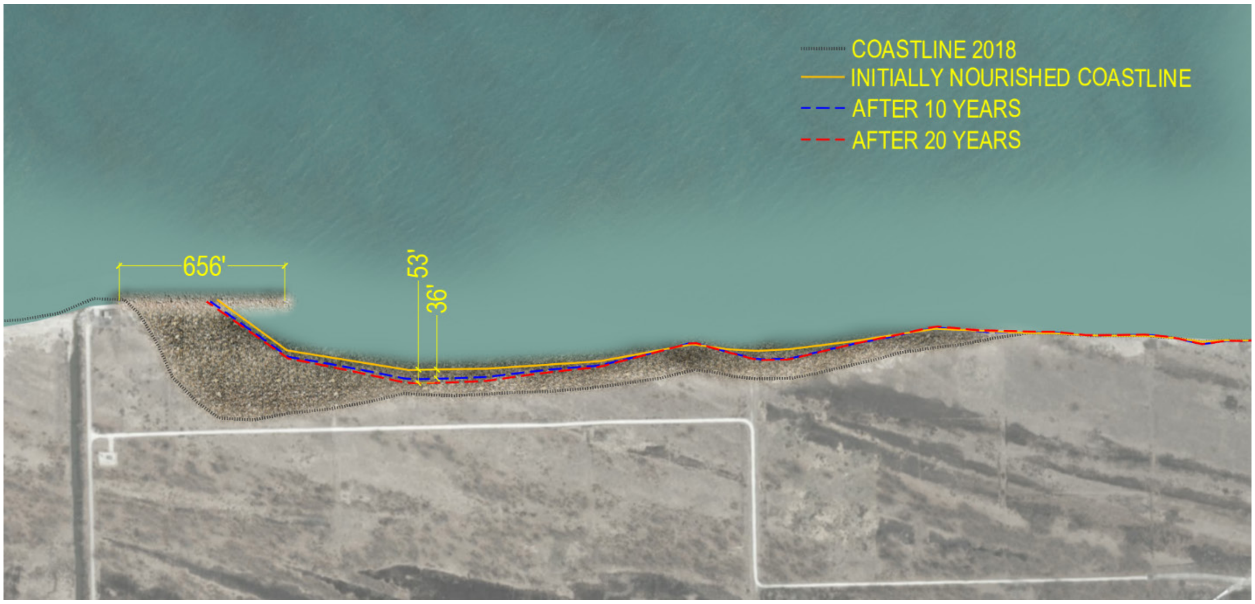


Figure C-15: Alternative 2.7 with shoreline recession after ten and twenty years

1.2.6. Alternative 2.8

Alternative 2.8 consists of a nearshore breakwater that connects to the existing revetment south of Kellogg Creek, two offshore breakwaters oriented to break the predominant wave direction, and a nearshore breakwater that promotes the formation of a tombolo behind it. This combination proved to be the most effective at reducing the littoral drift, stabilizing the coastline and avoids interfering with the existing Lake County Water District water intake pipe.

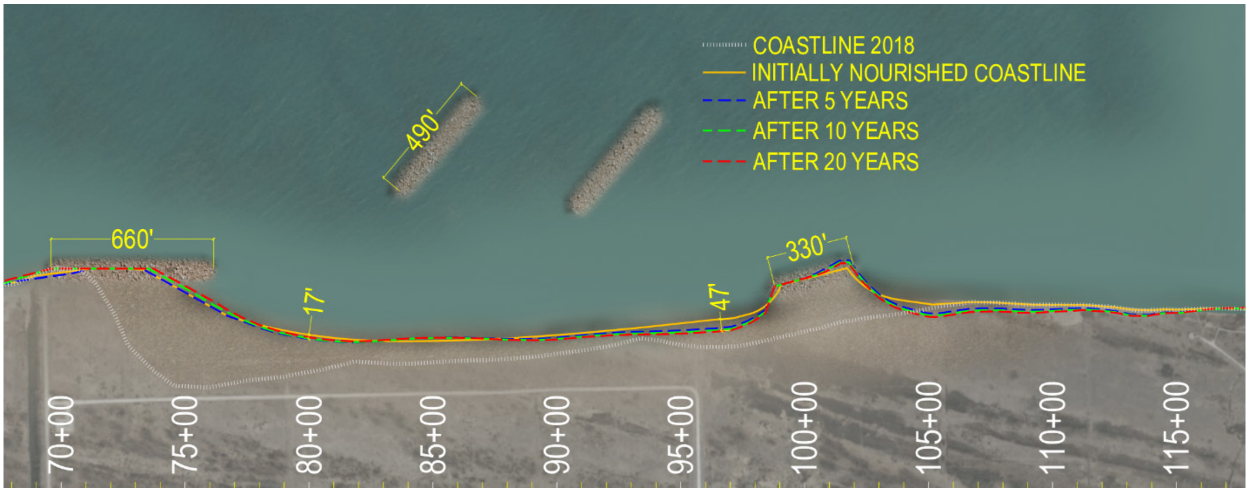


Figure C-16: Alternative 2.8 with shoreline recession after ten and twenty years

1.3. Area 3

Area 3, which represents the most visited section of Illinois Beach State Park, predominately has hardened edges to hold the shoreline. Beach sand is periodically nourished lakeward of these protections within the swimming beaches and is therefore able to erode downdrift.

A different approach to stabilizing the shoreline was undertaken in area 3 because of the location of the recreational beach and the understanding that an obstructed view would not be desirable. The stabilization strategy included submerged structures that would avoid obstructing the lake view for beach users.

Figure C-17: 5-year shoreline projection without mitigation, Area 3 (map data: Google, USDA Farm Service Agency)

1.3.1. Alternatives 3.1 and 3.2

Alternative 3.1 consists of pre-filling the nearshore, providing a submerged breakwater that would extend in 3 directions to protect the beach from all incoming waves, and a groin in front of the Illinois Beach Resort and Conference Center. This alternative would provide a stabilized beach in front of the parking lot and a smaller beach in front of the Conference Center just north of the groin.

Alternative 3.2 replaces the continuous U-shaped breakwater shown in 3.1 with smaller submerged breakwaters offshore of the recreational beach. A larger L-shaped groin is provided at the Convention Center to assess its effectiveness. Results of these alternatives show that the effect of both is similar and there's no need for the south L-head groin.

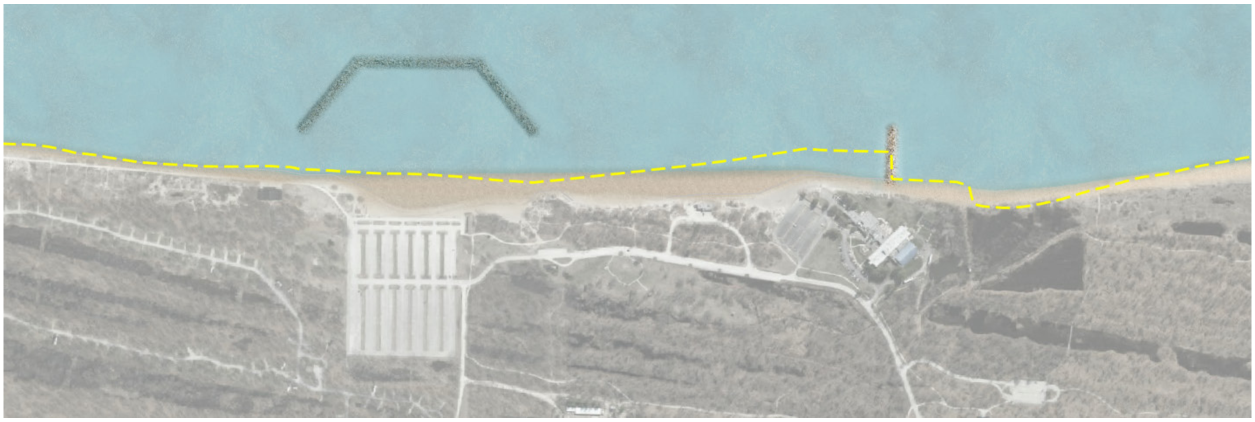


Figure C-18: Alternative 3.1 with shoreline recession after five years

Figure C-19: Alternative 3.2 with shoreline recession after five years

1.3.2. Alternative 3.3

The model results from alternative 3.2 indicated that fewer structures were required to protect the area in front of the recreational beach, and that by reconfiguring the south groin, potential negative effects downdrift could be avoided. Even though alternatives 3.1 and 3.2 show that the structures are effective in holding the coastline in front of the recreational beach, alternative 3.3 was developed with the intent to reduce costs by reducing the number of submerged structures and introducing a rotated south breakwater.

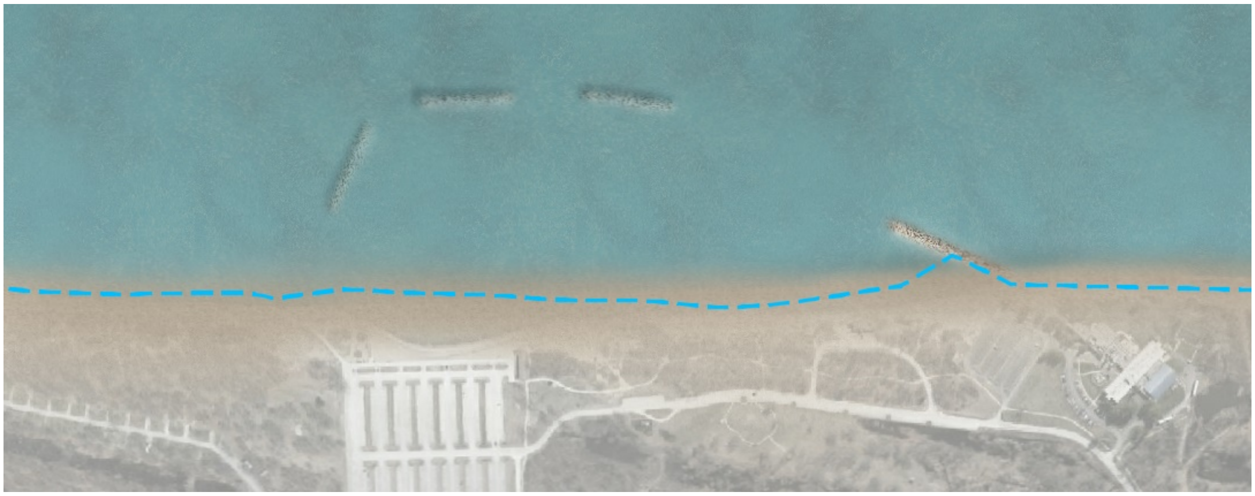


Figure C-20: Alternative 3.3 with shoreline recession after five years

1.3.3. Alternative 3.4

Alternative 3.4 consists of two angled shore attached breakwaters north and south of the area as well as two submerged breakwaters in front of the main swimming beach. While these structures achieve the goal of stabilizing the coastline in between, areas to the south also required stabilization, and so this configuration was enhanced and widened.

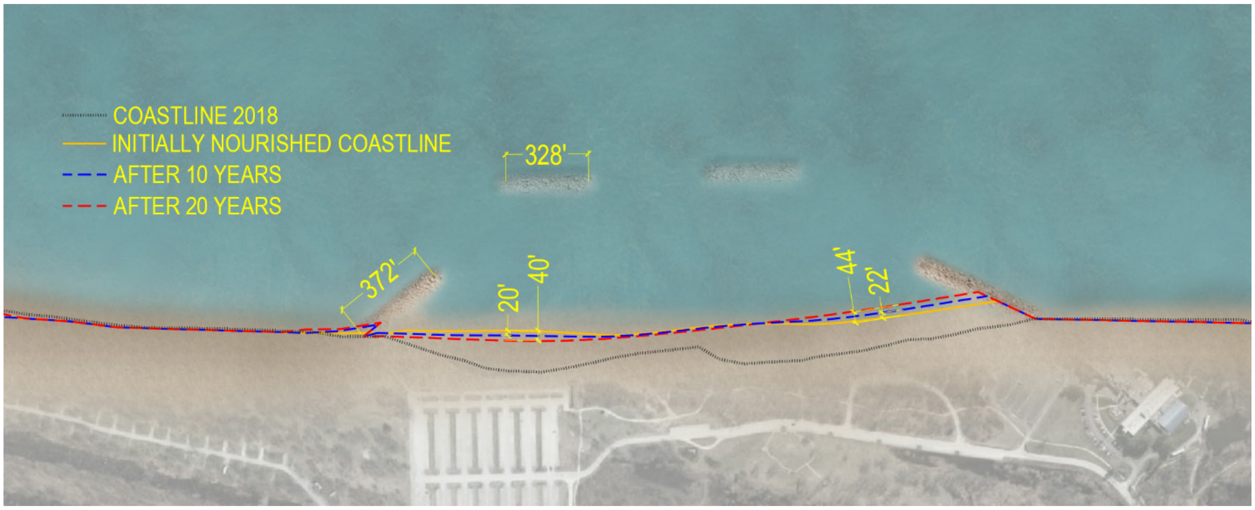


Figure C-21: Alternative 3.4 with shoreline recession after ten and twenty years

1.3.4. Alternative 3.5

The preferred alternative for Area 3, consists of two offshore submerged breakwaters which may become slightly emergent at low water. These breakwaters will cause passing waves to break, thereby reducing transport potential. To further maintain sand along this predominately recreational shoreline, updrift and downdrift structures create a closed cell trapping sand within. The northern structure is shore connected to an existing revetment and allows southerly transported sediment to enter the cell but stops any sediment from being pushed north by southerly waves. The southerly nearshore breakwater, similar to alternative 2.8, will be surrounded by sand at low water but 'offshore' at high water hindering down shore littoral drift. This nearshore breakwater was strategically located to provide the most protection for the valuable panne wetland in this area.

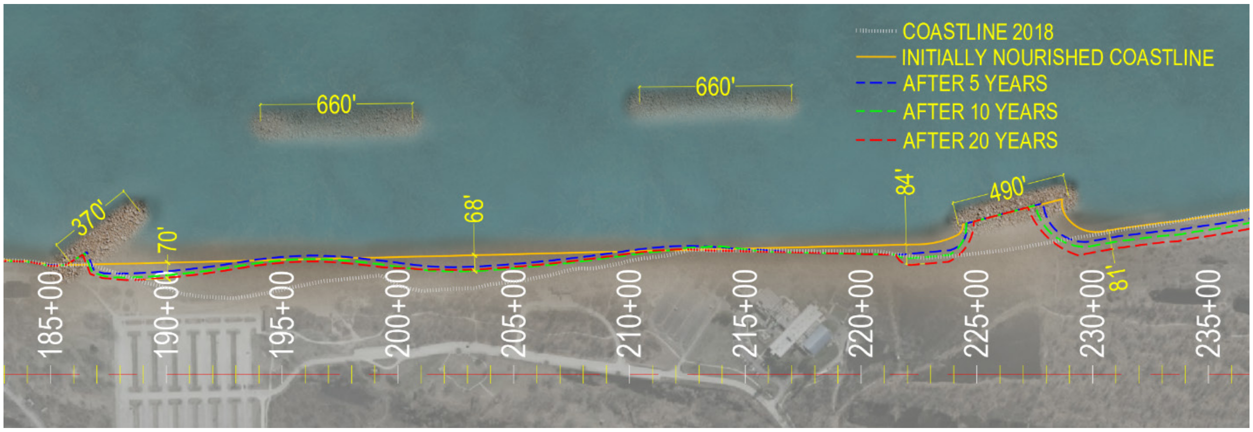


Figure C-22: Alternative 3.5 with shoreline recession after ten and twenty years

APPENDIX D

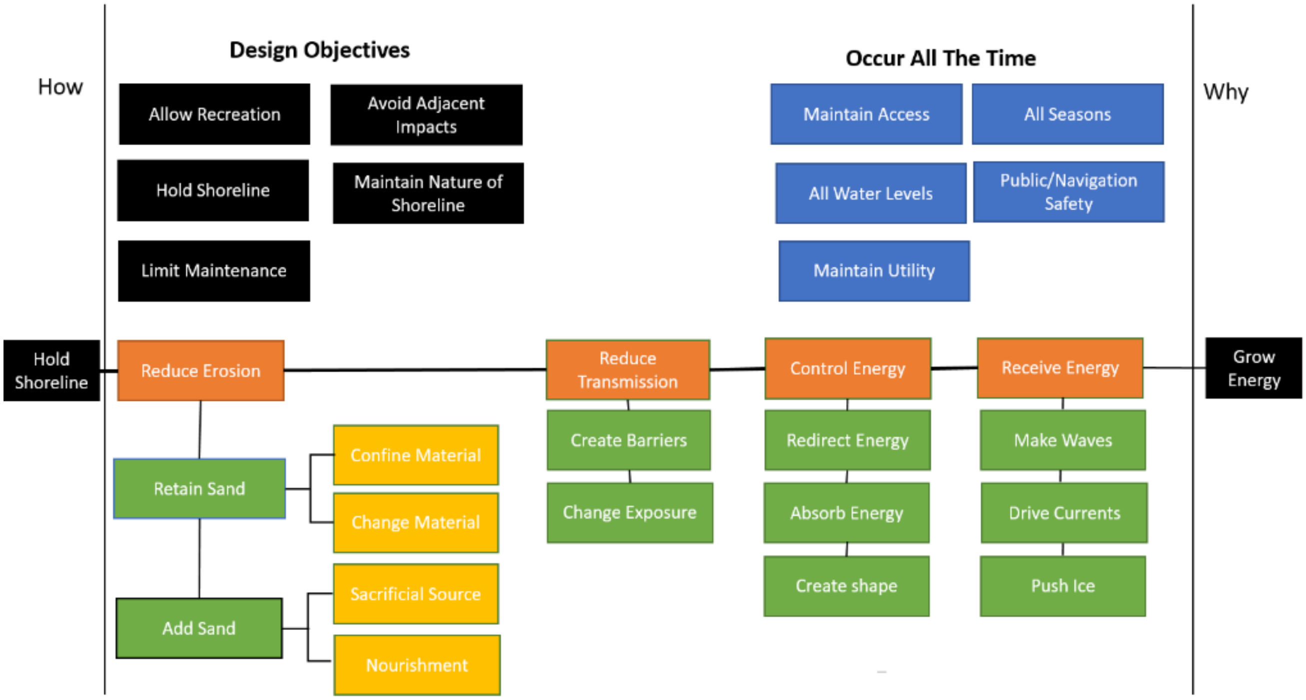
Value Engineering Analysis

VALUE ENGINEERING ANALYSIS OF SHORELINE STABILIZATION SOLUTIONS FOR ILLINOIS BEACH STATE PARK

To arrive at a rational list of shoreline stabilization solutions to consider for application at Illinois Beach State Park, a Value engineering approach was employed. Rather than simply picking typical design solutions, the approach was to develop a clear list of design objectives and attributes that the solution(s) must meet at all times. The analysis below breaks down the requirements into simple functions that must be achieved, and then modifying the designs to meet those functions with the objective of having the most efficient approach for least cost and greatest benefit. Following are the products of the analysis and resulting recommendations.

Functional Analysis

The following flow chart explains the design objectives, performance requirements and the functional relationships of how to address stabilizing the shoreline through various precedents. Based on these objectives, various alternatives for satisfying the functions where developed.



Alternative Development

Alternatives were generated that achieved the primary mission of holding the shoreline by either reducing erosion through either retaining sand or finding ways of adding sand. This was accomplished by reducing the transmission of wave energy reaching the shore, through redirecting the energy, absorbing the energy, or creating shapes that trap the energy.

IDEAS

Color Coding:

Qualitative selection of options best meeting Criteria

Could be merged/included with other ideas

ADD SAND

Feeder Beach
Eroding Artificial Dunes
Bypass Sand
Back pass sand
Offshore Mining of Sand (dredging pump to beach)
Offshore mining of Sand (barge nearshore placement)
Quarried Sand
Redirect Stream discharge
In stream sediment capture
Quarried crushed Rock
Degrading soft sandstone
TBM spoils
Allow sacrificial erosion zones

REDUCE EROSION

Offshore breakwater – detached continuous
Offshore Breakwater – detached segmental
Groin field – Straight or shaped groins, unfilled
Groin field – Straight or shaped groins, filled

Submerged breakwater (reef)

Rock Revetment

Terminal Jetties – Periodic long jetties at key beach areas

Bulkhead

Sandbags (geotubes)

Change beach grain size

Dunes

Beach vegetation

Pocket beach (headland features, w/wo groins or segmental detached breakwaters, and filled cell)

Perched beach

Seagrass

Beach grass or water tolerant deep rooting woody species

Floating attenuator

Changed wave refraction/diffraction pattern

Induce early wave breaking

Flattened beach slope – large area impact

Reduce reflections from shore

Remove seawall – reducing reflections slows down rate locally

Nourish beach (no structures)

Make shoreline more crescentic

RETAIN SAND

Groin field – pre-filled

Pocket beaches – shore attached breakwater arms

Segmental Submerged Breakwater – salient formation

Segmental Detached breakwater – salient formation

Segmental Detached breakwater – Tombolo

Submerged Breakwater – continuous

Nearshore Breakwater – continuous

Revetment – no beach access, narrow beach

Bulkhead – no beach access, narrow beach

Changed material – cobble beaches

Beach face dewatering - high power requirement

Freezing beach in winter – high power requirement, and equipment costs

Glue sand (polymer spray) – unnatural for adjacent uses

Crescentic shoreline

Sand traps – hazardous, inefficient but needed for bypassing or back passing

Flatten cross shore slope – large footprint

Add beach vegetation – conflict with beach use

Locally reduce transport rate

Paired Comparison Weighting Factors and Ideas Analysis

In order to assess how these different ideas would perform, relative to each other, and relative to the needs and objectives of the project, a list of criteria that needed to be met was developed. Though far from complete, the nine criteria selected reflected the major issues of concern as inferred by the VE team.

- Ability to obtain permits
- Project cost (or ability to be used broadly)
- Longevity
- Extent of Operational effort or magnitude of maintenance
- Material availability
- Ecological Impacts
- Changed Aesthetics
- Constructability
- Proven Technology

NOTE: for changed aesthetics, the consideration is NOT the actual appearance of the solution, but rather how important the appearance blends with its surrounding.

Through a paired inter-comparison, the importance of each of these factors was ranked to give a weighting in the final evaluation of ideas. The individual scores of the various reviewers illustrate the individual range of perceived importance resulting in a consensus weighting of results.

	Cost	Longevity	Ease of Maintenance	Material Availability	Ecological Impact	Aesthetics	Constructability	Proven Technology	Sum of Scores	Order of Importance of Criteria	Rob	Ale	Mauricio	Maggie	Jack	Bill
Permit Process									110	1	22	26	27	19	10	6
Cost									67	5	16	14	12	11	12	2
Longevity									71	3	13	4	5	15	22	12
Ease of Maintenance									27	8	6	4	3	1	5	8
Material Availability									36	7	9	13	2	6	0	6
Ecological Impact									52	6	16	13	8	2	7	6
Changed Aesthetics									26	9	0	0	15	6	2	3
Constructability									71	3	15	22	8	14	6	6
Proven Technology									72	2	22	7	9	8	15	11

The ability to obtain permits for the solutions was deemed as the most significant consideration in assessing ideas. This effectively eliminated some brainstorm ideas. Presuming any idea carried forward was then permittable, the most significant considerations would be solutions that have long life, rely on proven technology and are readily constructible. Cost and ecological impacts where secondary considerations.

Using the highlighted ideas above, filtered by the permit constraint, the following idea assessment was then developed. Each idea was ranked by weighting factor (10 – 1, highest to lowest, easiest versus hardest). The highest scoring of these would then be used singularly, or in combinations with other ideas to develop the most effective and appropriate shoreline solutions. The top 5 solutions are highlighted.

			Ability to Permit	Proven technology	constructability	Longevity	Capital Cost/Project extent	Ecological Impact	Operational/maintenance effort	Changed Aesthetics
		Importance Multiplier	4.23	2.77	2.73	2.73	2.58	2	1.04	1
Add Sand										
	122.32	Back Pass	8	9	7	4	6	5	1	7
			33.84	24.93	19.11	10.92	15.48	10	1.04	7

152.74	Feeder Beach	9	8	9	8	7	9	1	9
		38.07	22.16	24.57	21.84	18.06	18	1.04	9
147.67	Offshore Dredging	6	10	10	7	7	9	3	9
		25.38	27.7	27.3	19.11	18.06	18	3.12	9
152.74	Nearshore placement	9	8	10	7	7	9	1	9
		38.07	22.16	27.3	19.11	18.06	18	1.04	9
98.93	Instream capture	5	4	7	5	3	7	5	7
		21.15	11.08	19.11	13.65	7.74	14	5.2	7
148.78	Beach Rebuild	10	10	9	5	6	7	2	9
		42.3	27.7	24.57	13.65	15.48	14	2.08	9
Reduce Erosion									
144.87	Detached Brkwtr	8	10	7	8	7	5	8	6
		33.84	27.7	19.11	21.84	18.06	10	8.32	6
131.28	Terminal Jetties	8	10	8	8	3	4	8	2
		33.84	27.7	21.84	21.84	7.74	8	8.32	2
147.33	submerged brkwtr	8	8	6	9	7	7	8	10
		33.84	22.16	16.38	24.57	18.06	14	8.32	10
122.62	material size	10	6	10	4	4	4	4	3
		42.3	16.62	27.3	10.92	10.32	8	4.16	3
141.01	sacrificial dunes	10	6	10	5	7	7	2	7
		42.3	16.62	27.3	13.65	18.06	14	2.08	7
158.6	Pocket Beach	9	10	8	9	7	7	9	5
		38.07	27.7	21.84	24.57	18.06	14	9.36	5
Retain Sand (hold waterline)									
157.25	Pocket Beach	9	10	9	9	7	6	7	5
		38.07	27.7	24.57	24.57	18.06	12	7.28	5
152.44	submerged brkwtr	8	8	7	9	6	10	7	10
		33.84	22.16	19.11	24.57	15.48	20	7.28	10
156.56	emergent brkwtr	8	10	8	8	7	9	7	8
		33.84	27.7	21.84	21.84	18.06	18	7.28	8
127.33	Cobble beach	10	6	7	4	9	3	4	5
		42.3	16.62	19.11	10.92	23.22	6	4.16	5
151.18	Local transport control	8	7	8	7	10	8	5	10
		33.84	19.39	21.84	19.11	25.8	16	5.2	10
135.29	pre-filled Groin field	8	8	7	8	7	4	7	5
		33.84	22.16	19.11	21.84	18.06	8	7.28	5

Value Analysis Recommendations

The selection of the final approach was based on scores of essentially 20 or higher in a weighted criteria analysis and are carried forward to final considerations. Options must satisfy at least three categories besides being permittable with this scoring level. Based on this, the following coastal options will be used in the development of the alternatives.

ADD SAND

1) Create a feeder beach at the north end
    a. Feeder beach would be supplied by either deep water dredging with sand pumped to the feeder, or nearshore dump placement of sand (shallower than 10 ft)
2) Create multiple feeder beaches near high erosion areas

REDUCE EROSION

1) Employ detached breakwaters, with or without groin trunks, to form either salients along the beach or fixed pockets in equilibrium.
    a. Prefill pockets
2) Create one or more large terminal jetties to trap/create large beach areas
3) Submerged breakwater may be employed with proper consideration on construction method

RETAIN SAND

1) Create pocket beaches along the shoreline
2) Build submerged breakwaters in the nearshore
3) Build emergent detached breakwaters along shore
4) Configure offshore structures, or sculpt the bottom bathymetry to modify the wave pattern and transport rate/direction
5) Consider groin field as a less effective variant of pocket beach.

APPENDIX E

Select Site Photos

Figure E-1 Riprap Revetment, ST11+00 Looking North

Figure E-2 Riprap Revetment, ST11+50 Looking South

Figure E-3 Shoreline Erosion, ST12+00 Looking West

Figure E-4 Erosion at Revetment Tip, ST12+00 Looking South

Figure E-5 Southern End of Failing Block & Rubble Revetment, ST71+00 Looking Southwest

Figure E-6 Eroded Beach Behind Failed Revetment, ST71+50 Looking Southwest

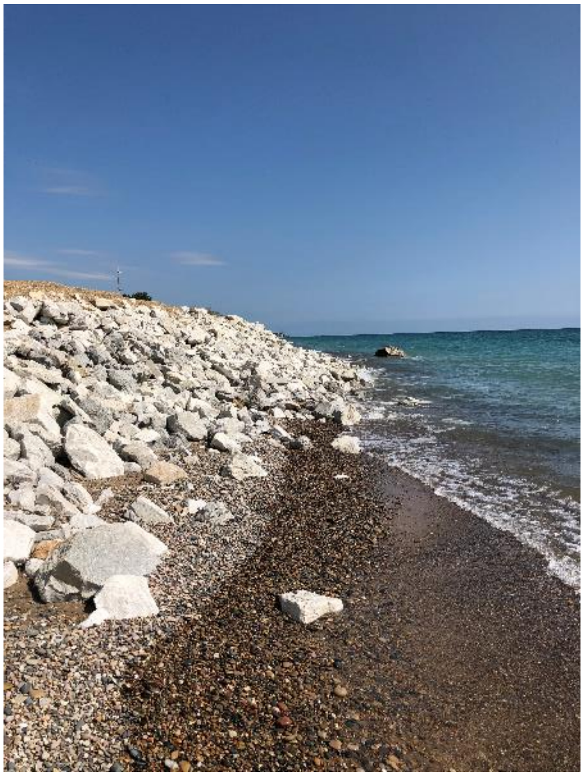


Figure E-7 Rubble Revetment, ST 185+50 Looking North

Figure E-8 Damage to Parking Lot, ST 190+50 Looking South

Figure E-9 Recreational Beach, ST 195+00 Looking North

Figure E-10 Bluff Slope Failure Behind Revetment, ST 198+50

Figure E-11 Concrete Block & Rubble, ST 207+00 Looking North

Figure E-12 Recreational Beach, ST210+00 Looking South

Figure E-13 Buried Sheetpile Wall in Rec Beach, ST 211+50

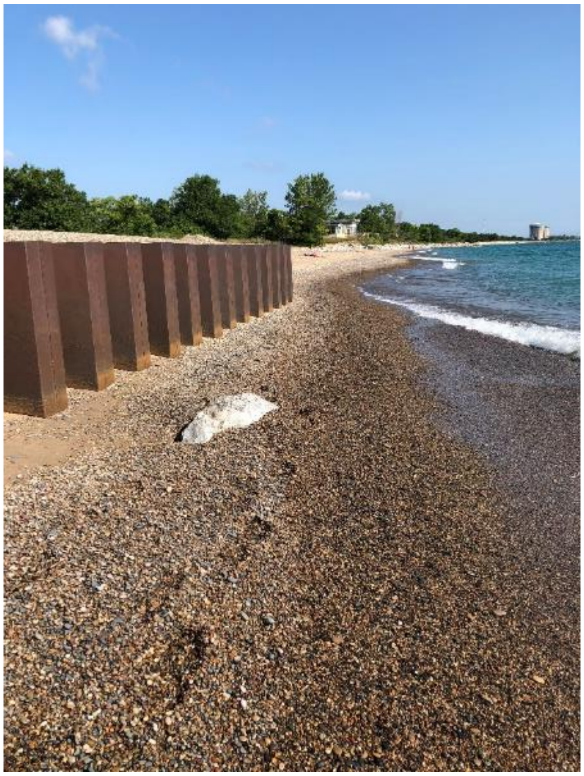


Figure E-14 Sheetpile Shoreline, ST 214+00 Looking North

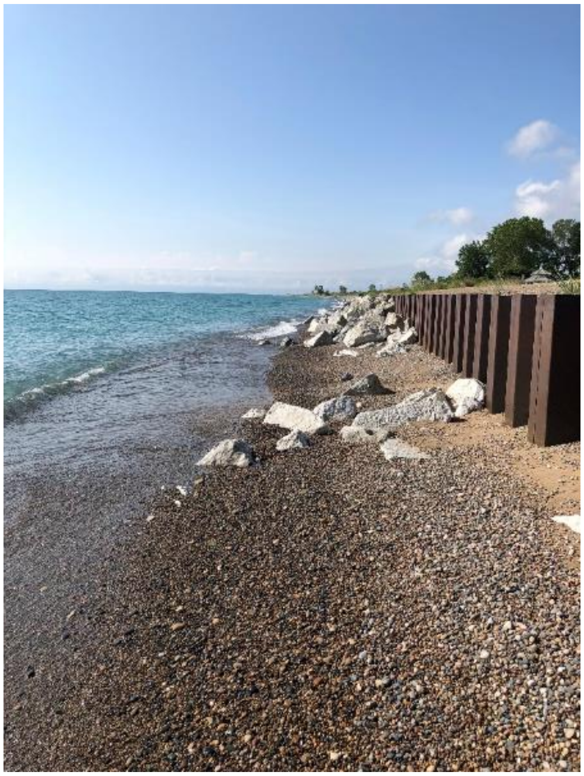


Figure E-15 Sheetpile Shoreline, ST 214+00 Looking South

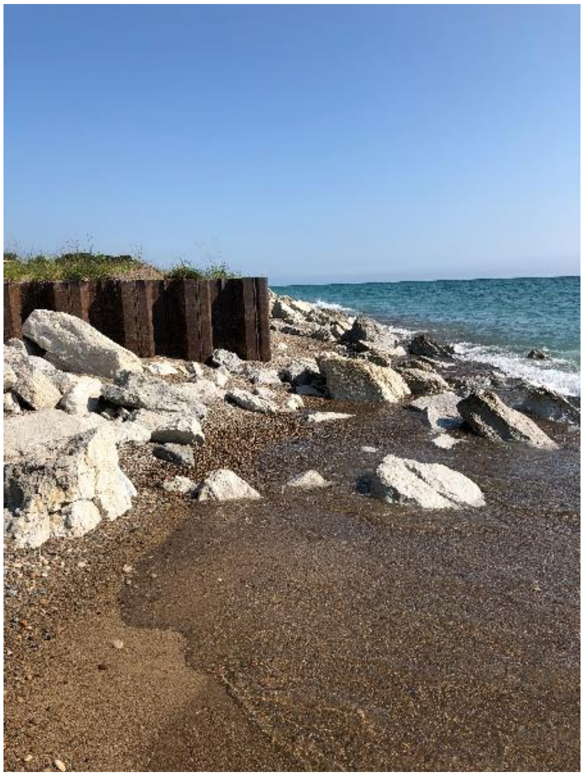


Figure E-16 End of Sheetpile Wall, ST 222+00 Looking North

APPENDIX F

Select Aerial / Drone Photos

Figure F-1: Area 1 Northpoint marina breakwater. ST12+50 Looking Southwest

Figure F-2: Area 1 Northpoint marina breakwater. ST 15+00 Looking Northeast

Figure F-3: Area 1 Northpoint marina breakwater. ST12+50 plan view.

Figure F-4: Erosion behind sheetpile ST42+50 Looking Southwest.

Figure F-5: Erosion behind sheetpile ST42+50 plan view

Figure F-6: Dilapidated eco-blocks ST55+00 Looking Southeast

Figure F-7: Dilapidated eco-blocks ST55+00 plan view.

Figure F-8: Erosion at Area 2 ST 71+00 Looking South

Figure F-9: Erosion at Area 2 ST 71+00 plan view

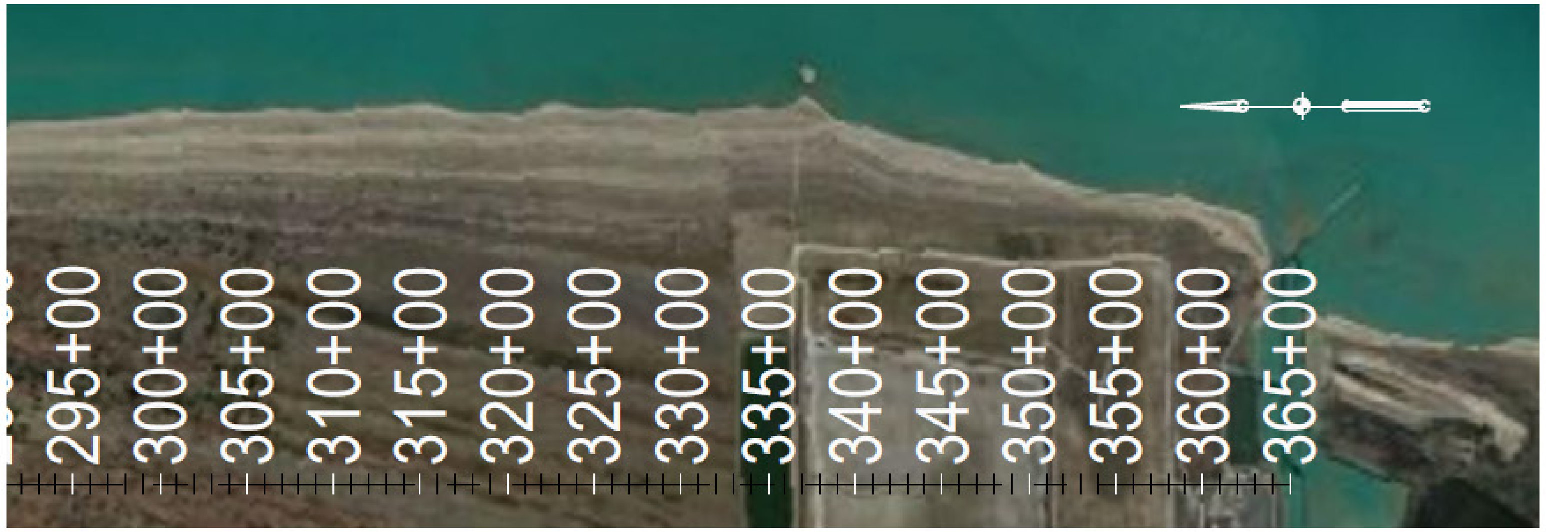
Figure F-10: Area 3 parking lot ST188+00 Looking Southwest

Figure F-11: Eco-block and riprap intersection at ST194+50 Looking South

APPENDIX G

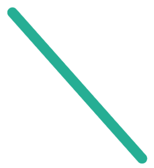
Illinois Beach State Park
Project Stationing

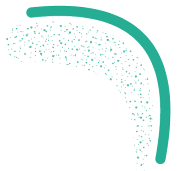
APPENDIX E: Illinois Beach Stationing

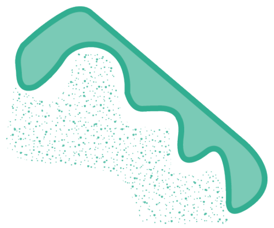


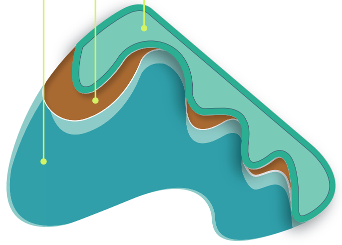
APPENDIX H

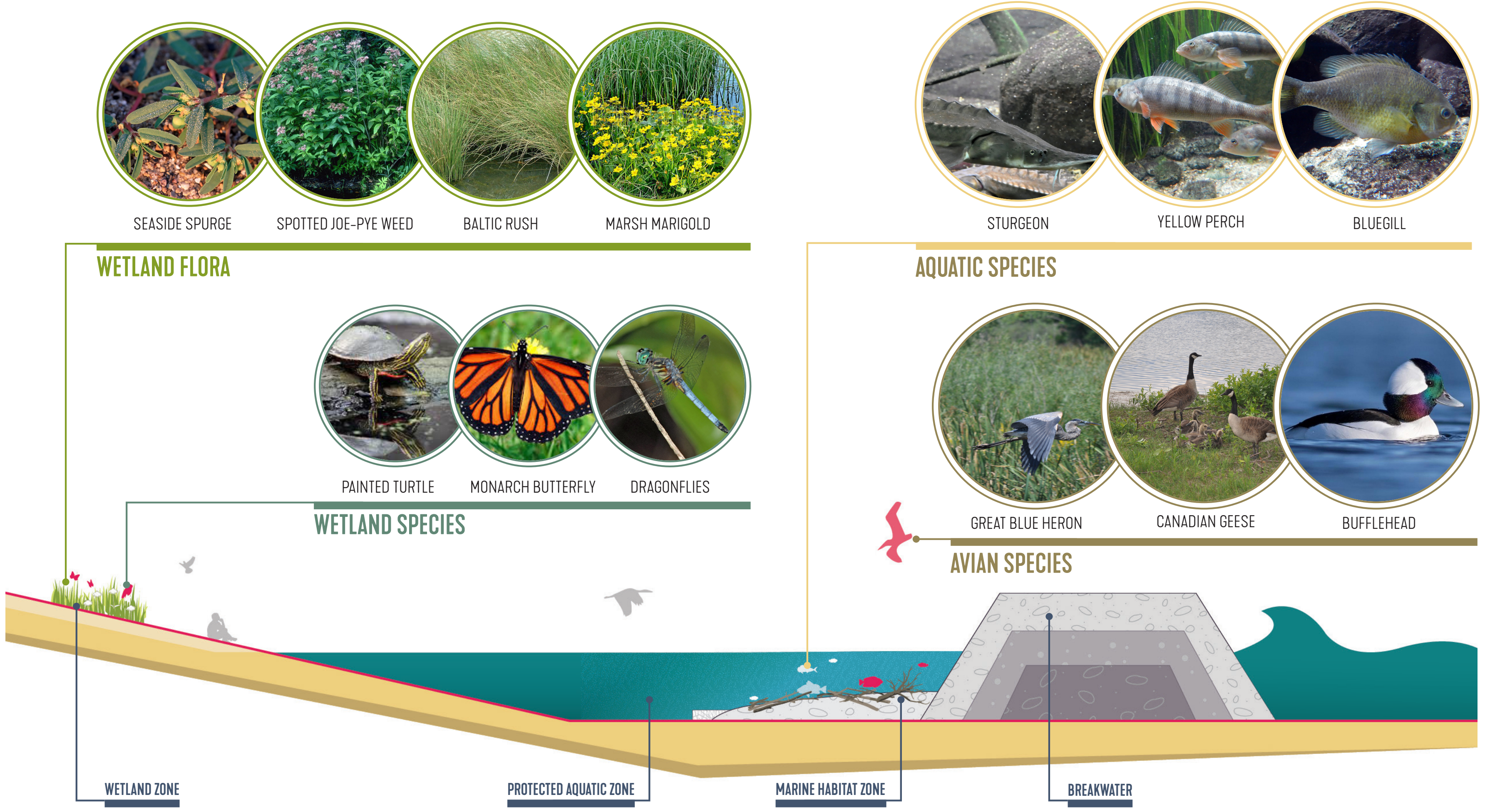
Structure Refinement & Bio Habitat Advancement Graphic


TYPICAL BREAKWATER


MODIFY FORM TO PROTECT CRITICAL HABITAT


MODIFY FORM FOR LOCALIZED, MICRO-SCALE COMPLEXITY


AQUATIC HABITAT PROTECTED BY BREAKWATER CONFIGURATION



APPENDIX I

Renderings


AREA 1


AREA 1


AREA 1


AREA 2


AREA 2

AREA 2


AREA 3


AREA 3

AREA 3

APPENDIX J

30% Design Drawings

Illinois Beach State Park

Erosion Control Structures

Zion, Illinois

September 06, 2019

30% Design Drawings

10793.000

IDNR # 2-17-008

Sheet List Table

G-001	COVER SHEET
G-003	OVERALL SITE KEY PLAN
V-100	OVERALL SURVEY
C-001	SITE ABBREVIATIONS, SYMBOLS, & NOTES
CV100	OVERALL EXISTING CONDITIONS - AREA 1
CV101	AREA 1 - OVERALL EXISTING CONDITIONS
CV102	AREA 1 - OVERALL EXISTING CONDITIONS
CV103	AREA 1 - OVERALL EXISTING CONDITIONS
CV104	OVERALL EXISTING CONDITIONS - AREA 2
CV105	AREA 2 - OVERALL EXISTING CONDITIONS
CV106	AREA 2 - OVERALL EXISTING CONDITIONS
CV107	AREA 2 - OVERALL EXISTING CONDITIONS
CV108	AREA 2 - OVERALL EXISTING CONDITIONS
CV109	OVERALL EXISTING CONDITIONS - AREA 3
CV110	AREA 3 - OVERALL EXISTING CONDITIONS
CV111	AREA 3 - OVERALL EXISTING CONDITIONS
CV112	AREA 3 - OVERALL EXISTING CONDITIONS
CV113	AREA 3 - OVERALL EXISTING CONDITIONS
CS100	OVERALL LAYOUT AND MATERIALS SHEET - AREA 1
CS101	AREA 1 - LAYOUT AND MATERIALS PLAN
CS102	AREA 1 - LAYOUT AND MATERIALS PLAN
CS103	AREA 1 - LAYOUT AND MATERIALS PLAN
CS104	OVERALL LAYOUT AND MATERIALS SHEET - AREA 2
CS105	AREA 2 - LAYOUT AND MATERIALS PLAN
CS106	AREA 2 - LAYOUT AND MATERIALS PLAN
CS107	AREA 2 - LAYOUT AND MATERIALS PLAN
CS108	AREA 2 - LAYOUT AND MATERIALS PLAN
CS109	OVERALL LAYOUT AND MATERIALS - AREA 3
CS110	AREA 3 - LAYOUT AND MATERIALS
CS111	AREA 3 - LAYOUT AND MATERIALS
CS112	AREA 3 - LAYOUT AND MATERIALS
CS113	AREA 3 - LAYOUT AND MATERIALS
CS500	BREAKWATER SECTIONS
CS501	BREAKWATER SECTIONS
CS502	BREAKWATER SECTIONS
CS503	BREAKWATER SECTIONS

Prepared for:

ILLINOIS DEPARTMENT
OF NATURAL
RESOURCES
One Natural Resources Way
Springfield, IL 62702

Prepared by:

SMITHGROUP

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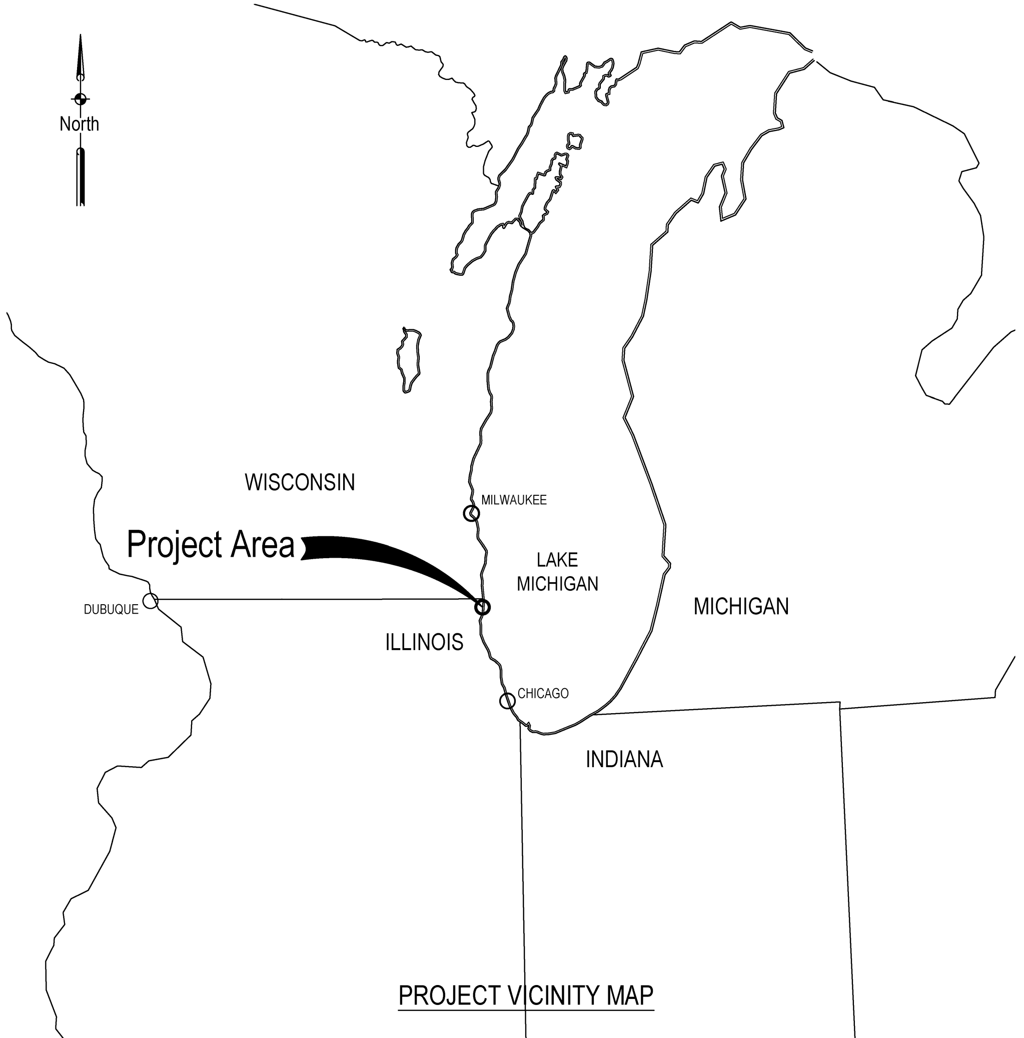
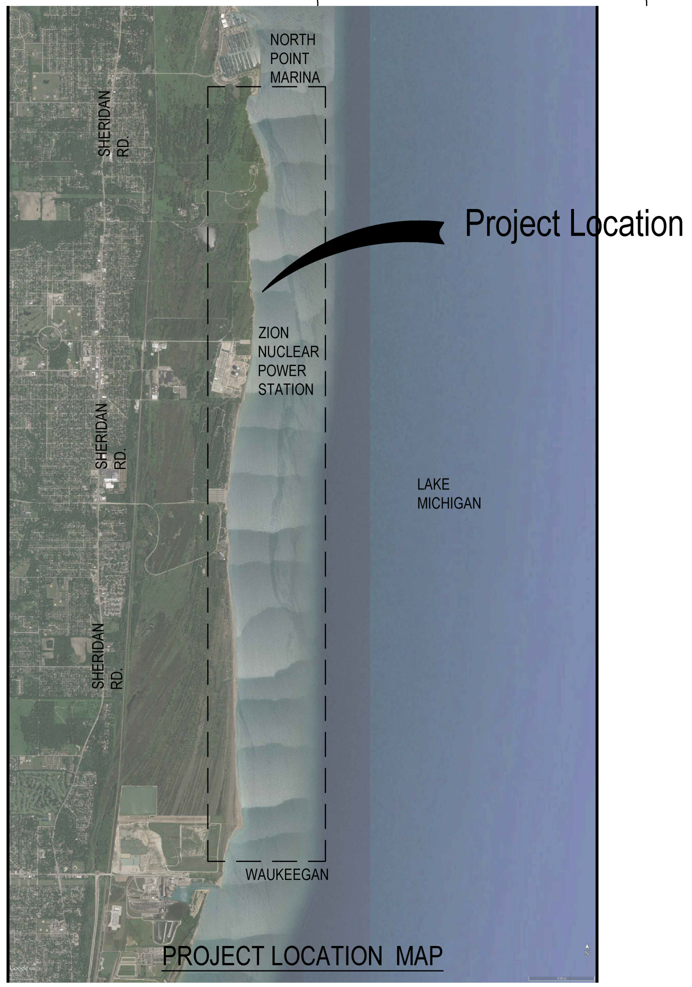


PROJECT VICINITY MAP



PROJECT LOCATION MAP

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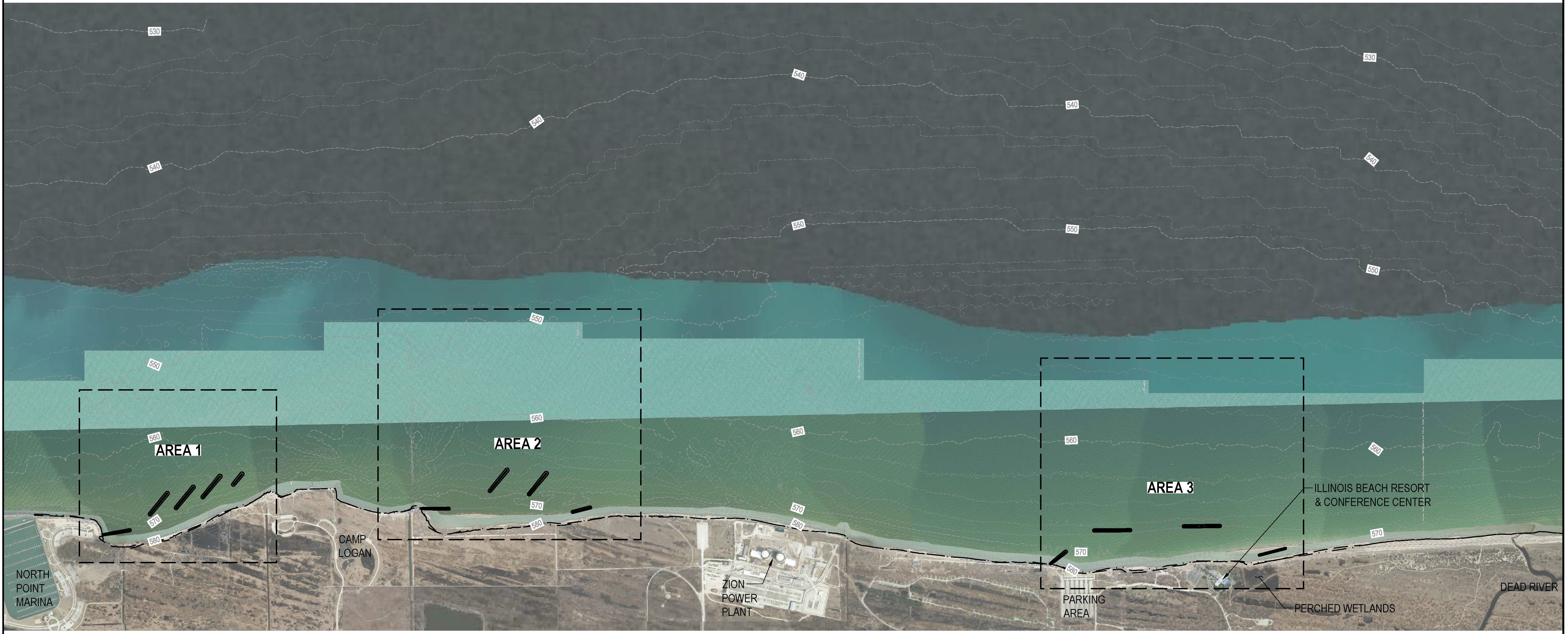

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IBSP RESTORATION

1 LAKE FRONT DRIVE, ZION, IL
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SEALS AND SIGNATURES

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KEY PLAN

PROJECT NORTH

DRAWING TITLE
OVERALL SITE KEY PLAN

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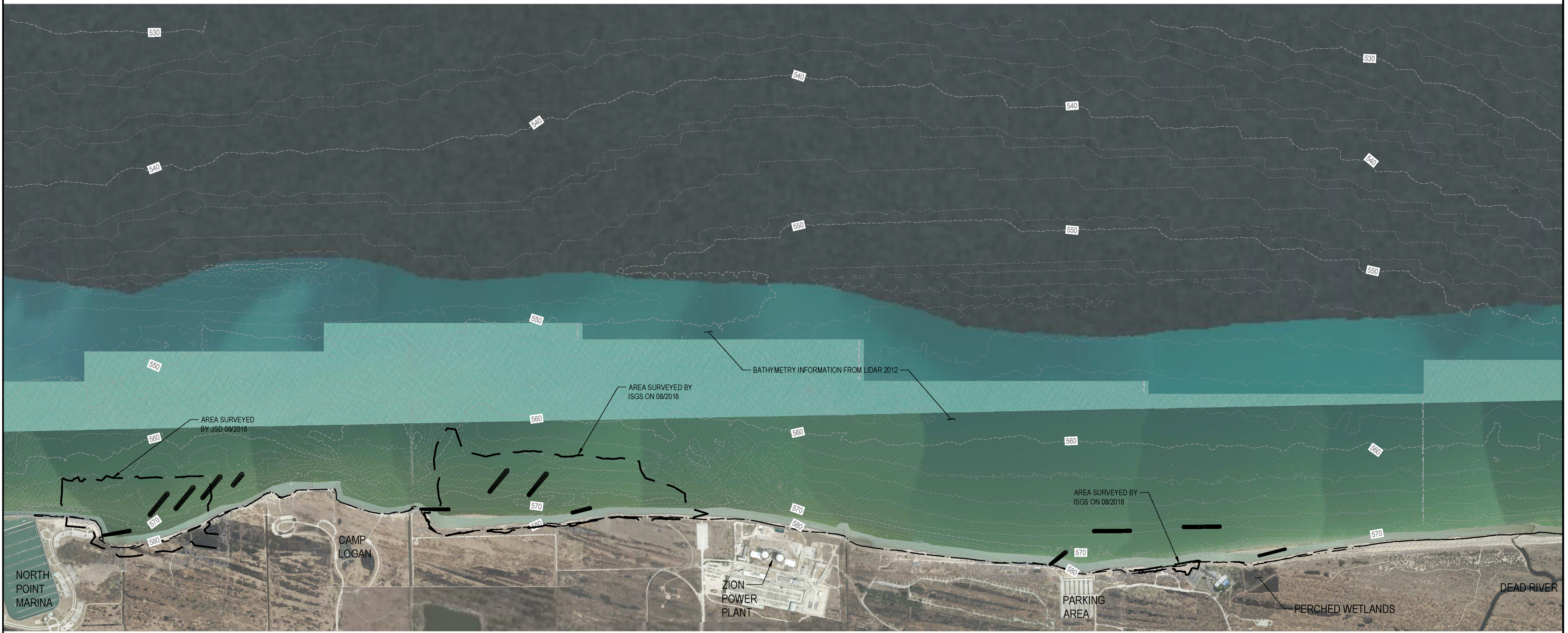
SCALE:

SCALE

10793.000
PROJECT NUMBER

G-003
DRAWING NUMBER



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KEY PLAN

PROJECT NORTH

DRAWING TITLE

OVERALL SURVEY

0' 500' 1000' 2000'

SCALE:

SCALE

10793.000

PROJECT NUMBER

V-100

DRAWING NUMBER

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SURVEY LEGEND

— — — — — —	WATER INTAKE PIPE
- - - -865- - - - -	EXISTING MAJOR CONTOUR
- - - -864- - - - -	EXISTING MINOR CONTOUR
—— · · ——	APPROXIMATE EDGE OF WATER DURING 08/2018 SURVEY
—— ——	SURVEY BOUNDARY

LAYOUT AND MATERIAL SYMBOLS

⁘	SAND MATERIAL
▨	PRIMARY BEACH
■	ARMOR STONE
—— - —— - ——	STRUCTURE CENTERLINE
⬡	RIPRAP
—— · · ——	APPROXIMATE EDGE OF WATER DURING 08/2018 SURVEY AND APPROXIMATED PER 07/2018 GOOGLE EARTH IMAGERY.

GRADING SYMBOLS

- - - -865- - - - -	EXISTING MAJOR CONTOUR
- - - -864- - - - -	EXISTING MINOR CONTOUR
——865——	PROPOSED MAJOR CONTOUR
——864——	PROPOSED MINOR CONTOUR
2.5% →	DRAINAGE SLOPE
▷ 1.5H:1V	COASTAL STRUCTURE SIDE SLOPE

SURVEY NOTES

1. THE HORIZONTAL DATUM IS ACCORDING TO THE NORTH AMERICAN DATUM OF 1983 (NAD83), STATE PLANE COORDINATES, OHIO NORTH ZONE (3401).
2. THE VERTICAL DATUM IS ACCORDING TO THE INTERNATIONAL GREAT LAKES DATUM OF 1985 (IGLD85).
3. AERIAL PHOTOGRAPHY OF THE NEARSHORE TAKEN ON 07/17/2019. DRONE PHOTOGRAPHY ALIGNED WITH GOOGLE EARTH IMAGERY. IMAGE NOT GEOREFERENCED AND FOR INFORMATIONAL PURPOSES ONLY.
4. SURVEY DATA WITHIN DEFINED BOUNDARIES IS FROM SURVEYS PERFORMED BY ISGS IN 08/2018 AND JSD IN 08/2018
5. DATA OUTSIDE THE DEFINED BOUNDARIES IS LIDAR INFORMATION FROM THE 2012 U.S. ARMY CORPS OF ENGINEERS NATIONAL COASTAL MAPPING PROGRAM.

LAYOUT AND MATERIALS NOTES

BEACH SAND:

A. SAND MATERIALS SHALL BE CLEAN, YELLOW/GOLDEN COLOR, NATURAL SAND FROM OFF-SITE SOURCES OR MINED FROM OFFSHORE SOURCES IF ALLOWABLE.
B. SAND MATERIAL SHALL BE FREE OF DEBRIS, ROOTS, WOOD, SCRAP MATERIAL, ORGANIC MATTER, REFUSE, SOFT UNSOUND PARTICLES, AND FROZEN, DELETERIOUS OR OBJECTIONABLE MATERIALS.
C. SAND MATERIAL FOR BEACHES SHALL BE ROUNDED, CLEAN, FREE DRAINING SAND OBTAINED FROM NATURAL DEPOSITS, PREFERABLY OF MARINE ORIGIN, MEETING THE GENERAL REQUIREMENTS OF SOIL MATERIALS ABOVE.

ABBREVIATIONS

NS BW	NEARSHORE BREAKWATER
O BW	OFFSHORE BREAKWATER
S BW	SUBMERGED BREAKWATER

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KEY PLAN

PROJECT NORTH

DRAWING TITLE
SITE ABBREVIATIONS, SYMBOLS, & NOTES

SCALE

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PROJECT NUMBER

C-001
DRAWING NUMBER

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KEY PLAN

DRAWING TITLE

OVERALL EXISTING
CONDITIONS - AREA 1

0' 75' 150' 300'

SCALE:

SCALE

10793.000

PROJECT NUMBER

CV100

DRAWING NUMBER

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SEALS AND SIGNATURES

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KEY PLAN

DRAWING TITLE

AREA 1 - OVERALL EXISTING CONDITIONS

0' 25' 50' 100'

SCALE:

SCALE

10793.000

PROJECT NUMBER

CV101

DRAWING NUMBER

APPROXIMATE WATER LINE ON 07/2019

APPROXIMATE WATER LINE ON 08/2018

IBSP RESTORATION

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60099

IDNR # 2-17-008

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30% DESIGN DRAWINGS		08/08/2019

SEALS AND SIGNATURES

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KEY PLAN

DRAWING TITLE

AREA 1 - OVERALL EXISTING
CONDITIONS

0' 25' 50' 100'

SCALE:

SCALE

10793.000

PROJECT NUMBER

CV102

DRAWING NUMBER

FILE: P:\10793.000\CAD\C\Sheet\10793-CV100.dwg USER: alira DATE: Sep. 06 2019 TIME: 10:50 am

IBSP RESTORATION

1 LAKE FRONT DRIVE, ZION, IL
60099

IDNR #2-17-008

Owner:

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SEALS AND SIGNATURES

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KEY PLAN

DRAWING TITLE

AREA 1 - OVERALL EXISTING CONDITIONS

0' 25' 50' 100'

SCALE:

SCALE

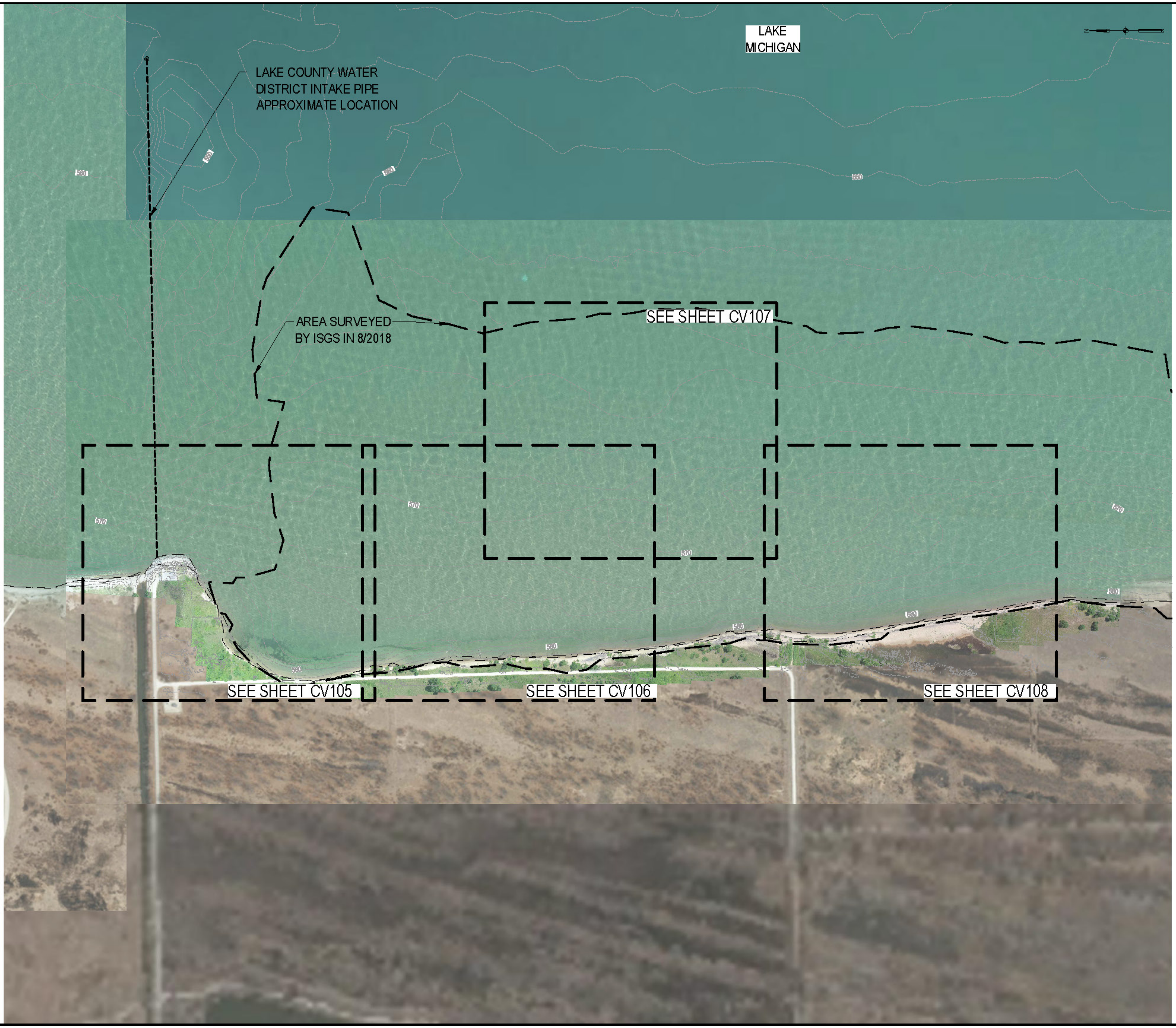
10793.000

PROJECT NUMBER

CV103

DRAWING NUMBER

FILE: P:\10793.000\CAD\C\Sheet\10793-CV100.dwg USER: alira DATE: Sep. 06 2019 TIME: 10:51 am



FILE:P:\10793.000\CAD\C\Sheet\10793-CV100.dwg USER:alira DATE: Sep, 06 2019 TIME: 10:52 am

IBSP RESTORATION

1 LAKE FRONT DRIVE, ZION, IL 60099

IDNR # 2-17-008

Owner:

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SEALS AND SIGNATURES

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KEY PLAN

DRAWING TITLE

OVERALL EXISTING CONDITIONS - AREA 2

0' 100' 200' 400'

SCALE:

SCALE

10793.000

PROJECT NUMBER

CV104

DRAWING NUMBER

FILE: P:\10793.000\CAD\C\Sheet\10793–CV100.dwg USER: alira DATE: Sep, 06 2019 TIME: 10:53 am

IBSP RESTORATION

1 LAKE FRONT DRIVE, ZION, IL
60099

IDNR # 2-17-008

Owner:

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SEALS AND SIGNATURES

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KEY PLAN

PROJECT NORTH

DRAWING TITLE

AREA 2 - OVERALL EXISTING CONDITIONS

0' 25' 50' 100'

SCALE:

SCALE

10793.000

PROJECT NUMBER

CV105

DRAWING NUMBER

FILE: P:\10793.000\CAD\C\Sheet\10793-CV100.dwg USER: alira DATE: Sep, 06 2019 TIME: 10:54 am

IBSP RESTORATION

1 LAKE FRONT DRIVE, ZION, IL
60099

IDNR # 2-17-008

Owner:

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SEALS AND SIGNATURES

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KEY PLAN

PROJECT NORTH

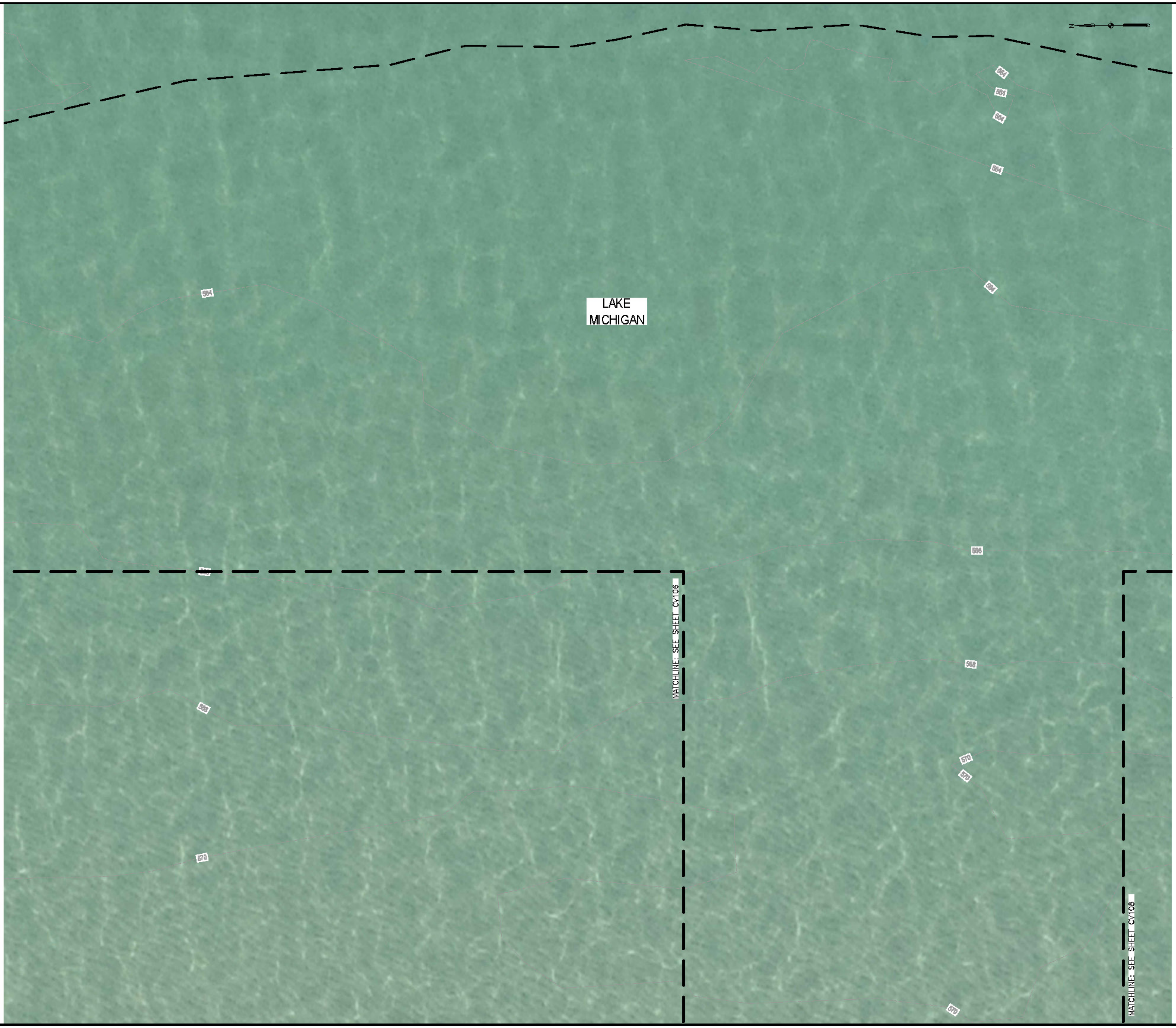
DRAWING TITLE
AREA 2 - OVERALL EXISTING CONDITIONS

0' 25' 50' 100'
SCALE:

SCALE

10793.000
PROJECT NUMBER

CV106
DRAWING NUMBER



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IBSP RESTORATION

1 LAKE FRONT DRIVE, ZION, IL
60099

IDNR # 2-17-008

Owner:

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SEALS AND SIGNATURES

NOT FOR CONSTRUCTION

KEY PLAN

PROJECT NORTH

DRAWING TITLE

AREA 2 - OVERALL EXISTING CONDITIONS

0' 25' 50' 100'

SCALE:

SCALE

10793.000

PROJECT NUMBER

CV107

DRAWING NUMBER

IBSP RESTORATION

1 LAKE FRONT DRIVE, ZION, IL
60099

IDNR # 2-17-008

Owner:

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SEALS AND SIGNATURES

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KEY PLAN

PROJECT NORTH

DRAWING TITLE

AREA 2 - OVERALL EXISTING
CONDITIONS

0' 25' 50' 100'

SCALE:

SCALE

10793.000

PROJECT NUMBER

CV108

DRAWING NUMBER

FILE: P:\10793.000\CAD\C\Sheet\10793-CV100.dwg USER: alira DATE: Sep, 06 2019 TIME: 10:55 am

IBSP RESTORATION

1 LAKE FRONT DRIVE, ZION, IL
60099

IDNR # 2-17-008

Owner:

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SEALS AND SIGNATURES

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KEY PLAN

PROJECT NORTH

DRAWING TITLE

OVERALL EXISTING CONDITIONS - AREA 3

0' 100' 200' 400'

SCALE:

SCALE

10793.000

PROJECT NUMBER

CV109

DRAWING NUMBER

FILE: P:\10793.000\CAD\C\Sheet\10793-CV100.dwg USER: alira DATE: Sep, 06 2019 TIME: 10:56 am

IBSP RESTORATION

1 LAKE FRONT DRIVE, ZION, IL
60099

IDNR # 2-17-008

Owner:

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30% DESIGN DRAWINGS		08/08/2019

SEALS AND SIGNATURES

NOT FOR CONSTRUCTION

KEY PLAN

PROJECT NORTH

DRAWING TITLE

AREA 3 - OVERALL EXISTING CONDITIONS

0' 25' 50' 100'

SCALE:

SCALE

10793.000

PROJECT NUMBER

CV110

DRAWING NUMBER

FILE: P:\10793.000\CAD\C\Sheet\10793-CV100.dwg USER: alira DATE: Sep, 06 2019 TIME: 10:57 am

IBSP RESTORATION

1 LAKE FRONT DRIVE, ZION, IL
60099

IDNR # 2-17-008

Owner:

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SEALS AND SIGNATURES

NOT FOR CONSTRUCTION

KEY PLAN

PROJECT NORTH

DRAWING TITLE

AREA 3 - OVERALL EXISTING CONDITIONS

0' 25' 50' 100'

SCALE:

SCALE

10793.000

PROJECT NUMBER

CV111

DRAWING NUMBER

FILE: P:\10793.000\CAD\C\Sheet\10793-CV100.dwg USER: alira DATE: Sep, 06 2019 TIME: 11:09 am

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IBSP RESTORATION

1 LAKE FRONT DRIVE, ZION, IL
60099

IDNR # 2-17-008

Owner:

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SEALS AND SIGNATURES

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KEY PLAN

PROJECT NORTH

DRAWING TITLE

AREA 3 - OVERALL EXISTING CONDITIONS

0' 25' 50' 100'

SCALE:

SCALE

10793.000

PROJECT NUMBER

CV112

DRAWING NUMBER

IBSP RESTORATION

1 LAKE FRONT DRIVE, ZION, IL
60099

IDNR # 2-17-008

Owner:

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SEALS AND SIGNATURES

NOT FOR CONSTRUCTION

KEY PLAN

PROJECT NORTH

DRAWING TITLE

AREA 3 - OVERALL EXISTING
CONDITIONS

0' 25' 50' 100'

SCALE:

SCALE

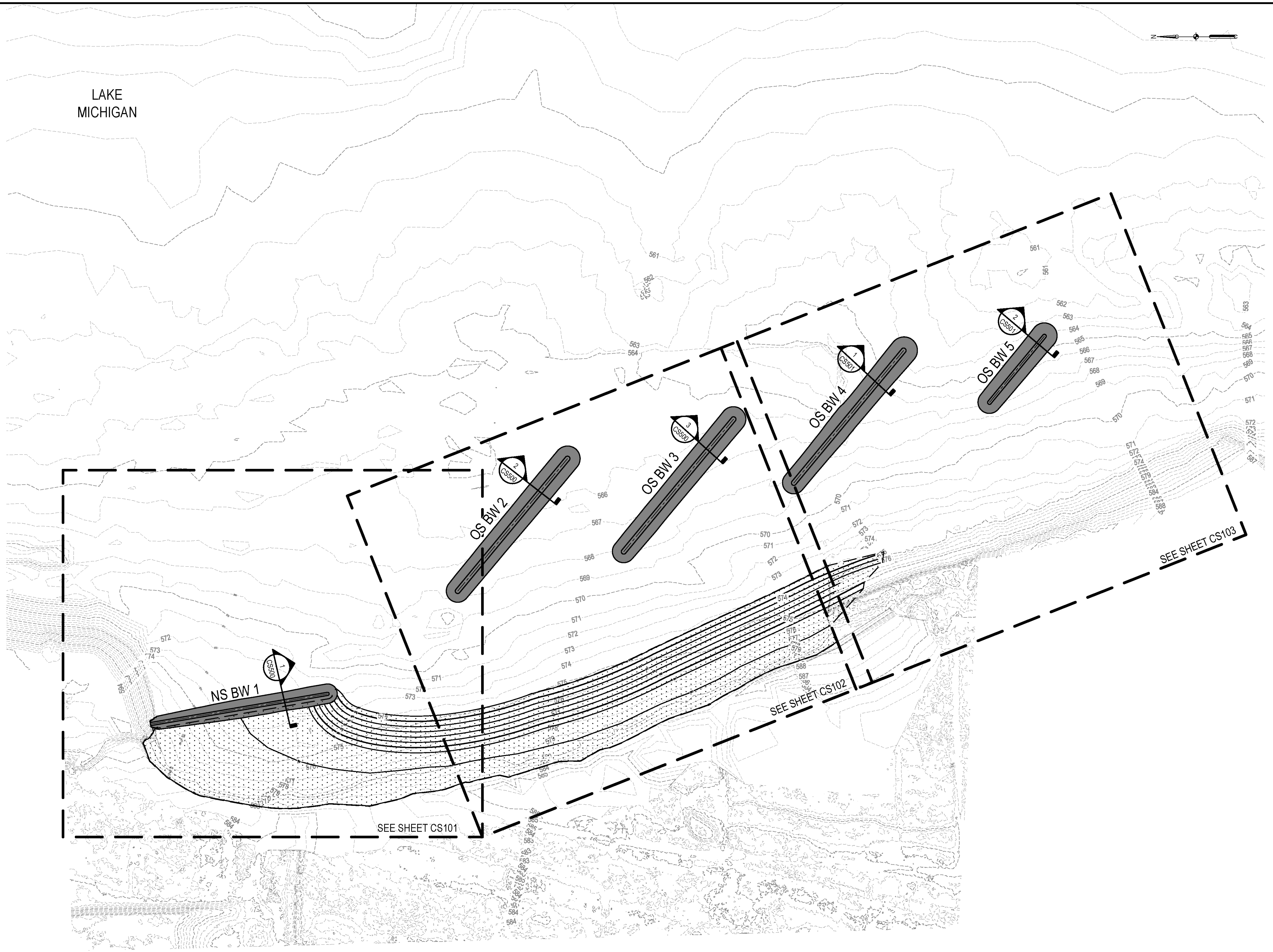
10793.000

PROJECT NUMBER

CV113

DRAWING NUMBER

FILE: P:\10793.000\CAD\C\Sheet\10793-CV100.dwg USER: alira DATE: Sep, 06 2019 TIME: 11:00 am



IBSP RESTORATION

1 LAKE FRONT DRIVE, ZION, IL
60099

IDNR # 2-17-008

Owner:

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SEALS AND SIGNATURES

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KEY PLAN

PROJECT NORTH

DRAWING TITLE

OVERALL LAYOUT AND
MATERIALS SHEET - AREA 1

0' 10' 20' 40'

SCALE:

SCALE

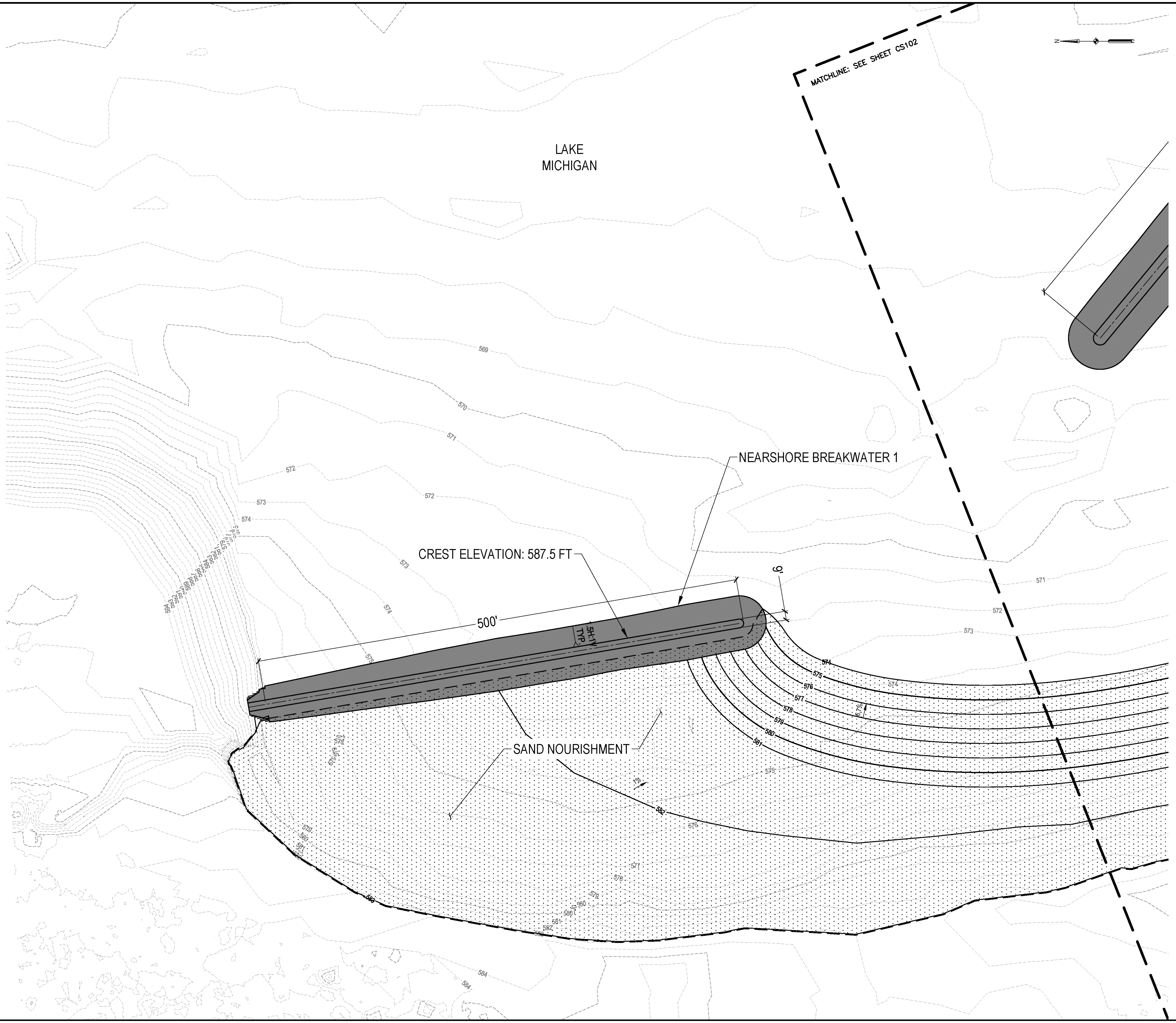
10793.000

PROJECT NUMBER

CS100

DRAWING NUMBER

FILE: P: \10793.000\CAD\C\Sheet\10793-CS100.dwg USER:alira DATE:Sep, 06 2019 TIME: 11:01 am



IBSP RESTORATION

1 LAKE FRONT DRIVE, ZION, IL
60099

IDNR # 2-17-008

Owner:

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SEALS AND SIGNATURES

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KEY PLAN

PROJECT NORTH

DRAWING TITLE

AREA 1 - LAYOUT AND MATERIALS PLAN

0' 10' 20' 40'

SCALE:

SCALE

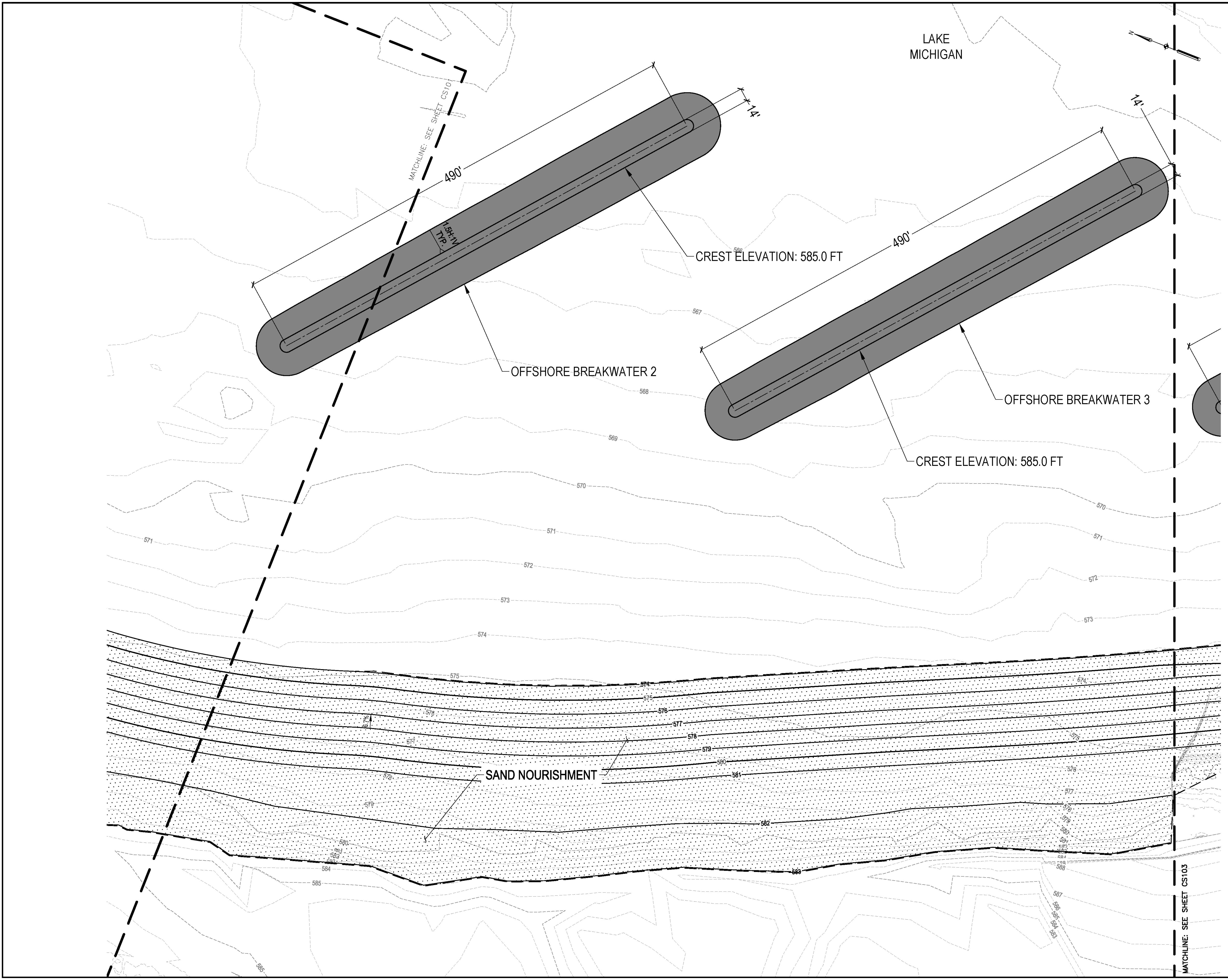
10793.000

PROJECT NUMBER

CS101

DRAWING NUMBER

FILE: P:\10793.000\CAD\C\Sheet\10793-CS100.dwg USER:alira DATE:Sep, 06 2019 TIME: 11:01 am



FILE: P:\10793.000\CAD\C\Sheet\10793-CS100.dwg USER:alira DATE: Sep, 06 2019 TIME: 11:01 am

IBSP RESTORATION

1 LAKE FRONT DRIVE, ZION, IL
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Owner:

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SEALS AND SIGNATURES

NOT FOR CONSTRUCTION

KEY PLAN

PROJECT NORTH

DRAWING TITLE

AREA 1 - LAYOUT AND
MATERIALS PLAN

0' 10' 20' 40'

SCALE:

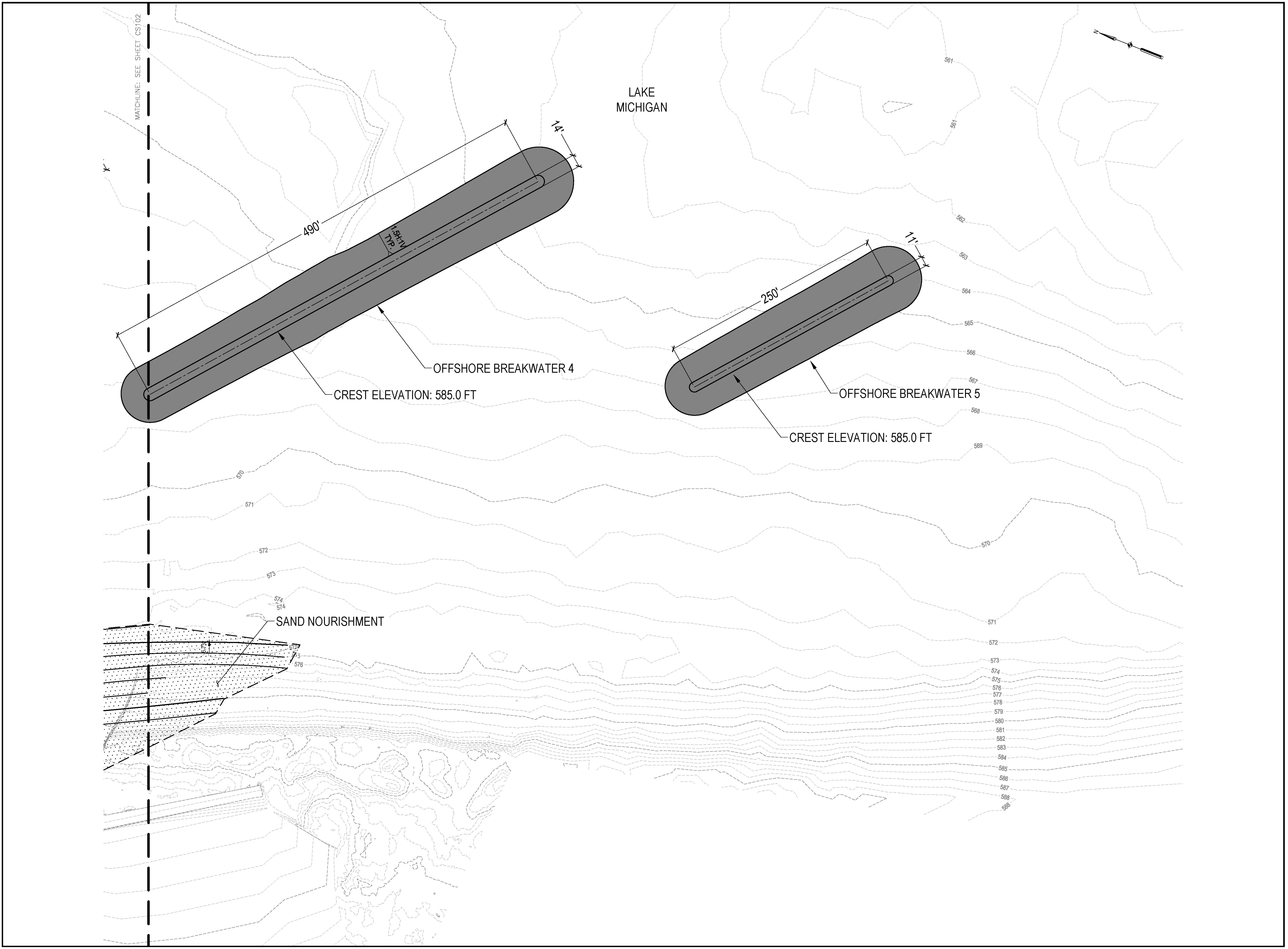
SCALE

10793.000

PROJECT NUMBER

CS102

DRAWING NUMBER



FILE: P:\10793.000\CAD\C\Sheet\10793-CS100.dwg USER: alira DATE: Sep, 06 2019 TIME: 11:01 am

IBSP RESTORATION

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IDNR # 2-17-008

Owner:

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30% DESIGN DRAWINGS		08/08/2019

SEALS AND SIGNATURES

NOT FOR CONSTRUCTION

KEY PLAN

PROJECT NORTH

DRAWING TITLE

AREA 1 - LAYOUT AND
MATERIALS PLAN

0' 10' 20' 40'

SCALE:

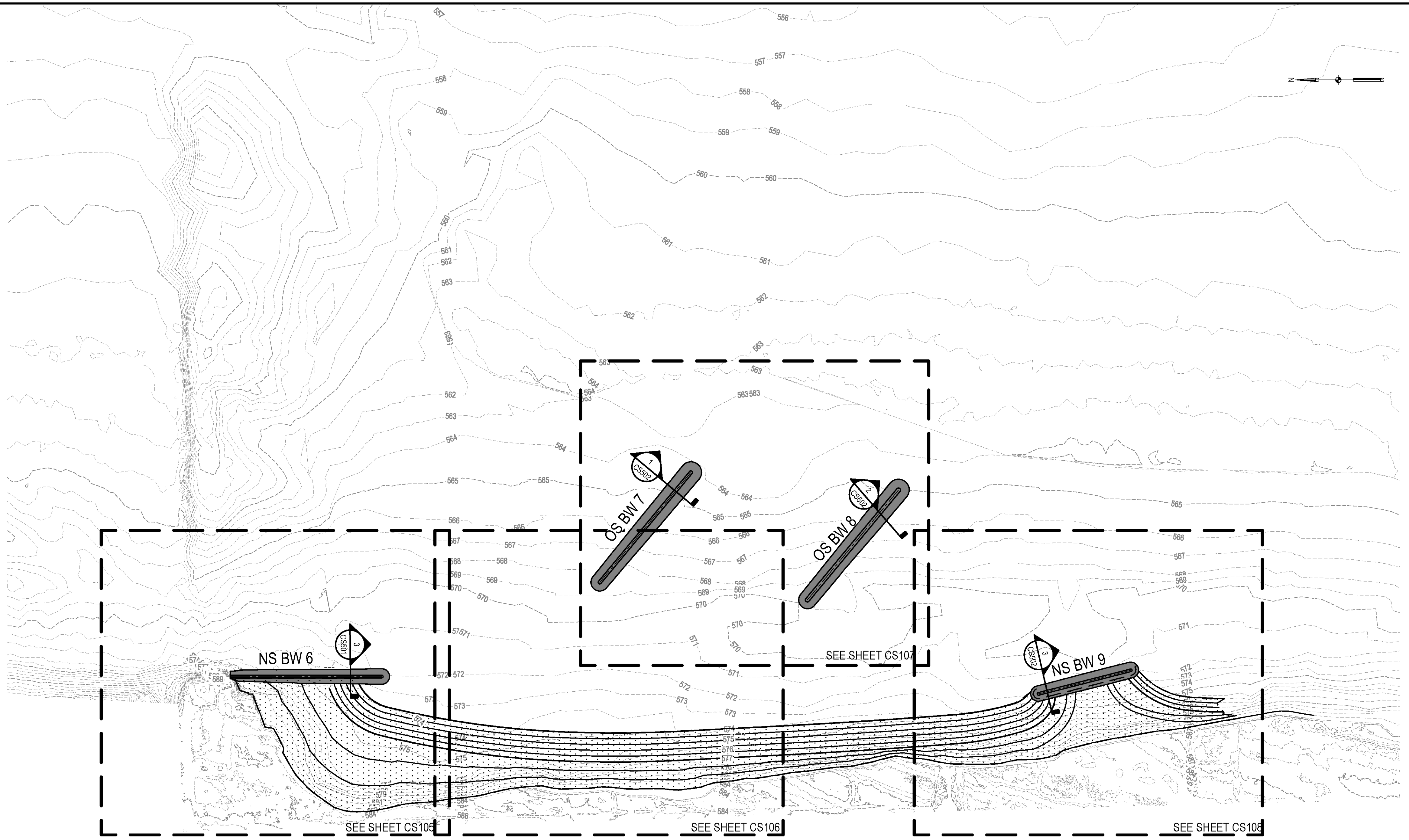
SCALE

10793.000

PROJECT NUMBER

CS103

DRAWING NUMBER



IBSP RESTORATION

1 LAKE FRONT DRIVE, ZION, IL
60099

IDNR # 2-17-008

Owner:

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SEALS AND SIGNATURES

NOT FOR CONSTRUCTION

KEY PLAN

PROJECT NORTH

DRAWING TITLE
OVERALL LAYOUT AND
MATERIALS SHEET - AREA 2

0' 10' 20' 40'

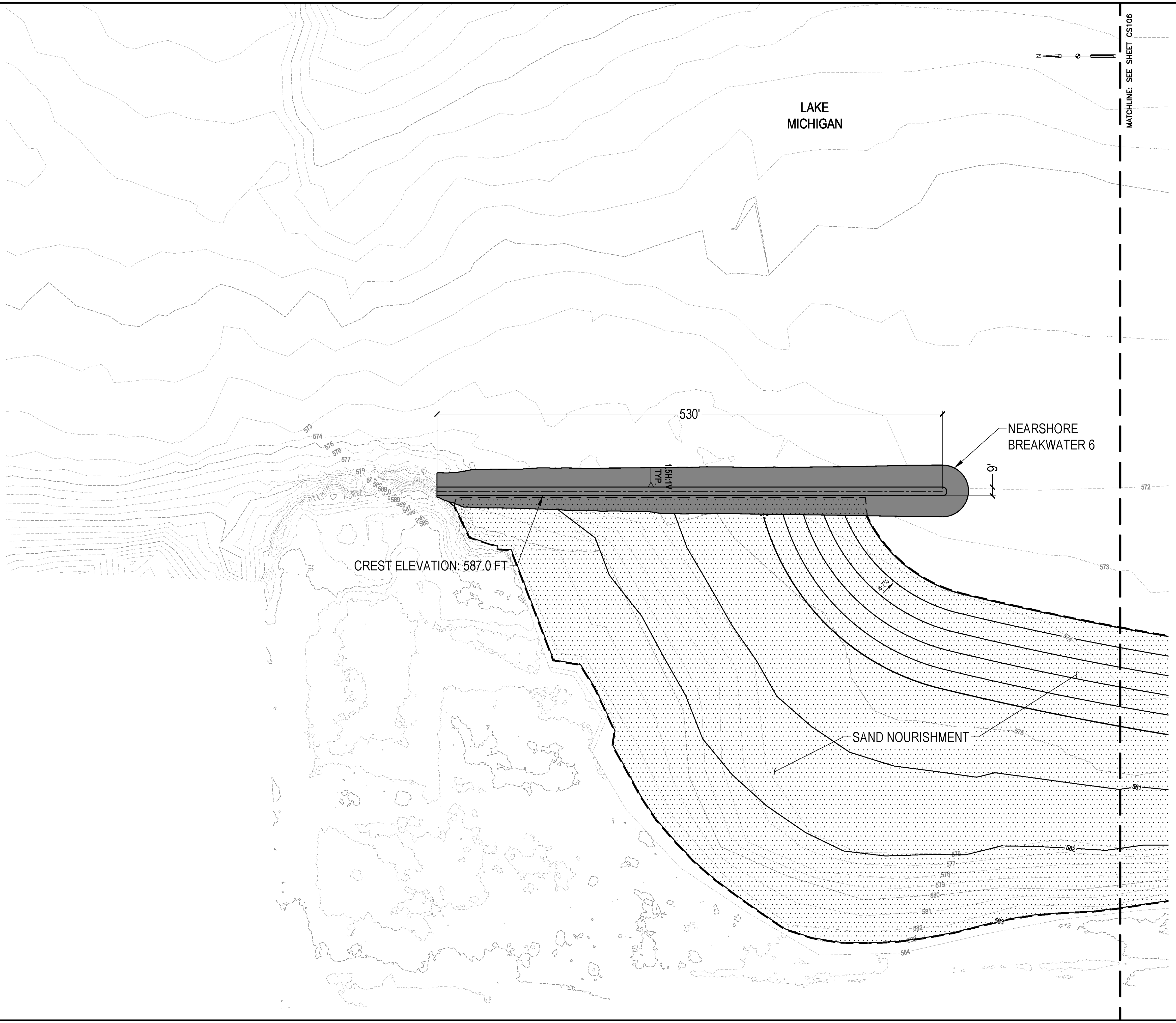
SCALE:

SCALE

10793.000
PROJECT NUMBER

CS104
DRAWING NUMBER

FILE: P:\10793.000\CAD\C\Sheet\10793-CS100.dwg USER: alira DATE: Sep, 06 2019 TIME: 11:01 am



IBSP RESTORATION

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60099

IDNR # 2-17-008

Owner:

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30% DESIGN DRAWINGS		08/08/2019

SEALS AND SIGNATURES

NOT FOR CONSTRUCTION

KEY PLAN

PROJECT NORTH

DRAWING TITLE

AREA 2 - LAYOUT AND
MATERIALS PLAN

0' 10' 20' 40'

SCALE:

SCALE

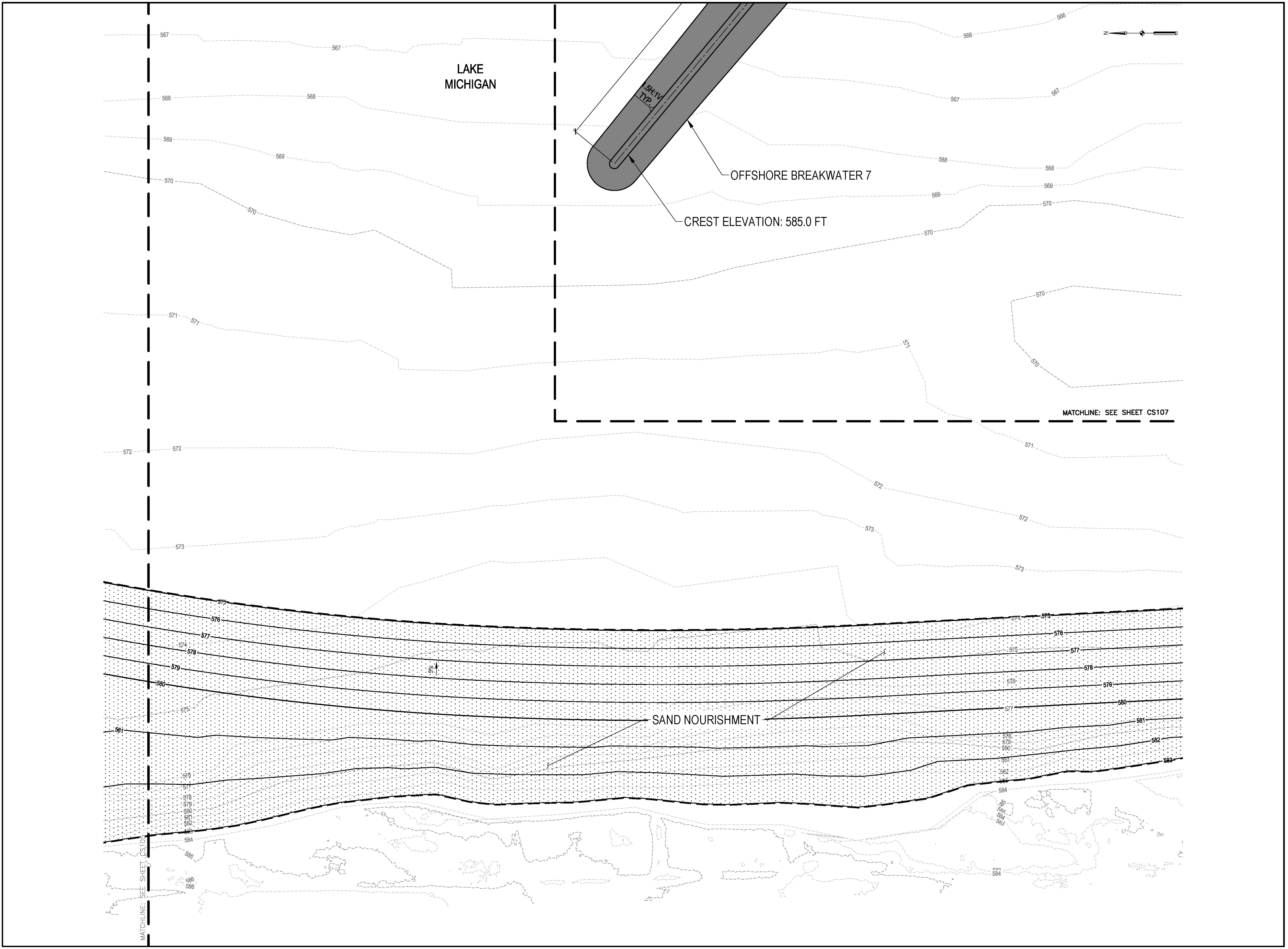
10793.000

PROJECT NUMBER

CS105

DRAWING NUMBER

FILE: P:\10793.000\CAD\C\Sheet\10793-CS100.dwg USER:alira DATE: Sep, 06 2019 TIME: 11:01 am



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IBSP RESTORATION

1 LAKE FRONT DRIVE, ZION, IL
60099

IDNR # 2-17-008

Owner:

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30% DESIGN DRAWINGS		08/08/2019

SEALS AND SIGNATURES

NOT FOR CONSTRUCTION

KEY PLAN

PROJECT NORTH

DRAWING TITLE

AREA 2 - LAYOUT AND
MATERIALS PLAN

0' 10' 20' 40'

SCALE:

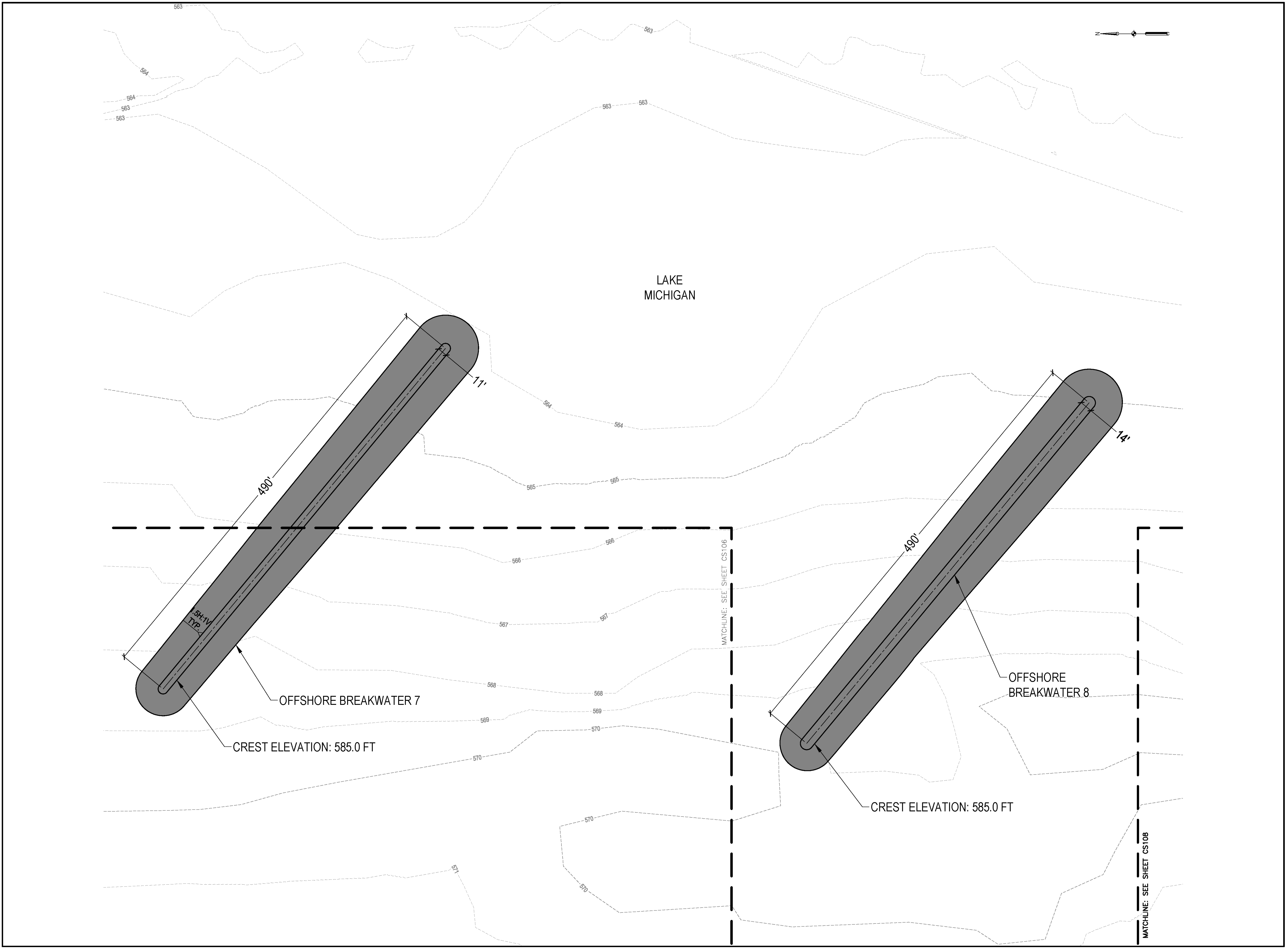
SCALE

10793.000

PROJECT NUMBER

CS106

DRAWING NUMBER



FILE: P:\10793.000\CAD\C\Sheet\10793-CS100.dwg USER: clira DATE: Sep. 06 2019 TIME: 11:01 am

IBSP RESTORATION

1 LAKE FRONT DRIVE, ZION, IL
60099

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Owner:

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30% DESIGN DRAWINGS		08/08/2019

SEALS AND SIGNATURES

NOT FOR CONSTRUCTION

KEY PLAN

PROJECT NORTH

DRAWING TITLE

AREA 2 - LAYOUT AND
MATERIALS PLAN

0' 10' 20' 40'

SCALE:

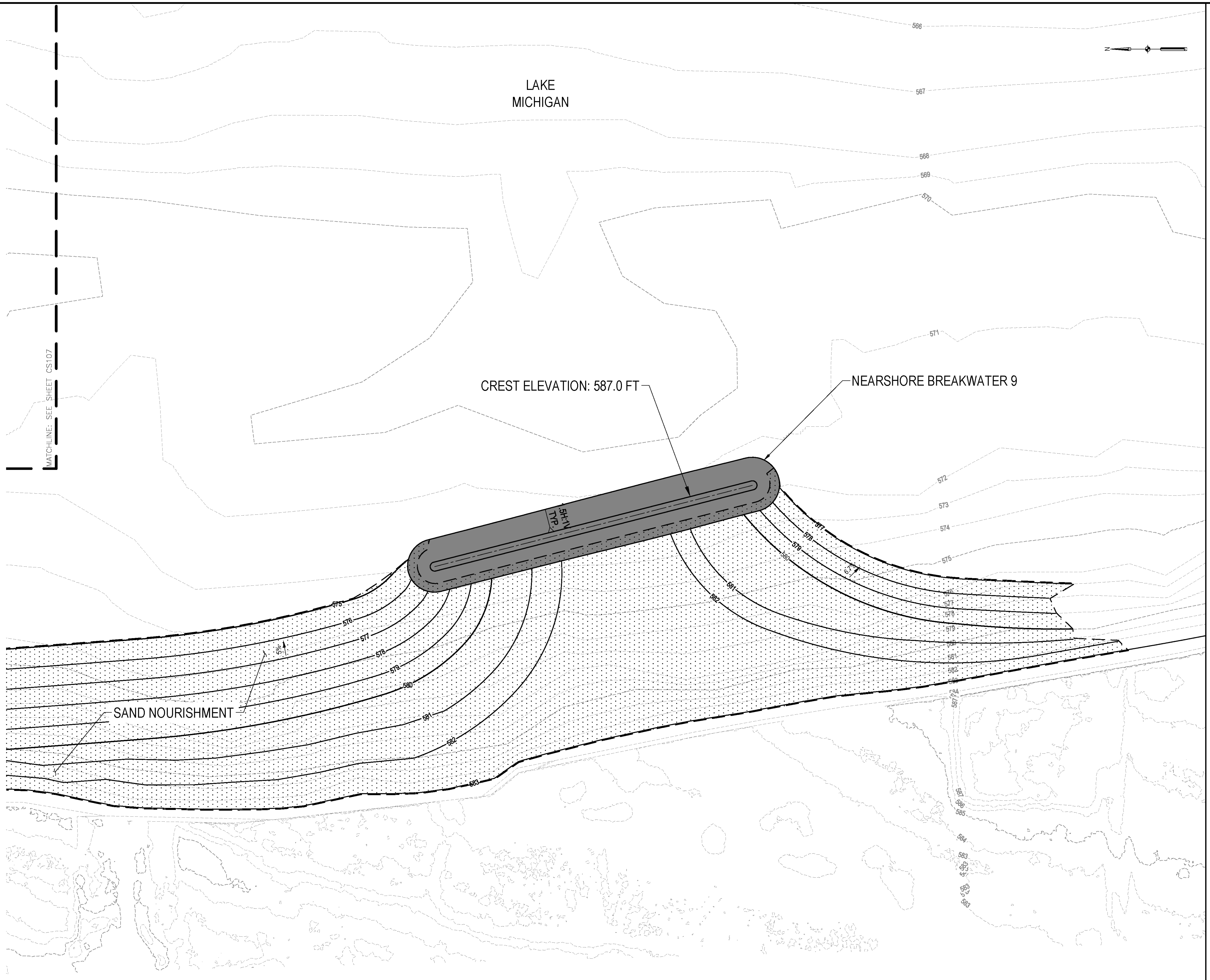
SCALE

10793.000

PROJECT NUMBER

CS107

DRAWING NUMBER



IBSP RESTORATION

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IDNR # 2-17-008

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ISSUED FOR	REV	DATE
30% DESIGN DRAWINGS		08/08/2019

SEALS AND SIGNATURES

NOT FOR CONSTRUCTION

KEY PLAN

PROJECT NORTH

DRAWING TITLE
AREA 2 - LAYOUT AND
MATERIALS PLAN

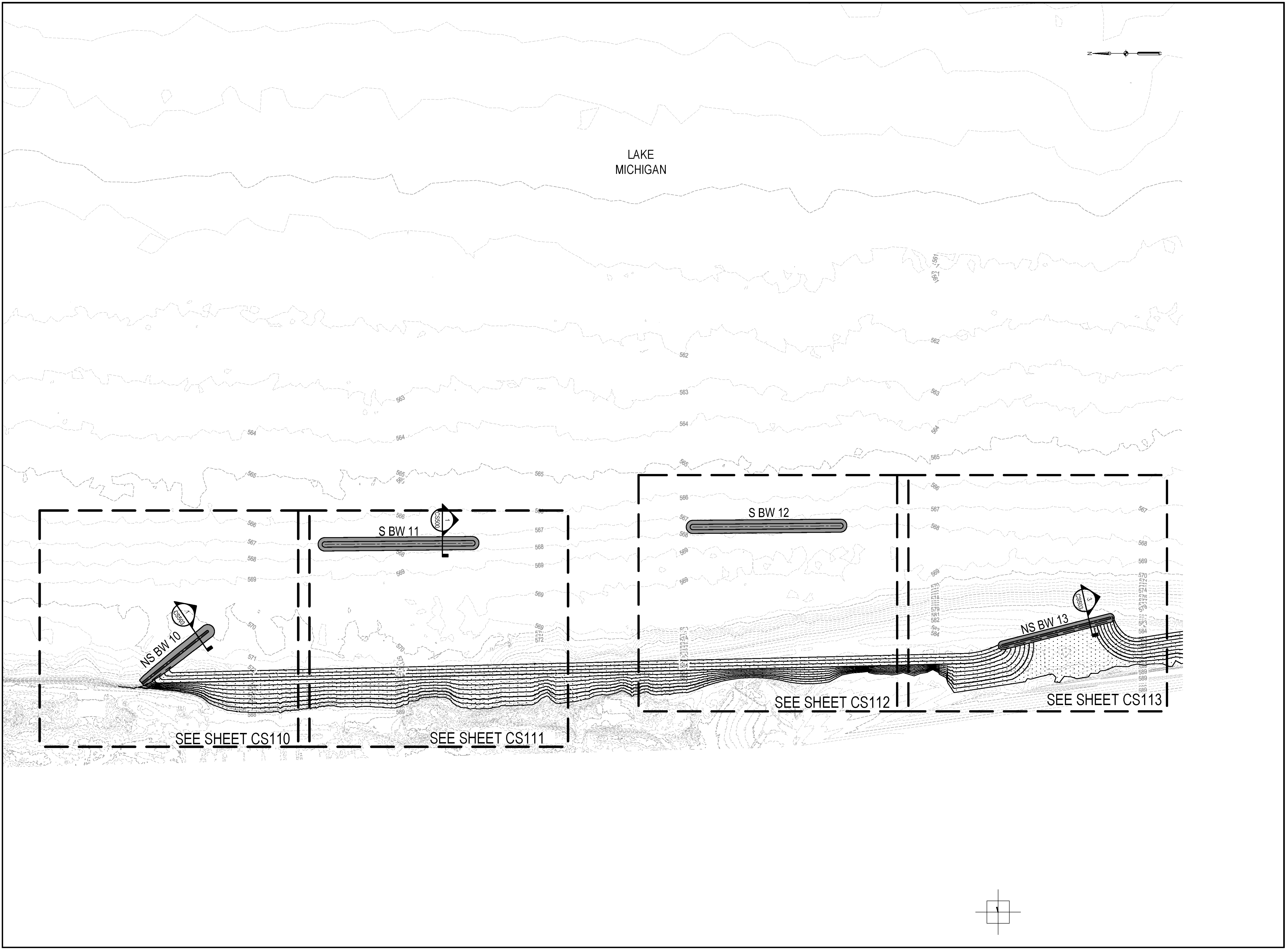
0' 10' 20' 40'
SCALE:

SCALE

10793.000
PROJECT NUMBER

CS108
DRAWING NUMBER

FILE: P:\10793.000\CAD\C\Sheet\10793-CS100.dwg USER:alira DATE: Sep, 06 2019 TIME: 11:01 am



IBSP RESTORATION

1 LAKE FRONT DRIVE, ZION, IL
60099

IDNR # 2-17-008

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30% DESIGN DRAWINGS		08/08/2019

SEALS AND SIGNATURES

NOT FOR CONSTRUCTION

KEY PLAN

PROJECT NORTH

DRAWING TITLE

OVERALL LAYOUT AND
MATERIALS - AREA 3

0' 10' 20' 40'

SCALE:

SCALE

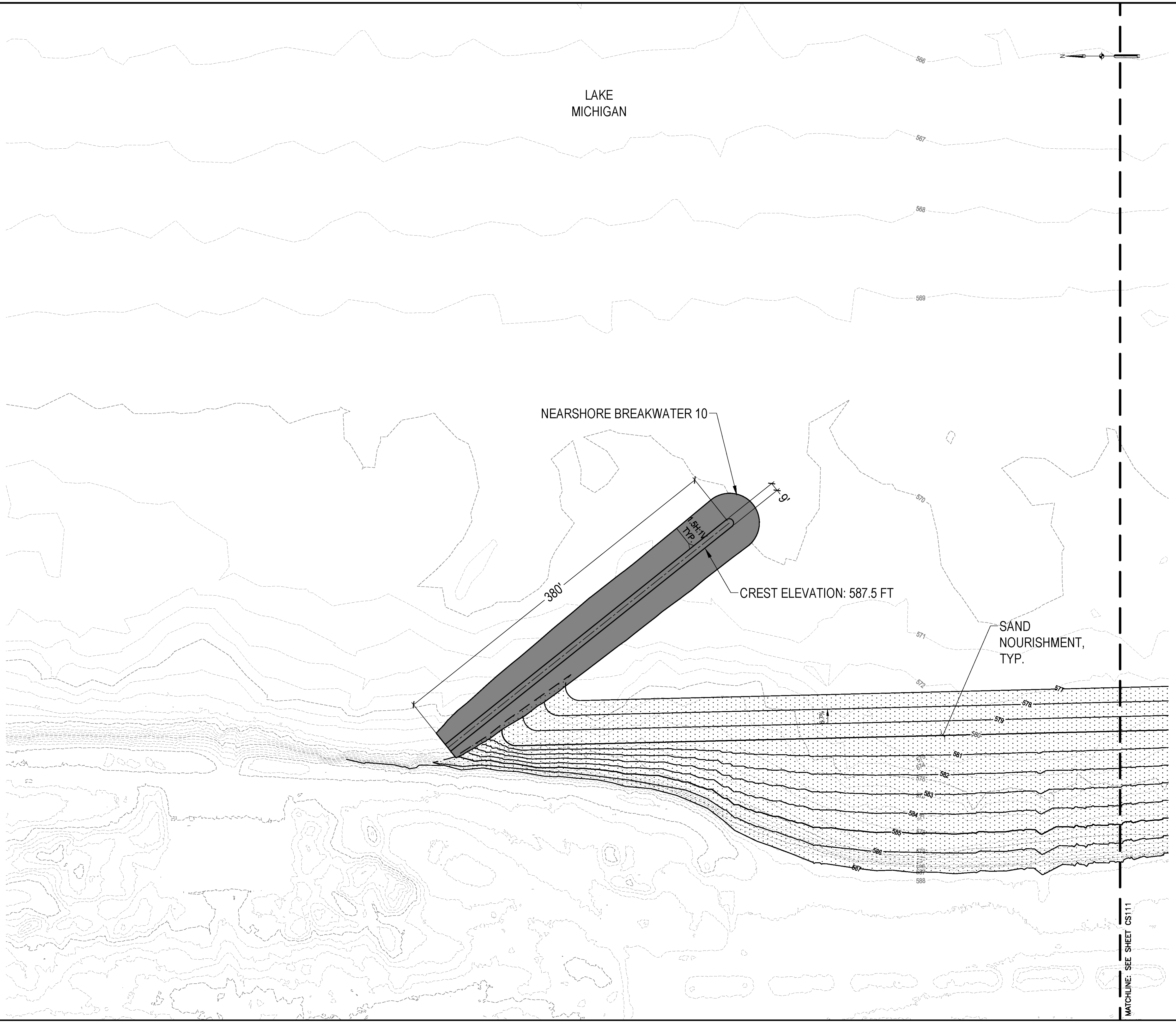
10793.000

PROJECT NUMBER

CS109

DRAWING NUMBER

FILE: P:\10793.000\CAD\C\Sheet\10793-CS100.dwg USER: alira DATE: Sep, 06 2019 TIME: 11:01 am



IBSP RESTORATION

1 LAKE FRONT DRIVE, ZION, IL
60099

IDNR # 2-17-008

Owner:

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ISSUED FOR	REV	DATE
30% DESIGN DRAWINGS		08/08/2019

SEALS AND SIGNATURES

NOT FOR CONSTRUCTION

KEY PLAN

PROJECT NORTH

DRAWING TITLE

AREA 3 - LAYOUT AND MATERIALS

0' 10' 20' 40'

SCALE:

SCALE

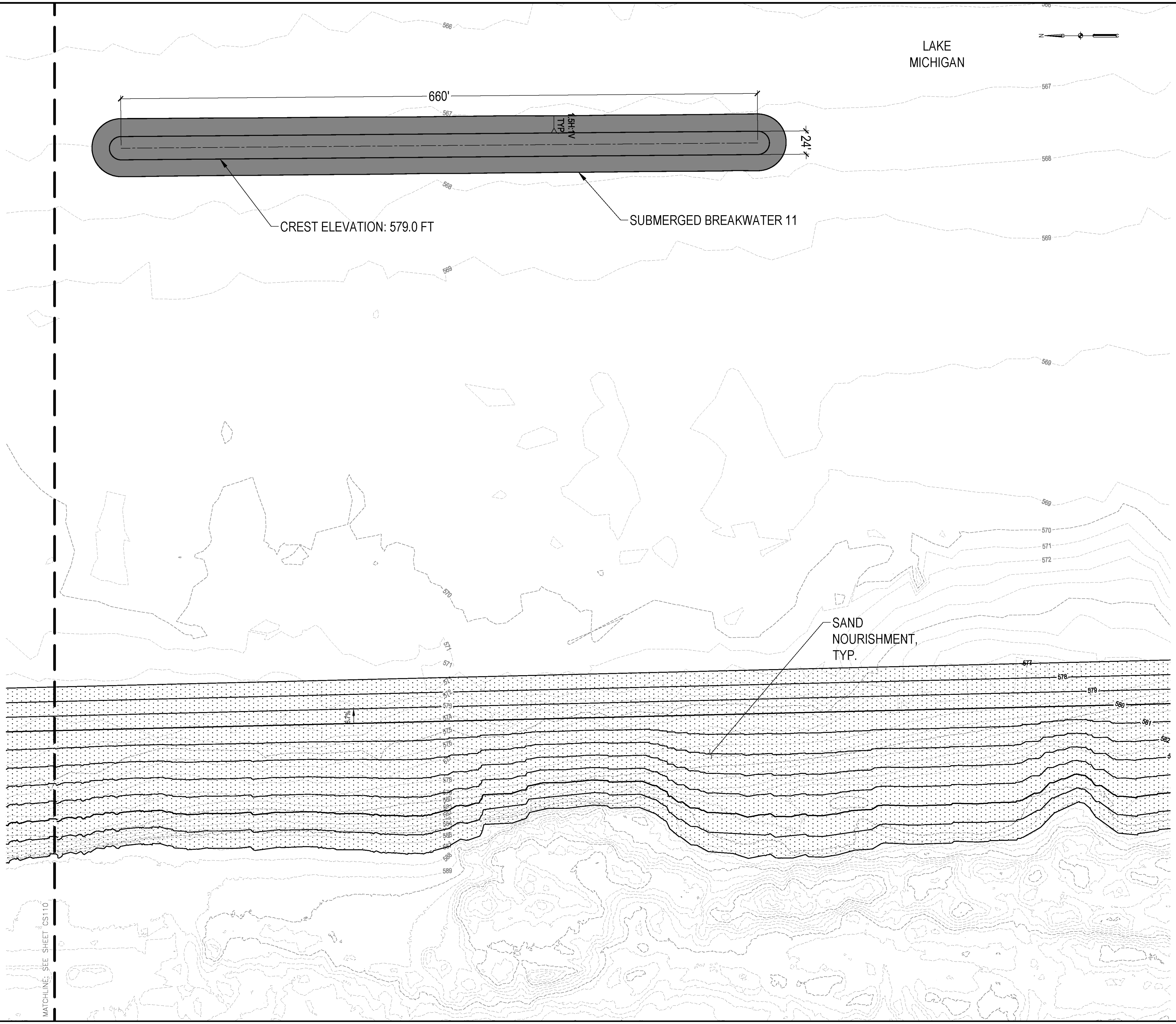
10793.000

PROJECT NUMBER

CS110

DRAWING NUMBER

FILE: P: \10793.000\CAD\C\Sheet\10793-CS100.dwg USER: alira DATE: Sep, 06 2019 TIME: 11:01 am



IBSP RESTORATION

1 LAKE FRONT DRIVE, ZION, IL
60099

IDNR # 2-17-008

Owner:

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ISSUED FOR	REV	DATE
30% DESIGN DRAWINGS		08/08/2019

SEALS AND SIGNATURES

NOT FOR CONSTRUCTION

KEY PLAN

PROJECT NORTH

DRAWING TITLE

AREA 3 - LAYOUT AND
MATERIALS

0' 10' 20' 40'

SCALE:

SCALE

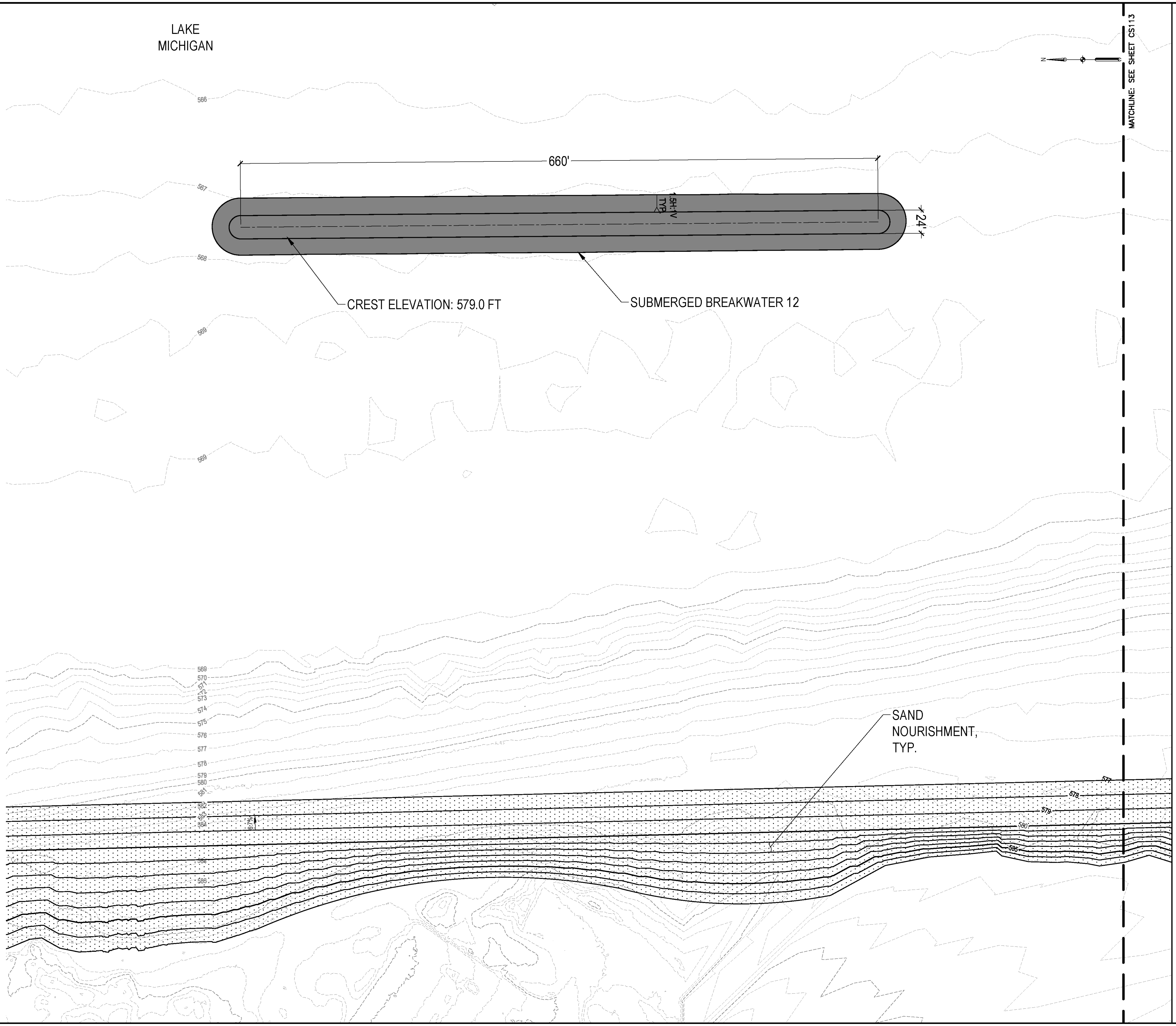
10793.000

PROJECT NUMBER

CS111

DRAWING NUMBER

FILE: P:\10793.000\CAD\C\Sheet\10793-CS100.dwg USER: alira DATE: Sep, 06 2019 TIME: 11:01 am



IBSP RESTORATION

1 LAKE FRONT DRIVE, ZION, IL
60099

IDNR # 2-17-008

Owner:

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ISSUED FOR	REV	DATE
30% DESIGN DRAWINGS		08/08/2019

SEALS AND SIGNATURES

NOT FOR CONSTRUCTION

KEY PLAN

PROJECT NORTH

DRAWING TITLE

AREA 3 - LAYOUT AND
MATERIALS

0' 10' 20' 40'

SCALE:

SCALE

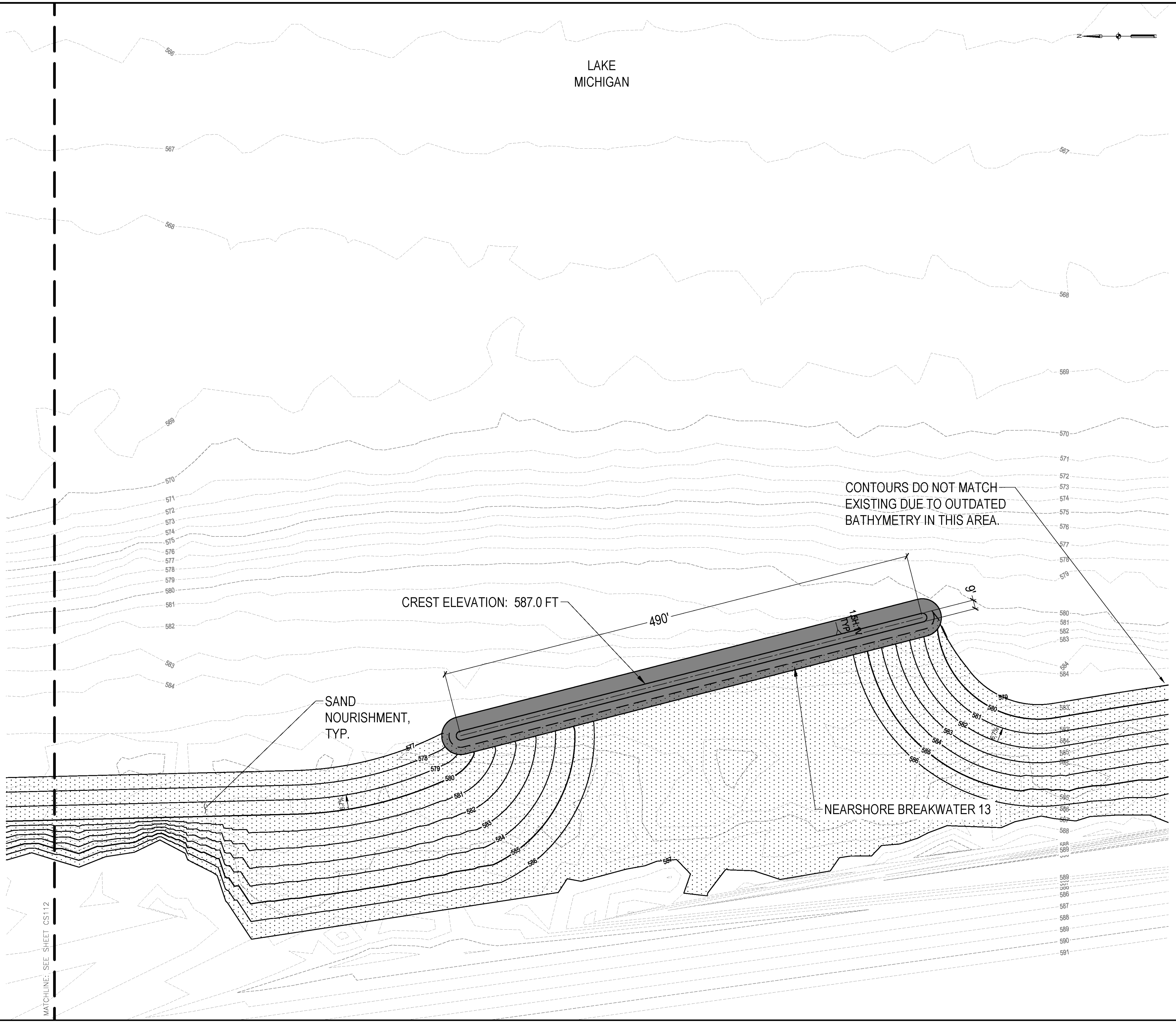
10793.000

PROJECT NUMBER

CS112

DRAWING NUMBER

FILE: P:\10793.000\CAD\C\Sheet\10793-CS100.dwg USER: alira DATE: Sep, 06 2019 TIME: 11:01 am



FILE: P:\10793.000\CAD\C\Sheet\10793-CS100.dwg USER: alira DATE: Sep, 06 2019 TIME: 11:01 am

IBSP RESTORATION

1 LAKE FRONT DRIVE, ZION, IL
60099

IDNR # 2-17-008

Owner:

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SEALS AND SIGNATURES

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KEY PLAN

PROJECT NORTH

DRAWING TITLE

AREA 3 - LAYOUT AND MATERIALS

0' 10' 20' 40'

SCALE:

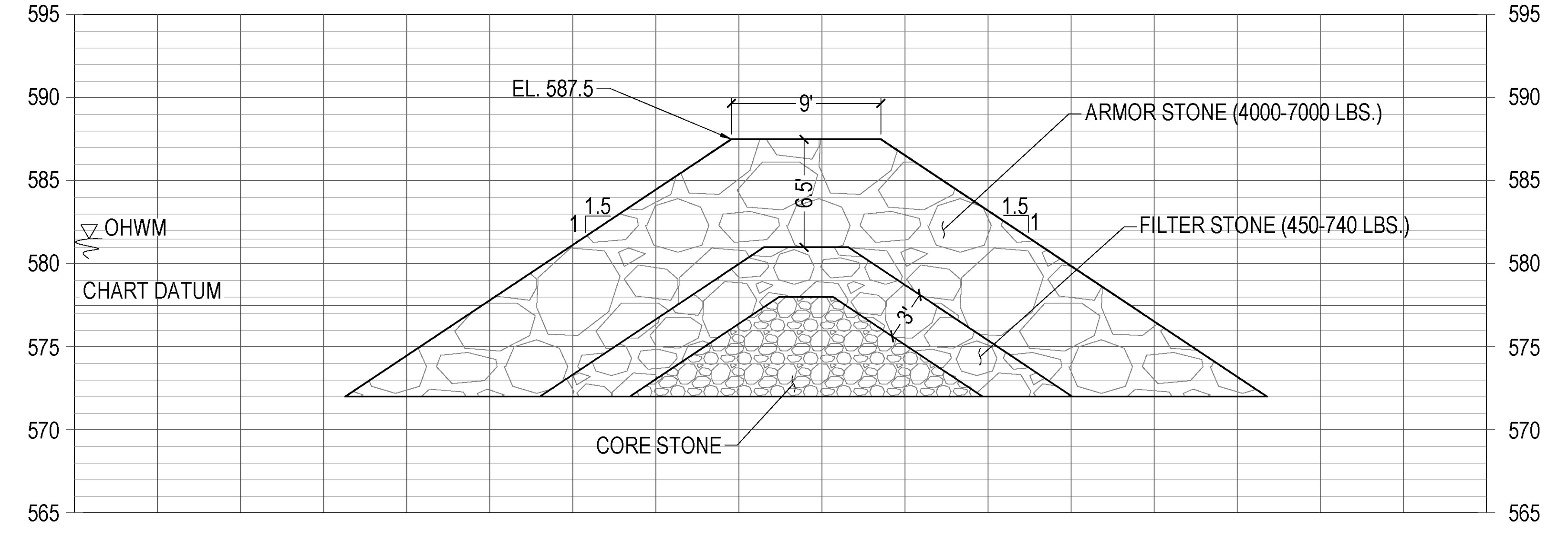
SCALE

10793.000

PROJECT NUMBER

CS113

DRAWING NUMBER



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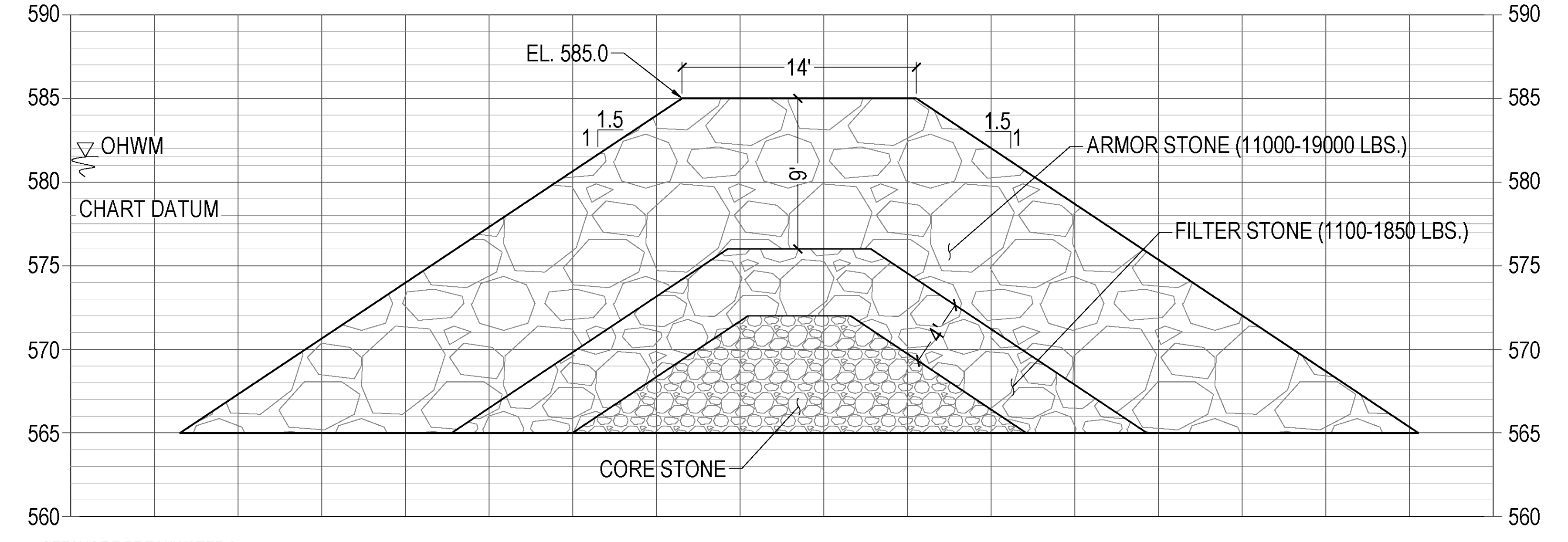

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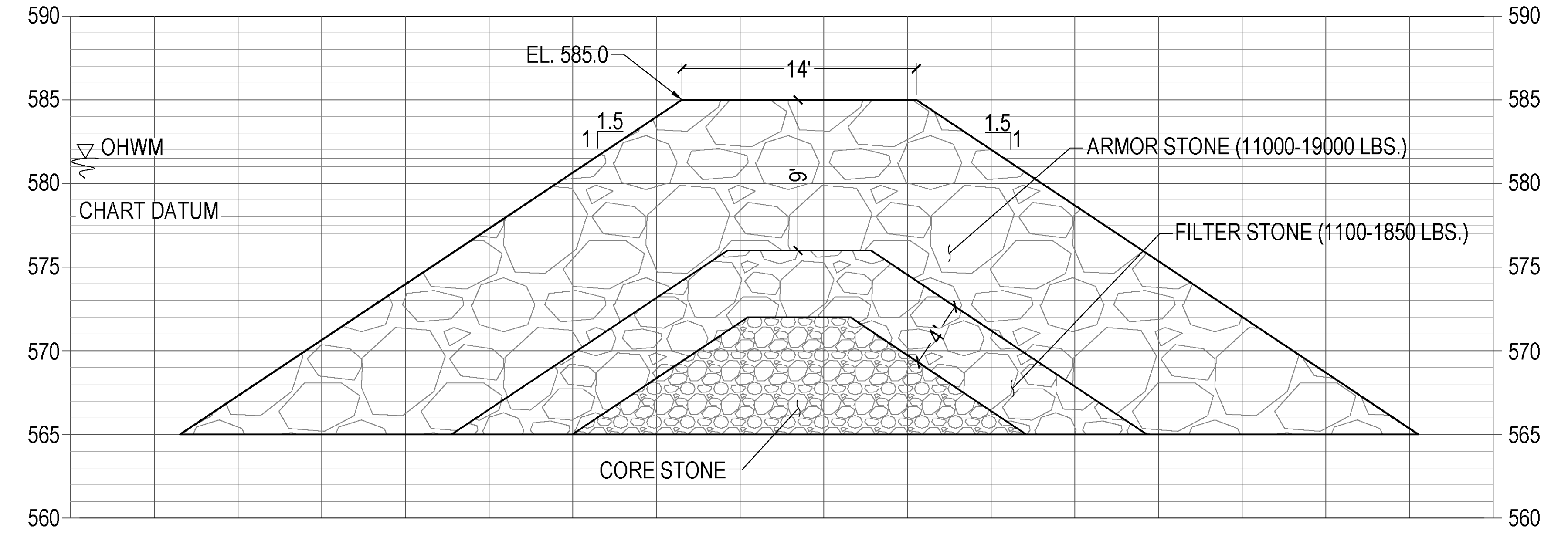

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IBSP RESTORATION

1 LAKE FRONT DRIVE, ZION, IL
60099

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KEY PLAN

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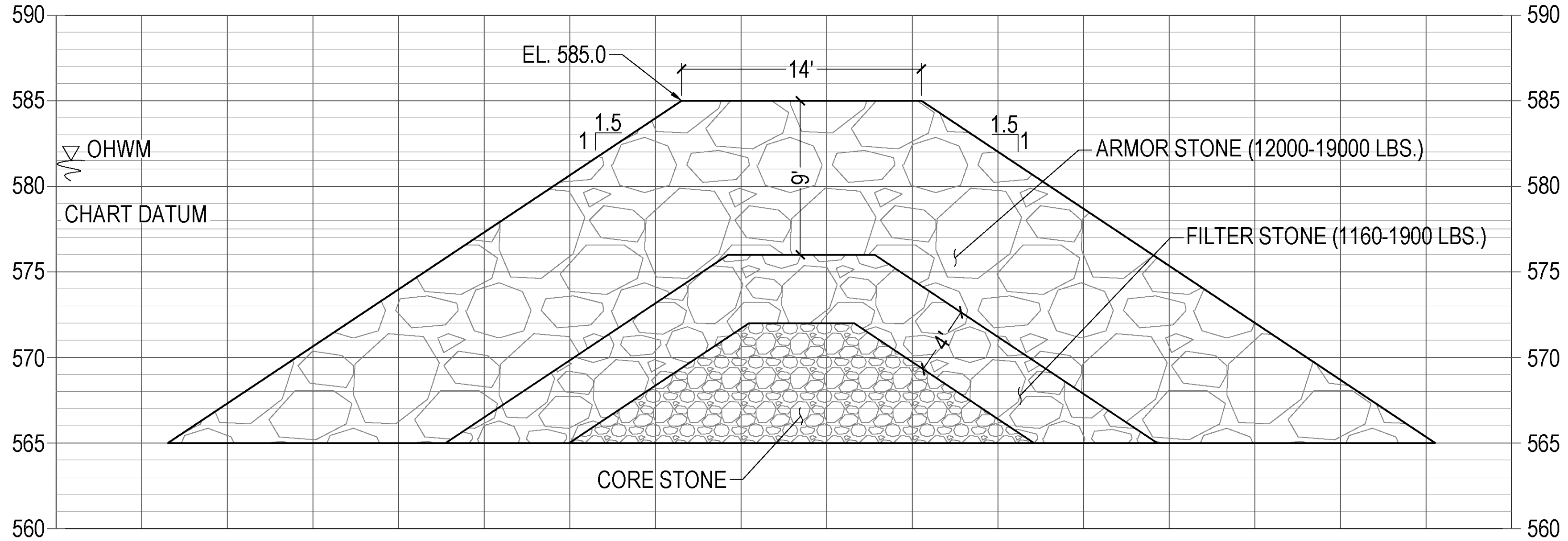
DRAWING TITLE
BREAKWATER SECTIONS

SCALE

10793.000
PROJECT NUMBER

CS500
DRAWING NUMBER



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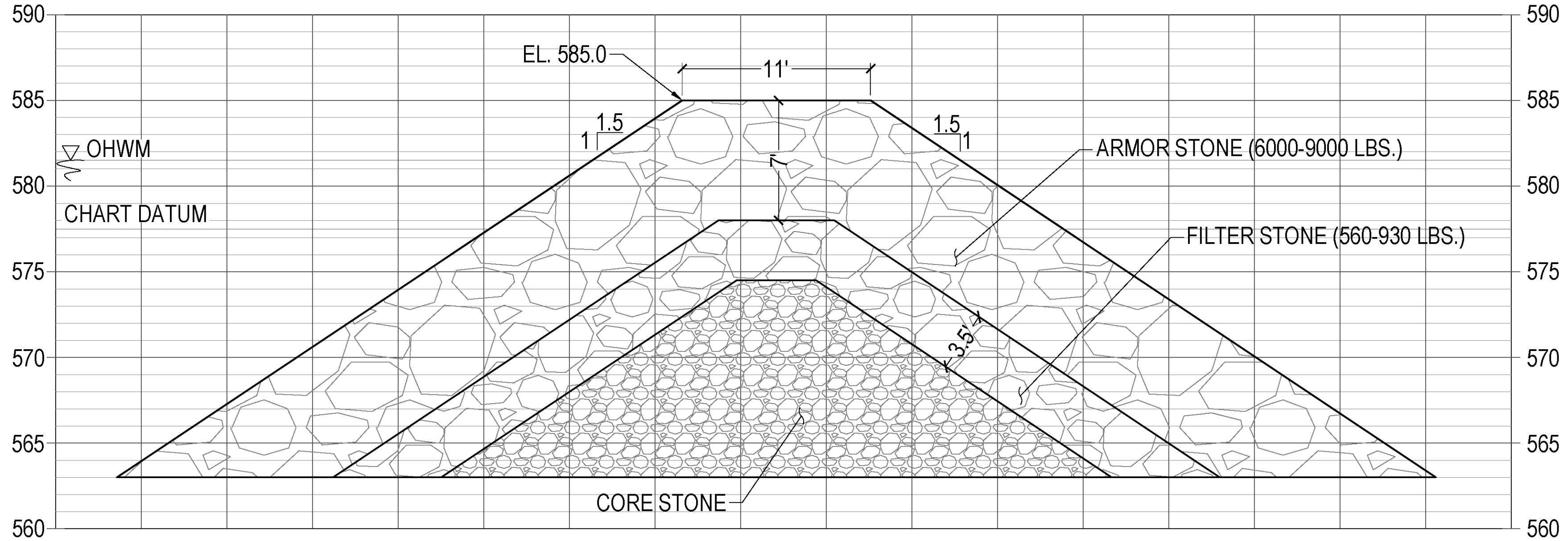

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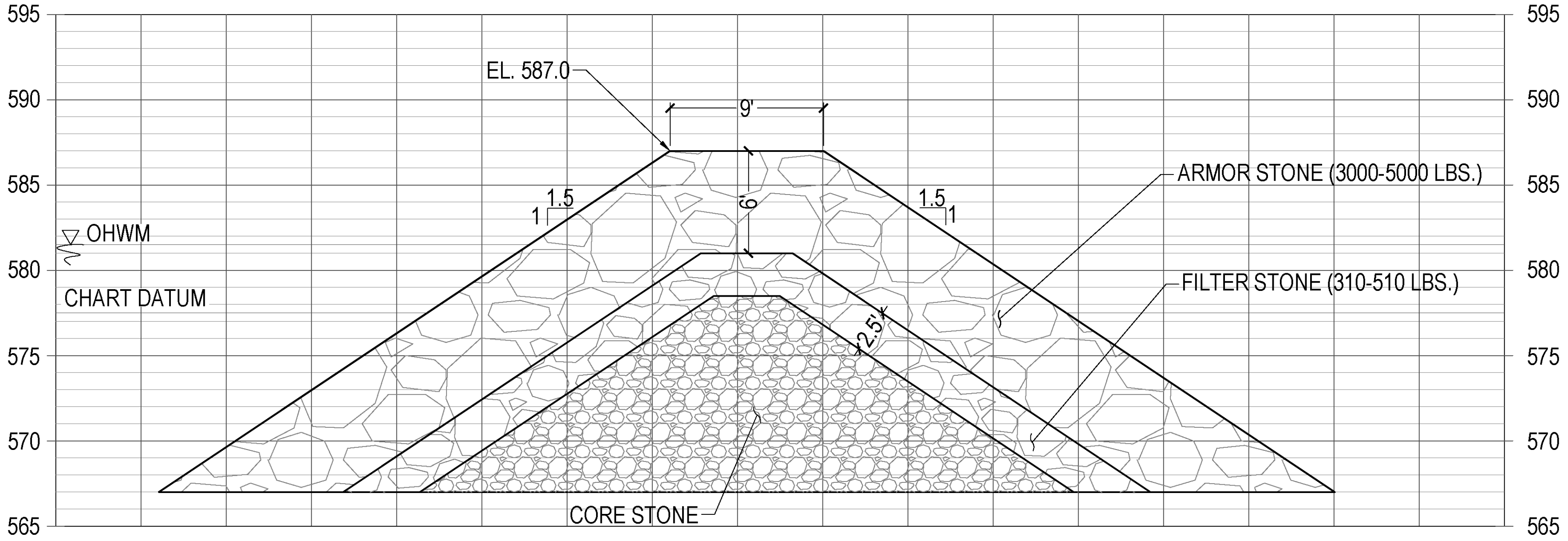

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KEY PLAN

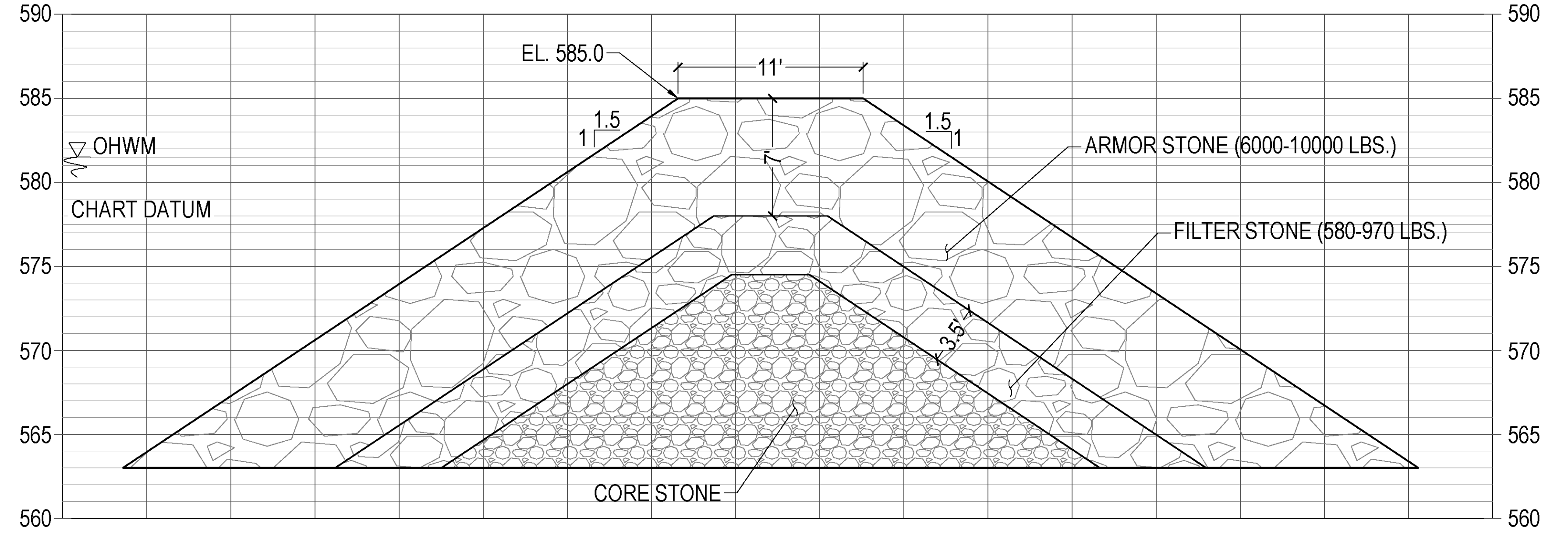
PROJECT NORTH

DRAWING TITLE
BREAKWATER SECTIONS

SCALE

10793.000
PROJECT NUMBER

CS501
DRAWING NUMBER



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SECTION
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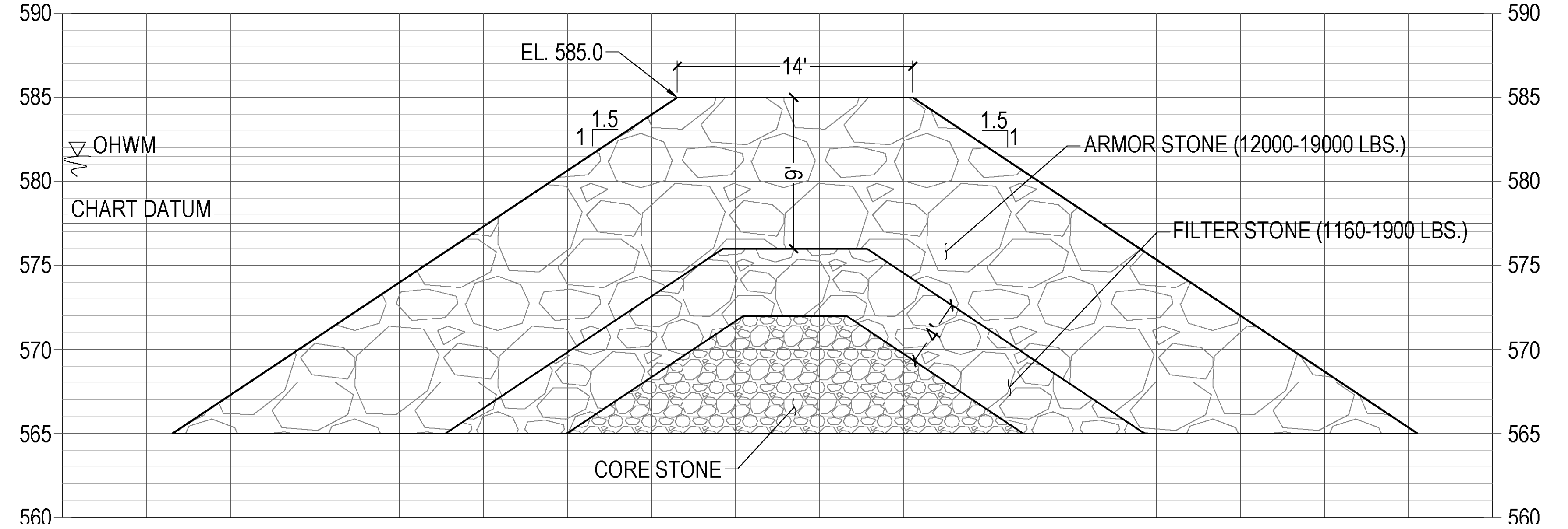

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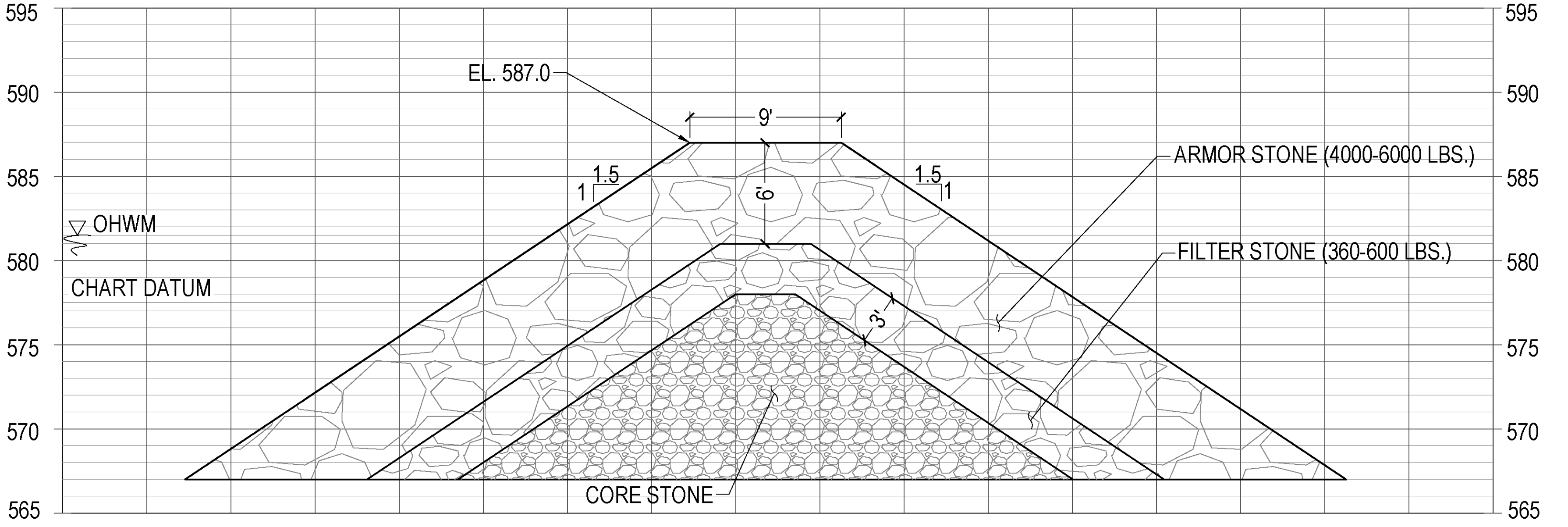
SCALE: 1"=5'



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SECTION
SCALE: 1"=5'

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1 LAKE FRONT DRIVE, ZION, IL
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KEY PLAN

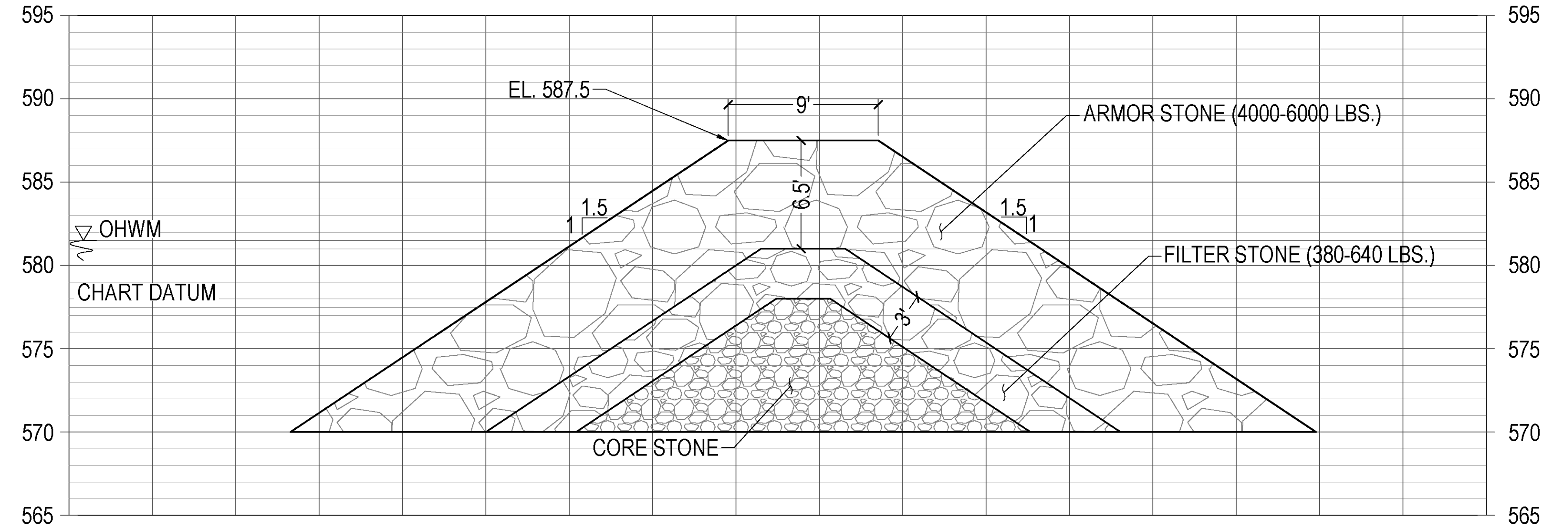
PROJECT NORTH

DRAWING TITLE
BREAKWATER SECTIONS

SCALE

10793.000
PROJECT NUMBER

CS502
DRAWING NUMBER



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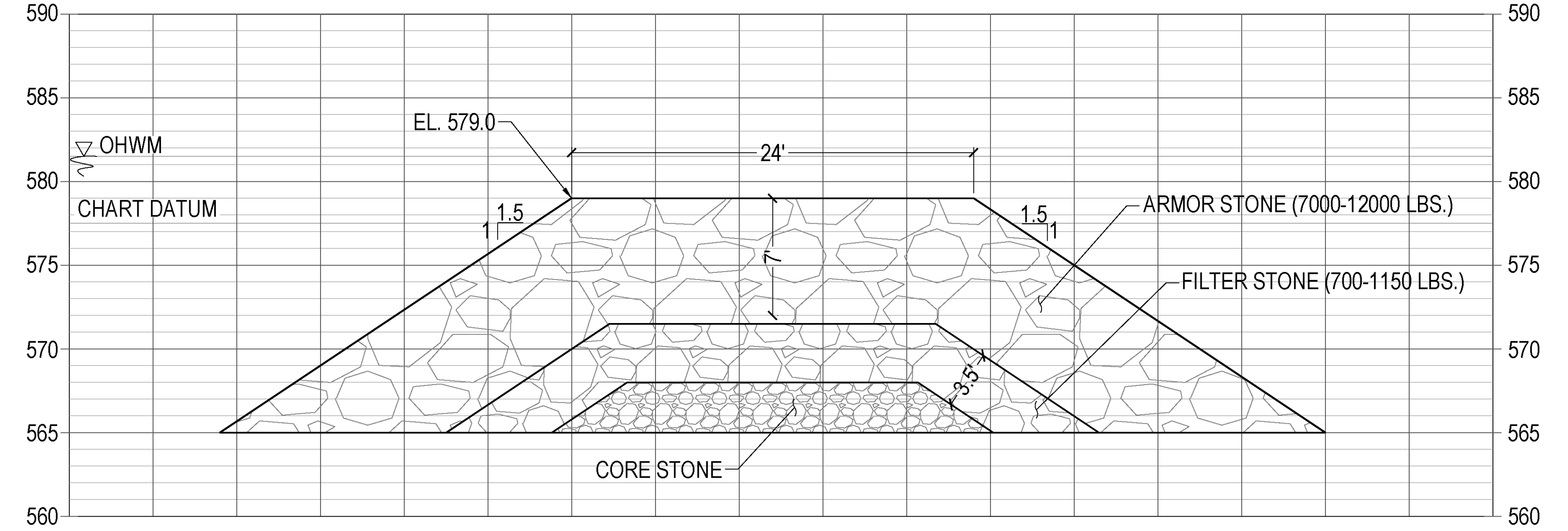
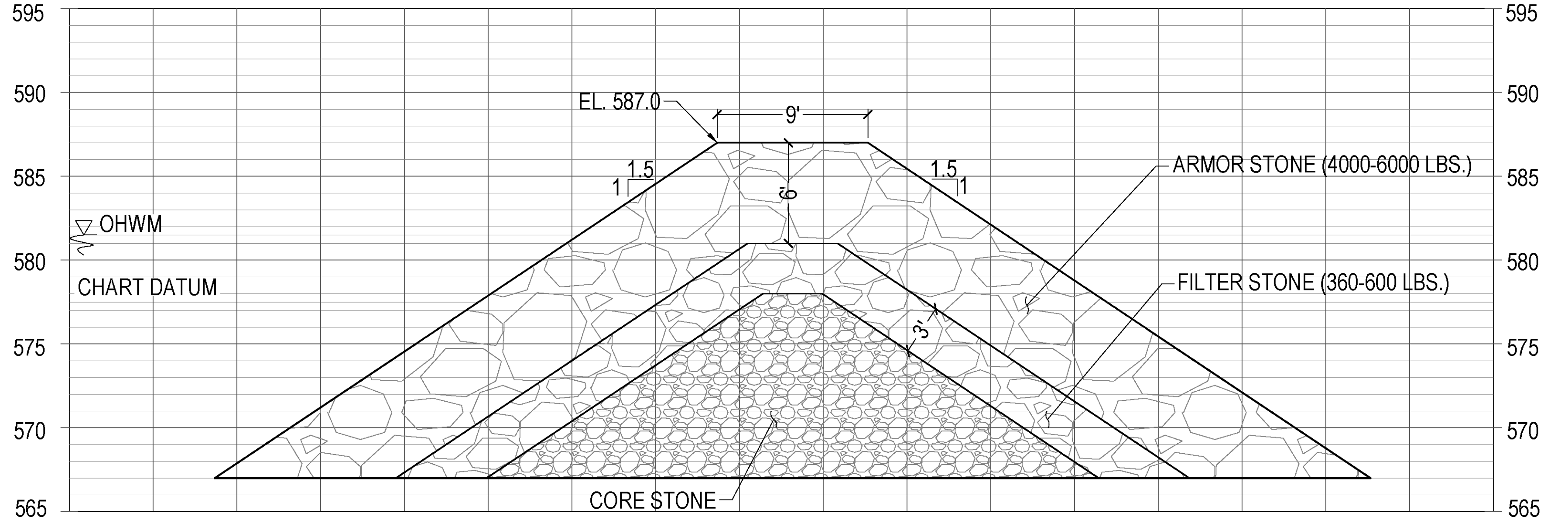


① SUBMERGED BREAKWATER 11 & 12 TYPICAL SECTION
SECTION
SCALE: 1"=5'



① NEARSHORE BREAKWATER 13
SECTION
SCALE: 1"=5'

IBSP RESTORATION

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60099

IDNR # 2-17-008

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KEY PLAN

PROJECT NORTH

DRAWING TITLE
BREAKWATER SECTIONS

SCALE

10793.000
PROJECT NUMBER

CS503
DRAWING NUMBER

APPENDIX K

Opinion of Probable Construction Cost

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Client	Illinois Department of Natural Resources
Project	Illinois Beach State Park Restoration
Project #	10793
Detail	30% Design
Date	8/16/2019

Item	Division	Item	Quantity	Unit	Unit Cost	Item Total	Subtotal
	01	General Requirements					$ 1,371,728.00
		Mobilization (4% Construction)	1	LS	$ 1,371,728.00	$ 1,371,728.00	
	02	Existing conditions					$ 80,000.00
		Subsurface Investigation	1	LS	$ 80,000.00	$ 80,000.00	
	35	Waterways and Marine Construction					$ 34,213,200.00
		Area 1 - North Point					**$ 14,449,800.00**
		Nourishment Sand	18600	Ton	$ 18.00	$ 334,800.00	
		Nearshore Breakwater 1				**$ 1,830,000.00**	
		Rip Rap Revetments & Breakwaters	12200	Ton	$ 150.00	$ 1,830,000.00	
		Offshore Breakwater 2				**$ 3,510,000.00**	
		Rip Rap Revetments & Breakwaters	23400	Ton	$ 150.00	$ 3,510,000.00	
		Offshore Breakwater 3				**$ 3,360,000.00**	
		Rip Rap Revetments & Breakwaters	22400	Ton	$ 150.00	$ 3,360,000.00	
		Offshore Breakwater 4				**$ 3,555,000.00**	
		Rip Rap Revetments & Breakwaters	23700	Ton	$ 150.00	$ 3,555,000.00	
		Offshore Breakwater 5				**$ 1,860,000.00**	
		Rip Rap Revetments & Breakwaters	12400	Ton	$ 150.00	$ 1,860,000.00	
		Area 2 - Camp Logan					**$ 11,490,600.00**
		Nourishment Sand	101700	Ton	$ 18.00	$ 1,830,600.00	
		Nearshore Breakwater 6				**$ 1,590,000.00**	
		Rip Rap Revetments & Breakwaters	10600	Ton	$ 150.00	$ 1,590,000.00	
		Offshore Breakwater 7				**$ 3,315,000.00**	
		Rip Rap Revetments & Breakwaters	22100	Ton	$ 150.00	$ 3,315,000.00	
		Offshore Breakwater 8				**$ 3,300,000.00**	
		Rip Rap Revetments & Breakwaters	22000	Ton	$ 150.00	$ 3,300,000.00	
		Nearshore Breakwater 9				**$ 1,455,000.00**	
		Rip Rap Revetments & Breakwaters	9700	Ton	$ 150.00	$ 1,455,000.00	
		Area 3 - Swimming Beach					**$ 8,272,800.00**
		Nourishment Sand	39600	Ton	$ 18.00	$ 712,800.00	
		Nearshore Breakwater 10				**$ 1,575,000.00**	
		Rip Rap Revetments & Breakwaters	10500	Ton	$ 150.00	$ 1,575,000.00	
		Submerged Breakwater 11				**$ 2,760,000.00**	
		Rip Rap Revetments & Breakwaters	18400	Ton	$ 150.00	$ 2,760,000.00	
		Submerged Breakwater 12				**$ 2,760,000.00**	
		Rip Rap Revetments & Breakwaters	18400	Ton	$ 150.00	$ 2,760,000.00	
		Nearshore Breakwater 13				**$ 465,000.00**	
		Rip Rap Revetments & Breakwaters	3100	Ton	$ 150.00	$ 465,000.00	
		Construction Subtotal					**$ 35,664,928**
		Bonds and Insurance	0%				$ -
		Contractor Fee	0%				$ -
		Phasing	0%				$ -
		Escalator	2.0%			0 years	$ -
		Construction Total					**$ 35,664,928**
		Design/Engineering/Permits	6%				$ 2,139,900.00
		Construction Contingency & Remaining Elements	20%				$ 7,133,000.00
		Project Total (Construction, design, contingency and permitting)					**$ 44,937,828**

APPENDIX L

Structure Design Methodology

Nearshore Breakwater 1

Wave Overtopping & Armor Calculation
Based on Method in EM1100-2-1100 Coastal Engineering Manual, USACE, Change 3, 2011

Input Data

WL_{Design} =	581	ft	Design Water Level, IGLD 1985	95% WL
H_s =	6	ft	Significant wave height	offshore, 100yr storm
T_p =	11	s	Peak wave period	
WL_{Toe} =	571.5	ft	Deepest Toe Location	
d =	9.50	ft	Depth of water at structure toe, shale lake bottom	
L_p =	189	ft	Local wave length, based on peak period	
cot θ =	1.5	-	Breakwater Slope	

Overtopping Estimation

Alternate Crest Elevation = 587.5 ft

Q' =	0.0001	-	Dimensionless overtopping	
F' =	0.343	-	Dimensionless freeboard	
Q =	0.006	cfs/ft	Wave overtopping	
	0.054	cfs/ft	Acceptable Value, rear side no damage	5 l/s per m

$$Q' = C_0 e^{C_1 F'} e^{C_2 m} \tag{2-9}$$

$$F' = \frac{F}{\left(H_{mo}^2 L_o\right)^{1/3}} \tag{2-11}$$

$$Q' = \frac{Q}{\left(g H_{mo}^3\right)^{1/2}} \tag{2-10}$$

*EM 1100-2-1614 Design of Coastal Revetments Seawalls and Bulkheads (USACE 1995)

Breakwater Design
Based on Method in EM 1110-2-1100 Coastal Engineering Manual, USACE, Change 3, 2011

Input Data

S_r =	2.6	-	Specific gravity, Limestone
ρ_{stone} =	165.0	lb/ft^3	Density of armorstone
ρ_{water} =	62.4	lb/ft^3	Density of fresh water
cot θ =	1.5	-	Revetment Slope

Armor Layer Breakwater Design, Hudson 1974, SPM 1984

K_d =	2	-	Stability coefficient, Rough anglular stone, 60% water depth breaking criteria
$H_{1/10}$ =	7.62	ft	
M_{50} =	5941	lbs	Medium mass of rocks
	3	tons	
D_{n50} =	3.3	ft	Equivalent cube length of median rock

$$M_{50} = \frac{\rho_s H^3}{K_D\left(\frac{\rho_s}{\rho_w} - 1\right)^3 \cot\alpha} \tag{VI-5-67}$$

K_D-values by SPM 1984, $H = H_{1/10}$.

Stone shape	Placement	Damage, D^4 = 0-5%	
		Breaking waves [1]	Nonbreaking waves [2]
Smooth rounded	Random	1.2	2.4
Rough angular	Random	2.0	4.0
Rough angular	Special [3]	5.8	7.0

*EM 1110-2-1100 Coastal Engineering Manual (USACE, 2002)

Armor Layer Gradation

M_{max} =	11.9	tons	D_{max} =	5.2	ft
M_{85} =	3.7	tons	D_{85} =	3.6	ft
M_{50} =	3.0	tons	D_{50} =	3.3	ft
M_{15} =	2.2	tons	D_{15} =	3.0	ft
M_{min} =	0.4	tons	D_{min} =	1.7	ft

Recommendation: 2 - 3.5 tons / 6.5 ft thickness

Filter Layer Gradation

M_{50} =	594	lbs	*10% of armor mass	D_{50} =	1.5	ft

Recommendation: 3 ft thickness

Geotextile

Recommendation: Due to fine-grained composition of the beach behind, a geotextile is recommended

Toe Protection

Assumption: Toe structure may be exposed to wave action. Therefore Hudson equation (VI-5-67, CEM) should be used to size stone per EM 1110-2-1614 direction. All variables remaining equal, armor stone should be used as toe protection stone.

Recommendation: 2 - 3.5 tons

Offshore Breakwater 2

Wave Overtopping & Armor Calculation
Based on Method in EM1100-2-1100 Coastal Engineering Manual, USACE, Change 3, 2011

Input Data

WL_{Design} =	581	ft	Design Water Level, IGLD 1985	95% WL
H_s =	8.2	ft	Significant wave height	offshore, 100yr storm
T_p =	11	s	Peak wave period	
WL_{Toe} =	564.5	ft	Deepest Toe Location	
d =	16.50	ft	Depth of water at structure toe, shale lake bottom	
L_p =	246	ft	Local wave length, based on peak period	
cot θ =	1.5	-	Revetment Slope	

Overtopping Estimation

Alternate Crest Elevation = 585.0 ft

Q' =	0.0160	-	Dimensionless overtopping	
F' =	0.157	-	Dimensionless freeboard	
Q =	2.138	cfs/ft	Wave overtopping	
	2.153	cfs/ft	Acceptable Value, rear side no damage	200 l/s per m

$$Q' = C_0 e^{C_1 F'} e^{C_2 m} \qquad (2\text{-}9)$$

$$F' = \frac{F}{\left(H_{mo}^2 L_o\right)^{1/3}} \qquad (2\text{-}11)$$

$$Q' = \frac{Q}{\left(g H_{mo}^3\right)^{1/2}} \qquad (2\text{-}10)$$

*EM 1100-2-1614 Design of Coastal Revetments Seawalls and Bulkheads (USACE 1995)

Breakwater Design
Based on Method in EM 1110-2-1100 Coastal Engineering Manual, USACE, Change 3, 2011

Input Data

S_r =	2.6	-	Specific gravity, Limestone
ρ_{stone} =	162.2	lb/ft^3	Density of armorstone
ρ_{water} =	62.4	lb/ft^3	Density of fresh water
cot θ =	1.5	-	Breakwater Slope

Armor Layer Breakwater Design, Hudson 1974, SPM 1984

K_d =	2	-	Stability coefficient, Rough anglular stone, 60% water depth breaking criteria
$H_{1/10}$ =	10.41	ft	
M_{50} =	14912	lbs	Medium mass of rocks
	7.5	tons	
D_{n50} =	4.5	ft	Equivalent cube length of median rock

$$M_{50} = \frac{\rho_s H^3}{K_D \left(\frac{\rho_s}{\rho_w} - 1\right)^3 \cot \alpha} \qquad (VI-5-67)$$

K_D-values by SPM 1984, $H = H_{1/10}$.

Stone shape	Placement	Damage, D^4 = 0-5%	
		Breaking waves [1]	Nonbreaking waves [2]
Smooth rounded	Random	1.2	2.4
Rough angular	Random	2.0	4.0
Rough angular	Special [3]	5.8	7.0

*EM 1110-2-1100 Coastal Engineering Manual (USACE, 2002)

Armor Layer Gradation

M_{max} =	29.8	tons	D_{max} =	7.2	ft
M_{85} =	9.3	tons	D_{85} =	4.9	ft
M_{50} =	7.5	tons	D_{50} =	4.5	ft
M_{15} =	5.6	tons	D_{15} =	4.1	ft
M_{min} =	0.9	tons	D_{min} =	2.3	ft

Recommendation: 5.5 - 9.5 tons / 9 ft thickness

Filter Layer Gradation

M_{50} = 1491 lbs *10% of armor mass D_{50} = 2.1 ft

Recommendation: 4 ft thickness

Geotextile

Recommendation: No Geotextile necessary for offshore breakwaters

Toe Protection

Assumption:

Toe structures in offshore breakwaters are unlikely to be exposed to wave action. To be refined in final engineering, filter stone should be used as a minimum as toe protection stone.

Recommendation: 1125 - 1875 lbs

Offshore Breakwater 3

Wave Overtopping & Armor Calculation
Based on Method in EM1100-2-1100 Coastal Engineering Manual, USACE, Change 3, 2011

Input Data

WL_{Design} =	581	ft	Design Water Level, IGLD 1985	95% WL
H_s =	8.2	ft	Significant wave height	offshore, 100yr storm
T_p =	11	s	Peak wave period	
WL_{Toe} =	565	ft	Deepest Toe Location	
d =	16.00	ft	Depth of water at structure toe, shale lake bottom	
L_p =	243	ft	Local wave length, based on peak period	
cot θ =	1.5	-	Revetment Slope	

Overtopping Estimation

Alternate Crest Elevation = 585.0 ft

Q' =	0.0157	-	Dimensionless overtopping	
F' =	0.158	-	Dimensionless freeboard	
Q =	2.090	cfs/ft	Wave overtopping	
	2.153	cfs/ft	Acceptable Value, rear side no damage	200 l/s per m

$$Q' = C_0 e^{C_1 F'} e^{C_2 m} \qquad (2\text{-}9)$$

$$F' = \frac{F}{\left(H_{mo}^2 L_o\right)^{1/3}} \qquad (2\text{-}11)$$

$$Q' = \frac{Q}{\left(g H_{mo}^3\right)^{1/2}} \qquad (2\text{-}10)$$

*EM 1100-2-1614 Design of Coastal Revetments Seawalls and Bulkheads (USACE 1995)

Breakwater Design
Based on Method in EM 1110-2-1100 Coastal Engineering Manual, USACE, Change 3, 2011

Input Data

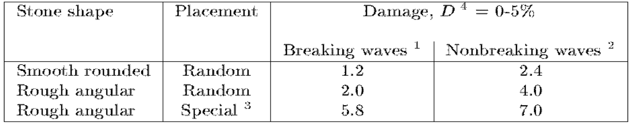
S_r =	2.6	-	Specific gravity, Limestone
ρ_{stone} =	162.2	lb/ft^3	Density of armorstone
ρ_{water} =	62.4	lb/ft^3	Density of fresh water
cot θ =	1.5	-	Breakwater Slope

Armor Layer Breakwater Design, Hudson 1974, SPM 1984

K_d =	2	-	Stability coefficient, Rough anglular stone, 60% water depth breaking criteria
$H_{1/10}$ =	10.41	ft	
M_{50} =	14912	lbs	Medium mass of rocks
	7.5	tons	
D_{n50} =	4.5	ft	Equivalent cube length of median rock

$$M_{50} = \frac{\rho_s H^3}{K_D \left(\frac{\rho_s}{\rho_w} - 1\right)^3 \cot\alpha} \qquad (VI-5-67)$$

K_D-values by SPM 1984, $H = H_{1/10}$.



Stone shape	Placement	Damage, D^4 = 0-5%	
		Breaking waves [1]	Nonbreaking waves [2]
Smooth rounded	Random	1.2	2.4
Rough angular	Random	2.0	4.0
Rough angular	Special [3]	5.8	7.0

*EM 1110-2-1100 Coastal Engineering Manual (USACE, 2002)

Armor Layer Gradation

M_{max} =	29.8	tons	D_{max} =	7.2	ft
M_{85} =	9.3	tons	D_{85} =	4.9	ft
M_{50} =	7.5	tons	D_{50} =	4.5	ft
M_{15} =	5.6	tons	D_{15} =	4.1	ft
M_{min} =	0.9	tons	D_{min} =	2.3	ft

Recommendation: 5.5 - 9.5 tons / 9 ft thickness

Filter Layer Gradation

M_{50} =	1491	lbs	*10% of armor mass	D_{50} =	2.1	ft

Recommendation: 4 ft thickness

Geotextile

Recommendation: No Geotextile necessary for offshore breakwaters

Toe Protection

Assumption:

Toe structures in offshore breakwaters are unlikely to be exposed to wave action. To be refined in final engineering, filter stone should be used as a minimum as toe protection stone.

Recommendation: 1125 - 1875 lbs

Offshore Breakwater 4

Wave Overtopping & Armor Calculation
Based on Method in EM1100-2-1100 Coastal Engineering Manual, USACE, Change 3, 2011

Input Data

WL_{Design} =	581	ft	Design Water Level, IGLD 1985	95% WL
H_s =	8.3	ft	Significant wave height	offshore, 100yr storm
T_p =	11	s	Peak wave period	
WL_{Toe} =	563.5	ft	Deepest Toe Location	
d =	17.50	ft	Depth of water at structure toe, shale lake bottom	
L_p =	253	ft	Local wave length, based on peak period	
cot θ =	1.5	-	Revetment Slope	

Overtopping Estimation

Alternate Crest Elevation = 585.0 ft

Q' =	0.0174	-	Dimensionless overtopping	
F' =	0.154	-	Dimensionless freeboard	
Q =	2.357	cfs/ft	Wave overtopping	
	2.153	cfs/ft	Acceptable Value, rear side no damage	200 l/s per m

$$Q' = C_0 e^{C_1 F'} e^{C_2 m} \qquad (2\text{-}9)$$

$$F' = \frac{F}{\left(H_{mo}^2 L_o\right)^{1/3}} \qquad (2\text{-}11)$$

$$Q' = \frac{Q}{\left(g H_{mo}^3\right)^{1/2}} \qquad (2\text{-}10)$$

*EM 1100-2-1614 Design of Coastal Revetments Seawalls and Bulkheads (USACE 1995)

Breakwater Design
Based on Method in EM 1110-2-1100 Coastal Engineering Manual, USACE, Change 3, 2011

Input Data

S_r =	2.6	-	Specific gravity, Limestone
ρ_{stone} =	162.2	lb/ft^3	Density of armorstone
ρ_{water} =	62.4	lb/ft^3	Density of fresh water
cot θ =	1.5	-	Breakwater Slope

Armor Layer Breakwater Design, Hudson 1974, SPM 1984

K_d =	2	-	Stability coefficient, Rough anglular stone, 60% water depth breaking criteria
$H_{1/10}$ =	10.54	ft	
M_{50} =	15464	lbs	Medium mass of rocks
	7.7	tons	
D_{n50} =	4.6	ft	Equivalent cube length of median rock

$$M_{50} = \frac{\rho_s H^3}{K_D \left(\frac{\rho_s}{\rho_w} - 1\right)^3 \cot\alpha} \qquad (VI-5-67)$$

K_D*-values by SPM 1984*, $H = H_{1/10}$.

Stone shape	Placement	Damage, D^4 = 0-5%	
		Breaking waves ^{1}	Nonbreaking waves ^{2}
Smooth rounded	Random	1.2	2.4
Rough angular	Random	2.0	4.0
Rough angular	Special ^{3}	5.8	7.0

*EM 1110-2-1100 Coastal Engineering Manual (USACE, 2002)

Armor Layer Gradation

M_{max} =	30.9	tons	D_{max} =	7.3	ft
M_{85} =	9.7	tons	D_{85} =	4.9	ft
M_{50} =	7.7	tons	D_{50} =	4.6	ft
M_{15} =	5.8	tons	D_{15} =	4.2	ft
M_{min} =	1.0	tons	D_{min} =	2.3	ft

Recommendation: 6 - 9.5 tons / 9 ft thickness

Filter Layer Gradation

M_{50} =	1546	lbs	*10% of armor mass	D_{50} =	2.1	ft

Recommendation: 4 ft thickness

Geotextile

Recommendation: No Geotextile necessary for offshore breakwaters

Toe Protection

Assumption:

Toe structures in offshore breakwaters are unlikely to be exposed to wave action. To be refined in final engineering, filter stone should be used as a minimum as toe protection stone.

Recommendation: 1150 - 1925 lbs

Offshore Breakwater 5

Wave Overtopping & Armor Calculation
Based on Method in EM1100-2-1100 Coastal Engineering Manual, USACE, Change 3, 2011

Input Data

WL_{Design} =	581	ft	Design Water Level, IGLD 1985	95% WL
H_s =	8.2	ft	Significant wave height	offshore, 100yr storm
T_p =	11	s	Peak wave period	
WL_{Toe} =	563	ft	Deepest Toe Location	
d =	18.00	ft	Depth of water at structure toe, shale lake bottom	
L_p =	257	ft	Local wave length, based on peak period	
cot θ =	1.5	-	Revetment Slope	

Overtopping Estimation

Alternate Crest Elevation = 585.0 ft

Q' =	0.0171	-	Dimensionless overtopping	
F' =	0.155	-	Dimensionless freeboard	
Q =	2.276	cfs/ft	Wave overtopping	
	2.153	cfs/ft	Acceptable Value, rear side no damage	200 l/s per m

$$Q' = C_0 e^{C_1 F'} e^{C_2 m} \qquad (2\text{-}9)$$

$$F' = \frac{F}{\left(H_{mo}^2 L_o\right)^{1/3}} \qquad (2\text{-}11)$$

$$Q' = \frac{Q}{\left(g H_{mo}^3\right)^{1/2}} \qquad (2\text{-}10)$$

*EM 1100-2-1614 Design of Coastal Revetments Seawalls and Bulkheads (USACE 1995)

Breakwater Design
Based on Method in EM 1110-2-1100 Coastal Engineering Manual, USACE, Change 3, 2011

Input Data

S_r =	2.6	-	Specific gravity, Limestone
ρ_{stone} =	162.2	lb/ft^3	Density of armorstone
ρ_{water} =	62.4	lb/ft^3	Density of fresh water
cot θ =	1.5	-	Breakwater Slope

Armor Layer Breakwater Design, Hudson 1974, SPM 1984

K_d =	4	-	Stability coefficient, Rough anglular stone, 60% water depth breaking criteria
$H_{1/10}$ =	10.41	ft	
M_{50} =	7456	lbs	Medium mass of rocks
	3.7	tons	
D_{n50} =	3.6	ft	Equivalent cube length of median rock

$$M_{50} = \frac{\rho_s H^3}{K_D \left(\frac{\rho_s}{\rho_w} - 1\right)^3 \cot\alpha} \qquad (VI-5-67)$$

K_D*-values by SPM 1984,* $H = H_{1/10}$.

Stone shape	Placement	Damage, D^4 = 0-5%	
		Breaking waves ^{1}	Nonbreaking waves ^{2}
Smooth rounded	Random	1.2	2.4
Rough angular	Random	2.0	4.0
Rough angular	Special ^{3}	5.8	7.0

*EM 1110-2-1100 Coastal Engineering Manual (USACE, 2002)

Armor Layer Gradation

M_{max} =	14.9	tons	D_{max} =	5.7	ft
M_{85} =	4.7	tons	D_{85} =	3.9	ft
M_{50} =	3.7	tons	D_{50} =	3.6	ft
M_{15} =	2.8	tons	D_{15} =	3.3	ft
M_{min} =	0.5	tons	D_{min} =	1.8	ft

Recommendation: 3 - 4.5 tons / 7 ft thickness

Filter Layer Gradation

M_{50} = 746 lbs *10% of armor mass — D_{50} = 1.7 ft

Recommendation: 3.5 ft thickness

Geotextile

Recommendation: No Geotextile necessary for offshore breakwaters

Toe Protection

Assumption:

Toe structures in offshore breakwaters are unlikely to be exposed to wave action. To be refined in final engineering, filter stone should be used as a minimum as toe protection stone.

Recommendation: 550 - 925 lbs

Nearshore Breakwater 6

Wave Overtopping & Armor Calculation
Based on Method in EM1100-2-1100 Coastal Engineering Manual, USACE, Change 3, 2011

Input Data

WL_{Design} =	581	ft	Design Water Level, IGLD 1985	95% WL
H_s =	5.3	ft	Significant wave height	offshore, 100yr storm
T_p =	11	s	Peak wave period	
WL_{Toe} =	572	ft	Deepest Toe Location	
d =	9.00	ft	Depth of water at structure toe, shale lake bottom	
L_p =	184	ft	Local wave length, based on peak period	
cot θ =	1.5	-	Breakwater Slope	

Overtopping Estimation

Alternate Crest Elevation = 587.0 ft

Q' =	0.0001	-	Dimensionless overtopping	
F' =	0.347	-	Dimensionless freeboard	
Q =	0.004	cfs/ft	Wave overtopping	
	0.054	cfs/ft	Acceptable Value, rear side no damage	5 l/s per m

$$Q' = C_0 e^{C_1 F'} e^{C_2 m} \qquad (2\text{-}9)$$

$$F' = \frac{F}{\left(H_{mo}^2 L_o\right)^{1/3}} \qquad (2\text{-}11)$$

$$Q' = \frac{Q}{\left(g H_{mo}^3\right)^{1/2}} \qquad (2\text{-}10)$$

*EM 1100-2-1614 Design of Coastal Revetments Seawalls and Bulkheads (USACE 1995)

Breakwater Design
Based on Method in EM 1110-2-1100 Coastal Engineering Manual, USACE, Change 3, 2011

Input Data

S_r =	2.6	-	Specific gravity, Limestone
ρ_{stone} =	165.0	lb/ft^3	Density of armorstone
ρ_{water} =	62.4	lb/ft^3	Density of fresh water
cot θ =	1.5	-	Revetment Slope

Armor Layer Breakwater Design, Hudson 1974, SPM 1984

K_d =	2	-	Stability coefficient, Rough anglular stone, 60% water depth breaking criteria
$H_{1/10}$ =	6.731	ft	
M_{50} =	4095	lbs	Medium mass of rocks
	2	tons	
D_{n50} =	2.9	ft	Equivalent cube length of median rock

$$M_{50} = \frac{\rho_s H^3}{K_D \left(\frac{\rho_s}{\rho_w} - 1\right)^3 \cot\alpha} \qquad (VI-5-67)$$

K_D-values by SPM 1984, $H = H_{1/10}$.

Stone shape	Placement	Damage, D [4] = 0-5%	
		Breaking waves [1]	Nonbreaking waves [2]
Smooth rounded	Random	1.2	2.4
Rough angular	Random	2.0	4.0
Rough angular	Special [3]	5.8	7.0

*EM 1110-2-1100 Coastal Engineering Manual (USACE, 2002)

Armor Layer Gradation

M_{max} =	8.2	tons	D_{max} =	4.6	ft
M_{85} =	2.6	tons	D_{85} =	3.1	ft
M_{50} =	2.0	tons	D_{50} =	2.9	ft
M_{15} =	1.5	tons	D_{15} =	2.7	ft
M_{min} =	0.3	tons	D_{min} =	1.5	ft

Recommendation: 1.5 - 2.5 tons / 6 ft thickness

Filter Layer Gradation

M_{50} = 409 lbs *10% of armor mass D_{50} = 1.4 ft

Recommendation: 2.5 ft thickness

Geotextile

Recommendation: Due to fine-grained composition of the beach behind, a geotextile is recommended

Toe Protection

Assumption: Toe structure may be exposed to wave action. Therefore Hudson equation (VI-5-67, CEM) should be used to size stone per EM 1110-2-1614 direction. All variables remaining equal, armor stone should be used as toe protection stone.

Recommendation: 1.5 - 2.5 tons

Offshore Breakwater 7

Wave Overtopping & Armor Calculation
Based on Method in EM1100-2-1100 Coastal Engineering Manual, USACE, Change 3, 2011

Input Data

WL_{Design} =	581	ft	Design Water Level, IGLD 1985	95% WL
H_s =	8.3	ft	Significant wave height	offshore, 100yr storm
T_p =	11	s	Peak wave period	
WL_{Toe} =	563	ft	Deepest Toe Location	
d =	18.00	ft	Depth of water at structure toe, shale lake bottom	
L_p =	257	ft	Local wave length, based on peak period	
cot θ =	1.5	-	Revetment Slope	

Overtopping Estimation

Alternate Crest Elevation = 585.0 ft

Q' =	0.0177	-	Dimensionless overtopping	
F' =	0.154	-	Dimensionless freeboard	
Q =	2.404	cfs/ft	Wave overtopping	
	2.153	cfs/ft	Acceptable Value, rear side no damage	200 l/s per m

$$Q' = C_0 e^{C_1 F'} e^{C_2 m} \qquad (2\text{-}9)$$

$$F' = \frac{F}{\left(H_{mo}^2 L_o\right)^{1/3}} \qquad (2\text{-}11)$$

$$Q' = \frac{Q}{\left(g H_{mo}^3\right)^{1/2}} \qquad (2\text{-}10)$$

*EM 1100-2-1614 Design of Coastal Revetments Seawalls and Bulkheads (USACE 1995)

Breakwater Design
Based on Method in EM 1110-2-1100 Coastal Engineering Manual, USACE, Change 3, 2011

Input Data

S_r =	2.6	-	Specific gravity, Limestone
ρ_{stone} =	162.2	lb/ft^3	Density of armorstone
ρ_{water} =	62.4	lb/ft^3	Density of fresh water
cot θ =	1.5	-	Breakwater Slope

Armor Layer Breakwater Design, Hudson 1974, SPM 1984

K_d =	4	-	Stability coefficient, Rough anglular stone, 60% water depth breaking criteria
$H_{1/10}$ =	10.54	ft	
M_{50} =	7732	lbs	Medium mass of rocks
	3.9	tons	
D_{n50} =	3.6	ft	Equivalent cube length of median rock

$$M_{50} = \frac{\rho_s H^3}{K_D \left(\frac{\rho_s}{\rho_w} - 1\right)^3 \cot \alpha} \qquad (\mathrm{VI}-5-67)$$

K_D*-values by SPM 1984,* $H = H_{1/10}$.

Stone shape	Placement	Damage, D^4 = 0-5%	
		Breaking waves ^{1}	Nonbreaking waves ^{2}
Smooth rounded	Random	1.2	2.4
Rough angular	Random	2.0	4.0
Rough angular	Special ^{3}	5.8	7.0

*EM 1110-2-1100 Coastal Engineering Manual (USACE, 2002)

Armor Layer Gradation

M_{max} =	15.5	tons	D_{max} =	5.8	ft
M_{85} =	4.8	tons	D_{85} =	3.9	ft
M_{50} =	3.9	tons	D_{50} =	3.6	ft
M_{15} =	2.9	tons	D_{15} =	3.3	ft
M_{min} =	0.5	tons	D_{min} =	1.8	ft

Recommendation: 3 - 5 tons / 7.5 ft thickness

Filter Layer Gradation

M_{50} = 773 lbs *10% of armor mass D_{50} = 1.7 ft

Recommendation: 3.5 ft thickness

Geotextile

Recommendation: No Geotextile necessary for offshore breakwaters

Toe Protection

Assumption:

Toe structures in offshore breakwaters are unlikely to be exposed to wave action. To be refined in final engineering, filter stone should be used as a minimum as toe protection stone.

Recommendation: 575 - 975 lbs

Offshore Breakwater 8

Wave Overtopping & Armor Calculation
Based on Method in EM1100-2-1100 Coastal Engineering Manual, USACE, Change 3, 2011

Input Data

WL_{Design} =	581	ft	Design Water Level, IGLD 1985	95% WL
H_s =	8.3	ft	Significant wave height	offshore, 100yr storm
T_p =	11	s	Peak wave period	
WL_{Toe} =	564	ft	Deepest Toe Location	
d =	17.00	ft	Depth of water at structure toe, shale lake bottom	
L_p =	250	ft	Local wave length, based on peak period	
cot θ =	1.5	-	Revetment Slope	

Overtopping Estimation

Alternate Crest Elevation = 585.0 ft

Q' =	0.0170	-	Dimensionless overtopping	
F' =	0.155	-	Dimensionless freeboard	
Q =	2.308	cfs/ft	Wave overtopping	
	2.153	cfs/ft	Acceptable Value, rear side no damage	200 l/s per m

$$Q' = C_0 e^{C_1 F'} e^{C_2 m} \qquad (2\text{-}9)$$

$$F' = \frac{F}{\left(H_{mo}^2 L_o\right)^{1/3}} \qquad (2\text{-}11)$$

$$Q' = \frac{Q}{\left(g H_{mo}^3\right)^{1/2}} \qquad (2\text{-}10)$$

*EM 1100-2-1614 Design of Coastal Revetments Seawalls and Bulkheads (USACE 1995)

Breakwater Design
Based on Method in EM 1110-2-1100 Coastal Engineering Manual, USACE, Change 3, 2011

Input Data

S_r =	2.6	-	Specific gravity, Limestone
ρ_{stone} =	162.2	lb/ft^3	Density of armorstone
ρ_{water} =	62.4	lb/ft^3	Density of fresh water
cot θ =	1.5	-	Breakwater Slope

Armor Layer Breakwater Design, Hudson 1974, SPM 1984

K_d =	2	-	Stability coefficient, Rough anglular stone, 60% water depth breaking criteria
$H_{1/10}$ =	10.54	ft	
M_{50} =	15464	lbs	Medium mass of rocks
	7.7	tons	
D_{n50} =	4.6	ft	Equivalent cube length of median rock

$$M_{50} = \frac{\rho_s H^3}{K_D \left(\frac{\rho_s}{\rho_w} - 1\right)^3 \cot\alpha} \qquad (VI-5-67)$$

K_D-values by SPM 1984, $H = H_{1/10}$.

Stone shape	Placement	Damage, D^4 = 0-5%	
		Breaking waves [1]	Nonbreaking waves [2]
Smooth rounded	Random	1.2	2.4
Rough angular	Random	2.0	4.0
Rough angular	Special [3]	5.8	7.0

*EM 1110-2-1100 Coastal Engineering Manual (USACE, 2002)

Armor Layer Gradation

M_{max} =	30.9	tons	D_{max} =	7.3	ft
M_{85} =	9.7	tons	D_{85} =	4.9	ft
M_{50} =	7.7	tons	D_{50} =	4.6	ft
M_{15} =	5.8	tons	D_{15} =	4.2	ft
M_{min} =	1.0	tons	D_{min} =	2.3	ft

Recommendation: 6 - 9.5 tons / 9 ft thickness

Filter Layer Gradation

M_{50} =	1546	lbs	*10% of armor mass	D_{50} =	2.1	ft

Recommendation: 4 ft thickness

Geotextile

Recommendation: No Geotextile necessary for offshore breakwaters

Toe Protection

Assumption:

Toe structures in offshore breakwaters are unlikely to be exposed to wave action. To be refined in final engineering, filter stone should be used as a minimum as toe protection stone.

Recommendation: 1150 - 1925 lbs

Nearshore Breakwater 9

Wave Overtopping & Armor Calculation
Based on Method in EM1100-2-1100 Coastal Engineering Manual, USACE, Change 3, 2011

Input Data

WL_{Design} =	581	ft	Design Water Level, IGLD 1985	95% WL
H_s =	5.6	ft	Significant wave height	offshore, 100yr storm
T_p =	11	s	Peak wave period	
WL_{Toe} =	571	ft	Deepest Toe Location	
d =	10.00	ft	Depth of water at structure toe, shale lake bottom	
L_p =	194	ft	Local wave length, based on peak period	
cot θ =	1.5	-	Breakwater Slope	

Overtopping Estimation

Alternate Crest Elevation = 587.0 ft

Q' =	0.0001	-	Dimensionless overtopping	
F' =	0.329	-	Dimensionless freeboard	
Q =	0.008	cfs/ft	Wave overtopping	
	0.054	cfs/ft	Acceptable Value, rear side no damage	5 l/s per m

$$Q' = C_0 e^{C_1 F'} e^{C_2 m} \qquad (2\text{-}9)$$

$$F' = \frac{F}{\left(H_{mo}^2 L_o\right)^{1/3}} \qquad (2\text{-}11)$$

$$Q' = \frac{Q}{\left(g H_{mo}^3\right)^{1/2}} \qquad (2\text{-}10)$$

*EM 1100-2-1614 Design of Coastal Revetments Seawalls and Bulkheads (USACE 1995)

Breakwater Design
Based on Method in EM 1110-2-1100 Coastal Engineering Manual, USACE, Change 3, 2011

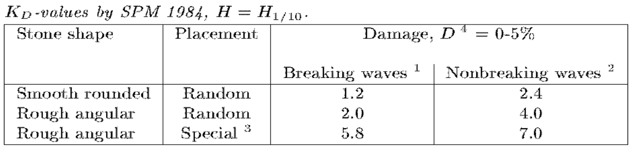
Input Data

S_r =	2.6	-	Specific gravity, Limestone
ρ_{stone} =	165.0	lb/ft^3	Density of armorstone
ρ_{water} =	62.4	lb/ft^3	Density of fresh water
cot θ =	1.5	-	Revetment Slope

Armor Layer Breakwater Design, Hudson 1974, SPM 1984

K_d =	2	-	Stability coefficient, Rough anglular stone, 60% water depth breaking criteria
$H_{1/10}$ =	7.112	ft	
M_{50} =	4830	lbs	Medium mass of rocks
	2	tons	
D_{n50} =	3.1	ft	Equivalent cube length of median rock

$$M_{50} = \frac{\rho_s H^3}{K_D \left(\frac{\rho_s}{\rho_w} - 1\right)^3 \cot\alpha} \qquad (VI-5-67)$$



K_D-values by SPM 1984, $H = H_{1/10}$.

Stone shape	Placement	Damage, D^4 = 0-5%	
		Breaking waves [1]	Nonbreaking waves [2]
Smooth rounded	Random	1.2	2.4
Rough angular	Random	2.0	4.0
Rough angular	Special [3]	5.8	7.0

*EM 1110-2-1100 Coastal Engineering Manual (USACE, 2002)

Armor Layer Gradation

M_{max} =	9.7	tons	D_{max} =	4.9	ft
M_{85} =	3.0	tons	D_{85} =	3.3	ft
M_{50} =	2.4	tons	D_{50} =	3.1	ft
M_{15} =	1.8	tons	D_{15} =	2.8	ft
M_{min} =	0.3	tons	D_{min} =	1.5	ft

Recommendation: 2 - 3 tons / 6 ft thickness

Filter Layer Gradation

M_{50} =	483	lbs	*10% of armor mass	D_{50} =	1.4	ft

Recommendation: 3 ft thickness

Geotextile

Recommendation: Due to fine-grained composition of the beach behind, a geotextile is recommended

Toe Protection

Assumption: Toe structure may be exposed to wave action. Therefore Hudson equation (VI-5-67, CEM) should be used to size stone per EM 1110-2-1614 direction. All variables remaining equal, armor stone should be used as toe protection stone.

Recommendation: 2 - 3 tons

Nearshore Breakwater 10

Wave Overtopping & Armor Calculation
Based on Method in EM1100-2-1100 Coastal Engineering Manual, USACE, Change 3, 2011

Input Data

WL_{Design} =	581	ft	Design Water Level, IGLD 1985	95% WL
H_s =	5.7	ft	Significant wave height	offshore, 100yr storm
T_p =	11	s	Peak wave period	
WL_{Toe} =	570	ft	Deepest Toe Location	
d =	11.00	ft	Depth of water at structure toe, shale lake bottom	
L_p =	203	ft	Local wave length, based on peak period	
cot θ =	1.5	-	Breakwater Slope	

Overtopping Estimation

Alternate Crest Elevation = 587.5 ft

Q' =	0.0001	-	Dimensionless overtopping	
F' =	0.347	-	Dimensionless freeboard	
Q =	0.005	cfs/ft	Wave overtopping	
	0.054	cfs/ft	Acceptable Value, rear side no damage	5 l/s per m

$$Q' = C_0 e^{C_1 F'} e^{C_2 m} \qquad (2\text{-}9)$$

$$F' = \frac{F}{\left(H_{mo}^2 L_o\right)^{1/3}} \qquad (2\text{-}11)$$

$$Q' = \frac{Q}{\left(g H_{mo}^3\right)^{1/2}} \qquad (2\text{-}10)$$

*EM 1100-2-1614 Design of Coastal Revetments Seawalls and Bulkheads (USACE 1995)

Breakwater Design
Based on Method in EM 1110-2-1100 Coastal Engineering Manual, USACE, Change 3, 2011

Input Data

S_r =	2.6	-	Specific gravity, Limestone
ρ_{stone} =	165.0	lb/ft^3	Density of armorstone
ρ_{water} =	62.4	lb/ft^3	Density of fresh water
cot θ =	1.5	-	Revetment Slope

Armor Layer Breakwater Design, Hudson 1974, SPM 1984

K_d =	2	-	Stability coefficient, Rough anglular stone, 60% water depth breaking criteria
$H_{1/10}$ =	7.239	ft	
M_{50} =	5094	lbs	Medium mass of rocks
	3	tons	
D_{n50} =	3.1	ft	Equivalent cube length of median rock

$$M_{50} = \frac{\rho_s H^3}{K_D \left(\frac{\rho_s}{\rho_w} - 1\right)^3 \cot\alpha} \qquad (VI-5-67)$$

K_D*-values by SPM 1984,* $H = H_{1/10}$.

Stone shape	Placement	Damage, D^4 = 0-5%	
		Breaking waves ^{1}	Nonbreaking waves ^{2}
Smooth rounded	Random	1.2	2.4
Rough angular	Random	2.0	4.0
Rough angular	Special ^{3}	5.8	7.0

*EM 1110-2-1100 Coastal Engineering Manual (USACE, 2002)

Armor Layer Gradation

M_{max} =	10.2	tons	D_{max} =	5.0	ft
M_{85} =	3.2	tons	D_{85} =	3.4	ft
M_{50} =	2.5	tons	D_{50} =	3.1	ft
M_{15} =	1.9	tons	D_{15} =	2.9	ft
M_{min} =	0.3	tons	D_{min} =	1.6	ft

Recommendation: 2 - 3 tons / 6.5 ft thickness

Filter Layer Gradation

M_{50} =	509	lbs	*10% of armor mass	D_{50} =	1.5	ft

Recommendation: 3 ft thickness

Geotextile

Recommendation: Due to fine-grained composition of the beach behind, a geotextile is recommended

Toe Protection

Assumption: Toe structure may be exposed to wave action. Therefore Hudson equation (VI-5-67, CEM) should be used to size stone per EM 1110-2-1614 direction. All variables remaining equal, armor stone should be used as toe protection stone.

Recommendation: 2 - 3 tons

Submerged Breakwater 11

Wave Armor Calculation
Based on Method in EM1100-2-1100 Coastal Engineering Manual, USACE, Change 3, 2011

Input Data

WL_{Design} =	577	ft	Low Water Level	
H_s =	7	ft	Significant wave height	offshore, 100yr storm
T_p =	11	s	Peak wave period	
WL_{Toe} =	567	ft	Deepest Toe Location	
d =	10.00	ft	Depth of water at structure toe, shale lake bottom	
L_p =	194	ft	Local wave length, based on peak period	
cot θ =	1.5	-	Revetment Slope	

Overtopping Estimation

Alternate Crest Elevation = 579.0 ft

Q' =	0.1011	-	Dimensionless overtopping	
F' =	0.094	-	Dimensionless freeboard	
Q =	10.620	cfs/ft	Wave overtopping	
	2.153	cfs/ft	Acceptable Value, rear side no damage	200 l/s per m

$$Q' = C_0 e^{C_1 F'} e^{C_2 m} \qquad (2\text{-}9)$$

$$F' = \frac{F}{\left(H_{mo}^2 L_o\right)^{1/3}} \qquad (2\text{-}11)$$

$$Q' = \frac{Q}{\left(g H_{mo}^3\right)^{1/2}} \qquad (2\text{-}10)$$

*EM 1100-2-1614 Design of Coastal Revetments Seawalls and Bulkheads (USACE 1995)

Breakwater Design
Based on Method in EM 1110-2-1100 Coastal Engineering Manual, USACE, Change 3, 2011

Input Data

S_r =	2.6	-	Specific gravity, Limestone
ρ_{stone} =	162.2	lb/ft^3	Density of armorstone
ρ_{water} =	62.4	lb/ft^3	Density of fresh water
cot θ =	1.5	-	Breakwater Slope

Armor Layer Breakwater Design, Hudson 1974, SPM 1984

K_d =	2	-	Stability coefficient, Rough anglular stone, 60% water depth breaking criteria
$H_{1/10}$ =	8.89	ft	
M_{50} =	9276	lbs	Medium mass of rocks
	4.6	tons	
D_{n50} =	3.9	ft	Equivalent cube length of median rock

$$M_{50} = \frac{\rho_s H^3}{K_D \left(\frac{\rho_s}{\rho_w} - 1\right)^3 \cot\alpha} \qquad (VI-5-67)$$

K_D-values by SPM 1984, $H = H_{1/10}$.

Stone shape	Placement	Damage, D^4 = 0-5%	
		Breaking waves [1]	Nonbreaking waves [2]
Smooth rounded	Random	1.2	2.4
Rough angular	Random	2.0	4.0
Rough angular	Special [3]	5.8	7.0

*EM 1110-2-1100 Coastal Engineering Manual (USACE, 2002)

Armor Layer Gradation

M_{max} =	18.6	tons	D_{max} =	6.1	ft
M_{85} =	5.8	tons	D_{85} =	4.1	ft
M_{50} =	4.6	tons	D_{50} =	3.9	ft
M_{15} =	3.5	tons	D_{15} =	3.5	ft
M_{min} =	0.6	tons	D_{min} =	1.9	ft

Recommendation: 3.5 - 6 tons / 7.5 ft thickness

Filter Layer Gradation

M_{50} = 928 lbs *10% of armor mass D_{50} = 1.8 ft

Recommendation: 3.5 ft thickness

Geotextile

Recommendation: No Geotextile necessary for offshore breakwaters

Toe Protection

Assumption:

Toe structures in offshore breakwaters are unlikely to be exposed to wave action. To be refined in final engineering, filter stone should be used as a minimum as toe protection stone.

Recommendation: 700 - 1150 lbs

Submerged Breakwater 12

Wave Armor Calculation
Based on Method in EM1100-2-1100 Coastal Engineering Manual, USACE, Change 3, 2011

Input Data

WL_{Design} =	577	ft	Low Water Level	
H_s =	7	ft	Significant wave height	offshore, 100yr storm
T_p =	11	s	Peak wave period	
WL_{Toe} =	567	ft	Deepest Toe Location	
d =	10.00	ft	Depth of water at structure toe, shale lake bottom	
L_p =	194	ft	Local wave length, based on peak period	
cot θ =	1.5	-	Revetment Slope	

Overtopping Estimation

Alternate Crest Elevation = 579.0 ft

Q' =	0.1011	-	Dimensionless overtopping	
F' =	0.094	-	Dimensionless freeboard	
Q =	10.620	cfs/ft	Wave overtopping	
	2.153	cfs/ft	Acceptable Value, rear side no damage	200 l/s per m

$$Q' = C_0 \, e^{C_1 F'} \, e^{C_2 m} \qquad (2\text{-}9)$$

$$F' = \frac{F}{\left(H_{mo}^2 L_o\right)^{1/3}} \qquad (2\text{-}11)$$

$$Q' = \frac{Q}{\left(g H_{mo}^3\right)^{1/2}} \qquad (2\text{-}10)$$

*EM 1100-2-1614 Design of Coastal Revetments Seawalls and Bulkheads (USACE 1995)

Breakwater Design
Based on Method in EM 1110-2-1100 Coastal Engineering Manual, USACE, Change 3, 2011

Input Data

S_r =	2.6	-	Specific gravity, Limestone
ρ_{stone} =	162.2	lb/ft^3	Density of armorstone
ρ_{water} =	62.4	lb/ft^3	Density of fresh water
cot θ =	1.5	-	Breakwater Slope

Armor Layer Breakwater Design, Hudson 1974, SPM 1984

K_d =	2	-	Stability coefficient, Rough anglular stone, 60% water depth breaking criteria
$H_{1/10}$ =	8.89	ft	
M_{50} =	9276	lbs	Medium mass of rocks
	4.6	tons	
D_{n50} =	3.9	ft	Equivalent cube length of median rock

$$M_{50} = \frac{\rho_s H^3}{K_D \left(\frac{\rho_s}{\rho_w} - 1\right)^3 \cot\alpha} \qquad (VI-5-67)$$

K_D*-values by SPM 1984,* $H = H_{1/10}$.

Stone shape	Placement	Damage, D^4 = 0-5%	
		Breaking waves [1]	Nonbreaking waves [2]
Smooth rounded	Random	1.2	2.4
Rough angular	Random	2.0	4.0
Rough angular	Special [3]	5.8	7.0

*EM 1110-2-1100 Coastal Engineering Manual (USACE, 2002)

Armor Layer Gradation

M_{max} =	18.6	tons	D_{max} =	6.1	ft
M_{85} =	5.8	tons	D_{85} =	4.1	ft
M_{50} =	4.6	tons	D_{50} =	3.9	ft
M_{15} =	3.5	tons	D_{15} =	3.5	ft
M_{min} =	0.6	tons	D_{min} =	1.9	ft

Recommendation: 3.5 - 6 tons / 7.5 ft thickness

Filter Layer Gradation

M_{50} =	928	lbs	*10% of armor mass	D_{50} =	1.8	ft

Recommendation: 3.5 ft thickness

Geotextile

Recommendation: No Geotextile necessary for offshore breakwaters

Toe Protection

Assumption:

Toe structures in offshore breakwaters are unlikely to be exposed to wave action. To be refined in final engineering, filter stone should be used as a minimum as toe protection stone.

Recommendation: 700 - 1150 lbs

Nearshore Breakwater 13

Wave Overtopping & Armor Calculation
Based on Method in EM1100-2-1100 Coastal Engineering Manual, USACE, Change 3, 2011

Input Data

WL_{Design} =	581	ft	Design Water Level, IGLD 1985	95% WL
H_s =	5.6	ft	Significant wave height	offshore, 100yr storm
T_p =	11	s	Peak wave period	
WL_{Toe} =	577	ft	Deepest Toe Location - assumes nearshore erosion of 3'	
d =	4.00	ft	Depth of water at structure toe, shale lake bottom	
L_p =	124	ft	Local wave length, based on peak period	
cot θ =	1.5	-	Breakwater Slope	

Overtopping Estimation

Alternate Crest Elevation = 587.0 ft

Q' =	0.0000	-	Dimensionless overtopping	
F' =	0.382	-	Dimensionless freeboard	
Q =	0.002	cfs/ft	Wave overtopping	
	0.054	cfs/ft	Acceptable Value, rear side no damage	5 l/s per m

$$Q' = C_0 e^{C_1 F'} e^{C_2 m} \qquad (2\text{-}9)$$

$$F' = \frac{F}{\left(H_{mo}^2 L_o\right)^{1/3}} \qquad (2\text{-}11)$$

$$Q' = \frac{Q}{\left(g H_{mo}^3\right)^{1/2}} \qquad (2\text{-}10)$$

*EM 1100-2-1614 Design of Coastal Revetments Seawalls and Bulkheads (USACE 1995)

Breakwater Design
Based on Method in EM 1110-2-1100 Coastal Engineering Manual, USACE, Change 3, 2011

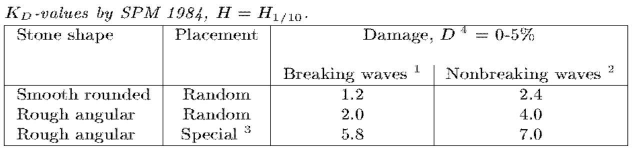
Input Data

S_r =	2.6	-	Specific gravity, Limestone
ρ_{stone} =	165.0	lb/ft^3	Density of armorstone
ρ_{water} =	62.4	lb/ft^3	Density of fresh water
cot θ =	1.5	-	Revetment Slope

Armor Layer Breakwater Design, Hudson 1974, SPM 1984

K_d =	2	-	Stability coefficient, Rough anglular stone, 60% water depth breaking criteria
$H_{1/10}$ =	7.112	ft	
M_{50} =	4830	lbs	Medium mass of rocks
	2	tons	
D_{n50} =	3.1	ft	Equivalent cube length of median rock

$$M_{50} = \frac{\rho_s H^3}{K_D \left(\frac{\rho_s}{\rho_w} - 1\right)^3 \cot\alpha} \qquad (\mathrm{VI}-5-67)$$


K_D-values by SPM 1984, $H = H_{1/10}$.

Stone shape	Placement	Damage, D^4 = 0-5%	
		Breaking waves [1]	Nonbreaking waves [2]
Smooth rounded	Random	1.2	2.4
Rough angular	Random	2.0	4.0
Rough angular	Special [3]	5.8	7.0

*EM 1110-2-1100 Coastal Engineering Manual (USACE, 2002)

Armor Layer Gradation

M_{max} =	9.7	tons	D_{max} =	4.9	ft
M_{85} =	3.0	tons	D_{85} =	3.3	ft
M_{50} =	2.4	tons	D_{50} =	3.1	ft
M_{15} =	1.8	tons	D_{15} =	2.8	ft
M_{min} =	0.3	tons	D_{min} =	1.5	ft

Recommendation: 2 - 3 tons / 6 ft thickness

Filter Layer Gradation

M_{50} = 483 lbs *10% of armor mass D_{50} = 1.4 ft

Recommendation: 3 ft thickness

Geotextile

Recommendation: Due to fine-grained composition of the beach behind, a geotextile is recommended

Toe Protection

Assumption: Toe structure may be exposed to wave action. Therefore Hudson equation (VI-5-67, CEM) should be used to size stone per EM 1110-2-1614 direction. All variables remaining equal, armor stone should be used as toe protection stone.

Recommendation: 2 - 3 tons

APPENDIX M

Kellogg Creek

2. Kellogg Creek

Kellogg Creek, shown in Figure M-1 below, is located at project station 70+00, directly north of Area 2 and the Lake County water intake. The creek transfers stormwater runoff from the surrounding lands into Lake Michigan. IDNR reported that in recent years, the confluence of the creek and the lake is frequently blocked resulting in water backing up in the creek resulting in minor flooding of the surrounding land. Due to the longshore littoral drift, the creek has been blocked by migrating sand but was easily manually opened. Large cobbles are now mixed with the sand making opening the creek by hand more difficult.

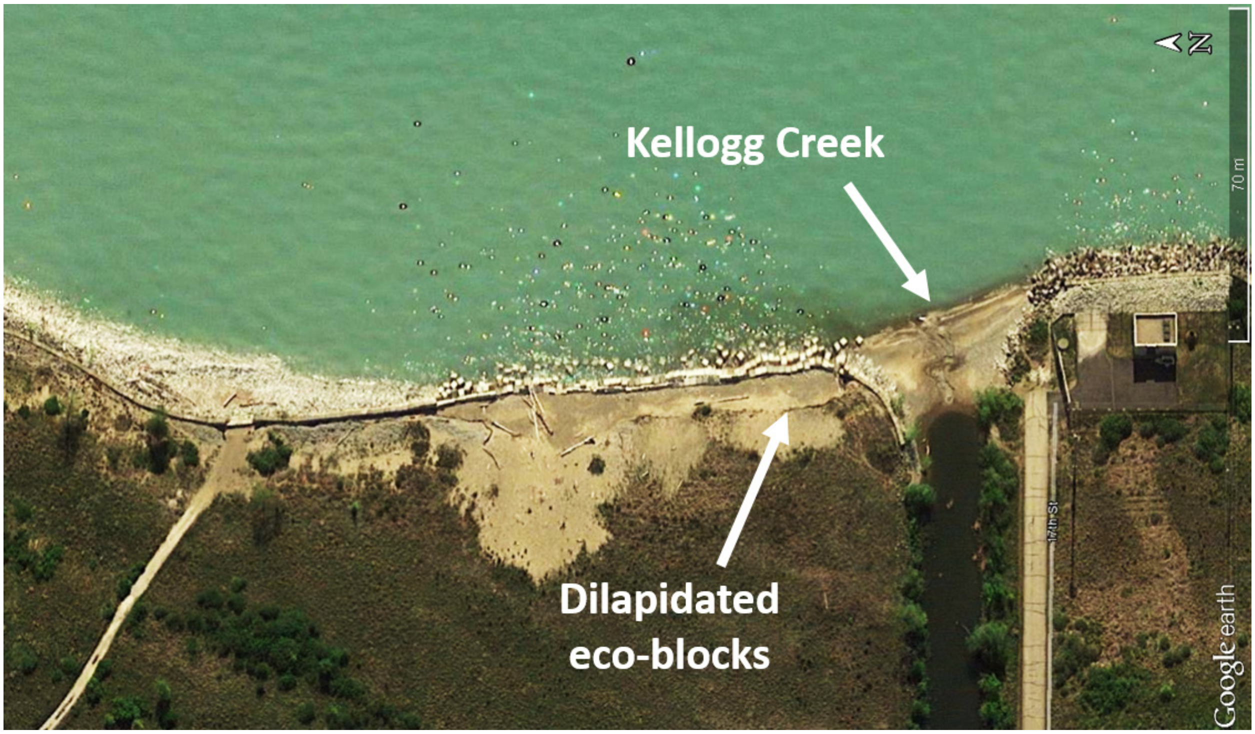


Figure M-1: Kellogg Creek (map data: Google, USDA Farm Service Agency)

The small rocks which litter the area do not appear to be native. Review of historic aerials in GoogleEarth show the cobble in 1999 to be purposely placed alongside the ecoblock revetment. It is unknown when the cobble was first installed. Subsequent aerials show the cobble moving downdrift whenever the water reaches it. The cobble's movement is likely due to large storm events. Once lake level's started to increase after 2013, the cobbles eventually migrated south to Kellogg Creek. The destruction of the ecoblock wall at this location allowed the cobbles to be pushed further onto the shore and into the mouth of the creek, as show in Figure M-2.

Figure M-2: White cobble north of Kellogg Creek

While the preference is to address this area by cleaning out the cobbles and the dilapidated wall and installing offshore structures to hold the shoreline, this area remains mostly stable as the wall is still holding much of the shoreline intact (see Figure 26 showing where erosion has begun). Therefore, to address the issues affecting Kellogg Creek, four updrift structures were explored. A structure will hold sediment and cobble from migrating further south. Cobbles and sand currently in the creek mouth will still need to be cleared but maintenance will be minimal thereafter.

2.1. Groin 1 – Shore Perpendicular

The first option explored was a shore perpendicular groin. A perpendicular groin captures sediment on its updrift side and, due to the starvation of sand immediately in its shadow, erosion occurs along its downdrift side. This would be acceptable as the downdrift side aligns with the mouth of Kellogg Creek and the revetment surrounding the water intake building. Shown in Figure M-3, currents running along the shoreline and around the groin create an eddy in its shadow. This can result in the deposition of sand though it is unlikely the cobble will be able to migrate around the structure. Overtime, sand will fill the groin and start to bypass, increasing the amount of sand deposited at the creek mouth.

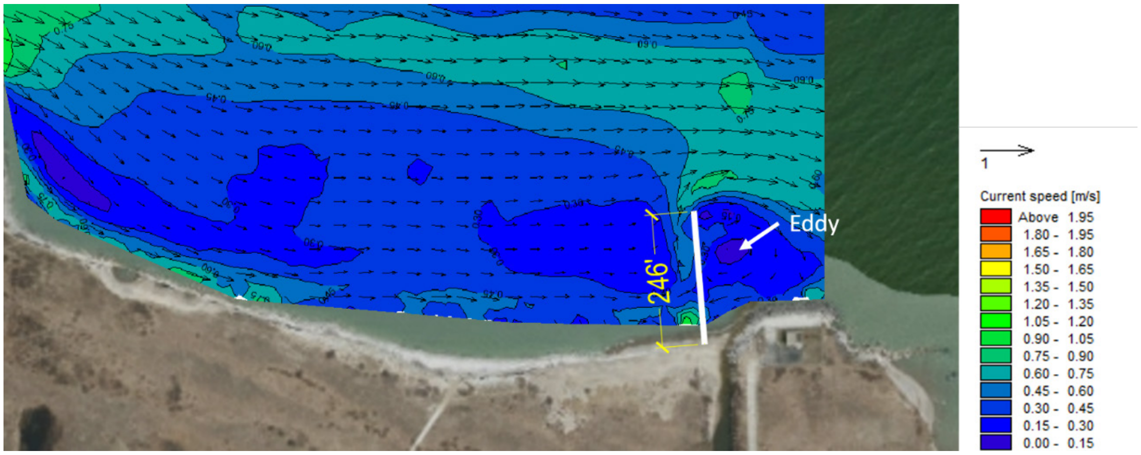


Figure M-3: Shore-perpendicular groin

2.2. Groin 2 – Angled

The second alternative explored included an angled groin projecting to the northeast. The intent is to capture southerly migrating sediment and cobbles. As with the shore-perpendicular option, this will result in some erosion on the downdrift side of the structure.

Shown in Figure M-4, the longshore currents do pass the groin and, although lessened, still create an eddy in front of the creek's mouth. This again will cause sediments to deposit. In addition, the angled groin provides less capacity for updrift sediment entrapment and therefore will result in sediment deposition at the creek mouth more quickly than the shore-perpendicular option.

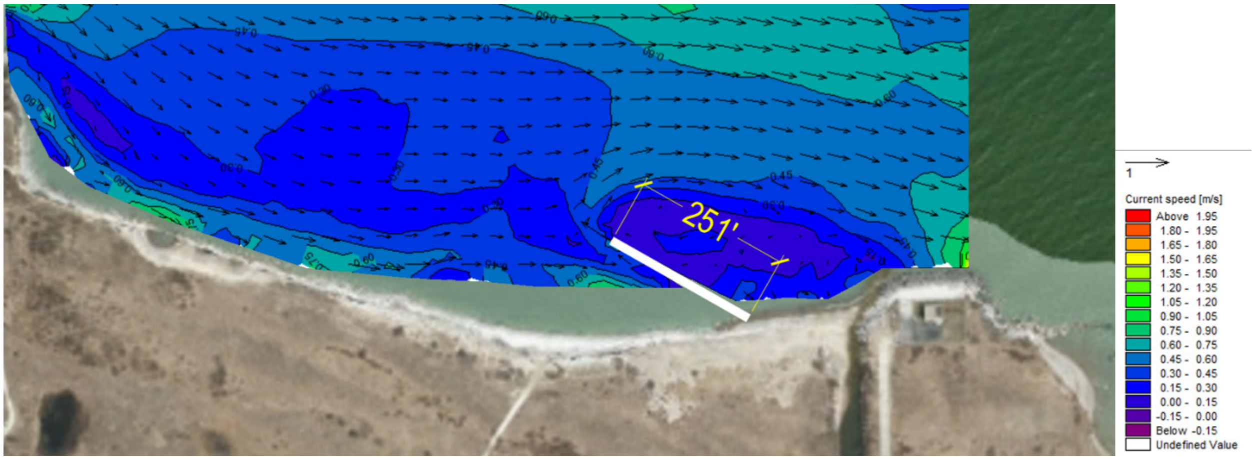


Figure M-4: Angled groin

2.3. Groin 3 – L-Head

The third alternative includes a northeast spur added to the end of the shore-perpendicular alternative. The intent of the spur is to redirect the currents, and the sediment with it, offshore and around the creek mouth. As shown in Figure

M-5, this is precisely how the currents behave. While an eddy forms along the backside of the spur, it is further offshore and therefore settling sediments will not block the creek mouth. This alternative provides both reduced maintenance and a larger holding capacity for migrating sediments.

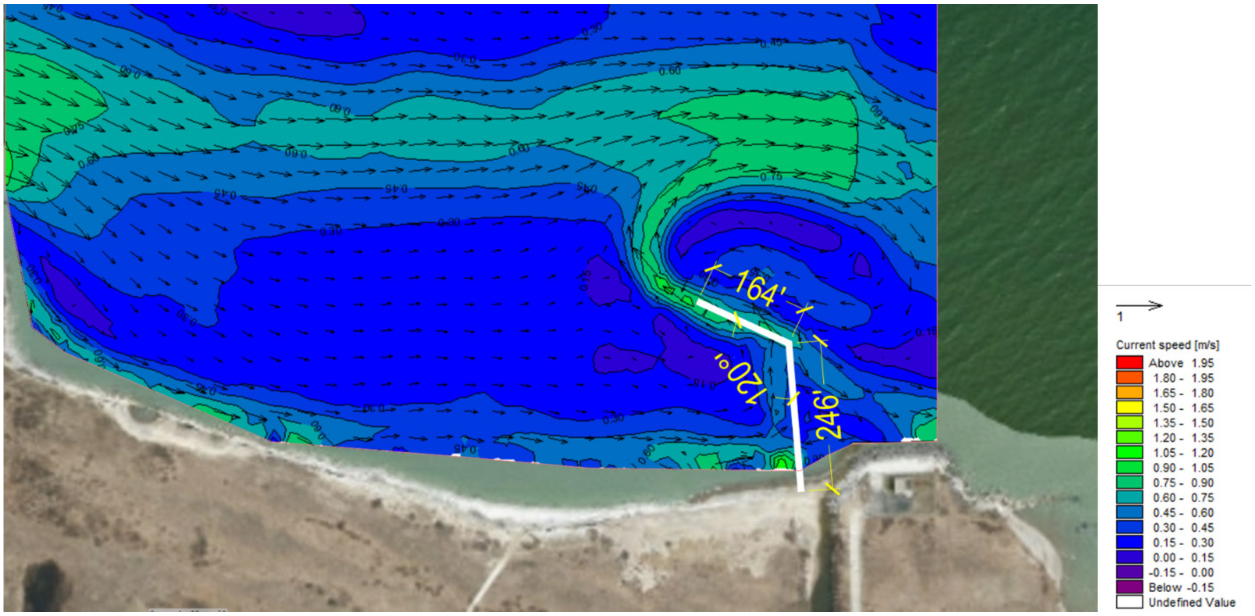


Figure M-5: L-Head Groin

2.4. Groin 4 – Hooked

Recognizing that a spur projecting to the northeast will push into deeper water and therefore have an increasing construction cost, the fourth alternative redirected the spur toward the northwest and into shallower water. Shown in Figure M-6, this alternative does redirect the currents around the structure and at a higher velocity near the creek mouth which will result in a reduction of sediment deposition. While this shape results in less sediment capture it has a higher rate of bypassing which is better for the downdrift shoreline.

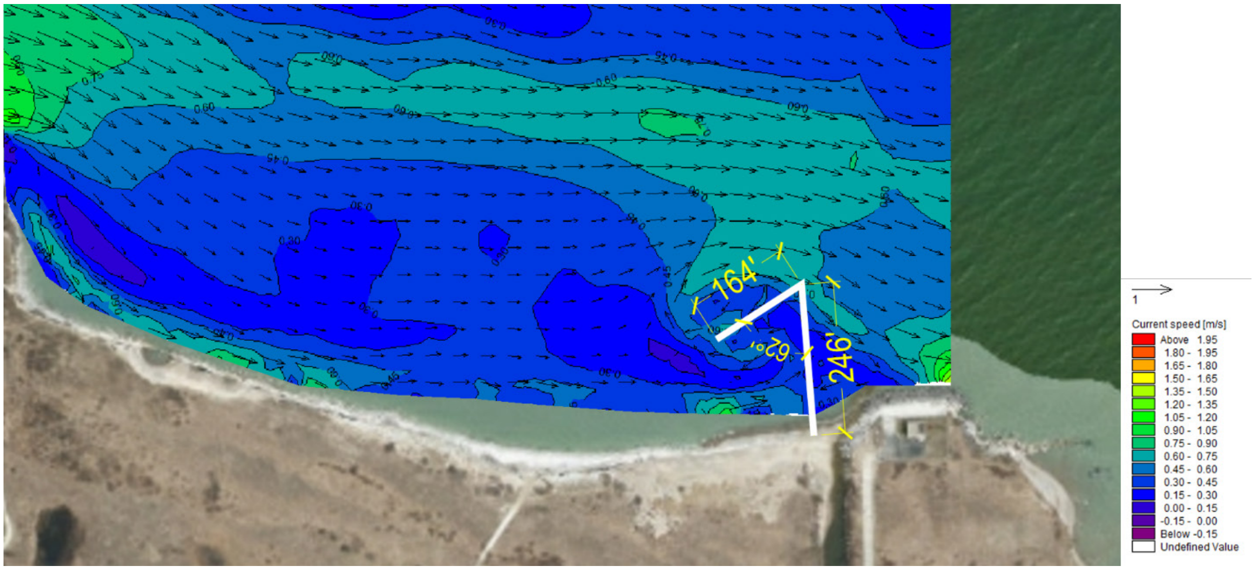


Figure M-6: Hooked Groin

2.5. Recommendation

In lieu of completely removing the cobble and dilapidated ecoblock revetment and replacing with a softer, more offshore approach, it is recommended that the hooked groin (Groin 4) be considered. The shape and dimensions can be further refined through additional modeling.

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