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ALTERNATIVES EVALUATION REPORT

DEER RIVER RESTORATION PROJECT
MOBILE COUNTY, ALABAMA

MAY 28, 2020

DRAFT

Prepared for:

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TABLE OF CONTENTS

LIST OF FIGURES

TABLE OF CONTENTS (Continued)

LIST OF TABLES

APPENDICES

1.0 INTRODUCTION

Purpose and Objectives

The purpose of this Alternatives Evaluation is to assist the Mobile Bay National Estuary Program (MBNEP) in making an informed decision on which design alternative best meets their goals, while taking into account budgetary and other constraints. MBNEP has established the overall goals for the restoration to be: (1) stabilize the eroding shoreline and protect an existing 275-acre tidal marsh along the western shore of Mobile Bay from Theodore Industrial Canal south approximately 5,600 feet and (2) improve water quality in the Middle Fork of Deer River by closing a breach allowing it to flow its full length and empty into the Theodore Industrial Canal, then dredge the silted-in portions of Deer River between the breach and the Canal improving water quality and wildlife habitat.

The proposed site was selected as part of the MBNEP's Western Shore Watershed Management Plan which includes the watersheds of Garrow's Bend, Deer River and Delchamps Bayou.

The goals and objectives of the Western Shore Watershed Management Plan are:

- Improving water quality and habitats necessary to support healthy populations of fish and shellfish;
- Protecting continued customary uses of biological resources to preserve culture, heritage, and ecology of the watershed;
- Mitigating impacts from industrial uses on the watershed, while embracing the economic benefit of resident industries;
- Reducing and mitigating impacts of coastal erosion on the shoreline;
- Improving watershed resiliency to sea level rise and changing climate impacts and
- Expanding opportunities for community access to the natural resources and waters of Mobile Bay.

The MBNEP secured funding from the National Fish and Wildlife Foundation (NFWF) Gulf Environmental Benefit Fund (GEBF) to undertake engineering and design to stabilize and enhance:

1) The 5,600-foot shoreline of the 275-acre Deer River salt marsh tract on the western shore of Mobile Bay directly south of the Theodore Industrial Canal. The shoreline has experienced significant recession from storms, tides, and ship wakes.

2) The network of tidal channels, including the South and Middle Forks of Deer River, which are extremely shallow and impaired by siltation, limiting tidal exchange and circulation necessary to sustain the currently healthy marsh.

A range of options were considered for the accomplishment of both objectives:

Objective 1 - Shoreline stabilization and enhancement: A number of options for breakwater systems to mitigate wave impacts to existing shoreline were analyzed, in addition to options for marsh creation (possibly using dredge materials for wetland fill) shoreline reclamation and stabilization.

Objective 2 – Water-quality improvement and flow/depth restoration in Deer River: Fewer options exist for accomplishing this objective. However, options analyzed included ultimate dredge depth, dredge method, dredge-material disposal/beneficial-use and breach-closure methods.

The objective of the Alternatives Evaluation phase of this project is to develop preliminary budgetary estimates for a range of options; assess the feasibility of those options; and determine to what extent the options will meet MBNEP's aforementioned overall objectives.

Scope

The Thompson Team (Thompson Engineering, ESA, Moffatt & Nichol, and Barry A. Vittor & Associates, Inc.) reviewed relevant information and data pertaining to Mobile Bay, Deer River, and the local area (see Figure 1 – Vicinity Map). A literature review conducted included coastal processes and data, living shorelines data, and US Army Corps of Engineers (USACE) dredging information. To supplement existing information and data available from literature review, field investigations were conducted to acquire site-specific survey information (bathymetric and topographic) and geotechnical (soils, sediments) data.

Conceptual design alternatives were developed and included: alternative breakwater/living shoreline designs and marsh-creation alternatives to include material sourcing and delivery methods.

Figure 1 –Vicinity Map - Source: USGS 7.5-minute Quadrangle, Hollinger's Island (2018)

2.0 GENERAL APPROACH FOR EVALUATION OF ALTERNATIVES

2.1 General Considerations

As noted previously, Deer River project goals are to: (1) stabilize the shoreline along the Mobile Bay western shore from Theodore Industrial Canal 5,600 south to protect the existing 275-acre marsh from further loss due to erosion, and (2) improve water quality in the Middle Fork of Deer River by increasing flow and depth. The evaluation of alternatives to address these goals has included:

- Alternate breakwater alignments (and marsh creation configuration)
- Alternate breakwater / living shoreline designs
- Alternate borrow sources and fill placement methods (for marsh creation fill)

Each is generally discussed in the subsections below. Following in Section 3 is a presentation of specific alternatives evaluated in more detail, including a summary tabulation of budgetary construction cost estimates. More detailed cost itemizations are contained in Appendix A. Preliminary drawings of the alternatives are included in Appendix B. A wave-climate analysis report is included as Appendix C. The complete geotechnical report is contained in Appendix D.

It should be noted, the budgetary cost information summarized in Section 3 and itemized in Appendix A are preliminary opinions of probable construction costs, and do not include related engineering (design, construction oversight) or permitting costs.

Certain alternatives require transport of materials and/or equipment to the site by barges and when optimally loaded, require a typical draft on the order of 6 feet or more. Where needed, the cost estimates include provision for an access channel from the existing Theodore Industrial Canal to the project work site and along the alignment of the proposed breakwater.

2.2 Shoreline Stabilization Methods

Four (4) alternative breakwater/living shoreline concepts were used for detailed cost comparisons during the initial planning, concept design, and feasibility-evaluation phases of the project. Each alternative breakwater system is planned along an alignment which best fits the characteristics of the particular breakwater while maximizing the potential for marsh creation:

- Alternative 1: Continuous Offshore Rock Dike Breakwater
- Alternative 2: Continuous OysterBreak™ Breakwater
- Alternative 3: Continuous Pile-supported Reefmaker™ Breakwater
- Alternative 4: Segmented Offshore Breakwater with Sand Fill (Pocket Beach)

Alternative 1 represents more conventional "rubble mound" breakwater construction offshore from the existing shoreline. Alternatives 2 and 3 represent a type of manufactured "living shoreline" breakwater. Alternative 4 would be similar to a beach re-nourishment with a sand berm being placed just offshore with a segmented rubble breakwater placed on the sand berm allowing pocket beaches to form between the segments.

Wave modeling conducted by Moffatt & Nichol (M&N) for the Deer River Restoration Project includes a regional (scale of Mobile Bay) MIKE21-Spectral Wave model as well as a local (scale of the project site) MIKE21-Boussinesq Wave model. Modeling was conducted to accurately determine the wave climate at the Deer River Restoration project site and the local wave transformation to be used for specific engineering and design components. These models were used to evaluate the effects of potential breakwater design alternatives (namely a low-crested rubble mound breakwater and Reefmaker™ wave-attenuator system) on wave conditions reaching the restored shoreline for various site-specific environmental scenarios (levels of extreme to operational waves).

Analysis of established tidal datums at nearby tidal gages and spatial interpolation were used to compute tidal datums at the project site, where an astronomical diurnal (one high and one low tide per day) tide range of approximately 1.54 feet (Table 1) is predicted. These findings agree with published literature documenting Mobile Bay as a micro-tidal environment.

Tidal Datums	Deer River [ft, NAVD88]
MHHW (Mean Higher High Water)	1.0
MHW (Mean High Water)	0.9
MTL (Mean Tide Level)	0.2
MSL (Mean Sea Level)	0.2
MLW (Mean Low Water)	-0.5
MLLW (Mean Lower Low Water)	-0.6

Table 1 - Interpolated tidal datums at the Deer River project site

Sea-level-rise planning scenarios from Sweet et al. (2017) were used to evaluate relative sea- level rise at the Deer River project site for a 25-year project design life (project life from 2020 to 2045). M&N recommends the Deer River Restoration project consider the use of the intermediate (1.0 meter, 3.28 feet) global mean sea level rise scenario and the representative concentration pathway 4.5 (RCP-4.5) emissions scenario combination. Evaluation of this scenario combination projects a water level increase of 0.79 feet over the assumed 25-year project life, with a 3% probability the projection will be exceeded. This increase in water level was considered during the modeling and evaluation of the Deer River project site's operational and extreme water levels, wave conditions, and breakwater design wave transmission performance.

A summary of the operational and extreme water levels and wave conditions (wave climate) calculated for the project site are shown in Table 2.

Type	Probability of Exceedance [%]	Return Period [yr]	Water Level [ft, NAVD88]	Significant Wave Height [ft, NAVD88]	Peak Wave Period [s]
Operational	50%	-	1.4	0.4	2.3
Operational	25%	-	1.8	0.5	2.3
Operational	10%	-	2.2	0.7	2.4
Operational	5%	-	2.5	0.7	2.4
Operational	1%	-	3.0	0.9	2.5
Extreme	-	1	4.0	1.2	2.6
Extreme	-	2	4.4	1.4	2.7
Extreme	-	5	5.0	1.6	2.8
Extreme	-	10	5.5	1.8	2.9
Extreme	-	25	6.2	2.1	3.0

Table 2 - Operational and extreme environmental scenarios at Deer River

Additionally, M&N evaluated the wave transmission predicted to occur for the operational and extreme environmental scenarios (Table 2) and structural shoreline design alternatives (rubble mound and Reefmaker^{TM} structures) with varying design crest elevations. This information was used to recommend crest elevations for the evaluated structural-shoreline design alternatives based on their predicted wave-height transmission values.

Based on the results of the wave-transmission analysis, a low-crested, rubble mound breakwater with a design crest elevation of +2.5 ft.NAVD88 would protect the restored shoreline from significant marsh erosion for the 1% probability of exceedance operational-level event, while a Reefmaker^{TM} wave-attenuator structure with a design crest elevation of +3.0 ft.NAVD88 would provide an equivalent level of protection.

Additionally, M&N performed local wave modeling to evaluate the wave-structure interactions (wave transmission, diffraction, etc.) predicted to occur for selected operational environmental scenarios (Table 2) and the Reefmaker^{TM} B system alternative breakwater configuration, provided by Thompson Engineering. Model results confirmed the proposed configuration, using 5-ft-wide fish gaps fronted by approximately 20-ft.-long overlapping sections would result in negligible additional wave energy penetration through the provided gaps.

Finally, an analysis of vessel-generated wave energy was performed for the adjacent Theodore Ship Channel. Predicted vessel-generated secondary waves for the 10 vessels most frequently transiting the channel in the year 2017 were computed and compared to the extreme wind-generated waves (Table ES-2). The wind-generated extreme waves (i.e. 1-yr Return Period and greater) were found to control and are recommended for use in project design.

Alternative 1&2: Continuous Rock Dike and/or Oyster Rings
Alternatives 1 and 2, the continuous Rock Dike and OysterBreak^{TM} alternatives are expected to perform comparably in terms of shoreline protection for the design conditions, so long as they are constructed to comparable crest elevations. Both can be classified as low-crested permeable structures and will provide the highest level of wave protection of the three alternatives by fully enclosing the project area.

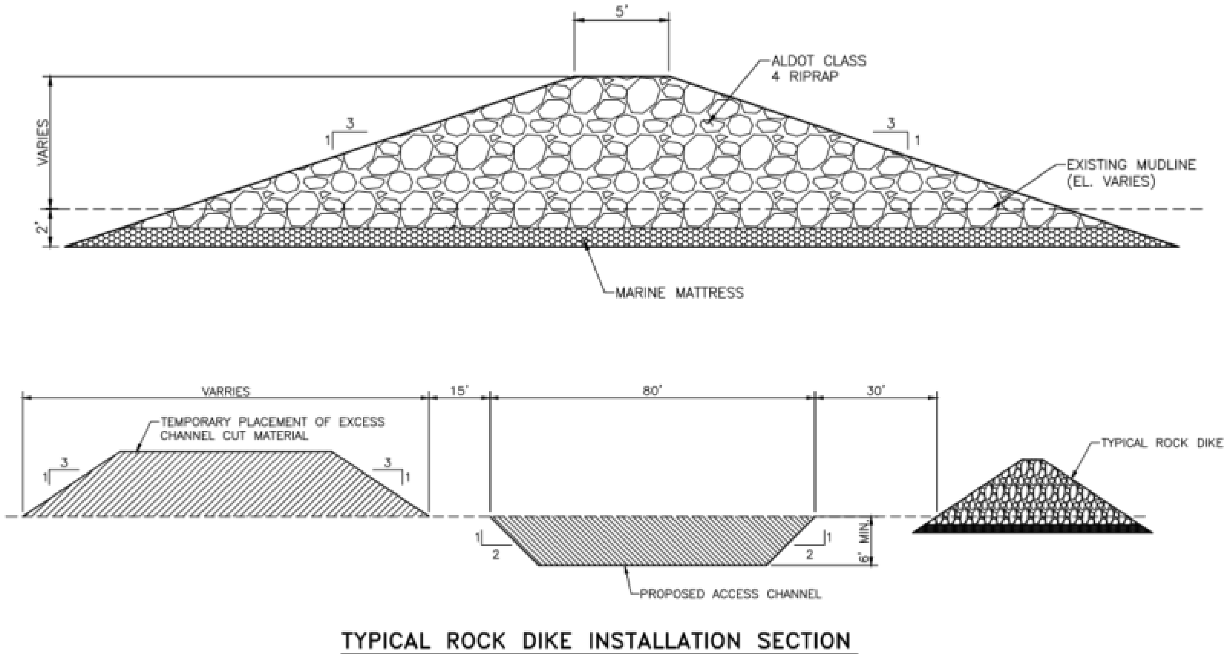


Figure 2 – Rubble Mound Breakwater Section

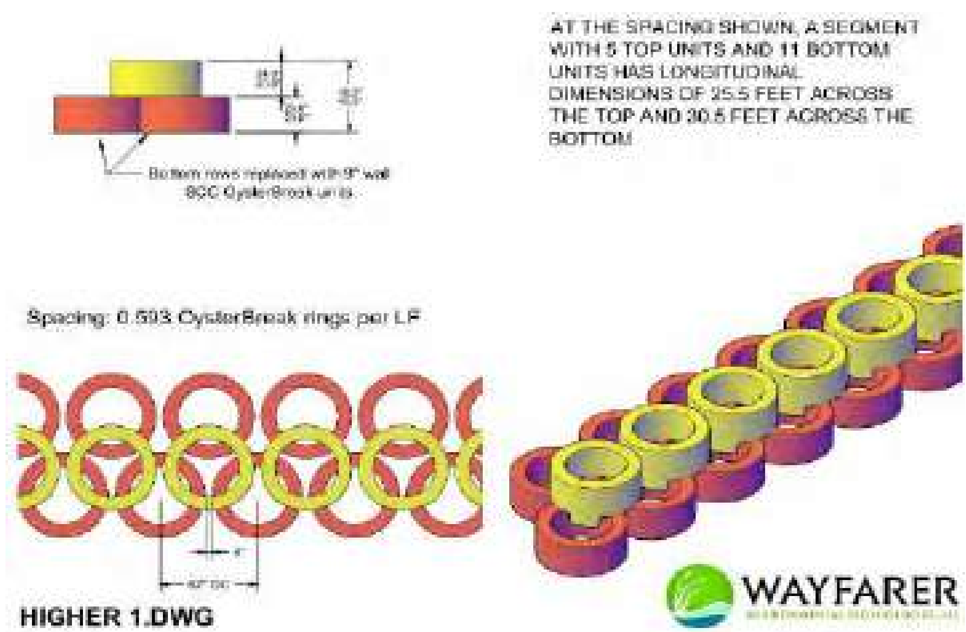


Figure 3 – Typical OysterBreak^{TM} Configuration

One factor differentiating the OysterBreak™ alternative (Figure 3) is its lighter overall weight, lower soil pressure, and less potential for settlement. One disadvantage of the system is its height limitation. The rings can be manufactured in a 20-inch height or 24-inch height and can be stacked multiple units high. Stacked 2-high, the top of the breakwater will be either 40 inches or 48 inches off the bottom (with slight variations depending on the foundation system used). With a rubble-mound breakwater, the height can vary along its length accommodating varying water depths. While the water depth along the proposed breakwater layout does not vary a great deal, there are some variations. With the OysterBreak™ system the breakwater must be laid out along a particular bathymetric contour in order to maintain design breakwater height.

Alternative 3: Continuous Pile-supported Reefmaker™ Breakwater

Due to poor soil conditions along most of the length of the proposed breakwater system, an alternative pile-supported breakwater system was considered. This system, known as *Reefmaker™* (Figure 4) is manufactured by Walter Marine of Orange Beach, Alabama and has been considered as a design alternative at other project sites, including Alabama Department of Conservation and Natural Resources (ADCNR) Marsh Island. The primary advantage of this system is that it is pile-supported with pile length adjustable to suit subsurface soil conditions. The breakwater height is also adjustable by adding additional 12-inch-tall segments to the structure. Thus the system can be deployed in water of varying depth and can also be easily raised in the future to account for sea-level rise.

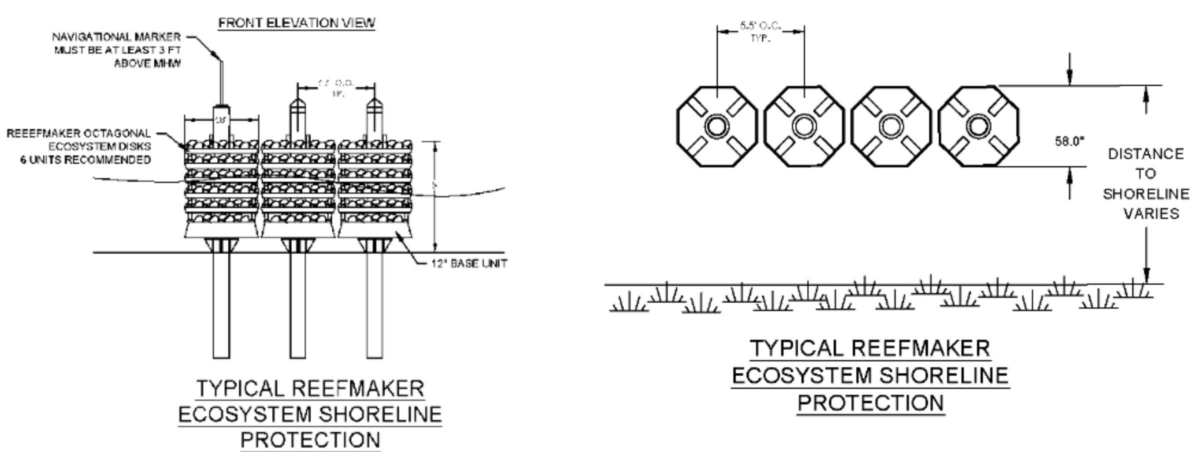


Figure 4 – Typical Reefmaker™ Configuration

Alternative 4: Continuous Sand Berm with Segmented Breakwater

Segmented breakwaters are designed to function by trapping sediment placed behind each structure, causing the shoreline to adjust in an undulating fashion (e.g. by the formation of salients/tombolos), resulting in negligible longshore sand transport during *design conditions. In a sediment-starved system, there is a risk that sand erosion, say due to conditions exceeding design, will be permanent and erosion may accumulate to the point of requiring additional sand placement.*

The presence of segmented breakwaters could result in increased erosion in some areas of the project site, if sediment nourishment is not included as a project component and re-nourishment intervals, i.e. maintenance events, based on a detailed sediment budget analysis (volumetric change over time) are not designed. The key to this design will be optimizing the headland structure array to create the greatest retention of sediment and longest interval between re-nourishment events.

2.3 Sources of Fill for Marsh Creation

During the initial conceptual phase of the project, a marsh-creation was envisioned as one of several measures to be utilized to stabilize and restore the eroding shoreline, and thus protect the marshes beyond from further degradation.

The shoreline is generally inaccessible by land and even if accessible, poor soil conditions along the length of the shoreline would preclude the use of conventional hauling equipment. Material would therefore have to be either dredged hydraulically directly to the marsh-creation site, or sourced from a nearby upland site and transported by truck to location where it could be transferred to barges and transferred either mechanically or hydraulically from the barges to the wetland site.

A pre-application meeting was held with the Corps of Engineers and various other agencies on January 14, 2020. At the meeting the design team was advised that material from the deepening and widening of the Mobile Ship Channel would not be made directly available for our use. Based on this comment and the final analysis of the geotechnical data, marsh creation was not considered a viable alternative. The design team moved forward with plans to simply place an offshore breakwater to reduce wave energy reaching the shoreline and mitigate further erosion of the shoreline.

However, in late March of 2020 we learned the Mobile District had set aside approximately 200,000 cubic yards of sandy material from the Mobile River turning basin for use at Deer River, with the dredging anticipated between July of 2022 and September of 2023. In a teleconference with the Corps of Engineers held April 22, 2020, we obtained further commitment to working with MBNEP and the design teams to develop beneficial-use scenarios between the dredging and restoration projects; though scheduling is still not final.

Should dredge material not be available from the Corps of Engineers, transport and delivery of fill materials to the site by truck and/or barge, with subsequent slurry placement on the site could be considered for several possible sources of fill. These include commercial “borrow pit” sources, as well as USACE dredged material management areas on Blakeley and Pinto Islands. All of these scenarios require duplicative handling of the materials (load to truck, transport and offload to barge, transport and offload at site) which substantially increases costs. Due to poor soil conditions and limited access, trucking of material directly to the site does not appear feasible.

Even though the fill material itself may be “free” or of nominal cost, transport and delivery costs have been estimated on the order of $30 to $45 per cubic yard, and such fill sources were ruled out from consideration at this phase of design. If, however, the material from the Mobile River Turning Basin should not become available, these alternative fill sources could be considered, but may increase project costs by $6 million to $9 million.

2.4 Geotechnical Analysis

A series of marine borings along he proposed breakwater alignment and vibracore probes of the silt deposits in Deer River were performed in December of 2019 and January of 2020. These borings can are detailed in Appendix D – Geotechnical Report.

Figure 5: Soil Boring and Vibracore Locations

The results of the marine borings showed, in certain areas, very soft soils, not only at the surface, but down to as deep as 30 ft. to 40 ft. Other areas were seen to have considerably better subsurface soils. Consolidation tests showed that potential settlement in the “poor soil areas” of a rubble-mound breakwater could be in the range of 18 inches to 24 inches while settlement in the “better soil areas” could be expected to be in the range of 6 inches to 12 inches, with an expectation of an initial 18 inches of initial soil displacement. Settlement of an Oysterbreak™ system, which is considerably lighter, would be expected to be approximately 6 inches in the “poor soil areas” and 3 inches in the "better soil areas".

Analysis of vertical and horizontal (wave-generated) stresses on a pile-supported breakwater yielded a required pile diameter of 12 inches and minimum pile length of 40 ft. with recommended length of 55 ft. if no load test is performed.

Mass fill, i.e. marsh fill is anticipated to settle between 36 inches and 40 inches in the "poor soil areas" and between 3 inches and 8 inches in "better soil areas;" depending on the chosen final marsh elevation.

As can be seen from the above geotechnical summary, soil conditions are highly variable across the shoreline to be protected, so a single breakwater system across the entire length may not be the best option, or even feasible. We have, therefore developed alternatives utilizing combinations of the breakwater systems previously described. Those alternatives are further discussed in Section 3.0.

2.5 Cultural Resources Assessment

A cultural resources assessment was performed by Jason Gardner of Gulf Past Recovery In November 2019 and March and April 2020 (Appendix E). A pedestrian and boat survey was performed of the entire Deer River Restoration project area. Shovel tests were excavated at 30-meter intervals along the existing shoreline.

One archaeological site was recorded as a result of this cultural resources assessment: Site 1Mb580, a generally linear scatter of clam shell and prehistoric and historic artifacts, as well as modern debris. It stretches from northeast to southwest along the eroding shoreline of the Theodore Ship Channel south and along Mobile Bay for a distance of approximately 1,500 ft. and a width of approximately 30 ft. (Figure 6).

Based on the finds from that site, it is recommended that the entire site be either avoided or further investigation (Phase II evaluation) be conducted. Avoidance would mean abandoning 25% of the planned marsh creation site as covering a site with fill would be considered an "impact". However once the Phase-II investigation is conducted we believe marsh creation would be allowed in that area. Therefore, the Phase-II investigation would be anticipated as a condition of the federal permitting process.

Figure 6: Archaeological Site 1Mb580 (Yellow-Shaded Area)

2.6 Submerged Aquatic Vegetation (SAV) Survey

Vittor & Associates inspected a 62.3-acre area of intertidal zone and shallow subtidal habitat at the Deer River Project site for submerged aquatic vegetation (SAV) on November 20, 2019, April 3, 2020 and April 21 2020. Aerial imagery acquired in late July 2019 for coast-wide SAV mapping was used to interpret potential SAV occurrence, and served as a guide for on-site verification. The report of the SAV survey can be found in Appendix F to this document.

Figures 7 shows the locations of SAV at the Project site. Field survey locations were logged in the field with GPS. Small, sparse SAV patches occurred in intertidal areas exposed at the time of the April survey, in addition to beds in shallow subtidal areas. SAV generally occurred at depths of < 2 ft. Larger areas containing multiple SAV patches were delineated as polygons in GIS, with most polygons classified as patchy SAV (< 50% cover). A 0.13-ac (5,663 ft2) polygon at the mouth of the South Fork is classified as continuous (> 50% cover). Small individual patches are reported as point data.

The total acreage of SAV polygons is 0.85 ac (37,157 ft2), with the largest proportion (0.61 ac [26,354 ft2]) occurring at the confluence of the Deer River Middle Fork and Theodore Industrial Canal (Figure 7).

While conservation of SAVs is a priority to regulators at the Alabama Department of Environmental Management (ADEM), it is expected that impacts to the few patchy areas along the Mobile Bay shoreline by marsh-creation and/or shore protection could be allowed; as those patches will eventually be lost due to erosion. The larger area to the north, near the mouth of Deer River will be avoided. All current preliminary design alternatives utilize the pile-supported, offshore breakwater system discussed previously for shore protection in the northern area to avoid SAV impacts.

Figure 7: Submerged Aquatic Vegetation (SAV) Areas

2.7 Deer River Water-Quality Enhancement

Prior to construction of the Theodore Industrial Canal, both north and middle forks of Deer River flowed into Mobile Bay at a location very near the current breach. At some point subsequent to the dredging of the Industrial Canal, the old mouth of Deer River silted in and all flow was directed into the canal.

In 2009/2010 the ongoing erosion of the shoreline reached a bend in the river, creating a breach, and a new “mouth” of the river emptying into Mobile Bay (Figure 8). This however cut off the section between the new breach and the canal from any substantial flow and caused it to silt in, with depths of 1 foot or less in some areas, cutting off other small inlets and tidal creeks from effective tidal exchange.

The loss of flow through the section of Deer River downstream of the breach, and subsequent loss of water-depth and flow capacity is the primary cause of the water-quality degradation in this area.

Therefore, the restoration of flow by either cutting off, or severely restricting flow through the breach is the only alternative to restoration of this flow.

Figure 8: Historic Site Photographs

As part of the original RFQ response, Thompson proposed two potential alternatives (Figure 9) for re-establishing flow in the lower reach of Deer River:

1. Construction of marsh in front of the breach
2. Maintaining the breach, but installing a tidal spillway

Figure 9: Possible Design Concepts (From RFQ Response)

Early in the evaluation process it was determined that a complete and permanent closure, by means of marsh construction at least in the front of the breach was superior to some sort of spillway structure, as it was the only way to ensure maximum flow through the lower reaches of Deer River by either tidal or rainwater flushing.

In addition to closing the breach, the restoration of flow would also require dredging the accumulated silt material from the lower reaches. Observation of our bathymetric data along with a historic survey from 1960, showed the water depths in the area upstream of the breach having remained essentially unchanged (Figures 8 and 9).

Therefore, dredging will only be required in the reach between the old mouth and the canal. In this area, depths vary between 1 ft. and 3 ft. but historically have been between 5 ft. to 6 ft. and as deep as 10 ft. to 13 ft. in bends. After discussing options for dredge depth, the decision was made to dredge the lower reach to a uniform depth of 6’ and allow natural processes to bring the overall riverbed profile back to near its original configuration.

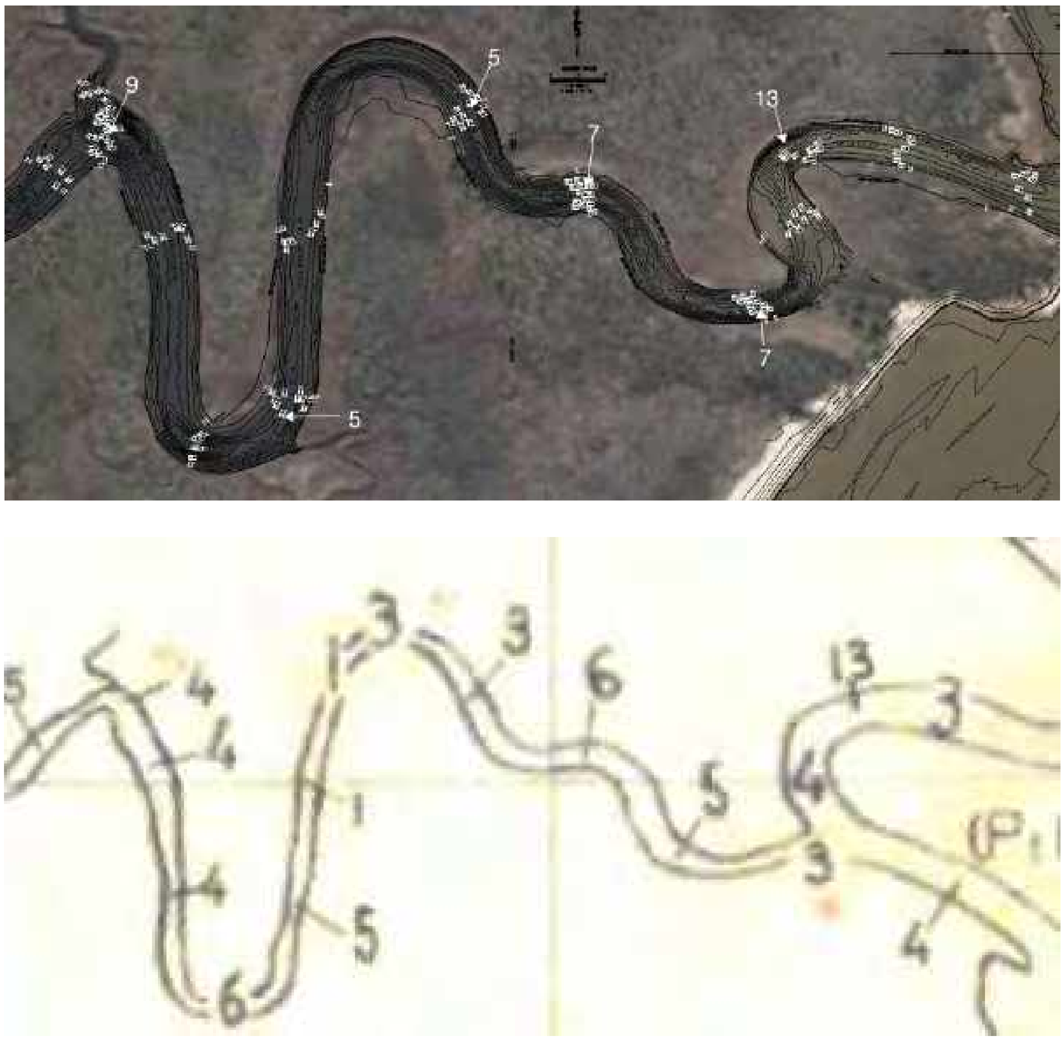


Figure 10: Upper Reach Deer River Modern vs. Historic Bathymetry

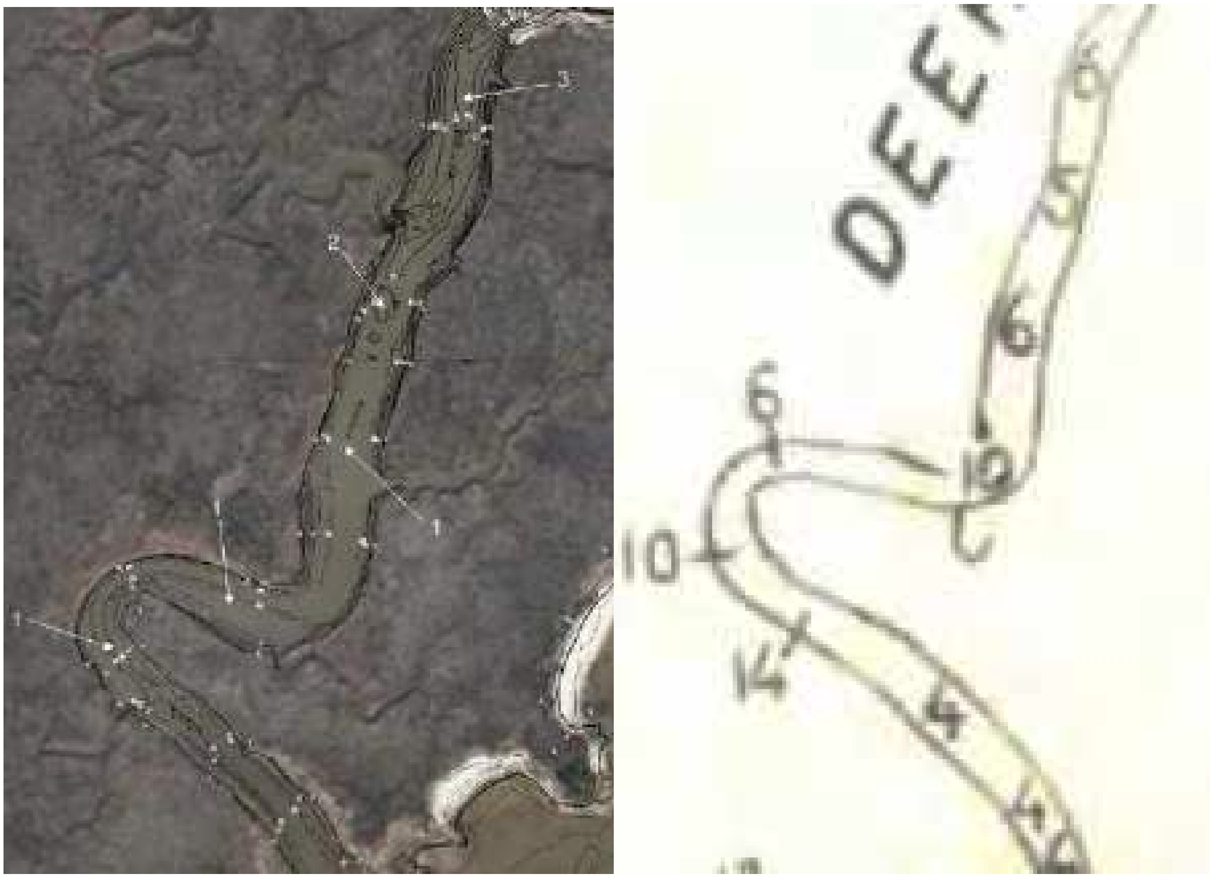


Figure 11: Lower Reach Deer River, Modern vs. Historic Bathymetry

Finally, the ultimate disposal location for the dredged material had to be considered. The two most practical options were (1) utilize the material for the breach-closure marsh creation, or (2) to dispose of the material in thin-layer disposal across the existing marsh to increase the elevation and provide more resilience from sea-level rise.

A total of thirteen (13) vibracore samples were taken of sediment within Deer River. Of those, samples 1-7 were taken in the lower reach where dredging is planned. Much of this material is classified as “muck” meaning essentially a slurry of very fine material with very high water content, not suitable for land-creation.

Borehole	Recovery Ft.	Sample No.	Sample Description	USCS	Sample Location (ft.)*
V-01-A	3.25	T-1	Dark gray, moist, ELASTIC SILT (MH), MUCK, with some organics	MH	0 - 0.5
V-01-A		T-2	Dark gray, moist, CLAY (CH), MUCK, with some organics	CH	0.5 - 3.25
V-02-A	2.00	S-1	Black, wet, SILTY SAND (SM), with some organics	SM	0 - 0.5
V-02-A		S-2	Dark brown, moist, FAT CLAY with SAND (CH), with organics	CH	0.5 - 2
V-02-B	3.25	S-1	Black, wet, FAT CLAY with SAND (CH), MUCK, with some organics	CH	0 - 1.33
V-02-B		S-2	Dark brown, moist, SANDY FAT CLAY (CH), with organics	CH	1.33 - 2.50
V-02-B		S-3	Gray and light brown, moist, CLAYEY SAND (SC), with some organics	SC	2.5 - 3.25
V-03-A	1.92	T-1	Black and gray, wet, ELASTIC SILT (MH), MUCK, with some organics	MH	0 - 1.5
V-03-A		T-2	Black and gray, wet, ELASTIC SILT (MH), MUCK, with some organics	MH	1.5 - 1.92
V-04-A	4.42	S-1	Dark gray, wet, FAT CLAY (CH), MUCK, with some organics	CH	0 - 1.0
V-04-A		S-2	Gray, moist, FAT CLAY (CH), with some organics	CH	1 - 4.42
V-05	3.58	S-1	Black and gray, wet, FAT CLAY (CH), MUCK, with some organics	CH	0 - 1.17
V-05		S-2	Black and gray, wet, FAT CLAY (CH), MUCK, with some organics	CH	1.17 - 3.58
V-06	2.75	S-1	Black and dark gray, wet, FAT CLAY with SAND (CH), soft MUCK, with some organics	CH	0 - 1.17
V-06		S-2	Black and dark gray, wet, FAT CLAY with SAND (CH), soft MUCH, with some organics	CH	1.17 - 2.75
V-07	2.75	S-1	Black and dark gray, wet, SANDY FAT CLAY (CH), MUCK, with some organics	CH	0 - 1.75
V-07		S-2	Black and dark gray, wet, FAT CLAY (CH), MUCK, with some organics	CH	1.75 - 2.75
V-08	2.67	S-1	Black, wet, fine grained, SILTY SAND (SM), with some organics	SM	0 - 0.33
V-08		S-2	Dark gray, wet, FAT CLAY with SAND (CH), with some organics	CH	0.33 - 2.67
V-09	1.67	S-1	Dark gray, wet, FAT CLAY with SAND (CH), with some organics	CH	0 - 1.67
V-10	3.42	S-1	Dark gray, wet, SANDY FAT CLAY (CH), with some organics	CH	0 - 3.42
V-11	3.42	S-1	Dark gray and black, wet, FAT CLAY with SAND (CH), MUCK, with organics	CH	0 - 0.42
V-11		S-2	Dark gray and black, wet, ORGANIC CLAY (CH), MUCK, with some organics	CH	0.42 - 3.42
V-12	2.58	S-1	Dark gray, wet, SANDY LEAN CLAY (CL), with some organics	CL	0 - 2.58
V-13	1.42	S-1	Dark gray, wet, ELASTIC SILT (MH), with organics	MH	0 - 1.42
V-14	4.42	S-1	Gray, wet, FAT CLAY (CH), with some organics	CH	0 - 2.50
V-14		S-2	Brown, wet, FAT CLAY with SAND (CH), with organics	CH	2.5 - 4.42

* All vibracore sampling was performed from mud line to a depth of 10 ft. These sample locations/ depths are within the recorded portions of the vibracore sample.

Table 3 – Summary of Vibracore Sampling Results – from Thompson Engineering Geotechnical Data Report – February, 2020.

The total estimated volume of material to be removed is approximately 46,000 cubic yards. At a thin-layer deposition thickness of approximately 6 inches could potentially increase the elevation of 57 acres of marsh; or if utilized for marsh creation in front of the breach could potentially create approximately 3 to 4 acres of new marsh (assuming 50% settlement/consolidation).

3.0 DISCUSSION AND COST OPINIONS OF ALTERNATIVES

Preliminary investigations of several breakwater alternatives/alignments for the restoration and enhancement of the Mobile Bay shoreline in front of the 275-acre Deer River Marsh have been performed. Whenever possible, quoted prices were compared to identical items on bid tabs from recent projects similar in nature to ensure cost reasonableness. The primary objective of this section of the report is to provide the MBNEP with rough budgetary pricing by which each alternative can be evaluated and to establish an overall budget. A summary comparison of the investigated alternatives and associated cost opinions are provided below in *Table 4: Construction Cost Comparison of Alternatives*. See Appendix A for more detailed cost itemizations and Appendix B for full-size drawings.

Alternative	Cost Opinion
Alternative No. 1 – Continuous Rubble Mound Breakwater	$16,500,000
Alternative No. 2 - Continuous OysterBreakTM	$14,500,000
Alternative No. 3 – Pile Supported Breakwater	$13,300,000
Alternative No. 4 – Sand-Berm Containment with Pocket Beaches and Headland Breakwater	$12,500,000

Table 4 – Construction Cost Comparison of Alternatives

Alternative No. 1 – Continuous Rubble Mound Breakwater:
As discussed in previous sections the geotechnical variability across the site precludes the use of one breakwater type/section across the entire length for most alternatives. The first alternative evaluated consists primarily of a continuous rock dike breakwater structure using DOTD Class IV riprap to construct a continuous dike following an alignment outward from 500' south of the Theodore Ship Channel (Figure 12). The dike would be constructed over Marine Mattresses filled with a bedding stone along the project length. Typical proposed dimensions of the dike are 30 ft. wide at the base and 5 ft. wide at the top with side slopes of 1:3 (V:H). The constructed elevation of the dike will vary along the length depending on soil conditions and expected settlement (see Appendix D) with a final design elevation of the dike at +2.5 ft. NAVD88. Preliminary geotechnical engineering indicates approximately 20 inches of "worst case" settlement, with slightly less in some areas. In "worst case" areas a lightweight core may be used to reduce the bearing pressure and ensure slope stability. The cost estimate associated with this option assumes the use of such a section. Final design of the selected shoreline protection alternative will require additional field and geotechnical analysis to determine

more precise settlement rates across the alignment. However some field adjustment and follow-on maintenance may be required to maintain a consistent top elevation.

The construction of such a dike will require digging an access channel to allow for passage of construction equipment. The channel is recommended to be dredged at a minimum of 6 ft. deep, with an 80 ft. top width. This dictates a certain amount of material being removed and stockpiled. The material excavated from this access channel will be stockpiled on the side of the channel opposite the dike. After placement of all riprap is complete, a portion of the stockpile may be used to create marsh on the inside of the dike. Any material not used for marsh creation behind the dike must be used to fill the access channel.

Because of the poor soil conditions at the northern end of the alignment, combined with the proximity to the shipping channel and the nearby seagrass beds, a pile-supported Reefmaker™ breakwater is proposed to protect the north shore. A breakdown of costs for Alternative No. 1 is attached in *Appendix A: Preliminary Opinion of Probable Costs.*

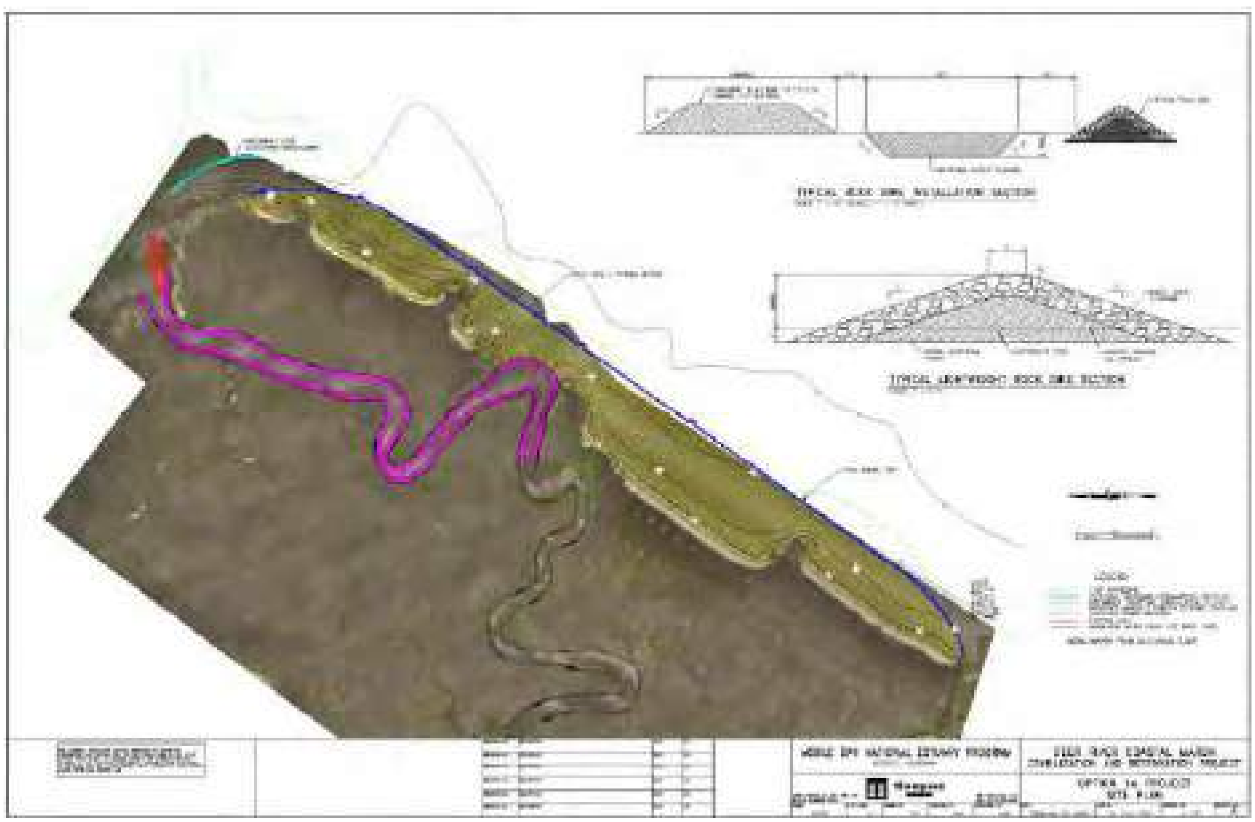


Figure 12: Alternative 1 – Rubble Mound Breakwater

Alternative No. 2 - Continuous OysterBreak™

The second alternative evaluated is a Continuous OysterBreak™ Breakwater following an alternative alignment along the -2 ft. bathymetric contour (Figure 13). The OysterBreak™ system is essentially a finite-height system so its alignment must generally follow an existing contour in order to maintain a consistent top-of-breakwater elevation. This alternative also calls for a base preparation of woven geotextile marine mattresses filled with bedding stone to be installed. Alternative No. 2 calls for an OysterBreak™ design with two layers of 58 inch outside diameter (OD) and 46 inch inside diameter (ID) for each unit, with the top layer interlocked into the base layer. The crest elevation of the OysterBreak™ Armor Unit is anticipated to be approximately +2 NAVD88. *(See Figure 3 OysterBreak™ Typical Configuration)*

OysterBreak™ alternative has certain advantages as it is a lightweight system which would see considerably less settlement than the rubble-mound system. However the risk associated with settlement is much greater because of the finite dimensions of the system. If settlement occurs which exceed projections, the height of the breakwater is reduced along with the anticipated level of wave protection. With a rubble-mound system, rock can always be added during or post-construction to compensate for unanticipated levels of settlement in certain areas. The current preliminary design envisions a 48-inch-high profile with 3 rings on the bottom and 2 rings on top. This will allow for future increase in top elevation of 24 inches to account for sea-level rise or unanticipated settlement.

For this reason, we would not anticipate using the OysterBreak™ alternative in areas with poor, highly compressible marine sediments; but only where soil conditions are less severe, and utilize the pile-supported breakwater in those soft-soil areas. The alignment below in Figure 13 shows an alignment following the -2 ft. contour, turning in before the "poor soil area," and the marsh area will be protected by an offshore pile-supported Reefmaker™ breakwater.

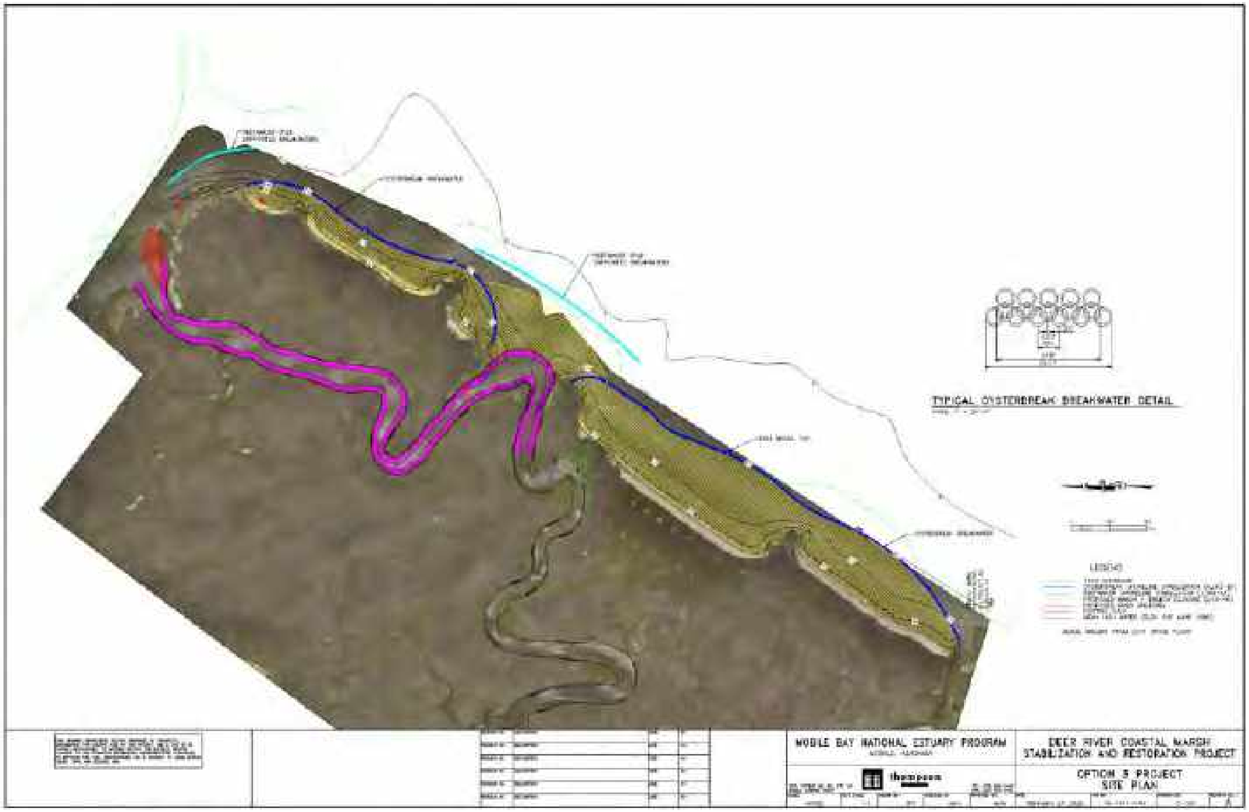


Figure 13: Alternative 2 – OysterBreak™ Breakwater

Alternative No. 3 – Pile Supported Breakwater

The third alternative calls for a pile-supported Reefmaker™ breakwater system along the entire alignment (Figure 12). This system was initially considered as the primary alternative due to the poor soil conditions along the proposed alignment, and because the lack of available material for marsh creation between the breakwater and the existing shoreline. The primary purpose of an offshore breakwater without marsh creation would be to reduce wave action and mitigate ongoing erosion of the existing shoreline.

However, with the potential availability of marsh-fill material from the Turning Basin dredging project, the option remains viable to construct both the pile-supported breakwater and the marsh fill. The Reefmaker™ breakwater system is, however, a flow-through system which would not serve as primary containment for dredge fill. Some other form of containment would need to be placed along the inside of the breakwater. This containment could consist of silt curtain or hay bales or could involve the placement of a sand berm some distance inside the alignment. Additionally, a rock dike or other containment structure may also be utilized to close off the ends of the breakwater which do not tie back into shore if dredged material is used for marsh creation.

The cost of the Reefmaker™ breakwater system is estimated from bids received for Marsh Island in 2015 where the average bid cost was approximately $920/linear foot. For this project, we have conservatively estimated a per-foot cost of $1,200 for the pile-supported breakwater.

Figure 14: Alternative 3 – Pile-Supported Breakwater

Alternative No. 4 – Sand-Berm Containment with Pocket Beaches and Headland Breakwater

The final alternative is the construction of a sand containment berm offshore of the existing shoreline extending out to approximately -2.5 feet of water depth. The sand material could be hydraulically pumped to the site from barges as part of the 200,000 cubic yards of sandy material earmarked for the project from the USACE dredging of the turning basin. The area between the containment berm and the existing shoreline can then be filled with material and planted to form a protective sediment berm. A small dune feature can also be incorporated to add resiliency and protection to the back marsh platform from storm surge and sea level rise.

The sand berm will be protected intermittently with a segmented headland breakwater of Class IV riprap placed atop a marine mattress. The design of the headlands and pocket beaches are based on a combination of the work of Bodge (2003) on the *Design Aspects of Groins and Jetties and Engineering Aspects of Coastal Geomorphology*, and of

Sylvester and Hsu (1993) *Crenulate Shaped Bays*. Headland segments will be 80 to 150 feet long by 30 feet wide with 80-120 foot gaps between headlands; with the expectation that natural pocket beaches will form between the headland segments. The headlands would have a crest elevation of +2.5 feet NAVD 88, and could range from 100 to 200 feet offshore of the existing shoreline. The Bodge (2003) literature also indicates that a stem may be added landward of the headland, creating a T-shape, which would help retain sand in each cell and help facilitate placement of the rip rap headland from shore. This may be of particular interest in including in the design in the northern section of the project to reduce any sediment transport north towards the Theodore Industrial Canal.

Calculations suggest that the Mean High Water Line (beach fill berm) would be offset approximately 30-50 feet from the alignment of the breakwaters. This offset (erosion) could be mitigated, by placing the breakwaters 30-50 feet seaward of the toe of fill. If geotechnical data indicates the substrate in this location will not support rip rap, then the headlands can be moved to the toe of sandy fill to take advantage of a sand base that has surcharged the existing substrate.

The headlands could be constructed in two options. 1) The headlands could follow the same alignment as the continuous rubble-mound breakwater in Figure 12, or 2) work in 3-4 different segments utilizing the existing headlands as keystone locations, filling with sediment in between these salient natural features like the OysterBreak™ alignment in Figure 13. A detailed sediment budget will be developed at the 60% design stage to determine the rate of erosion and the corresponding nourishment interval for all of the selected alternatives.

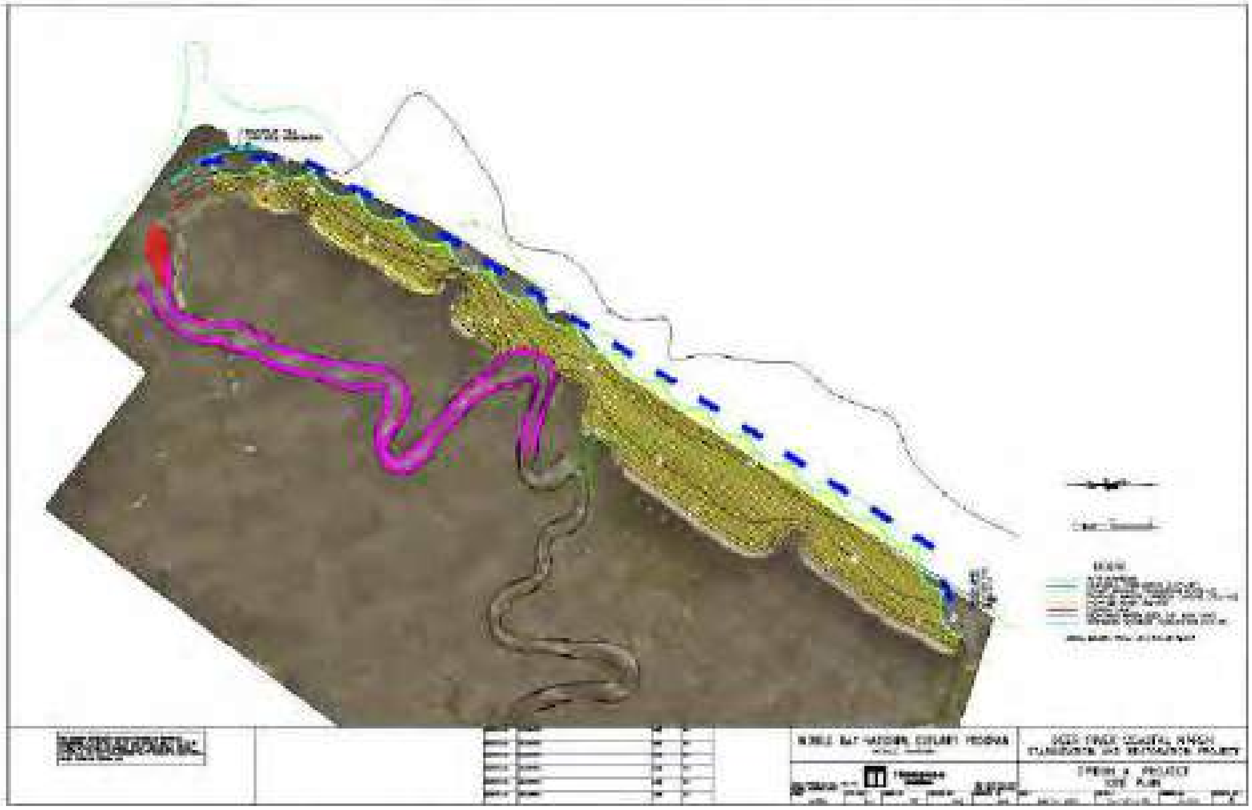


Figure 15: Alternative 4 – Segmented Headland Breakwater

Shoreline Protection Pros/Cons

<u>*Alternatives 1: Continuous Rubble Mound*</u>

Alternative 1, the continuous rubble mound alternative, is generally the industry-preferred shore-protection system due to a long history of success and vast depth of experimental and empirical knowledge about the design and performance of such structures. As a low-crested permeable structure, it will provide the highest level of wave protection by fully enclosing the project area.

However, in this case, major portions of the shoreline are unsuitable for construction of traditional heavy rubble-mound breakwater systems due to deep soft soils, therefore, a lightweight-core breakwater is proposed in these sections to reduce the structure unit weight. Even with the lightweight core, substantial settlement, on the order of 2' to 3' is anticipated over the life of the project.

The lightweight core adds some additional cost due to the increased material cost and increased complexity of installation, but does offset some of those costs in decreased settlement and "lost" material. However, there are additional risks associated with the lightweight core. Should the armor layer somehow be compromised and the lightweight core exposed, the lightweight material could be lost resulting in a collapse of the structure, and loss of protection.

Alternative 2: OysterBreak^{TM} Breakwater System

The OysterBreak^{TM} system, like the rubble mound, can be classified as a low-crested permeable structure and will provide a similar level of wave protection should the crest elevations be the same as the rubble mound. We preliminarily anticipate a final crest elevation of approximately 2.0 ft. NAVD88 (vs. 2.5 for the rubble mound) for the OysterBreak^{TM} breakwater, which will provide a slightly lower level of shoreline protection than the rubble mound.

Due to its fixed, non-adjustable height, the OysterBreak^{TM} system must be placed along a consistent depth contour and cannot tolerate significant settlement without losing its wave-attenuation ability.

The two major advantages of the OysterBreakTM system are its light weight and low relative cost. The anticipated settlement of the OysterBreakTM breakwater is only a small fraction of the settlement calculated for the rubble mound structure; and with the use of pre-loads and marine mattresses, the settlement can be reduced to near zero.

Alternatives 3: Pile-Supported Reefmaker^{TM} Alternative

Alternative 3 involves the installation of a pile-supported breakwater system along the entire 5,600-foot length of the Deer River shoreline.

The primary advantage of this system is its pile-supported foundation which takes advantage of deeper dense soil layers to provide support instead of the loose, compressible surface soils. This system would see virtually no post-construction settlement compared to the other systems which could see 3 in. to 3 ft. of settlement over the life of the structure.

The primary disadvantage, compared to other systems, is the lack of significant historical and/or experimental data to determine the actual anticipated wave transmission through the structure, thus presenting a greater risk for shoreline-protection performance.

Additionally, the porous structure, while providing certain ecological advantages of water-movement through the structure and habitat for marine organisms, provides poor containment for marsh fill. Should it be used in combination with marsh fill, some form of secondary containment would be necessary, though none of the proposed breakwater systems is completely non-porous. Should the *Reefmaker*^{TM} system be used as marsh containment, scour would be a concern since the system generally does not extend below the mudline. However, scour would be less of a concern if the system is used for offshore breakwaters.

Alternative 4: Headland/Breakwater and Pocket Beach Alternative

The advantage of a headland or segmented breakwater approach is a reduction in the amount of Class IV rip rap; between 50%-60% over the continuous breakwater. This design also would reduce fixed structure interference in the cross-shore sediment transport regime. Another advantage is the restoration of beach habitat. Segmented breakwaters are designed to function within a sediment-rich system by trapping sediment behind each structure, causing the shoreline to prograde toward the breakwaters in an undulating fashion (e.g. by the formation of salients/tombolos).

This promotion of sediment deposition behind each structure, however, comes at the cost of potentially increased erosion between the structures, caused by the focusing of wave energy and nearshore currents through the segment gaps. In a sediment-starved system, the little sediment that is available will be trapped behind the updrift breakwaters in the series, causing increased erosion further downshore.

Another disadvantage of the segmented breakwater is that it places certain limitation on the construction sequencing and the choice of fill material. While marsh creation can be performed with a wide variety of sediment types, a very high-quality sand is needed for the outside containment berm and pocket beaches. Additionally, the placement of sand and breakwaters will result in a wider beach than presently exists and any erosion will supply sand to adjacent shores, to potentially be deposited in the navigation channel of the Industrial Canal.

It is also noted that the costs of future maintenance and/or re-nourishment has not been factored in to the estimated construction costs at this time.

4.0 CONCLUSIONS AND RECOMMENDATIONS

[to be added after review of this Draft]

5.0 REFERENCES CITED

Bodge, K. R., 2003. "Design Aspects of Groins and Jetties." American Society of Civil Engineers (ASCE). Reston, VA. ISBN 0-7844-0689-8. Pp. 181-199

Sylvester and Hsu (1993) - Beaches Downcoast of Harbours and Bays. Coastal Eng., 19 (1-2): 163-181.

Sweet et al. (2017) Global and Regional Sea Level Rise Scenarios for the United States. NOAA Technical Report NOS CO-OPS 083

Thompson Engineering (Feb. 2020) – Geotechnical Data Report – Deer River Coastal Marsh Stabilization and Restoration Project

APPENDIX A

PRELIMINARY OPINIONS OF PROBABLE COSTS

Alternative 1 Rubble Mound Breakwater				
Item Description	Unit of Measure	Estimated Quantity	Estimated Unit Price	Estimated Cost
Mob/Demob	lump sum	1	$400,000.00	$400,000
Project Access Channel	linear foot	5600	$225.00	$1,260,000
Pile-Supported Breakwater	linear foot	515	$1,200.00	$618,000
Marine Mattress	square yards	17000	$125.00	$2,125,000
Rip-Rap	tons	40000	$75.00	$3,000,000
Lightweight Aggregate	tons	20000	$85.00	$1,700,000
Deer River Dredge & Placement	cubic yards	46,000	$17.00	$782,000
Marsh Creation	acres	30	$125,000.00	$3,750,000
Settlement Plate	each	12	$2,700.00	$32,400
Warning Sign (Perm)	each	12	$4,200.00	$50,400
Warning Sign (Temp)	each	12	$2,500.00	$30,000
			Subtotal	$13,747,800
			Miscellaneous Contingency (20%)	$2,749,560
			TOTAL ESTIMATED CONSTRUCTION COST	$16,497,360

Alternative 2 Oysterbreak Breakwater				
Item Description	Unit of Measure	Estimated Quantity	Estimated Unit Price	Estimated Cost
Mob/Demob	lump sum	1	$400,000.00	$400,000
Project Access Channel	linear foot	5600	$225.00	$1,260,000
Pile-Supported Breakwater	linear foot	1600	$1,200.00	$1,920,000
Marine Mattress/Preload	square yards	4000	$150.00	$600,000
Oyster Break Breakwater 3:2	linear foot	4300	$900.00	$3,870,000
Marsh Creation	acres	25	$125,000.00	$3,125,000
Deer River Dredge & Placement	cubic yards	46,000	$17.00	$782,000
Settlement Plate	each	12	$2,700.00	$32,400
Warning Sign (Perm)	each	12	$4,200.00	$50,400
Warning Sign (Temp)	each	12	$2,500.00	$30,000
			Subtotal	$12,069,800
			Miscellaneous Contingency (20%)	$2,413,960
			TOTAL ESTIMATED CONSTRUCTION COST	$14,483,760

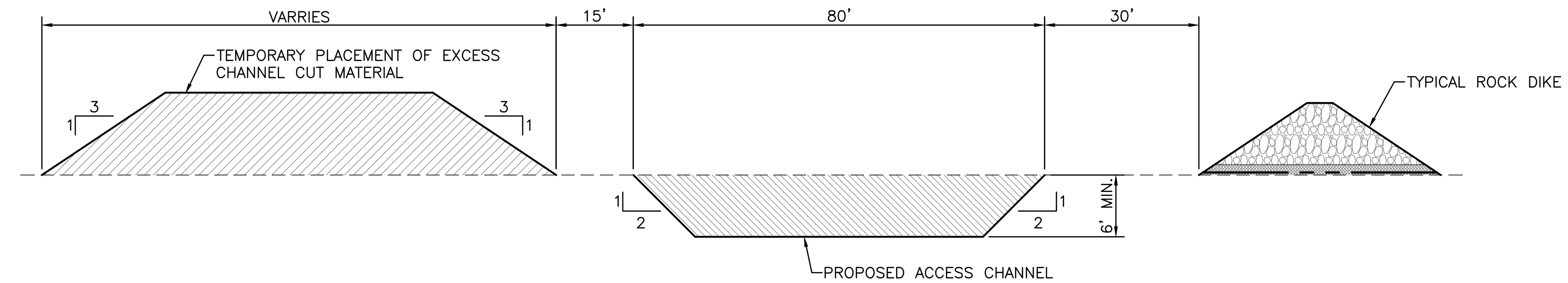
Alternative 3 Reefmaker Breakwater				
Item Description	Unit of Measure	Estimated Quantity	Estimated Unit Price	Estimated Cost
Mob/Demob	lump sum	1	$400,000.00	$400,000
Pile-Supported Breakwater	linear foot	4,800	$1,200.00	$5,760,000
Closure Dikes	linear foot	800	$350.00	$280,000
Marsh Creation	acres	30	$125,000.00	$3,750,000
Deer River Dredge & Placement	cubic yards	46,000	$17.00	$782,000
Settlement Plate	each	12	$2,700.00	$32,400
Warning Sign (Perm)	each	12	$4,200.00	$50,400
Warning Sign (Temp)	each	12	$2,500.00	$30,000
			Subtotal	$11,084,800
			Miscellaneous Contingency (20%)	$2,216,960
			TOTAL ESTIMATED CONSTRUCTION COST	$13,301,760

Alternative 4 Pocket Beach with Headland Breakwaters				
Item Description	Unit of Measure	Estimated Quantity	Estimated Unit Price	Estimated Cost
Mob/Demob	lump sum	1	$400,000.00	$400,000
Project Access Channel	linear foot	5,600	$225.00	$1,260,000
Pile-Supported Breakwater	linear foot	515	$1,200.00	$618,000
Marine Mattress	square yards	8,500	$125.00	$1,062,500
Rip-Rap	tons	20,000	$75.00	$1,500,000
Lightweight Aggregate	tons	10,000	$85.00	$850,000
Deer River Dredge & Placement	cubic yards	46,000	$17.00	$782,000
Marsh Creation	acres	30	$125,000.00	$3,750,000
Settlement Plate	each	20	$2,700.00	$54,000
Warning Sign (Perm)	each	20	$4,200.00	$84,000
Warning Sign (Temp)	each	20	$2,500.00	$50,000
			Subtotal	$10,410,500
			Miscellaneous Contingency (20%)	$2,082,100
			*TOTAL ESTIMATED CONSTRUCTION COST	$12,492,600

*future maintenance not inlcuded

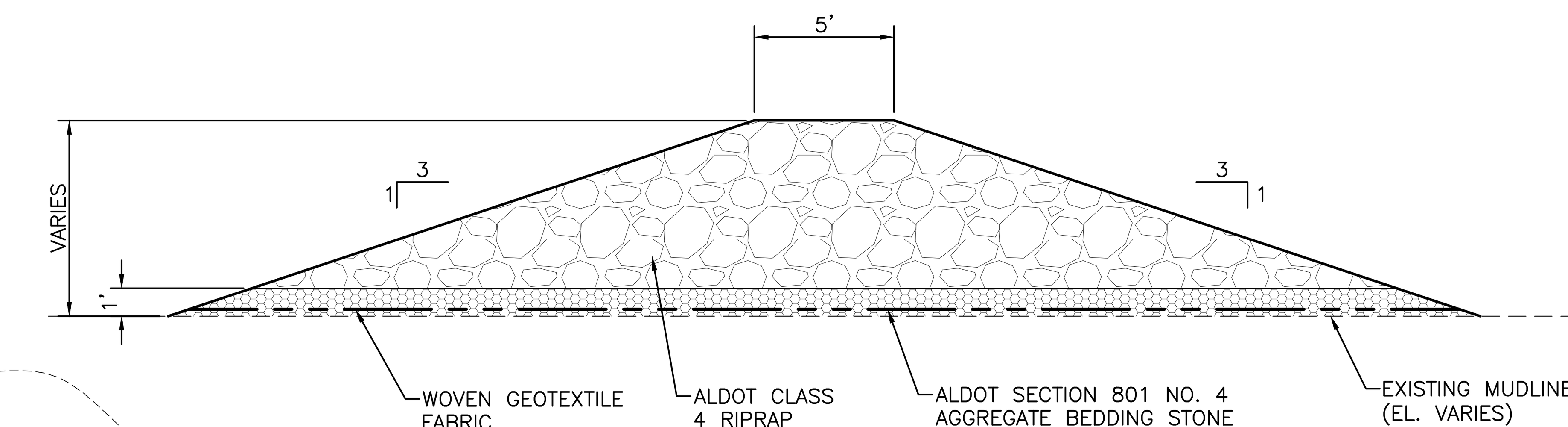
APPENDIX B

PRELIMINARY DRAWINGS



TYPICAL ROCK DIKE INSTALLATION SECTION

SCALE: 1" = 20' (HORIZ.); 1" = 10' (VERT.)



TYPICAL ROCK DIKE SECTION

SCALE: 1" = 5'-0"

LEGEND

- 1979 SHORELINE
- PROPOSED SHORELINE STABILIZATION (4570-LF)
- PROPOSED MARSH / BREECH CLOSURE
- PROPOSED RIVER DREDGING
- EXISTING S.A.V.
- MEAN HIGH WATER (ELEV. 0.8' NAVD 1988)

AERIAL IMAGERY FROM 2019 DRONE FLIGHT

REVISION NO.	DESCRIPTION	DATE	BY:

MOBILE BAY NATIONAL ESTUARY PROGRAM
MOBILE, ALABAMA

thompson ENGINEERING
2970 COTTAGE HILL RD., STE. 190
MOBILE, ALABAMA 36606
TEL: (251) 666-2443
FAX: (251) 666-6422

DEER RIVER COASTAL MARSH STABILIZATION AND RESTORATION PROJECT

OPTION 1 PROJECT SITE PLAN

SCALE:	PLOT SCALE:	DRAWN BY:	CHECKED BY:	APPROVED BY:	DATE :	JOB NO. :	DRAWING NO.	REVISION NO. :
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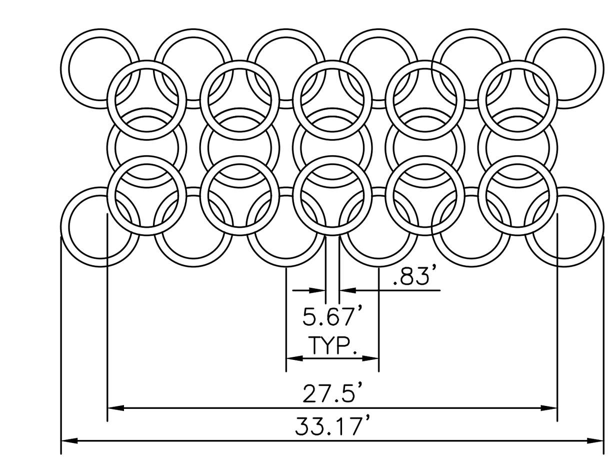

TYPICAL OYSTERBREAK BREAKWATER DETAIL

SCALE: 1" = 20'-0"

0 200 400 Feet

LEGEND

- 1979 SHORELINE
- OYSTERBREAK SHORELINE STABILIZATION (4,263-LF)
- REEFMAKER SHORELINE STABILIZATION (1,569-LF)
- PROPOSED MARSH / BREECH CLOSURE (24.8-AC)
- PROPOSED RIVER DREDGING
- EXISTING S.A.V.
- MEAN HIGH WATER (ELEV. 0.8' NAVD 1988)

AERIAL IMAGERY FROM 2019 DRONE FLIGHT

REVISION NO.	DESCRIPTION	DATE	BY:
REVISION NO.	DESCRIPTION	DATE	BY:
REVISION NO.	DESCRIPTION	DATE	BY:
REVISION NO.	DESCRIPTION	DATE	BY:
REVISION NO.	DESCRIPTION	DATE	BY:
REVISION NO.	DESCRIPTION	DATE	BY:

MOBILE BAY NATIONAL ESTUARY PROGRAM
MOBILE, ALABAMA

DEER RIVER COASTAL MARSH STABILIZATION AND RESTORATION PROJECT

OPTION 2 PROJECT SITE PLAN

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2970 COTTAGE HILL RD., STE. 190
MOBILE, ALABAMA 36606
TEL: (251) 666-2443
FAX: (251) 666-6422

SCALE:	PLOT SCALE:	DRAWN BY:	CHECKED BY:	APPROVED BY:	DATE :	JOB NO. :	DRAWING NO.	REVISION NO. :
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TYPICAL PILE SUPPORTED BREAKWATER DETAIL

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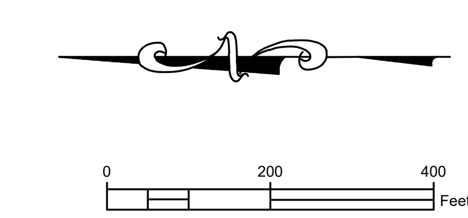

LEGEND

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PROPOSED SHORELINE STABILIZATION (4790–LF)
PROPOSED MARSH / BREECH CLOSURE (31–AC)
PROPOSED RIVER DREDGING
EXISTING S.A.V.
MEAN HIGH WATER (ELEV. 0.8' NAVD 1988)

AERIAL IMAGERY FROM 2019 DRONE FLIGHT

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REVISION NO.	DESCRIPTION	DATE	BY:
REVISION NO.	DESCRIPTION	DATE	BY:
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REVISION NO.	DESCRIPTION	DATE	BY:

MOBILE BAY NATIONAL ESTUARY PROGRAM
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TEL: (251) 666–2443
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DEER RIVER COASTAL MARSH STABILIZATION AND RESTORATION PROJECT

OPTION 3 PROJECT
SITE PLAN

SCALE:	PLOT SCALE:	DRAWN BY:	CHECKED BY:	APPROVED BY:	DATE :	JOB NO. :	DRAWING NO.	REVISION NO. :
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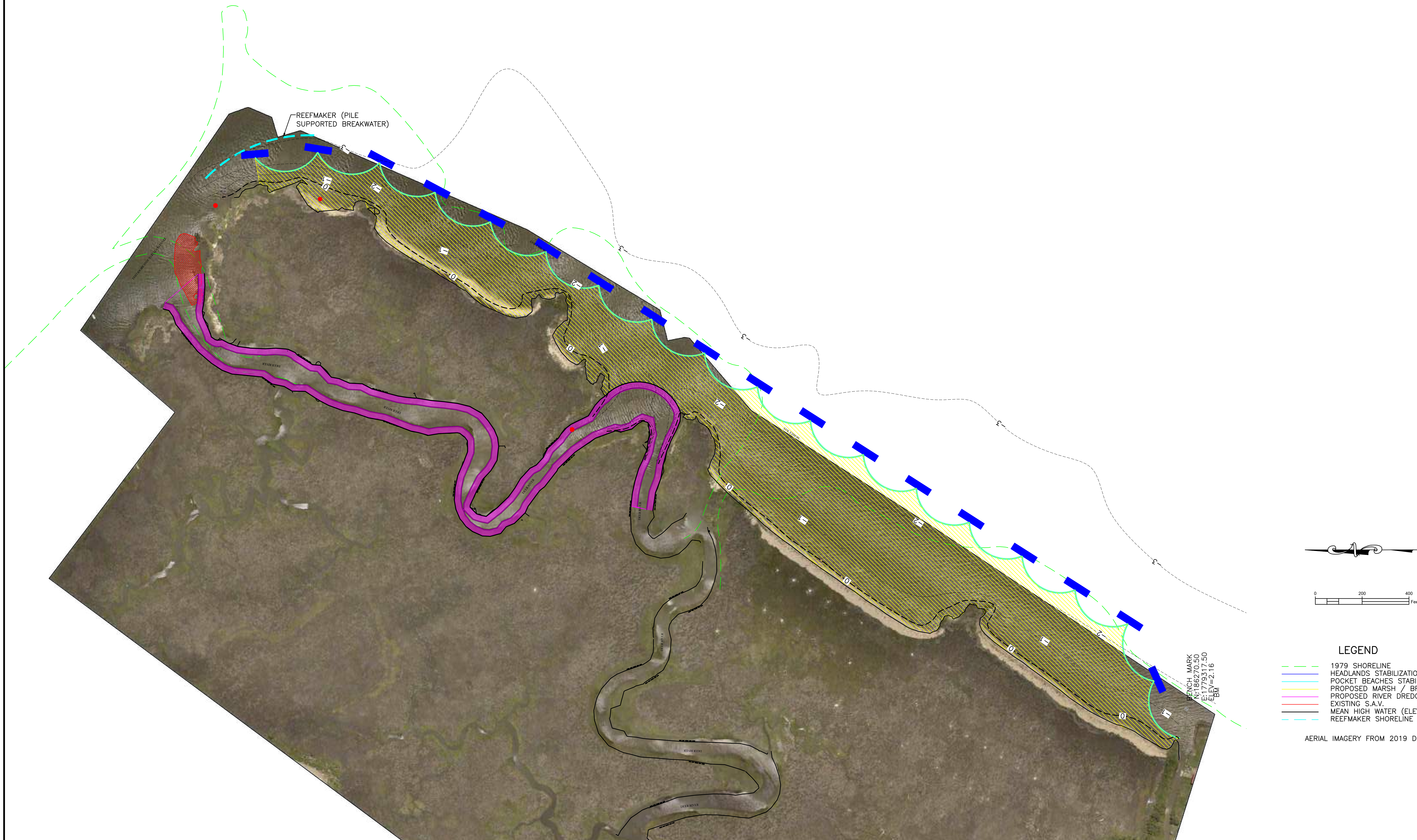

LEGEND

1979 SHORELINE
HEADLANDS STABILIZATION (2,070–LF)
POCKET BEACHES STABILIZATION (5,658–LF)
PROPOSED MARSH / BREECH CLOSURE (30.2–AC)
PROPOSED RIVER DREDGING
EXISTING S.A.V.
MEAN HIGH WATER (ELEV. 0.8' NAVD 1988)
REEFMAKER SHORELINE STABILIZATION (513–LF)

AERIAL IMAGERY FROM 2019 DRONE FLIGHT

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REVISION NO.	DESCRIPTION	DATE	BY:
REVISION NO.	DESCRIPTION	DATE	BY:
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REVISION NO.	DESCRIPTION	DATE	BY:

MOBILE BAY NATIONAL ESTUARY PROGRAM
MOBILE, ALABAMA

DEER RIVER COASTAL MARSH STABILIZATION AND RESTORATION PROJECT

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FAX: (251) 666–6422

OPTION 4 PROJECT SITE PLAN

SCALE:	PLOT SCALE:	DRAWN BY:	CHECKED BY:	APPROVED BY:	DATE :	JOB NO. :	DRAWING NO.	REVISION NO. :
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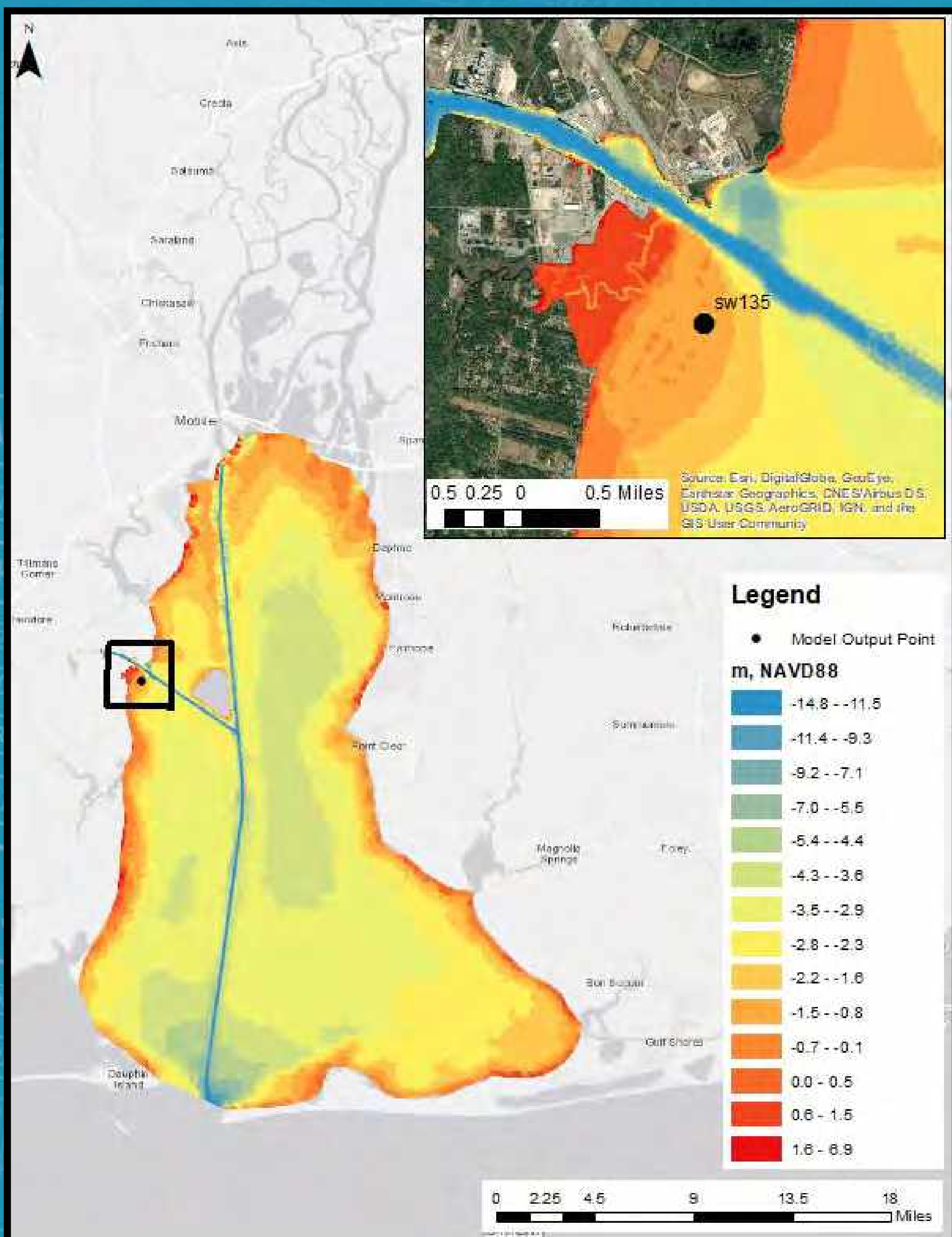
APPENDIX C
WAVE CLIMATE ANALYSIS

moffatt & nichol

Deer River Wave Modeling Summary Report
Produced for Thompson Engineering
May 13, 2020



Document Verification

Client	Thompson Engineering
Project name	Deer River Restoration
Document title	Deer River Wave Modeling Summary Report
Document sub-title	–
Status	FINAL
Date	May 13, 2020
Project number	10852
File reference	Q:\BR\10852-Deer River E-D\400_Reports\402_Memo_Coastal\M&N_Wave_Summary_Report_DeerRiver_20200513_FINAL.docx

Revision	Description	Issued by	Date	Checked
A	Draft Report	KH, KPS, CT	04/16/2020	JS
0	Final Report	KH, KPS, CT	05/13/2020	JS

Produced by:
Moffatt & Nichol
11 North Water Street, Suite 20220
Mobile, AL 36602
(251) 378-9000
www.moffattnichol.com

Executive Summary

This report summarizes the wave modeling (including the incorporation of sea level rise) performed by Moffatt & Nichol (M&N) conducted in support of the shoreline structure design for the Deer River Restoration Project. Wave modeling conducted by M&N for the Deer River Restoration Project includes a regional (scale of Mobile Bay) MIKE21-Spectral Wave model as well as a local (scale of the project site) MIKE21-Boussinesq Wave model. Modeling was conducted to accurately determine the wave climate at the Deer River Restoration project site and the local wave transformation to be used for specific engineering and design components. These models were used to evaluate the effects of potential breakwater design alternatives (namely a low-crested rubble mound breakwater and Reefmaker© wave-attenuator system) on wave conditions reaching the restored shoreline for various site-specific environmental scenarios (levels of extreme to operational waves).

Analysis of established tidal datums at nearby tidal gages and spatial interpolation were used to compute tidal datums at the project site, where an astronomical diurnal (one high and one low tide per day) tide range of approximately 1.54 feet (Table ES-1) is predicted. These findings agree with published literature that document Mobile Bay as a micro-tidal environment.

Table ES-1: Interpolated tidal datums at the Deer River project site.

Tidal Datums	Deer River [ft, NAVD88]
MHHW (Mean Higher High Water)	**1.0**
MHW (Mean High Water)	**0.9**
MTL (Mean Tide Level)	**0.2**
MSL (Mean Sea Level)	**0.2**
MLW (Mean Low Water)	**-0.5**
MLLW (Mean Lower Low Water)	**-0.6**

Sea level rise planning scenarios from Sweet et al. (2017) were used to evaluate relative sea level rise at the Deer River project site for a 25-year project design life (project life from 2020 to 2045). M&N recommends the Deer River Restoration project consider the use of the intermediate (1.0 meter, 3.28 feet) global mean sea level rise scenario and the representative concentration pathway 4.5 (RCP-4.5) emissions scenario combination. Evaluation of this scenario combination projects a water level increase of 0.79 feet over the assumed 25-year project life provides, with a 3% probability that the projection will be exceeded. This increase in water level was considered during the modeling and evaluation of the Deer River project site's operational and extreme water levels, wave conditions, and breakwater design wave transmission performance.

A summary of the operational and extreme water levels and wave conditions (wave climate) calculated for the project site are shown in Table ES-2.

Table ES-2: Operational and extreme environmental scenarios at Deer River, AL. Each scenario includes a water level, significant wave height, and associated peak wave period.

Type	Probability of Exceedance [%]	Return Period [yr]	Water Level [ft, NAVD88]	Significant Wave Height [ft, NAVD88]	Peak Wave Period [s]
Operational	50%	-	1.4	0.4	2.3
Operational	25%	-	1.8	0.5	2.3
Operational	10%	-	2.2	0.7	2.4
Operational	5%	-	2.5	0.7	2.4
Operational	1%	-	3.0	0.9	2.5
Extreme	-	1	4.0	1.2	2.6
Extreme	-	2	4.4	1.4	2.7
Extreme	-	5	5.0	1.6	2.8
Extreme	-	10	5.5	1.8	2.9
Extreme	-	25	6.2	2.1	3.0

Additionally, M&N evaluated the wave transmission predicted to occur for the operational and extreme environmental scenarios (Table ES-2) and structural shoreline design alternatives (rubble mound and Reefmaker© structures) with varying structure design crest elevations. This information was used to recommend crest elevations for the evaluated structural shoreline design alternatives based on their predicted wave height transmission values. Table ES-3 and Table ES-4 summarize the wave height transmission results predicted to occur for structural shoreline design crest elevations ranging from 1.0 ft NAVD88 to 5.0 ft NAVD88 for both rubble mound and Reefmaker© design alternatives, respectively. Bolded wave transmission values in Table ES-3 and Table ES-4 indicate occurrences where the breakwater is sized such that the transmitted significant wave height for a particular environmental scenario is less than approximately 0.5 ft, an assumed threshold below which the leeward marsh shoreline is not expected to be significantly eroded.

Based on the results of the wave transmission analysis, a low-crested, rubble mound breakwater with a design crest elevation of +2.5 ft NAVD88 would protect the restored shoreline from significant marsh erosion for the 1% probability of exceedance operational-level event, while a Reefmaker© wave-attenuator structure with a design crest elevation of +3.0 ft NAVD88 would provide an equivalent level of protection.

Additionally, M&N performed local wave modeling to evaluate the wave-structure interactions (wave transmission, diffraction, etc.) predicted to occur for selected operational environmental scenarios (Table ES-2) and the Reefmaker© system alternative breakwater configuration, provided by Thompson Engineering. Model results confirmed that the proposed configuration, using 5 ft wide fish gaps fronted by approximately 20 ft long overlapping sections results in negligible additional wave energy penetration through the provided gaps.

Finally, an analysis of vessel-generated wave energy was performed for the adjacent Theodore Ship Channel. Predicted vessel-generated secondary waves for the 10 vessels that most-frequently transited the channel in the year 2017 were computed and compared to the extreme wind-generated waves (Table ES-2). The wind-generated extreme waves (i.e. 1-yr Return Period and greater) were found to control and are recommended for use in project design.

Table ES-3: Rubble mound breakwater wave transmission results for various operational and extreme environmental scenarios and breakwater crest elevation alternatives. Crest breadth and foreslope were held constant at 5 ft and 1:3 (Horizontal:vertical), respectively.

Type	Prob. of Exceedance [%]	Return Period [yr]	Water Level [ft, NAVD88]	Significant Wave Height [ft, NAVD88]	Crest Elevation [ft, NAVD88]	1.0	1.5	2.0	2.5	3.0	4.0	5.0
					Peak Wave Period [s]	Transmitted Wave Height [ft]						
Operational	50%	-	1.4	0.4	2.3	0.25	0.05	-	-	-	-	-
Operational	25%	-	1.8	0.5	2.3	0.46	0.26	0.06	-	-	-	-
Operational	10%	-	2.2	0.7	2.4	0.65	0.45	0.26	0.06	-	-	-
Operational	5%	-	2.5	0.7	2.4	0.72	0.56	0.36	0.17	-	-	-
Operational	1%	-	3.0	0.9	2.5	0.92	0.81	0.61	0.42	0.21	-	-
Extreme	-	1	4.0	1.2	2.6	1.21	1.21	1.10	0.90	0.69	0.28	-
Extreme	-	2	4.4	1.4	2.7	1.38	1.38	1.31	1.12	0.91	0.50	0.11
Extreme	-	5	5.0	1.6	2.8	1.61	1.61	1.61	1.42	1.21	0.80	0.41
Extreme	-	10	5.5	1.8	2.9	1.84	1.84	1.84	1.70	1.49	1.08	0.69
Extreme	-	25	6.2	2.1	3.0	2.13	2.13	2.13	2.07	1.86	1.46	1.06

Table ES-4: Reefmaker Ecosystem: Wave Attenuator© wave transmission results for various operational and extreme environmental scenarios and breakwater crest elevation alternatives.

Type	Prob. of Exceedance [%]	Return Period [yr]	Water Level [ft, NAVD88]	Significant Wave Height [ft, NAVD88]	Crest Elevation [ft, NAVD88]	1.0	1.5	2.0	2.5	3.0	4.0	5.0
					Peak Wave Period [s]	Transmitted Wave Height [ft]						
Operational	50%	-	1.4	0.4	2.3	0.32	0.20	0.17	0.17	0.17	0.17	0.17
Operational	25%	-	1.8	0.5	2.3	0.42	0.36	0.24	0.21	0.21	0.21	0.21
Operational	10%	-	2.2	0.7	2.4	0.52	0.52	0.41	0.29	0.26	0.26	0.26
Operational	5%	-	2.5	0.7	2.4	0.58	0.58	0.50	0.38	0.29	0.29	0.29
Operational	1%	-	3.0	0.9	2.5	0.73	0.73	0.72	0.60	0.47	0.37	0.37
Extreme	-	1	4.0	1.2	2.6	0.97	0.97	0.97	0.97	0.88	0.63	0.49
Extreme	-	2	4.4	1.4	2.7	1.10	1.10	1.10	1.10	1.07	0.82	0.58
Extreme	-	5	5.0	1.6	2.8	1.29	1.29	1.29	1.29	1.29	1.08	0.84
Extreme	-	10	5.5	1.8	2.9	1.47	1.47	1.47	1.47	1.47	1.33	1.09
Extreme	-	25	6.2	2.1	3.0	1.71	1.71	1.71	1.71	1.71	1.66	1.42

Table of Contents

List of Figures

List of Tables

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1. Introduction

The Mobile Bay National Estuary Program (MBNEP) is pursuing a project to restore the Deer River coastal marsh and shoreline along the western shore of Mobile Bay. The project would seek to stabilize and enhance:

- The 5,600-foot shoreline of the 275-acre Deer River salt marsh tract on the western shore of Mobile Bay directly south of the Theodore Industrial Canal. The shoreline has experienced significant recession from storms, tides, and ship wakes.
- The network of tidal channels, including the South and Middle Forks of Deer River, which are extremely shallow and impaired by siltation, limiting tidal exchange and circulation necessary to sustain the currently healthy marsh.

An engineering and design team led by Thompson Engineering has been selected by the MBNEP to develop engineering and design documents and secure permits for the Deer River Restoration project. As part of the engineering and design team, Moffatt and Nichol (M&N) has been scoped with performing wave modeling and breakwater wave transmission analysis to inform project component design.

This report summarizes the wave modeling (including the incorporation of sea level rise) performed by M&N conducted to support the shoreline structure design (by others) for the Deer River Restoration Project. Wave modeling conducted by M&N for the Deer River Restoration Project includes a regional (scale of Mobile Bay) MIKE21-Spectral Wave model as well as a local (scale of the project site) MIKE21-Boussinesq Wave model.

Modeling was conducted to accurately determine the wave climate at the Deer River Restoration project site and the local wave transformation to be used for specific engineering and design components. These models were used to evaluate the effects of potential breakwater design alternatives (namely a low-crested rubble mound breakwater and Reefmaker© wave-attenuator system) on wave conditions reaching the restored shoreline for various site-specific environmental scenarios (levels of extreme to operational waves).

2. Water Levels

2.1. Approach

Water level measurements are not available directly at the project site. However, water level measurements are available from the National Oceanic and Atmospheric Administration's (NOAA) Center for Operational Oceanographic Products and Services (CO-OPS) (NOAA, 2020) for several nearby stations. Four (4) NOAA CO-OPS stations were identified in the vicinity of Deer River and collectively provide water level measurements encompassing portions of the time period from 2001 through 2019. Table 1 provides a list of NOAA CO-OPS stations used for this project, including the length of the station's respective water level records and the station's approximate proximity to the project site.

Table 1: NOAA CO-OPS stations with water level measurements relevant to Deer River.

NOAA CO-OPS Station	NOAA CO-OPS Station ID	Water Level Period of Record	Proximity to Deer River (Miles)
Coast Guard Sector	8736897	2007-2019	9.1
Dog River Bridge	8735391	2011-2019	3.1
East Fowl River Bridge	8735523	2011-2019	5.5
Dauphin Island	8735180	2001-2019	18.5

The location of the project site relative to various measurement gages is given in Figure 1. Dog River Bridge and East Fowl River Bridge measured water levels are more representative of those at the project site due to the short distance between the locations, but Deer River water levels are expected to differ slightly between those measured at these gages. Additionally, the approximately 9-year period of record at Dog River Bridge and East Fowl River Bridge is too short to characterize the operational and extreme water level conditions, so a procedure was developed to extend the record of water levels at the project site based on interpolation and curve fitting.

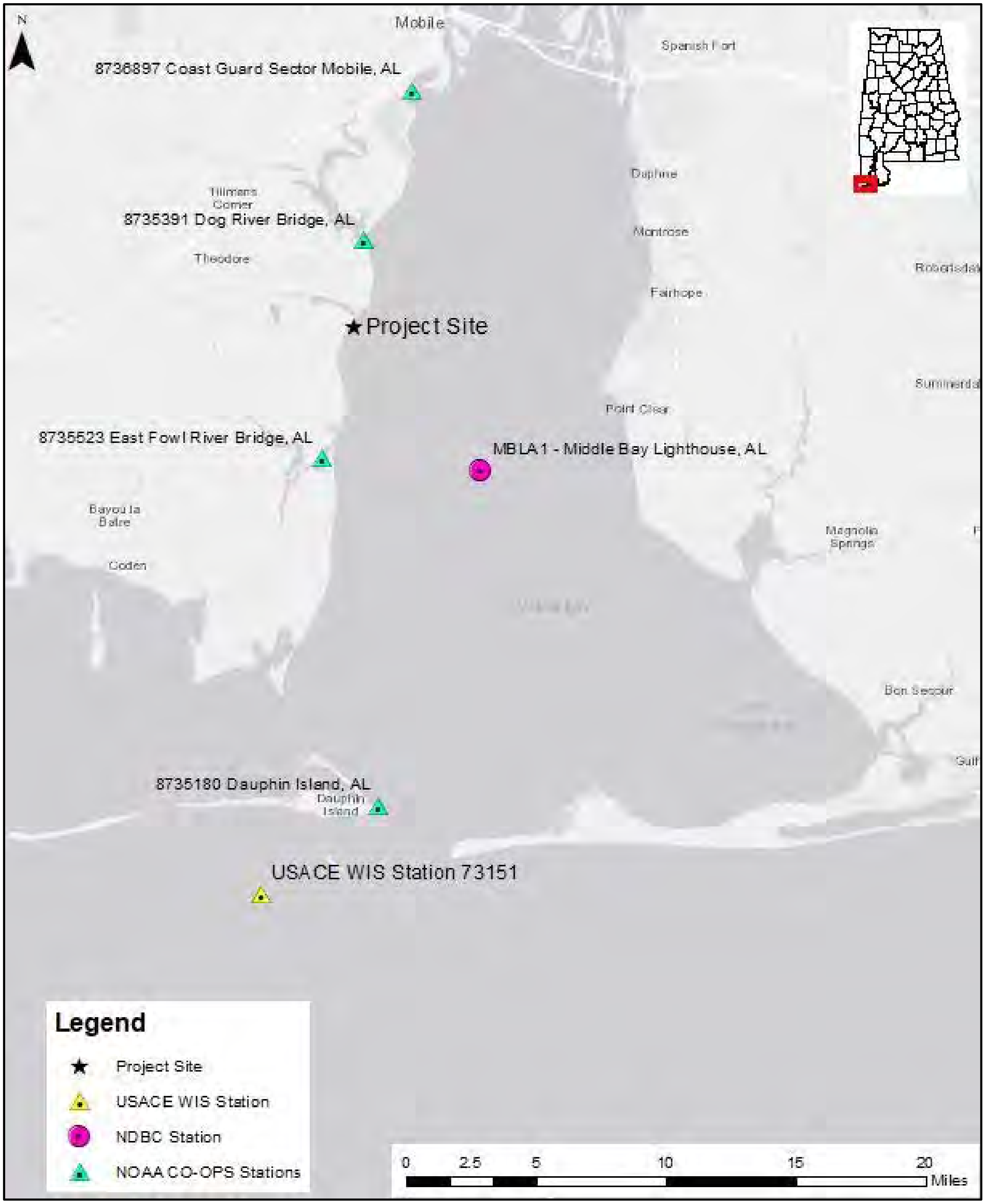


Figure 1: Location of the project site relative to the USACE WIS Station, NDBC Station, and NOAA CO-OPS stations.

For the overlapping measurement time period from 2011 through 2019 when data is available at both Dog River Bridge and East Fowl River Bridge, water levels at Deer River were interpolated based on the relative distance to each gage. Deer River interpolated water levels were then extrapolated for the time period from 2001-2011 with a best fit polynomial as a function of the Dauphin Island water level (see Figure 2). This derived record of water levels at Deer River was then analyzed to determine the levels corresponding to various frequencies of occurrence and extreme recurrence intervals (return periods). Results of these analyses are given in the following sections, both for current conditions and taking into account the Relative Sea Level Rise (RSLR) expected to occur during the project design life.

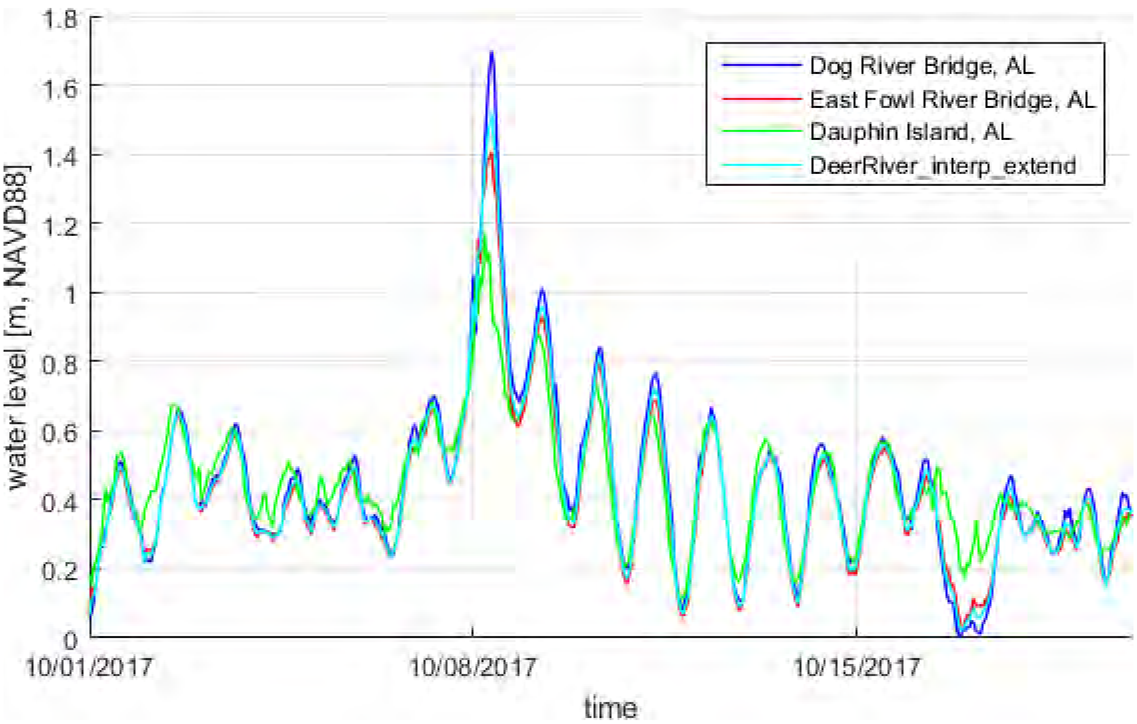


Figure 2: Example of the water level interpolation between Dog River Bridge, AL, East Fowl River Bridge, AL, and Dauphin Island, AL NOAA CO-OPS water level gages for a period of overlapping records.

2.2. Tidal Datums

Tidal datums at the project site were derived from datums at the nearby Dog River Bridge and East Fowl River Bridge NOAA CO-OPS tidal gages using distance-based interpolation, similar to the procedure used to derive the full water level time series. Tidal datum data obtained from the Dog River Bridge NOAA CO-OPS station was adjusted to reference the North American Vertical Datum of 1988 (NAVD88) by M&N as the station data as provided by NOAA is not surveyed and tied to a standard vertical engineering datum. Due to the close distance and limited variation in water levels across Mobile Bay, the tidal datums at Deer River were interpolated based on the relative distance to those at Dog River Bridge and East Fowl River Bridge. Table 2 gives the tidal gage datums and interpolated tidal datums for Deer River.

Table 2: Tidal Datums at NOAA CO-OPS Gages in project vicinity and interpolated datums at Deer River project site.

Tidal Datums [ft, NAVD88]	Coast Guard Sector (8736897)	Dauphin Island (8735180)	Dog River Bridge (8735391)	East Fowl River Bridge (8735523)	Deer River (Interpolated)
MHHW (Mean Higher High Water)	1.1	0.7	1.0	0.8	**1.0**
MHW (Mean High Water)	1.1	0.7	1.0	0.8	**0.9**
MTL (Mean Tide Level)	0.3	0.1	0.2	0.1	**0.2**
MSL (Mean Sea Level)	0.3	0.1	0.2	0.1	**0.2**
MLW (Mean Low Water)	-0.5	-0.5	-0.5	-0.6	**-0.5**
MLLW (Mean Lower Low Water)	-0.5	-0.5	-0.6	-0.7	**-0.6**

2.3. Current Conditions Results

The following section details the results of the water level analysis using the derived Deer River water level time series without taking into account any additional RSLR. Figure 3 and Table 3 give the results of the cumulative frequency analysis, where water levels associated with various probabilities of exceedance are established. The operational 1% water level is the level that is on average exceeded for only 1% of the time annually.

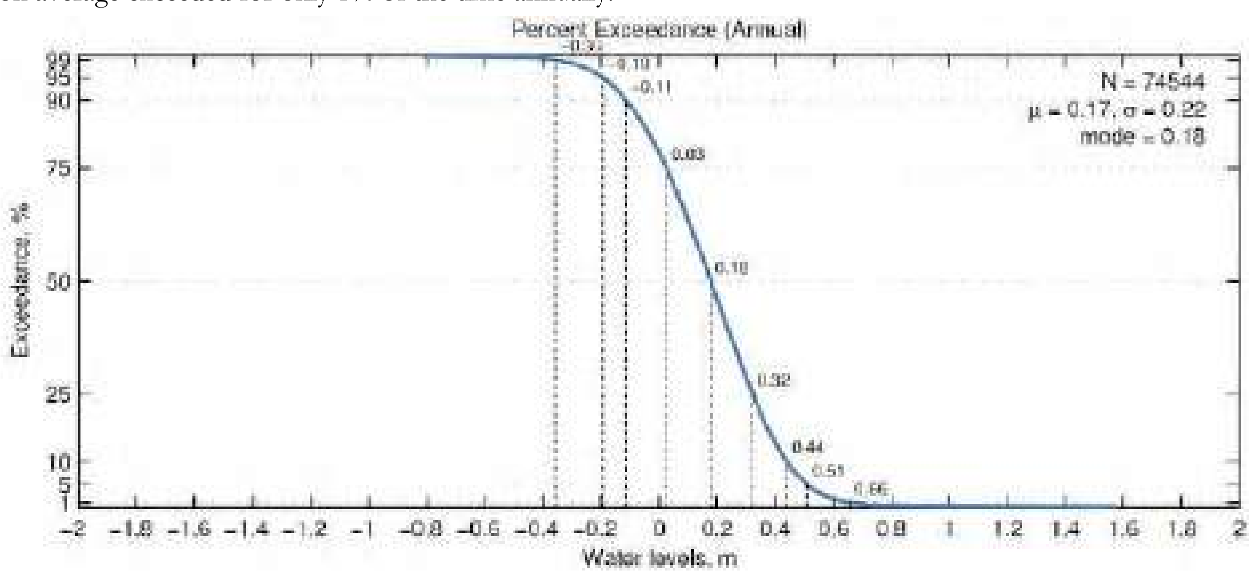


Figure 3: Cumulative exceedance plot for the interpolated Deer River water level time series for current conditions (without RSLR).

Table 3: Interpolated Deer River water levels associated with various probabilities of exceedance for current conditions (without RSLR).

Probability of Exceedance [%]	Water Level [m, NAVD88]	Water Level [ft, NAVD88]
99%	-0.36	**-1.2**
95%	-0.19	**-0.6**
90%	-0.11	**-0.4**
75%	0.03	**0.1**
50%	0.18	**0.6**
25%	0.32	**1.1**
10%	0.44	**1.4**
5%	0.51	**1.7**
1%	0.66	**2.2**

An extreme analysis was then performed to characterize the less-frequently occurring high water levels associated with various return periods. For this process, a peaks-over-threshold (POT) analysis returns the highest events in the time series that exceed a certain threshold value. These peaks were then analyzed with various candidate extreme value distributions fit using a linear least-squares regression (Goda, 2010). The candidate extreme value distribution that produced the best fit with the peak samples was used to estimate the extreme values corresponding to return periods from 1-year to 25-year. Figure 4 shows the extreme value distribution fit for current condition water levels, while Table 4 gives the return periods and corresponding water levels. Note that this method computed 50-year and 100-year return period levels, but the less than 20-year period of record can only be used to reliably extrapolate to a 25-year return period value. As such, the 50-year and 100-year values are not used for this study.

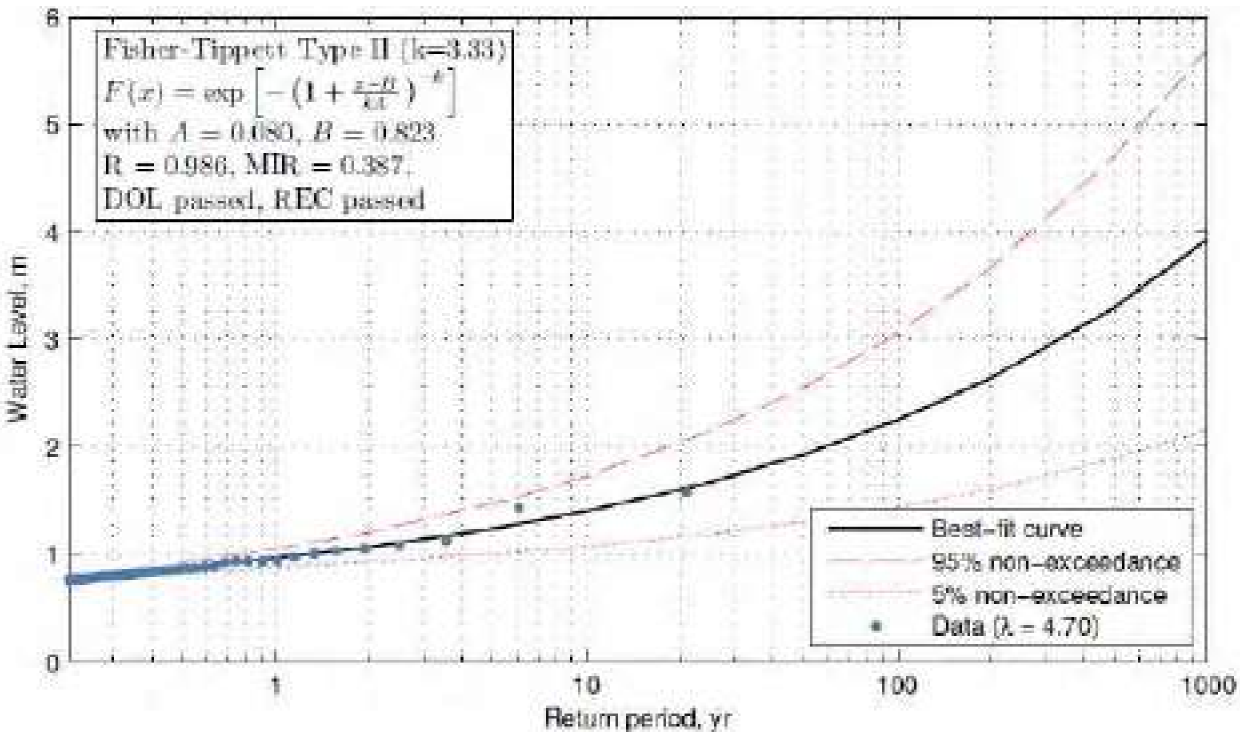


Figure 4: Extreme analysis of interpolated Deer River water levels for current conditions (without RSLR). The plot shows the peaks in the interpolated water level time series compared to the best fit Fisher-Tippett Type II extreme value distribution.

Table 4: Extreme interpolated Deer River water levels associated with various return periods for current conditions (without RSLR). Note that the computed 50- and 100-year return period values for this study are not used since the period of record is considered too short to reliably extrapolate beyond the 25-year return period level.

Return Period [yr]	Deer River Interp. Computed Water Level [m, NAVD88]	Deer River Interp. Computed Water Level [ft, NAVD88]
0.5	0.87	2.8
1	0.97	3.2
2	1.07	3.5
5	1.24	4.1
10	1.40	4.6
25	1.67	5.5
50	1.93	6.3
100	2.25	7.4

2.4. Relative Sea Level Rise

Strong evidence from coastal tide gauges and satellite altimetry data indicate that water levels along the coast of the United States are experiencing a long-term rise, likely due to anthropogenic causes driven by an increase in the emissions of greenhouse gasses. This rise in water levels is generally termed Eustatic Sea Level Rise (SLR), and projections of global mean sea level (GMSL) rise are available from sources such as the USACE and NOAA based on emissions guidelines termed Representative Concentration Pathways (RCPs) that are provided by the Intergovernmental Panel on Climate Change (IPCC). Further, changes in coastal water levels are influenced by many physical drivers including ocean temperatures, water salinity, atmospheric winds and pressures, and ocean currents. SLR is therefore regionally dependent on climate change induced modifications to these processes.

In order to provide regional SLR guidance in the United States, the U.S. Global Change Research Program (USGCRP), together with the National Ocean Council (NOC) put together the Sea Level Rise and Coastal Flood Hazard Scenarios and Tools Interagency Task Force (Task Force hereafter). Released in January 2017, "NOAA Technical Report NOS CO-OPS 083" (Sweet et al., 2017) provides an update to SLR projections for the United States coastline. The report provides an update to global mean sea level (GMSL) rise, and a 1° by 1° gridded regional SLR database which details varying regional SLR rates across the US coastline.

In addition to the previously mentioned regional processes that affect water levels, another element to changing water levels along the Gulf Coast is land subsidence likely due to the extraction of water and fossil fuels as well as consolidation of soils. When combined with Eustatic Sea Level Rise, subsidence leads to even higher rates of Relative Sea Level Rise. The Task Force RSLR guidelines already include regional subsidence rates within the 1° by 1° gridded projections.

Task Force SLR guidelines are based on the ICCP RCPs scenarios and the six (6) GMSL SLR scenarios. Further, a probability of exceedance is provided for each scenario. Using this probabilistic assessment, a risk-based approach can be used to guide SLR design criteria. For example, Table 5 shows the "Low" GMSL scenario provides a high probability of exceedance for all 3 RCP scenarios. Designing to this level of SLR therefore provides a high risk that the Deer River project would experience severe inundation that may put the project at greater risk by the end of the design life. On the other hand, the "Extreme" SMSL scenario is highly unlikely to be exceeded and should be reserved for infrastructure projects where failure would be catastrophic (such as a nuclear power plant).

For a project design that remains robust and sustainable at the end of the design life, the expected relative sea level rise (both eustatic SLR and subsidence) must be taken into account. For the Deer River Restoration project, Moffatt & Nichol explored the various global and regional projections for SLR and vertical land movement applicable to the project, concluding that the RCP 4.5 emissions scenario and an Intermediate GMSL scenario provide an acceptable amount of risk (3% probability of exceedance) while protecting against unacceptable water levels when considering a design life of 25 years. Based on this reasoning, the M&N design team recommends a design assuming a 0.79 ft (0.24 m) rise in water levels due to Relative Sea Level Rise should be used based on the assumed project life from 2020-2045.

Table 5 and Figure 5 provide graphic and tabular illustrations of the RSLR and Representative Concentration Pathways (RCP) scenarios provided by the Sweet et al. 2017 report.

Table 5: Relative Sea Level Rise between the years 2020 and 2045 (25-year project design life). Scenarios and Probability of Exceedance data extracted at Dauphin Island (Sweet et al., 2017).

Relative Sea Level Rise Scenarios and Probability of Exceedance (all RSLR values displayed in ft)			
GMSL Scenarios	**RCP 2.6** **(strong emissions mitigation by 2100)**	**RCP 4.5** **(moderate emissions mitigation by 2100)**	**RCP 8.5** **(no emissions mitigation by 2100)**
Low (0.3 m)	0.34 (94%)	0.44 (98%)	0.54 (100%)
Intermediate-Low (0.5 m)	0.48 (49%)	0.52 (73%)	0.61 (96%)
Intermediate (1.0 m)	0.74 (2%)	**0.79 (3%)**	0.87 (17%)
Intermediate-High (1.5 m)	0.97(0.4%)	1.08 (0.5%)	1.18 (1.3%)
High (2.0 m)	1.35 (0.1%)	1.44 (0.1%)	1.53 (0.3%)
Extreme (2.5 m)	1.41 (0.05%)	1.69 (0.05%)	1.82 (0.1%)

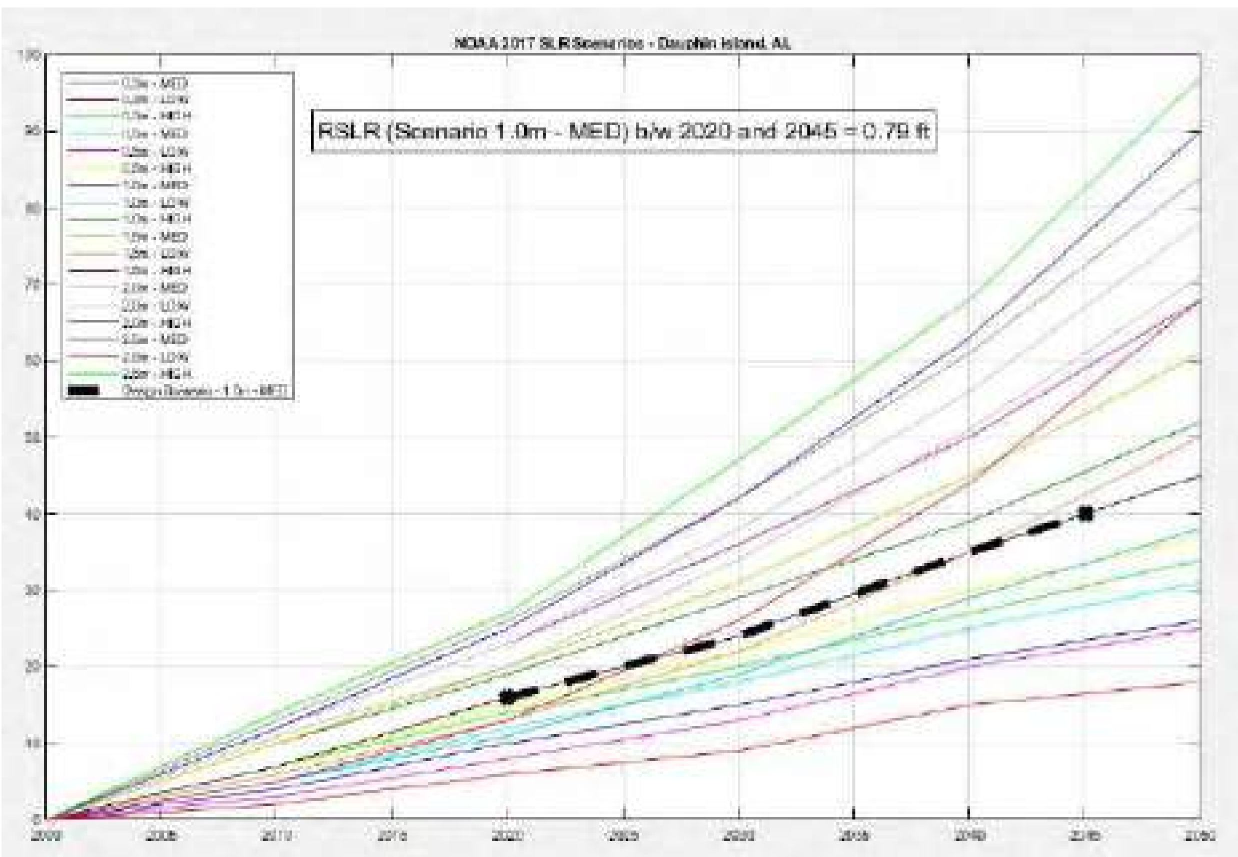


Figure 5: Relative Sea Level Rise Scenarios and Probability of Exceedance data extracted at Dauphin Island (Sweet et al., 2017). Black Dashed line indicates the 1.0m – MED scenario during the 25-year project design life from 2020 to 2045.

This study in particular takes the RSLR into account by re-analyzing the water level time series after adding 0.79 ft to the full record. Additionally, waves are computed based on the water levels + RSLR, so the increased wave height associated with deeper fetch conditions and less depth-limited breaking are captured.

2.5. Future with RSLR Results

The following section details the results of the water level analysis using the derived Deer River water level time series with the addition of 0.79 ft (0.24 m) RSLR. Figure 6 and Table 6 give the results of the cumulative frequency analysis, where water levels associated with various probabilities of exceedance are established. Figure 7 and Table 7 then give the results of the extreme analysis of the water levels, taking into account RSLR. Each of the operational and extreme water levels with RSLR is greater than the corresponding value without RSLR by an increment approximately equal to the imposed 0.79 ft (0.24 m) of relative sea level change.

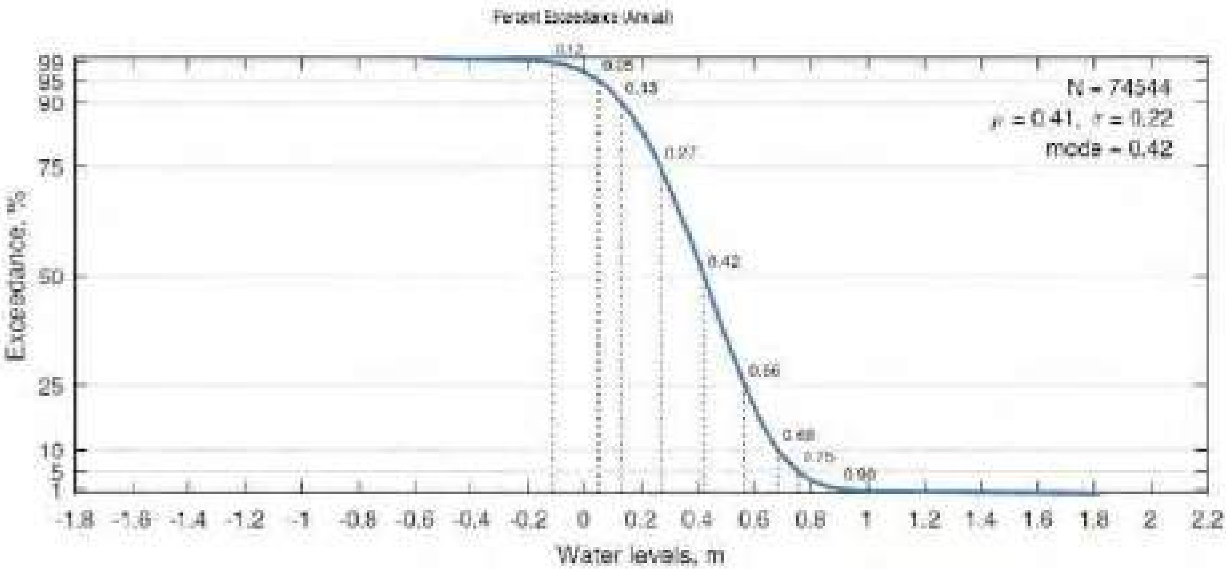


Figure 6: Cumulative exceedance plot for the interpolated Deer River water level time series for future conditions (with RSLR).

Table 6: Interpolated Deer River water levels associated with various probabilities of exceedance for future conditions (with RSLR).

Probability of Exceedance [%]	Water Level [m, NAVD88]	Water Level [ft, NAVD88]
99%	-0.12	**-0.4**
95%	0.05	**0.2**
90%	0.13	**0.4**
75%	0.27	**0.9**
50%	0.42	**1.4**
25%	0.56	**1.8**
10%	0.68	**2.2**
5%	0.75	**2.5**
1%	0.90	**3.0**

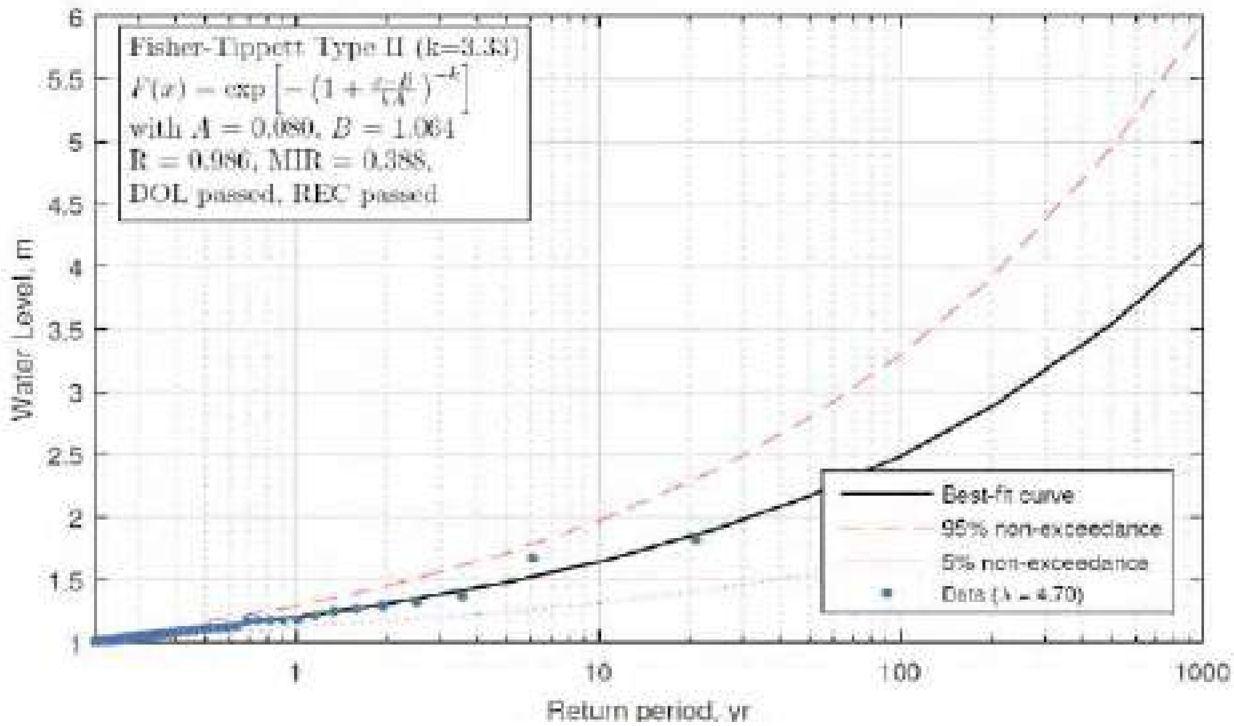


Figure 7: Extreme analysis of interpolated Deer River water levels for future conditions (with RSLR). The plot shows the peaks in the interpolated water level time series compared to the best fit Fisher-Tippett Type II extreme value distribution.

Table 7: Extreme interpolated Deer River water levels associated with various return periods for future conditions (with RSLR). Note that the computed 50- and 100-year return period values for this study are not used since the period of record is considered too short to reliably extrapolate beyond the 25-year return period level.

Return Period [yr]	Deer River Interpolated Water Level [m, NAVD88]	Deer River Interpolated Water Level [ft, NAVD88]
0.5	1.12	3.7
1	1.21	4.0
2	1.31	4.3
5	1.48	4.9
10	1.64	5.4
25	1.91	6.3
50	2.17	7.1
100	2.49	8.2

3. Regional Spectral Wave Modeling

3.1. Approach

With no available wave measurements near the project site, a wave model was developed to hindcast the long-term wave conditions at Deer River to aid in the project design.

A MIKE21-Spectral Wave (SW) model was developed to calculate the wind-wave generation within Mobile Bay and wave transformation processes as the waves propagate towards the Deer River shoreline. The software simulates both the growth of waves due to wind stress and the transformation of waves as they approach the nearshore environment due to shoaling, refraction, and diffraction. The model calculates both direction and frequency spectral wave parameters over a flexible mesh computational grid that allows for computationally-efficient high-resolution representation of the area of interest without imposing unnecessarily-high resolutions in model areas where it is not required.

Waves were simulated for a period from 2001-2019 for which model boundary conditions are available. Specific boundary conditions are detailed in subsequent sections. For wave conditions at the end of the project design life which are used as input into the design, waves are modeled using the water level record with the additional 0.79 ft (0.24 m) of RSLR.

3.2. MIKE21-SW Model Development

3.2.1. Model Domain

The model domain was chosen to encompass all of Mobile Bay, covering the full area over which wind-generated waves impacting the project site could be generated. Figure 8 shows the model domain, bathymetry, and the selected output point (sw135) used for subsequent wave analyses. Bathymetry data in the project vicinity and Mobile Bay for interpolation to the model grid was assembled from several sources. For all source datasets, the depth points were projected to the North American Datum of 1983 (NAD83), UTM 16N coordinate system and adjusted to reference the NAVD88 vertical datum. Input boundary conditions for the 2001-2019 hindcast included a time-varying water level and time-varying wind speed and direction forcings.

In general, model settings were chosen based on recommended values for sheltered locations given in the MIKE21-SW documentation (DHI, 2012), or adjusted to improve model calibration to measured wave data (see *Model Calibration*).

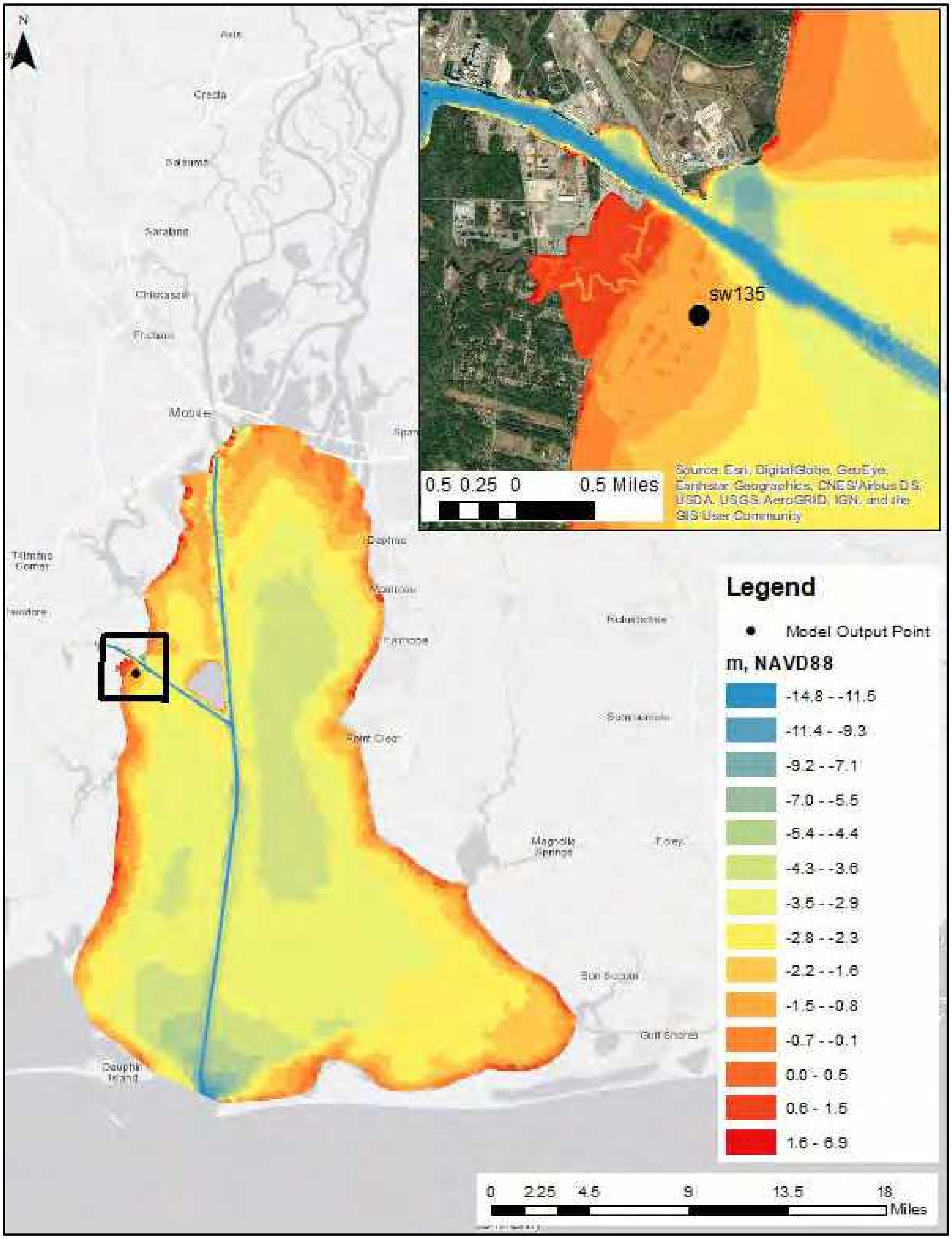


Figure 8: MIKE21-Spectral Wave (SW) model domain and bathymetry (in m, NAVD88) covering Mobile Bay and project vicinity (see inlay). The model output point (sw135) was used to characterize wave conditions at the project site.

3.2.2. Bathymetry

For most of the model domain, topographic and bathymetric data was obtained from the United States Geological Service (USGS) Coastal National Elevation Database Digital Elevation Model of Mobile Bay and vicinity[1]. More detailed bathymetry for the Mobile Bay and Theodore Ship navigation channels was obtained from USACE navigation surveys. Additionally, countywide LiDAR data from 2014 was used for wetland areas in the vicinity of the project site. Finally, project-specific topographic and nearshore bathymetric surveys for Deer River were commissioned, and data were incorporated into the model once they became available from Thompson Engineering, Inc.

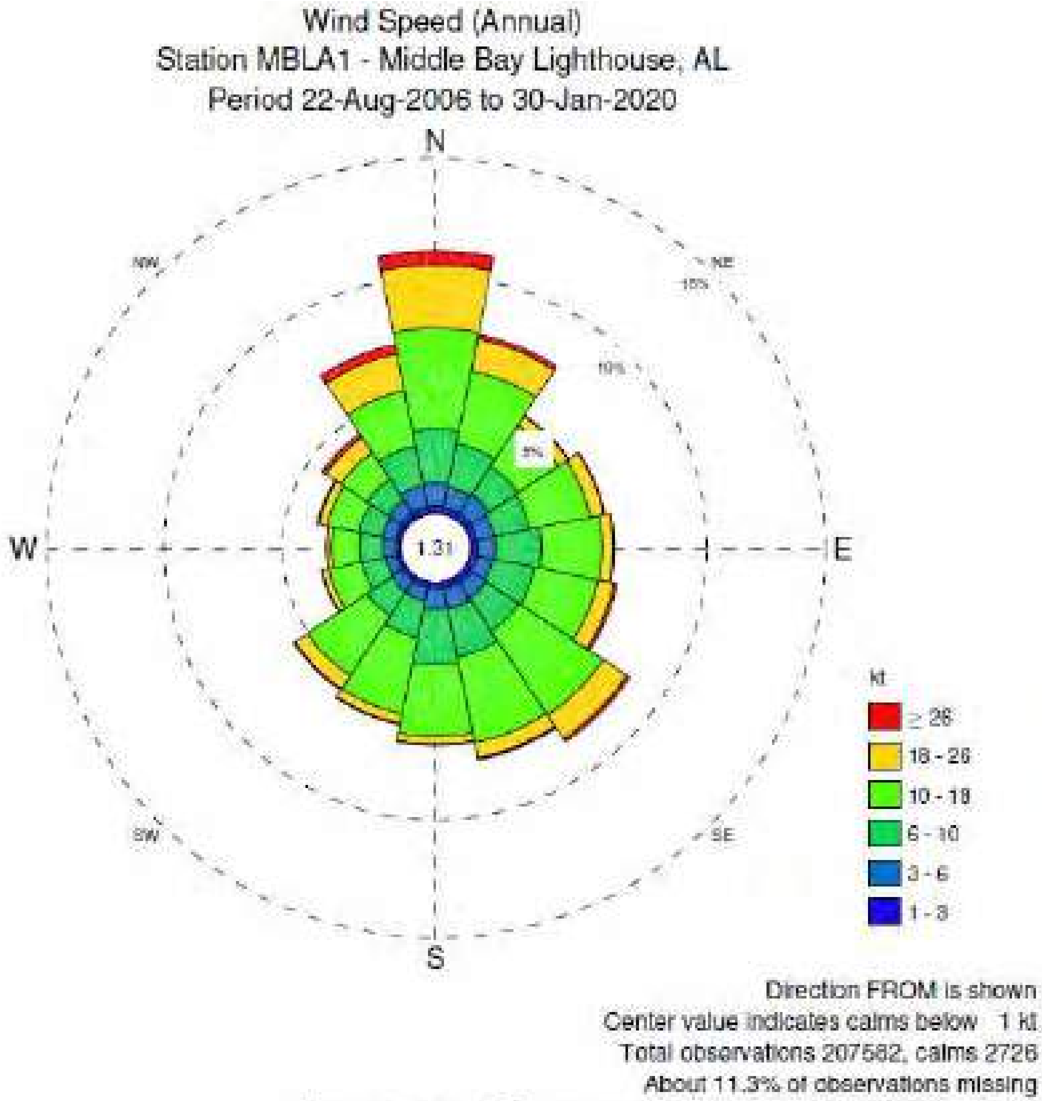
3.2.3. Boundary Conditions

Input boundary conditions for the 2001-2019 hindcast included a time-varying water level and time-varying wind speed and direction forcings. For each model run, the water level was assumed to be constant over the model domain. While in reality the water level varies across the modeled area due to various factors, the water level only affects the wave results in shallow areas near the shore where the waves interact with the bottom and to a small degree across the main wind-generating fetches, where deeper water produces slightly higher waves. The variation in water depth in the deeper potions of the model domain away from shorelines would not significantly affect the wave generation and propagation. However, the transformation processes of waves near Deer River are adequately represented because modeled water levels were defined based on the interpolated Deer River water level time series. The derived time series provides high surges for notable recent historic hurricanes, including Hurricanes Ivan, Katrina, and Nate. With these extreme events properly represented in the measurements and derived Deer River water levels, extreme values can be more reliably computed.

Additionally, wind speed and direction were also assumed to be constant over the full domain. With the relatively small modeled area, this is a valid assumption, and eliminated the complications of implementing spatially-varying wind fields. Wind over the model domain is represented using the measured wind speed and directions from the NDBC gage MBLA1, located at the Middle Bay Lighthouse. Gaps in Middle Bay Lighthouse (MBLA1) wind record were first filled with gage measurements from the Dauphin Island NOAA CO-OPS station (8735180) and then filled with gage measurements from the Coast Guard Sector NOAA CO-OPS station (8736897). These recorded wind measurements are representative of that generating waves over the exposed fetches impacting Deer River. Additional analyses were conducted comparing MBLA1 wind to the windspeed and directions measured at other nearby gages which confirm that there is little variability among the gages and that MBLA1, 8735180, and 8736897 are representative.

Wind speeds measured at Middle Bay Lighthouse (MBLA1) and Dauphin Island (8735180) wind gages are averaged over a 2-minute duration taken at hourly intervals from 2002-2020. For wind-wave generation models in semi-enclosed areas, the wind averaging interval is sometimes adjusted to larger durations that are more representative of the typical time required for waves to reach fetch-limited conditions. For this relatively small fetch area, the wind speeds were adjusted to 1-hour averages. Figure 9 gives the computed annual wind rose for the wind measurements at Middle Bay Light House (MBLA1).

[1] https://topotools.cr.usgs.gov/topobathy_viewer/dwndata.htm



Percentage of Occurrence

Wind Speed, kt	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Total	11.08	7.72	5.26	5.78	6.04	6.31	8.57	7.67	6.88	6.13	5.74	3.42	3.14	3.61	4.28	7.28	98.69
≥ 26	0.61	0.18				0.12	0.14								0.17	0.37	2.05
18–26	2.67	1.38	0.49	0.59	0.46	0.69	1.21	0.67	0.36	0.40	0.66	0.27	0.19	0.36	0.76	1.61	12.76
10–18	4.26	3.14	2.06	2.61	2.56	2.71	4.39	3.70	3.03	3.31	3.03	1.44	1.20	1.37	1.36	2.38	42.45
6–10	2.26	1.89	1.46	1.54	1.94	1.81	1.68	2.08	2.32	1.59	1.24	1.02	0.96	1.00	1.04	1.70	25.52
3–6	1.01	0.87	0.94	0.76	0.90	0.71	0.77	0.88	0.88	0.60	0.56	0.47	0.57	0.60	0.70	0.94	12.08
1–3	0.27	0.26	0.28	0.25	0.25	0.27	0.29	0.26	0.25	0.19	0.18	0.18	0.19	0.22	0.25	0.25	3.83

Figure 9: Annual wind rose of measured wind speed (in Knots) and direction at NDBC meteorological station MBLA1– Middle Bay Lighthouse, AL (2006-2020). See Figure 1 for station location.

Finally, an offshore wave boundary was evaluated so that any wave energy (swell) that propagates into Mobile Bay that could potentially reach the project site could be accounted for. Offshore wave conditions from a short distance seaward of Dauphin Island (station 73151) were taken from the USACE Wave Information Studies hindcast model[2].

The large majority of offshore waves in the area are of less than 1 m height with peak periods of between 4 and 6 seconds. To analyze the potential for offshore swell energy to reach the site, a series of model sensitivity runs were performed imposing offshore waves with significant wave heights of 1 m, peak periods of 4 and 6 seconds, and incident directions covering the full southeasterly to southwesterly exposed sector. Model results showed that the transformed swell waves at the project site never exceeded 0.39 inches (1.5 centimeter), representing negligible wave energy compared to that associated with wind-generated waves. Therefore, offshore swell was not incorporated as a boundary condition to model simulations.

3.2.4. Model Settings

The MIKE21-SW model was run in quasi-stationary mode utilizing the directionally decoupled parametric spectral formulation. This frequency spectrum was discretized using a directional spectrum of 16 directions over the full 360 degrees with no separation of wind sea and swell. Diffraction, wave breaking with a specified breaking parameter of 0.8, and bottom friction with a Nikuradse roughness of 0.04 m were included.

3.3. Model Calibration

The MIKE21-SW model was calibrated using wave measurements collected in Mobile Bay at NOAA's National Data Buoy Center (NDBC) station Middle Bay Lighthouse (MBLA1) during 2013 by the Dauphin Island Sea Lab (DISL). In general, model settings were chosen based on recommended values for sheltered locations given in the MIKE21-SW documentation (DHI, 2012), or adjusted based on engineering judgement to improve model calibration to measured wave data. Figure 10 provides the calibration results of significant wave height measured at Middle Bay Lighthouse compared to predicted model calculations for significant wave height. Similarly, Figure 11 provides the calibration results of peak wave period measured at Middle Bay Lighthouse compared to predicted model calculations for peak wave period. While modelled wave heights were consistently larger than measured values at this location, the patterns of variability throughout the year are very well represented, and slight overprediction was preferred to maintain conservatism in subsequent design tasks. Similarly, the modelled wave period is consistent with the longer-term (several days to weeks) patterns in the measured wave period data, though higher frequency oscillations in the measurements were generally not reproduced. Located in a more-exposed location that the project site, the gage could be impacted by long-period, low-amplitude swell entering Mobile Bay that would elevate measured peak periods. The project site is sheltered from offshore swell so is only impacted by wind-generated waves of lower peak period. Overall, the model matches measurements to a degree sufficient for determining the long-term wave climate at the project site, with a small positive bias in predicted wave heights to maintain design conservatism.

[2] http://wis.usace.army.mil/

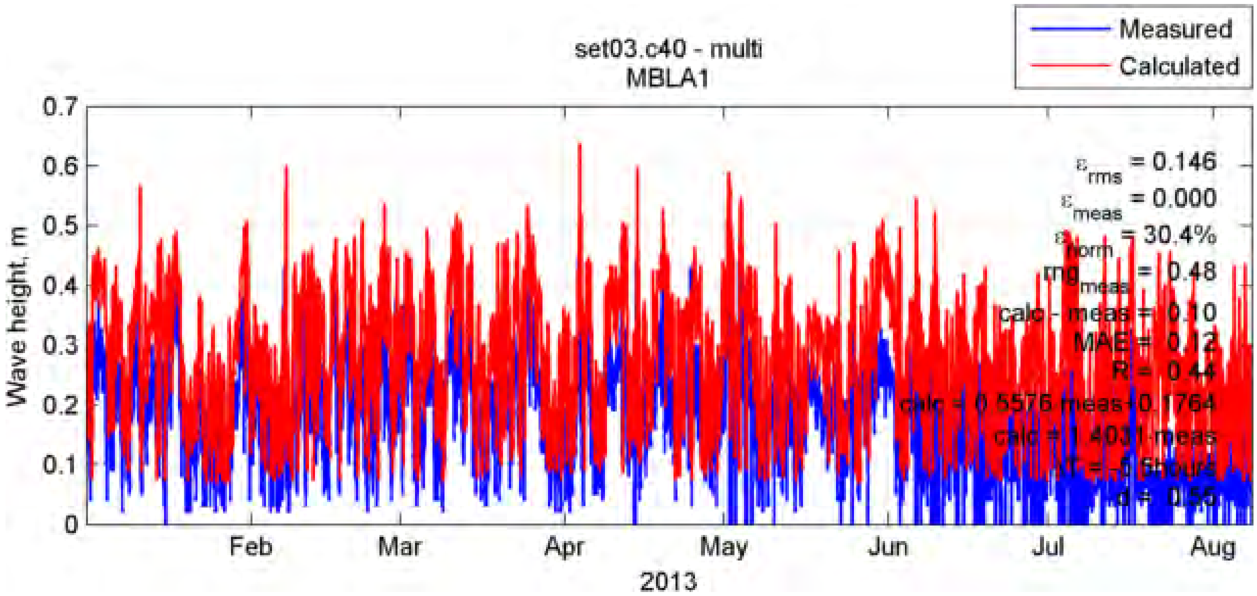


Figure 10: Model calibration results comparing measured and calculated significant (Hmo) wave heights

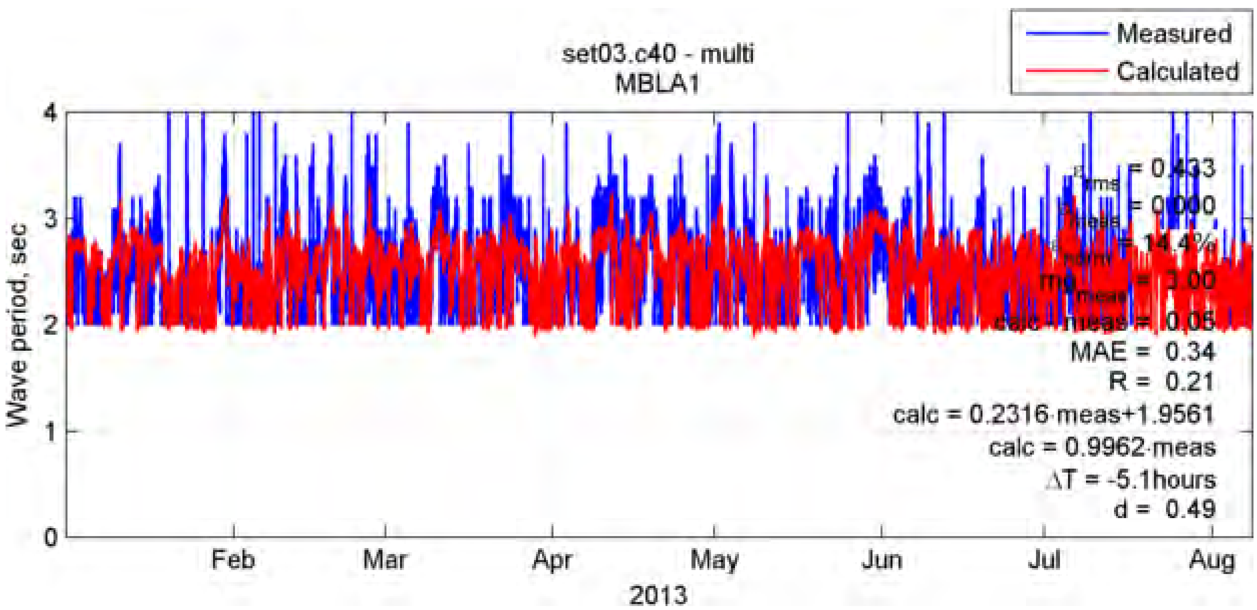


Figure 11: Model calibration results comparing measured and calculated peak wave periods

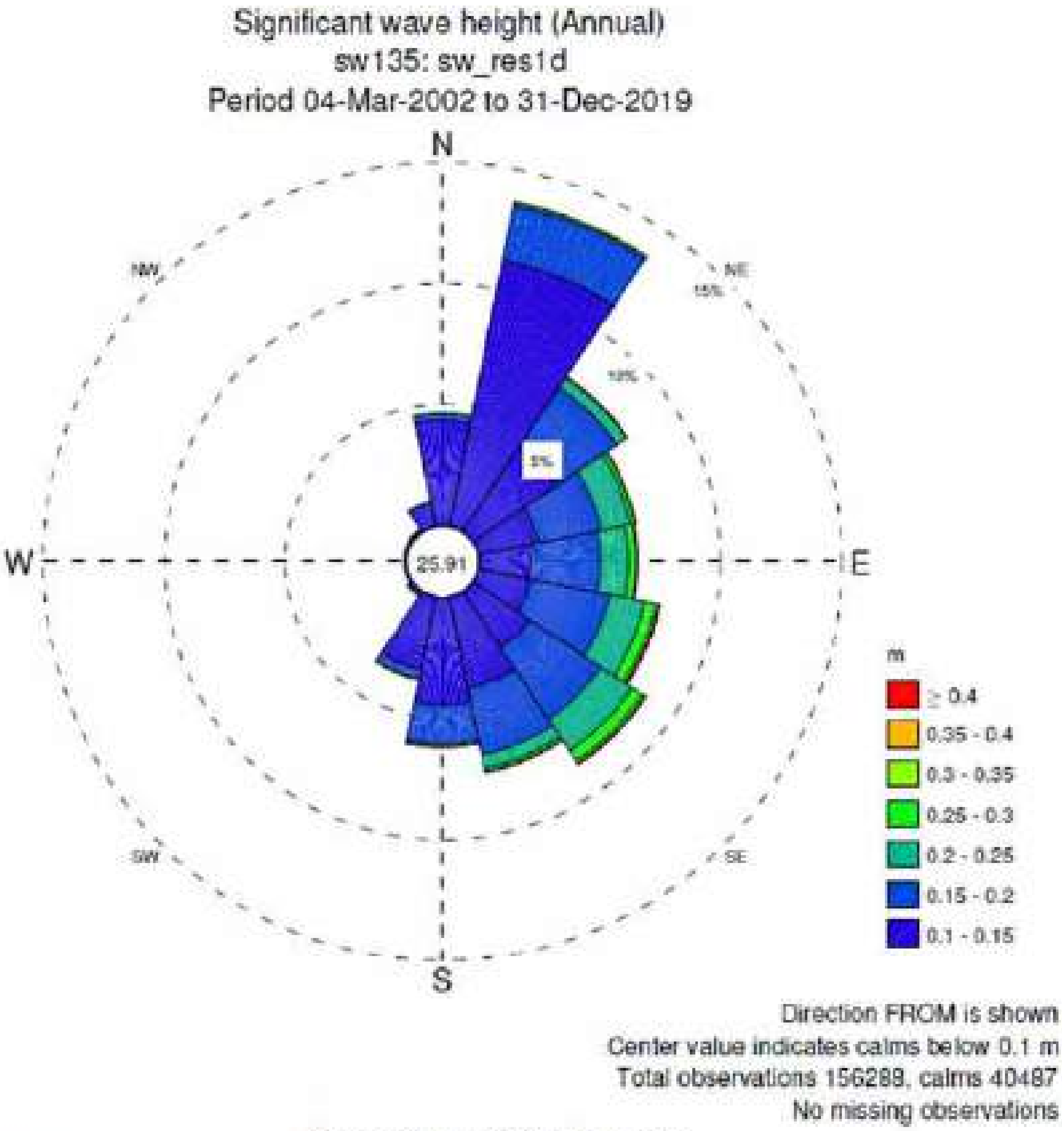
3.4. Future with RSLR Results

As discussed in the Approach Section, wave heights provided to inform project design are based on the future conditions at the end of the project design life. These are computed by running the wave model hindcast for the years 2001 through 2019, but with water levels that have been increased by the 0.79 ft (0.24 m) of RSLR at each timestep in the 18.5-year period. These timeseries of wave results at an output point representative of conditions impacting the project (sw135, see Figure 8) were then analyzed to derive operational and extreme conditions.

Table 8 shows the wave heights associated with varying exceedance probabilities for the future condition model run. A modeled wave rose at the project site is given in Figure 12. While the dominant northerly wind (Figure 9) translates into frequent northerly waves, southeasterly waves are larger and more-prominently contribute to the operational and extreme statistics due to the fetch lengths.

Table 8: Modeled significant wave heights at the project site associated with various probabilities of exceedance for future conditions (with RSLR).

Probability of Exceedance [%]	Significant Wave Height [m]	Significant Wave Height [ft]
75%	0.10	**0.33**
50%	0.13	**0.43**
25%	0.16	**0.52**
10%	0.20	**0.66**
5%	0.22	**0.72**
1%	0.28	**0.92**



Percentage of Occurrence

Significant wave height, m	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Total	4.62	[illegible]	7.68	6.63	6.56	7.66	8.61	7.42	6.19	3.48	0.23				0.14	1.00	74.09
0.4																	
0.35 – 0.4																	
0.3 – 0.35						0.12	0.13										0.34
0.25 – 0.3				0.18	0.29	0.46	0.50	0.14									1.64
0.2 – 0.25		0.13	0.48	1.24	1.17	1.46	1.58	0.58	0.11								6.81
0.15 – 0.2	0.26	2.32	2.82	2.65	2.90	3.32	3.58	2.82	1.60	0.38							22.71
0.1 – 0.15	4.34	[illegible]	4.34	2.53	2.13	2.21	2.81	3.86	4.45	3.06	0.19				0.13	0.97	[illegible]

Figure 12: Annual wave rose of hindcast significant wave heights at the project site.

Table 9 gives the computed extreme values for the future conditions (with RSLR) run, with the best-fit extreme value distribution shown in Figure 13.

Table 9: Extreme project site significant wave heights associated with various return periods for future conditions (with RSLR). Note that the computed 50- and 100-year return period values for this study are not used since the period of record is considered too short to reliably extrapolate beyond the 25-year return period level.

Return Period [yr]	Significant Wave Height [m]	Significant Wave Height [ft]
0.5	0.33	**1.08**
1	0.37	**1.21**
2	0.42	**1.38**
5	0.49	**1.61**
10	0.56	**1.84**
25	0.65	**2.13**
50	0.73	**2.40**
100	0.81	**2.66**

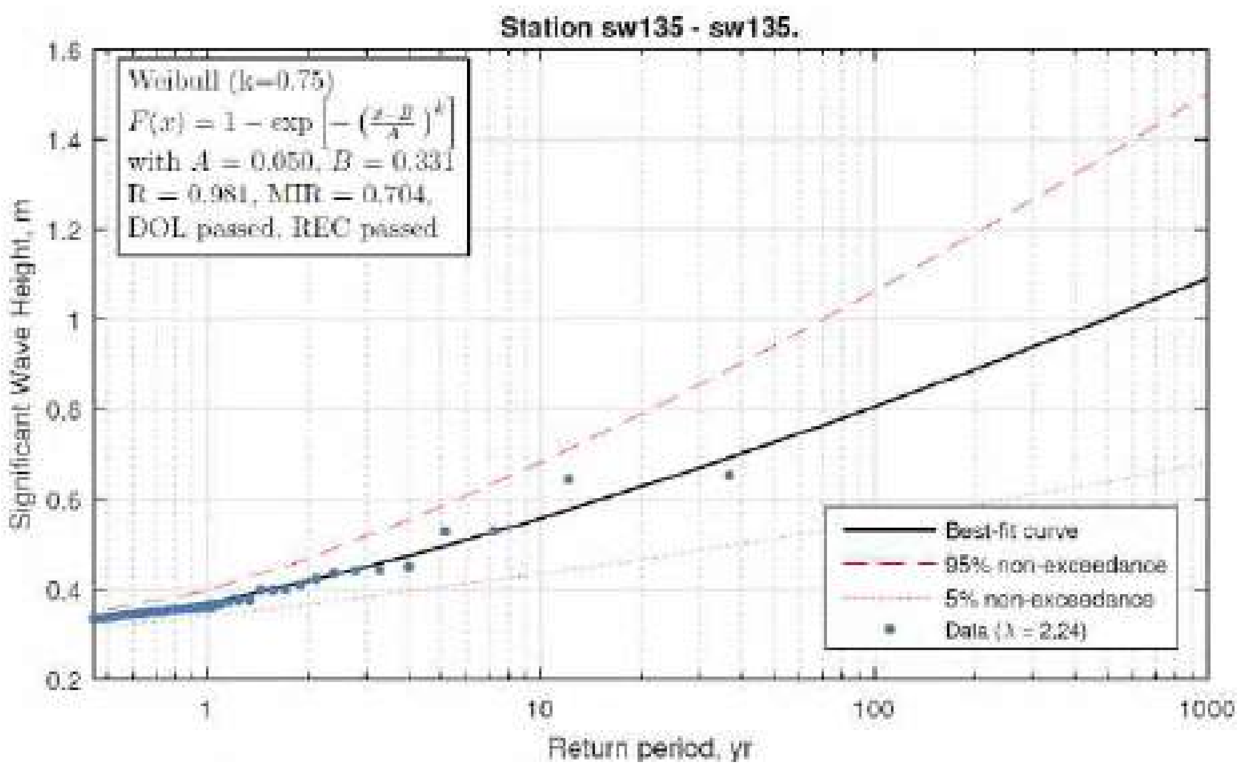


Figure 13: Extreme analysis of significant wave heights at the project site for future conditions (with RSLR). The plot shows the peaks in the modeled wave height time series compared to the best fit Weibull extreme value distribution.

3.5. Design Environmental Scenarios

The previous sections detailed the computation of operational and extreme levels for both water levels and wave heights at the project site. To help determine the degree to which varying breakwater alternatives will protect the restored shoreline, a series of environmental scenarios are developed from analysis results.

While wave height is the most significant variable when determining wave-structure interactions, the peak wave period has some influence on the resulting values of transmission and breakwater sizing. Since the previous statistical analyses were only performed on wave height, an associated peak wave period must be associated with each statistical level. The most straightforward approach is to examine the correlation between modeled Hs vs. Tp values to develop a functional relationship. Figure 14 shows the Tp vs. Hs for all modeled time periods at the project site. Most relevant to the design are the high wave heights that have periods more typical of steep waves generated by wind within Mobile Bay. A linear trend was identified for these higher wave height, wind-generated waves and used to compute an associated peak wave period for each operational and extreme significant wave height. Typical of smaller, semi-enclosed basins, the associated wave periods show little variability and range from 2 to 3 seconds.

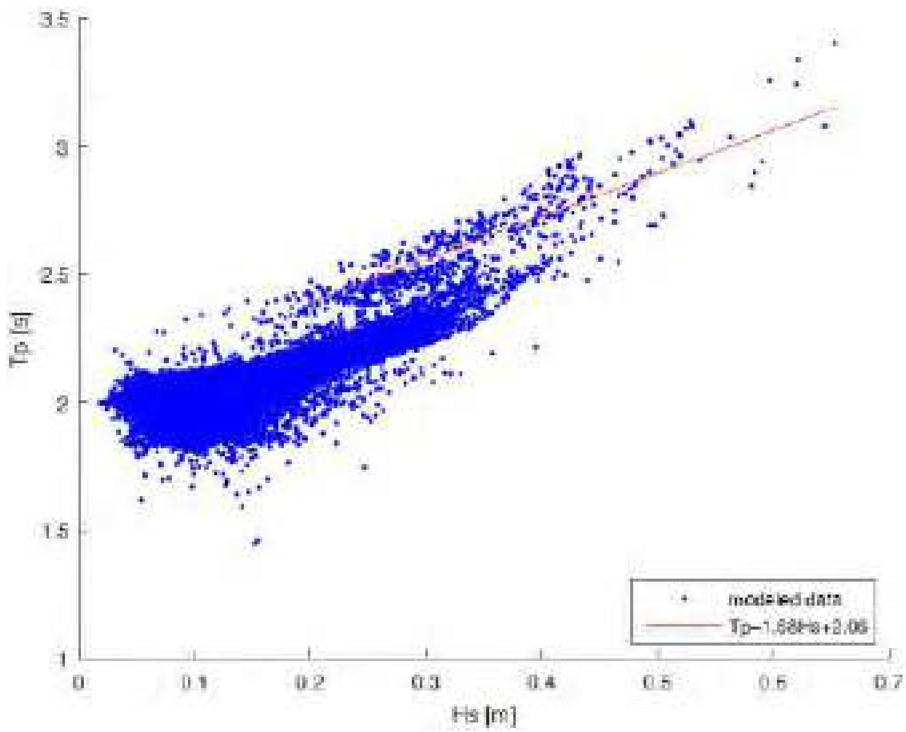


Figure 14: Plot of modeled peak wave period vs. significant wave height at the project site, along with a linear best fit limited to the higher waves where wind-generated peak periods are dominant.

Along the U.S. Gulf Coast, the highest waves almost always occur coincidently with the highest water levels when tropical systems produce large storm surges and high wind speeds. Because high wind-generated waves and high water levels are produced by the same meteorological events and show particularly strong correlation, it is a very appropriate assumption to apply water levels and wave heights of the same statistical level (exceedance probability or recurrence interval) simultaneously for design purposes. For use in subsequent shoreline protection alternative designs, the operational and extreme water levels and wave heights of the same probability are combined into 10 environmental scenarios, detailed in Table 10 below.

Table 10: Operational and extreme environmental scenarios used in the breakwater coastal engineering and design. Each scenario includes a water level, significant wave height, and associated peak wave period.

Type	Probability of Exceedance [%]	Return Period [yr]	Water Level [ft, NAVD88]	Significant Wave Height [ft, NAVD88]	Peak Wave Period [s]
Operational	50%	-	1.4	0.4	2.3
Operational	25%	-	1.8	0.5	2.3
Operational	10%	-	2.2	0.7	2.4
Operational	5%	-	2.5	0.7	2.4
Operational	1%	-	3.0	0.9	2.5
Extreme	-	1	4.0	1.2	2.6
Extreme	-	2	4.4	1.4	2.7
Extreme	-	5	5.0	1.6	2.8
Extreme	-	10	5.5	1.8	2.9
Extreme	-	25	6.2	2.1	3.0

4. Breakwater Wave Transmission Analysis

4.1. Introduction

With the design environmental scenarios established based on the previously detailed water level analysis and wave modeling study, the information on operational and extreme conditions at the project site and geotechnical conditions of the project site can be used to evaluate important breakwater technologies and design parameters.

The primary function of the breakwater feature for this project is to reduce the wave energy impacting the Deer River shoreline to a degree that the restored shoreline can provide the desired ecological benefits. Greater protection is achieved with a more massive structure; however, the level of protection must be balanced with cost, aesthetic, and geotechnical and ecological considerations that could favor a smaller structure. The following sections detail an approach that establishes an approximate wave height threshold for marsh edge erosion and computes wave transmission for various breakwater dimensions and designs of a traditional rubble mound structure as well as the Reefmaker Ecosystem Wave Attenuator© (hereafter referred to as the Reefmaker system) such that the marsh erosion threshold is not exceeded.

4.2. Marsh Erosion Thresholds

Previous work has been performed in the project vicinity examining the wave climates associated with the presence or absence of stable marsh shorelines. Generally, marsh shorelines occur where exposed to lower wave heights, while shorelines with higher wave heights show eroding marsh or lack vegetation completely. Figure 15 from Roland and Douglass (2005) shows the computed thresholds for the presence or absence of spartina alterniflora marshes at the shorelines in Coastal Alabama (Roland & Douglass, 2005).

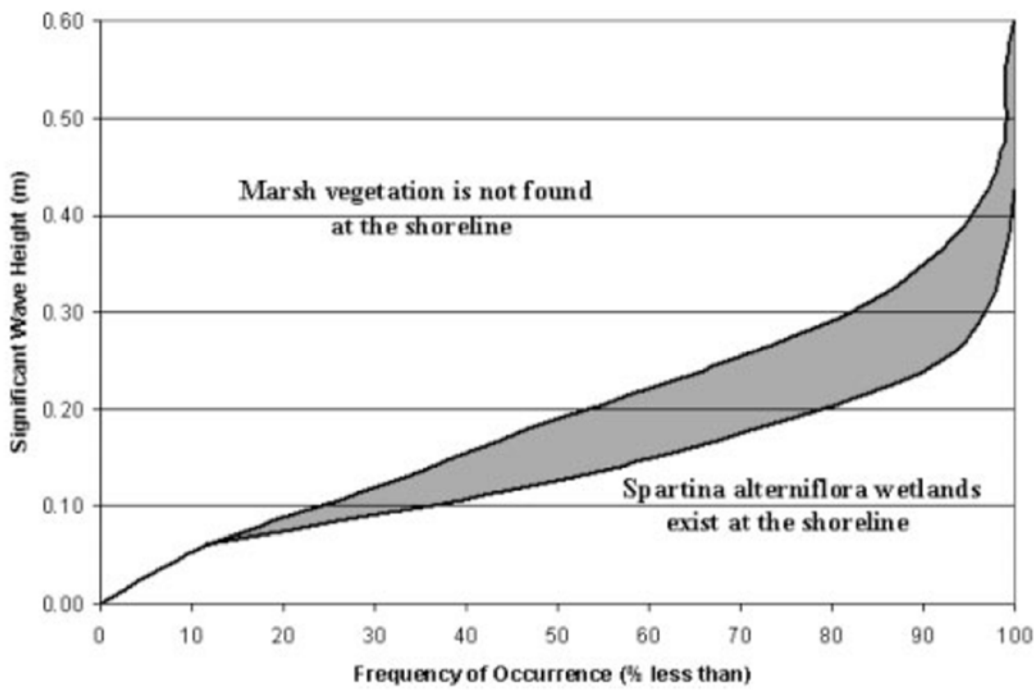


Figure 15: Threshold wave cumulative frequencies for the presences or absence of Spartina alterniflora wetlands in coastal Alabama, from Roland and Douglass (2005).

Other recent work has focused on modeling the response of a marsh shoreline to varying wave conditions. The model of Mariotti and Fagherazzi correlates the rate of marsh edge retreat with the wave power over a certain threshold value for stability, ranging from 3 to 15 watts/meter (Mariotti & Fagherazzi, 2010). Additional work by Trosclair (2013) in Lake Borgne, Louisiana used the methods of Mariotti and Fagherazzi to model edge erosion during cold fronts (Trosclair, 2013). In this work, the critical wave power of 15 watts/meter is computed to correspond to a significant wave height of approximately 15 cm (or 0.5 ft). Higher wave heights produce more wave power, so wave heights above this threshold would erode the marsh edge at a rate proportional to the difference over the threshold value. If the breakwater is sized such that the transmitted wave height for a particular environmental scenario is less than approximately 0.5 ft, then it is assumed that the marsh shoreline will not be significantly damaged.

The shoreline at Deer River is composed of marsh vegetation and sandy scarps, though there has been significant shoreline retreat over the past several decades. Erosional scarps are prevalent, and continued retreat is expected to occur without intervention.

4.3. Wave Transmission

4.3.1. Modeling wave transmission across breakwaters

Wave transmission across breakwaters is difficult to model numerically because of the complex physics of waves shoaling, breaking, traveling across the crest, and reforming in the lee. There is also a contribution of transmission through the pores of structures. The most reliable method of determining transmission is physical modeling in a laboratory, which is outside the scope of this project. Direct numerical modeling of these processes is possible, but required computational time is high on such a large scale (i.e. where the spatial domain is larger than a few wavelengths).

4.3.2. Empirical prediction formulae for wave transmission (rubble mound breakwaters)

Extensive physical modeling of rubble mound breakwater structures has been conducted over the past few decades and data have been collated, analyzed, and used to produce empirical prediction formulae. These formulae are derived from the pool of data and account for the influence of the major parameters of the incident wave conditions and breakwater geometry. However, similar efforts to physically model, collate, and analyze data to produce empirical prediction formulae for newly emerging technologies such as the Reefmaker system have not been conducted.

The current authoritative guidance on wave transmission over and/or past a structure is the recently revised *EurOtop – Manual on wave overtopping of sea defenses and related structures (Van der Meer et al., 2017),* supported jointly by the U.K. Environment Agency and Rijkswaterstaat, the Netherlands Expertise Network on Flood Protection. Since its conception in 1999, guidance by EurOtop has been incorporated into worldwide engineering publications related to breakwater design; see CIRIA / CUR / CETMEF Rock Manual (2007), British Standard 6349 (2000), and US Army Corps Coastal Engineering Manual.

The equation for predicting wave transmission across rubble mound structures is given below, taken directly from Chapter 4.2.5 of EurOtop. This equation is applicable to narrow rubble mound structures, where crest breadth B / wave height H_{m0} < 10, and is valid for negative freeboards (i.e. when the structure is submerged). Indeed, many tests were conducted specifically to investigate overtopping and transmission at low to negative freeboards. This equation is the exact same as Equation 5.66 in the Rock Manual. It is noted that porosity is not included as a variable in the equation. This is because the equation is applicable specifically to rubble mounds and the typical range of porosity within this class of structure does not change enough to exert significant influence on transmission.

$$K_t = -0.4\frac{R_c}{H_{m0}} + 0.64\left(\frac{B}{H_{m0}}\right)^{-0.31} \times \left(1 - \exp(-0.5\xi_{op})\right)\ for\ 0.075 \le K_t \le 0.8$$

$where,$
$K_t = tranmission\ coefficient$
$R_c = crest\ freeboard\ [m]$
$H_{m0} = significant\ wave\ height, spectrally\ derived\ [m]$
$B = crest\ breadth$
$\xi_{op} = surf\ similarity\ parameter$

4.3.3. Empirical prediction formulae for wave transmission (Reefmaker system)

Unlike for low-crested rubble-mound structures, the Reefmaker system has not been extensively studied in scale model or prototype situations such that there are robust, reliable empirical equations to predict wave transmission. Instead, the system has been installed for several projects, and some information on wave transmission has been provided by the manufacturer. In particular, some limited wave transmission performance data for the Reefmaker system is available for the Shark Island, LA demonstration project (Jadhav, 2018) and the Brunswick Town/ Fort Anderson shoreline protection project in North Caroline (Walter, 2019). The wave transmission monitoring data as a function of the structure relative crest elevation is plotted in Figure 16.

While these few data points are inadequate to develop a robust empirical wave transmission equation, they were compared to a widely used "Rule of Thumb"-type equation provided in the Rock Manual (CIRIA; CUR; CETMEF, 2007) for wave transmission over low-crested structures that is only dependent on the relative crest elevation. Figure 16 shows that this "Rule of Thumb" equation somewhat resembles the sparse measured data for the Reefmaker system; however, the measured transmission is greater than predicted by the "Rule of Thumb" equation for positive relative crest elevations (i.e. where the structure crest is higher than the water surface), suggesting that there is a minimum amount of wave energy that is always transmitted through the relatively-porous structure regardless of relative crest height. To account for this effect and ensure any predicted transmission results are conservative, the "Rule of Thumb" equation was modified to better match the limited available data (see Figure 16). This modified relationship was subsequently used to predict the transmitted wave heights for the Reefmaker system for a range of crest elevations under the various design environmental scenarios.

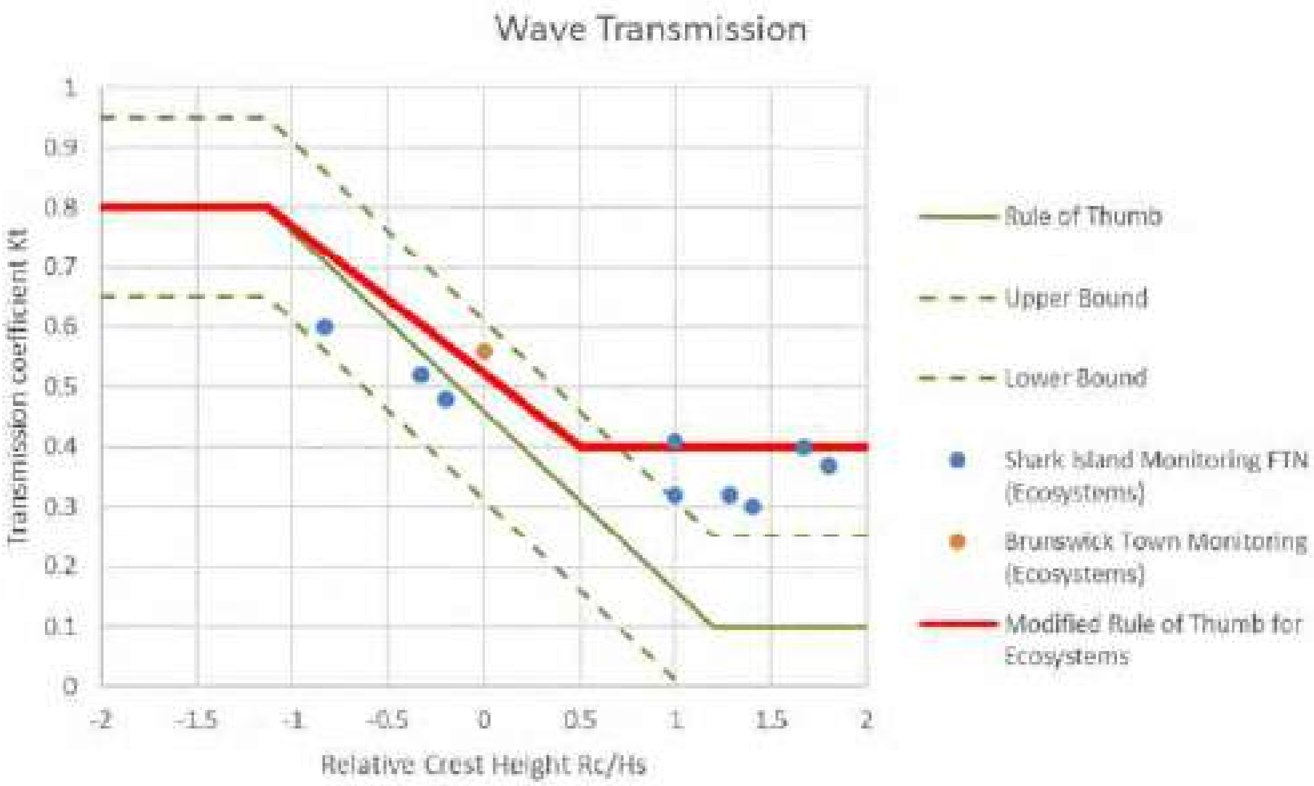


Figure 16: Wave transmission formula developed for the Reefmaker system.

4.4. Transmission Results - Rubble Mound Structures

Using the equation for transmission over low-crested, rubble-mound structures detailed above, the transmission coefficient and resulting transmitted wave height were computed for all design environmental scenarios for varying breakwater crest dimensions. The crest elevation exerts the most influence on transmission, while crest breadth and slope are less influential and chosen more for constructability. A typical crest breadth of 5 ft with a foreslope of 1:3 (vertical:horizontal) is assumed, and the crest elevation is varied for +1 ft, +1.5 ft, +2.0 ft, +2.5 ft, +3.0 ft, +4.0 ft, and +5.0 ft (all relative to NAVD88). Note that these alternatives are not all necessarily considered for design but are useful in demonstrating the influence of crest level on the level of protection.

Table 11 shows the results of the wave transmission calculations for the various environmental scenarios and rubble mound breakwater crest elevation alternatives. Where transmitted wave heights are greater than 0.5 ft, it is expected that the protected restored shoreline feature could experience erosion.

Table 11: Rubble mound wave transmission results for various operational and extreme environmental scenarios and breakwater crest elevation alternatives. Crest breadth and foreslope were held constant at 5 ft and 1:3 (Horizontal:vertical), respectively.

Type	Prob. of Exceedance [%]	Return Period [yr]	Water Level [ft, NAVD88]	Significant Wave Height [ft, NAVD88]	Crest Elevation [ft, NAVD88]	1.0	1.5	2.0	2.5	3.0	4.0	5.0
					Peak Wave Period [s]	Transmitted Wave Height [ft]						
Operational	50%	-	1.4	0.4	2.3	0.25	0.05	-	-	-	-	-
Operational	25%	-	1.8	0.5	2.3	0.46	0.26	0.06	-	-	-	-
Operational	10%	-	2.2	0.7	2.4	0.65	0.45	0.26	0.06	-	-	-
Operational	5%	-	2.5	0.7	2.4	0.72	0.56	0.36	0.17	-	-	-
Operational	1%	-	3.0	0.9	2.5	0.92	0.81	0.61	0.42	0.21	-	-
Extreme	-	1	4.0	1.2	2.6	1.21	1.21	1.10	0.90	0.69	0.28	-
Extreme	-	2	4.4	1.4	2.7	1.38	1.38	1.31	1.12	0.91	0.50	0.11
Extreme	-	5	5.0	1.6	2.8	1.61	1.61	1.61	1.42	1.21	0.80	0.41
Extreme	-	10	5.5	1.8	2.9	1.84	1.84	1.84	1.70	1.49	1.08	0.69
Extreme	-	25	6.2	2.1	3.0	2.13	2.13	2.13	2.07	1.86	1.46	1.06

4.5. Transmission Results - Reefmaker System

Using the formula described previously for wave transmission over and through Reefmaker system, the transmission coefficient and resulting transmitted wave height were computed for all design environmental scenarios for varying breakwater crest elevations. As with the low-crested rubble-mound structure alternative, typical crest elevations were assumed for +1 ft, +1.5 ft, +2.0 ft, +2.5 ft, +3.0 ft, +4.0 ft, and +5.0 ft (all relative to NAVD88). Note that these alternatives are not all necessarily considered for design but are useful in demonstrating the influence of crest level on the level of protection.

Table 12 shows the results of the wave transmission calculations for the various environmental scenarios and the Reefmaker system breakwater crest elevation alternatives. Where transmitted wave heights are greater than 0.5 ft, it is expected that the protected shoreline restoration could experience erosion.

Table 12: Reefmaker Ecosystem: Wave Attenuator© wave transmission results for various operational and extreme environmental scenarios and breakwater crest elevation alternatives.

Type	Prob. of Exceedance [%]	Return Period [yr]	Water Level [ft, NAVD88]	Significant Wave Height [ft, NAVD88]	Crest Elevation [ft,NAVD88]	1.0	1.5	2.0	2.5	3.0	4.0	5.0
					Peak Wave Period [s]	Transmitted Wave Height [ft]						
Operational	50%	-	1.4	0.4	2.3	0.32	0.20	0.17	0.17	0.17	0.17	0.17
Operational	25%	-	1.8	0.5	2.3	0.42	0.36	0.24	0.21	0.21	0.21	0.21
Operational	10%	-	2.2	0.7	2.4	0.52	0.52	0.41	0.29	0.26	0.26	0.26
Operational	5%	-	2.5	0.7	2.4	0.58	0.58	0.50	0.38	0.29	0.29	0.29
Operational	1%	-	3.0	0.9	2.5	0.73	0.73	0.72	0.60	0.47	0.37	0.37
Extreme	-	1	4.0	1.2	2.6	0.97	0.97	0.97	0.97	0.88	0.63	0.49
Extreme	-	2	4.4	1.4	2.7	1.10	1.10	1.10	1.10	1.07	0.82	0.58
Extreme	-	5	5.0	1.6	2.8	1.29	1.29	1.29	1.29	1.29	1.08	0.84
Extreme	-	10	5.5	1.8	2.9	1.47	1.47	1.47	1.47	1.47	1.33	1.09
Extreme	-	25	6.2	2.1	3.0	1.71	1.71	1.71	1.71	1.71	1.66	1.42

4.6. Breakwater Crest Elevation Alternatives

Based on wave and water level conditions at the project site, a traditional rubble-mound, low-crested breakwater with a crest width of 5 ft and 3:1 (H:V) slope would need to be built to a design crest elevation of approximately +2.5 ft NAVD88 to achieve transmitted waves of less than 0.5 ft for the operational, 1% exceedance conditions (waves expected to be exceeded for 1% of a typical year). Conversely, a low-crested breakwater constructed using the Reefmaker system would need be built to a design crest elevation of +3.0 ft NAVD88 to achieve similar wave transmission results and associated level of protection against significant erosion of the restored shoreline.

5. Local Boussinesq Wave Modeling

5.1. Introduction

The preliminary design of the Deer River restoration includes segmented, Reefmaker-type breakwaters as the shoreline protection feature. The gaps in the breakwaters, often referred to as "fish gaps", enable tidal exchange and nekton access to the newly created marsh edge and tidal creek habitat behind the structures. Gaps are placed approximately every 700 ft along the shoreline protection feature and are sized to be 5 ft wide. Short (approximately 20 ft long) segments are placed in an overlapping configuration in front of the gaps such that the gaps do not expose the protected side directly to wave attack. Still, the process of diffraction can enable wave energy penetration behind the breakwater, which would be superimposed with the wave energy transmitted over and through the breakwater crest (see section 4).

Additional modeling was performed to simulate the wave-by-wave propagation in the shoreline protection feature vicinity conditions in the MIKE21-Boussinesq (BW) Wave Model. This model was used for this because of its accurate diffraction, reflection and transmission formulation, which is key to simulating waves penetrating behind segmented breakwaters. The following sections detail the development of this local, high-resolution wave model as well as the results for the 1-yr return period wave conditions (see section 3 for detailed description of how these were derived).

5.2. Model Development

The grid extents of the local MIKE21-BW model were chosen to encompass the local area of interest around the project, enabling nearshore wave conditions computed using the MIKE21-SW model to be propagated to the shoreline protection features with computational efficiency. The grid, shown below in Figure 17, covers an area approximately 2400 m by 1500 m with a 0.5 m by 0.5 m resolution. The bathymetry sources are identical to those used in the regional MIKE21-SW modeling (see section 3).

The Reefmaker breakwaters are designed as low-crested structures that will be inundated during the higher operational and more-frequently occurring extreme events. When the elevated water level is near or above the breakwater crest, additional wave energy can enter the protected area through transmission of waves over the breakwater, in addition to the energy transmitting through the porous Reefmaker structure. The process of waves passing over a breakwater, breaking, and entering the protected area is complex and difficult to model numerically. Instead, empirically-derived analytical equations were used to estimate the transmission coefficient for wave passing over and through the Reefmaker breakwater alternative (see section 4). The breakwater was then implemented in the MIKE21-BW as an area of reduced porosity corresponding to the planform limits of the Reefmaker alternative breakwater configuration. This porosity value was adjusted so that the computed transmission in the model matched the analytically-determined value.

To investigate how the processes of diffraction and transmission influence the waves reaching the protected areas, a single environmental scenario corresponding to the 1% exceedance event conditions was simulated with the Reefmaker alternative with crest elevation of +3.0 ft, NAVD88. The 1% exceedance conditions correspond to a water level of approximately +3.0 ft with nearshore significant wave heights of approximately 1.0 ft. A +3.0 ft crest elevation transmits waves of 0.5 ft significant wave height, indicating a computed transmission coefficient (K_t) of approximately 0.5 (see Table 12). Note that the wave conditions at the nearshore boundary wave generation lines were adjusted such that the waves impacting the breakwater alignment were approximately equal to the 1% exceedance conditions.

The BW model's ability to match this target K_t value was confirmed by comparing the significant wave heights directly in front of and behind the breakwater for the 1-yr RP wave conditions imposed at 135° (which is approximately head-on to the shoreline and breakwater alignment). For this initial case, the breakwater was implemented as a continuous feature with no fish gaps so that the wave heights in the protected zone would only be produced by transmission.

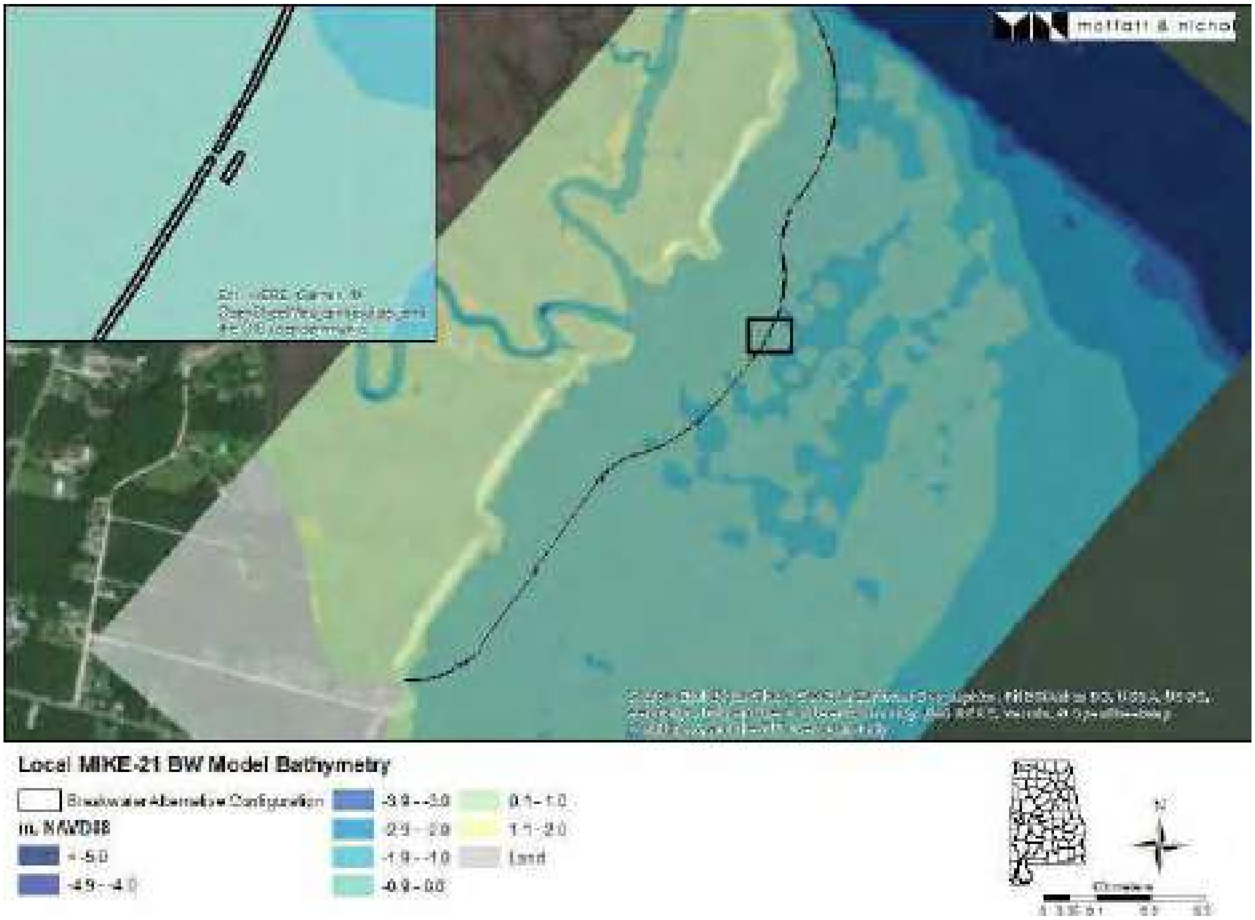


Figure 17: Local MIKE21-Boussinesq Wave (BW) model domain and bathymetry (in m, NAVD88) covering the direct project vicinity along with an outline of the proposed Reefmaker alternative breakwater configuration.

Figure 18 and Figure 19 show the instantaneous wave field and resulting significant wave heights for this transmission confirmation simulation. Analysis of the wave heights offshore and landward of the continuous breakwater will determine if the transmission coefficient goal is being achieved. Figure 20 gives the significant wave heights in front of and behind the breakwater for 1-yr return period, head-on waves, while Figure 21 gives the corresponding computed transmission coefficients. The target K_t value of 0.5 is approximately equalled in the lee of most portions of the breakwater, confirming that the modeled transmission is matching what is predicted by the empirical tools.

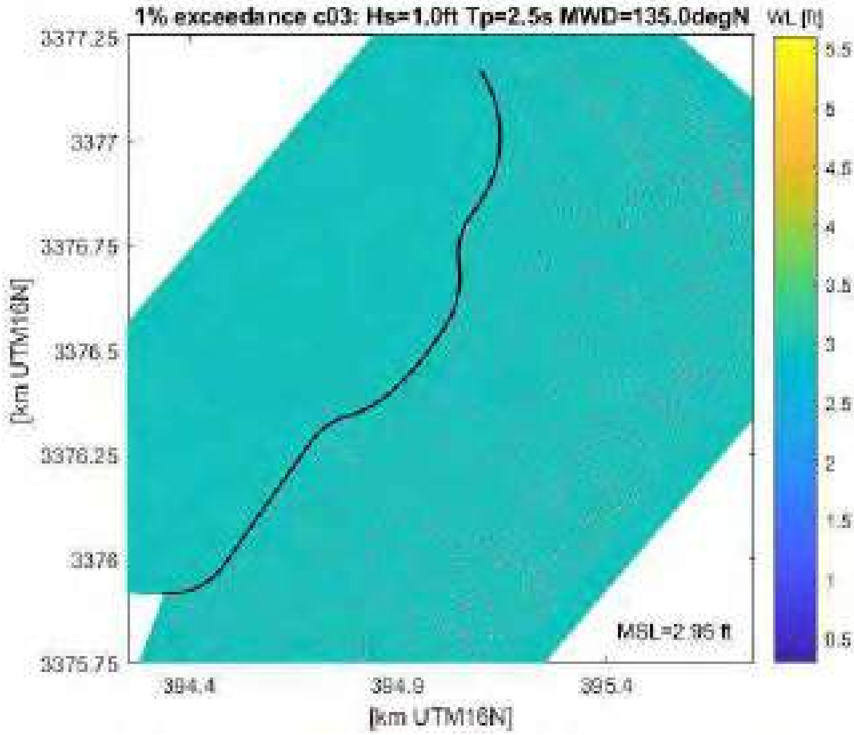


Figure 18: Snapshot of the wave field (actual instantaneous water surface) at the last timestep of the simulation for the 1-yr RP event with 135 degree incidence, impacting a continuous breakwater with a goal transmission coefficient of 0.51.

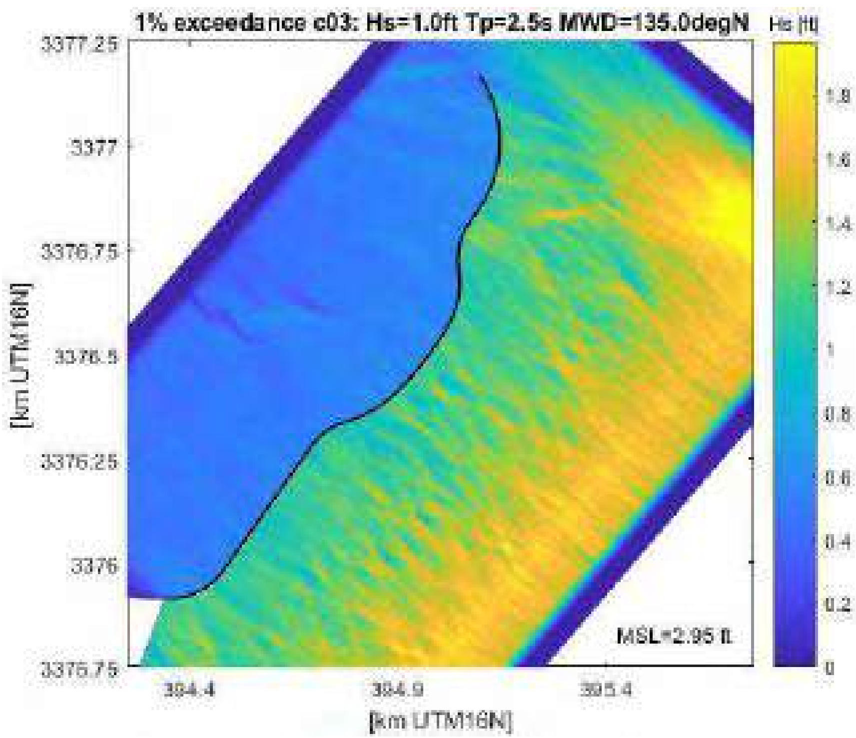


Figure 19: Significant wave height for the 1-yr RP event with 135 degree incidence, impacting a continuous breakwater with a transmission goal coefficient of 0.51.

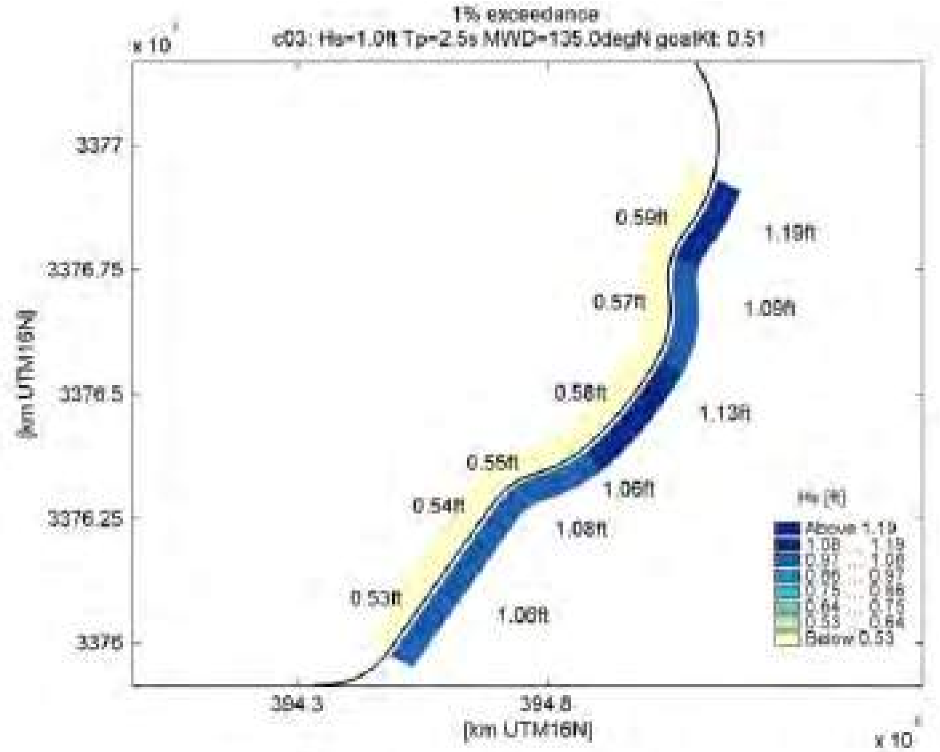


Figure 20: Significant wave heights in front of and behind the breakwater for head-on (135°) 1-yr return period waves.

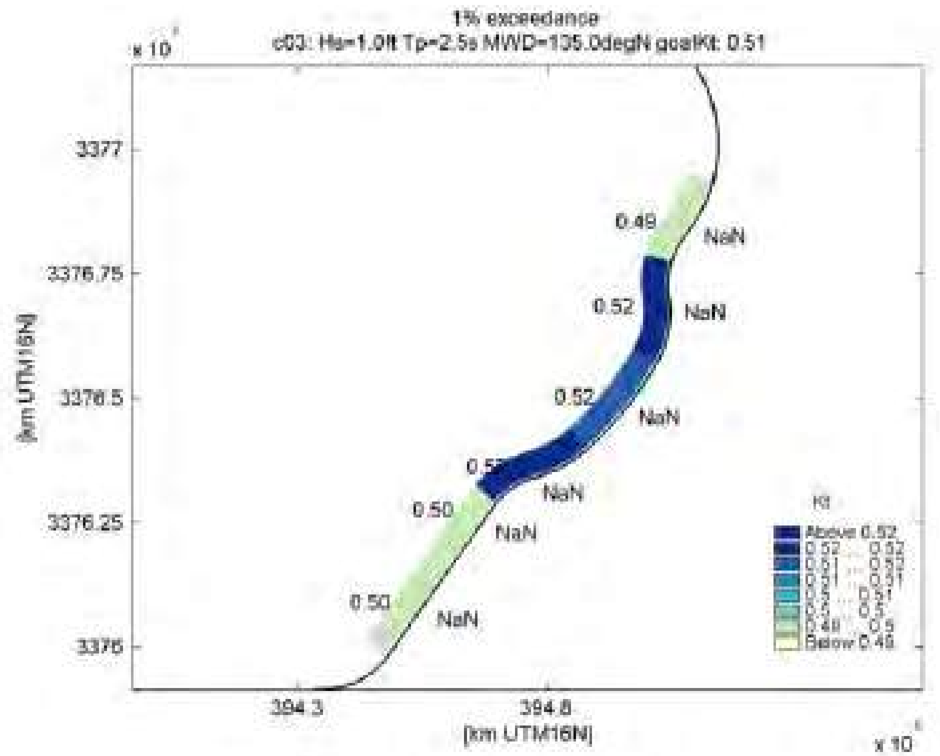


Figure 21: Computed transmission coefficient for head-on (135°) 1-yr return period waves.

5.3. Results

With the imposed porosity values adequately representing the expected breakwater transmission, the actual Reefmaker alternative breakwater footprint was implemented in the model so that the combined effects of transmission and diffraction through the fish gaps could be analyzed. Though the dominant wave direction is South-Southeast (SSE), waves still approach the site from a range of directions from East (E) through South (S). To account for this variability, the 1-yr RP wave conditions were modeled with the incident wave angles 105°, 120°, 135°, 150°and 165°. The following figures plot the computed significant wave height in the project vicinity for each of the 5 simulated wave directions. The results indicate that the Reefmaker breakwater alternative configuration provides sufficient overlap such that wave penetration through the fish gaps is negligible. A low-crested, rubble mound breakwater with a similar overlapping configuration will likely provide the same level of protection against wave penetration through the fish gaps.

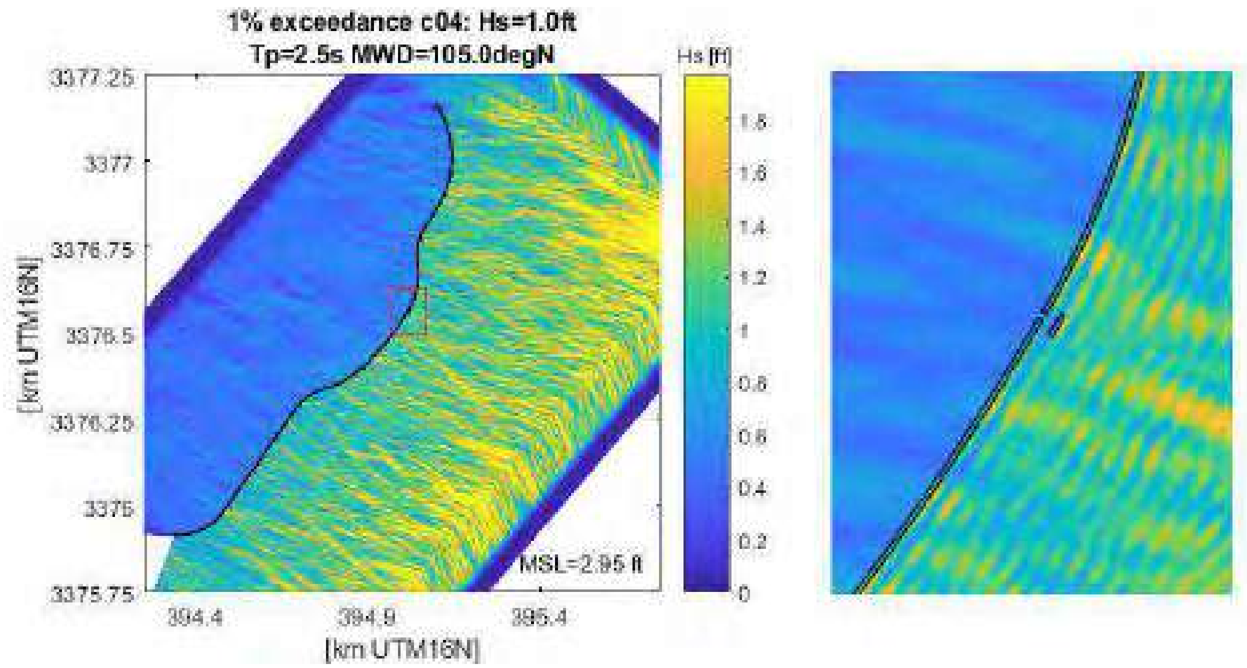


Figure 22: Significant wave height for the 1-yr RP event with 105-degree incidence, impacting the Reefmaker alternative breakwater configuration with a transmission coefficient of 0.51. The right subplot shows a zoomed-in view of the Hs at a fish gap, with extents indicated by the red box in the left subplot.

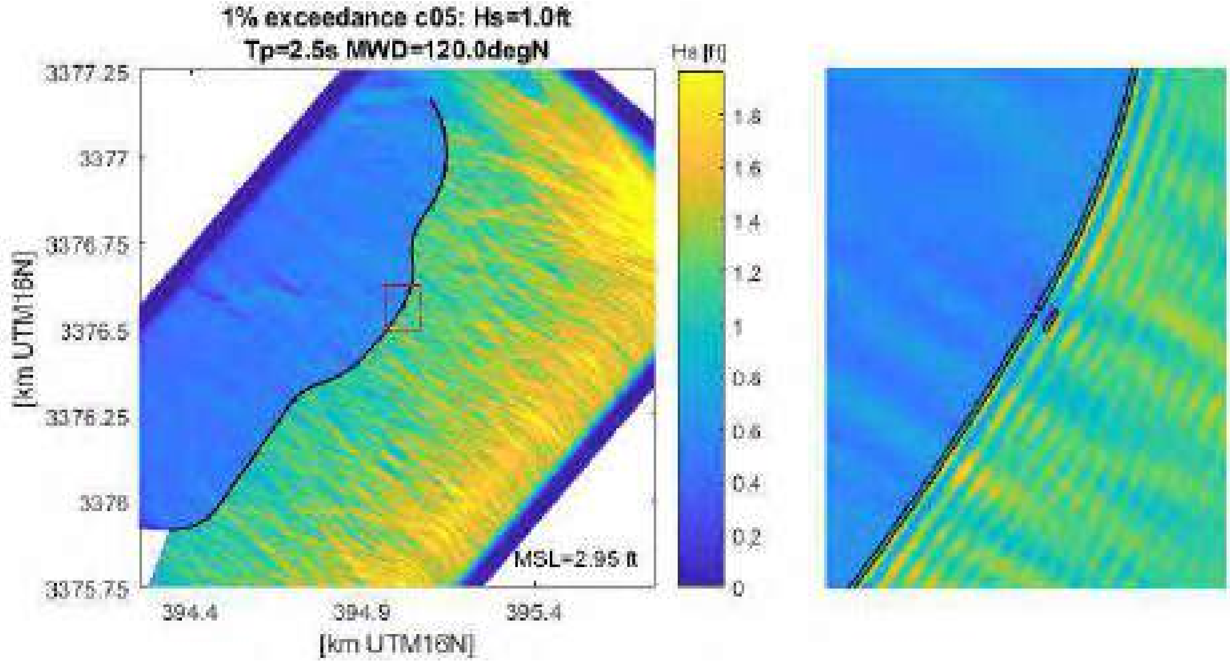


Figure 23: Significant wave height for the 1-yr RP event with 120-degree incidence, impacting the Reefmaker alternative breakwater configuration with a transmission coefficient of 0.51. The right subplot shows a zoomed-in view of the Hs at a fish gap, with extents indicated by the red box in the left subplot.

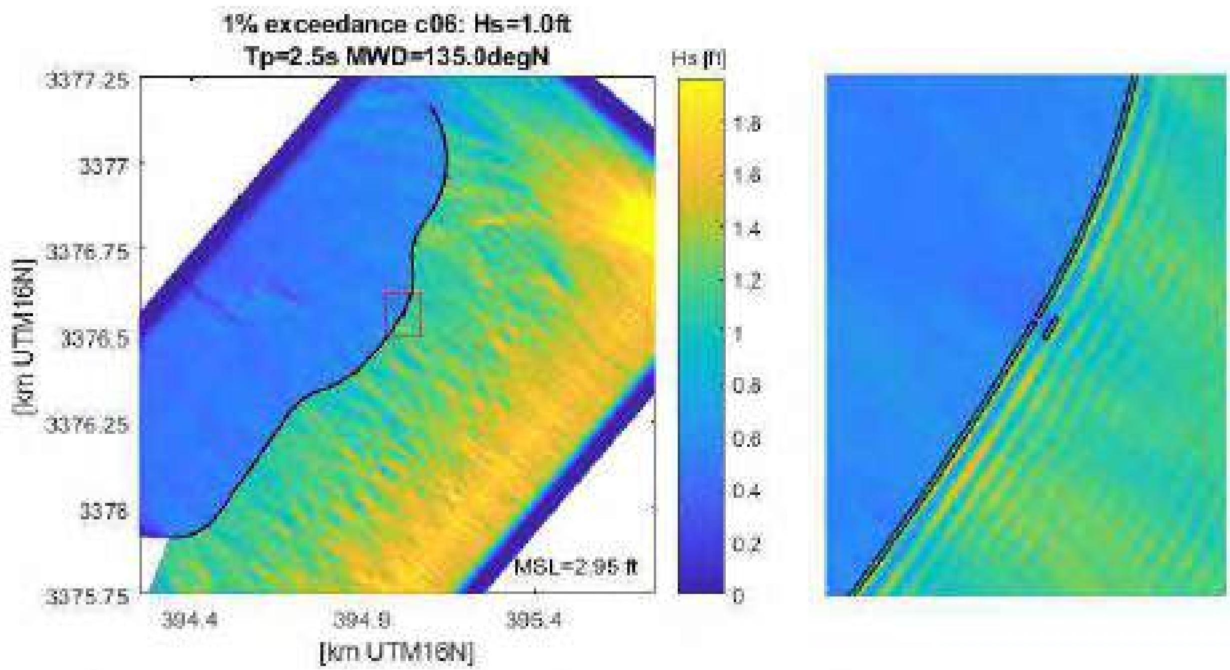


Figure 24: Significant wave height for the 1-yr RP event with 135-degree incidence, impacting the Reefmaker alternative breakwater configuration with a transmission coefficient of 0.51. The right subplot shows a zoomed-in view of the Hs at a fish gap, with extents indicated by the red box in the left subplot.

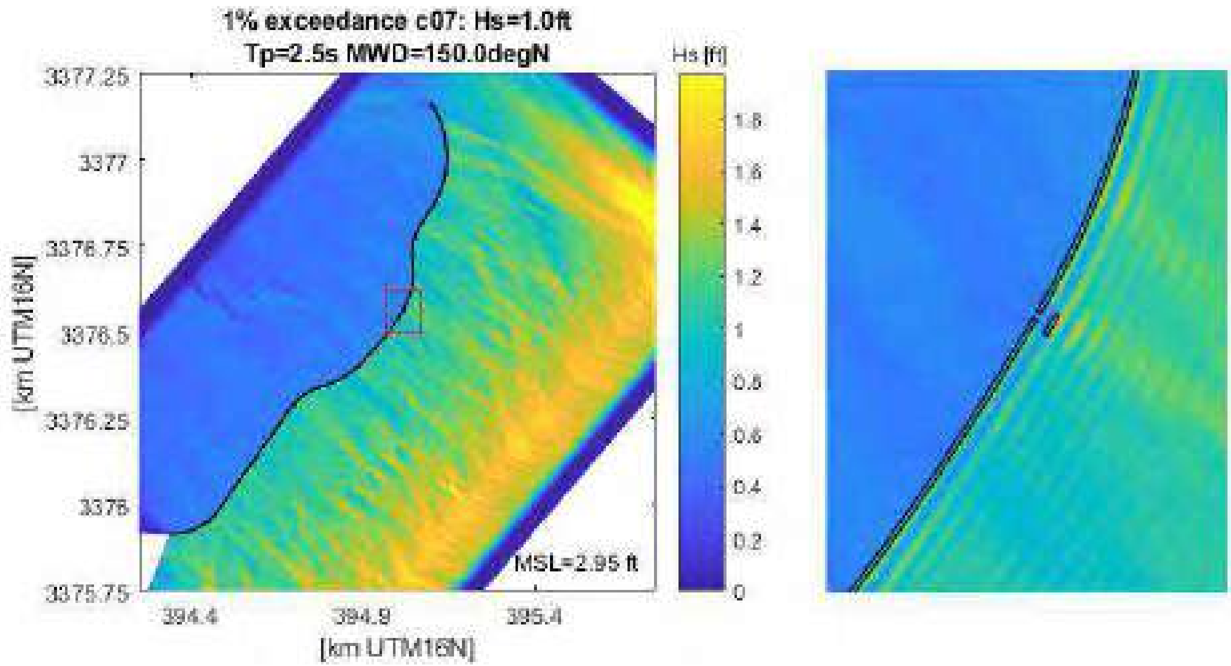


Figure 25: Significant wave height for the 1-yr RP event with 150-degree incidence, impacting the Reefmaker alternative breakwater configuration with a transmission coefficient of 0.51. The right subplot shows a zoomed-in view of the Hs at a fish gap, with extents indicated by the red box in the left subplot.

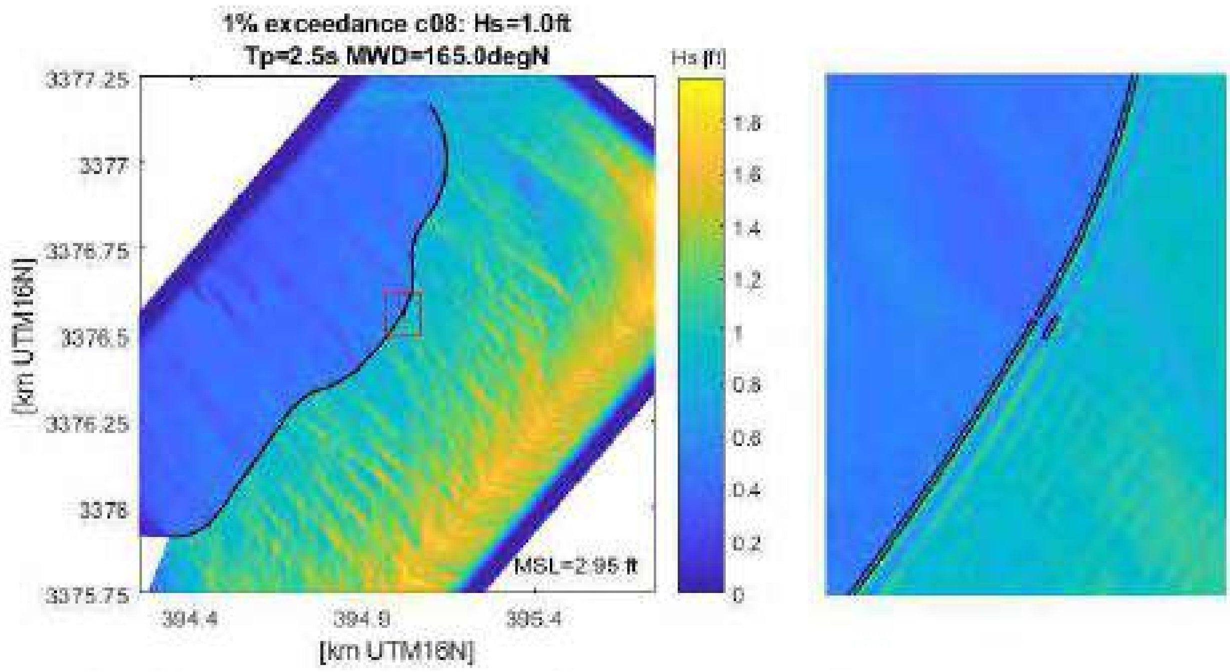


Figure 26: Significant wave height for the 1-yr RP event with 165-degree incidence, impacting the Reefmaker alternative breakwater configuration with a transmission coefficient of 0.51. The right subplot shows a zoomed-in view of the Hs at a fish gap, with extents indicated by the red box in the left subplot.

6. Vessel Wake Analysis

6.1. Introduction

Theodore Ship Channel, located directly north of the Deer River project site, experiences commercial and recreational ship traffic which produces wakes that may be disruptive to the proposed breakwater structures. The following section includes data and calculations for the water waves produced from the wakes of the 10 most frequent vessels passing through the Theodore Ship Channel in 2017, in order to better inform the design of the proposed breakwater structures. Additionally, data collected and assessed by the USACE (2019) as part of their *Mobile Harbor, Mobile, Alabama Integrated Final General Evaluation Report with Supplemental Environmental Impact Statement, Mobile County, Alabama (GRR/SEIS)* was used to supplement the evaluation and analysis of vessel wakes.

6.2. AIS Data Collection & Analysis

Vessel Automatic Identification System (AIS) data was collected for the year 2017 in the Theodore Ship Channel area of interest. All of the identified tracks for this analysis area and period are plotted in Figure 27. The reported vessel speeds from the identified ship tracks were then analyzed, with the frequency distribution also included in Figure 27. From the analysis, it is noted that most of the vessel speeds (appx. 75%) are under 7 knots. After analyzing vessel speeds, the transit frequency of particular vessels was determined, and the 10 vessels with the highest number of trips in the analysis period were selected for additional vessel wake analysis that would be representative of the marine traffic near the project site.

The methodology presented in Schiereck (2011) for vessel-generated secondary waves in navigation channels was applied to estimate the height of ship wakes produced by the vessels (Schiereck & Verhagen, 2011). The length, breadth, draft, and speed of the vessels were determined from the AIS data or supplemented with information available from the vessel operator websites if needed. This data was used for wake calculations (see Table 13), with the resulting wave heights as a function of vessel speed, vessel dimensions, and channel area.

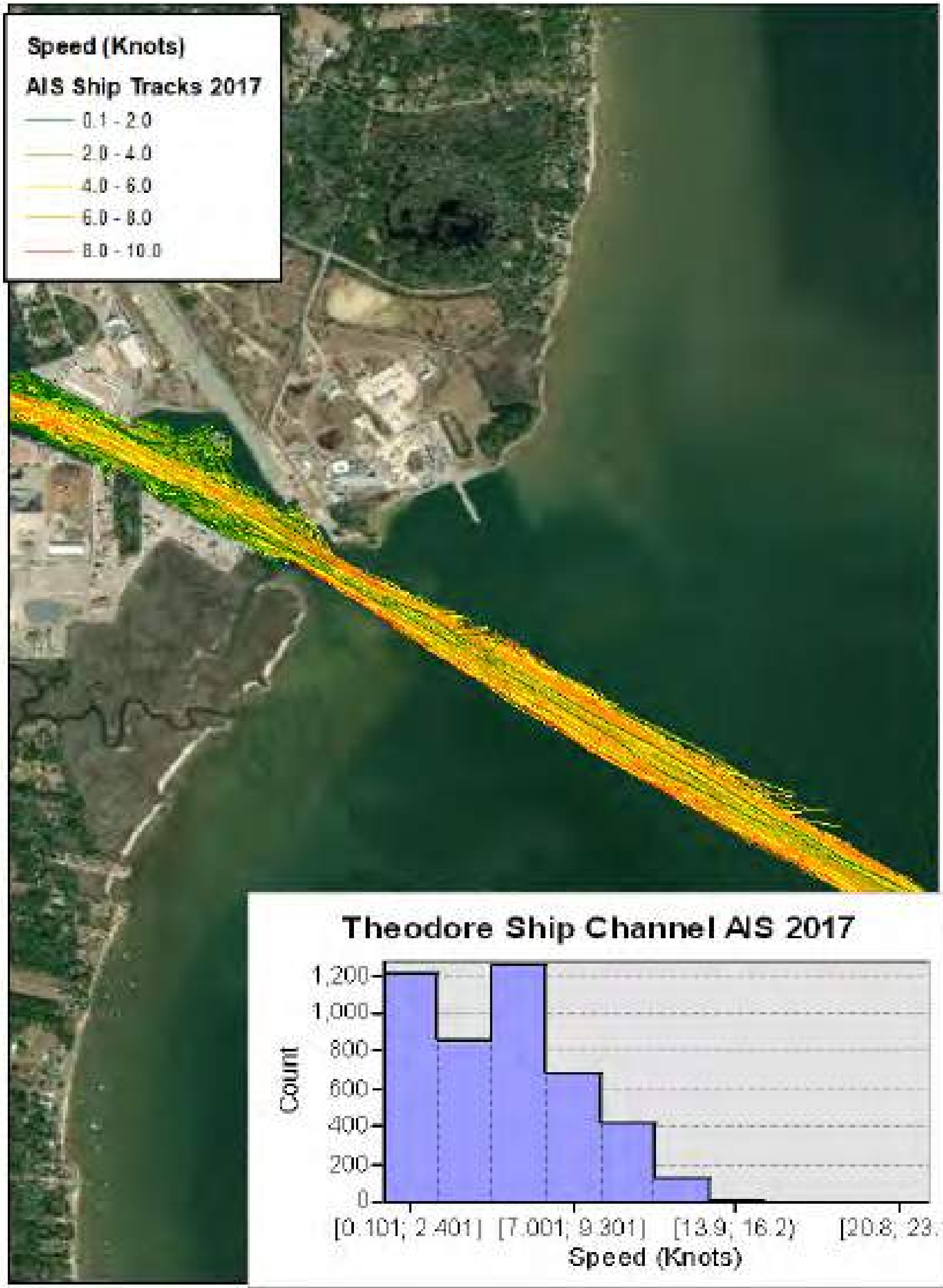


Figure 27. Theodore Ship Channel AIS data for year 2017

Table 13. Most Frequent Vessels of Theodore Ship Channel in 2017 and associated vessel-generated ship wakes.

Name	Vessel Length [ft]	Vessel Beam [ft]	Vessel Draft [ft]	Vessel Speed [knot]	Wake Wave Height [ft]	Wake Wave Period [s]
JUDY D	128	30	6.6	8.3	0.6	2.2
ENTERPRISE	64	28	9.5	9.9	0.7	2.4
SCOTT QUEST	57	22	8.0	8.0	0.5	2.2
SEA ANGEL	160	30	7.5	8.0	0.5	2.2
MS JOY	139	30	7.2	8.2	0.5	2.2
HONOR	94	32	13.1	9.0	0.8	2.4
HARRY BRINDELL	61	26	9.5	8.5	0.6	2.3
MR HENRY	151	30	8.2	9.8	1.1	2.7
SEA EAGLE	123	35	11.2	9.1	0.8	2.5
SABINE	98	33	13.1	6.4	0.2	1.7

The predicted wave heights generated from vessel wakes provided in Table 13 are lower than the extreme wind-generated wave heights predicted for use in the design of the breakwaters (Table 10). Additionally, supporting information was available from the comparison of vessel wake data provided by USACE (2019) which reports "the Average Vessel Generated Wave Energy (VGWE) represented as the statistically significant wave height for all sites ranged between 0.02 ft to 0.15 ft with the highest values being closer to the Mobile Harbor Federal Navigation Channel, decreasing in height moving further from the channel. (U.S. Army Corps of Engineers, 2019)" In summary, analysis of vessel generated wave heights confirms that extreme conditions (Table 10) predicted to result from wind-generated conditions are higher than typical vessel-generated waves and should control the wave design of the Deer River Restoration Project.

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APPENDIX D

GEOTECHNICAL EVALUATION REPORT

GEOTECHNICAL REPORT

Deer River Coastal Marsh Stabilization and Restoration Project
Phase I Engineering and Design
Theodore, Mobile County, Alabama

Mobile Bay National Estuary Program
118 North Royal Street, Suite 601
Mobile, AL 36602

Thompson Engineering Project No.: 19-1101-0184

May 28, 2020

knowledge · ideas · impact

thompson
ENGINEERING

May 28, 2020

Mobile Bay National Estuary Program
118 North Royal Street, Suite 601
Mobile, AL 36602

Attention: **Jason Kudulis, Restoration Project Manager**

Subject: **Geotechnical Report of Shoreline Alternatives and Marsh Settlement**
Deer River Shoreline Stabilization and Marsh Creation Project
Phase I Engineering and Design
Theodore, Mobile County, Alabama
Thompson Project No.: 19-1101-0184

Dear Mr. Kudulis:

Thompson Engineering (Thompson) is pleased to present this geotechnical report of shoreline stabilization alternatives for the Deer River Shoreline Stabilization and Marsh Creation Project. This report presents the results of geotechnical analyses for alternative restoration options, which may be used in support of project design and construction activities. Details on alternative restoration options are presented below and in the attached Appendices. Thompson has previously issued a data report (dated February 14, 2020) detailing our field activities and the results of soil borings and laboratory testing programs for this project. The data report provides the basis for the geotechnical analyses presented herein.

Project Description: The project site is located in Mobile County at the mouth of Deer River. The subject project consists of stabilizing approximately 5,600 feet of shoreline along the western shore of Mobile Bay located near the south side of the mouth of the Theodore Industrial Ship Channel. Besides stabilizing the shoreline, the overall goal of the project is to enhance aquatic, wetland, and inland habitats to the extent possible. A range of options is being considered for the restoration including marsh creation, possibly using dredged soils from Deer River for wetland fill as well as alternative shoreline reclamation and stabilization systems such as conventional rubble mound breakwaters, pile supported Reefmaker™ structure, OysterBreak™ breakwater systems, or a combination of these. The objective of the project is to develop a cost-effective and functional design that when implemented will meet the aforementioned goals while enhancing overall ecosystem functionality to the extent practicable. At present, dredging within the Deer River is being considered as a potential borrow source for the fill required to create the new marsh area.

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Soil Description: From a review of the boring and laboratory test data, it appears that there are 3 key areas where soils in the upper layers are significantly soft and lower in shear strength than at other areas of the site. These three areas are associated with (1) boring MB-01, (2) borings MB-05 through MB-07, and (3) boring MB-10. When used in this report, the term "Soft Soil" refers to these 3 areas. Soils within the "Soft Soil" areas generally consist of the following layers:

- 0 – 35 ft.: very soft interbedded layers of clay and clayey sand with varying amounts of organics (avg. undrained shear strength 150 psf)
- 35 – 52 ft.: soft clay (avg. undrained shear strength 250 psf)
- 52 – 63 ft.: medium dense sands.

Outside of the soft areas, the existing subgrade encountered relatively stiffer soil which we have termed in this report as "Better Soil" areas. In the better soil areas, the average soil profile generally consisted the following layers:

- 0 – 5 ft.: soft clay with sand and organics (avg. undrained shear strength 250 psf).
- 5 – 11 ft.: Interbedded layers of sand and clay (avg. undrained shear strength 500 psf).
- 11-21': soft clay with sand and organics (avg. undrained shear strength 300-500 psf).
- 21-43': Interbedded layers of sand and clay (avg. undrained shear strength 1000 psf).
- 43 – 63 ft.: medium dense sands

The Test Location Plan attached in **Appendix A** indicates the general boundaries of what are considered to be the "Soft Soil" and "Better Soil" areas.

GEOTECHNICAL ANALYSES OF BREAKWATER ALTERNATIVES

In order to assist the design team, we have performed stability and settlement analyses for 3 potential breakwater systems. As indicated below it is likely possible to construct conventional rubble mound breakwaters in some areas of the project. However, as previously described, extremely soft soil conditions exist within certain areas of the project, and it is likely that conventional rubble mound breakwaters cannot be supported in these soft areas without significant and costly ground improvement measures. The following report sections address the design considerations for:

1. Conventional Rubble Mound Breakwater Structures
2. Reefmaker™ Breakwater Structures
3. OysterBreak™ Breakwater Structures

1. RUBBLE MOUND BREAKWATER

A breakwater will be constructed using imported rubble along the planned shoreline of the project boundary. An access channel will be required to allow for construction to take place by barge. The access channel will likely be constructed east of the proposed breakwater. The dredge spoil from access channel dredging will be temporarily placed on the east side of the access channel. The construction of breakwater will likely take place from the southern end of the access channel progressing to the north. Backfilling the access channel will like occur coincidentally with the breakwater construction.

In constructing the rubble mound breakwater system, a marine mattress (12 inch thick Triton Marine Mattress or approved equal) will be first be placed on the existing mudline within the entire breakwater footprint. The rubble mound riprap will then be placed to the design crest elevation of +2.5 (or higher to account for settlement). The analysis graphics, Run-1 through Run-3 in **Appendix B** shows the cross section of the rubble mound. Due to the concern of slope stability and excessive settlement, the rubble mound breakwater system is only recommended within the "Better Soil" areas as indicated on the Test Location Plan in **Appendix A**. We anticipate the marine mattress may 'sink' up to 2 ft. into underlying mudline during construction due to displacement of the upper soft soils. In estimating the volume of riprap, the initial displacement of 2 ft. should be considered in addition to the anticipated long-term settlement of the breakwater discussed later in this report.

Lightweight Aggregate Fill Breakwater: As an alternative to the rubble mound breakwater described above, as breakwater using lightweight fill can be constructed. The use of a lightweight aggregate can reduce the overall weight of breakwater structure improving stability and reducing long-term settlement. The lightweight fill core of the breakwater should be composed of Expanded Shale, Clay and Slate (ESCS) lightweight. The total unit weight of the lightweight fill is should be 75 pcf. We have utilized a friction angle of 35 degrees for the stability analyses. In order to reduce the risk of excessive settlement and slope stability, the lightweight aggregate fill breakwater system is recommended within the "Soft Soil" areas as shown on Test Location Plan in **Appendix A**.

In constructing the lightweight aggregate breakwater system, a marine mattress (12 inch thick Triton Marine Mattress or approved equal) will be first be placed on the existing mudline within the entire breakwater footprint. Then lightweight aggregate wrapped with a filter fabric should be placed on the marine mattress. The lightweight aggregate section should then protected all around by riprap. The design thickness of the riprap (assumed 3 ft.) should be confirmed by the project engineer. In estimating the volume of the lightweight aggregate and riprap should considered displacement of up to 2-ft. of the soil near mudline soils that is expected to occur during construction of the breakwater. This initial displacement of up to 2 ft. should be considered in addition to the anticipated long-term settlement of the breakwater discussed later in this report.

Slope Stability Analysis of Breakwaters

During construction process of breakwater, the most critical scenario from a slope stability perspective is while a rubble mound is being placed adjacent to the access channel but the channel has not been backfilled with the dredging fill yet. We performed the slope stability analysis for the most critical scenario using the slope stability analysis program "Slope/W" from Geoslope, Inc™. Slope/W is a limit equilibrium computerized solution used to perform the slope and bearing related stability analyses. It incorporates Bishop (1955), Spencer (1967), and Morgenstern and Price (1965) slope evaluation methods to determine the minimum factor of safety for a soil profile using the soil parameters and loading conditions specified. Factor of safety is defined as the ratio of shear strength of the rotating soil mass along the critical slip plane to the shear strength mobilized. A factor of safety less than 1.0 indicates imminent slope failure. For temporary construction condition (such as the dredge spoil slopes), a minimum factor of safety of 1.1 is generally required for considering stable slope condition. For the end of construction a minimum factor of safety of 1.3 is recommended; whereas, for permanent construction condition, a minimum factor of safety of 1.4 is generally considered suitable to represent a stable slope for long term conditions.

In determining the most critical factor of safety, the program defines a potential failure surface and subdivides the bounded soil mass into a finite number of vertical slices, using an iterative procedure to compute the factor of safety. The program develops alternate trial failure surfaces based on specified failure surface entry and exit limits and computes a factor of safety for each trial surface. The final results of the analysis provide a list of factors of safety with a corresponding diagram of the model and failure surface. The printed output for each slope stability model (**Appendix B**) consists of a graphical representation of worst-case failure surface determined by the program and the associated calculated factor of safety for stability along the failure surface.

All slope runs assume the modeled geometry can be constructed as depicted. Constructions methods may result in different settlements depending on mud-waving or other effects of construction means and methods. Means and methods should be further evaluated by the project design team to determine suitable guidelines. Tables 1 & 2 below provide a summary of our stability analyses at for a typical rubble mound breakwater constructed with "Better Soil" areas and a lightweight aggregate core breakwater constructed in "Soft Soil" areas, respectively. Additional details regarding these analyses are also provided this report section.

Table 1. Summary of Slope Stability Analyses - Better Soils Area

Model #	Modeled Conditions	F.S. for Worst-Case Failure Plane
RUN-1	Temporary slope stability of Dredge Spoil. A factor of safety of 1.1 or greater is recommended for this temporary condition.	2.47
RUN-2	Short-term slope stability of Rubble Mound with marine mattress. Short-term factor of safety of 1.3 or greater is recommended.	2.09
RUN-3	Long-term slope stability of slope configuration from Run-2 after the access channel is backfilled with dredged fill. Long-term factor of safety of 1.4 or greater is recommended.	2.10

Table 2. Summary of Slope Stability Analyses - Soft Soils Area

Model #	Modeled Conditions	F.S. for Worst-Case Failure Plane
RUN-4	Temporary slope stability of Dredge Spoil. A factor of safety of 1.1 or greater is recommended for this temporary condition.	1.16
RUN-5	Short-term slope stability of lightweight aggregate core encased with 3-ft. Riprap cover underlain with marine mattress. Short-term factor of safety of 1.3 or greater is recommended.	1.32
RUN-6	Long-term slope stability of slope configuration from Run-5 after the access channel is backfilled with dredged fill. Long-term factor of safety of 1.4 or greater is recommended..	1.61

Each Run of Tables 1 and 2 are described in more detail below:

- Run 1: The intent was to determine a stable geometry for the dredge fill placement within the retained area during construction of the shoreline breakwater in the “Better Soil” areas. The minimum factor of safety requirement is 1.1 for temporary stability condition.
- Runs 2 & 3: The intent is to show that conventional construction (access channel and placement of marine mattress and rubble mound) of the rubble mound breakwater is feasible within the “Better Soil” areas. We expect mud-waving to occur within the upper 24 inches on average, so the berm geometry is set 24 inches below the current mudline elevation. Runs 2 and 3 show the short term (during or end of construction) and long term (access channel has been backfilled) stability of rubble mound in the “Better soils” areas.
- Run 4: The intent was to determine a stable geometry for the dredge fill placement within the retained area during construction of the shoreline breakwater in the “Soft Soil” areas. The general geometry of the dredge spoil is indicated in the output. Note that there should be at least 10 feet setback between the top of shore side (inboard) dredge cut slope and toe of the temporary dredge spoil stockpile.
- Runs 5 & 6: Conventional rubble mound breakwaters are not recommended for the “Soft Soil” areas.. The intent of these runs is to show that construction of a breakwater system using a lightweight aggregate core is feasible from a stability perspective. These runs consider the use of a lightweight aggregate (max 75 pcf) core wrapped in a filter fabric over marine mattress in the “Soft soils” areas. The model considers a 3 ft. thick riprap cover of the lightweight aggregate core. Runs 5 and 6 show the short-term (during or end of construction) and long-term (access channel has been backfilled) stability of lightweight aggregate in the “Soft soils” area, respectively.

Settlement Evaluation of Breakwater

In general, two factors contribute to subsurface settlement, the elastic settlement of sands and consolidation of clays. The former occurs significantly faster than the latter. It is anticipated that nearly 100 percent of the anticipated elastic settlement of subsurface sands will occur during fill placement. Consolidation of subsurface clays will begin during filling activities and will continue for a significant period of time after the marsh fill has been placed. The portion of consolidation, which has not occurred during construction and continues after filling has been completed is referred to in this report as either “post-construction settlement” or “post-construction consolidation”. Immediate and post-construction settlement estimates assist in the estimation of total fill material required to attain finish grades, by including the amount of fill “lost” due to settlement.

The subsurface findings suggest some variability of soil types and strata thickness in the shallow subsurface stratigraphy throughout the site. The settlement estimates have been based on an average subsurface profiles for the “soft soil areas” and the “better soil areas”. Actual settlement magnitudes and time rates may vary from those indicated herein.

The rate of vertical consolidation is a function of the coefficient of consolidation (Cv), the thickness of the clay stratum, and drainage characteristics of the subsurface clays. For the time-rate calculations, reliance on the results of laboratory consolidation test and our experience with similar soil conditions were relied upon for consolidation estimates.

Settlement analyses were performed using settlement analysis program “Settle3D” for fill thicknesses that result in finish marsh elevations ranging from -3.0 to +2.5 ft.-NAVD88. The post-construction settlement of the lightweight aggregate core breakwater mound in the “Soft Soil” areas is estimated to be on the order of 18 to 24 inches over a period of 20 years. The post-construction settlement of the typical rubble mound breakwater constructed within the “Better Soil” areas constructed is estimated to be in the range of 6 to 12 inches over a period of 20 years. Please note these settlements do not include the previously mentioned initial settlement that may occur during construction due to displacement of the near mudline soils which may result in mud waves.

2. REEFMAKER BREAKWATER SYSTEM

An alternative to a rubble mound breakwater is a Reefmaker™ breakwater system. The Reefmaker™ system consists of 12 inch tall concrete disc-reefs which are mounted vertically in series on a pipe pile. The weight of the reefs are approximately 4,000 lbs., 4 to 6 feet tall and approximately 5 feet in diameter. The artificial reefs are available in various shapes such as Pyramids, Grouper Reefs, and EcoSystem Discs. For this project, we have analyzed a 12-inch diameter fiberglass composite pile with wall thickness of 0.5 inch. The axial compression (dead weight load) and lateral load on the proposed reef maker is estimated to be approximately 4 tons and 2 tons, respectively.

Provided pile load testing is performed, a factor of safety of 2.0 applied to the design axial capacities can be utilized to determine the pile embedment depths. If pile load testing is not performed, a factor of safety of 3.0 should be utilized to determine the pile embedment depth. The pile capacity analysis attached in **Appendix C** indicates a 12-inch fiberglass composite pile will need to be installed to a minimum 40 feet below the existing mudline in order to support the anticipated axial load with factor of safety of 2.0. If pile load testing is not performed, a minimum depth of 55 feet below the mudline will be required.

Lateral pile analyses were performed using the program LPile. The results of the analyses are attached in **Appendix C.** When considering that the bottom 12-inch tall Reefmaker disc is located below the mudline to improve lateral capacity, a pile head deflection of 2.1 inches from the 2-ton lateral load was calculated. When considering that the bottom 12-inch tall Reefmaker disc is located above the mudline, a pile head deflection of 2.7 inches from the 2-ton lateral load was calculated. As indicated on the attached LPile output, the deflections mentioned above are considered to be at the mudline, and the lateral load is applied at the mudline.

3. OYSTERBREAK BREAKWATER SYSTEM

The OysterBreak™ is a patented technology designed to use the oyster's inherent nature of clustering to enhance a strategic coastal protection structure for coastal and estuary shorelines. They may be applied to any shoreline project that calls for any combination of wave attenuation, and shoreline erosion mitigation. OysterBreak™ units are designed to serve dual functions by creating a reef structure for habitat and robust structure for shoreline protection. OysterBreak™ units may be custom designed to meet project and regional needs. The OysterBreak™ system consists of unique interlocking armor units that in themselves can form an effective coastal engineering structure. The modular units can be arranged in any configuration to fit project needs. Given the right conditions, the units will become increasingly covered with oyster growth, enhancing the structure’s stability and wave attenuation performance over time.

For our settlement analysis, we have considered an OysterBreak breakwater that is three units wide at the base to allow for a potential stack height of up to three 24-inch tall rings, resulting in a breakwater that is approximately 6 feet tall with a base width of 15 feet. The estimated area load for this configuration is 250 psf. Please note that a marine mattress (or geogrid reinforced aggregate section that is 1 to 2 feet thick) will be required below the oyster rings to distribute the area load. In the “Soft Soil” areas, the post-construction settlement of the OysterBreak is estimated to be on the order of 6 inches over a period of 20 years. In the better soil areas, the post-construction settlement is estimated to be on the order of 3 inches over a period of 20 years.

MASS FILL SETTLEMENT

The previous sections addressed the geotechnical analyses associated with the breakwater structures. We have also performed settlement analyses related to mass fill placement within the potential marsh creation area. **Figures 1 and 2** in **Appendix D** presents the results of settlement modeling for various fill heights within both the “soft soil areas” and “better soil areas” of the project.

GENERAL

This report is prepared for the exclusive use by Mobile Bay National Estuary Program and is prepared in accordance with the Standard of Care reasonably expected of similar geotechnical engineers, providing similar services in a similar locale. No warranty is expressed or implied and all such warranties are disclaimed. This report is prepared for a limited purpose as further detailed by the objectives and/or scope work identified herein.

The evaluation and recommendations submitted in this geotechnical report are based in part upon the data obtained from the field exploration program. The nature or extent of variations throughout the subsurface profile may not become evident until the time of construction. If variations then appear evident, it may be necessary to reevaluate our recommendations as provided in this geotechnical report. Furthermore, the recovered samples were not examined, either visually or analytically, for chemical composition or environmental hazards.

The soil borings or other subsurface tests presented in this report were performed in support of the geotechnical analyses and recommendations as defined by the scope of services and not for determining the presence or extent of any subsurface debris which may exist at the site. Depending on project location, subsurface conditions, and the history of the site, buried debris, environmental contamination, or other soil types and conditions not identified may be encountered during construction.

This report has not been prepared as, and should not be used as, a design or specification document to be directly implemented by the Contractor. In addition, this report was not prepared for the purposes of bid development and the accuracy of this report is limited. If included as part of a bid package, it is for informational purposes only, and it shall remain the Contractor’s responsibility to retain and confer with a geotechnical engineer to obtain specific types of information the Contractor or others may need or prefer to interpret this report, or perform additional geotechnical testing prior to bidding and construction.

We appreciate the opportunity to continue to assist the Mobile Bay National Estuary Program and the Dauphin Island Sea Lab with project-related geotechnical matters. Please do not hesitate to contact our office with any questions concerning this submittal.

Respectfully,

THOMPSON ENGINEERING, INC

ALABAMA LICENSED No. 26300 PROFESSIONAL ENGINEER CAMERON LEE CRIGLER

Debashis Sikdar, P.E., Ph.D.
Project Geotechnical Engineer

Cameron Crigler, P.E.
Principal Geotechnical Engineer

Attachments: Appendix A – Subsurface Soil Profile Sheet
Appendix B – Stability Analysis of Rubble Mound Breakwater
Appendix C – Axial and Lateral Load Analysis of Reef Maker Pile
Appendix D – Mass Fill Settlement

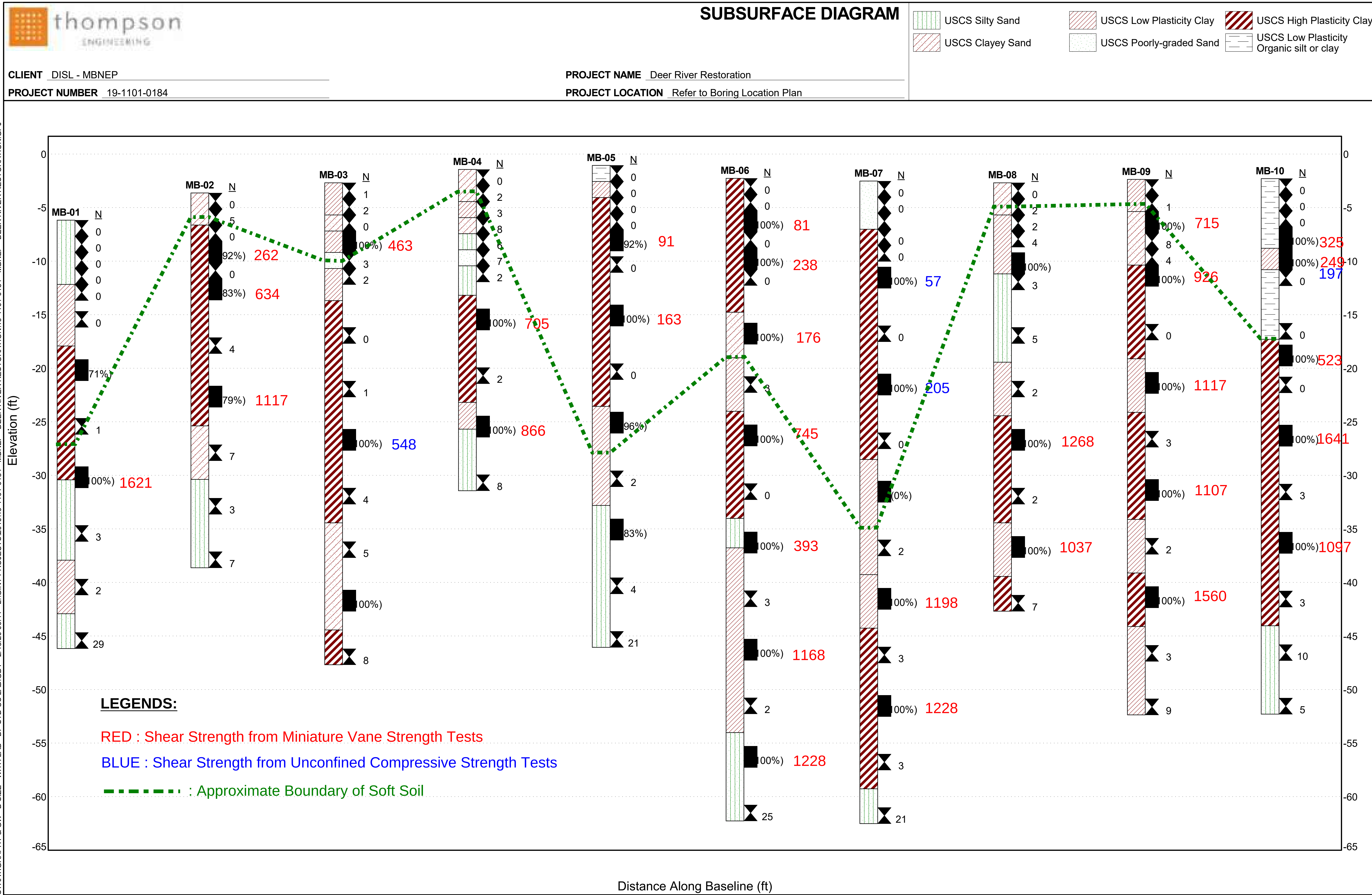
APPENDIX A

Subsurface Soil Profile Sheet

SUBSURFACE DIAGRAM

thompson ENGINEERING

CLIENT	DISL - MBNEP	PROJECT NAME	Deer River Restoration
PROJECT NUMBER	19-1101-0184	PROJECT LOCATION	Refer to Boring Location Plan



Deer River Restoration Project
Theodore, Mobile County, Alabama

thompson ENGINEERING

TEST LOCATION PLAN

Project No. 19-1101-0184

DATE: February 14, 2020

APPENDIX B

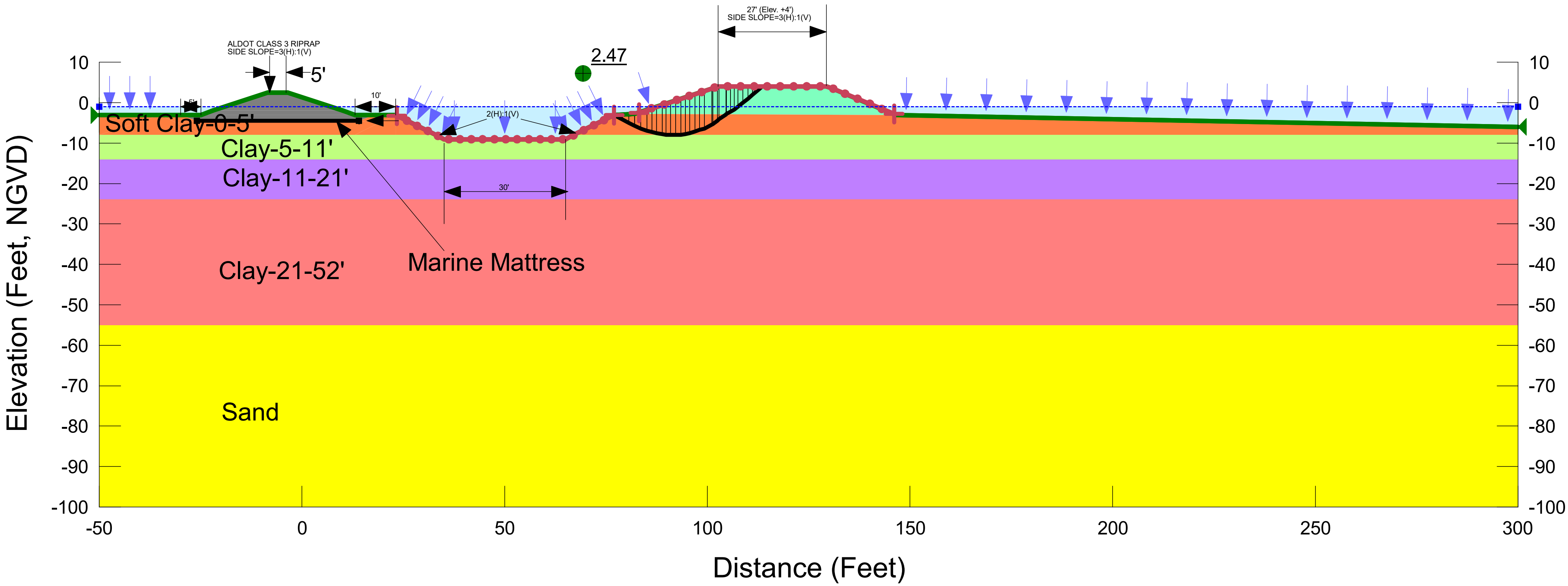
Stability Analysis of Rubble Mound Breakwater

Run-1

Restoration Of Deer River - Better Area
Proposed Continuous Rock Dike

N
E (Bay)

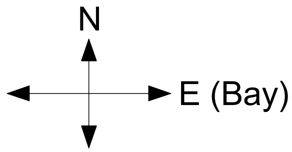
Short Term Stability
Factor of Safety: 2.47



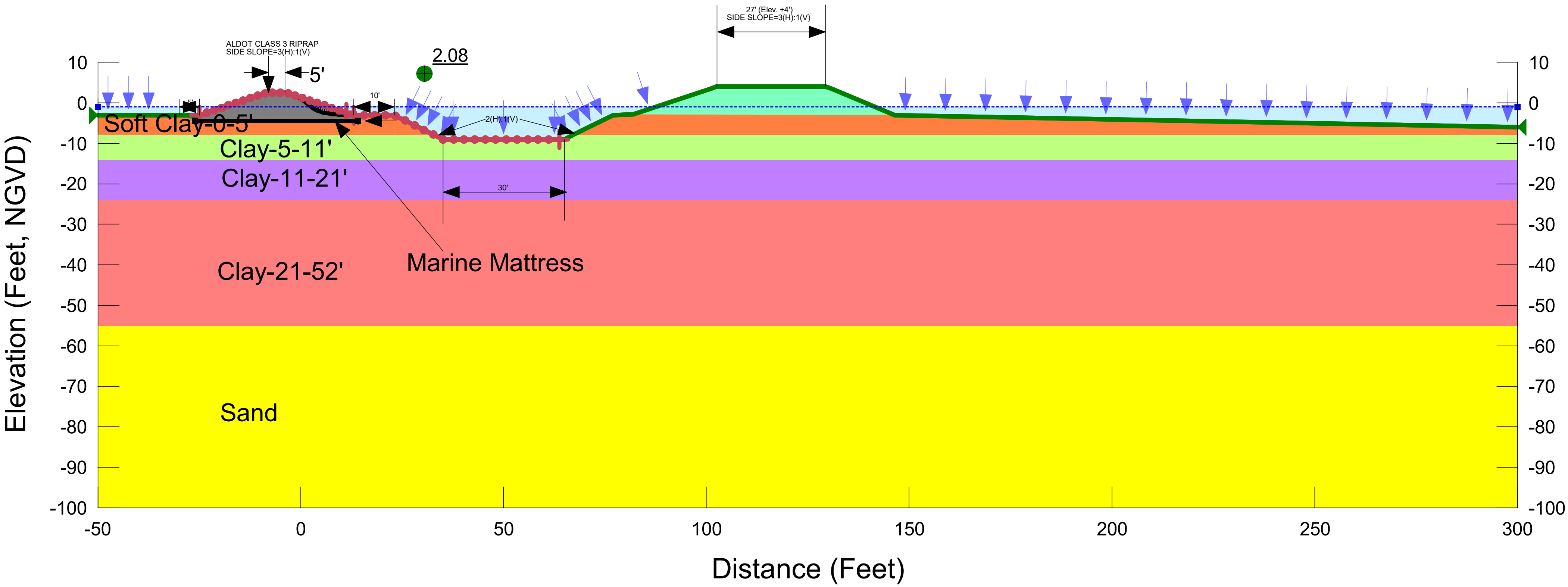
Name: SOFT CLAY-0-5' Unit Weight: 97 pcf Cohesion: 250 psf Phi: 0 °
Name: CLAY-5-11' Unit Weight: 100 pcf Cohesion: 500 psf Phi: 0 °
Name: SAND Unit Weight: 115 pcf Cohesion: 0 psf Phi: 31 °
Name: DREDGE FILL Unit Weight: 95 pcf Cohesion: 100 psf Phi: 0 °
Name: RIPRAP Unit Weight: 108 pcf Cohesion: 0 psf Phi: 38 °
Name: CLAY-11-21' Unit Weight: 200 pcf C-Top of Layer: 300 psf C-Rate of Change: 25 psf/ft Limiting C: 500 psf
Name: CLAY-21-52' Unit Weight: 200 pcf Cohesion: 1000 psf Phi: 0 °

Run-2

Restoration Of Deer River - Better Area
Proposed Continuous Rock Dike



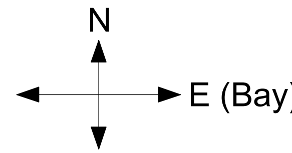
Short Term Stability
Factor of Safety: 2.08



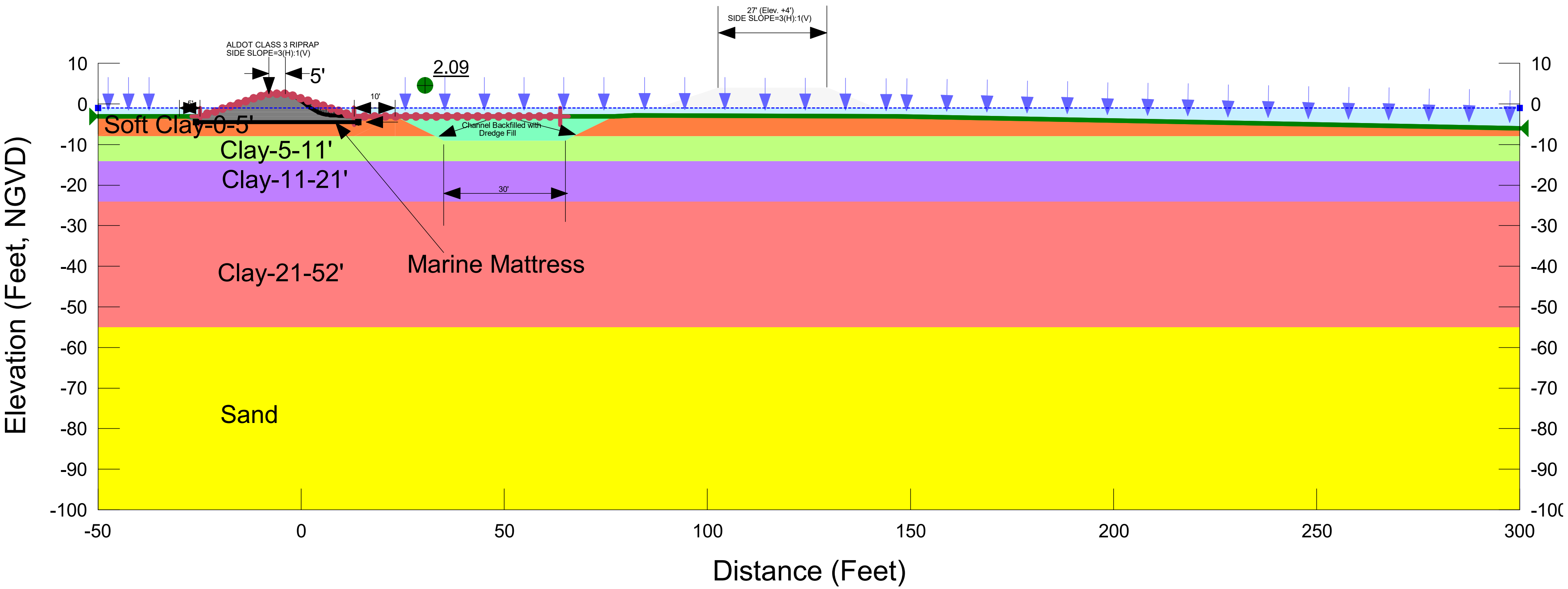
Name: SOFT CLAY-0-5' Unit Weight: 97 pcf Cohesion: 250 psf Phi: 0 °
Name: CLAY-5-11' Unit Weight: 100 pcf Cohesion: 500 psf Phi: 0 °
Name: SAND Unit Weight: 115 pcf Cohesion: 0 psf Phi: 31 °
Name: DREDGE FILL Unit Weight: 95 pcf Cohesion: 100 psf Phi: 0 °
Name: RIPRAP Unit Weight: 108 pcf Cohesion: 0 psf Phi: 38 °
Name: CLAY-11-21' Unit Weight: 200 pcf C-Top of Layer: 300 psf C-Rate of Change: 25 psf/ft Limiting C: 500 psf
Name: CLAY-21-52' Unit Weight: 200 pcf Cohesion: 1000 psf Phi: 0 °

Run-3

Restoration Of Deer River - Better Area
Proposed Continuous Rock Dike



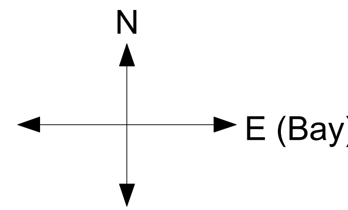
Long Term Stability
Factor of Safety: 2.09



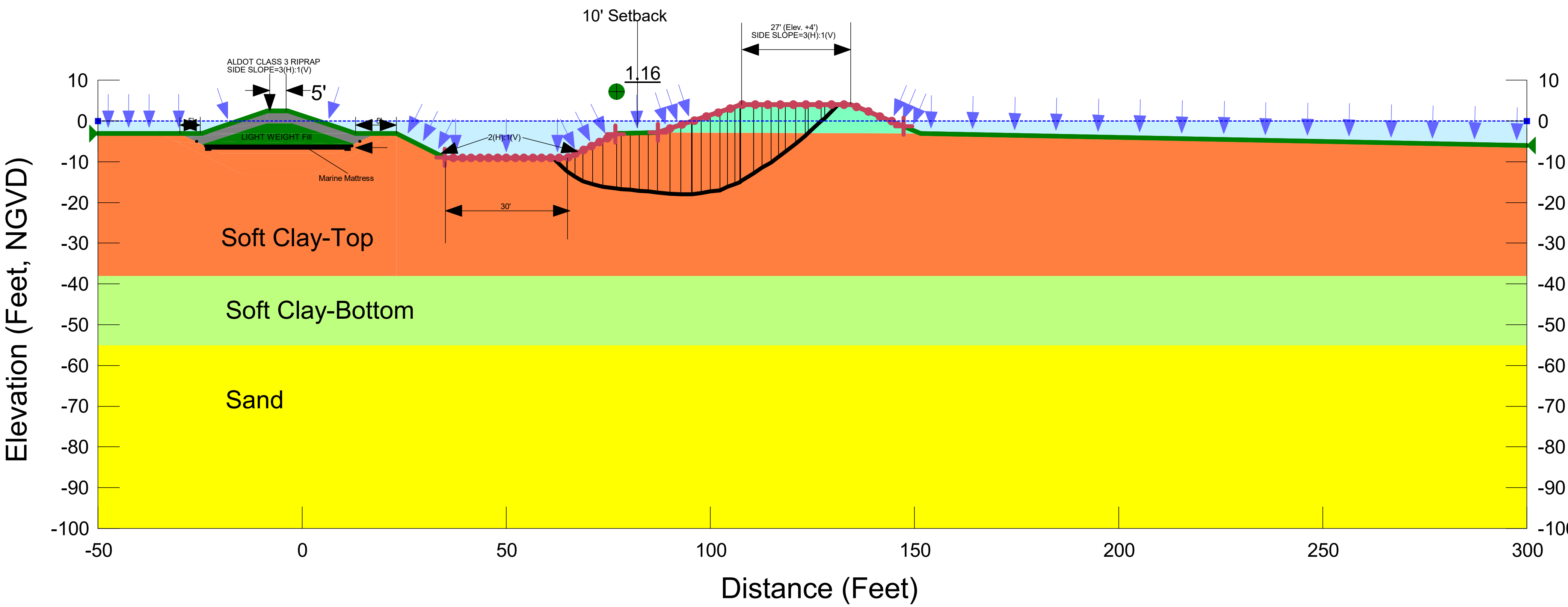
Name: SOFT CLAY-0-5' Unit Weight: 97 pcf Cohesion: 250 psf Phi: 0 °
Name: CLAY-5-11' Unit Weight: 100 pcf Cohesion: 500 psf Phi: 0 °
Name: SAND Unit Weight: 115 pcf Cohesion: 0 psf Phi: 31 °
Name: DREDGE FILL Unit Weight: 95 pcf Cohesion: 100 psf Phi: 0 °
Name: RIPRAP Unit Weight: 108 pcf Cohesion: 0 psf Phi: 38 °
Name: CLAY-11-21' Unit Weight: 200 pcf C-Top of Layer: 300 psf C-Rate of Change: 25 psf/ft Limiting C: 500 psf
Name: CLAY-21-52' Unit Weight: 200 pcf Cohesion: 1000 psf Phi: 0 °

Run-4

Restoration Of Deer River-Soft Area
Proposed Continuous Rock Dike

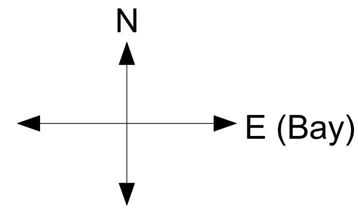


Short Term Stability
Factor of Safety: 1.16



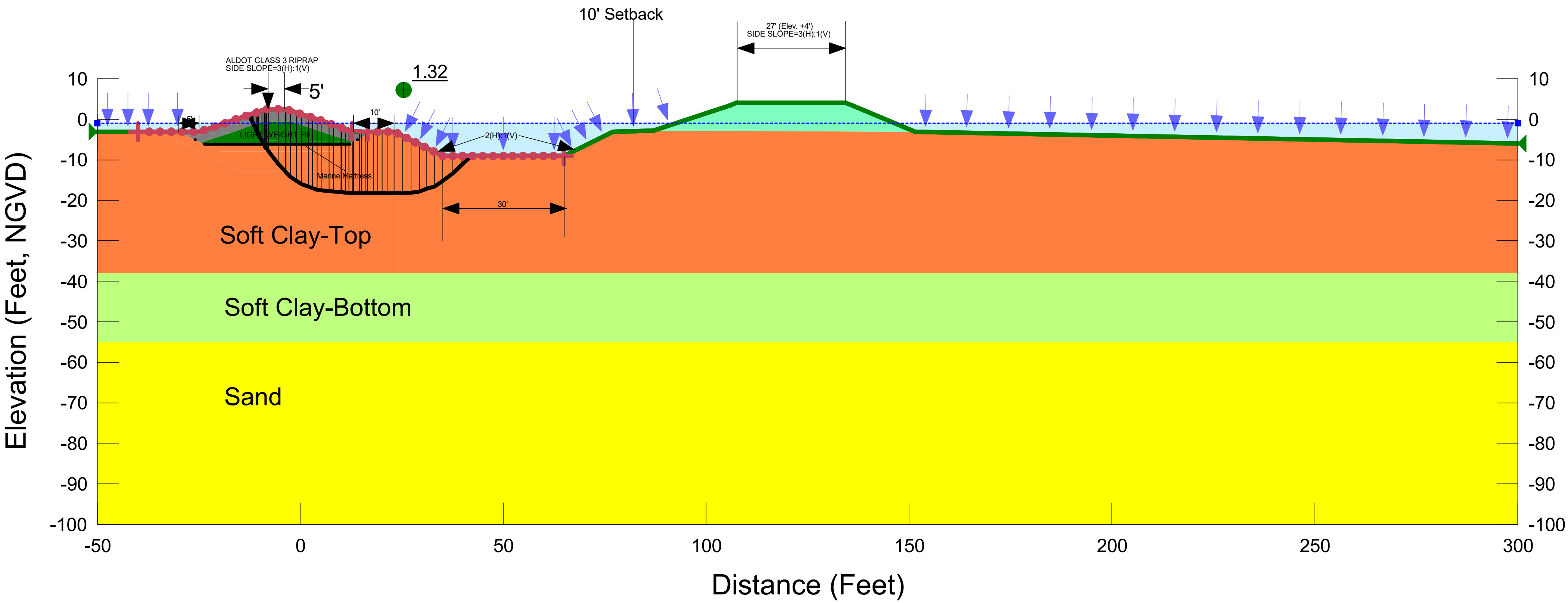
Name: SOFT CLAY-TOP Unit Weight: 97 pcf C-Top of Layer: 100 psf C-Rate of Change: 3 psf/ft Limiting C: 200 psf
Name: SOFT CLAY-BOT Unit Weight: 100 pcf C-Top of Layer: 200 psf C-Rate of Change: 6 psf/ft Limiting C: 300 psf
Name: SAND Unit Weight: 115 pcf Cohesion: 0 psf Phi: 31 ° Vol. WC. Function: Silty Sand
Name: DREDGE FILL Unit Weight: 95 pcf Cohesion: 75 psf Phi: 0 ° Vol. WC. Function: Silty Clay
Name: RIPRAP Unit Weight: 108 pcf Cohesion: 0 psf Phi: 38 ° Vol. WC. Function: Rip Rap
Name: LIGHT WEIGHT FILL Unit Weight: 75 pcf Cohesion: 0 psf Phi: 35 ° Vol. WC. Function: Coarse-Grained Fill

Run-5



Restoration Of Deer River-Soft Area
Proposed Continuous Rock Dike

Short Term Stability
Factor of Safety: 1.32



Name: SOFT CLAY-TOP Unit Weight: 97 pcf C-Top of Layer: 75 psf C-Rate of Change: 3 psf/ft Limiting C: 200 psf
Name: SOFT CLAY-BOT Unit Weight: 100 pcf C-Top of Layer: 200 psf C-Rate of Change: 6 psf/ft Limiting C: 300 psf
Name: SAND Unit Weight: 115 pcf Cohesion: 0 psf Phi: 31 ° Vol. WC. Function: Silty Sand
Name: DREDGE FILL Unit Weight: 95 pcf Cohesion: 75 psf Phi: 0 ° Vol. WC. Function: Silty Clay
Name: RIPRAP Unit Weight: 108 pcf Cohesion: 0 psf Phi: 38 ° Vol. WC. Function: Rip Rap
Name: LIGHT WEIGHT FILL Unit Weight: 75 pcf Cohesion: 0 psf Phi: 35 ° Vol. WC. Function: Coarse-Grained Fill

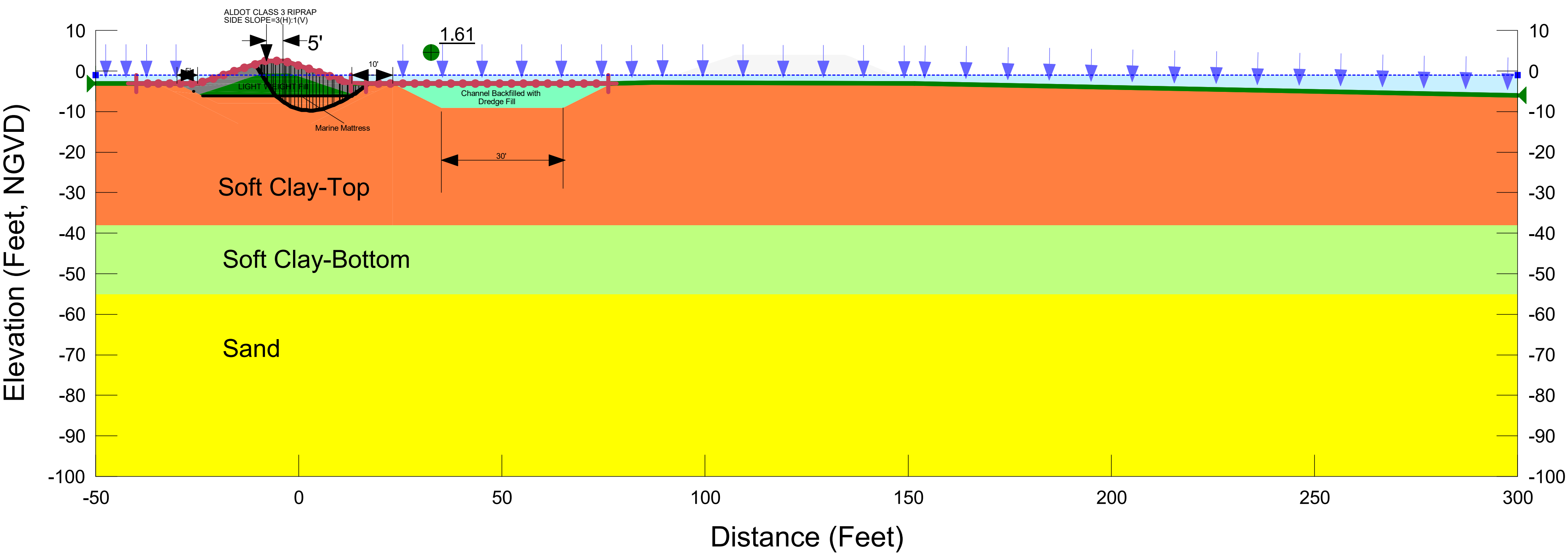
Run-6

Restoration Of Deer River-Soft Area
Proposed Continuous Rock Dike

N

E (Bay)

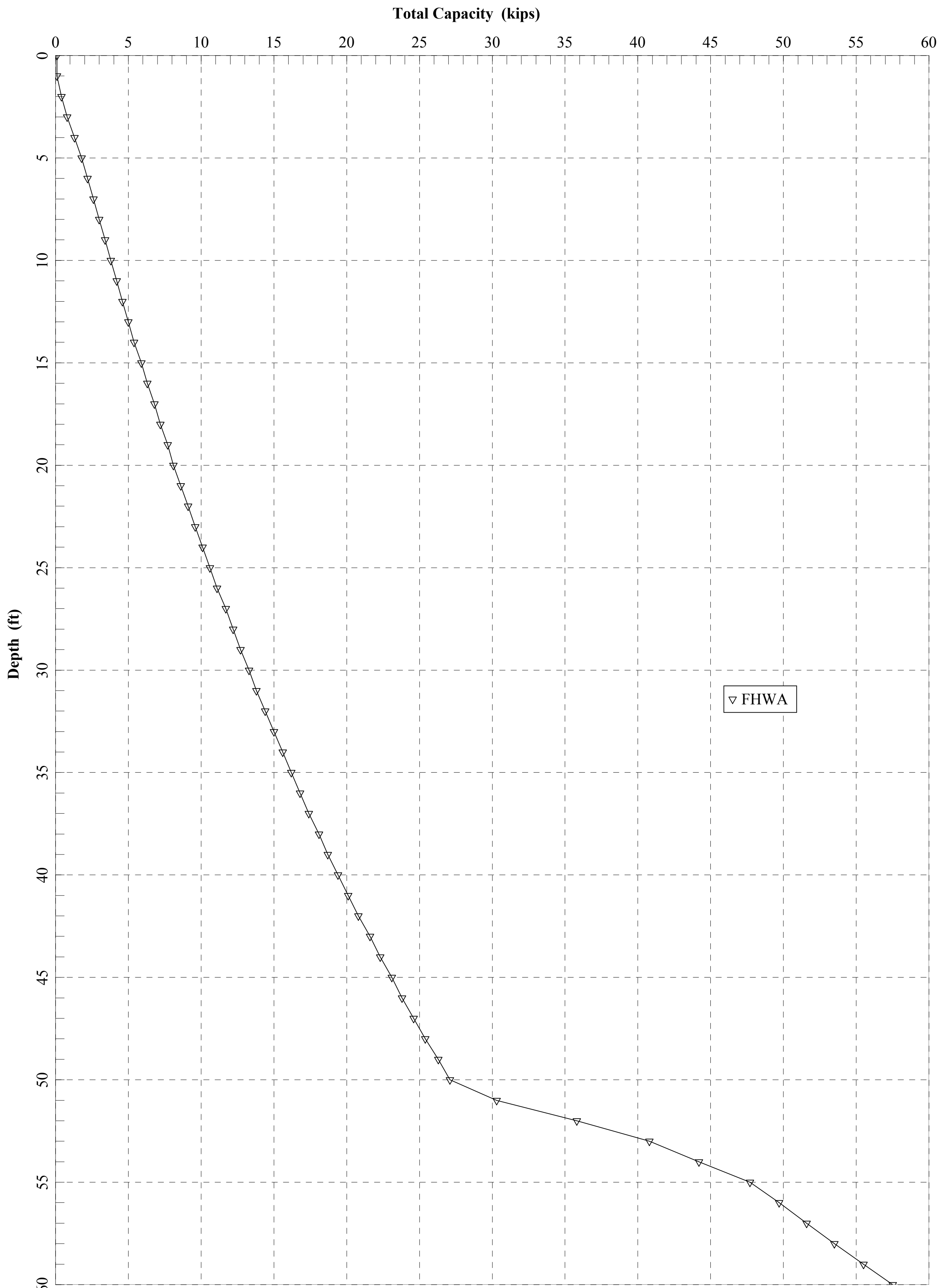
Long Term Stability
Factor of Safety: 1.61



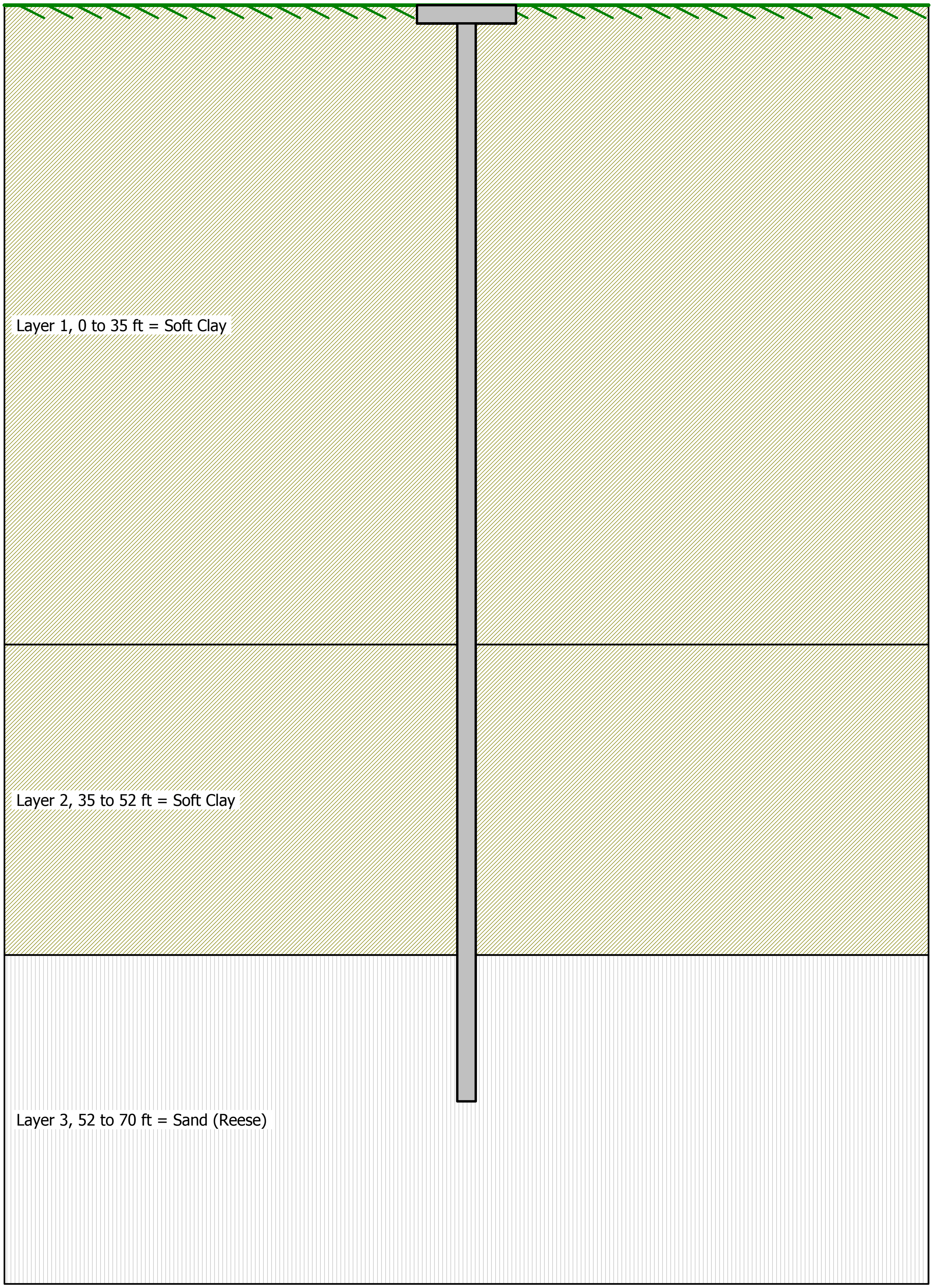
Name: SOFT CLAY-TOP Unit Weight: 97 pcf C-Top of Layer: 75 psf C-Rate of Change: 3 psf/ft Limiting C: 200 psf
Name: SOFT CLAY-BOT Unit Weight: 100 pcf C-Top of Layer: 200 psf C-Rate of Change: 6 psf/ft Limiting C: 300 psf
Name: SAND Unit Weight: 115 pcf Cohesion: 0 psf Phi: 31 ° Vol. WC. Function: Silty Sand
Name: DREDGE FILL Unit Weight: 95 pcf Cohesion: 75 psf Phi: 0 ° Vol. WC. Function: Silty Clay
Name: RIPRAP Unit Weight: 108 pcf Cohesion: 0 psf Phi: 38 ° Vol. WC. Function: Rip Rap
Name: LIGHT WEIGHT FILL Unit Weight: 75 pcf Cohesion: 0 psf Phi: 35 ° Vol. WC. Function: Coarse-Grained Fill

APPENDIX C

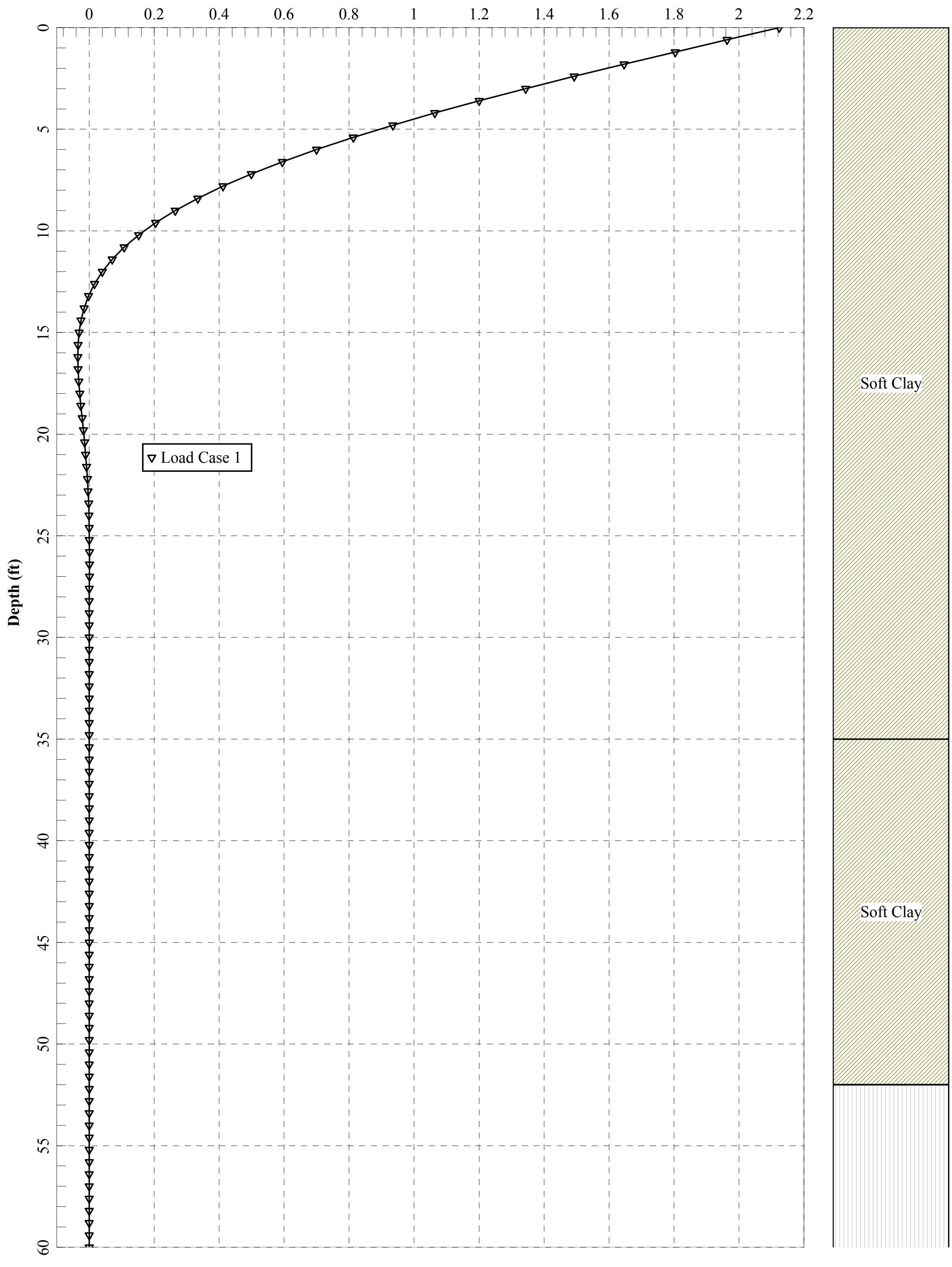
Axial and Lateral Load Analysis of Reef Maker Pile

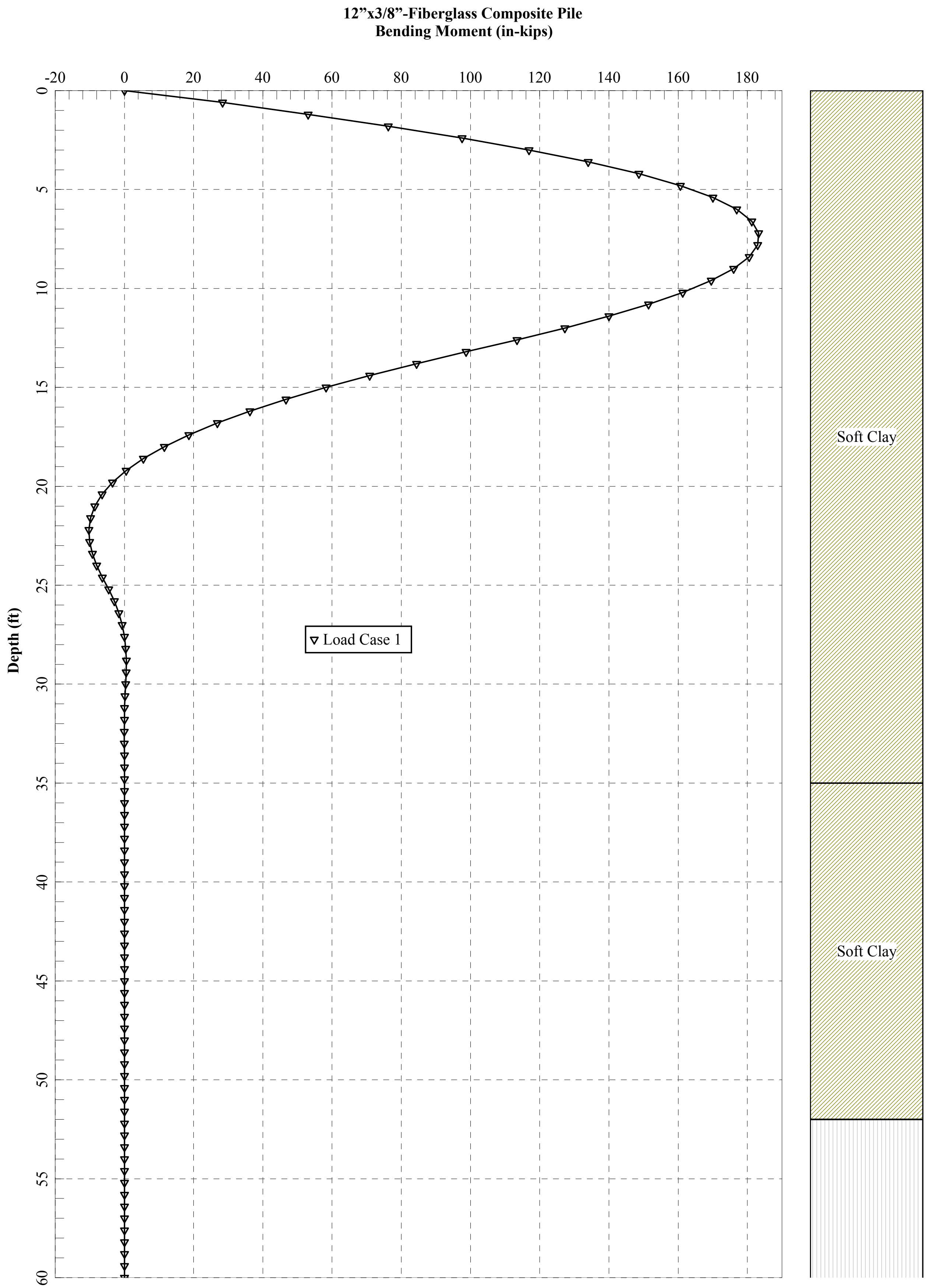


Total Capacity (kips)
0
5
10
15
20
25
30
35
40
45
50
55
60
Depth (ft)
FHWA

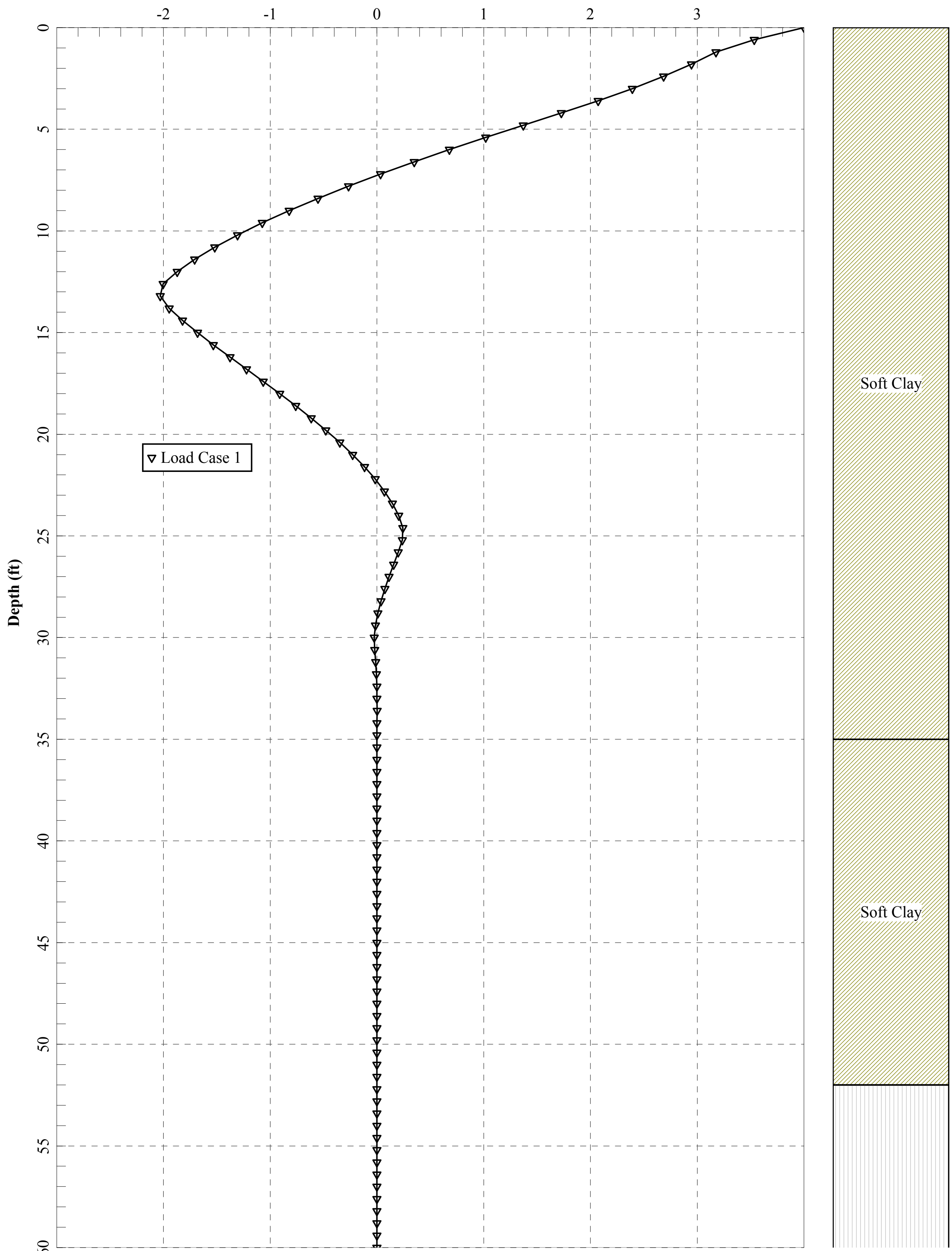

Layer 1, 0 to 35 ft = Soft Clay
Layer 2, 35 to 52 ft = Soft Clay
Layer 3, 52 to 70 ft = Sand (Reese)

12”x3/8”-Fiberglass Composite Pile
Lateral Pile Deflection (inches)




12”x3/8”-Fiberglass Composite Pile
Bending Moment (in-kips)
-20
0
20
40
60
80
100
120
140
160
180
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
Load Case 1
Soft Clay
Soft Clay

12”x3/8”-Fiberglass Composite Pile
Shear Force (kips)



==

LPile for Windows, Version 2016-09.010

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

==

This copy of LPile is being used by:

Thompson Engineering
Mobile

Serial Number of Security Device: 253582197

Files Used for Analysis

Path to file locations:
\Users\dsikdar\Desktop\Lpile Projects\Deer River\

Name of input data file:
Deer River-Boring B-6.lp9d

Name of output report file:
Deer River-Boring B-6.lp9o

Name of plot output file:
Deer River-Boring B-6.lp9p

Name of runtime message file:
Deer River-Boring B-6.lp9r

Date and Time of Analysis

Date: April 21, 2020   Time: 20:46:56

Problem Title

Project Name: Deer River

Job Number: 19-1101-0184

Client: Deer River Coastal Restoration

Engineer: DS /CC

Description: Reef Maker Pile-Boring: B-6

Program Options and Settings

Computational Options:
 - Use unfactored loads in computations (conventional analysis)
Engineering Units Used for Data Input and Computations:
 - US Customary System Units (pounds, feet, inches)

Analysis Control Options:

```
 - Maximum number of iterations allowed                    =         1000
 - Deflection tolerance for convergence                    =   1.0000E-04 in
 - Maximum allowable deflection                            =     100.0000 in
 - Number of pile increments                               =          100

Loading Type and Number of Cycles of Loading:
 - Static loading specified

 - Use of p-y modification factors for p-y curves not selected
 - No distributed lateral loads are entered
 - Loading by lateral soil movements acting on pile not selected
 - Input of shear resistance at the pile tip not selected
 - Computation of pile-head foundation stiffness matrix not selected
 - Push-over analysis of pile not selected
 - Buckling analysis of pile not selected

Output Options:
 - Output files use decimal points to denote decimal symbols.
 - Values of pile-head deflection, bending moment, shear force, and
   soil reaction are printed for full length of pile.
 - Printing Increment (nodal spacing of output points) = 1
 - No p-y curves to be computed and reported for user-specified depths
 - Print using wide report formats




--------------------------------------------------------------------------------
                     Pile Structural Properties and Geometry
--------------------------------------------------------------------------------

Number of pile sections defined                            =            2
Total length of pile                                       =       60.000 ft
Depth of ground surface below top of pile                  =       0.0000 ft

Pile diameters used for p-y curve computations are defined using 4 points.

p-y curves are computed using pile diameter values interpolated with depth over
the length of the pile. A summary of values of pile diameter vs. depth follows.

            Depth Below          Pile
Point        Pile Head         Diameter
 No.           feet             inches
```

1	0.000	65.0000
2	1.000	65.0000
3	1.000	12.0000
4	60.000	12.0000

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is a round drilled shaft, bored pile, or CIDH pile
Length of section = 1.000000 ft
Shaft Diameter = 65.000000 in
Shear capacity of section = 0.0000 lbs

Pile Section No. 2:

Section 2 is a steel pipe pile
Length of section = 59.000000 ft
Pile diameter = 12.000000 in
Shear capacity of section = 0.0000 lbs

Ground Slope and Pile Batter Angles

Ground Slope Angle = 0.000 degrees
= 0.000 radians

Pile Batter Angle = 0.000 degrees
= 0.000 radians

Soil and Rock Layering Information

The soil profile is modelled using 3 layers

```
Layer 1 is soft clay, p-y criteria by Matlock, 1970

   Distance from top of pile to top of layer          =      0.0000 ft
   Distance from top of pile to bottom of layer       =   35.000000 ft
   Effective unit weight at top of layer              =   97.000000 pcf
   Effective unit weight at bottom of layer           =   97.000000 pcf
   Undrained cohesion at top of layer                 =  100.000000 psf
   Undrained cohesion at bottom of layer              =  200.000000 psf
   Epsilon-50 at top of layer                         =    0.020000
   Epsilon-50 at bottom of layer                      =    0.018000


Layer 2 is soft clay, p-y criteria by Matlock, 1970

   Distance from top of pile to top of layer          =   35.000000 ft
   Distance from top of pile to bottom of layer       =   52.000000 ft
   Effective unit weight at top of layer              =  100.000000 pcf
   Effective unit weight at bottom of layer           =  100.000000 pcf
   Undrained cohesion at top of layer                 =  200.000000 psf
   Undrained cohesion at bottom of layer              =  300.000000 psf
   Epsilon-50 at top of layer                         =    0.018000
   Epsilon-50 at bottom of layer                      =    0.016000


Layer 3 is sand, p-y criteria by Reese et al., 1974

   Distance from top of pile to top of layer          =   52.000000 ft
   Distance from top of pile to bottom of layer       =   70.000000 ft
   Effective unit weight at top of layer              =  115.000000 pcf
   Effective unit weight at bottom of layer           =  115.000000 pcf
   Friction angle at top of layer                     =   32.000000 deg.
   Friction angle at bottom of layer                  =   32.000000 deg.
   Subgrade k at top of layer                         =   60.000000 pci
   Subgrade k at bottom of layer                      =   60.000000 pci


 (Depth of the lowest soil layer extends 10.000 ft below the pile tip)



--------------------------------------------------------------------------------
                        Summary of Input Soil Properties
```

Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Undrained Cohesion psf	Angle of Friction deg.	E50 or krm	kpy pci
1	Soft	0.00	97.0000	100.0000	--	0.02000	--
	Clay	35.0000	97.0000	200.0000	--	0.01800	--
2	Soft	35.0000	100.0000	200.0000	--	0.01800	--
	Clay	52.0000	100.0000	300.0000	--	0.01600	--
3	Sand	52.0000	115.0000	--	32.0000	--	60.0000
	(Reese, et al.)	70.0000	115.0000	--	32.0000	--	60.0000

Static Loading Type

Static loading criteria were used when computing p-y curves for all analyses.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length
1	1	V = 4000. lbs	M = 0.0000 in-lbs	6743.	No

V = shear force applied normal to pile axis
M = bending moment applied to pile head
y = lateral deflection normal to pile axis
S = pile slope relative to original pile batter angle
R = rotational stiffness applied to pile head
Values of top y vs. pile lengths can be computed only for load types with
specified shear loading (Load Types 1, 2, and 3).
Thrust force is assumed to be acting axially for all pile batter angles.

Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness

---

Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 2

Pile Section No. 1:

Dimensions and Properties of Drilled Shaft (Bored Pile):

Length of Section	=	1.000000	ft
Shaft Diameter	=	65.000000	in
Concrete Cover Thickness	=	3.000000	in
Number of Reinforcing Bars	=	8	bars
Yield Stress of Reinforcing Bars	=	60000.	psi
Modulus of Elasticity of Reinforcing Bars	=	29000000.	psi
Gross Area of Shaft	=	3318.	sq. in.
Total Area of Reinforcing Steel	=	32.000000	sq. in.
Area Ratio of Steel Reinforcement	=	0.96	percent
Edge-to-Edge Bar Spacing	=	19.457606	in
Maximum Concrete Aggregate Size	=	0.750000	in
Ratio of Bar Spacing to Aggregate Size	=	25.94	
Offset of Center of Rebar Cage from Center of Pile	=	0.0000	in

Axial Structural Capacities:

Nom. Axial Structural Capacity = 0.85 Fc Ac + Fy As	=	13093.445	kips
Tensile Load for Cracking of Concrete	=	-1473.870	kips
Nominal Axial Tensile Capacity	=	-1920.000	kips

Reinforcing Bar Dimensions and Positions Used in Computations:

Bar Number	Bar Diam. inches	Bar Area sq. in.	X inches	Y inches
1	2.257000	4.000000	28.371500	0.00000

2	2.257000	4.000000	20.061680	20.061680
3	2.257000	4.000000	0.00000	28.371500
4	2.257000	4.000000	-20.061680	20.061680
5	2.257000	4.000000	-28.371500	0.00000
6	2.257000	4.000000	-20.061680	-20.061680
7	2.257000	4.000000	0.00000	-28.371500
8	2.257000	4.000000	20.061680	-20.061680

NOTE: The positions of the above rebars were computed by LPile

Minimum spacing between any two bars not equal to zero = 19.458 inches
between bars 1 and 2.

Ratio of bar spacing to maximum aggregate size = 25.94

Concrete Properties:

Compressive Strength of Concrete	=	4000. psi
Modulus of Elasticity of Concrete	=	3604997. psi
Modulus of Rupture of Concrete	=	-474.341649 psi
Compression Strain at Peak Stress	=	0.001886
Tensile Strain at Fracture of Concrete	=	-0.0001154
Maximum Coarse Aggregate Size	=	0.750000 in

Number of Axial Thrust Force Values Determined from Pile-head Loadings = 1

Number	Axial Thrust Force kips
1	6.743

Definitions of Run Messages and Notes:

C = concrete in section has cracked in tension.
Y = stress in reinforcing steel has reached yield stress.
T = ACI 318 criteria for tension-controlled section met, tensile strain in
reinforcement exceeds 0.005 while simultaneously compressive strain in

concrete more than 0.003. See ACI 318, Section 10.3.4.
Z = depth of tensile zone in concrete section is less than 10 percent of section depth.

Bending Stiffness (EI) = Computed Bending Moment / Curvature.
Position of neutral axis is measured from edge of compression side of pile.
Compressive stresses and strains are positive in sign.
Tensile stresses and strains are negative in sign.

Axial Thrust Force = 6.743 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in2	Depth to N Axis in	Max Comp Strain in/in	Max Tens Strain in/in	Max Conc Stress ksi	Max Steel Stress ksi	Run Msg
4.16667E-07	1678.	4027064895.	33.5917264	0.00001400	-0.00001309	0.0585715	0.4019729	
8.33333E-07	3348.	4018163363.	33.0474241	0.00002754	-0.00002663	0.1148153	0.7907919	
0.00000125	5012.	4009253501.	32.8659962	0.00004108	-0.00004017	0.1706549	1.1796111	
0.00000167	6667.	4000341557.	32.7752869	0.00005463	-0.00005371	0.2260904	1.5684305	
0.00000208	8315.	3991428780.	32.7208649	0.00006817	-0.00006725	0.2811217	1.9572502	
0.00000250	9956.	3982515586.	32.6845867	0.00008171	-0.00008079	0.3357488	2.3460700	
0.00000292	11590.	3973602154.	32.6586763	0.00009525	-0.00009433	0.3899718	2.7348901	
0.00000333	13216.	3964688573.	32.6392459	0.0001088	-0.0001079	0.4437907	3.1237104	
0.00000375	13216.	3524167621.	15.9830818	0.00005994	-0.0001838	0.2451563	-5.2952461	C
0.00000417	13216.	3171750858.	15.9379279	0.00006641	-0.0002044	0.2711470	-5.8890629	C
0.00000458	13216.	2883409871.	15.9007543	0.00007288	-0.0002250	0.2970431	-6.4829101	C
0.00000500	13216.	2643125715.	15.8700706	0.00007935	-0.0002456	0.3228549	-7.0767149	C
0.00000542	13216.	2439808353.	15.8448272	0.00008583	-0.0002663	0.3485923	-7.6704064	C
0.00000583	13216.	2265536327.	15.8238605	0.00009231	-0.0002869	0.3742551	-8.2639845	C
0.00000625	13216.	2114500572.	15.8063171	0.00009879	-0.0003075	0.3998432	-8.8574488	C
0.00000667	13216.	1982344287.	15.7915570	0.0001053	-0.0003281	0.4253565	-9.4507990	C
0.00000708	13216.	1865735799.	15.7790908	0.0001118	-0.0003486	0.4507948	-10.0440347	C
0.00000750	13216.	1762083810.	15.7685378	0.0001183	-0.0003692	0.4761580	-10.6371555	C
0.00000792	13216.	1669342557.	15.7595976	0.0001248	-0.0003898	0.5014460	-11.2301612	C
0.00000833	13216.	1585875429.	15.7520296	0.0001313	-0.0004104	0.5266587	-11.8230512	C
0.00000875	13216.	1510357552.	15.7456394	0.0001378	-0.0004310	0.5517960	-12.4158253	C
0.00000917	13216.	1441704936.	15.7402677	0.0001443	-0.0004515	0.5768578	-13.0084830	C
0.00000958	13216.	1379022112.	15.7357830	0.0001508	-0.0004721	0.6018438	-13.6010240	C
0.00001000	13216.	1321562858.	15.7320757	0.0001573	-0.0004927	0.6267541	-14.1934480	C
0.00001042	13216.	1268700343.	15.7290539	0.0001638	-0.0005132	0.6515884	-14.7857545	C
0.00001083	13216.	1219904176.	15.7266395	0.0001704	-0.0005338	0.6763467	-15.3779432	C
0.00001125	13216.	1174722540.	15.7247663	0.0001769	-0.0005543	0.7010289	-15.9700137	C

0.00001167	13216.	1132768164.	15.7233774	0.0001834	-0.0005749	0.7256347	-16.5619656	C
0.00001208	13216.	1093707193.	15.7224238	0.0001900	-0.0005954	0.7501641	-17.1537985	C
0.00001250	13216.	1057250286.	15.7218631	0.0001965	-0.0006160	0.7746169	-17.7455121	C
0.00001292	13216.	1023145438.	15.7216583	0.0002031	-0.0006365	0.7989931	-18.3371059	C
0.00001333	13216.	991172143.	15.7217770	0.0002096	-0.0006570	0.8232925	-18.9285795	C
0.00001375	13216.	961136624.	15.7221908	0.0002162	-0.0006776	0.8475149	-19.5199326	C
0.00001417	13216.	932867900.	15.7228747	0.0002227	-0.0006981	0.8716603	-20.1111648	C
0.00001458	13216.	906214531.	15.7238064	0.0002293	-0.0007186	0.8957285	-20.7022756	C
0.00001500	13216.	881041905.	15.7249662	0.0002359	-0.0007391	0.9197194	-21.2932647	C
0.00001542	13216.	857229962.	15.7263365	0.0002424	-0.0007596	0.9436328	-21.8841316	C
0.00001583	13216.	834671279.	15.7279016	0.0002490	-0.0007801	0.9674686	-22.4748760	C
0.00001625	13216.	813269451.	15.7296473	0.0002556	-0.0008006	0.9912267	-23.0654974	C
0.00001708	13426.	785894461.	15.7336312	0.0002688	-0.0008416	1.0385093	-24.2463697	C
0.00001792	14069.	785230487.	15.7382008	0.0002820	-0.0008826	1.0854795	-25.4267453	C
0.00001875	14711.	784593038.	15.7432840	0.0002952	-0.0009236	1.1321361	-26.6066206	C
0.00001958	15353.	783978546.	15.7488213	0.0003084	-0.0009645	1.1784781	-27.7859924	C
0.00002042	15994.	783384021.	15.7547629	0.0003217	-0.0010054	1.2245044	-28.9648571	C
0.00002125	16635.	782806944.	15.7610669	0.0003349	-0.0010463	1.2702138	-30.1432113	C
0.00002208	17275.	782245172.	15.7676978	0.0003482	-0.0010872	1.3156054	-31.3210515	C
0.00002292	17914.	781696871.	15.7746253	0.0003615	-0.0011281	1.3606778	-32.4983741	C
0.00002375	18553.	781160466.	15.7818234	0.0003748	-0.0011689	1.4054301	-33.6751755	C
0.00002458	19191.	780634589.	15.7892698	0.0003882	-0.0012098	1.4498609	-34.8514520	C
0.00002542	19828.	780118054.	15.7969450	0.0004015	-0.0012506	1.4939693	-36.0271998	C
0.00002625	20465.	779609822.	15.8048323	0.0004149	-0.0012914	1.5377539	-37.2024153	C
0.00002708	21101.	779108979.	15.8129169	0.0004283	-0.0013322	1.5812137	-38.3770947	C
0.00002792	21736.	778614721.	15.8211858	0.0004417	-0.0013729	1.6243473	-39.5512340	C
0.00002875	22371.	778126333.	15.8296277	0.0004551	-0.0014136	1.6671536	-40.7248294	C
0.00002958	23005.	777643181.	15.8382327	0.0004685	-0.0014544	1.7096314	-41.8978769	C
0.00003042	23639.	777164697.	15.8469918	0.0004820	-0.0014951	1.7517794	-43.0703726	C
0.00003125	24272.	776690374.	15.8558973	0.0004955	-0.0015358	1.7935964	-44.2423122	C
0.00003208	24904.	776219754.	15.8649420	0.0005090	-0.0015764	1.8350810	-45.4136919	C
0.00003292	25535.	775752425.	15.8741199	0.0005225	-0.0016171	1.8762321	-46.5845073	C
0.00003375	26166.	775288013.	15.8834253	0.0005361	-0.0016577	1.9170483	-47.7547543	C
0.00003458	26796.	774826181.	15.8928533	0.0005496	-0.0016983	1.9575284	-48.9244285	C
0.00003542	27425.	774366619.	15.9023995	0.0005632	-0.0017389	1.9976709	-50.0935257	C
0.00003625	28054.	773909046.	15.9120599	0.0005768	-0.0017794	2.0374746	-51.2620414	C
0.00003708	28682.	773453203.	15.9218310	0.0005904	-0.0018200	2.0769381	-52.4299712	C
0.00003792	29310.	772999347.	15.9313946	0.0006041	-0.0018605	2.1160254	-53.5976560	C
0.00003875	29936.	772546969.	15.9409074	0.0006177	-0.0019010	2.1547514	-54.7649374	C
0.00003958	30562.	772095784.	15.9505151	0.0006314	-0.0019415	2.1931313	-55.9316491	C
0.00004042	31187.	771645570.	15.9602155	0.0006451	-0.0019820	2.2311635	-57.0977878	C
0.00004125	31812.	771196156.	15.9700067	0.0006588	-0.0020225	2.2688468	-58.2633491	C
0.00004208	32436.	770747383.	15.9798868	0.0006725	-0.0020629	2.3061797	-59.4283287	C

0.00004292	33059.	770299104.	15.9898542	0.0006862	-0.0021034	2.3431608	-60.0000000	CY
0.00004375	33681.	769851180.	15.9999075	0.0007000	-0.0021438	2.3797887	-60.0000000	CY
0.00004458	34303.	769403535.	16.0100453	0.0007138	-0.0021841	2.4160621	-60.0000000	CY
0.00004542	34916.	768796362.	16.0191741	0.0007275	-0.0022245	2.4518484	-60.0000000	CY
0.00004625	35472.	766962916.	16.0198396	0.0007409	-0.0022653	2.4862445	-60.0000000	CY
0.00004708	35936.	763242942.	16.0071506	0.0007537	-0.0023067	2.5186345	-60.0000000	CY
0.00004792	36359.	758800628.	15.9891833	0.0007661	-0.0023484	2.5499745	-60.0000000	CY
0.00004875	36781.	754484450.	15.9720283	0.0007786	-0.0023901	2.5810039	-60.0000000	CY
0.00004958	37203.	750303740.	15.9557614	0.0007911	-0.0024318	2.6117362	-60.0000000	CY
0.00005292	38883.	734803005.	15.8973693	0.0008412	-0.0025983	2.7314971	-60.0000000	CY
0.00005625	40437.	718884063.	15.8297846	0.0008904	-0.0027658	2.8437910	-60.0000000	CY
0.00005958	41074.	689351612.	15.6299846	0.0009313	-0.0029416	2.9327139	-60.0000000	CY
0.00006292	41559.	660533739.	15.4309640	0.0009709	-0.0031187	3.0153814	-60.0000000	CY
0.00006625	42040.	634564545.	15.2527678	0.0010105	-0.0032958	3.0948235	-60.0000000	CY
0.00006958	42513.	610970657.	15.0849001	0.0010497	-0.0034733	3.1700183	-60.0000000	CY
0.00007292	42984.	589498931.	14.9343137	0.0010890	-0.0036506	3.2422131	-60.0000000	CY
0.00007625	43453.	569870476.	14.7987715	0.0011284	-0.0038278	3.3113756	-60.0000000	CY
0.00007958	43918.	551853112.	14.6764121	0.0011680	-0.0040049	3.3774726	-60.0000000	CY
0.00008292	44379.	535226519.	14.5616684	0.0012074	-0.0041822	3.4399431	-60.0000000	CY
0.00008625	44836.	519834744.	14.4543595	0.0012467	-0.0043596	3.4989256	-60.0000000	CY
0.00008958	45290.	505559249.	14.3567439	0.0012861	-0.0045368	3.5548434	-60.0000000	CY
0.00009292	45741.	492279401.	14.2678092	0.0013257	-0.0047139	3.6076609	-60.0000000	CY
0.00009625	46190.	479891249.	14.1866842	0.0013655	-0.0048908	3.6573412	-60.0000000	CY
0.00009958	46635.	468304747.	14.1126154	0.0014054	-0.0050675	3.7038465	-60.0000000	CY
0.0001029	47078.	457441498.	14.0449484	0.0014455	-0.0052441	3.7471378	-60.0000000	CY
0.0001063	47517.	447217891.	13.9827805	0.0014857	-0.0054206	3.7871404	-60.0000000	CY
0.0001096	47890.	437017087.	13.9089582	0.0015242	-0.0055987	3.8221913	-60.0000000	CY
0.0001129	48134.	426279027.	13.8127909	0.0015597	-0.0057799	3.8516383	-60.0000000	CY
0.0001163	48253.	415081580.	13.6961517	0.0015922	-0.0059641	3.8761775	-60.0000000	CY
0.0001196	48317.	404041330.	13.5748417	0.0016233	-0.0061496	3.8975950	-60.0000000	CY
0.0001229	48379.	393593140.	13.4609645	0.0016546	-0.0063350	3.9170445	-60.0000000	CY
0.0001263	48441.	383689979.	13.3539386	0.0016859	-0.0065203	3.9345049	-60.0000000	CY
0.0001296	48496.	374247812.	13.2468608	0.0017166	-0.0067063	3.9495621	-60.0000000	CY
0.0001329	48549.	365257448.	13.1434872	0.0017470	-0.0068926	3.9625645	-60.0000000	CY
0.0001363	48601.	356701318.	13.0459212	0.0017775	-0.0070787	3.9736675	-60.0000000	CY
0.0001396	48652.	348548208.	12.9537543	0.0018081	-0.0072648	3.9828511	-60.0000000	CY
0.0001429	48702.	340769814.	12.8666161	0.0018389	-0.0074507	3.9900949	-60.0000000	CY
0.0001462	48751.	333340409.	12.7841702	0.0018697	-0.0076366	3.9953783	-60.0000000	CY
0.0001496	48800.	326236561.	12.7061106	0.0019006	-0.0078223	3.9986802	-60.0000000	CY
0.0001529	48844.	319416431.	12.6281387	0.0019311	-0.0080085	3.9999728	-60.0000000	CY
0.0001562	48885.	312866096.	12.5515631	0.0019612	-0.0081951	3.9932593	-60.0000000	CY
0.0001596	48926.	306584677.	12.4788382	0.0019914	-0.0083815	3.9971386	-60.0000000	CY
0.0001629	48966.	300556085.	12.4096956	0.0020217	-0.0085678	3.9993861	-60.0000000	CY

0.0001662	49005.	294764447.	12.3439800	0.0020522	-0.0087541	3.9988017	-60.0000000	CY
0.0001696	49042.	289193172.	12.2817249	0.0020828	-0.0089401	3.9929717	-60.0000000	CY
0.0001729	49080.	283833290.	12.2223925	0.0021135	-0.0091261	3.9967454	-60.0000000	CY
0.0001762	49116.	278672751.	12.1658228	0.0021442	-0.0093120	3.9990900	60.0000000	CY
0.0001796	49152.	273700397.	12.1118678	0.0021751	-0.0094978	3.9999899	60.0000000	CY
0.0001829	49187.	268902284.	12.0607418	0.0022061	-0.0096835	3.9929143	60.0000000	CY
0.0002029	49366.	243280095.	11.7737658	0.0023891	-0.0108005	3.9907498	60.0000000	CY
0.0002229	49524.	222165870.	11.5491420	0.0025745	-0.0119151	3.9886523	60.0000000	CY
0.0002429	49669.	204471272.	11.3722088	0.0027625	-0.0130271	3.9975542	60.0000000	CY
0.0002629	49788.	189366451.	11.2155135	0.0029487	-0.0141408	3.9973720	60.0000000	CY
0.0002829	49884.	176321869.	11.0970028	0.0031395	-0.0152501	3.9821624	60.0000000	CYT
0.0003029	49960.	164931459.	11.0089834	0.0033348	-0.0163548	3.9991069	60.0000000	CYT
0.0003229	50004.	154850688.	10.9549424	0.0035375	-0.0174520	3.9801579	60.0000000	CYT
0.0003429	50037.	145915474.	10.9157101	0.0037432	-0.0185464	3.9984016	60.0000000	CYT
0.0003629	50037.	137874208.	10.9740292	0.0039827	-0.0196069	3.9764859	60.0000000	CYT

Summary of Results for Nominal (Unfactored) Moment Capacity for Section 1

Moment values interpolated at maximum compressive strain = 0.003
or maximum developed moment if pile fails at smaller strains.

Load No.	Axial Thrust kips	Nominal Mom. Cap. in-kip	Max. Comp. Strain
1	6.743	49813.602	0.00300000

Note that the values of moment capacity in the table above are not
factored by a strength reduction factor (phi-factor).

In ACI 318, the value of the strength reduction factor depends on whether
the transverse reinforcing steel bars are tied hoops (0.65) or spirals (0.70).

The above values should be multiplied by the appropriate strength reduction
factor to compute ultimate moment capacity according to ACI 318, Section
9.3.2.2 or the value required by the design standard being followed.

The following table presents factored moment capacities and corresponding
bending stiffnesses computed for common resistance factor values used for
reinforced concrete sections.

Axial Load No.	Resist. Factor for Moment	Nominal Moment Cap in-kips	Ult. (Fac) Ax. Thrust kips	Ult. (Fac) Moment Cap in-kips	Bend. Stiff. at Ult Mom kip-in^2
1	0.65	49814.	4.382950	32379.	770788232.
1	0.70	49814.	4.720100	34870.	768842521.
1	0.75	49814.	5.057250	37360.	748849908.

Pile Section No. 2:

Dimensions and Properties of Steel Pipe Pile:

Length of Section	=	59.000000 ft
Outer Diameter of Pipe	=	12.000000 in
Pipe Wall Thickness	=	0.375000 in
Yield Stress of Pipe	=	54.289000 ksi
Elastic Modulus	=	4588. ksi
Cross-sectional Area	=	13.695380 sq. in.
Moment of Inertia	=	231.591024 in^4
Elastic Bending Stiffness	=	1062537. kip-in^2
Plastic Modulus, Z	=	50.695312in^3
Plastic Moment Capacity = Fy Z	=	2752.in-kip

Axial Structural Capacities:

Nom. Axial Structural Capacity = Fy As	=	743.509 kips
Nominal Axial Tensile Capacity	=	-743.509 kips

Number of Axial Thrust Force Values Determined from Pile-head Loadings = 1

Number	Axial Thrust Force kips
1	6.743

Definition of Run Messages:

Y = part of pipe section has yielded.

Axial Thrust Force = 6.743 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in2	Depth to N Axis in	Max Total Stress ksi	Run Msg
0.00004168	44.2912224	1062536.	8.5744390	1.6283678	
0.00008337	88.5824447	1062536.	7.2872195	2.7643797	
0.0001251	132.8736671	1062536.	6.8581463	3.9003917	
0.0001667	177.1648894	1062536.	6.6436098	5.0364036	
0.0002084	221.4561118	1062536.	6.5148878	6.1724156	
0.0002501	265.7473341	1062536.	6.4290732	7.3084275	
0.0002918	310.0385565	1062536.	6.3677770	8.4444395	
0.0003335	354.3297788	1062536.	6.3218049	9.5804514	
0.0003752	398.6210012	1062536.	6.2860488	10.7164634	
0.0004168	442.9122235	1062536.	6.2574439	11.8524753	
0.0004585	487.2034459	1062536.	6.2340399	12.9884873	
0.0005002	531.4946682	1062536.	6.2145366	14.1244992	
0.0005419	575.7858906	1062536.	6.1980338	15.2605112	
0.0005836	620.0771129	1062536.	6.1838885	16.3965231	
0.0006253	664.3683353	1062536.	6.1716293	17.5325351	
0.0006670	708.6595576	1062536.	6.1609024	18.6685471	
0.0007086	752.9507800	1062536.	6.1514376	19.8045590	
0.0007503	797.2420023	1062536.	6.1430244	20.9405710	
0.0007920	841.5332247	1062536.	6.1354968	22.0765829	
0.0008337	885.8244470	1062536.	6.1287220	23.2125949	
0.0008754	930.1156694	1062536.	6.1225923	24.3486068	
0.0009171	974.4068917	1062536.	6.1170200	25.4846188	
0.0009587	1019.	1062536.	6.1119321	26.6206307	
0.0010004	1063.	1062536.	6.1072683	27.7566427	
0.0010421	1107.	1062536.	6.1029776	28.8926546	
0.0010838	1152.	1062536.	6.0990169	30.0286666	
0.0011255	1196.	1062536.	6.0953496	31.1646785	
0.0011672	1240.	1062536.	6.0919443	32.3006905	
0.0012088	1284.	1062536.	6.0887738	33.4367025	
0.0012505	1329.	1062536.	6.0858146	34.5727144	
0.0012922	1373.	1062536.	6.0830464	35.7087264	

0.0013339	1417.	1062536.	6.0804512	36.8447383	
0.0013756	1462.	1062536.	6.0780133	37.9807503	
0.0014173	1506.	1062536.	6.0757188	39.1167622	
0.0014590	1550.	1062536.	6.0735554	40.2527742	
0.0015006	1594.	1062536.	6.0715122	41.3887861	
0.0015423	1639.	1062536.	6.0695794	42.5247981	
0.0015840	1683.	1062536.	6.0677484	43.6608100	
0.0016257	1727.	1062536.	6.0660113	44.7968220	
0.0017091	1816.	1062536.	6.0627912	47.0688459	
0.0017924	1905.	1062536.	6.0598707	49.3408698	
0.0018758	1993.	1062536.	6.0572098	51.6128937	
0.0019592	2082.	1062536.	6.0547753	53.8849176	
0.0020425	2166.	1060232.	6.0562228	54.2890000	Y
0.0021259	2232.	1050036.	6.0614230	54.2890000	Y
0.0022093	2284.	1033702.	6.0649628	54.2890000	Y
0.0022926	2326.	1014628.	6.0671632	54.2890000	Y
0.0023760	2362.	994001.	6.0688683	54.2890000	Y
0.0024594	2393.	972947.	6.0700863	54.2890000	Y
0.0025428	2420.	951635.	6.0714885	54.2890000	Y
0.0026261	2444.	930465.	6.0723470	54.2890000	Y
0.0027095	2465.	909649.	6.0730992	54.2890000	Y
0.0027929	2484.	889278.	6.0737726	54.2890000	Y
0.0028762	2501.	869419.	6.0743883	54.2890000	Y
0.0029596	2516.	850115.	6.0749629	54.2890000	Y
0.0030430	2530.	831398.	6.0755101	54.2890000	Y
0.0031263	2543.	813284.	6.0760414	54.2890000	Y
0.0032097	2554.	795783.	6.0765664	54.2890000	Y
0.0032931	2565.	778876.	6.0768881	54.2890000	Y
0.0033764	2575.	762530.	6.0769282	54.2890000	Y
0.0034598	2584.	746738.	6.0774343	54.2890000	Y
0.0035432	2592.	731536.	6.0779612	54.2890000	Y
0.0036265	2600.	716837.	6.0777449	54.2890000	Y
0.0037099	2607.	702637.	6.0782739	54.2890000	Y
0.0037933	2613.	688961.	6.0784417	54.2890000	Y
0.0038767	2620.	675724.	6.0785850	54.2890000	Y
0.0039600	2625.	662973.	6.0790093	54.2890000	Y
0.0040434	2631.	650623.	6.0789072	54.2890000	Y
0.0041268	2636.	638718.	6.0793663	54.2890000	Y
0.0042101	2641.	627180.	6.0792517	54.2890000	Y
0.0042935	2645.	616051.	6.0795308	54.2890000	Y
0.0043769	2649.	605259.	6.0796291	54.2890000	Y
0.0044602	2653.	594844.	6.0795148	54.2890000	Y
0.0045436	2657.	584739.	6.0800498	54.2890000	Y

0.0046270	2660.	574959.	6.0797719	54.2890000	Y
0.0047103	2664.	565494.	6.0802000	54.2890000	Y
0.0047937	2667.	556302.	6.0802393	54.2890000	Y
0.0048771	2670.	547411.	6.0799641	54.2890000	Y
0.0049604	2673.	538781.	6.0806057	54.2890000	Y
0.0052939	2682.	506705.	6.0808117	54.2890000	Y
0.0056274	2691.	478125.	6.0809468	54.2890000	Y
0.0059609	2697.	452516.	6.0811870	54.2890000	Y
0.0062944	2703.	429452.	6.0815453	54.2890000	Y
0.0066278	2708.	408582.	6.0817836	54.2890000	Y
0.0069613	2712.	389608.	6.0811213	54.2890000	Y
0.0072948	2716.	372282.	6.0817271	54.2890000	Y
0.0076283	2719.	356423.	6.0820154	54.2890000	Y
0.0079617	2722.	341833.	6.0816729	54.2890000	Y
0.0082952	2724.	328386.	6.0823173	54.2890000	Y
0.0086287	2726.	315937.	6.0818767	54.2890000	Y
0.0089622	2728.	304397.	6.0820660	54.2890000	Y
0.0092956	2730.	293656.	6.0823585	54.2890000	Y
0.0096291	2731.	283646.	6.0816179	54.2890000	Y
0.0099626	2733.	274290.	6.0826071	54.2890000	Y

Summary of Results for Nominal (Unfactored) Moment Capacity for Section 2

Load No.	Axial Thrust kips	Nominal Moment Capacity in-kips
1	6.7430000000	2733.

Note that the values in the above table are not factored by a strength reduction factor for LRFD.

The value of the strength reduction factor depends on the provisions of the LRFD code being followed.

The above values should be multiplied by the appropriate strength reduction factor to compute ultimate moment capacity according to the LRFD structural design standard being followed.

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	0.00	0.00	N.A.	No	0.00	47142.
2	35.0000	35.0000	Yes	No	47142.	38250.
3	52.0000	52.0000	No	No	85392.	N.A.

Notes: The F0 integral of Layer n+1 equals the sum of the F0 and F1 integrals for Layer n. Layering correction equivalent depths are computed only for soil types with both shallow-depth and deep-depth expressions for peak lateral load transfer. These soil types are soft and stiff clays, non-liquefied sands, and cemented c-phi soil.

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 1

Pile-head conditions are Shear and Moment (Loading Type 1)

Shear force at pile head = 4000.0 lbs
Applied moment at pile head = 0.0 in-lbs
Axial thrust load on pile head = 6743.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness in-lb^2	Soil Res. p lb/inch	Soil Spr. Es*h lb/inch	Distrib. Lat. Load lb/inch
0.00	2.1239	0.6662	4000.	-0.02222	0.00	4.03E+12	-58.7584	99.5961	0.00

0.6000	1.9639	28356.	3535.	-0.02222	0.00	4.03E+12	-70.4460	258.2716	0.00
1.2000	1.8039	53060.	3175.	-0.02204	1867.	1.06E+09	-29.4259	117.4519	0.00
1.8000	1.6464	76221.	2946.	-0.02161	2467.	1.06E+09	-34.1650	149.4063	0.00
2.4000	1.4927	97586.	2685.	-0.02102	3021.	1.06E+09	-38.5720	186.0471	0.00
3.0000	1.3438	116920.	2392.	-0.02029	3521.	1.06E+09	-42.6219	228.3673	0.00
3.6000	1.2006	134005.	2072.	-0.01944	3964.	1.06E+09	-46.2901	277.6133	0.00
4.2000	1.0639	148646.	1727.	-0.01848	4343.	1.06E+09	-49.5529	335.3670	0.00
4.8000	0.9344	160670.	1370.	-0.01743	4655.	1.06E+09	-49.6583	382.6393	0.00
5.4000	0.8128	170067.	1018.	-0.01631	4898.	1.06E+09	-48.1459	426.4915	0.00
6.0000	0.6995	176912.	677.1377	-0.01514	5076.	1.06E+09	-46.5030	478.6698	0.00
6.6000	0.5948	181287.	348.7067	-0.01392	5189.	1.06E+09	-44.7278	541.4228	0.00
7.2000	0.4990	183285.	33.5442	-0.01269	5241.	1.06E+09	-42.8173	617.8438	0.00
7.8000	0.4121	183003.	-267.3593	-0.01145	5234.	1.06E+09	-40.7670	712.3028	0.00
8.4000	0.3341	180547.	-552.9732	-0.01022	5170.	1.06E+09	-38.5702	831.1763	0.00
9.0000	0.2650	176032.	-822.2039	-0.00901	5053.	1.06E+09	-36.2161	984.1516	0.00
9.6000	0.2044	169582.	-1074.	-0.00784	4886.	1.06E+09	-33.6880	1187.	0.00
10.2000	0.1521	161329.	-1307.	-0.00672	4672.	1.06E+09	-30.9574	1465.	0.00
10.8000	0.1077	151419.	-1519.	-0.00566	4415.	1.06E+09	-27.9744	1871.	0.00
11.4000	0.07064	140009.	-1708.	-0.00467	4120.	1.06E+09	-24.6420	2512.	0.00
12.0000	0.04043	127275.	-1872.	-0.00376	3790.	1.06E+09	-20.7400	3693.	0.00
12.6000	0.01644	113424.	-2002.	-0.00295	3431.	1.06E+09	-15.5770	6823.	0.00
13.2000	-0.00202	98729.	-2030.	-0.00223	3050.	1.06E+09	7.7954	27749.	0.00
13.8000	-0.01567	84405.	-1946.	-0.00161	2679.	1.06E+09	15.7101	7220.	0.00
14.4000	-0.02519	70867.	-1822.	-0.00108	2328.	1.06E+09	18.6487	5330.	0.00
15.0000	-0.03126	58273.	-1682.	-6.45E-04	2002.	1.06E+09	20.2976	4675.	0.00
15.6000	-0.03449	46712.	-1532.	-2.90E-04	1703.	1.06E+09	21.2386	4434.	0.00
16.2000	-0.03543	36237.	-1378.	-8.63E-06	1431.	1.06E+09	21.6991	4409.	0.00
16.8000	-0.03461	26875.	-1221.	2.05E-04	1189.	1.06E+09	21.7960	4534.	0.00
17.4000	-0.03248	18634.	-1065.	3.59E-04	975.1125	1.06E+09	21.5993	4788.	0.00
18.0000	-0.02943	11506.	-910.9399	4.62E-04	790.4519	1.06E+09	21.1552	5175.	0.00
18.6000	-0.02583	5471.	-760.9949	5.19E-04	634.1059	1.06E+09	20.4962	5713.	0.00
19.2000	-0.02196	497.3413	-616.4821	5.39E-04	505.2408	1.06E+09	19.6463	6441.	0.00
19.8000	-0.01807	-3458.	-478.7088	5.29E-04	581.9541	1.06E+09	18.6241	7422.	0.00
20.4000	-0.01434	-6447.	-348.8639	4.96E-04	659.3947	1.06E+09	17.4440	8758.	0.00
21.0000	-0.01093	-8530.	-228.0451	4.45E-04	713.3521	1.06E+09	16.1168	10618.	0.00
21.6000	-0.00793	-9775.	-117.2851	3.83E-04	745.5910	1.06E+09	14.6498	13295.	0.00
22.2000	-0.00542	-10256.	-17.5842	3.15E-04	758.0710	1.06E+09	13.0449	17344.	0.00
22.8000	-0.00340	-10058.	70.0332	2.46E-04	752.9437	1.06E+09	11.2933	23934.	0.00
23.4000	-0.00187	-9272.	144.3880	1.81E-04	732.5630	1.06E+09	9.3608	36041.	0.00
24.0000	-7.95E-04	-7997.	203.7316	1.22E-04	699.5312	1.06E+09	7.1235	64502.	0.00
24.6000	-1.10E-04	-6350.	242.9532	7.36E-05	656.8639	1.06E+09	3.7713	245885.	0.00
25.2000	2.65E-04	-4505.	238.5031	3.68E-05	609.0774	1.06E+09	-5.0075	136307.	0.00
25.8000	4.20E-04	-2919.	199.1819	1.17E-05	567.9778	1.06E+09	-5.9151	101491.	0.00

26.4000	4.32E-04	-1638.	156.1442	-3.78E-06	534.7976	1.06E+09	-6.0398	100582.	0.00
27.0000	3.65E-04	-670.0522	113.6326	-1.16E-05	509.7153	1.06E+09	-5.7690	113755.	0.00
27.6000	2.65E-04	-0.7549	74.0035	-1.39E-05	492.3754	1.06E+09	-5.2392	142217.	0.00
28.2000	1.65E-04	396.9451	38.8647	-1.25E-05	502.6397	1.06E+09	-4.5216	196942.	0.00
28.8000	8.47E-05	560.1138	9.4043	-9.29E-06	506.8671	1.06E+09	-3.6618	311142.	0.00
29.4000	3.15E-05	533.2689	-13.4738	-5.59E-06	506.1716	1.06E+09	-2.6932	615686.	0.00
30.0000	4.27E-06	366.6334	-24.5710	-2.54E-06	501.8544	1.06E+09	-0.3894	656429.	0.00
30.6000	-5.07E-06	179.6927	-21.7028	-6.88E-07	497.0112	1.06E+09	1.1861	1685750.	0.00
31.2000	-5.64E-06	54.1802	-12.8523	1.04E-07	493.7595	1.06E+09	1.2724	1625603.	0.00
31.8000	-3.56E-06	-5.3900	-4.6616	2.70E-07	492.4954	1.06E+09	1.0028	2027135.	0.00
32.4000	-1.75E-06	-12.9737	-0.4512	2.08E-07	492.6919	1.06E+09	0.1668	685810.	0.00
33.0000	-5.73E-07	-11.9075	0.3478	1.23E-07	492.6643	1.06E+09	0.05517	693225.	0.00
33.6000	2.38E-08	-7.9772	0.5381	5.59E-08	492.5625	1.06E+09	-0.00232	700668.	0.00
34.2000	2.32E-07	-4.1646	0.4477	1.47E-08	492.4637	1.06E+09	-0.02277	708139.	0.00
34.8000	2.36E-07	-1.5311	0.2813	-4.56E-09	492.3955	1.06E+09	-0.02346	715639.	0.00
35.4000	1.66E-07	-0.1135	0.1364	-1.01E-08	492.3587	1.06E+09	-0.01678	728498.	0.00
36.0000	9.01E-08	0.4342	0.04246	-9.05E-09	492.3670	1.06E+09	-0.00932	744130.	0.00
36.6000	3.56E-08	0.4988	-0.00461	-5.88E-09	492.3687	1.06E+09	-0.00376	759886.	0.00
37.2000	5.40E-09	0.3685	-0.02023	-2.95E-09	492.3653	1.06E+09	-5.82E-04	775768.	0.00
37.8000	-6.83E-09	0.2078	-0.01962	-9.94E-10	492.3612	1.06E+09	7.51E-04	791776.	0.00
38.4000	-8.91E-09	0.08609	-0.01331	2.04E-12	492.3580	1.06E+09	0.00100	807913.	0.00
39.0000	-6.80E-09	0.01615	-0.00691	3.48E-10	492.3562	1.06E+09	7.78E-04	824180.	0.00
39.6000	-3.90E-09	-0.01344	-0.00247	3.58E-10	492.3561	1.06E+09	4.55E-04	840578.	0.00
40.2000	-1.65E-09	-0.01945	-1.26E-04	2.46E-10	492.3563	1.06E+09	1.96E-04	857109.	0.00
40.8000	-3.50E-10	-0.01528	7.34E-04	1.29E-10	492.3562	1.06E+09	4.25E-05	873775.	0.00
41.4000	2.03E-10	-0.00890	7.97E-04	4.67E-11	492.3560	1.06E+09	-2.51E-05	890578.	0.00
42.0000	3.22E-10	-0.00381	5.60E-04	3.63E-12	492.3559	1.06E+09	-4.06E-05	907519.	0.00
42.6000	2.55E-10	-8.29E-04	2.96E-04	-1.21E-11	492.3558	1.06E+09	-3.28E-05	924599.	0.00
43.2000	1.48E-10	4.55E-04	1.09E-04	-1.34E-11	492.3558	1.06E+09	-1.93E-05	941821.	0.00
43.8000	6.27E-11	7.36E-04	8.92E-06	-9.33E-12	492.3558	1.06E+09	-8.36E-06	959187.	0.00
44.4000	1.35E-11	5.84E-04	-2.78E-05	-4.85E-12	492.3558	1.06E+09	-1.84E-06	976697.	0.00
45.0000	-7.18E-12	3.36E-04	-3.08E-05	-1.74E-12	492.3558	1.06E+09	9.91E-07	994354.	0.00
45.6000	-1.15E-11	1.40E-04	-2.14E-05	0.00	492.3558	1.06E+09	1.61E-06	1012161.	0.00
46.2000	-8.91E-12	2.78E-05	-1.10E-05	0.00	492.3558	1.06E+09	1.28E-06	1030118.	0.00
46.8000	-5.00E-12	-1.87E-05	-3.83E-06	0.00	492.3558	1.06E+09	7.28E-07	1048227.	0.00
47.4000	-2.01E-12	-2.74E-05	-1.37E-07	0.00	492.3558	1.06E+09	2.97E-07	1066491.	0.00
48.0000	0.00	-2.07E-05	1.12E-06	0.00	492.3558	1.06E+09	5.21E-08	1084912.	0.00
48.6000	0.00	-1.13E-05	1.14E-06	0.00	492.3558	1.06E+09	-4.66E-08	1103491.	0.00
49.2000	0.00	-4.31E-06	7.46E-07	0.00	492.3558	1.06E+09	-6.26E-08	1122231.	0.00
49.8000	0.00	-5.61E-07	3.56E-07	0.00	492.3558	1.06E+09	-4.59E-08	1141133.	0.00
50.4000	0.00	8.12E-07	1.04E-07	0.00	492.3558	1.06E+09	-2.41E-08	1160201.	0.00
51.0000	0.00	9.35E-07	-1.21E-08	0.00	492.3558	1.06E+09	-8.08E-09	1179435.	0.00
51.6000	0.00	6.39E-07	-3.80E-08	0.00	492.3558	1.06E+09	8.89E-10	1198839.	0.00

52.2000	0.00	3.89E-07	-3.09E-08	0.00	492.3558	1.06E+09	1.08E-09	270605.	0.00
52.8000	0.00	1.95E-07	-2.24E-08	0.00	492.3558	1.06E+09	1.27E-09	273715.	0.00
53.4000	0.00	6.62E-08	-1.39E-08	0.00	492.3558	1.06E+09	1.09E-09	276826.	0.00
54.0000	0.00	-5.60E-09	-7.15E-09	0.00	492.3558	1.06E+09	7.84E-10	279936.	0.00
54.6000	0.00	-3.68E-08	-2.60E-09	0.00	492.3558	1.06E+09	4.81E-10	283046.	0.00
55.2000	0.00	-4.31E-08	7.75E-12	0.00	492.3558	1.06E+09	2.42E-10	286157.	0.00
55.8000	0.00	-3.68E-08	1.18E-09	0.00	492.3558	1.06E+09	8.27E-11	289267.	0.00
56.4000	0.00	-2.62E-08	1.45E-09	0.00	492.3558	1.06E+09	-7.42E-12	292378.	0.00
57.0000	0.00	-1.59E-08	1.25E-09	0.00	492.3558	1.06E+09	-4.72E-11	295488.	0.00
57.6000	0.00	-8.13E-09	8.83E-10	0.00	492.3558	1.06E+09	-5.55E-11	298598.	0.00
58.2000	0.00	-3.21E-09	5.12E-10	0.00	492.3558	1.06E+09	-4.74E-11	301709.	0.00
58.8000	0.00	-7.47E-10	2.25E-10	0.00	492.3558	1.06E+09	-3.25E-11	304819.	0.00
59.4000	0.00	3.31E-11	5.15E-11	0.00	492.3558	1.06E+09	-1.57E-11	307930.	0.00
60.0000	0.00	0.00	0.00	0.00	492.3558	1.06E+09	1.38E-12	155520.	0.00

* This analysis computed pile response using nonlinear moment-curvature relationships. Values of total stress due to combined axial and bending stresses are computed only for elastic sections only and do not equal the actual stresses in concrete and steel. Stresses in concrete and steel may be interpolated from the output for nonlinear bending properties relative to the magnitude of bending moment developed in the pile.

Output Summary for Load Case No. 1:

```
Pile-head deflection             =     2.12387965 inches
Computed slope at pile head      =    -0.02222379 radians
Maximum bending moment           =        183285. inch-lbs
Maximum shear force              =          4000. lbs
Depth of maximum bending moment  =     7.20000000 feet below pile head
Depth of maximum shear force     =       0.000000 feet below pile head
Number of iterations             =             24
Number of zero deflection points =             11
```

Summary of Pile-head Responses for Conventional Analyses

Definitions of Pile-head Loading Conditions:

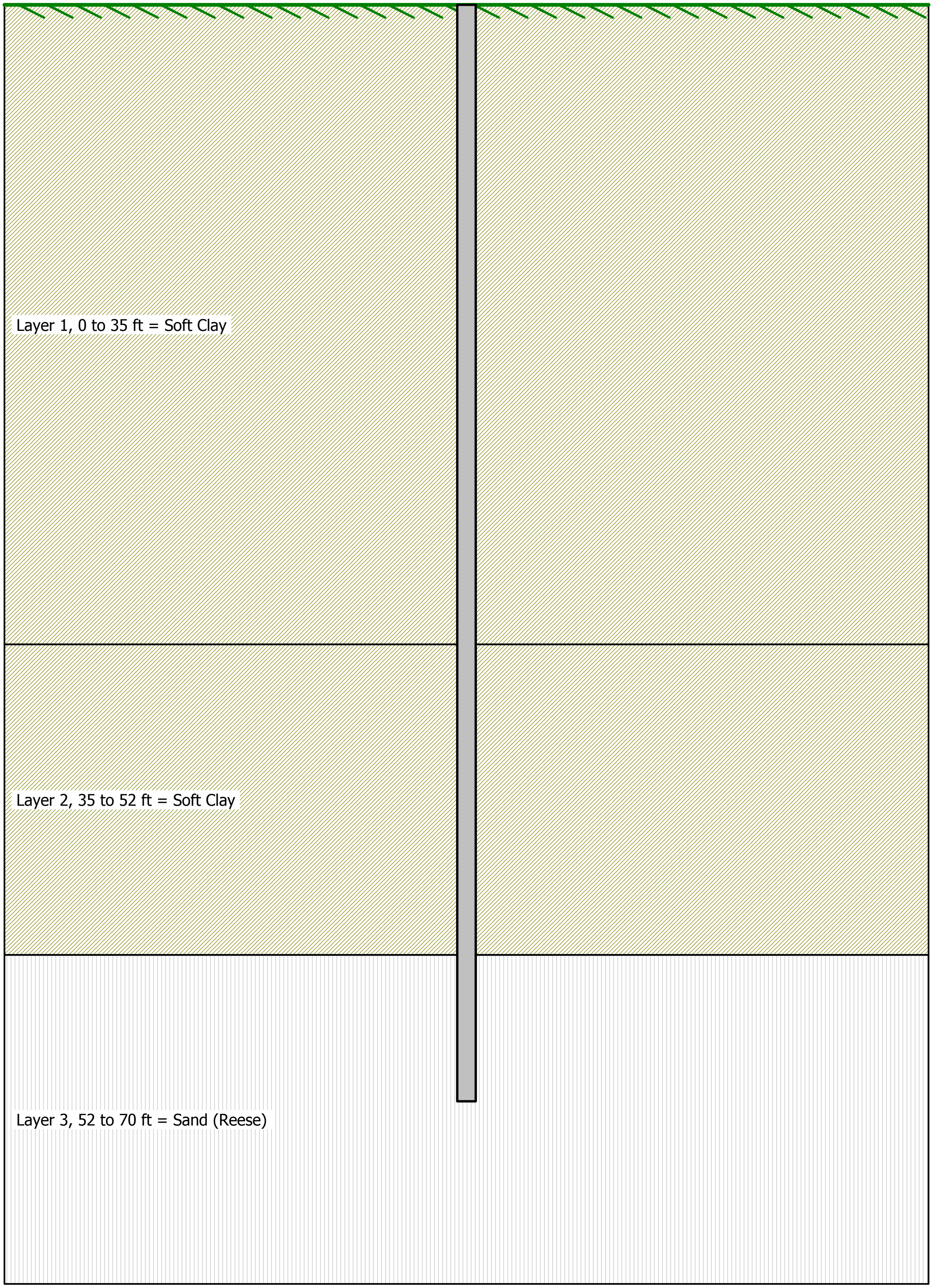
Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians

Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

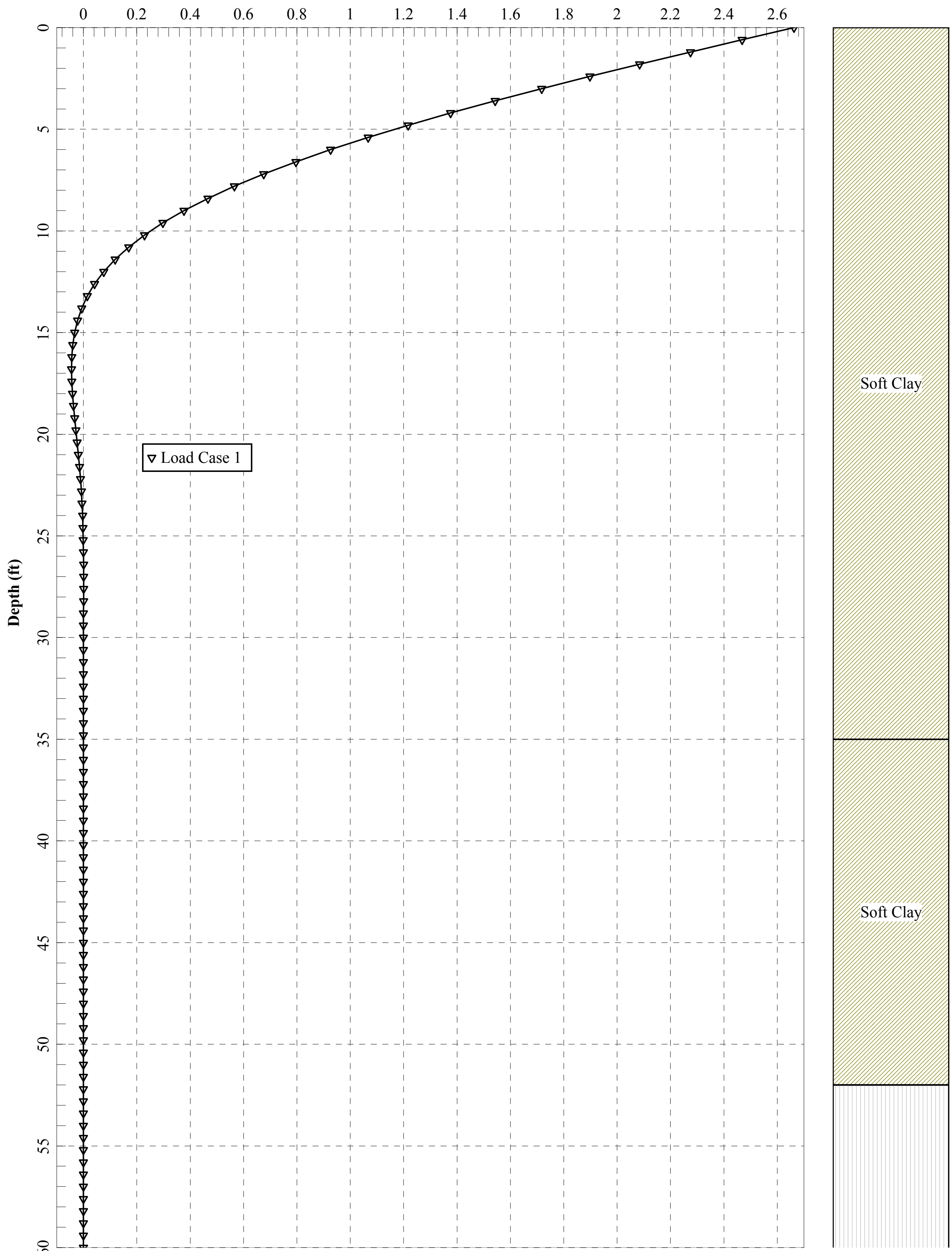
Load Case No.	Load Type 1	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	V, lb	4000.	M, in-lb	0.00	6743.	2.1239	-0.02222	4000.	183285.

Maximum pile-head deflection = 2.1238796463 inches
Maximum pile-head rotation = -0.0222237924 radians = -1.273330 deg.

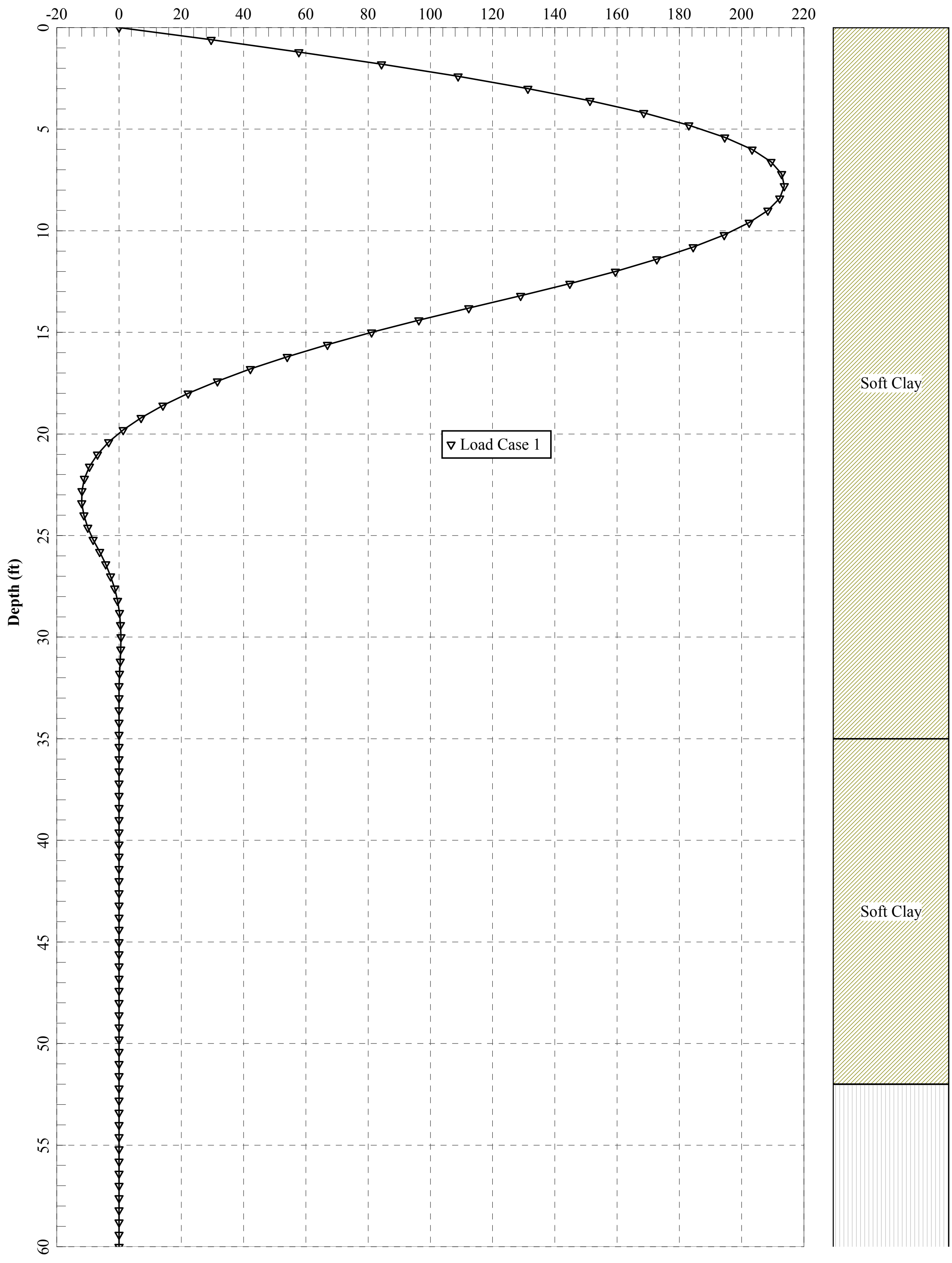
The analysis ended normally.

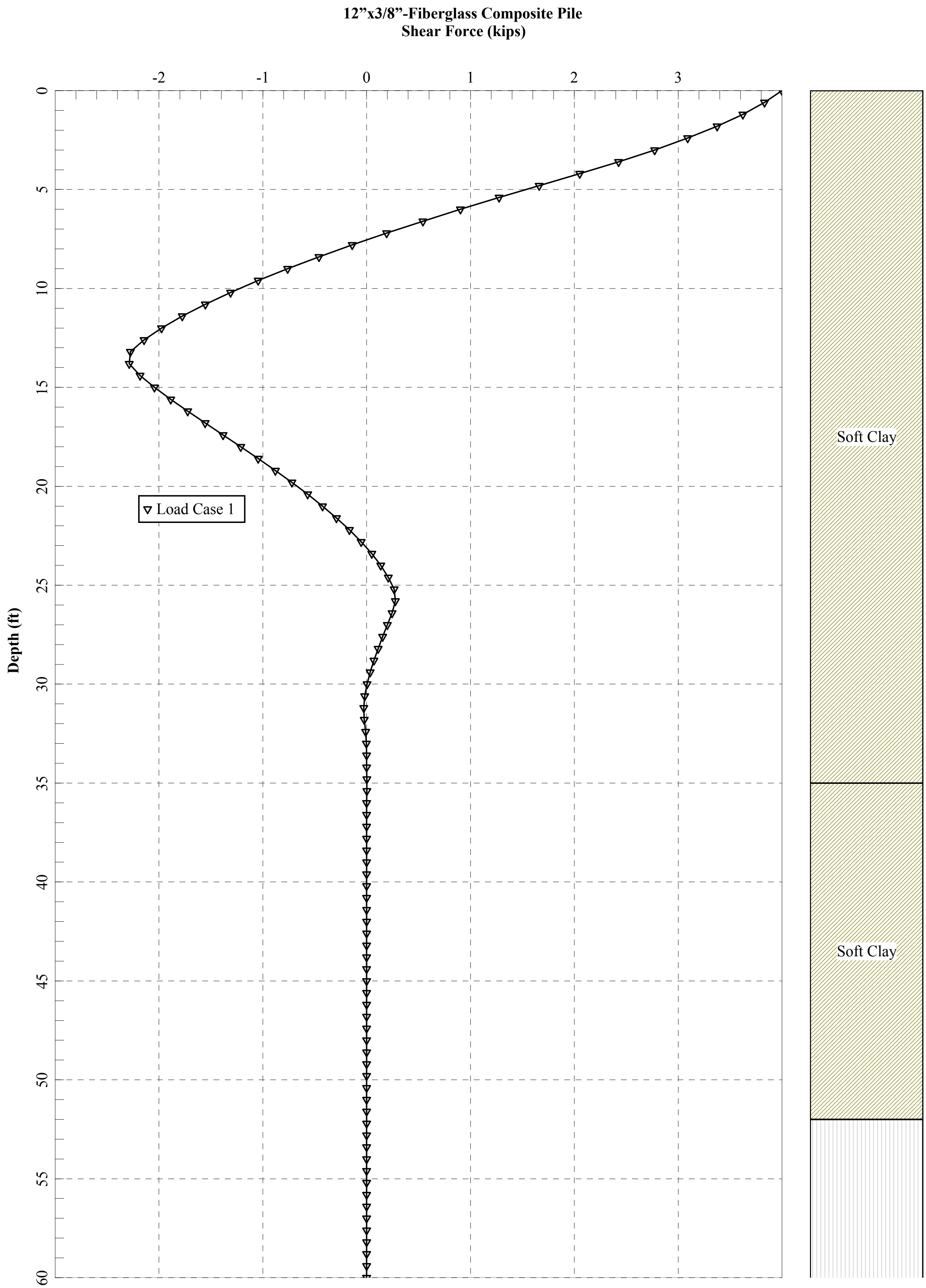

Layer 1, 0 to 35 ft = Soft Clay
Layer 2, 35 to 52 ft = Soft Clay
Layer 3, 52 to 70 ft = Sand (Reese)

12”x3/8”-Fiberglass Composite Pile
Lateral Pile Deflection (inches)



12”x3/8”-Fiberglass Composite Pile
Bending Moment (in-kips)




12”x3/8”-Fiberglass Composite Pile
Shear Force (kips)
-2
-1
0
1
2
3
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
Load Case 1
Soft Clay
Soft Clay

```
================================================================================

                LPile for Windows, Version 2016-09.010

              Analysis of Individual Piles and Drilled Shafts
             Subjected to Lateral Loading Using the p-y Method
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--------------------------------------------------------------------------------
                             Files Used for Analysis
--------------------------------------------------------------------------------

Path to file locations:
\Users\dsikdar\Desktop\Lpile Projects\Deer River\

Name of input data file:
Deer River-Boring B-6-No Concrete.lp9d

Name of output report file:
Deer River-Boring B-6-No Concrete.lp9o

Name of plot output file:
Deer River-Boring B-6-No Concrete.lp9p

Name of runtime message file:
Deer River-Boring B-6-No Concrete.lp9r
```

Date and Time of Analysis

Date: April 30, 2020 Time: 10:39:32

Problem Title

Project Name: Deer River

Job Number: 19-1101-0184

Client: Deer River Coastal Restoration

Engineer: DS /CC

Description: Reef Maker Pile-Boring: B-6

Program Options and Settings

Computational Options:
- Use unfactored loads in computations (conventional analysis)

Engineering Units Used for Data Input and Computations:
- US Customary System Units (pounds, feet, inches)

Analysis Control Options:

```
 - Maximum number of iterations allowed                    =        1000
 - Deflection tolerance for convergence                    =   1.0000E-04 in
 - Maximum allowable deflection                            =     100.0000 in
 - Number of pile increments                               =         100

Loading Type and Number of Cycles of Loading:
 - Static loading specified

 - Use of p-y modification factors for p-y curves not selected
 - No distributed lateral loads are entered
 - Loading by lateral soil movements acting on pile not selected
 - Input of shear resistance at the pile tip not selected
 - Computation of pile-head foundation stiffness matrix not selected
 - Push-over analysis of pile not selected
 - Buckling analysis of pile not selected

Output Options:
 - Output files use decimal points to denote decimal symbols.
 - Values of pile-head deflection, bending moment, shear force, and
   soil reaction are printed for full length of pile.
 - Printing Increment (nodal spacing of output points) = 1
 - No p-y curves to be computed and reported for user-specified depths
 - Print using wide report formats



--------------------------------------------------------------------------------
                     Pile Structural Properties and Geometry
--------------------------------------------------------------------------------

Number of pile sections defined                            =           1
Total length of pile                                       =      60.000 ft
Depth of ground surface below top of pile                  =      0.0000 ft

Pile diameters used for p-y curve computations are defined using 2 points.

p-y curves are computed using pile diameter values interpolated with depth over
the length of the pile. A summary of values of pile diameter vs. depth follows.

            Depth Below          Pile
Point        Pile Head         Diameter
 No.           feet             inches
```

```
-----      -------------     -------------
  1             0.000          12.0000
  2            60.000          12.0000


Input Structural Properties for Pile Sections:
----------------------------------------------

Pile Section No. 1:

   Section 1 is a steel pipe pile
   Length of section                                      =    60.000000 ft
   Pile diameter                                          =    12.000000 in
   Shear capacity of section                              =       0.0000 lbs



--------------------------------------------------------------------------------
                       Ground Slope and Pile Batter Angles
--------------------------------------------------------------------------------

Ground Slope Angle                                        =        0.000 degrees
                                                          =        0.000 radians

Pile Batter Angle                                         =        0.000 degrees
                                                          =        0.000 radians


--------------------------------------------------------------------------------
                       Soil and Rock Layering Information
--------------------------------------------------------------------------------

The soil profile is modelled using 3 layers

Layer 1 is soft clay, p-y criteria by Matlock, 1970

   Distance from top of pile to top of layer              =       0.0000 ft
   Distance from top of pile to bottom of layer           =    35.000000 ft
   Effective unit weight at top of layer                  =    97.000000 pcf
   Effective unit weight at bottom of layer               =    97.000000 pcf
   Undrained cohesion at top of layer                     =   100.000000 psf
   Undrained cohesion at bottom of layer                  =   200.000000 psf
```

Epsilon-50 at top of layer = 0.020000
Epsilon-50 at bottom of layer = 0.018000

Layer 2 is soft clay, p-y criteria by Matlock, 1970

Distance from top of pile to top of layer = 35.000000 ft
Distance from top of pile to bottom of layer = 52.000000 ft
Effective unit weight at top of layer = 100.000000 pcf
Effective unit weight at bottom of layer = 100.000000 pcf
Undrained cohesion at top of layer = 200.000000 psf
Undrained cohesion at bottom of layer = 300.000000 psf
Epsilon-50 at top of layer = 0.018000
Epsilon-50 at bottom of layer = 0.016000

Layer 3 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer = 52.000000 ft
Distance from top of pile to bottom of layer = 70.000000 ft
Effective unit weight at top of layer = 115.000000 pcf
Effective unit weight at bottom of layer = 115.000000 pcf
Friction angle at top of layer = 32.000000 deg.
Friction angle at bottom of layer = 32.000000 deg.
Subgrade k at top of layer = 60.000000 pci
Subgrade k at bottom of layer = 60.000000 pci

(Depth of the lowest soil layer extends 10.000 ft below the pile tip)

Summary of Input Soil Properties

Layer Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Undrained Cohesion psf	Angle of Friction deg.	E50 or krm	kpy pci
1	Soft	0.00	97.0000	100.0000	--	0.02000	--
	Clay	35.0000	97.0000	200.0000	--	0.01800	--
2	Soft	35.0000	100.0000	200.0000	--	0.01800	--

	Clay	52.0000	100.0000	300.0000	--	0.01600	--
3	Sand	52.0000	115.0000	--	32.0000	--	60.0000
	(Reese, et al.)	70.0000	115.0000	--	32.0000	--	60.0000

Static Loading Type

Static loading criteria were used when computing p-y curves for all analyses.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length
1	1	V = 4000. lbs	M = 0.0000 in-lbs	6743.	No

V = shear force applied normal to pile axis
M = bending moment applied to pile head
y = lateral deflection normal to pile axis
S = pile slope relative to original pile batter angle
R = rotational stiffness applied to pile head
Values of top y vs. pile lengths can be computed only for load types with specified shear loading (Load Types 1, 2, and 3).
Thrust force is assumed to be acting axially for all pile batter angles.

Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness

---

Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 1

Pile Section No. 1:

Dimensions and Properties of Steel Pipe Pile:

Length of Section	=	60.000000 ft
Outer Diameter of Pipe	=	12.000000 in
Pipe Wall Thickness	=	0.375000 in
Yield Stress of Pipe	=	54.289000 ksi
Elastic Modulus	=	4588. ksi
Cross-sectional Area	=	13.695380 sq. in.
Moment of Inertia	=	231.591024 in^4
Elastic Bending Stiffness	=	1062537. kip-in^2
Plastic Modulus, Z	=	50.695312in^3
Plastic Moment Capacity = Fy Z	=	2752.in-kip

Axial Structural Capacities:

Nom. Axial Structural Capacity = Fy As	=	743.509 kips
Nominal Axial Tensile Capacity	=	-743.509 kips

Number of Axial Thrust Force Values Determined from Pile-head Loadings = 1

Number	Axial Thrust Force kips
1	6.743

Definition of Run Messages:

Y = part of pipe section has yielded.

Axial Thrust Force = 6.743 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in2	Depth to N Axis in	Max Total Stress ksi	Run Msg

0.00004168	44.2912224	1062536.	8.5744390	1.6283678
0.00008337	88.5824447	1062536.	7.2872195	2.7643797
0.0001251	132.8736671	1062536.	6.8581463	3.9003917
0.0001667	177.1648894	1062536.	6.6436098	5.0364036
0.0002084	221.4561118	1062536.	6.5148878	6.1724156
0.0002501	265.7473341	1062536.	6.4290732	7.3084275
0.0002918	310.0385565	1062536.	6.3677770	8.4444395
0.0003335	354.3297788	1062536.	6.3218049	9.5804514
0.0003752	398.6210012	1062536.	6.2860488	10.7164634
0.0004168	442.9122235	1062536.	6.2574439	11.8524753
0.0004585	487.2034459	1062536.	6.2340399	12.9884873
0.0005002	531.4946682	1062536.	6.2145366	14.1244992
0.0005419	575.7858906	1062536.	6.1980338	15.2605112
0.0005836	620.0771129	1062536.	6.1838885	16.3965231
0.0006253	664.3683353	1062536.	6.1716293	17.5325351
0.0006670	708.6595576	1062536.	6.1609024	18.6685471
0.0007086	752.9507800	1062536.	6.1514376	19.8045590
0.0007503	797.2420023	1062536.	6.1430244	20.9405710
0.0007920	841.5332247	1062536.	6.1354968	22.0765829
0.0008337	885.8244470	1062536.	6.1287220	23.2125949
0.0008754	930.1156694	1062536.	6.1225923	24.3486068
0.0009171	974.4068917	1062536.	6.1170200	25.4846188
0.0009587	1019.	1062536.	6.1119321	26.6206307
0.0010004	1063.	1062536.	6.1072683	27.7566427
0.0010421	1107.	1062536.	6.1029776	28.8926546
0.0010838	1152.	1062536.	6.0990169	30.0286666
0.0011255	1196.	1062536.	6.0953496	31.1646785
0.0011672	1240.	1062536.	6.0919443	32.3006905
0.0012088	1284.	1062536.	6.0887738	33.4367025
0.0012505	1329.	1062536.	6.0858146	34.5727144
0.0012922	1373.	1062536.	6.0830464	35.7087264
0.0013339	1417.	1062536.	6.0804512	36.8447383
0.0013756	1462.	1062536.	6.0780133	37.9807503
0.0014173	1506.	1062536.	6.0757188	39.1167622
0.0014590	1550.	1062536.	6.0735554	40.2527742
0.0015006	1594.	1062536.	6.0715122	41.3887861
0.0015423	1639.	1062536.	6.0695794	42.5247981
0.0015840	1683.	1062536.	6.0677484	43.6608100
0.0016257	1727.	1062536.	6.0660113	44.7968220
0.0017091	1816.	1062536.	6.0627912	47.0688459
0.0017924	1905.	1062536.	6.0598707	49.3408698
0.0018758	1993.	1062536.	6.0572098	51.6128937

0.0019592	2082.	1062536.	6.0547753	53.8849176	
0.0020425	2166.	1060232.	6.0562228	54.2890000	Y
0.0021259	2232.	1050036.	6.0614230	54.2890000	Y
0.0022093	2284.	1033702.	6.0649628	54.2890000	Y
0.0022926	2326.	1014628.	6.0671632	54.2890000	Y
0.0023760	2362.	994001.	6.0688683	54.2890000	Y
0.0024594	2393.	972947.	6.0700863	54.2890000	Y
0.0025428	2420.	951635.	6.0714885	54.2890000	Y
0.0026261	2444.	930465.	6.0723470	54.2890000	Y
0.0027095	2465.	909649.	6.0730992	54.2890000	Y
0.0027929	2484.	889278.	6.0737726	54.2890000	Y
0.0028762	2501.	869419.	6.0743883	54.2890000	Y
0.0029596	2516.	850115.	6.0749629	54.2890000	Y
0.0030430	2530.	831398.	6.0755101	54.2890000	Y
0.0031263	2543.	813284.	6.0760414	54.2890000	Y
0.0032097	2554.	795783.	6.0765664	54.2890000	Y
0.0032931	2565.	778876.	6.0768881	54.2890000	Y
0.0033764	2575.	762530.	6.0769282	54.2890000	Y
0.0034598	2584.	746738.	6.0774343	54.2890000	Y
0.0035432	2592.	731536.	6.0779612	54.2890000	Y
0.0036265	2600.	716837.	6.0777449	54.2890000	Y
0.0037099	2607.	702637.	6.0782739	54.2890000	Y
0.0037933	2613.	688961.	6.0784417	54.2890000	Y
0.0038767	2620.	675724.	6.0785850	54.2890000	Y
0.0039600	2625.	662973.	6.0790093	54.2890000	Y
0.0040434	2631.	650623.	6.0789072	54.2890000	Y
0.0041268	2636.	638718.	6.0793663	54.2890000	Y
0.0042101	2641.	627180.	6.0792517	54.2890000	Y
0.0042935	2645.	616051.	6.0795308	54.2890000	Y
0.0043769	2649.	605259.	6.0796291	54.2890000	Y
0.0044602	2653.	594844.	6.0795148	54.2890000	Y
0.0045436	2657.	584739.	6.0800498	54.2890000	Y
0.0046270	2660.	574959.	6.0797719	54.2890000	Y
0.0047103	2664.	565494.	6.0802000	54.2890000	Y
0.0047937	2667.	556302.	6.0802393	54.2890000	Y
0.0048771	2670.	547411.	6.0799641	54.2890000	Y
0.0049604	2673.	538781.	6.0806057	54.2890000	Y
0.0052939	2682.	506705.	6.0808117	54.2890000	Y
0.0056274	2691.	478125.	6.0809468	54.2890000	Y
0.0059609	2697.	452516.	6.0811870	54.2890000	Y
0.0062944	2703.	429452.	6.0815453	54.2890000	Y
0.0066278	2708.	408582.	6.0817836	54.2890000	Y
0.0069613	2712.	389608.	6.0811213	54.2890000	Y

0.0072948	2716.	372282.	6.0817271	54.2890000	Y
0.0076283	2719.	356423.	6.0820154	54.2890000	Y
0.0079617	2722.	341833.	6.0816729	54.2890000	Y
0.0082952	2724.	328386.	6.0823173	54.2890000	Y
0.0086287	2726.	315937.	6.0818767	54.2890000	Y
0.0089622	2728.	304397.	6.0820660	54.2890000	Y
0.0092956	2730.	293656.	6.0823585	54.2890000	Y
0.0096291	2731.	283646.	6.0816179	54.2890000	Y
0.0099626	2733.	274290.	6.0826071	54.2890000	Y

Summary of Results for Nominal (Unfactored) Moment Capacity for Section 1

Load No.	Axial Thrust kips	Nominal Moment Capacity in-kips
1	6.7430000000	2733.

Note that the values in the above table are not factored by a strength reduction factor for LRFD.

The value of the strength reduction factor depends on the provisions of the LRFD code being followed.

The above values should be multiplied by the appropriate strength reduction factor to compute ultimate moment capacity according to the LRFD structural design standard being followed.

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer	Top of Layer Below	Equivalent Top Depth Below	Same Layer Type As	Layer is Rock or	F0 Integral	F1 Integral

No.	Pile Head ft	Grnd Surf ft	Layer Above	is Below Rock Layer	for Layer lbs	for Layer lbs
1	0.00	0.00	N.A.	No	0.00	45907.
2	35.0000	35.0000	Yes	No	45907.	38250.
3	52.0000	52.0000	No	No	84157.	N.A.

Notes: The F0 integral of Layer n+1 equals the sum of the F0 and F1 integrals for Layer n. Layering correction equivalent depths are computed only for soil types with both shallow-depth and deep-depth expressions for peak lateral load transfer. These soil types are soft and stiff clays, non-liquefied sands, and cemented c-phi soil.

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 1

Pile-head conditions are Shear and Moment (Loading Type 1)

Shear force at pile head = 4000.0 lbs
Applied moment at pile head = 0.0 in-lbs
Axial thrust load on pile head = 6743.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness in-lb^2	Soil Res. p lb/inch	Soil Spr. Es*h lb/inch	Distrib. Lat. Load lb/inch
0.00	2.6627	7.28E-08	4000.	-0.02703	492.3558	1.06E+09	-20.5420	27.7728	0.00
0.6000	2.4681	29580.	3831.	-0.02693	1259.	1.06E+09	-26.3102	76.7517	0.00
1.2000	2.2750	57786.	3622.	-0.02663	1989.	1.06E+09	-31.7920	100.6172	0.00
1.8000	2.0847	84325.	3375.	-0.02615	2677.	1.06E+09	-36.9608	127.6552	0.00
2.4000	1.8984	108920.	3091.	-0.02549	3314.	1.06E+09	-41.7901	158.4924	0.00
3.0000	1.7175	131313.	2774.	-0.02468	3894.	1.06E+09	-46.2545	193.9004	0.00
3.6000	1.5431	151264.	2426.	-0.02372	4411.	1.06E+09	-50.3289	234.8385	0.00
4.2000	1.3759	168558.	2051.	-0.02264	4859.	1.06E+09	-53.9888	282.5120	0.00
4.8000	1.2170	182996.	1661.	-0.02145	5233.	1.06E+09	-54.2306	320.8254	0.00
5.4000	1.0671	194564.	1276.	-0.02017	5533.	1.06E+09	-52.7181	355.7065	0.00
6.0000	0.9266	203334.	902.6965	-0.01882	5760.	1.06E+09	-51.0720	396.8384	0.00
6.6000	0.7961	209390.	541.3914	-0.01742	5917.	1.06E+09	-49.2905	445.8037	0.00

7.2000	0.6757	212822.	193.4096	-0.01599	6006.	1.06E+09	-47.3711	504.7384	0.00
7.8000	0.5658	213728.	-140.2417	-0.01455	6030.	1.06E+09	-45.3098	576.5927	0.00
8.4000	0.4663	212215.	-458.5214	-0.01310	5990.	1.06E+09	-43.1012	665.5559	0.00
9.0000	0.3771	208397.	-760.3371	-0.01168	5891.	1.06E+09	-40.7365	777.7815	0.00
9.6000	0.2981	202400.	-1045.	-0.01029	5736.	1.06E+09	-38.2027	922.7033	0.00
10.2000	0.2290	194355.	-1310.	-0.00894	5528.	1.06E+09	-35.4788	1116.	0.00
10.8000	0.1693	184407.	-1555.	-0.00766	5270.	1.06E+09	-32.5295	1383.	0.00
11.4000	0.1187	172713.	-1777.	-0.00645	4967.	1.06E+09	-29.2924	1777.	0.00
12.0000	0.07647	159442.	-1975.	-0.00532	4623.	1.06E+09	-25.6434	2414.	0.00
12.6000	0.04203	144791.	-2144.	-0.00429	4244.	1.06E+09	-21.2886	3647.	0.00
13.2000	0.01465	128987.	-2275.	-0.00337	3834.	1.06E+09	-15.1884	7463.	0.00
13.8000	-0.00643	112354.	-2288.	-0.00255	3403.	1.06E+09	11.6648	13061.	0.00
14.4000	-0.02203	96289.	-2182.	-0.00184	2987.	1.06E+09	17.8333	5828.	0.00
15.0000	-0.03294	81117.	-2043.	-0.00124	2594.	1.06E+09	20.6545	4515.	0.00
15.6000	-0.03988	66988.	-1889.	-7.38E-04	2228.	1.06E+09	22.2940	4025.	0.00
16.2000	-0.04356	53994.	-1725.	-3.28E-04	1891.	1.06E+09	23.2463	3842.	0.00
16.8000	-0.04460	42186.	-1555.	-1.97E-06	1585.	1.06E+09	23.7199	3829.	0.00
17.4000	-0.04359	31595.	-1384.	2.48E-04	1311.	1.06E+09	23.8262	3936.	0.00
18.0000	-0.04103	22228.	-1213.	4.30E-04	1068.	1.06E+09	23.6330	4147.	0.00
18.6000	-0.03739	14079.	-1045.	5.53E-04	857.1019	1.06E+09	23.1861	4465.	0.00
19.2000	-0.03306	7127.	-880.3979	6.25E-04	676.9989	1.06E+09	22.5178	4904.	0.00
19.8000	-0.02839	1340.	-721.3846	6.54E-04	527.0778	1.06E+09	21.6526	5492.	0.00
20.4000	-0.02365	-3324.	-569.2422	6.47E-04	578.4857	1.06E+09	20.6092	6275.	0.00
21.0000	-0.01907	-6920.	-425.1987	6.12E-04	671.6299	1.06E+09	19.4028	7327.	0.00
21.6000	-0.01483	-9507.	-290.3856	5.57E-04	738.6560	1.06E+09	18.0452	8763.	0.00
22.2000	-0.01105	-11155.	-165.8601	4.87E-04	781.3652	1.06E+09	16.5452	10782.	0.00
22.8000	-0.00782	-11942.	-52.6301	4.09E-04	801.7583	1.06E+09	14.9076	13733.	0.00
23.4000	-0.00517	-11953.	48.3082	3.28E-04	802.0278	1.06E+09	13.1308	18303.	0.00
24.0000	-0.00310	-11279.	135.8940	2.49E-04	784.5600	1.06E+09	11.1986	26025.	0.00
24.6000	-0.00158	-10020.	208.8023	1.77E-04	751.9557	1.06E+09	9.0537	41224.	0.00
25.2000	-5.53E-04	-8289.	264.6787	1.15E-04	707.1064	1.06E+09	6.4675	84160.	0.00
25.8000	7.03E-05	-6220.	276.5776	6.55E-05	653.5002	1.06E+09	-3.1623	323984.	0.00
26.4000	3.90E-04	-4313.	244.2416	2.98E-05	604.0880	1.06E+09	-5.8200	107337.	0.00
27.0000	5.00E-04	-2706.	200.2567	6.07E-06	562.4557	1.06E+09	-6.3981	92114.	0.00
27.6000	4.78E-04	-1430.	154.2929	-7.94E-06	529.3932	1.06E+09	-6.3697	95987.	0.00
28.2000	3.86E-04	-483.1621	109.7897	-1.44E-05	504.8734	1.06E+09	-5.9923	111850.	0.00
28.8000	2.70E-04	152.7833	68.8699	-1.55E-05	496.3141	1.06E+09	-5.3743	143257.	0.00
29.4000	1.62E-04	510.0740	33.0511	-1.33E-05	505.5707	1.06E+09	-4.5754	203429.	0.00
30.0000	7.87E-05	630.0099	3.5138	-9.43E-06	508.6779	1.06E+09	-3.6294	332255.	0.00
30.6000	2.61E-05	561.5889	-18.6709	-5.40E-06	506.9053	1.06E+09	-2.5330	698739.	0.00
31.2000	9.50E-07	361.6731	-28.1085	-2.27E-06	501.7259	1.06E+09	-0.08858	671065.	0.00
31.8000	-6.55E-06	157.0465	-22.5488	-5.10E-07	496.4245	1.06E+09	1.6329	1793843.	0.00
32.4000	-6.40E-06	37.0197	-10.8025	1.47E-07	493.3149	1.06E+09	1.6299	1834653.	0.00

33.0000	-4.43E-06	1.4762	-3.3983	2.78E-07	492.3940	1.06E+09	0.4268	693225.	0.00
33.6000	-2.40E-06	-11.9424	-1.0220	2.42E-07	492.6652	1.06E+09	0.2333	700668.	0.00
34.2000	-9.44E-07	-13.2643	0.1520	1.57E-07	492.6994	1.06E+09	0.09284	708139.	0.00
34.8000	-1.38E-07	-9.7687	0.5356	7.88E-08	492.6089	1.06E+09	0.01372	715639.	0.00
35.4000	1.91E-07	-5.5588	0.5153	2.69E-08	492.4998	1.06E+09	-0.01936	728498.	0.00
36.0000	2.49E-07	-2.3504	0.3528	1.11E-10	492.4167	1.06E+09	-0.02578	744130.	0.00
36.6000	1.93E-07	-0.4778	0.1867	-9.47E-09	492.3682	1.06E+09	-0.02036	759886.	0.00
37.2000	1.13E-07	0.3394	0.06958	-9.94E-09	492.3646	1.06E+09	-0.01218	775768.	0.00
37.8000	4.98E-08	0.5251	0.00602	-7.01E-09	492.3694	1.06E+09	-0.00547	791776.	0.00
38.4000	1.21E-08	0.4268	-0.01858	-3.79E-09	492.3669	1.06E+09	-0.00136	807913.	0.00
39.0000	-4.74E-09	0.2579	-0.02152	-1.47E-09	492.3625	1.06E+09	5.43E-04	824180.	0.00
39.6000	-9.00E-09	0.1171	-0.01578	-1.96E-10	492.3588	1.06E+09	0.00105	840578.	0.00
40.2000	-7.56E-09	0.03069	-0.00876	3.05E-10	492.3566	1.06E+09	9.00E-04	857109.	0.00
40.8000	-4.61E-09	-0.00907	-0.00351	3.78E-10	492.3560	1.06E+09	5.60E-04	873775.	0.00
41.4000	-2.11E-09	-0.01982	-5.51E-04	2.80E-10	492.3563	1.06E+09	2.61E-04	890578.	0.00
42.0000	-5.74E-10	-0.01704	6.48E-04	1.55E-10	492.3562	1.06E+09	7.24E-05	907519.	0.00
42.6000	1.29E-10	-0.01050	8.49E-04	6.21E-11	492.3561	1.06E+09	-1.66E-05	924599.	0.00
43.2000	3.20E-10	-0.00482	6.39E-04	1.02E-11	492.3559	1.06E+09	-4.18E-05	941821.	0.00
43.8000	2.76E-10	-0.00130	3.56E-04	-1.05E-11	492.3558	1.06E+09	-3.67E-05	959187.	0.00
44.4000	1.68E-10	3.13E-04	1.42E-04	-1.39E-11	492.3558	1.06E+09	-2.28E-05	976697.	0.00
45.0000	7.57E-11	7.46E-04	2.24E-05	-1.03E-11	492.3558	1.06E+09	-1.04E-05	994354.	0.00
45.6000	1.97E-11	6.36E-04	-2.52E-05	-5.62E-12	492.3558	1.06E+09	-2.77E-06	1012161.	0.00
46.2000	-5.22E-12	3.83E-04	-3.25E-05	-2.16E-12	492.3558	1.06E+09	7.47E-07	1030118.	0.00
46.8000	-1.15E-11	1.68E-04	-2.38E-05	0.00	492.3558	1.06E+09	1.67E-06	1048227.	0.00
47.4000	-9.46E-12	4.02E-05	-1.28E-05	0.00	492.3558	1.06E+09	1.40E-06	1066491.	0.00
48.0000	-5.51E-12	-1.54E-05	-4.73E-06	0.00	492.3558	1.06E+09	8.30E-07	1084912.	0.00
48.6000	-2.31E-12	-2.80E-05	-4.66E-07	0.00	492.3558	1.06E+09	3.55E-07	1103491.	0.00
49.2000	0.00	-2.22E-05	1.08E-06	0.00	492.3558	1.06E+09	7.53E-08	1122231.	0.00
49.8000	0.00	-1.24E-05	1.20E-06	0.00	492.3558	1.06E+09	-4.22E-08	1141133.	0.00
50.4000	0.00	-4.90E-06	8.12E-07	0.00	492.3558	1.06E+09	-6.57E-08	1160201.	0.00
51.0000	0.00	-7.52E-07	3.93E-07	0.00	492.3558	1.06E+09	-5.08E-08	1179435.	0.00
51.6000	0.00	7.62E-07	1.05E-07	0.00	492.3558	1.06E+09	-2.93E-08	1198839.	0.00
52.2000	0.00	7.57E-07	-1.15E-08	0.00	492.3558	1.06E+09	-2.97E-09	270605.	0.00
52.8000	0.00	5.98E-07	-2.47E-08	0.00	492.3558	1.06E+09	-7.18E-10	273715.	0.00
53.4000	0.00	4.02E-07	-2.57E-08	0.00	492.3558	1.06E+09	4.61E-10	276826.	0.00
54.0000	0.00	2.29E-07	-2.07E-08	0.00	492.3558	1.06E+09	9.04E-10	279936.	0.00
54.6000	0.00	1.03E-07	-1.42E-08	0.00	492.3558	1.06E+09	9.19E-10	283046.	0.00
55.2000	0.00	2.46E-08	-8.23E-09	0.00	492.3558	1.06E+09	7.34E-10	286157.	0.00
55.8000	0.00	-1.57E-08	-3.80E-09	0.00	492.3558	1.06E+09	4.97E-10	289267.	0.00
56.4000	0.00	-3.02E-08	-9.82E-10	0.00	492.3558	1.06E+09	2.85E-10	292378.	0.00
57.0000	0.00	-2.99E-08	5.12E-10	0.00	492.3558	1.06E+09	1.30E-10	295488.	0.00
57.6000	0.00	-2.29E-08	1.09E-09	0.00	492.3558	1.06E+09	3.11E-11	298598.	0.00
58.2000	0.00	-1.42E-08	1.12E-09	0.00	492.3558	1.06E+09	-2.29E-11	301709.	0.00

58.8000	0.00	-6.74E-09	8.62E-10	0.00	492.3558	1.06E+09	-4.86E-11	304819.	0.00
59.4000	0.00	-1.80E-09	4.69E-10	0.00	492.3558	1.06E+09	-6.08E-11	307930.	0.00
60.0000	0.00	0.00	0.00	0.00	492.3558	1.06E+09	-6.94E-11	155520.	0.00

* This analysis computed pile response using nonlinear moment-curvature relationships. Values of total stress due to combined axial and bending stresses are computed only for elastic sections only and do not equal the actual stresses in concrete and steel. Stresses in concrete and steel may be interpolated from the output for nonlinear bending properties relative to the magnitude of bending moment developed in the pile.

Output Summary for Load Case No. 1:

```
Pile-head deflection               =     2.66272344 inches
Computed slope at pile head        =    -0.02702643 radians
Maximum bending moment             =        213728. inch-lbs
Maximum shear force                =          4000. lbs
Depth of maximum bending moment    =     7.80000000 feet below pile head
Depth of maximum shear force       =       0.000000 feet below pile head
Number of iterations               =             25
Number of zero deflection points   =             10
```

Summary of Pile-head Responses for Conventional Analyses

Definitions of Pile-head Loading Conditions:

Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

Load Case No.	Load Type 1	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	V, lb	4000.	M, in-lb	0.00	6743.	2.6627	-0.02703	4000.	213728.

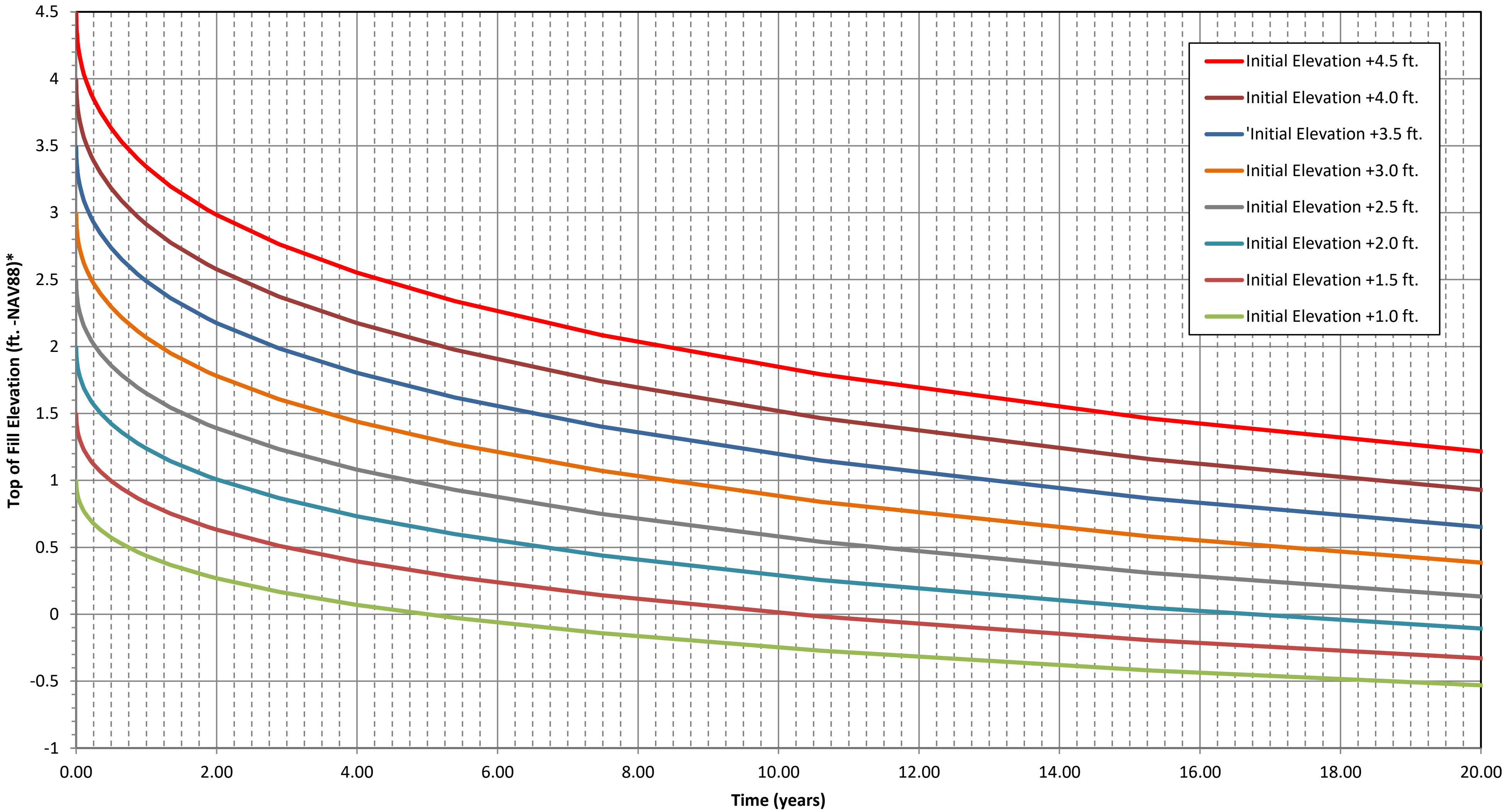
```
Maximum pile-head deflection = 2.6627234411 inches
Maximum pile-head rotation   = -0.0270264296 radians = -1.548500 deg.

The analysis ended normally.
```

APPENDIX D

Mass Fill Settlement

Deer River Project - Time vs. Total Settlement - Soft Soil Area

(Total Settlement includes immediate and consolidation settlement)

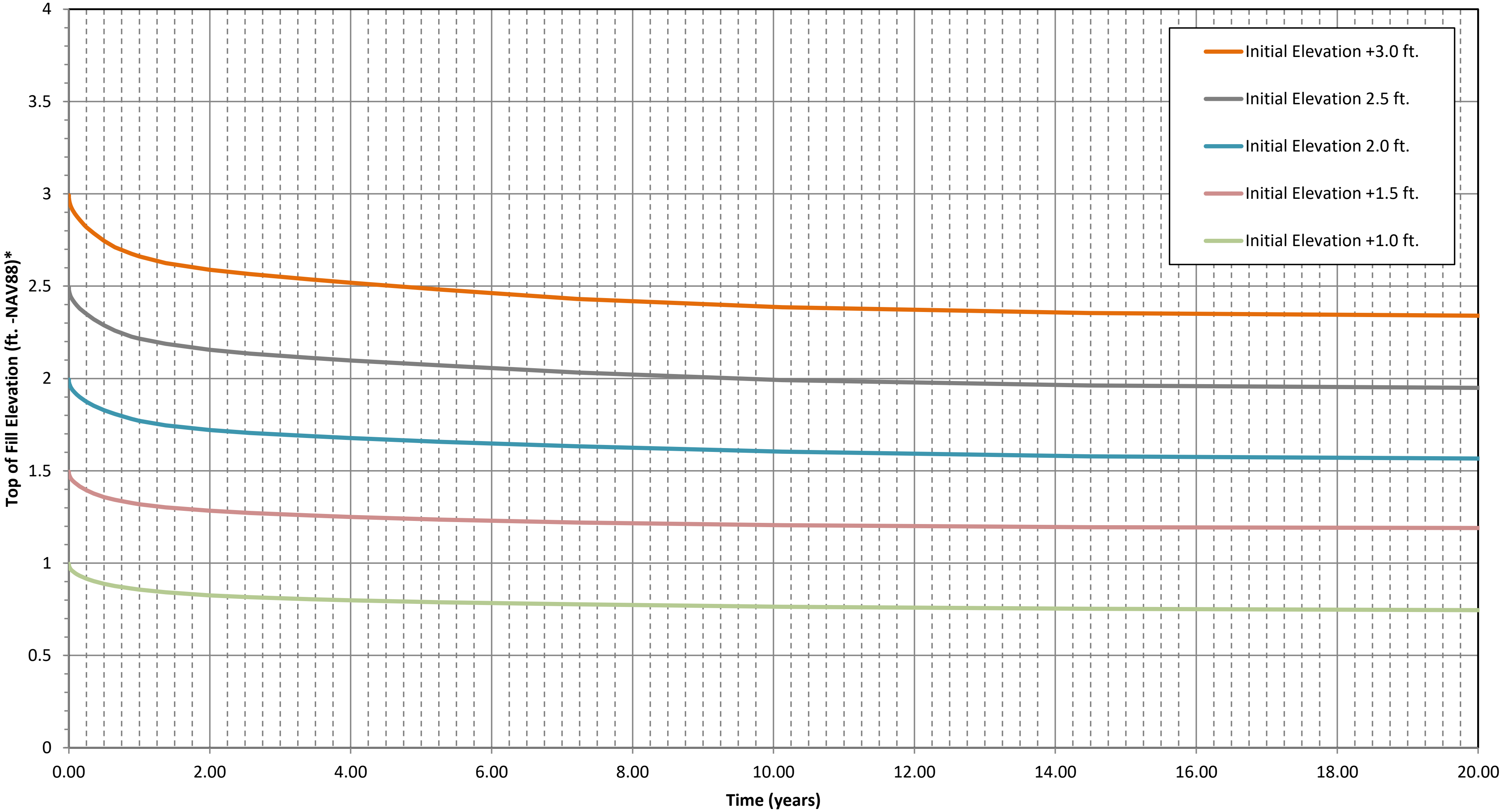


*Note - Sea floor elevation assumed to be -2.0 ft.-msl. Settlements are calculated as if fill was instantaneously placed on sea floor. This will assist in calculating required fill volumes. The occurrence of immediate settlement and settlement during filling operations should be considered when establishing the end of construction finish grade. Settlements presented above are based on generalized soil conditions for the soft soil areas identified in the geotechnical report. Variations in actual settlement may occur due to variable soil conditions.

Figure 1

Deer River Project - Time vs. Total Settlement - Better Soil Areas

(Total Settlement includes immediate and consolidation settlement)



*Note - Sea floor elevation assumed to be -2.0 ft.-msl. Settlements are calculated as if fill was instantaneously placed on sea floor. This will assist in calculating required fill volumes. The occurrence of immediate settlement and settlement during filling operations should be considered when establishing the end of construction finish grade. Settlements presented above are based on generalized soil conditions for the better soil areas identified in the geotechnical report. Variations in actual settlement may occur due to variable soil conditions.

Figure 2

APPENDIX E

CULTURAL RESOURCES ASSESSMENT

A PHASE I CULTURAL RESOURCES ASSESSMENT OF THE PROPOSED RESTORATION OF THE DEER RIVER TIDAL MARSH SHORELINE, MOBILE COUNTY, ALABAMA

Oblique Aerial View of the Project Area to the Southwest, Taken April 13, 2020 (Used by Permission of Sam St. John, Flythecoast.com)

Gulf South Past Recovery
Archaeology and Historic Preservation Consulting

A PHASE I CULTURAL RESOURCES ASSESSMENT OF THE PROPOSED RESTORATION OF THE DEER RIVER TIDAL MARSH SHORELINE, MOBILE COUNTY, ALABAMA

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April 2020

Table of Contents

List of Appendices

List of Figures

List of Tables

Introduction

This report describes a cultural resources assessment (archaeological and historical survey) of the proposed restoration of Mobile Bay's western shore, from the Theodore Ship Channel south to a man made canal just north of Renee Drive in Mobile County, Alabama. The project will be located in Section 5, Township 7 South, Range 1 West (U.S.G.S. Hollinger's Island, AL 7.5' quadrangle) (Figures 1-2). Project elevation is approximately 0-1 feet above mean sea level. The survey area includes approximately 13.4 acres of the larger 30-acre restoration area that is situated offshore.

The assessment conforms to the guidelines established by the State Historic Preservation Officer, Alabama Historical Commission, Montgomery. The assessment included a state site file search, historic literature search and field survey with subsurface testing. Principal Investigator for this project is Jason A. Gardner. The objective for this project was to identify any cultural resources that would be impacted by the proposed restoration activities along the existing shoreline. One archaeological site, 1Mb580, was located.

Project Description

The proposed project involves placement of structures and sediments along the shoreline adjoining the remnants of the north and middle forks of Deer River and the Theodore Ship Channel on the west side of Mobile Bay, for the purpose of restoring marshlands that had occurred there prior to 1979. Shoreline protection would be accomplished by placing linear breakwaters roughly 400 feet off the present shoreline, in water approximately 3 feet deep. The breakwaters would be comprised of rock or fabricated concrete and are intended to reduce wave energy at the shoreline. Restoration of eroded tidal marshes would be accomplished by placement of about 200,000 cubic yards of sediment along the shoreline, from the mean high-water line to water depths of about 3 feet in the Bay. Areas above the mean high-water mark are outside the project area and would not be disturbed. Sediment that has accumulated in Middle Fork of Deer River would also be removed as part of this project. This project would result in reestablishment of approximately 30 acres of emergent brackish marsh (Figure 3).

In accordance with guidance from the Mobile District Corps of Engineers, the study area for this cultural resource investigation was defined as the existing shoreline and immediate subtidal zone along the shoreline of the proposed restoration area. The Area of Potential Effects (APE) should be considered the entire project area, which comprises approximately 30 acres. There should be no additional effects on the surrounding area once restoration activities are completed.

Figure 1. Project Location Hollinger's Island, AL 7.5' USGS Topographic Map

Figure 2. 2017 Aerial Photograph of Project Area (usda.gov)

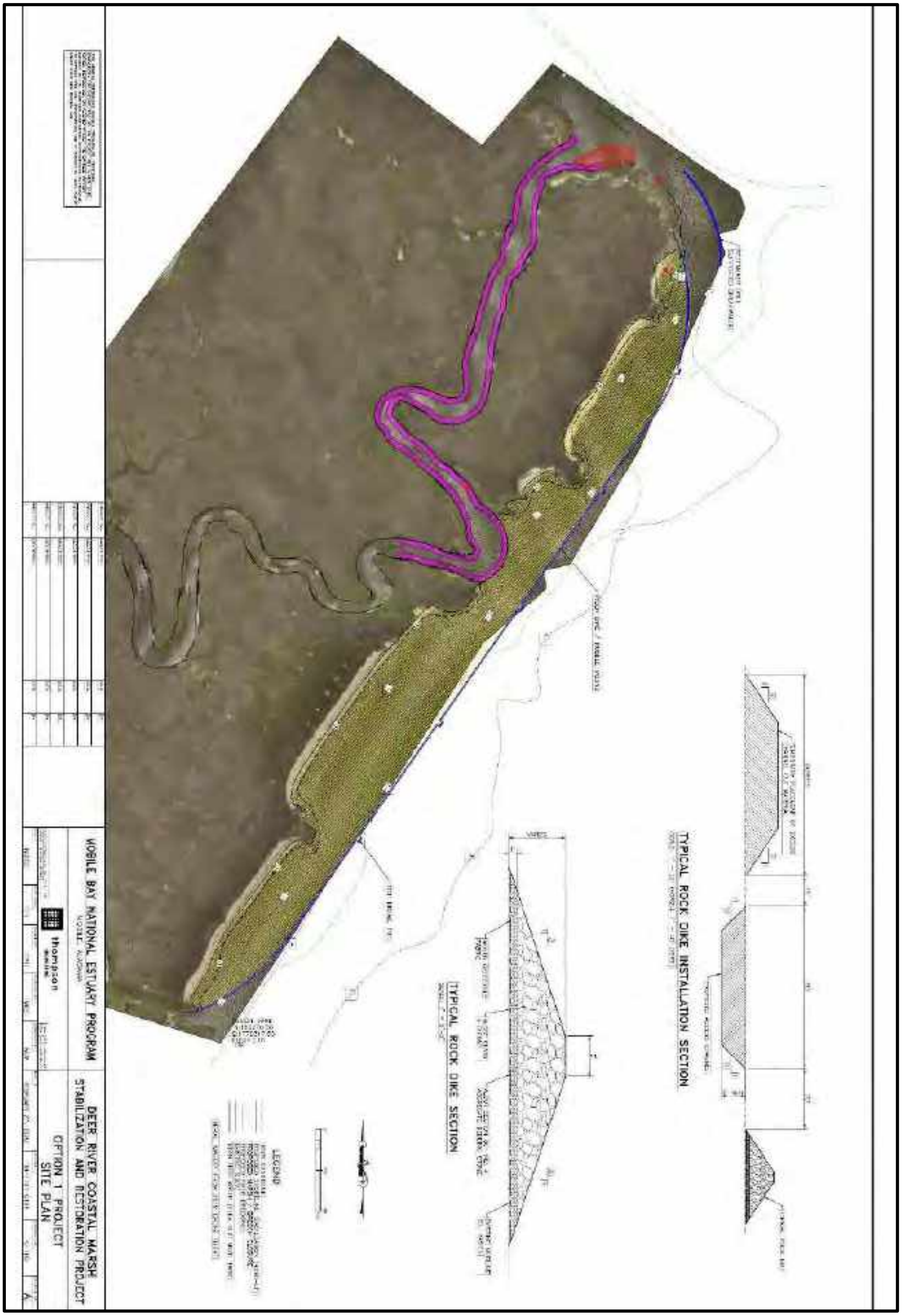

Figure 3. Proposed Layout of Restoration Activities

Archaeological and Historical Literature Search

Several Phase I cultural resources assessments and one Phase II site evaluation have been conducted within one mile of the proposed development (Figure 4; Table 1). These include:

Table 1. Phase I and II Cultural Resource Studies Within a 1-Mile Radius of the Project Area

Fuller, Richard S. and Diane E. Silvia	1983	Report on a Cultural Resources Reconnaissance of North American Gulf Terminals' Proposed Bulk Handling Facility, Mobile County, Alabama
Fuller, Richard S. and Diane E. Silvia	1984	Phase II Investigations of Sites 1Mb19 and 1Mb20
US Army Corps of Engineers	1987	Theodore Ship Channel Project Historical Report
Spies, Gregory C.	1998	An Archaeological Reconnaissance of South Deer River Estates near Bellefontaine in Mobile County, Alabama
Stowe, Noel R. and Rebecca N. Stowe	2000	A Cultural Resources Assessment of the Proposed Theodore Marine Terminal Development, Mobile County, Alabama
Stowe, Noel R. and Rebecca N. Stowe	2001	A Cultural Resources Assessment of the Proposed Bredero Price Company Facility Expansion in Southern Mobile County, Alabama
McDuffie, Julie E.	2011	A Phase I Cultural Resources Assessment of Two Parcels on Middle Road on Hollinger's Island, Mobile County, Alabama

The Alabama State Site Files (ASSF) were consulted in May 2019. The ASSF indicates that there are four previously recorded archaeological sites within one mile of the project area (Table 2).

Table 2. Previously Recorded Archaeological Sites Within 1 Mile of the Proposed Project

Site #	Cultural Affiliation	National Register Eligibility
1Mb19	Middle Gulf Formational; Mississippian	Undetermined (possibly destroyed)
1Mb20	Late Gulf Formational	Undetermined; (possibly destroyed)
1Mb338	Middle Woodland	Undetermined, (possibly destroyed
1Mb339	Middle Woodland	Undetermined (possibly destroyed)

Figure 4. Previous Surveys and Recorded Sites Within a 1-Mile Radius of the Survey Area

Sites 1Mb19 and 1Mb20 were further investigated during a Phase II in 1984 (Fuller and Fuller 1984). This report and its results are not available. These sites will not be affected by the proposed project.

Prior to the cultural resources assessment, the *Alabama Register of Landmarks and Heritage* and the *National Register of Historic Places* was reviewed. No previously identified structures currently listed on the *Alabama Register* or the *National Register of Historic Places* are located within the project area or within a one-mile radius of the proposed project area.

Historic aerial photographs and maps were also reviewed (www.alabamamaps.com; earthexplorer.usgs.gov; glorecords.blm.gov). No structures or other cultural or historical features were observed on these images.

The Natural Setting

Paleo Environment

When humans first occupied what is now the southeastern United States, approximately 11,500 B.P. (in some areas earlier), the environment was much cooler and drier than in subsequent periods. The coastlines extended much further south than their present locations, and plant and animal resources would have been adapted to the cooler, drier conditions (Anderson and Sassaman 2012:3-45). The cooler drier period lasted until the onset of the Hypsithermal during the Middle Archaic Period (ca. 4,000 to 3,000 B.C.), which corresponded to the retreat of the glacial sheets to their present stand. Climatic and other environmental conditions have been generally stable in the Southeast since that time (Brooks and Twaroski 2015, Delcourt and Delcourt 1979, Smith 1986, Watts 1980), with a dramatic increase in global temperatures and sea levels since the late 19th century.

Modern Environment

The project area falls into the Southern Coastal Plain Level III Ecoregion and Gulf Coast Flatwoods Level IV Ecoregion of Alabama (Figure 5). The Southern Coastal Plain ecoregion stretches across a large section of the Southeast and is generally flat with numerous waterways, swamps, marshes, and lagoons. It was once dominated by the longleaf pine, but now more diverse pine and other hardwood species have become the primary vegetation.

Further delineation of Level III ecoregions into smaller ecological units is possible (Level IV mapping):

> The Southern Coastal Plain extends from South Carolina and Georgia through much of central Florida, and along the Gulf coast lowlands of the Florida Panhandle, Alabama, and Mississippi. From a national perspective, it appears to be mostly flat plains, but it is a heterogeneous region also containing barrier islands,

coastal lagoons, marshes, and swampy lowlands along the Gulf and Atlantic coasts (Griffith et al. 2001).

and

the Gulf Coast Flatwoods is a narrow region of nearly level terraces and delta deposits composed of Quaternary sands and clays. Wet, sandy flats and broad depressions that are locally swampy are usually forested, while some of the better-drained land has been cleared for pasture or crops. Most of the Mobile urban area is also contained in this region. (Griffith et al. 2001).

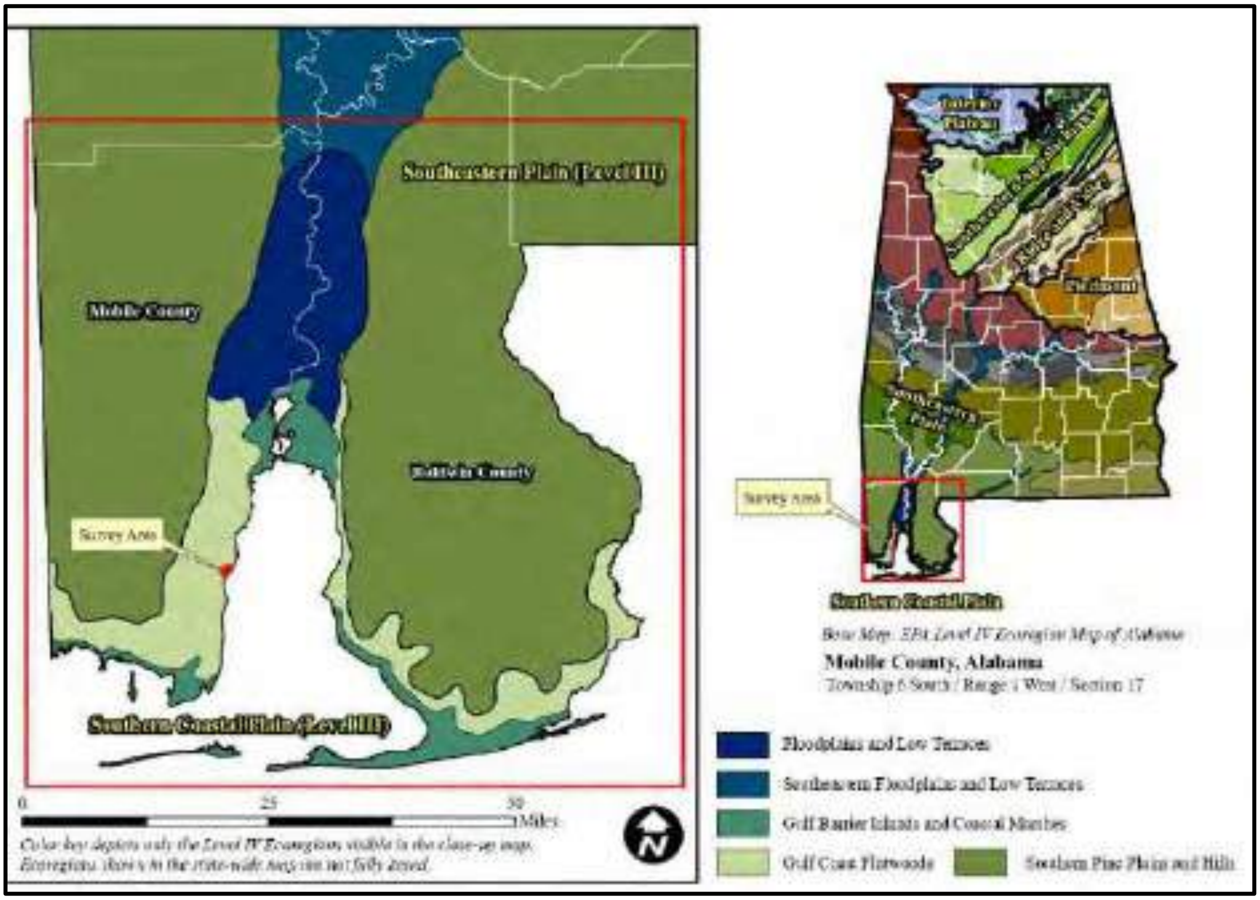

Figure 5. Level III and IV Ecoregions in the Vicinity of the Project Area (epa.gov)

Project Environment

The proposed project is located on Mobile Bay, south of the Theodore Ship Channel and the remnants of North Fork of Deer River. It is bordered on the south by a man-made canal that parallels Renee Drive. The western boundary is formed by the marshes associated with the North and South Forks of Deer River, and the eastern boundary is Mobile Bay. Soils in the project area have been described as mucky loams and are classified into the Lafitte Muck and Pactolus sandy loam associations (Figure 6, Table 3). Vegetation along the shore line includes various marsh grasses such Southern Cattail, Black Needlrush, Sturdy Bullrush, Sawgrass, Torpedo Grass, Reed Grass, Smooth Cordgrass, Giant Cordgrass, Salt meadow Cordgrass, Poison Bean, Hairy pod Cowpea, Alligator Weed, Morning-glory, Groundsel, Sump weed, Sweet scent, and Goldenrod (Figures 7-10). The project drains east into Mobile Bay. Jurisdictional wetlands comprise 13.4 acres or 100% of the project area. The project area has been previously disturbed by severe erosion associated with sea level rise, flooding, tropical storms, and the construction of the Theodore Ship Channel.

Mobile Bay is a drowned river valley that has been filling since the end of the Pleistocene. However, sea-level rise and tropical storm activity have resulted in a greater rate of erosion than deposition by the Mobile and Tensaw Rivers (Smith 1997). Geological research suggests the small Deer River watershed with its three forks are late Pleistocene or early Holocene in age (Smith 1997:112-113; 133).

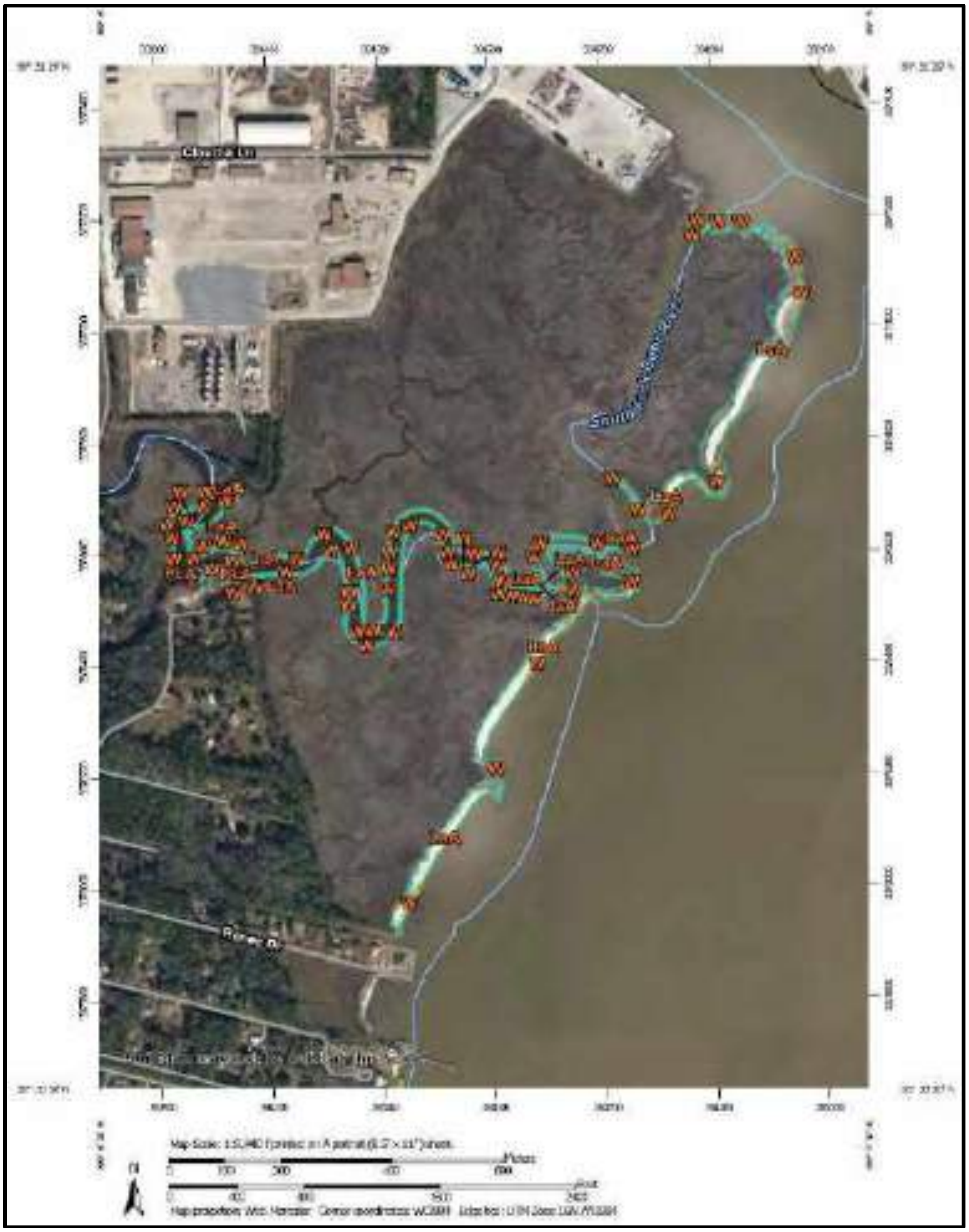

Figure 6. Soils Mapped Within the Project Area (usda.gov)

Table 3. Soil Types Mapped Within the Project Area (usda.gov)

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
LaA	Lafitte muck, 0 to 1 percent slopes	10.1	75.3%
PsA	Pactolus loamy sand, 0 to 2 percent slopes, rarely flooded	0.1	0.8%
W	Water	3.2	23.9%
Totals for Area of Interest		13.4	100.0%

Figure 7. South Fork of Deer River, View to the West

Figure 8. General Project View to the South Along Bay Shore


Figure 9. General Project View to the North Along Bay Shore (Mouth of Theodore Ship Channel)


Figure 10. General Project View to the West (Site 1Mb580 in Foreground)

The Cultural and Historical Setting

The following section is an overview of the regional prehistoric and historic cultural stages which provide a chronological framework for evaluating the research potential of Site 1Mb580 (Figure 11). The identified cultural periods related to the archaeological site date generally to the prehistoric Late Archaic and Early, Middle, and Late Historic Periods.

Stage	Period	Phase (Complex)	Date Range	Characteristics
	Late	(Ft. Stoddard)	A.D. 1770-1830	Early Americans; Mt. Vernon Arsenal; Alabama River Forts; Removal
Historic	Early	Port Dauphin	A.D. 1700-1770	Choctaw; Apalache; Confederation
	Early	Doctor Lake	A.D. 1699-1770	French Colonization; Colonoware
Protohistoric		Bear Point	A.D. 1550-1699	DeSoto/Luna 1st Contact; Depopulation from disease, upheaval
	Late	Bottle Creek II	A.D. 1400-1550	SECC Apogee
Mississippian	Middle	Bottle Creek I	A.D. 1250-1400	Pensacola Culture; Bottle Creek construction
	Early	(Andrews Place)	A.D. 1150-1250	Shell Tempering; Moundville, LMV Ceramic Styles Appear
	Late	Coden	A.D. 900-1150	Wakulla/Weeden Island II
	Late	Tensaw Lake	A.D. 750-900	Very Coarse Sand Tempering
Woodland	Late	Tates Hammock	A.D. 500-900	Weeden Island I; Gulf III and IV
	Middle	Porter	A.D. 200-500	Hopewell, Marksville, Santa Rosa, Swift Creek; Gulf II
	Middle	Blakeley	100 B.C. - A.D. 200	Deptford, Early McLeod; Gulf I Ceramic Tradition
	Late	Bryants Landing	700-100 B.C.	Alexander Culture
(Early Woodland)	Middle	(View Point)	1000-700 B.C.	Bayou La Batre Culture
Gulf Formational	Early	(Coon Neck)	1400-1200 B.C.	Fiber tempering
	Late	(Cypress Point)	3000-1400 B.C.	McIntire; Extensive trade in lithic raw materials; steatite appears
Archaic	Middle	Unnamed	6000-3000 B.C.	Savannah River, Morrow Mountain, Broad, High Median Ridge PPKs
	Early	Unnamed	8000-6000 B.C.	Kirk Serrated, little Big Sandys, Bellarcate Stemmed
	Late	Seed Tick?	8500-8000 B.C.	San Patrice, Dalton
Paleoindian	Middle	Unnamed	8800-8500 B.C.	Cumberland, Suwanee, Quad, Folsom?
	Early	Unnamed	8500 B.C.	Clovis; coastal sites probably eroded/buried/submerged

Figure 11. Outline of Southwest Alabama Provisional Culture History

Archaic (7000 B.C. to 1400 B.C.)

The Archaic Period has been divided temporally into Early, Middle, and Late periods (Anderson et al. 1996). The Early Archaic is dated from circa 7,000 to 4,000 B.C. Walthall (1980) notes that within the southeastern United States, the climactic changes from the Late Pleistocene to the Early Holocene were gradual and that a corresponding gradual variation can be seen in the material culture from Late Paleoindian to Early Archaic sites. It is widely considered that cultural adaptations were not markedly different between the two cultural divisions (Anderson et al. 1996).

Many of the lithic tools associated with the Late Paleoindian Period have also been identified at Early Archaic sites. Diagnostic notched or stemmed projectile points recognized to have been used during the Early Archaic in Alabama include Big Sandy, Palmer-Kirk series, Kirk Corner Notched, Kirk Stemmed, and bifurcate base types such as LeCroy (Coe 1964; Broyles 1968).

The Middle Archaic Period is dated from 4,000 to 3,000 B.C. It is differentiated from the Early Archaic by the prevalence of stemmed projectile points. Material culture markers for the Middle Archaic include the Stanly Stemmed, and Morrow Mountain Stemmed projectile points. Elliot and Sassaman (1995) note that the biface typology for the later Middle Archaic is both highly localized and divergent. Other Middle Archaic artifact types include a variety of ground and polished stone tools. The Middle Archaic Period is very poorly documented in southwest Alabama. One intact site upriver from Site 1Mb414 was reported by collectors and briefly visited by Archaeological Services, Inc. It appeared to be a stratified Archaic period site eroding from the bank of the Tombigbee River northeast of McCarty's Landing. This site, probably associated with the Tallahatta quartzite quarry at McCarty's Landing, has produced a large number of broad-stemmed Middle Archaic Points (Stowe Personal Communication 2008).

Late Archaic (3000 B.C. to ca. 1000 B.C.)

The Late Archaic Period is considered to have begun ca. 3000 B.C. Elliot and Sassaman (1995) note that by 1,000 B.C., the widespread diffusion of pottery throughout the Southeast, reorganization of local populations, and dispersed upland occupations mark the transition from the Late Archaic Period to the Woodland. Late Archaic populations experienced growth and continued regional adaptation, including exploitation of shellfish, riverine environments, and the development of soapstone cooking technology (Elliot and Sassaman 1995).

Late Archaic sites can be identified through the presence of large triangular bifaces with broad stems, referred to as Savannah River Stemmed projectile points. Steatite vessels are common throughout sites in the Southeast from this period. Around 2,500 B.C., the development of fiber-tempered ceramics is first noted along the Atlantic Coast, although the current evidence suggests that pottery was not used in the Piedmont until ca. 1,500 B.C. The earliest wares are the Stalling and St. Simons Island types in the east and small amounts of fiber-tempered pottery have been found in the west Georgia Coastal Plain (Elliot and Sassaman 1995). The chronological placement of fiber-tempered pottery in the vicinity of the area proposed for development is approximately 1200 B.C. (Stowe 1990). A large Late Archaic site was recently excavated on the eastern shore of Mobile Bay and led Stowe et al. (2007) to propose the Cypress Point complex for other sites in southwest Alabama demonstrating similar Late Archaic traditions such as the use of ferruginous sandstone in tool making.

Cypress Point: A Proposed Late Archaic Complex

This complex was first recognized in 2006 during data recovery excavations at1Ba556, the Cypress Point Site in Baldwin County (Stowe et al. 2007). It was further observed in the lowest levels of Site 1Mb414 in northern Mobile County (Gardner 2010). With just two sites looked at so far, it is hard to speculate on the geographic distribution, especially because some of the complex's characteristics may not have been recognized in earlier work in the region.

This complex is characterized by both the use of localized lithic materials for tool production (ferruginous sandstone) as well as the acquisition of far-flung materials for similar purposes such as TQ and agate (could be considered local), but also Coastal Plain Chert from southeastern Alabama or northwest Florida, Ft. Payne chert from the Tennessee Valley and hematite from the Ohio River Valley. However, Pickwick-like, McIntire-like, Tombigbee

Stemmed-like points were observed in the later deposits at 1Mb414, which suggests this type of lithic acquisition strategy was not unique to the Late Archaic Period.

Anderson and Sassaman (2012) place the onset of the Late Archaic Period in the Southeast at ca. 3800 B.C. The earliest cultural materials recovered from the excavations at 1Ba556 are relatively and absolutely dated to the Late Archaic period ca. 3980 B.C. to 2250 B.C. and 2190 B.C. to 940 B.C. at 1Mb414. At both sites these dates and artifacts were found stratigraphically 50 centimeters and below the site surface in the excavation units.

Other than the raw material for stone tools, there does not appear to be much participation in the so called "Poverty Point Interaction Sphere" common elsewhere in the lower Southeast. One pit feature interpreted as an earth-oven excavated at 1Mb414 produced hundreds of crudely made amorphous baked clay chunks and charcoal from this feature returned a Late Archaic date of 1720 B.C. No steatite sherds were recovered from either site; a few plain fiber-tempered sherds recovered from later levels at 1Mb414.

Another major characteristic of the Cypress Point Complex observed at both 1Ba556 and 1Mb414 was clusters of artifacts such as pecked and ground ferruginous sandstone slabs in direct association with a large quartz hammerstones, and occasionally a chunk of raw material such as TQ, or a few flakes or a core, usually in 2-4 artifact clusters; two or three of these clusters were observed in the Late Archaic zone at 1Ba556, and it looks like 5 were observed at 1Mb414 is the deepest part of the site, and which also returned Late Archaic C14 dates from their direct vicinity (Figures 12-14).

Figure 12. Late Archaic Toolkit from 1Mb414

Figure 13. Late Archaic Toolkit from 1Mb414

Figure 14. Late Archaic Toolkit from 1Ba556

Historic Period (1519-present)

Spanish I (1519-1699/1700)

During the sixteenth century, Spanish conquistadors, missionaries, and colonists began exploring the Southeast and contacted numerous native peoples during their journeys. On the north-central Gulf Coast, the earliest documented Spanish expedition was that of Alonso Alvarez Pineda in 1519, whose primary goal was "to find a passage west of Florida, [which was] still supposed to be an island" (Hamilton 1910:10). He named the bay and river he found Espiritu Santo, The Holy Spirit (McLaurin and Thomason 1981:9), (Delaney 1981), (Butler 2003), (Higginbotham 2001) (Figure 15).

In 1528, Panfilo de Narvaez visited Mobile Bay while exploring Florida to repair his boats and search for fresh water (possibly near present-day Bellefontaine) (Hamilton 1910, McLaurin and Thomason 1981, Delaney 1968). After many hardships, the entire expedition was lost, and the French later surmised "that the bones of his men were those found bleaching on Massacre (our Dauphine) Island" (Hamilton 1910:13).

Hernando de Soto was the next Spanish explorer to appear near the project area during his 1539-1542 expedition across the Southeast. Much of his exploration was of the inland areas, and culminated in a tremendous battle in October 1540 with native groups at the town of Maubila (Mabila, Mavila, Mauvila), thought to be located in the lower Tombigbee-Alabama River watershed (McLaurin and Thomason 1981; Hamilton 1910). After many losses on both sides, the expedition continued westward to the Mississippi River, and eventually the few surviving members reached Mexico two years later.

In 1559, Tristan de Luna y Arellano's fleet of 13 ships with 1500 colonists left Veracruz Mexico with the goal of colonizing Florida (Higginbotham 2001, Hamilton 1910). After reaching Mobile Bay, many hardships befell the group, including the first recorded hurricane on August 20, 1559, which resulted in a tremendous loss of Luna's armada and supplies (Higginbotham 2001, Delaney 1968). Despite several attempts to reorganize and recover, "bitter strife gradually developed, culminating in near mutiny and finally driving and desperate and defeated Luna (like others before him) to abandon this dreams of fame and fortune and sail back to Spain via Havana" (Higginbotham 2001:13-14).

The expeditions of Pineda, de Soto, and Luna are three of the major attempts at colonizing "La Florida" in the sixteenth century. Several smaller explorations were also made, but all ended unsuccessfully. A "notable pessimism among the authorities in Spain" (Higginbotham 2001:14) dissuaded any further attempts at exploration during remaining years of the sixteenth century and throughout the seventeenth century. Further attempts were not made at establishing permanent settlements on the Gulf coast until rivals France and England began their successful attempts in Canada and the north Atlantic coast, with hopes of spreading influence across the Gulf region.

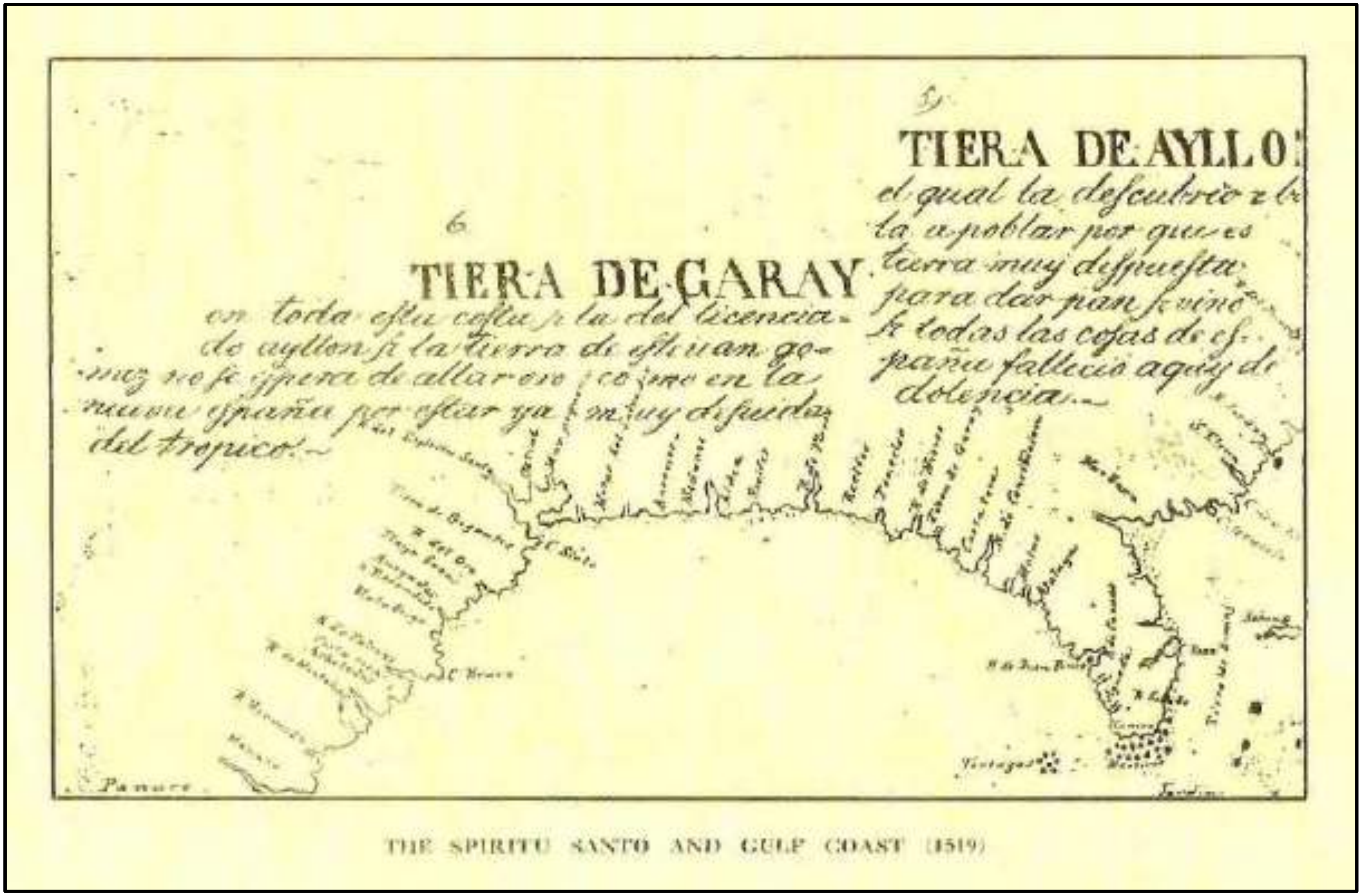

Figure 15. The Spiritu Santo and Gulf Coast (1519) (Hamilton 1910).

French Settlement and Occupation (1699-1763)

The French colonists of Canada were eager to establish themselves across the newly settled continent in the late 1600s, and began making a serious attempt at exploring the mouth of the recently discovered (by the French) Mississippi River. The explorer Robert Cavelier, known as La Salle, eventually reached the river mouth and the Gulf, and in 1682 “he took possession of the valley of the Mississippi and its tributaries and named it Louisiana for Louis XIV” (Hamilton 1910:41).

Soon after, La Salle returned to France to petition the crown for resources to begin serious colonization of the northern Gulf coast and Mississippi river valley. The French Canadian LeMoyne brothers Pierre (later Sieur d’Iberville) and Jean Baptiste (Sieur de Bienville), as well as Antoine (Sieur de Chateauguay), Joseph (Sieur de Serigny), and Jacques (de Sainte-Helene) were chosen to lead the colonization efforts of the region to thwart attempts at English settlement and the continued Spanish interests in the area (Hamilton 1910; McLaurin and Thomason 1981).

While exploring the Gulf coast in January of 1699, d’Iberville found the Spanish newly entrenched at Pensacola, so he pressed further west and established Ft. Maurepas in present day Ocean Springs, Mississippi (McLaurin and Thomason 1981; McWilliams 1981). Shortly after, he and Bienville decided to move the colony to the Mobile area, first to Twenty-Seven Mile bluff on the Mobile River in 1702, and later (because of a devastating month-long flood of the town) to the present location of the city at the mouth of the river in 1711.

Despite a few of the same setbacks that had hindered Spanish attempts at permanent settlement, including hurricanes, food shortages, attacks by natives, and yellow fever epidemics, the French-Canadian colonists were a great deal more successful in their efforts. The French colony of Louisiane with Mobile (and later New Orleans) as its capitol would remain until 1763, when France finally lost the French and Indian wars and "relinquished to Great Britain all claims to territory east of the Mississippi, except for the Isle of Orleans" (McLaurin and Thomason 1981:17).

Iberville named most of the tributaries of the Bay during their initial exploration in the early 1700s (McWillams 1991). Dog River (Riviere aux Chiens) and Deer or Roebuck River (Riviere aux Chevreuil) were named at this time, as French settlements were being established at the mouth of Dog River.

North of the project area, extending to Dog River, was the Rochon-Demouy-Hollinger land holdings, that most likely included the current project area and newly recorded site 1Mb580. Deer River is mentioned as the southern boundary of these holdings in several references (Hamilton 1910; Owen 1921; Waselkov and Gums 2000; Figures 16-17; See also Figure 20).

Additionally, the area in the general vicinity of the project area and Site 1Mb580 may have been part of Charles Mioux's plantation in the 1700s. Hamilton (1910:515) states that he "had a house and plantation there, part of his unsurveyed tract extending from Pierre Baptiste on the south to Deer River (Rio del Gamo)."

To the south of the project area, the area of Mon Louis Island was among the earliest French land grants in the region and was given to Nicholas Boudin in 1710 (McWilliams 1981). Descendants of Boudin are still found in the area.

Figure 16. Detail of 1733 Map by Baron DeCrenay (Waselkov and Gums 2000:1)

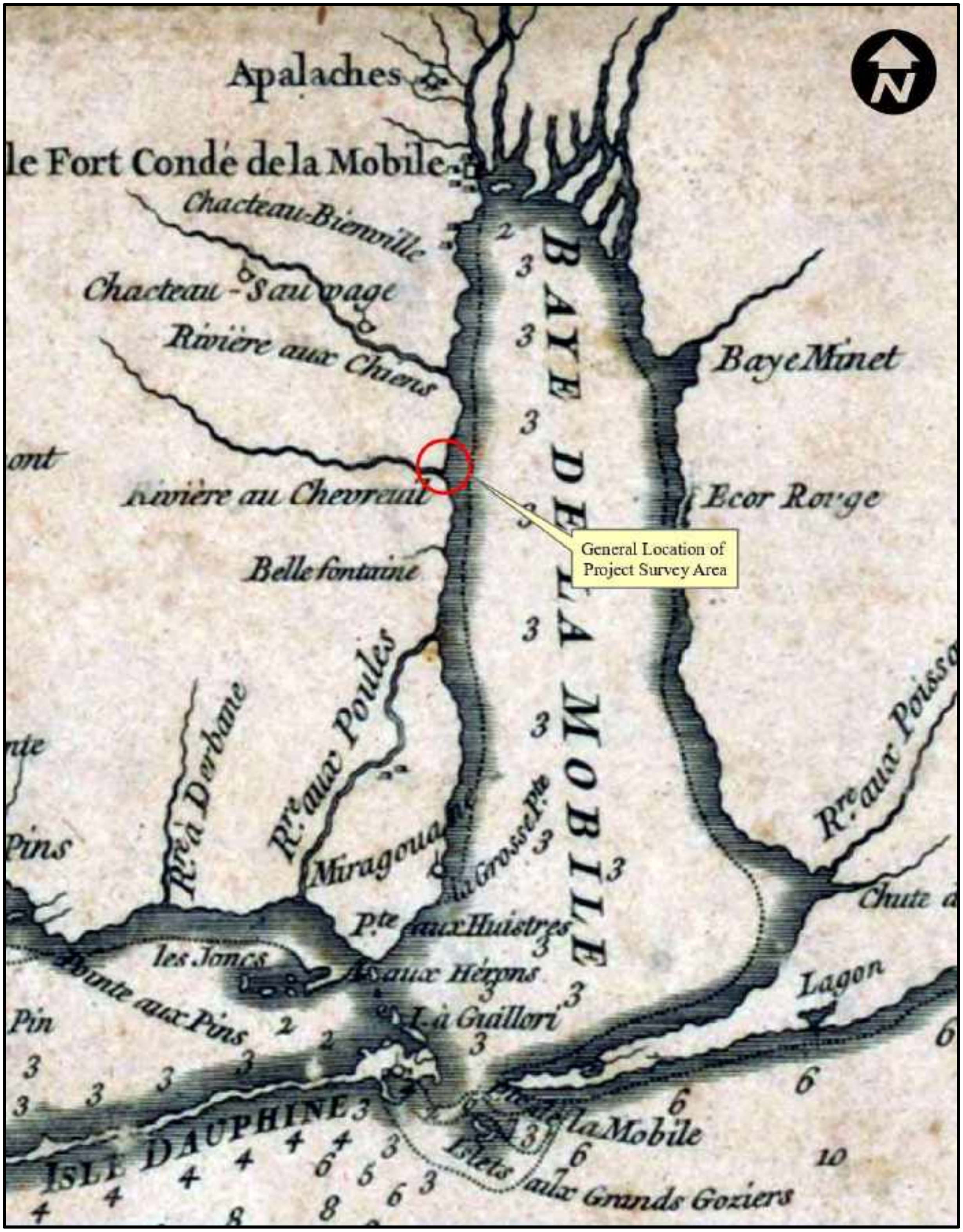

Figure 17. 1752 Plan of the Bay and Island of Mobile by D'Anville (alabamamaps.com)

British Occupation (1763-1776)

During the short 13-year period when Britain occupied Mobile and "West Florida", the primary activities taking place were fur trading, and stabilizing relationships with the numerous fractious native groups (Butler 2003). In 1779 Urbane (Orbanne) Demouy bought the Rochon holdings totaling about 8,866 acres (Waselkov and Gums 2000:79) (Hamilton 1910:496-497) (Figures 18-19).

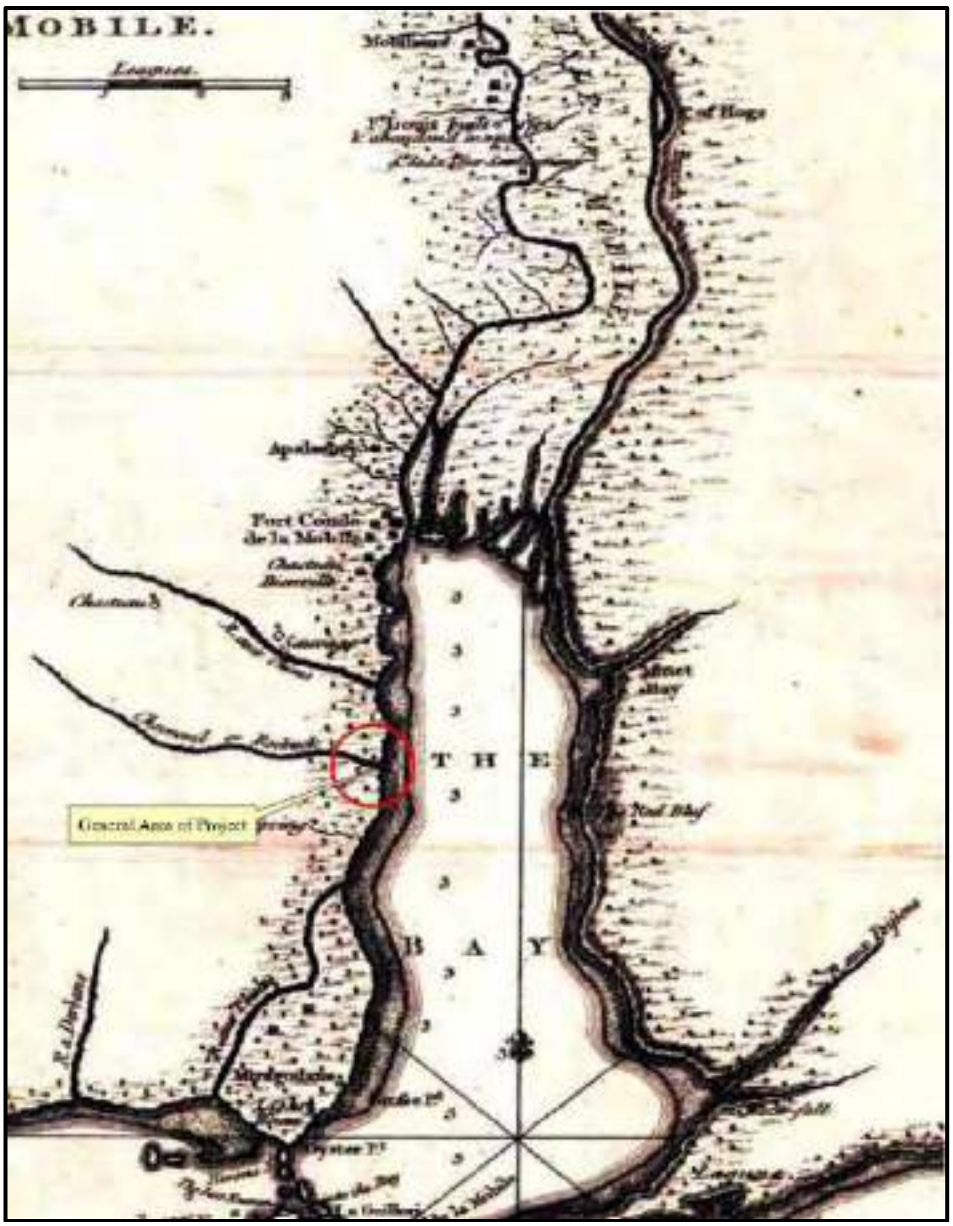

Figure 18. 1763 Plan of the Bay and Island of Mobile by Thomas Jefferys (alabamamaps.com)

Figure 19. 1775 Anonymous, Detail of "Land Granted and Surveyed on the River and Bay of Mobile" (Waselkov and Gums 2000) With General Project Location in Red Circle

Spanish II (1776-1814)

With Britain's loss in the American Revolution, Spain maneuvered and captured their holdings in West and East Florida, including Mobile, New Orleans, Natchez and Pensacola (McLaurin and Thomason 1981). Spain allowed numerous British and American settlers to remain in the region, but the Treaty of San Lorenzo in 1795 favored the newly established United States claims in the area, and greatly reduced Spanish influence (McLaurin and Thomason 1981). In 1799 Spain gave much of its land holdings north of the Rio Grande River to France, which then sold the holdings to the United States in 1803 as the Louisiana Purchase. President Thomas Jefferson considered Mobile a part of this claim, although Spain disputed this (McLaurin and Thomason 1981, Butler 2003).

As a result of the War of 1812 a few years later, the American forces captured Mobile from the much-weakened Spanish in 1813. A victory over the British during a naval engagement at Ft. Bowyer (now Ft. Morgan) (McLaurin and Thomason 1981) and Andrew Jackson's victory over the British and Creek Indians at the Battle of Horseshoe Bend in 1814 were two decisive final battles and "this victory and Mobile's capture ultimately resulted in the opening of the interior of Alabama and Mississippi to American settlement" (Butler 2003).

Early American (1814-1861)

In the early nineteenth century, Mobile began experiencing an unprecedented rate of growth, spurred by newly arriving American settlers, and improvements and expansion of the port. Cotton produced by plantations on the Alabama River in the interior made its way through the port at an increasing rate every year, and its economic impact helped Mobile transition from a frontier town to a more mature, established city (McLaurin and Thomason 1981, Doss 2001) (Figures 20-23).

Figure 20. 1828 Plat Map (blm.glo.gov)

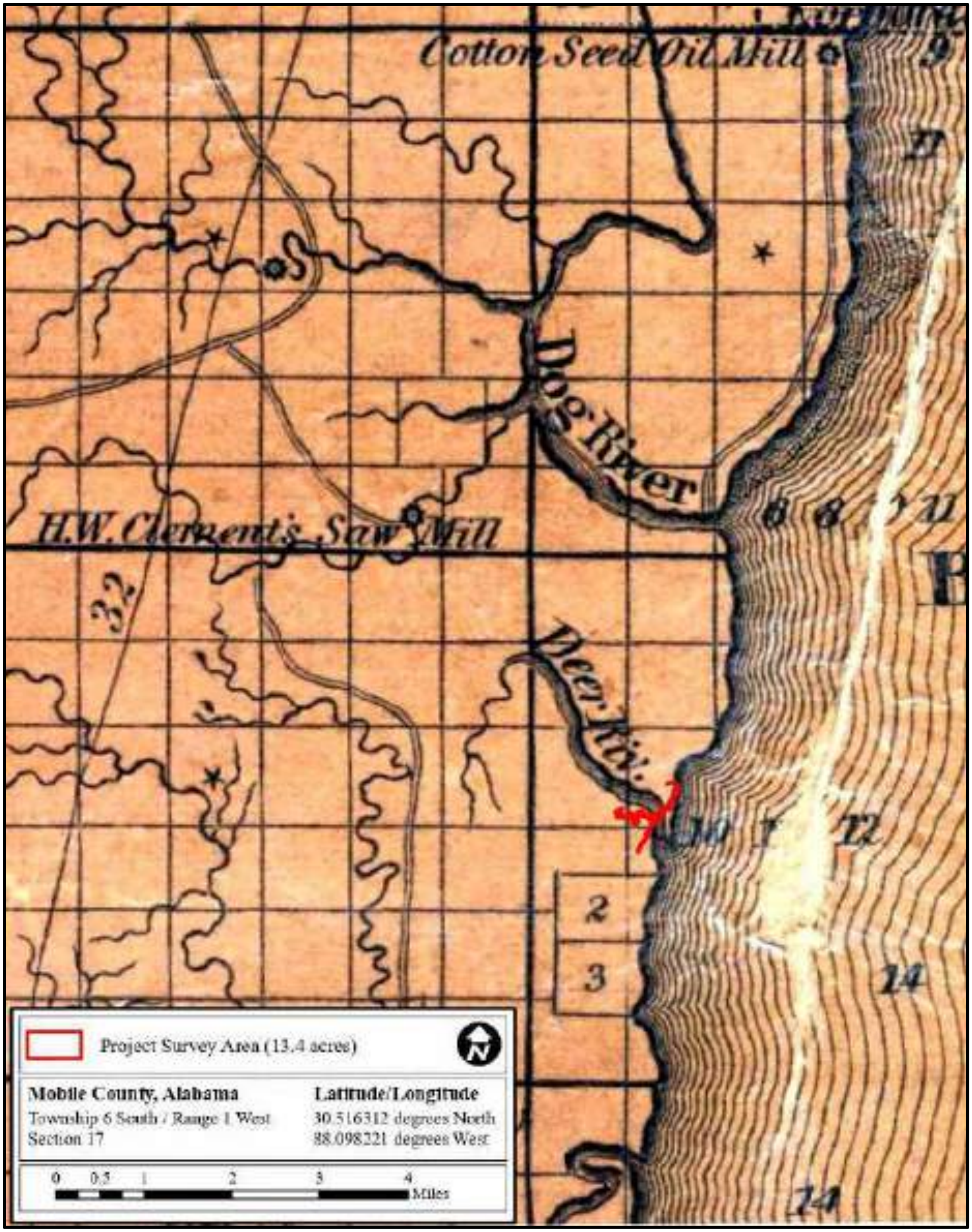

Figure 21. 1837 "An Accurate Map of Alabama and West Florida" by John LaTourrette (alabamamaps.com)

Figure 22. 1849 U.S. Coast Survey Map (alabamamaps.com)

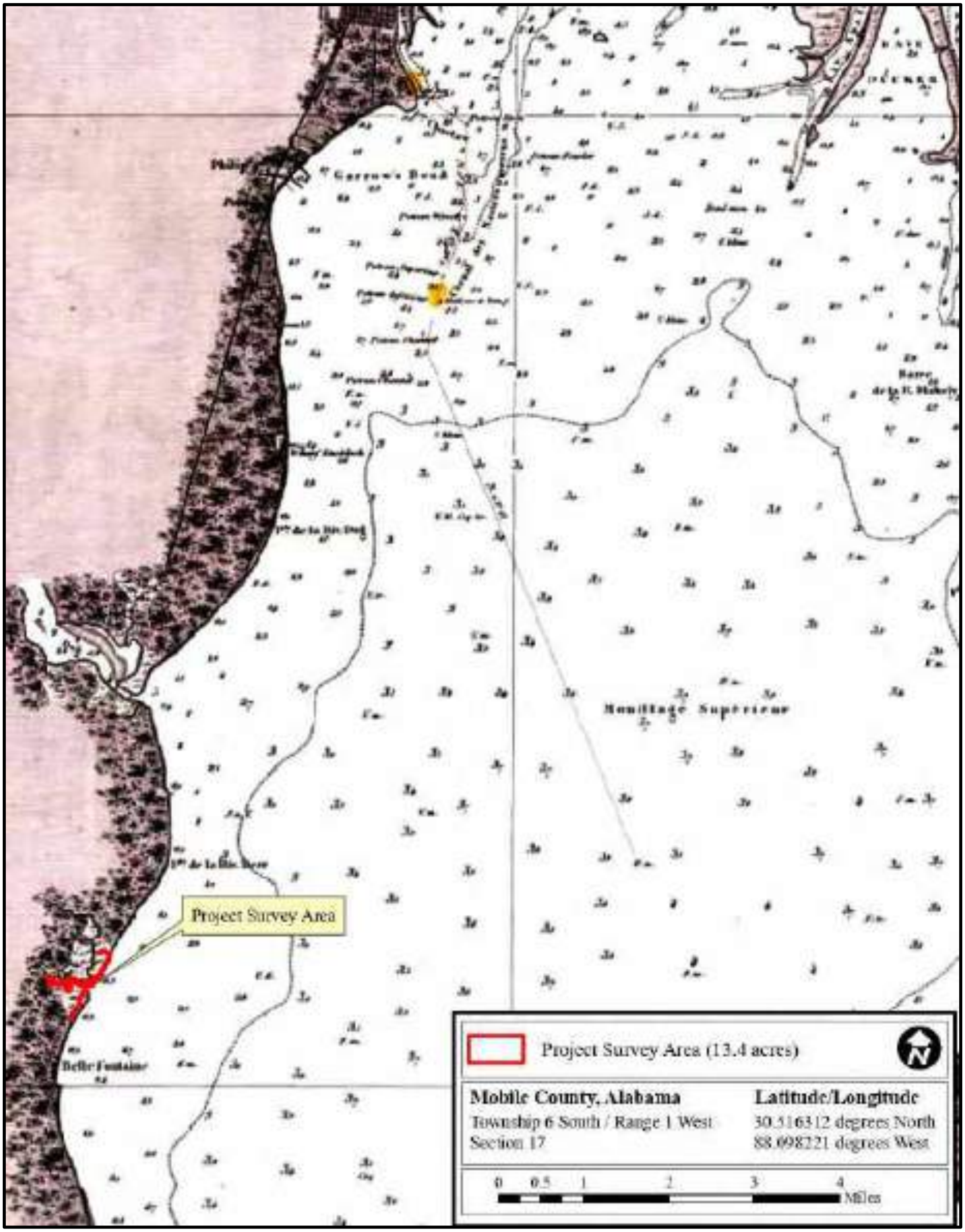

Figure 23.1856-67 “Baie de Mobile” (alabamamaps.com)

"Modern" History

The area surrounding the project area remained in the Demouy family until the early 1830s, when about a third of it was sold to the Montgomery family (Waselkov and Gums 2000). This was a brief occupation and in 1834 Adam Hollinger, Jr. bought around 6000 acres of the former Rochon land holdings in the area, and built two saw mills on the property, including one on Deer River "at the southern tip of his island property" (Waselkov and Gums 2000:106). From the mid-to-late 1800s until the early 1930s the property changed hands numerous times but was generally not occupied aside from the sawmills, and other agricultural activities such as cattle grazing, hunting and fishing (Waselkov and Gums 2000) (Figures 24-26).

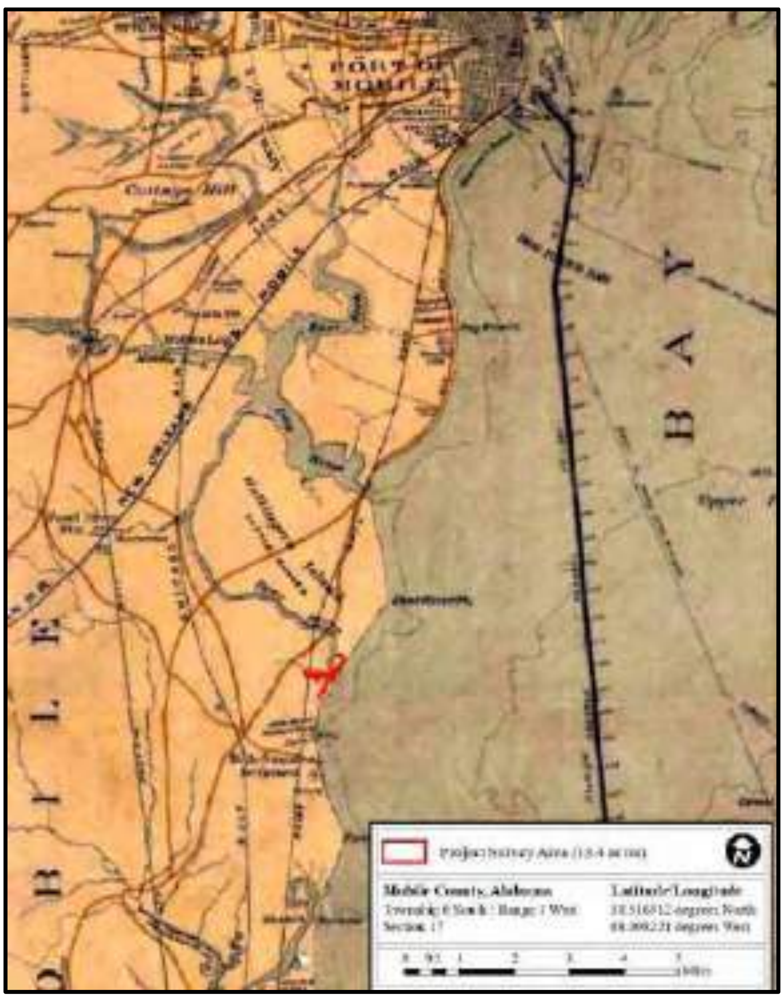

Figure 24. 1889 "Reference Map of Mobile and Vicinity" by Paul C. Boudousquie (alabamamamps.com)

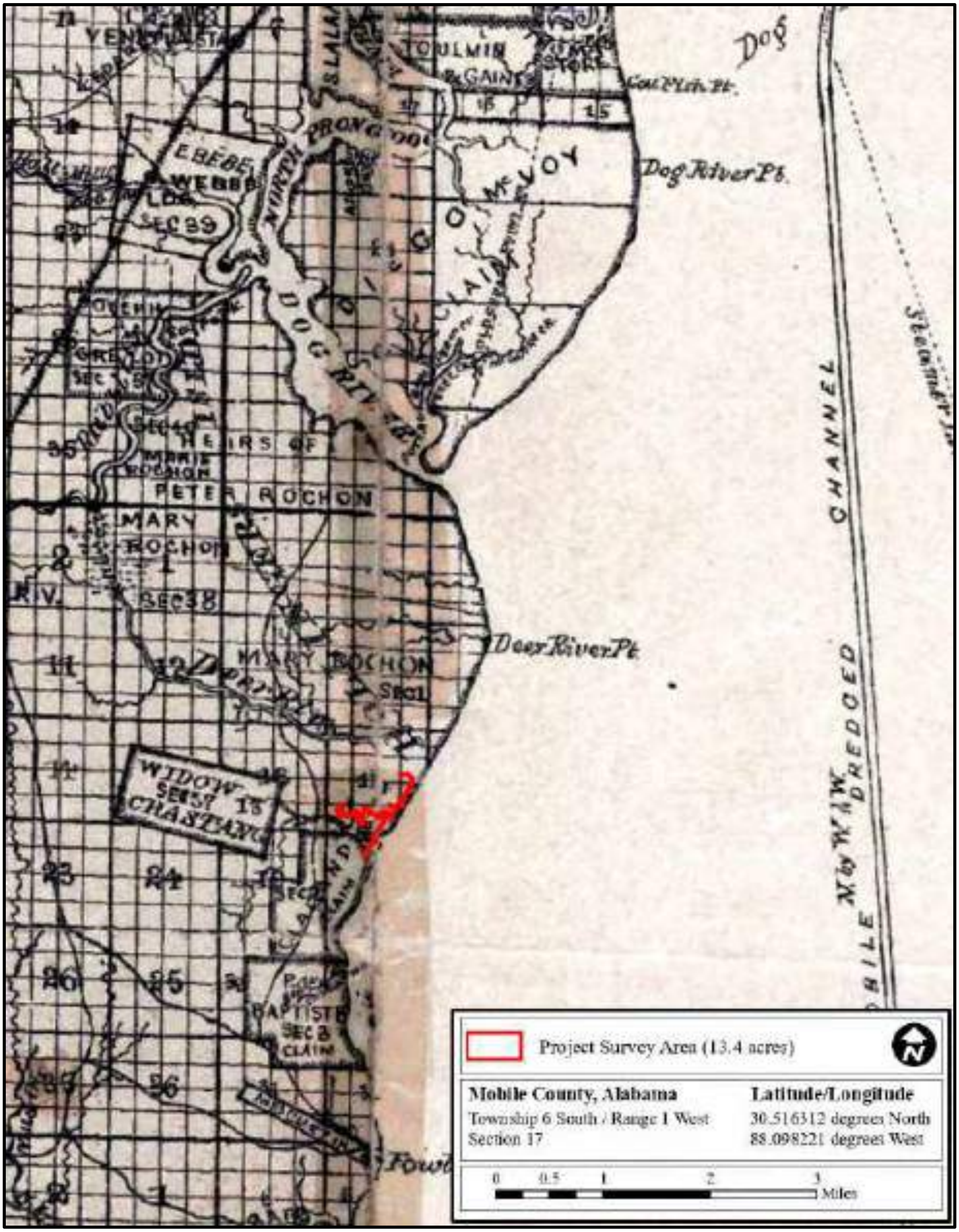

Figure 25. 1895 "Township and Sectional Map of Mobile County" by Henry Fonde (alabamamaps.com)

Figure 26. 1907 "Widell's New Sectional Map of Mobile County" by Theodore Widell (alabamamaps.com)

Beginning in the 1940s, an ammunition depot was established about 2 miles inland from the Bay, so the Hollinger's Island Ship Channel was created from the Mobile ship channel to the Bay shore in the vicinity of the current expanded barge canal. As industrial development increased in this area throughout the mid to late 20th century, this channel was expanded into a large canal that significantly impacted the original course of North and Middle Fork of Deer River, and most likely impacted Site 1Mb580, at least in part (Figures 27-32).

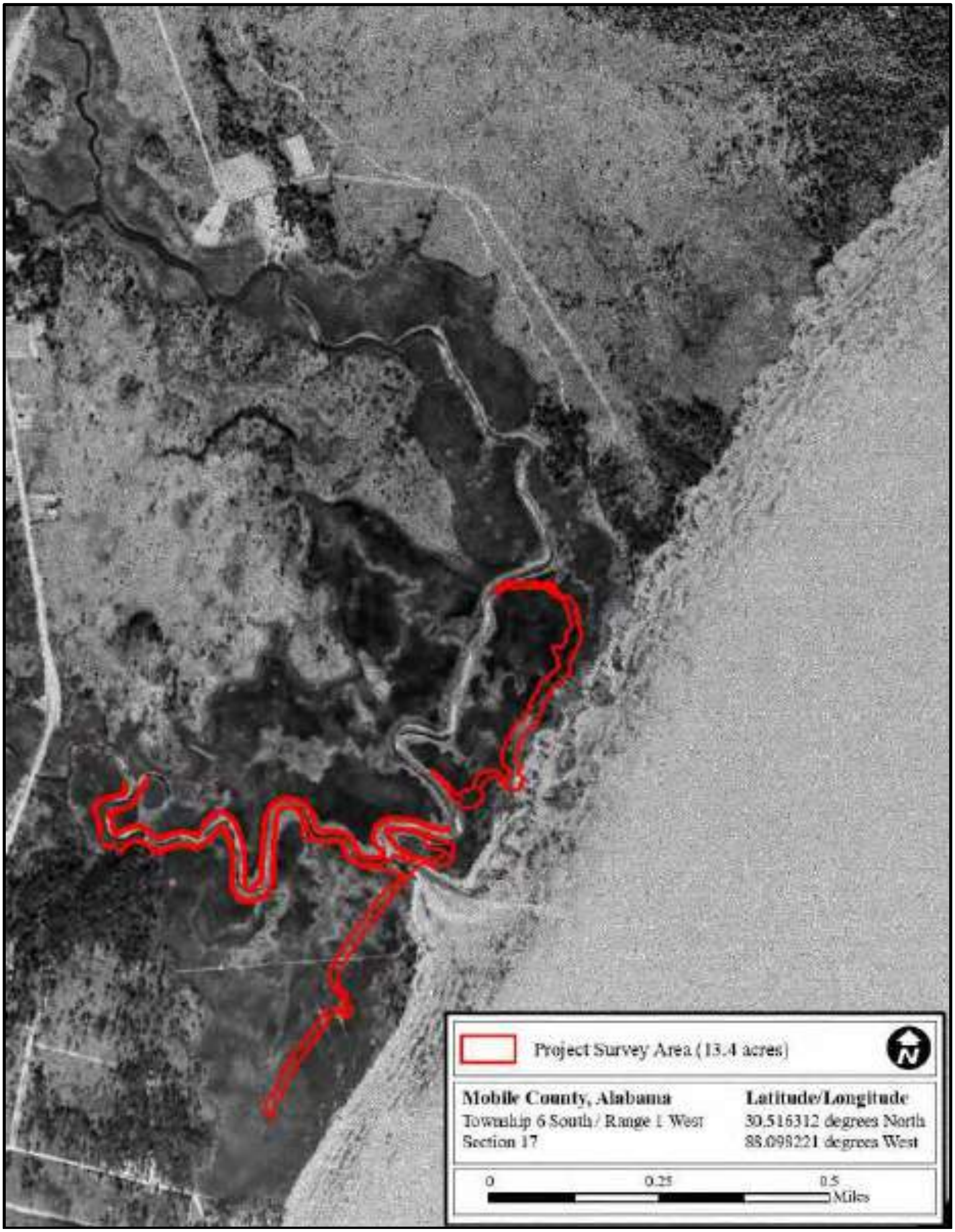

Figure 27. 1938 Aerial Photograph (alabamamaps.com)

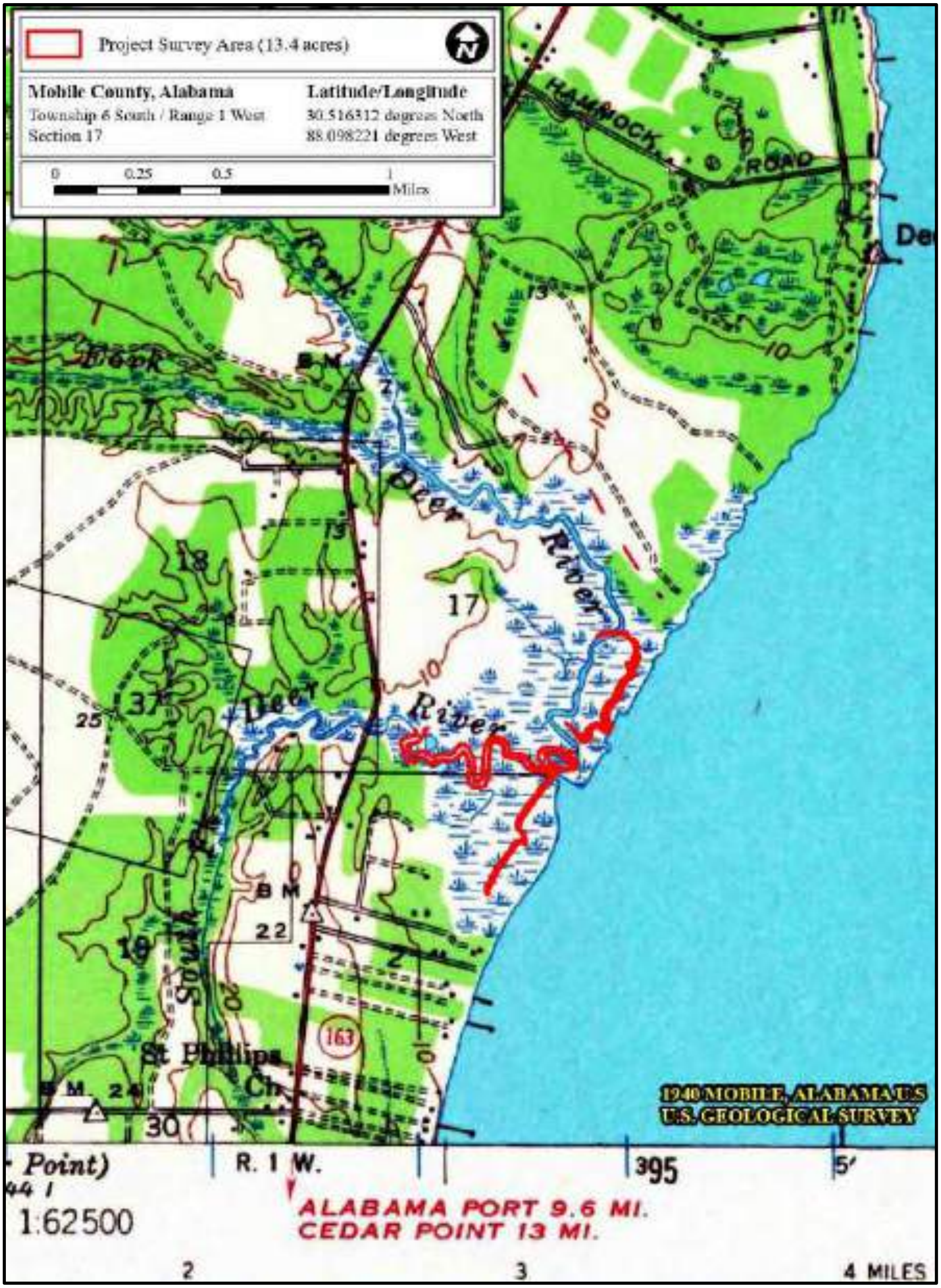

Figure 28. 1940 Map; The 1Mb580 Site Area was Clearly Once Connected to the Land to the North (alabamamaps.com)

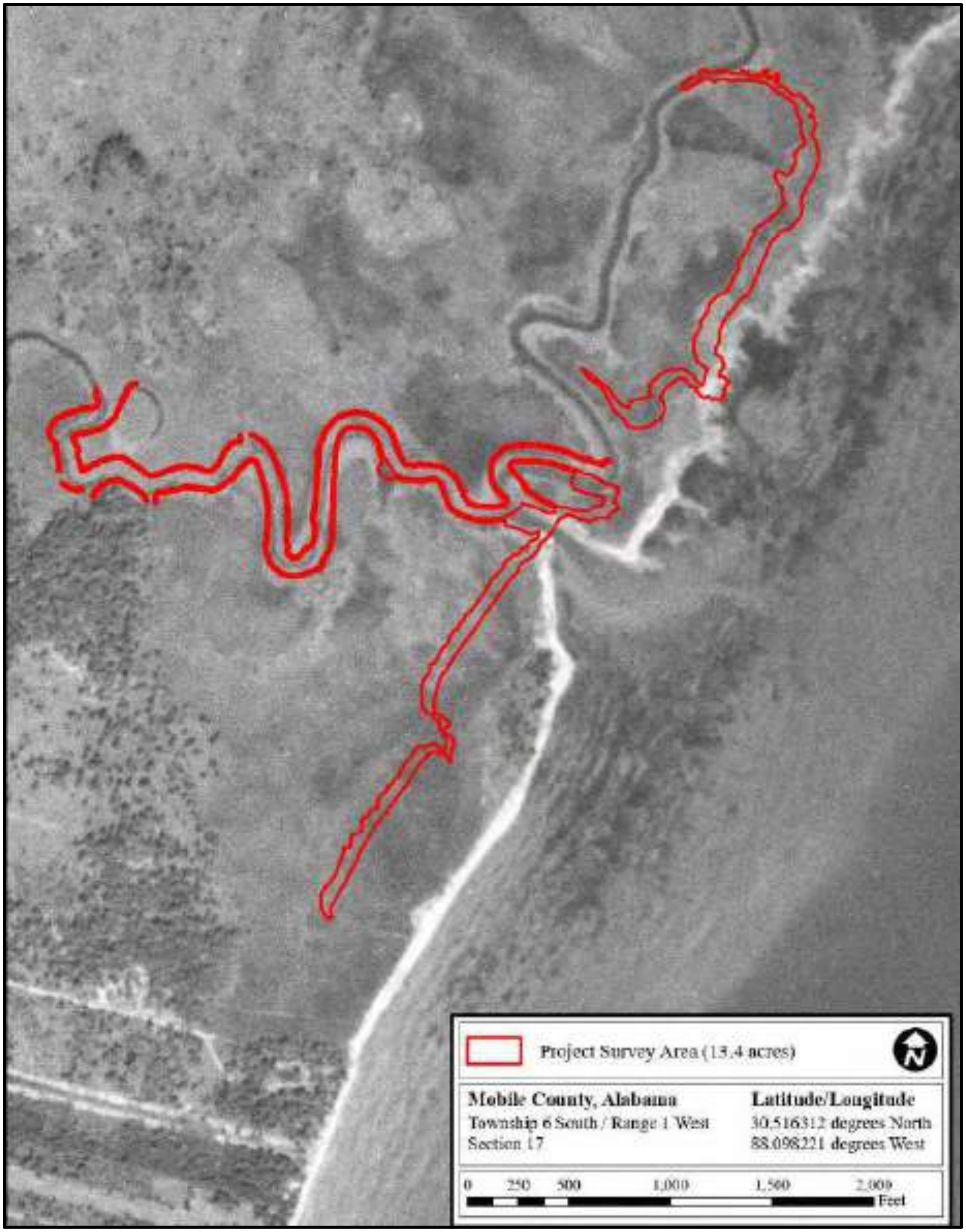

Figure 29. 1940 Aerial No Indication of Hammock or Site is Evident (alabamamaps.com)

Figure 30. 1952 Aerial (The New Ship Channel is in Evidence in the Upper Right) No Indication of Hammock or Site is Evident (alabamamaps.com)

Figure 31. 1967 Aerial; No Indication of Hammock or Site is Evident (alabamamaps.com)

Figure 32. 1974 Aerial Photograph with Nascent Barge Canal Excavated into the Bay Shore (alabamamap.com)

Field and Laboratory Methods

In November 2019 and March and April 2020, Jason Gardner, Jeremy Jones, Matthew Stowe, and Tim Thibaut conducted a pedestrian and boat survey of the entire project. Shovel tests were excavated at 30-meter intervals along the existing shoreline. Eighty-seven shovel tests were excavated across the survey area (Figure 33). The tests measured approximately 30 centimeters in diameter and were excavated to sterile subsoil. All soil was screened through 6.5 mm (¼") hardware cloth. All shovel tests were plotted on a map, recorded in the field and backfilled. Most of the shovel tests revealed a profile of dark grey muck typical of marsh environments; other shovel tests contained sandy clays associated with beach zones (see Figures 40-41). Subtidal areas immediately adjacent to the shoreline were probed with a 60-inch long steel rod, to test for potential shell deposits below the sediment surface. Since shore of the South Fork of Deer River's banks are primarily salt marsh, this area was visually examined from the boat with occasional stops for probing with the metal rod, particularly of a small oak hummock present of the south shore of the fork. The most extensive shovel testing took place on the Bay front, particularly in the vicinity of the documented archaeological site.

Because of the narrow footprint of the shoreline we usually were not able to delineate the site in all 4 cardinal directions, so we used a combination of surface distribution of cultural materials and shell and 10-meter intervals (combined with the probing of intermediate areas between these intervals) to delineate the site boundary. Multiple digital photographs were taken.

Figure 33. Shovel Test Locations (Black Dots)

All recovered artifacts were returned to the laboratory facilities of Gulf South Past Recovery. Provenience assigned in the field was continued. The artifacts were washed, air-dried, and rough sorted into material classes. Bottles were the primary artifact recovered, although 2 prehistoric projectile points, fragments of ferruginous sandstone and quartz hammerstones, as well as historic ceramics and metal were also collected. These artifacts were counted and weighed to the nearest 10^{th} of a gram (Table 4).

Results of the Survey

One archaeological site was recorded as a result of this cultural resources assessment. Site 1Mb580 was located in the northern part of the project area (Figure 34), along the eroding marsh edge along the Theodore Ship Channel and south along the Bay front.

Site 1Mb580

Site 1Mb580 is a generally linear scatter of clam shell and prehistoric and historic artifacts, as well as modern debris. It stretches from northeast to southwest along the eroding shoreline of the Theodore Ship Channel south along the Bay front for a distance of approximately 461.77 meters, with a width that averages 10 meters.

Based on the recovered diagnostic artifacts, it appears this site originated as a Late Archaic Cypress Point Complex Shell Midden on or near the shore of the North Fork of Deer River and as later used as a refuse dump during the historic period, beginning with the later occupants of the Rochon and Demouy plantations in the middle 1700s, and continued in this manner until the middle 20th century. A large quantity and diversity of material exists along this now active Bay and Canal shore, much of it covered by water most of the time. Because archaeological sites of this age and size with such a large quantity of material are rare on Mobile Bay's western shore, we recommend avoidance or Phase II evaluation for this site, since its information potential is unknown at this stage of investigation.

Figure 34. 2017 View of Site 1Mb580 in Yellow and “Modern” Shorelines

Fifty-two shovel tests were excavated at 10-meter intervals along the length of this scatter and although artifacts are concentrated on the site surface, none were recovered below the surface (see Figure 35). A great deal of dark mucky soil is present in this area that may be attributed to midden material, but also may be indicative of the former salt marsh substrate (Figures 36-39).

Figure 35. Site 1Mb580 Sketch Map, Includes Shovel Tests and Surface Finds

Figure 36. Dark Soil and Shell on the Surface of 1Mb580, View to the Northwest

Figure 37. General View of Site 1Mb580 to the North


Figure 38. General View of SiteMb580 to the South


Figure 39. General View of Site 1Mb580 to the North


Figure 40. Typical Shovel Test Profile at 1Mb580


Figure 41. Typical Shovel Test Along the Beach and Hummock Areas

Table 4. Artifacts Recovered From Site 1Mb580

Provenience	Artifact Type	N=	Weight (g)	Comments
Surface	Clear Bottles (whole)	15	2536.25	Alcohol, Cosmetics, Medicine, Condiments, Soda
	Dark Green Bottle Glass (fragments)	3	306.49	Mid-18th century wine bottle fragments
	Slip Decorated Redware	3	35.89	Mid-18th Century Plate/Bowl Fragments
	Aqua glass	4	1121.55	3 Whole bottles and 1 base fragment
	Cobalt Blue Bottle (whole)	1	142.42	Phillips Milk of Magnesia
	Albany Slip Stoneware Jug fragments	4	295.91	Interior Slipped Only
	Salt Glazed Stoneware Jug fragment	1	45.61	Exterior Glaze Only
	Herty Cup Fragment	1	40.96	
	Flint Creek/Pontchartrain PP/K	2	63.42	Citronelle Gravel Chert
	Ferruginous Sandstone Slabs	14	871.38	Pecked and Ground
	Quartz Hammerstone Fragments	4	532.17	
	TOTALS	**53**	**5992.05**	

Figure 42. Late Archaic Flint Creek/Pontchartrain Points/Knives from 1Mb580 Surface

Figure 43. Cypress Point Complex-Related Artifacts, Including Pecked and Ground Ferruginous Sandstone Slabs and Quartz Hammerstone Fragments

Figure 44. Mid-Late 18th Century Artifacts From Site 1Mb580, Top Row: Slip Decorated Redware; Bottom Row: Dark Green Olive Wine Bottle Fragments

Figure 45 shows how the shoreline in the project area has changed just since 1938, with almost 300 feet of shoreline loss. It appears that 1Mb580 was not located on the Bay shore originally and may have even been accessed from the north fork of Deer River or by land before it was cut off by the Ship Channel and erosion.

Figure 45. Shoreline Change 1938-2017

There have been few if any prehistoric sites systematically evaluated during the modern period on Mobile Bay's western shore. Archaic sites and shell middens of this age in particular are rare in the Mobile Bay area. Additionally, historic refuse dumps related to the colonial and early American settlements along the Bay have only been identified and investigated at 1Mb161 the Dog River site, and additional investigations of these prehistoric and historic features could expand our knowledge of Mobile's past significantly.

Summary and Recommendations

This report describes a cultural resources assessment of a proposed shoreline restoration area at Deer River on the western shore of Mobile Bay in Mobile County, Alabama. This assessment included a review of the archaeological literature (state site files), historic literature and records and a field survey with subsurface testing. As a result of this survey one new archaeological site was recorded (1Mb580). Site 1Mb580is a large scatter of shell, midden, and artifacts located on the northern part of the proposed restoration area. Because of the age, quantity, and ubiquity of the artifact collection, and the rarity of sites of this type on Mobile's western shore, we recommend avoidance or Phase II testing of Site 1Mb580 in any proposed restoration activities in the site's vicinity.

Curation

All field notes, photographs, artifacts, and a copy of this report will be curated at the Erskine Ramsay Archaeological Repository at Moundville, Alabama (see Appendix A for curation agreement). This repository meets the Department of Interior 36 CFR Part 79 guidelines for curation of materials. A copy of this report will also be kept on file at Gulf South Past Recovery, Mobile, Alabama.

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APPENDIX A

CURATION AGREEMENT

THE UNIVERSITY OF ALABAMA | University Museums
Office of Archaeological Research

April 9, 2019

Jason A. Gardner
Gulf South Past Recovery
Box 8072
Mobile AL 36689

Dear Jason:

As per your request, this letter is to establish an agreement to provide curation services to Gulf South Past Recovery on an as-needed basis.. We are recognized by a variety of Federal agencies as a repository meeting the standards in 36 CFR Part 79 and have formal agreements to provide curation under these guidelines to agencies such as the Department of Defense, National Park Service, U.S. Fish and Wildlife Service, U.S. Soil Conservation Service, U.S. Army Corps of Engineers, Tennessee Valley Authority, National Forest Service, etc.

Please be advised that once a year we must be notified of all reports in which we were named as the repository. Project collections must be submitted within one calendar year of completion. Small projects may be compiled for periodic submission. For Alabama, the AHC survey policy specifies which materials must be curated (Administrative Code of Alabama, Chapter 460-X-9). Archaeological documentation must be curated even if no artifacts are recovered. Renewal of this agreement is contingent upon compliance.

We appreciate having the opportunity to assist you with curation services and look forward to working with you whenever we can be of service.

Sincerely,

Eugene Futato

Eugene M. Futato RPA
Deputy Director

13075 Moundville Archaeological Park | Moundville, AL 35474 | 205-371-2266 | Fax 205-371-2494

APPENDIX B

ALABAMA STATE SITE FORM

1Mb580

ALABAMA STATE SITE FILE

SITE NUMBER: ____________________

SITE IDENTIFICATION **TEMPORARY SITE NUMBER:** ____________________

SITE NAME: __________Deer River Industrial Canal-South______________________________

SITE FLAG: __________ (Office Use Only)

SITE LOCATION AND SIZE

7.5' QUADRANGLE MAP: ________________Hollinger's Island__________________

UTM COORDINATES (Center of Site): ZONE: __16_ EASTING: __395947.20_____ NORTHING: ______3377050.85_

TOWNSHIP: _6 South___ RANGE: __1 West___ SECTION: _17_____ ______¼ of ______¼ of ______¼

ELEVATION: __1___ ft AMSL SITE SIZE: MAJOR AXIS: _325____ m MINOR AXIS: _20___ m

MAXIMUM DEPTH: __25____cm

PRESERVATION INFORMATION

PRESERVATION STATE: __03, 08, 10____

01-Unmodified
02-Erosion
03-Severe Erosion
04-Inundated
05-Intermittent Flooding
06-Cultivation
07-Construction
08-Logged, Clear Cut
09-Borrow Pit/Surface Mine
10-Deposition (buried)
11-Potholed
00-Other (specify)____________________

IMMEDIATE DESTRUCTION PENDING (y/n): __y_____ LOOTING/VANDALISM (y/n): __n____

PERCENT DESTROYED: __75____%

NATIONAL REGISTER STATUS: __01_

01-Undetermined
02-Considered Eligible
03-Considered Ineligible
04-Registered
05-Ineligible

DESCRIPTION/COMMENTS/REFERENCES

This is a large rangia clam and historic trash midden located south of the Threemile Industrial Canal on Mobile Bay. Collected artifacts include Late Archaic projectile points and other tool making elements of the Cypress Point Complex. As well as mid to late 18th century bottle and ceramic fragments, 19th century glass bricks and ceramics, as well as 20th century trash in the same vein. The site is much reduced from erosion into the bay and the creation of the Industrial Canal, but pockets of midden remain, and the artifact scatter overall is very extensive, numbering in the Hundreds of artifacts, only a small sample of which was collected.

ARCHAEOLOGICAL INFORMATION

LEVEL OF INVESTIGATION: _03__

01-Volunteered Report	02-Reconnaissance Survey	03-Intensive (100%) Survey

EXCAVATION STATUS: _04_

01-No Collection	03-Shovel Tests	05-Limited Testing	07-Excavation
02-Surface Collection	04-Surface Collection & Shovel Tests	06-Extensive Testing	08-Total Excavation

TOPOGRAPHIC ASSOCIATION: _07___

01-Upland Crest	03- Upland Base	05-Terrace	07-Tidal Marsh
02-Upland Slope	04-Floodplain	06-Island	

PHYSIOGRAPHIC DISTRICT: _32___

01-Northern Piedmont Upland	09-Blount Mountain	17-Warrior Basin	25-Eastern Red Hills
02-Southern Piedmont Upland	10-Jackson Co. Mountains	18-Wills Valley	26-Fall Line Hills
03-Big Canoe Valley	11-Little Mountain	19-Tennessee Valley	27-Flatwoods
04-Cahaba Ridges	12-Lookout Mountain	20-Outer Nashville Basin	28-Lime Hills
05-Cahaba Valley	13-Moulton Valley	21-Black Prairie	29-Southern Pine Hills
06-Coosa Ridges	14-Murphrees Valley	22-Buhrstone Hills	30-Western Red Hills
07-Coosa Valley	15-Sand Mountain	23-Chunnennuggee Hills	31-Coastal Strip
08-Weisner Ridges	16-Sequatchie Valley	24-Dougherty Plain	32-Mobile Delta

NEAREST WATER SOURCE: _10____ AT CONFLUENCE (y/n): ___y___

01-Sink	05-Lake	09-Third Order Stream	13-Estuary
02-Well	06-Oxbow Lake	10-Fourth Order Stream	14-Ocean/Bay
03-Spring	07-First Order Stream	11-Major Tributary	
04-Swamp	08-Second Order Stream	12-River	

DIRECTION TO WATER: _E__

DISTANCE TO WATER: _1___m

DRAINAGE BASIN: _20___

01-Alabama	07-Choctawhatchee	13-Pea	19-Yellow
02-Apalachicola	08-Conecuh	14-Perdido	20-Coastal Estuary/Bay
03-Black Warrior	09-Coosa	15-Sipsey	
04-Buttahatchee	10-Escambia	16-Tallapoosa	
05-Cahaba	11-Escatawpa	17-Tennessee	
06-Chattahoochee	12-Mobile-Tensaw	18-Tombigbee	

GROUND COVER: _06, 10____

01-Grassland	04-Unimproved Forest	07-Inundated	09-Roadway
02-Cultivation	05-Improved Forest/Orchard	08-Developed (Urban Residential/Industrial)	10-Open and Eroded
03-Secondary Growth	06-Intermittent Flooding		99-Other (specify)________

SOIL TEXTURE CLASS: __09___

01-Coarse Sand	09-Coarse Sandy Loam	17-Clay Loam
02-Sand	10-Sandy Loam	18-Silty Clay Loam
03-Fine Sand	11-Fine Sandy Loam	19-Sandy Clay
04-Very Fine Sand	12-Very Fine Sandy Loam	20-Silty Clay
05-Loamy Coarse Sand	13-Loam	21-Clay
06-Loamy Sand	14-Silt Loam	22-Rockland
07-Loamy Fine Sand	15-Silt	
08-Loamy Very Fine Sand	16-Sandy Clay Loam	

SOIL TYPE: _____Lafitte Muck________________________________

NATURE OF DEPOSIT: __02__

01-Entire Site Disturbed 02-Upper Portion Disturbed 03-Deep Disturbance 04-Undisturbed

SITE CHARACTERISTICS:

____	HUMAN REMAINS	____	WEIR
____	FEATURES	____	HISTORIC STRUCTURE (STANDING)
____	ROCKSHELTER	X	HISTORIC STRUCTURE SITE (NOT STANDING)
____	CAVE	____	HISTORIC CEMETERY
X	ARTIFACT SCATTER	____	QUARRY
____	MIDDEN	____	STILL
X	SHELL MIDDEN	____	MILL
____	SINGLE EARTHEN MOUND	____	ENGINEERING
____	MULTIPLE EARTHEN MOUNDS		(Specify): ____
____	PETROGLYPH/PICTOGRAPH	____	OTHER (Specify) ____
____	STONE MOUND(S)		

CULTURAL AFFILIATION(S):

--CULTURE--			--PHASES, CULTURES, HORIZONS, IF KNOWN--
____	PALEOINDIAN (Unidentified)		
____		EARLY	____
____		MIDDLE	____
____		LATE	____
____	ARCHAIC (Unidentified)		
____		EARLY	____
____		MIDDLE	____
X		LATE	Cypress Point Complex
____	GULF FORMATIONAL (Unidentified)		
____		EARLY	____
____		MIDDLE	____
____		LATE	____
____	WOODLAND (Unidentified)		
____		EARLY	____
____		MIDDLE	____
____		LATE	____
____	MISSISSIPPIAN (Unidentified)		
____		EARLY	____
____		MIDDLE	____
____		LATE	____
____	PROTOHISTORIC		____
____	HISTORIC ABORIGINAL		____
____	UNKNOWN ABORIGINAL		____
____	NON-ABORIGINAL		
____		16th CENTURY	____
____		17th CENTURY	____
X		18th CENTURY	____
X		19th CENTURY	____
X		20th CENTURY	____
		SPECIFIC DATE RANGE	1790s-1920s

MAP OF SITE

7.5' USGS TOPOGRAPHIC MAP: Hollinger's Island

SITE FORM AUTHOR IDENTIFICATION

DATE: 3-24-20

AUTHOR--NAME: Jason Gardner, Gulf South Past Recovery

ADDRESS:

jasongardner76@gmail.com

CITY: Mobile

STATE: AL ZIP:

APPENDIX F

SUBMERGED AQUATIC VEGETATION (SAV) SURVEY

Mobile Bay National Estuary Program Deer River Restoration Project
Submerged Aquatic Vegetation Survey
Barry A. Vittor & Associates, Inc.
April 27, 2020

Vittor & Associates inspected a 62.3-acre area of intertidal zone and shallow subtidal habitat at the Deer River Project site for submerged aquatic vegetation (SAV) on November 20, 2019, April 3, and April 21 2020. The April surveys were necessary due to typical seasonal patterns in presence/absence and density of different SAV species, which often die back in the late fall. Aerial imagery acquired in late July 2019 for coast-wide SAV mapping was used to interpret potential SAV occurrence, and served as a guide for on-site verification.

Figures 1 and 2 show the locations of SAV at the Project site. Field survey locations were logged in the field with GPS. Small, sparse SAV patches occurred in intertidal areas that were exposed at the time of the April survey, in addition to beds in shallow subtidal areas. SAV generally occurred at depths of < 2 ft. Larger areas containing multiple SAV patches were delineated as polygons in GIS, with most polygons classified as patchy SAV (< 50% cover). A 0.13-ac (5,663 ft^2) polygon at the mouth of the South Fork is classified as continuous (> 50% cover). Small individual patches are reported as point data.

The total acreage of SAV polygons is 0.85 ac (37,157 ft^2), with the largest proportion (0.61 ac [26,354 ft^2]) occurring at the confluence of the Deer River Middle Fork and Theodore Industrial Canal (Figure 1). Except for one field point inside the Deer River South Fork with Eurasian watermilfoil (*Myriophyllum spicatum*) (Figure 2), all SAV in the survey area is widgeon grass (*Ruppia maritima*). GIS shapefiles for GPS points, polygons, and survey area are provided in State Plane Alabama West, NAD1983 U.S. Survey (feet).


Barry A. Vittor & Associates, Inc.
0.61 AC
0.003 AC
0.04 AC
DEER RIVER
MOBILE BAY
SURVEY AREA
0.13 AC
Figure 1. Mobile Bay NEP Deer River Restoration Project - SAV Survey.
Submerged Aquatic Vegetation (SAV) April 23, 2020
GPS Field Point with SAV
GPS Field Point without SAV
0
100
200
400
Feet
Base Map: 2019 Aerial Imagery


SURVEY AREA
0.07 AC
Figure 1. Mobile Bay NEP Deer River Restoration Project - SAV Survey.
Submerged Aquatic Vegetation (SAV) April 23, 2020
GPS Field Point with SAV
0 100 200 400 Feet
Barry A. Vittor & Associates, Inc.
Base Map: 2019 Aerial Imagery