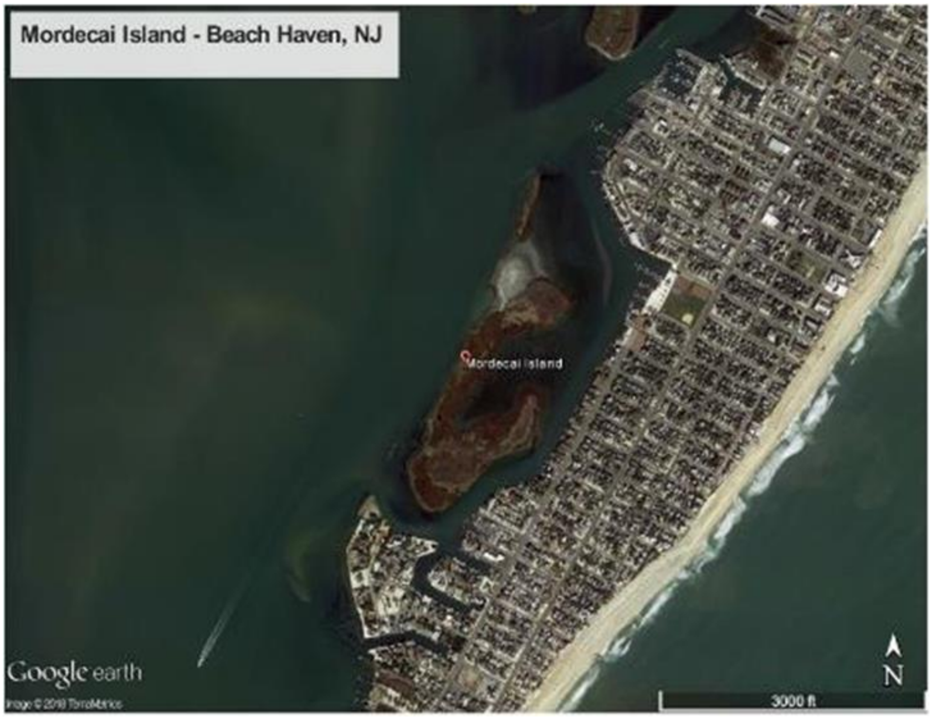
Mordecai Island
Beach Haven, New Jersey
Ecosystem Restoration

Feasibility Study and Integrated Environmental Assessment



APPENDIX A:
Engineering Technical Appendix
January 2022

U.S. Army Corps of Engineers
Philadelphia District

New Jersey Department of Environmental Protection

Table of Contents

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Section A1 – Geotechnical

Table of Contents

List of Figures

List of Tables

A1-1) INTRODUCTION

The objective of the feasibility study is to investigate and recommend implementable solutions to prevent future degradation of the area and to restore aquatic habitats for fish and wildlife at Mordecai Island. Several alternative plans were considered to lessen the erosion of Mordecai Island and increase habitat. Such alternatives consist of rock mounds, concrete Wave Attenuation Devices (WADs®), etc. The Tentative Selected Plan (TSP) to attenuate erosion of the island is to place a ruble mound breakwater structure to lessen wave impacts to the island shoreline. Placement of such structures in coastal/marsh areas may be subject to settlement which can impact performance. Consequently, a geotechnical evaluation was performed to estimate the settlement that may occur due to the additional loads imposed over the structure area. The project area is shown in Figure A1-1.

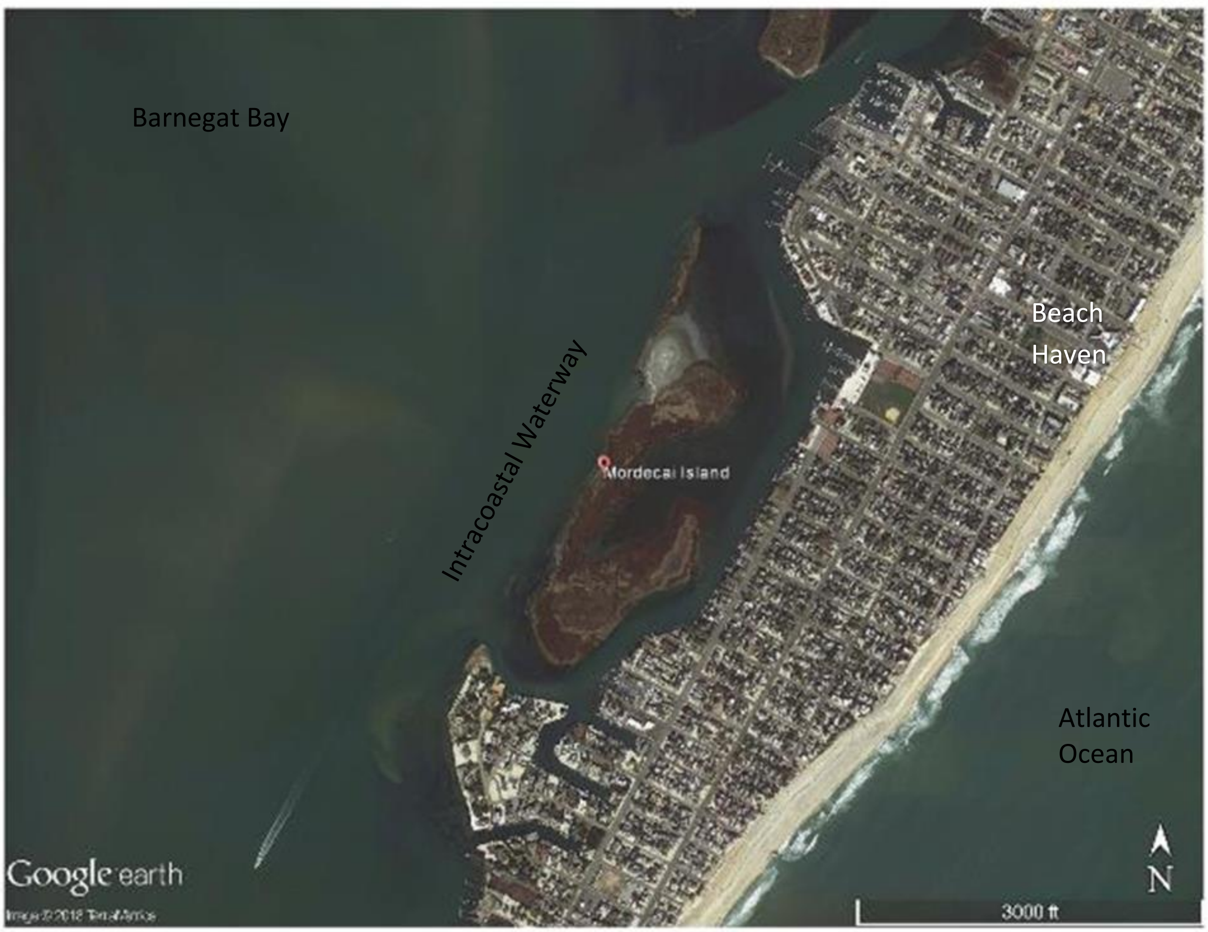


Figure A1-1: Mordecai Island and Surrounding Area

A1-2) REGIONAL GEOLOGY

Based on the Bedrock Geologic Map of New Jersey, dated 2014, the project site lies within the outer coastal plain Physiographic Province. The coastal plain Province is characterized by the Kirkwood formation, Belleplain Member (middle Miocene) which consist of gray to white, fine to medium grained, micaceous sand, wood and shell fragments. The lower part consists of gray brown, laminated silty clay, diatoms and shell fragments.

Surficial Deposits

Based on the soil survey of Ocean County (USDA-NRCS, 2015) shown in Figure A1-2, the soils within the approximate project limits consists of Appoquinimink-Transquaking-Mispillion complex (AptAv), Psammaquents, sulfidic substratum, (PstAt), Dredge Channel (WDC4A), Indian River sand flat (WIr1) and Indian River sand tidal inlet (WIr3).

The general characteristics of Appoquinimink-Transquaking-Mispillion complex (AptAv) include: Appoquinimink - mucky silt loam (0-12 in.) underlain by silt loam (12 to 30 in.) and mucky peat (30 to 80 in.) ; Transquaking - mucky peat (0 to 14 in.) underlain by muck (14 to 60 in.) and silty clay (60 to 90 in.); Mispillion - mucky peat (0 to 10 in.) underlain by muck (10 to 26 in.) and silt loam (26-90 in.). The soils develop along tidal marshes with 0 to 1 percent slopes are derived from loamy fluviomarine deposits over herbaceous organic material.

The general characteristics of Psammaquents, sulfidic substratum, (PstAt) include coarse sand (0 to 12 in.) underlain by gravelly sand (12 to 36 in.) and mucky peat (36 to 80 in.). The soils develop along flats with 0 to 2 percent slopes are derived from sandy lateral spread deposits over organic material

The Dredge Channel (WDC4) encompasses approximately 3.3 to 13.1 ft. of water depth. The Indian River sand flat (Wlr1) includes sand from approximately 0 to 79 ft. The soils along the Indian River deposit develop along flood-tidal delta flats with 0 to 3.3 ft. of water depth. The Indian River deposit (Wlr3) includes sand from approximately 0 to 70 ft. The soil develops along sandy flood-tidal delta lagoon deposits with 6.5 to 16.4 ft. of water depth.

The Appoquinimink-Transquaking-Mispillion complex soils are very poorly drained, very frequently flooded and strong saline content. The Psammaquents, sulfidic substratum soils are very poorly drained, frequently flooded and very slightly saline to strong saline content. The Indian River sand is frequently flooded, very frequently flooded and strong saline content. The Indian River sand flat soil is very frequently flooded, contains 5 percent calcium carbonate and strong saline.

Figure A1-2: Mordecai Island Soils Map

A1-3) SUBSURFACE EXPLORATION

A site-specific geotechnical investigation and laboratory testing program were performed for USACE Philadelphia District by Schnabel Engineering, Inc. The work was performed under Contract W912BU-05-D-0001, Task Order 007 and included the advancement of 3 geotechnical borings drilled from the water adjacent to Mordecai Island. Drilling was conducted from 24 February 2005 to 18 March 2005. Soils were sampled continuously in the borings using the Standard Penetration Methods (SPT) and Procedures. All SPT borings were drilled using a Diedrich D-25 drill rig equipped with an automatic hammer. As per the SPT Methods and Procedures, a 140 lb. hammer with a 30-inch drop was used to advance a 2-inch diameter split spoon sampler for drilling. When cohesive soils were present, a 3-inch diameter undisturbed piston Shelby tube was used to collect undisturbed samples. All soil sampling was continuous to the termination depth of the borings, 50 ft. Figure A1-3 shows the 3 soil borings that were performed in the project area.

Figure A1-3: Map of Soil Borings Performed

A1-4) SUBSURFACE CONDITIONS

The geotechnical borings encountered soils consisting of very loose to medium dense Silty Sand (SM) and coarse-grained soil to depths of 12 to 14 ft. below the mud line (el. -17.2 to -19.1). The upper coarse-grained soil is underlain by a fine grained/organic layer consisting of very soft Fat Clay (CH), Lean Clay (CL) and Silty Clay (CL-ML). A layer of Peat from 25.5 to 27.5 ft. below the mud line (el. -30.7 to -32.6) was encountered in Boring MIB-2. Below the fine grained strata, medium dense to very dense granular soil consisting of Silty Sand (SM) and Sand with Silt (SP-SM) was encountered and extended to the termination depths of 48.0 to 50 ft. below the mud line (el. -53.1 to 55.6). Refer to the boring logs included in Attachment A1-1 for detailed description of the encountered materials. A generalization of the subsurface profile is presented in Attachment A1-2, Subsurface Soil Profile.

Laboratory tests included moisture content, Atterberg limits, specific gravity, unconfined compressive strength, and consolidation testing. The results of the laboratory tests are presented in the Geotechnical Laboratory Testing, Mordecai Island, Beach Heaven New Jersey report, prepared by Schnabel Engineering, Inc. dated January 06, 2006, Attachment A1-4.

A1-5) SETTLEMENT ANALYSES

Settlement analyses were performed on the soils encountered in the borings performed in the nearby vicinity of the proposed hardened structures. Settlement was calculated based on the subsurface profile at each boring location and are generalized into the 4 strata as shown in table A1-1.

Table A1-1: Summary of Stratigraphic Unit Properties

Strata	Depth (ft.)	Average Uncorrected N-value(bpf)	Minimum Uncorrected N-Value (bpf)	Maximum Uncorrected N-Value (bpf)
Silty Sand	0-14	7	3	20
Fat Clay/Silty Clay	14-22	3	WOH*	5
Peat (only considered in MIB-2)	22-25.5	WOH*	WOH*	4
Silty Sand	25.5-50	28	4	50/3”

*WOH: Weight of Hammer

Soil Parameters

Geotechnical design parameters were based on the available existing field and laboratory test data obtained from the geotechnical investigation. The laboratory testing was performed by Schnabel Engineering, Inc. The initial void ratio, compression index, recompression index, and the pre-consolidation stress soil parameters shown in Table A1-2 were taken from the consolidation test report and were used to analyze the settlement.

Table A1-2: Settlement soil parameters from Schnabel Engineering, Inc. Test Results

Boring	Initial Void Ratio, e_0	Compression Index C_c	Recompression Index, C_r	Pre-Consolidation Stress, σ'_p (tsf)
MIB-1	2.07	0.661	0.0914	0.87
MIB-2	1.24	0.330	0.0266	0.75
MIB-3	1.21	0.351	0.0351	1.20

Structure Settlement

To lessen wave impacts and erosion, the use of a rubble mound structure was selected over the concrete WADs®. Since the WADs® are no longer considered as an option, settlement calculations were only performed for the rubble mound breakwater structure.

The rubble mound breakwater was designed as a rock embankment with a maximum height of approximately 7.6 ft., crest width of 3 ft., and 2H:1V side slopes. The rubble mound (riprap aggregate) breakwater was analyzed as an embankment with an assumed unit weight of 105 pcf. The crest height is being established at elevation +3.6 ft. NAVD88, the Mean Low Water at elevation -1.07 and the ground surface at approximate elevation -4.0 ft. NAVD88. The embankment load was calculated using the simplified stress under an embankment formula by calculating the stress at the top, the middle, and the bottom of the embankment and taking the average to determine the stress increase (see Attachment A1-3).

For the sand soil layer, an immediate settlement analysis was performed under the embankment using the modified Hough method for estimating immediate settlement of an embankment. For the clay soils, settlements were estimated using consolidation theory. Primary settlement analyses were performed in each boring assuming a normally consolidated state. This is a conservative approach due to the relatively close approximations of the pre-consolidated stresses and the vertical effective stresses of the soils and it will provide the largest estimated settlement at each test boring.

Based on the settlement calculations and the data provided, the predicted settlement ranges from 5 to 11 inches and it would take approximately 3 years to reach this settlement. Secondary consolidation was also calculated; however, it is considered insignificant since the additional settlement will be approximately 1 inch over 20 years. Due to potential variations in the subsurface conditions and to minimize differential settlement of the rubble mound breakwater structure, it is recommended that the structure be placed on a geo-composite fabric.

A1-6) RISK

A potential geotechnical risk associated with the planned structure is an inaccurate estimate of the settlement of the structure. This can result from inaccuracy of the laboratory testing results and potential for variations in the subsurface conditions which may not be identified based on the available data.

A1-7) REFERENCES

1. Das, Braja M. 1984, Principles of Foundation Engineering
2. USDA-NRCS, 2019, NRCS Web Soil Survey: Custom Soil Resource Report for Ocean County, New Jersey, United States Department of Agriculture and Natural Resources Conservation Service, Generated May 21, 2019
3. NAVFAC DM 7.1, May 1982, Soil Mechanics

A1-8) ATTACHMENTS:

Attachment A1-1: Boring Logs

Attachment A1-2: Subsurface Soil Profile

Attachment A1-3: Settlement Calculations

Attachment A1-4: Geotechnical Laboratory Testing, Mordecai Island, Beach Heaven New Jersey, dated January 06, 2006, Schnabel Engineering, Inc.

ATTACHMENT A1-1: BORING LOGS

DRILLING LOG — Hole No. MIB-1 (Sheet 1 of 3 sheets)

DIVISION	North Atlantic Division
INSTALLATION	Philadelphia District
1. PROJECT	Mordecai Island
2. LOCATION (Coordinates or Station)	561546.75 E, 264291.72 N (NAD 83)
3. DRILLING AGENCY	ASI – Aqua Surveys, Inc.
4. HOLE NO. (As shown on drawing title and file number)	MIB-1
5. NAME OF DRILLER	Fred Kuchner
6. DIRECTION OF HOLE	☒ VERTICAL ☐ INCLINED --- DEG. FROM VERT.
7. THICKNESS OF OVERBURDEN	
8. DEPTH DRILLED INTO ROCK	
9. TOTAL DEPTH OF HOLE	50.0 ft.
10. SIZE AND TYPE OF BIT	5" Casing/ 2" Split Spoon Sampler
11. DATUM FOR ELEVATION SHOWN (TBM or MSL)	MLW
12. MANUFACTURER'S DESIGNATION OF DRILL	Diedrich D-25
13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN	DISTURBED: UNDISTURBED:
14. TOTAL NUMBER CORE BOXES	
15. ELEVATION GROUND WATER	
16. DATE HOLE	STARTED 16 March 2005; COMPLETED 18 March 2005
17. ELEVATION TOP OF HOLE	-5.2 MLW
18. TOTAL CORE RECOVERY FOR BORING	ft
19.	

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g	BLOW COUNT PER 0.5' h
	0.0–2.0		Dark gray fine sand, some to and silt (loose/ wet) with shell fragments	1.33	S-1		1 3 1 2
	2.0–4.0			1.17	S-2		5 4 1 1
	4.0–6.0			0.75	S-3		2 2/ 12" 1
	6.0–7.0		Gray clayey silt, some fine sand (soft/ wet)	1.75	S-4		2 3 1 1
	7.0–8.0		Gray fine sand, some/and clayey silt, with shell fragments (very loose/ wet)				
	8.0–10.0		grading with fine-medium sand and shell fragments at 9.5'	1.58	S-5		2 6 2 1
	10.0–12.0			0.92	S-6		3 3 1/ 12"
	12.0–14.0		Gray clayey silt, some fine sand (very soft/ wet)	0.75	S-7		WOH WOH WOH WOH

ENG FORM MAR 71 1836 PREVIOUS EDITIONS ARE OBSOLETE. PROJECT: Mordecai Island HOLE NO. MIB-1

DRILLING LOG (Cont Sheet) — Hole No. MIB-1 (Sheet 2 of 3 sheets)

ELEVATION TOP OF HOLE: -5.2 MLW
PROJECT: Mordecai Island INSTALLATION: Philadelphia District

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS g	BLOW COUNT PER 0.5' h
	14.0–16.0		Tube T-1	1.83	T-1	Push tube 14'-16'	
	16.0–18.0			2	S-8		WOH WOH WOH WOH
	18.0–20.0		grading with little peat fibers at 19.5'	2	S-9		WOH WOH WOH WOH
	20.0–22.0		Tube T-2	2	T-2	Push tube 20'-22'	
	22.0–24.0		grading with little to some peat chunks at 22.5'	1.25	S-10		2 1 1 2
	24.0–25.5			2	S-11	End of day at 26'; Drive casing for 26'-28' spoon - did not wash out	1 1 1 4
	25.5–28.0		Gray-brown fine sand and silt, trace fine-coarse gravel (very loose)(wet) grading light brown, light gray, and olive fine sand, little-some silt (very dense/ wet) at 26'	1.67	S-12	Restart 3/17/2005 at 26'	14 41 27 21
	28.0–30.0		grading (dense) at 28'	1.42	S-13		20 25 14 23
	30.0–32.0		grading (very dense) at 30'	1.17	S-14		17 33 21 20

ENG FORM JUN 67 1836-A PROJECT: Mordecai Island HOLE NO. MIB-1

DRILLING LOG (Cont Sheet) — Hole No. MIB-1 (Sheet 3 of 3 sheets)

ELEVATION TOP OF HOLE: -5.2 MLW
PROJECT: Mordecai Island INSTALLATION: Philadelphia District

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS g	BLOW COUNT PER 0.5' h
	32.0–34.0		grading (dense) at 32'	1.33	S-15		17 26 19 18
	34.0–36.0		grading with trace fine-coarse gravel (medium dense) at 34'	1.42	S-16		6 11 16 18
	36.0–38.0		grading light gray/ white/ light brown in color	1.17	S-17		11 21 12 18
	38.0–40.0		grading with fine-coarse sand lenses (±2") at 39'	1.42	S-18		12 18 15 19
	40.0–42.0		grading (medium dense) at 40' grading with fine-medium sand lenses (±3") at 40.5'	1.33	S-19		14 14 9 7
	42.0–44.0		grading fine-medium sand at 42.5'	1.5	S-20		12 12 10 11
	44.0–46.0				S-21	End of day at 44'; Drive casing for 44'-46' spoon - did not wash out. Restart 3/18/2005 at 44'	5 11 10 9
	46.0–48.0		Light gray/white/light brown well graded fine-medium sand, trace silt, trace fine-coarse gravel (medium dense) at 46'		S-22		5 8 12 10
	48.0–50.0				S-23	End of Boring at 50' on 3/18/2005	7 5 10 9

ENG FORM JUN 67 1836-A PROJECT: Mordecai Island HOLE NO. MIB-1

DRILLING LOG — Hole No. MIB-2 (Sheet 1 of 3 sheets)

DIVISION	North Atlantic Division
INSTALLATION	Philadelphia District
1. PROJECT	Mordecai Island
2. LOCATION (Coordinates or Station)	561906.01 E, 265021.92 N (NAD 83)
3. DRILLING AGENCY	ASI – Aqua Surveys, Inc.
4. HOLE NO. (As shown on drawing title and file number)	MIB-2
5. NAME OF DRILLER	Fred Kuchner
6. DIRECTION OF HOLE	☒ VERTICAL ☐ INCLINED --- DEG. FROM VERT.
7. THICKNESS OF OVERBURDEN	
8. DEPTH DRILLED INTO ROCK	
9. TOTAL DEPTH OF HOLE	50.0 ft.
10. SIZE AND TYPE OF BIT	5" Casing/ 2" Split Spoon Sampler
11. DATUM FOR ELEVATION SHOWN (TBM or MSL)	MLW
12. MANUFACTURER'S DESIGNATION OF DRILL	Diedrich D-25
13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN	DISTURBED: UNDISTURBED:
14. TOTAL NUMBER CORE BOXES	
15. ELEVATION GROUND WATER	
16. DATE HOLE	STARTED 10 March 2005; COMPLETED 14 March 2005
17. ELEVATION TOP OF HOLE	-5.6' MLW
18. TOTAL CORE RECOVERY FOR BORING	ft
19.	

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS g	BLOW COUNT PER 0.5' h
	0.0–0.3		4" Dark gray silt, some fine sand (very soft/ wet)	1.33	S-1		WOH 3 4 3
	0.3–2.0		Gray fine sand, little-some silt (loose/ wet)				
	2.0–4.0			1.75	S-2		3 4 1 1
	4.0–6.0			1.5	S-3		5 4 1 2
	6.0–7.0			1.92	S-4		2 2 2 1
	7.0–8.0		grading with fine shell fragments at 7' (slight organic odor)				
	8.0–10.0			1.67	S-5		3 4 2 2
	10.0–12.0		grading with larger shell fragments at 10'	1.83	S-6		WOR 3 2 1
	12.0–14.0			0.92	S-7		WOH 2 1 /12"

ENG FORM MAR 71 1836 PREVIOUS EDITIONS ARE OBSOLETE. PROJECT: Mordecai Island HOLE NO. MIB-2

DRILLING LOG (Cont Sheet) — Hole No. MIB-2 (Sheet 2 of 3 sheets)

ELEVATION TOP OF HOLE: -5.6' MLW
PROJECT: Mordecai Island INSTALLATION: Philadelphia District

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS g	BLOW COUNT PER 0.5' h
	14.0–16.0		Gray silty clay, little fine sand (very soft/ wet)	2	S-8		WOR 2 1 1
	16.0–18.0		Tube T-1	2	T-1	Push tube 16'-18'	
	18.0–20.0		grading with little peat fibers at 18'	2	S-9		WOH WOH 1/ 12"
	20.0–22.0		grading with some peat fibers	2	S-10	End of day at 22'. Push casing but do not clean out	1 1 2 3
	22.0–22.5		grading and peat at 22'	2	S-11	Restart at 22'	2 2 3 3
	22.5–24.0		Dark brown peat, some silt (soft/ very moist/ organic odor)				
	24.0–25.5		Tube T-2	1.5	T-2	Push tube 24'-25.5' 0-500-1500. Advance casing to 26'	
	25.5–26.0		Gray fine sand, little to some clayey silt				
	26.0–28.0		grading light brown/ gray fine-medium sand, trace to little silt at 26' (very dense/ wet)	1.42	S-12		10 37 37 24
	28.0–30.0		grading gray in color, little to some silt, with medium sand lenses (±1") occasionally (medium dense/ wet at 28')	1.5	S-13		8 14 16 17
	30.0–32.0			1.25	S-14		9 9 10 8

ENG FORM JUN 67 1836-A PROJECT: Mordecai Island HOLE NO. MIB-2

DRILLING LOG (Cont Sheet) — Hole No. MIB-2 (Sheet 3 of 3 sheets)

ELEVATION TOP OF HOLE: -5.6' MLW
PROJECT: Mordecai Island INSTALLATION: Philadelphia District

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS g	BLOW COUNT PER 0.5' h
	32.0–34.0		grading trace to little silt (loose) at 32'	0.5	S-15	Drove casing to next interval (34') but do not finish. End of day at 34' on 3/14/2005	5 4 3 4
	34.0–36.0		grading gray fine-medium sand, trace silt (medium dense/ wet)	1.17	S-16		8 5 5 6
	36.0–38.0		grading very fine sand, some silt at 36.5' (very loose/ wet)	1	S-17		4 2/ 12" 2
	38.0–40.0		grading gray fine-medium sand, little silt, trace fine gravel (medium dense/ wet) at 38'	0.67	S-18		8 11 11 15
	40.0–42.0			0.92	S-19		5 8 7 9
	42.0–44.0		Gray fine sand, trace silt with 3"-5" fine-coarse sand lenses (medium dense/ wet)	1.75	S-20		4 7 6 7
	44.0–46.0		grading gray fine-medium sand, trace silt with 2"-3" fine sand lenses (medium dense/ wet)	1.42	S-21		2 10 7 14
	46.0–48.0		grading fine-coarse sand, trace silt (medium dense/ wet) at 46'. grading fine sand, trace silt (very dense/ wet) at 46.5'	1.5	S-22		14 29 24 30
	48.0–50.0		grading (medium dense) at 48'. Tip of spoon ±1" of gray clayey silt, little-some fine sand	2	S-23	End of Boring at 50' on 3/14/2005	3 6 12 13

ENG FORM JUN 67 1836-A PROJECT: Mordecai Island HOLE NO. MIB-2

US Army Corps of Engineers
Philadelphia District

Mark	Description	Date	Appr.	Mark	Description	Date	Appr.

U.S. ARMY ENGINEER DISTRICT, PHILADELPHIA
CORPS OF ENGINEERS
PHILADELPHIA, PENNSYLVANIA

Designed by: TJK		Date: XXXXXXXXX	Rev.
Dwn by: JRI	Ckd by: TJK	IFB XXXXXXXXX	
Reviewed by: DJK		Drawing code:	
Submitted by: JRA	Chief, E & C Division	File name: ; Plot Scale: 1:1; Dwg. scale: N.T.S.	

MORDECAI ISLAND
COASTAL WETLANDS RESTORATION
BEACH HAVEN, NEW JERSEY

BORING LOGS MIB 1 AND 2

Sheet drawing number: C-4

Hole No. MIB-3

DRILLING LOG	DIVISION North Atlantic Division	INSTALLATION Philadelphia District	SHEET 1 OF 3 SHEETS
1. PROJECT Mordecai Island		10. SIZE AND TYPE OF BIT 5" Casing/ 2" Split Spoon Sampler	
2. LOCATION (Coordinates or Station) 562349.01 E, 265835.18 N (NAD 83)		11. DATUM FOR ELEVATION SHOWN (TBM or MSL) MLW	
3. DRILLING AGENCY ASI – Aqua Surveys, Inc.		12. MANUFACTURER'S DESIGNATION OF DRILL Diedrich D-25	
4. HOLE NO. (As shown on drawing title and file number) MIB-3		13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN	DISTURBED / UNDISTURBED
5. NAME OF DRILLER Fred Kuchner		14. TOTAL NUMBER CORE BOXES	
6. DIRECTION OF HOLE ☒ VERTICAL ☐ INCLINED --- DEG. FROM VERT.		15. ELEVATION GROUND WATER	
7. THICKNESS OF OVERBURDEN		16. DATE HOLE STARTED 24 February 2005 COMPLETED 05 March 2005	
8. DEPTH DRILLED INTO ROCK		17. ELEVATION TOP OF HOLE -5.1' MLW	
9. TOTAL DEPTH OF HOLE 46.0 ft.		18. TOTAL CORE RECOVERY FOR BORING ft	
		19.	

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g	BLOW COUNT PER 0.5' h
	2.0		Dark gray clayey silt, little fine sand (very soft/ wet)	0.75	S-1		WOR WOH WOH 3
	4.0		Gray fine sand, little to some silt (loose/ wet)	1.17	S-2		9 6 3 3
	6.0		grading with little peat fibers at 4'	1.42	S-3		5 4 2 1
	8.0			1.17	S-4		5 3 2 4
	10.0		grading with shell fragments at 8' (medium dense/ wet)	1.5	S-5		4 10 10 8
	12.0			0.92	S-6		8 5 7 5
	14.0			0.67	S-7		13 12 4 2

ENG FORM MAR 71 1836 PREVIOUS EDITIONS ARE OBSOLETE. PROJECT Mordecai Island HOLE NO. MIB-3

Hole No. MIB-3

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE -5.1' MLW		
PROJECT Mordecai Island		INSTALLATION Philadelphia District	SHEET 2 OF 3 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g	BLOW COUNT PER 0.5' h
	16.0		Gray clayey silt, some fine sand (very soft/ wet)	2	S-8		WOH WOH WOH WOH
	18.0		grading with shell fragments at 17'	2	S-9		WOH WOH WOH 2
	20.0		T-1		T-1	Push tube 18'-20'	
	22.0		grading with little to some peat at 20'	2	S-10		WOH WOH WOH WOH
	24.0			2	S-11		WOH 1 1 2
	26.0			1.67	S-12		WOH 1 3 3
	27.5		grading gray silt, little-some fine sand at 26.5' (stiff)	2	S-13		3 6 9 19
	28.0		Gray fine sand, little silt (medium dense/ wet)				
	30.0		grading gray/ brown at 28' (very dense)	0.75	S-14		25 50/ 3"
	32.0			0.38	S-15		50/ 4.5"

ENG FORM JUN 67 1836-A PROJECT Mordecai Island HOLE NO. MIB-3

Hole No. MIB-3

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE -5.1' MLW		
PROJECT Mordecai Island		INSTALLATION Philadelphia District	SHEET 3 OF 3 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth weathering, etc., if significant) g	BLOW COUNT PER 0.5' h
	33.0		grading light brown to gray fine-medium sand, trace silt (dense/ wet) at 32'	1.5	S-16		10 22 25 32
	34.0		grading gray to light brown fine sand, trace silt (very dense) at 33'				
	36.0		grading with some silt (medium dense) at 34'	1.33	S-17		16 14 11 12
	38.0		grading (dense) at 36'	1.17	S-18		12 18 24 32
	40.0		grading with coarse-medium sand lenses at 39' (medium dense)	1.25	S-19		7 13 10 17
	42.0		grading light gray/ gray very fine sand, trace silt (medium dense/ wet) at 40'	1.67	S-20		9 8 7 8
	44.0			1.42	S-21		6 5 8 7
	46.0			1.5	S-22	End of Boring at 46' on 3/5/2005	6 8 7 11

ENG FORM JUN 67 1836-A PROJECT Mordecai Island HOLE NO. MIB-3

US Army Corps of Engineers Philadelphia District

Mark	Description	Date	Appr.	Mark	Description	Date	Appr.

U.S. ARMY ENGINEER DISTRICT, PHILADELPHIA
CORPS OF ENGINEERS
PHILADELPHIA, PENNSYLVANIA

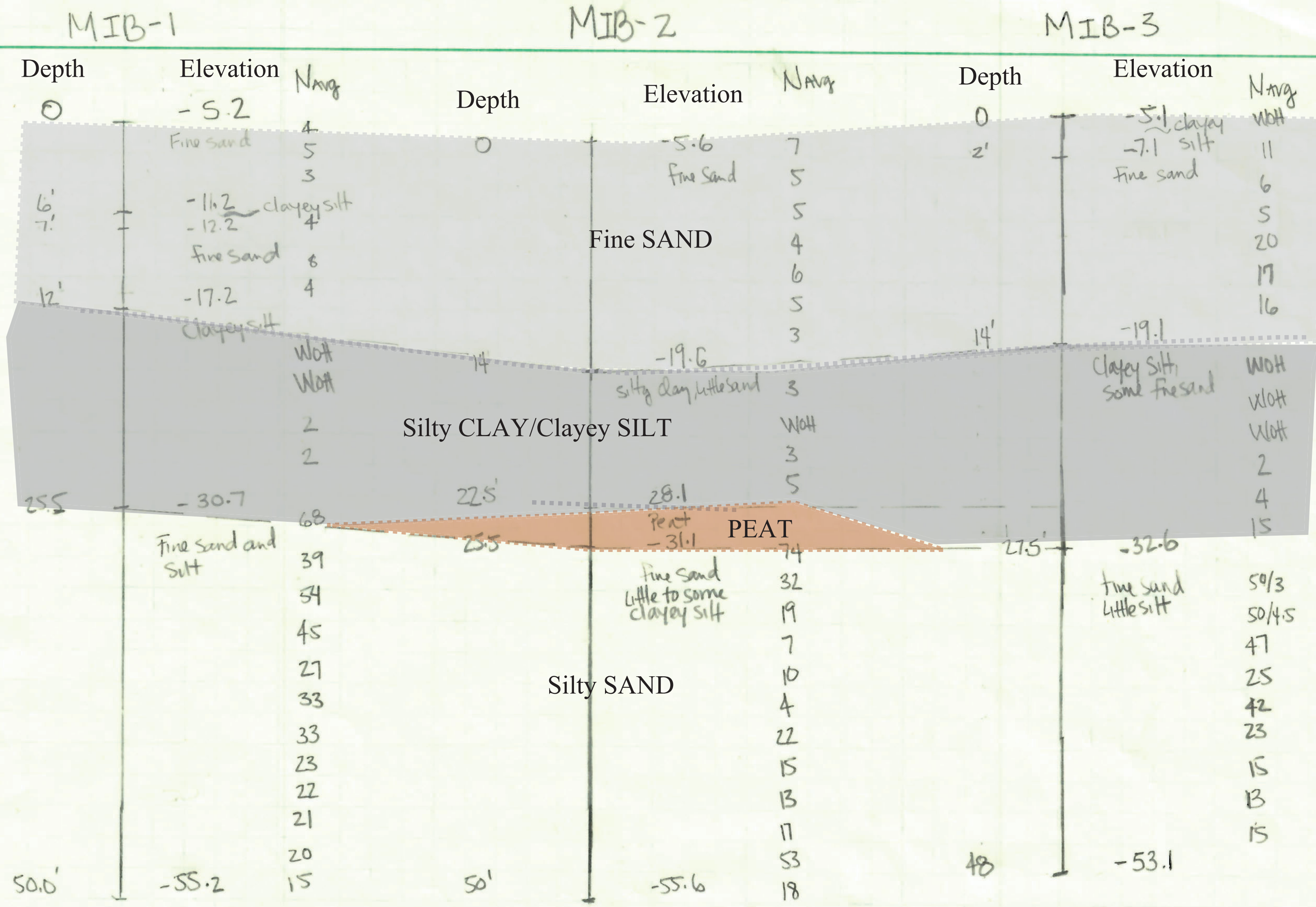
Designed by: TJE	Date: XXXXXXXXX	Rev.
Dwn by: JBJ	Ckd by: TJE	IFB XXXXXXXXX
Reviewed by: DJK	Drawing code:	
Submitted by: JRA	File name:	
Chief, E & C Division	Plot Scale: xx	Dwg. scale: N.T.S.

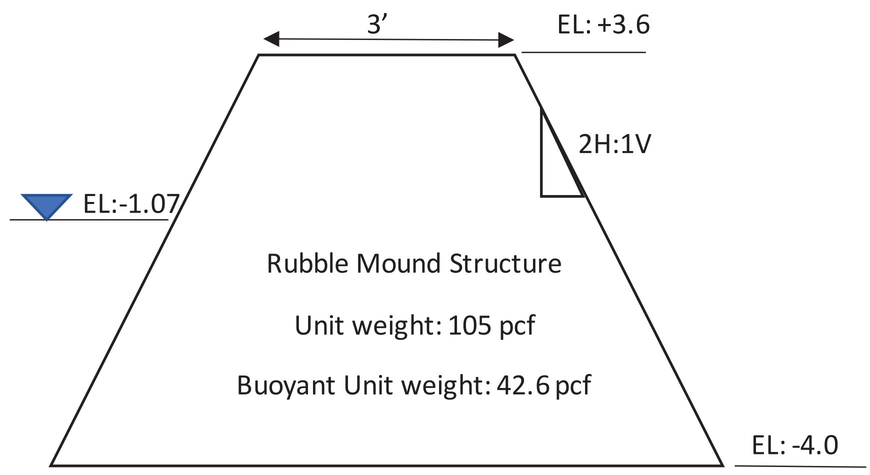
MORDECAI ISLAND
COASTAL WETLANDS RESTORATION
BEACH HAVEN, NEW JERSEY

BORING LOG MIB 3

Sheet drawing number: C-5

ATTACHMENT A1-2: SUBSURFACE SOIL PROFILE


MIB-1
MIB-2
MIB-3
Depth
Elevation
Navg
Fine SAND
Silty CLAY/Clayey SILT
PEAT
Silty SAND
-5.2
Fine sand
-11.2 clayey silt
-12.2
fine sand
-17.2
Clayey silt
-30.7
Fine sand and silt
-55.2
50.0'
-5.6
fine Sand
-19.6
silty clay little sand
28.1
Peat
-31.1
Fine Sand little to some clayey silt
-55.6
50'
-5.1 clayey silt
-7.1
Fine sand
-19.1
Clayey Silt, some fine sand
-32.6
fine sand little silt
-53.1
48
WoH
50/3
50/4.5



Rubble Mound Sketch * Not to scale*

ATTACHMENT A1-3: SETTLEMENT CALCULATIONS

Project: Mordecai Island Settlement Analysis
Objective: Evaluate immediate settlement on the thickest sand layer from borings MIB-1 through MIB-3. The immediate settlement will be computed using the Modified Hough Method.

$H_{sand} := 15.6\ ft$

$\gamma_{sand} := 105\ pcf$

$\gamma_w := 62.4\ pcf$

$N_{avg} := 5$

$\gamma_{rock} := 105\ pcf$

$E := 80$

$H_{emb} := 7.6\ ft$

$N_{1\_60} := N_{avg} \cdot \frac{E}{60} = 7$

$H_w := 2.93\ ft$

$C' := 43$

C': bearing capacity index NHI-06-088

Settlement Equation:

$\sigma'_o := \frac{H_{sand}}{2} \cdot \left(\gamma_{sand} - \gamma_w\right) = 332.3\ psf$ effective stress at the midpoint of the sand layer

$\Delta\sigma'_f := \gamma_{rock} \cdot \left(H_{emb} - H_w\right) + \left(\gamma_{rock} - \gamma_w\right) \cdot H_w = 615.2\ psf$

Settlement in the sand:

$$\Delta H := H_{sand} \cdot \left(\frac{1}{C'}\right) \cdot \log\left(\frac{\sigma'_o + \Delta\sigma'_f}{\sigma'_o}\right) = 2\ in$$

with depth, or the D'Appolonia (1968, 1970) method, which takes into account the effect of preconsolidation. Both methods provide equally suitable results. Schmertmann's modified method is presented in Chapter 8 (Shallow Foundations).

7.4.1 Modified Hough Method for Estimating Immediate Settlements of Embankments

The following steps are used in Modified Hough method to estimate immediate settlement:

Step 1. Determine the bearing capacity index (C′) by entering Figure 7-7 with $N1_{60}$ value and the visual description of the soil.

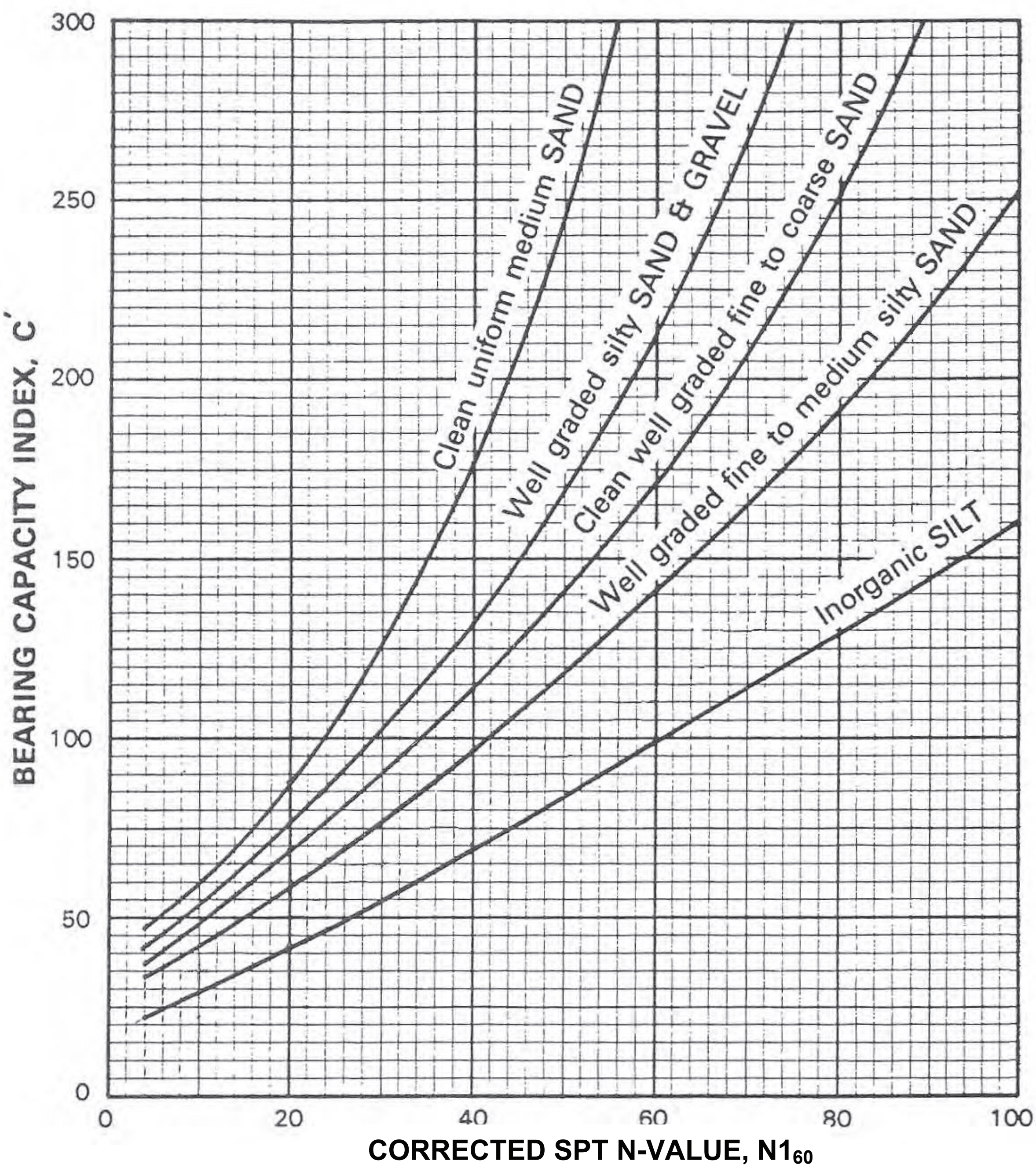
Step 2. Compute settlement by using the following equation. Subdivide the total thickness of the layer impacted by the applied loads into 10 ft ± (3 m ±) increments and sum the incremental solutions:

$$\Delta H = H\left(\frac{1}{C'}\right)\log_{10}\frac{p_o + \Delta p}{p_o} \qquad 7\text{-}1$$

where:
- ΔH = settlement of subdivided layer (ft)
- H = thickness of subdivided soil layer considered (ft)
- C' = bearing capacity index (Figure 7-7)
- p_o = existing effective overburden pressure (psf) at center of the subdivided layer being considered. For shallow surface deposits, a minimum value of 200 psf should be used to prevent unrealistic settlement predictions.
- Δp = distributed embankment pressure (psf) at center of the subdivided layer being considered

Note that the term $p_o + \Delta p$ represents the final pressure applied to the foundation subsoil, p_f.

A key point is that the logarithm term in Equation 7-1 incorporates the fundamental feature of dissipation of applied stress with depth. The use of Modified Hough method is illustrated numerically in Example 7-2.



(Note: The "Inorganic SILT" curve should generally not be applied to soils that exhibit plasticity because N-values in such soils are unreliable)

Figure 7-7. Bearing capacity index (C') values used in Modified Hough method for computing immediate settlements of embankments (AASHTO, 2004 with 2006 Interims; modified after Hough, 1959).

Project: Mordecai Island Settlement Analysis
Objective: Evaluate consolidation settlement on the clay layer in borings MIB-1 through MIB-3.
Calc. by: ME Check by:EF

Laboratory parameters obtained from Boring MIB-1

$\sigma'_p := 0.87\ \frac{ton}{ft^2} = 1740\ \frac{lb}{ft^2}$ (pre-consolidation stress)

$w := 0.391$ (initial water content)

$\gamma_{m1} := 55.1\ pcf \cdot (1+w) = 76.6\ pcf$ (moist unit weight)

$C_{c1} := 0.661$ (Compression Index)

$C_{r1} := 0.0914$ (Recompression Index)

$e_{o1} := 2.07$ (Initial void ratio)

Rubble Mound (rm) Parameters:

$\gamma_{rock} := 105\ pcf$ (Unit weight of rm)

$\gamma_w := 62.4\ pcf$ (Unit weight of water)

$\gamma_{sand} := \gamma_{rock}$

$EL_{top} := 3.6\ ft$ (top of rm embankment el.)

$EL_{bot} := -4.0\ ft$ (bot. of rm embankment el.)

$H_{rm} := EL_{top} - EL_{bot} = 7.6\ ft$ (height of rm embankment)

$EL_{topW} := -1.07\ ft$ (top of rm embankment water el.)

$H_w := EL_{topW} - EL_{bot} = 2.9\ ft$ (height of water in embankment)

Calculate the average effective stress of the rubble mound

$q_o := \gamma_{rock} \cdot (H_{rm} - H_w) + (\gamma_{rock} - \gamma_w) \cdot H_w = 615.2\ psf$

Calculate the stress increase under the Rubble Mound under MIB-1.

The stress increase is calculating the stress reduction of the compressible clay stratum using the simplified stress under an embankment formula. It is calculated by computing the stress at the tip, middle and bottom of the compressible clay layer and taking the average.

$\Delta\sigma'_f = \frac{1}{6}\left(\Delta\sigma_{top} + 4\ \Delta\sigma_{mid} + \Delta\sigma_{bottom}\right)$

$I = Influence\_factor\_\Delta\sigma_{top}$
Fig.5.11 :Das, Principles of Engineering

$I_{top} = Influence\_factor\_\Delta\sigma_{top}$

$B_2 := 15.6\ ft$

$B_1 := 1.5\ ft$ $\quad Z_{1t} := 13.2\ ft$

$\frac{B_1}{Z_{1t}} = 0.1$ $\quad \frac{B_2}{Z_{1t}} = 1.2$ $\quad I_{top1} := 0.44$

$I_{mid} = Influence\_factor\_\Delta\sigma_{mid}$

$B_2 := 15.6\ ft$ $\quad B_1 := 1.5\ ft$

$Z_{1m} := 13.2\ ft + \frac{13.5}{2}\ ft = 20\ ft$

$\frac{B_1}{Z_{1m}} = 0.1$ $\quad \frac{B_2}{Z_{1m}} = 0.8$

$I_{mid1} := 0.38$

$$I_{bottom} = Influence\_factor\_\Delta\sigma_{bottom}$$

$$B_2 := 15.6\ ft \quad B_1 := 1.5\ ft$$

$$Z_{1b} := 13.2\ ft + 13.5\ ft = 26.7\ ft$$

$$\frac{B_1}{Z_{1b}} = 0.1 \qquad \frac{B_2}{Z_{1b}} = 0.6 \qquad I_{bottom1} := 0.33$$

$$\Delta\sigma_{top} := q_o \cdot \left(I_{top1} \cdot 2\right) = 541.3\ psf$$

$$\Delta\sigma_{mid} := q_o \cdot \left(I_{mid1} \cdot 2\right) = 467.5\ psf$$

$$\Delta\sigma_{bottom} := q_o \cdot \left(I_{bottom1} \cdot 2\right) = 406\ psf$$

$$\Delta\sigma'_{f1} := \frac{1}{6}\left(\Delta\sigma_{top} + 4\ \Delta\sigma_{mid} + \Delta\sigma_{bottom}\right) = 469.6\ psf$$

Calculating the effective stress at the midpoint of the clay layer for MIB-1:

$$H_{sand} := 13.2\ ft \qquad H_{clay} := 13.5\ ft$$

$$\sigma'_o := \left(\gamma_{sand} - \gamma_w\right) \cdot H_{sand} + \left(\gamma_{m1} - \gamma_w\right) \cdot \frac{H_{clay}}{2} = 658.5\ psf$$

The settlement is calculated as normally consolidated clay

$$S_{cMIB-1} := \frac{C_{c1} \cdot H_{clay}}{1 + e_{o1}} \cdot \log\left(\frac{\sigma'_o + \Delta\sigma'_{f1}}{\sigma'_o}\right) = 8.2\ in$$

Laboratory parameters obtained from Boring MIB-2

$\sigma'_c := 0.75\ \frac{ton}{ft^2} = 1500\ \frac{lb}{ft^2}$ pre-consolidation stress

$w := 0.44$

$C_{c2} := 0.330 \quad C_{r2} := 0.0266 \quad e_{o2} := 1.24 \quad \gamma_{m2} := 76.5\ pcf \cdot (1 + w) = 110.2\ pcf$

Calculate the stress increase under the Rubble Mound under MIB-2.

$\Delta\sigma'_f = \frac{1}{6}\left(\Delta\sigma_{top} + 4\ \Delta\sigma_{mid} + \Delta\sigma_{bottom}\right)$

$I = Influence\_factor\_\Delta\sigma_{top}$

Fig.5.11 :Das, Principles of Engineering

$I_{top} = Influence\_factor\_\Delta\sigma_{top}$

$B_{2t} := 15.6\ ft \quad B_1 := 1.5\ ft \qquad Z_{1t} := 15.6\ ft$

$\frac{B_1}{Z_{1t}} = 0.1 \qquad \frac{B_{2t}}{Z_{1t}} = 1 \qquad I_{top2} := 0.42$

$I_{mid} = Influence\_factor\_\Delta\sigma_{mid}$

$B_{2m} := 15.6\ ft \quad B_1 := 1.5\ ft$

$Z_{1m} := 15.6\ ft + \frac{8.5}{2}\ ft = 19.85\ ft$

$\frac{B_1}{Z_{1m}} = 0.1 \qquad \frac{B_{2m}}{Z_{1m}} = 0.79 \qquad I_{mid2} := 0.38$

$I_{bottom} = Influence\_factor\_\Delta\sigma_{bottom}$

$B_{2b} := 15.6\ ft \quad B_1 := 1.5\ ft$

$Z_{1b} := 15.6\ ft + 8.5\ ft = 24.1\ ft$

$\frac{B_1}{Z_{1b}} = 0.1 \qquad \frac{B_{2b}}{Z_{1b}} = 0.6 \qquad I_{bottom2} := 0.33$

$\Delta\sigma_{top2} := q_o \cdot \left(I_{top2} \cdot 2\right) = 516.7\ psf$

$\Delta\sigma_{mid2} := q_o \cdot \left(I_{mid2} \cdot 2\right) = 467.5\ psf$

$\Delta\sigma_{bottom2} := q_o \cdot \left(I_{bottom2} \cdot 2\right) = 406\ psf$

$\Delta\sigma'_{f2} := \frac{1}{6}\left(\Delta\sigma_{top2} + 4\ \Delta\sigma_{mid2} + \Delta\sigma_{bottom2}\right) = 465.5\ psf$

Calculating the effective stress at the midpoint of the clay layer for MIB-2:

$H_{sand2} := 15.6\ ft \qquad H_{clay2} := 8.5\ ft$

$\sigma'_o := \left(\gamma_{sand} - \gamma_w\right) \cdot H_{sand2} + \left(\gamma_{m2} - \gamma_w\right) \cdot \frac{H_{clay2}}{2} = 867.5\ psf$

The settlement is calculated as normally consolidated clay at MIB -2

$$S_{cMIB-2} := \frac{C_{c2} \cdot H_{clay2}}{1 + e_{o2}} \cdot \log\left(\frac{\sigma'_o + \Delta\sigma'_{f2}}{\sigma'_o}\right) = 2.8\ in$$

Laboratory parameters obtained from Boring MIB-3

$\sigma'_c := 1.2\ \frac{ton}{ft^2} = 2400\ \frac{lb}{ft^2}$ pre-consolidation stress

$w := 0.44$

$\gamma_{m3} := 76.5\ pcf \cdot (1 + w) = 110.2\ pcf$

$C_{c3} := 0.351 \quad C_{r3} := 0.0365 \quad e_{o3} := 1.21$

Calculate the stress increase under the Rubble Mound under MIB-3.

$\Delta\sigma'_f = \frac{1}{6}\left(\Delta\sigma_{top} + 4\ \Delta\sigma_{mid} + \Delta\sigma_{bottom}\right)$

$I = Influence\_factor\_\Delta\sigma_{top}$

Fig.5.11 :Das, Principles of Engineering

$I_{top} = Influence\_factor\_\Delta\sigma_{top}$

$B_{2t} := 15.6\ ft \quad B_1 := 1.5\ ft \quad Z_{1t} := 15.1\ ft$

$\frac{B_1}{Z_{1t}} = 0.1 \quad \frac{B_{2t}}{Z_{1t}} = 1 \quad I_{top3} := 0.42$

$I_{mid} = Influence\_factor\_\Delta\sigma_{mid}$

$B_{2m} := 15.6\ ft \quad B_1 := 1.5\ ft$

$Z_{1m} := 15.6\ ft + \frac{13.5}{2}\ ft = 22.35\ ft$

$\frac{B_1}{Z_{1m}} = 0.1 \quad \frac{B_{2m}}{Z_{1m}} = 0.7 \quad I_{mid3} := 0.358$

$I_{bottom} = Influence\_factor\_\Delta\sigma_{bottom}$

$B_{2b} := 15.6\ ft \quad B_1 := 1.5\ ft$

$Z_{1b} := 15.6\ ft + 13.5\ ft = 29.1\ ft$

$\frac{B_1}{Z_{1b}} = 0.1 \quad \frac{B_{2b}}{Z_{1b}} = 0.5 \quad I_{bottom3} := 0.295$

$\Delta\sigma_{top3} := q_o \cdot (I_{top3} \cdot 2) = 516.7\ psf$

$\Delta\sigma_{mid3} := q_o \cdot (I_{mid3} \cdot 2) = 440.5\ psf$

$\Delta\sigma_{bottom3} := q_o \cdot (I_{bottom3} \cdot 2) = 362.9\ psf$

$\Delta\sigma'_{f3} := \frac{1}{6}\left(\Delta\sigma_{top3} + 4\ \Delta\sigma_{mid3} + \Delta\sigma_{bottom3}\right) = 440.3\ psf$

Calculating the effective stress at the midpoint of the clay layer for MIB-3:

$H_{sand3} := 15.1\ ft$ $\qquad H_{clay3} := 13.5\ ft$

$$\sigma'_o := (\gamma_{sand} - \gamma_w) \cdot H_{sand3} + (\gamma_{m2} - \gamma_w) \cdot \frac{H_{clay3}}{2} = 965.6\ psf$$

The settlement is calculated as normally consolidated clay at MIB -3

$$S_{cMIB\_}3 := \frac{C_{c3} \cdot H_{clay3}}{1 + e_{o3}} \cdot \log\left(\frac{\sigma'_o + \Delta\sigma'_{f3}}{\sigma'_o}\right) = 4.2\ in$$

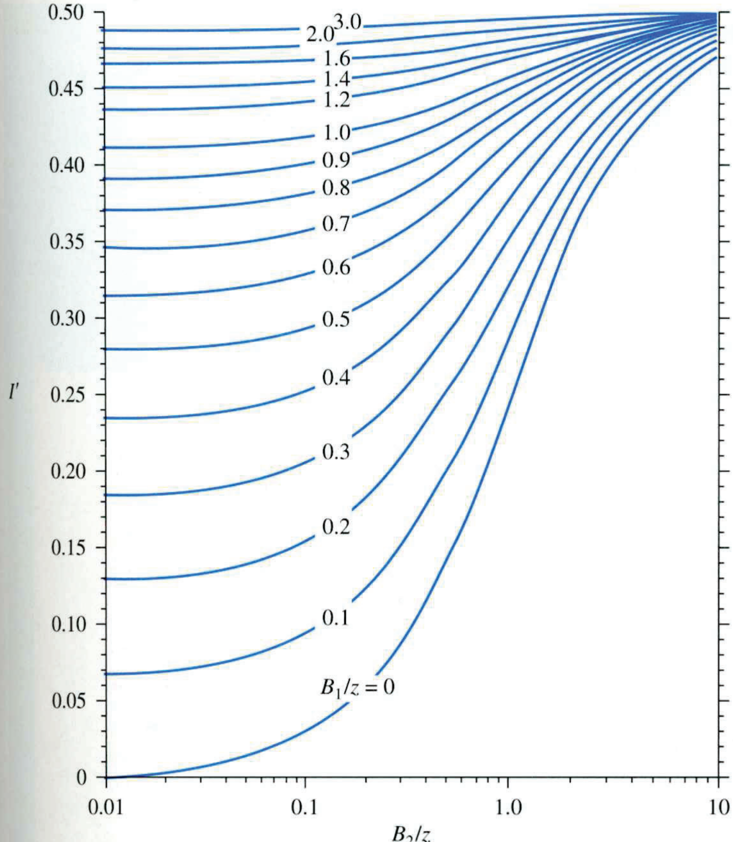


Figure 5.11 Influence value I' for embankment loading (After Osterberg, 1957) (Osterberg, J. O. (1957). "Influence Values for Vertical Stresses in Semi-Infinite Mass Due to Embankment Loading," Proceedings, Fourth International Conference on Soil Mechanics and Foundation Engineering, London, Vol. 1. pp. 393–396. With permission from ASCE.)

For a detailed derivation of Eq. (5.20), see Das (2008). A simplified form of the equation is

$$\Delta\sigma = q_o I' \tag{5.23}$$

where I' = a function of B_1/z and B_2/z.

The variation of I' with B_1/z and B_2/z is shown in Figure 5.11. An application of this diagram is given in Example 5.3.

Example 5.3

An embankment is shown in Figure 5.12a. Determine the stress increase under the embankment at points A_1 and A_2.

Solution

We have

$$\gamma H = (17.5)(7) = 122.5 \text{ kN/m}^2$$

Project: Mordecai Island Settlement Analysis
Objective: Determine the secondary settlement at the end of primary consolidation and at t=5, t=10 and t=20 years after end of primary consolidation.

MIB-1 Determine the time rate of consolidation for U=90%

$T_{v1} := 0.848$ $\quad H_{d1} := 6.85\ ft$ $\quad C_{v1} := 13\ \frac{ft^2}{yr}$ From MIB-1 Lab data

$$t_{90\_1} := \frac{T_{v1} \cdot H_{d1}^{\ 2}}{C_{v1}}$$

$$t_{90\_1} = 3.061\ yr$$

MIB-2 Determine the time rate of consolidation for U=90%

$T_{v2} := 0.848$ $\quad H_{d2} := 4.25\ ft$ $\quad C_{v2} := 42\ \frac{ft^2}{yr}$ From MIB-2 Lab data

$$t_{90\_2} := \frac{T_{v2} \cdot H_{d2}^{\ 2}}{C_{v2}}$$

$$t_{90\_2} = 0.365\ yr$$

MIB-3 Determine the time rate of consolidation for U=90%

$T_{v3} := 0.848$ $\quad H_{d3} := 6.75\ ft$ $\quad C_{v3} := 245\ \frac{ft^2}{yr}$ From MIB-3 Lab data

$$t_{90\_3} := \frac{T_{v3} \cdot H_{d3}^{\ 2}}{C_{v3}}$$

$$t_{90\_3} = 0.158\ yr$$

MIB-1 - Secondary Settlement

$w := 38$ w=final moisture content

$c'_\alpha := 0.0001 \cdot w$

$c'_\alpha = 0.004$

Find secondary consolidation after 5 years after end of primary consolidation

$t_1 := 3.06\ yr$

$t_2 := 5\ yr$

$H_c := 13.7\ ft$ Height of clay layer in MIB-1

$$S_{cs\_1a} := c'_\alpha \cdot H_c \cdot \log\left(\frac{t_2}{t_1}\right) = 0.133\ in$$

Find secondary consolidation after 10 years after end of primary consolidation

$t_{1a} := 3.06\ yr$

$t_{2a} := 10\ yr = 10\ yr$

$$S_{cs\_1b} := c'_\alpha \cdot H_c \cdot \log\left(\frac{t_{2a}}{t_{1a}}\right) = 0.321\ in$$

Find secondary consolidation after 20 years after end of primary consolidation

$t_{1b} := 3.06\ yr$

$t_{2b} := 20\ yr = 20\ yr$

$$S_{cs\_1c} := c'_\alpha \cdot H_c \cdot \log\left(\frac{t_{2b}}{t_{1b}}\right) = 0.509\ in$$

MIB-2 - Secondary Settlement

$w := 26.8$ w=final moisture content

$c'_{\alpha} := 0.0001 \cdot w$
$c'_{\alpha} = 0.003$
Find secondary consolidation after 5 years after end of primary consolidation

$t_1 := t_{90\_2} = 0.365\ yr$
$t_2 := 5\ yr$

$H_c := 8.5\ ft$ Height of clay layer in MIB-2

$$S_{cs\_2a} := c'_{\alpha} \cdot H_c \cdot \log\left(\frac{t_2}{t_1}\right) = 0.311\ in$$

Find secondary consolidation after 10 years after end of primary consolidation

$t_1 = 0.365\ yr$
$t_{2b} := 10\ yr$

$$S_{cs\_2b} := c'_{\alpha} \cdot H_c \cdot \log\left(\frac{t_{2b}}{t_1}\right) = 0.393\ in$$

Find secondary consolidation after 20 years after end of primary consolidation

$t_1 = 0.365\ yr$
$t_{2c} := 20\ yr$

$$S_{cs\_2c} := c'_{\alpha} \cdot H_c \cdot \log\left(\frac{t_{2c}}{t_1}\right) = 0.475\ in$$

MIB-3 - Secondary Settlement

$w := 30$ w=final moisture content

$c'_{\alpha} := 0.0001 \cdot w$
$c'_{\alpha} = 0.003$
Find secondary consolidation after 5 years after end of primary consolidation

$t_1 := t_{90\_3} = 0.158\ yr$
$t_2 := t_1 + 5\ yr = 5.158\ yr$

$H_c := 13.5\ ft$ Height of clay layer in MIB-3

$$S_{cs\_3a} := c'_{\alpha} \cdot H_c \cdot \log\left(\frac{t_2}{t_1}\right) = 0.736\ in$$

Find secondary consolidation after 10 years after end of primary consolidation

$t_1 = 0.158\ yr$
$t_{2b} := t_1 + 10\ yr = 10.158\ yr$

$$S_{cs\_3b} := c'_{\alpha} \cdot H_c \cdot \log\left(\frac{t_{2b}}{t_1}\right) = 0.879\ in$$

Find secondary consolidation after 20 years after end of primary consolidation

$t_1 = 0.158\ yr$
$t_{2c} := t_1 + 20\ yr = 20.158\ yr$

$$S_{cs\_3c} := c'_{\alpha} \cdot H_c \cdot \log\left(\frac{t_{2c}}{t_1}\right) = 1.024\ in$$

ATTACHMENT A1-4: SCHNABEL ENGINEERING, INC. GEOTECHNICAL TESTING

GEOTECHNICAL LABORATORY TESTING
MORDECAI ISLAND, BEACH HAVEN
NEW JERSEY

U.S. ARMY CORPS OF ENGINEERS
PHILADELPHIA DISTRICT
CONTRACT W912BU-05-D-001
TASK ORDER 0007

Schnabel Reference 04151127.07

FOR

U.S. ARMY CORPS OF ENGINEERS
PHILADELPHIA DISTRICT

WANAMAKER BUILDING
100 PENN SQUARE EAST
PHILADELPHIA, PA 19107-3396

March 9, 2006

PREPARED BY

Schnabel Engineering

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GEOTECHNICAL LABORATORY TESTING
MORDECAI ISLAND, BEACH HAVEN
NEW JERSEY

TABLE OF CONTENTS

1.0 INTRODUCTION

In accordance with Contract W912BU-05-D-0001, Task Order 0007, Schnabel Engineering, Inc. (Schnabel), has completed the geotechnical laboratory testing of samples obtained during a previous field investigation program that took place during the 1st quarter of 2005.

2.0 PROJECT DESCRIPTION

The Philadelphia District of the U.S. Army Corps of Engineers (USACE) conducted a subsurface exploration immediately adjacent to Mordecai Island, Beach Haven, New Jersey. This was accomplished by means of subaqueous drive sample borings and undisturbed sample borings, in accordance with ASTM D1586 and D1587. The purpose of the work described herein is to determine the soil properties including strength and compressibility characteristics of the subsurface materials obtained from this exploration program completed in the 1st quarter of 2005. The material is comprised of interlayered marine sediments (silts and clays) and sands.

3.0 LABORATORY TESTING

Jar samples and tube samples were provided to our office in September 2005. These samples were tested for moisture content, density, Atterberg limits, specific gravity, unconfined compressive strength, and consolidation characteristics in accordance with the testing schedule provided by Dennis Zeveney of the Philadelphia District.

The following summary of tests was completed and is included as Appendices A and B for your use.

Test Description	**No. of Tests Completed**	**No. of Tests Proposed**	**Remarks**
Natural Moisture Content and Unit Weight	4	15	Notes 1 & 2
Consolidation Time Compression Tests	3 12 + 9	3 12 + 9	Note 3
Atterberg Limits	4	6	Note 1
Sieve Analysis w/o Hydrometer	10	15	Note 4
Unconfined Compression	3	3	

Notes:

1. Samples MIB-1, S-11, 24 to 26 ft; and MIB-3, S-13, 26 to 28 ft were not included with the samples provided to our laboratory. Therefore, the Atterberg Limit, moisture content and density testing requested was not performed.
2. The samples delivered for the majority of the testing consisted of jar samples. Due to the disturbance of these samples when placing them in the jars, and the long duration of storage prior to delivering these to the laboratory, accurate measurement of field moisture content and density was not practical. Therefore, this type of testing was only completed on the undisturbed tube samples delivered to the laboratory.
3. As part of the base consolidation testing fee, four Time Compression Curves are generated per test. Three additional curves were generated per test, at your request.
4. One extra sample was completed and six samples not presented in the report.

The consolidation testing and unconfined compression testing were completed on samples as extruded from the undisturbed tubes. No remolding of these samples was required.

4.0 DISCUSSION

Three laboratory consolidation tests were completed as proposed in the lab testing schedule. The consolidation parameters (P_p', C_r, C_c) were calculated by the COE and are presented on the Consolidation Test Report sheets.

It is also important to note that the possibility of sample disturbance was noted in the three Shelby tube samples. The extended period between sample collection (March 2005) and testing (October 2005) raises the question of disturbance and quality of the test results. The Norwegian Geotechnical Institute (NGI) has proposed a method of evaluating sample disturbance that is gaining acceptance in the industry: Sample Disturbance Effects in Soft Low Plastic Norwegian Clay, *Symposium on Recent Developments in Soil and Pavement Mechanics Rio de Janeiro*, as referenced by Lunne, T., Berre, T., and Strandvik, S. (1997). The basic premise is that the amount of relative change in void ratio that a specimen experiences, when consolidating back to in situ stresses, is an indication of the sampling disturbance in the specimen.

The relative change in void ratio is defined as the change in void ratio (Δe) divided by the initial void ratio (e_o). You indicated that the in situ effective overburden stress for each of the three consolidation test samples is roughly 0.5 tsf, which corresponds to a loading increment in the test. Therefore, the computed relative void ratio changes in the tests are as follows:

- MIB-1, 20 to 22 ft = 0.11
- MIB-2, 24 to 25.5 ft = 0.13
- MIB-3, 18 to 20 ft = 0.06

The NGI evaluation system for soils with an overconsolidation ratio of 1 to 2 is as follows:

- Very Good to Excellent Quality (relative void ratio change <0.04)
- Good to Fair (relative void ratio change 0.04 to 0.07)
- Poor (relative void ratio change 0.07 to 0.14)
- Very Poor (relative void ratio change >0.14)

Based on these criteria, the NGI evaluation system indicates that the sample quality ranges from fair to poor. We recommend consideration of the sample quality in interpretation and use of these results.

5.0 LIMITATIONS

We have endeavored to complete the services identified herein in a manner consistent with that level of care and skill ordinarily exercised by members of the profession currently practicing in the same locality and under similar conditions as this project. No other representation, express or implied, is included or intended, and no warranty or guarantee is included or intended in this report, or any other instrument of service.

APPENDIX A

Jar Sample Test Results

Boring No.	Sample Depth (ft)	Elevation	Sample Type	Description of Soil Specimen	Stratum	Atterberg Limits LL	PL	PI	Natural Moisture (%)	% Passing No. 200 Sieve	Remarks
MIB-1	10.0		Jar	SILTY SAND, trace rock fragments, contains mica - gray (SM)						40	
MIB-1	24.0		Jar	SILTY SAND, contains rock fragments and mica - gray-brown (SM)						42	
MIB-1	26.0		Jar	SILTY SAND, trace mica - gray-brown (SM)						14	
MIB-1	34.0		Jar	SILTY SAND, trace gravel, contains mica - gray (SM)						26	
MIB-1	46.0		Jar	WELL GRADED SAND WITH SILT, trace rock fragments - gray (SW-SM)						8	
MIB-2	12.0		Jar	SILT WITH SAND, trace gravel, contains seashells - gray (ML)						79	
MIB-2	38.0		Jar	SILTY SAND, contains mica and gravel - gray (ML)						20	
MIB-2	48.0		Jar	POORLY GRADED SAND WITH SILT, contains mica and gravel - brown (SP-SM)						8	

Notes:
1. Soil tests were performed in accordance with applicable ASTM Standards.
2. Soil classifications are in accordance with ASTM D2487, based on testing indicated and visual identification.
3. Key to abbreviations: LL=Liquid Limit; PL=Plastic Limit; PI=Plasticity Index; NP=Non-Plastic.
4. Soil tests were conducted by: IG.

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SUMMARY OF SOIL LABORATORY TESTS

Project: MORDECAI ISLAND

Boring No.	Sample Depth (ft) / Elevation	Sample Type	Description of Soil Specimen	Stratum	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI	Natural Moisture (%)	% Passing No. 200 Sieve	Remarks
MIB-3	28.0	Jar	SILTY SAND , contains mica - gray brown (SM)						15	
MIB-3	44.0	Jar	SILT, trace sand, contains mica - gray (MH)						86	

Notes:
1. Soil tests were performed in accordance with applicable ASTM Standards.
2. Soil classifications are in accordance with ASTM D2487, based on testing indicated and visual identification.
3. Key to abbreviations: LL=Liquid Limit; PL=Plastic Limit; PI=Plasticity Index; NP=Non-Plastic.
4. Soil tests were conducted by: IG.

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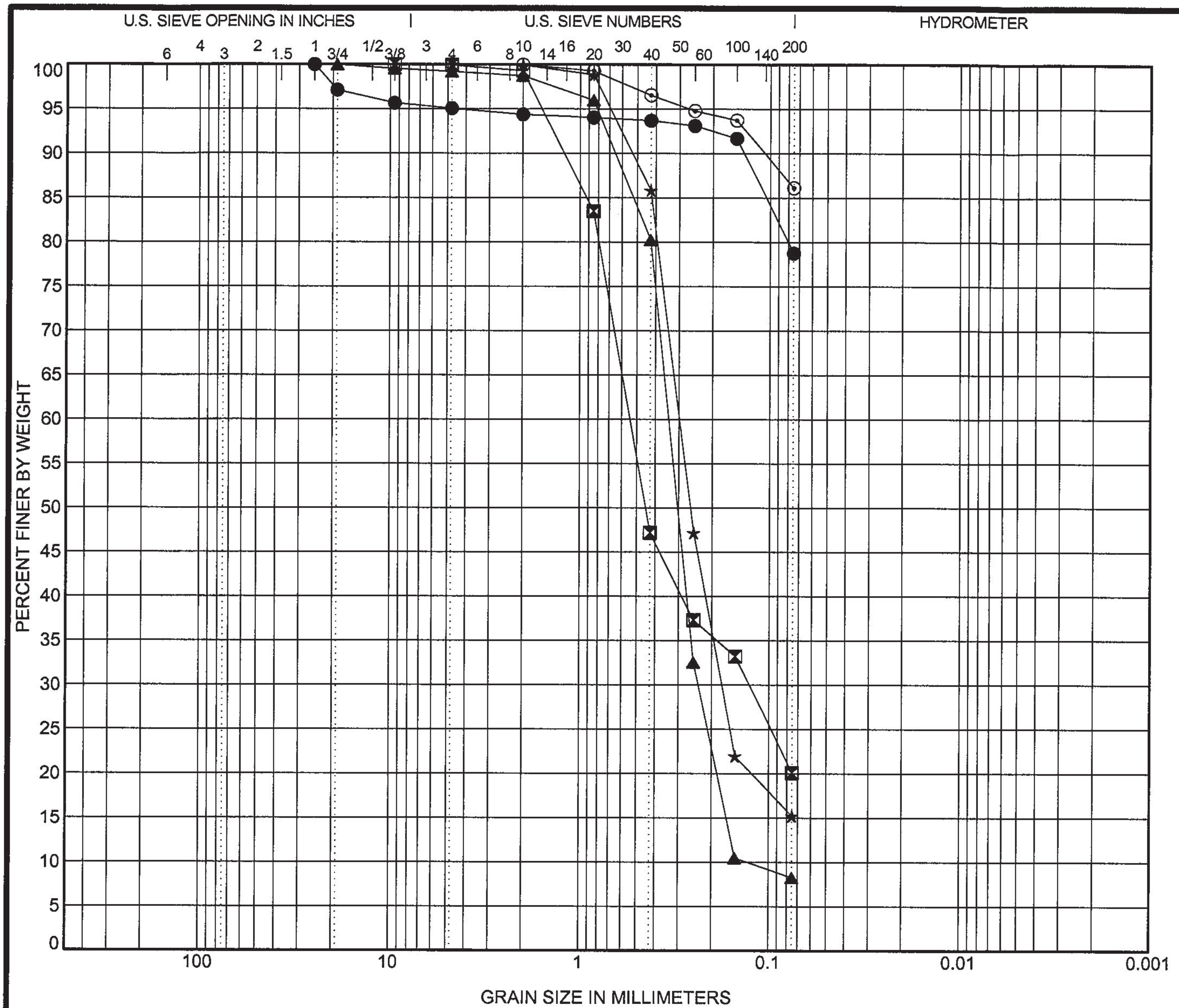
SUMMARY OF SOIL LABORATORY TESTS

Project: MORDECAI ISLAND



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen		Classification	LL	PL	PI	Cc	Cu
●	MIB-2	12.0 ft	SILT WITH SAND, trace gravel, contains seashells - gray (ML)					
⊠	MIB-2	38.0 ft	SILTY SAND, contains mica and gravel - gray (ML)					
▲	MIB-2	48.0 ft	POORLY GRADED SAND WITH SILT, contains mica and gravel - brown (SP-SM)				1.27	2.63
★	MIB-3	28.0 ft	SILTY SAND , contains mica - gray brown (SM)					
⊙	MIB-3	44.0 ft	SILT, trace sand, contains mica - gray (MH)					

	Specimen		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	MIB-2	12.0 ft	25				4.9	16.3	78.8	
⊠	MIB-2	38.0 ft	9.5	0.543	0.127		0.1	79.8	20.1	
▲	MIB-2	48.0 ft	19	0.339	0.236	0.129	0.8	91.0	8.3	
★	MIB-3	28.0 ft	4.75	0.298	0.177		0.0	84.8	15.2	
⊙	MIB-3	44.0 ft	4.75				0.0	13.9	86.1	

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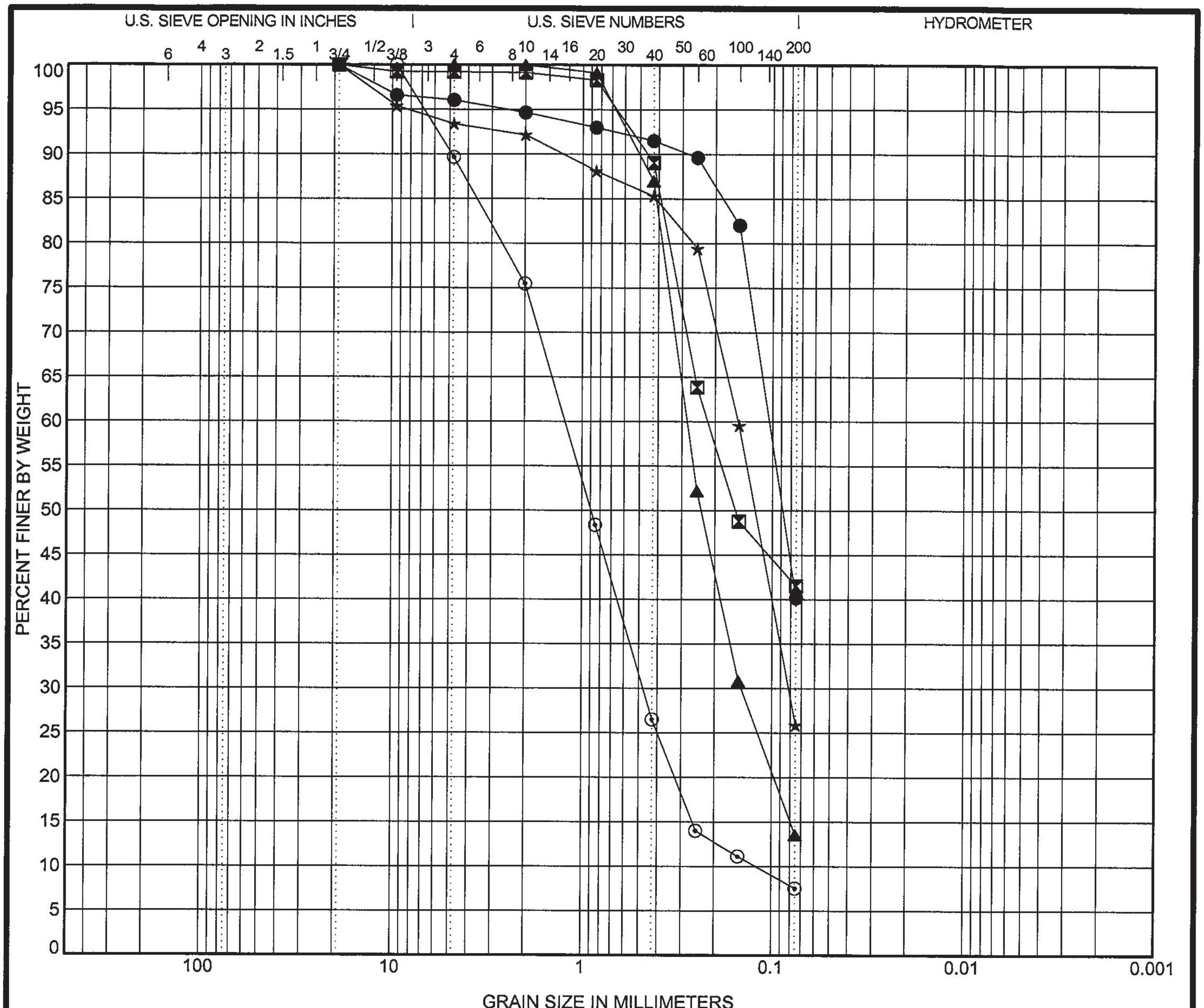
GRADATION CURVES

Project: MORDECAI ISLAND

Contract: 04151127.07

US GRAIN SIZE 04151127.07.GPJ SCHNABEL.GDT 3/9/06



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen		Classification	LL	PL	PI	Cc	Cu
●	MIB-1	10.0 ft	SILTY SAND, trace rock fragments, contains mica - gray (SM)					
⊠	MIB-1	24.0 ft	SILTY SAND, contains rock fragments and mica - gray-brown (SM)					
▲	MIB-1	26.0 ft	SILTY SAND, trace mica - gray-brown (SM)					
★	MIB-1	34.0 ft	SILTY SAND, trace gravel, contains mica - gray (SM)					
⊙	MIB-1	46.0 ft	WELL GRADED SAND WITH SILT, trace rock fragments - gray (SW-SM)				1.52	10.17

	Specimen		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	MIB-1	10.0 ft	19	0.104			4.0	55.9	40.2	
⊠	MIB-1	24.0 ft	19	0.219			0.8	57.7	41.5	
▲	MIB-1	26.0 ft	4.75	0.282	0.145		0.0	86.4	13.6	
★	MIB-1	34.0 ft	19	0.152	0.082		6.6	67.5	25.9	
⊙	MIB-1	46.0 ft	9.5	1.227	0.475	0.121	10.3	82.1	7.5	

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GRADATION CURVES

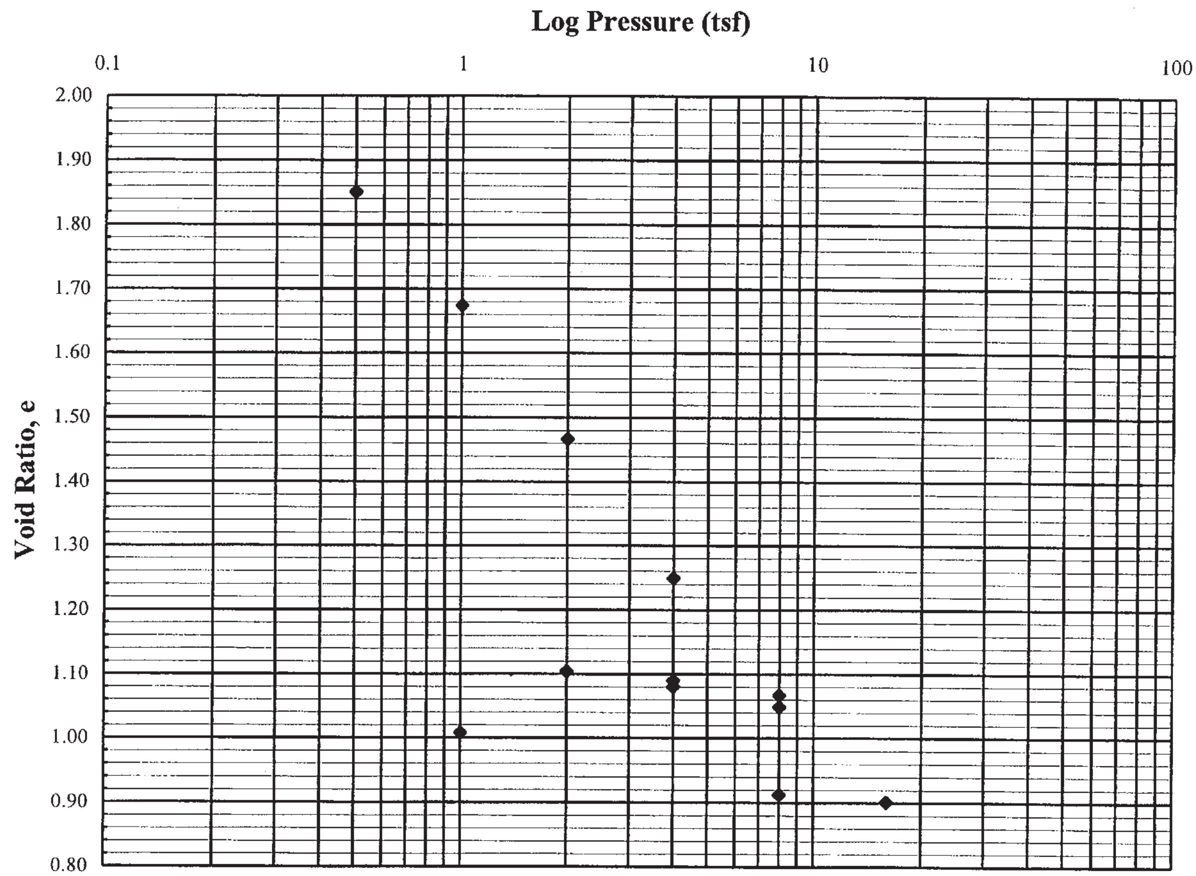
Project: MORDECAI ISLAND

Contract: 04151127.07

US GRAIN SIZE 04151127.07.GPJ SCHNABEL.GDT 3/9/06

APPENDIX B

Shelby Tube Test Results



Probable Preconsolidation Pressure (Pp), tsf:						*0.87*	
Type of Specimen:	*Tube Sample*						
Description:	*FAT CLAY (CH), trace sand, contains mica - gray*						
LL:	*65*	**PI:**	*36*	**Gs:**	*2.71*	**P_o' (tsf):**	*0.50*
% < No. 200:	*NA*	**Test Method:**	*ASTM D2435 Method A*				
Test Condition:	*Inundated @ 0.05 tsf*						
Remarks:							

Recompression Index (Cr):		*0.0914*
Compression Index (Cc):		*0.661*
	Initial	**Final**
Water Content, %	*39.1*	*38.0*
Void Ratio	*2.07*	*1.01*
Saturation, %	*51*	*100*
Dry Unit Weight, pcf	*55.1*	*84.2*

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Project:	Mordecai Island Geotechnical Laboratory Testing		
Location:	*Beach Haven, NJ*		
Boring:	*MIB-1*	**Sample No.:**	*T-2*
Depth:	*20-22 ft.*	**Elevation:**	
Date:	*12/8/05*	**Schnabel No.:**	*04151127.07*

Consolidation Test Report

Consolidation Test Data Sheet

Consolidometer ID: G
Schnabel Contract: *04151127.07*
Project: *Mordecai Island Geotechnical Laboratory Testing*
Boring No.: *MIB-1*
Depth: *20-22 ft.*

Test Method: *ASTM D2435 Method A*
Test Condition: *Inundated @ 0.05 tsf*

Initial Height of Specimen (H_o), in.: 0.7490
Height of Solids (H_s), in.: 0.2442
Seating Press. (tsf): 0.05
Initial Dial Gauge Reading (D_o), in.: 0.0032
Initial Apparatus Correction (D_{co}), in.: -0.0007

Pressure, P (tsf)	TimeReadings Required	Date Load Applied	Time Load Applied	Time Load Effective (min.)	A: Final[1] Dial Reading, D_{fi} x 10^{-4} in.	B: Apparatus Correction[2], D_{ci} x 10^{-4} in.	C: Cumulative Change in Height[3], ΔH_i in.	D: Height of Voids[4], H_{vi} in.	Vertical Strain[5], ε_i (%)	Void Ratio[6], e_i
0.5	X	10/5/2005	9:15		573	5	0.0529	0.4519	7.06	1.851
1	X	10/6/2005	9:05		1011	11	0.0961	0.4087	12.83	1.674
2	X	10/7/2005	8:55		1524	18	0.1467	0.3581	19.59	1.467
4	X	10/8/2005	8:50		2062	25	0.1998	0.3050	26.68	1.249
8	X	10/11/2005	10:45		2513	32	0.2442	0.2606	32.61	1.067
4		10/12/2005	10:00		2475	25	0.2411	0.2637	32.19	1.080
2		10/13/2005	9:25		2409	18	0.2352	0.2696	31.40	1.104
4	X	10/14/2005	9:25		2450	25	0.2386	0.2662	31.86	1.090
8	X	10/15/2005	9:25		2558	32	0.2487	0.2561	33.21	1.049
16	X	10/17/2005	9:25		2929	41	0.2849	0.2199	38.04	0.901
8		10/18/2005	9:25		2891	32	0.2820	0.2228	37.65	0.912
1		10/19/2005	9:25		2637	11	0.2587	0.2461	34.54	1.008

Notes:
1. "Final" based on test method; 24 hrs for Method A, end of primary for Method B.
2. Correction value, for the current pressure, from the consolidometer's calibration curve.
3. $\Delta H = D_{fi} - D_o - D_{ci} + D_{co}$ = Col. A - D_o - Col. B + D_{co}
4. $H_{vi} = (H_o - H_s) - \Delta H$
5. $\varepsilon_i = (\Delta H / H_o) \times 100$ = (Col. C / H_o) x 100
6. $e_i = H_{vi}$ / Hs = Col. D / Hs

Load Time Readings

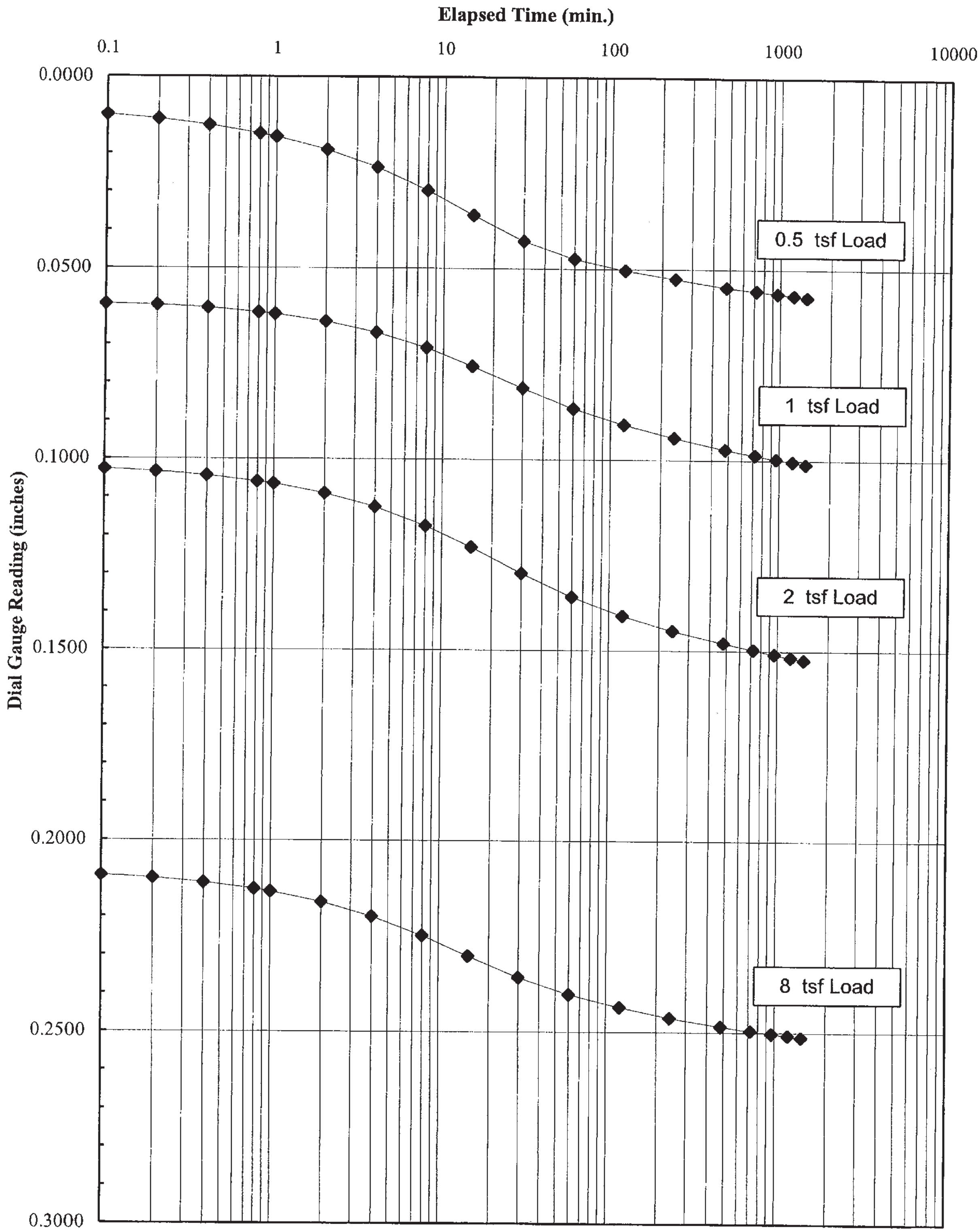
Project: Mordecai Island
Schnabel Contract: 04151127.07
Boring No.: MIB-1 Depth: 20-22 ft.

Consol. ID: G

Elapsed Time (min.)	Dial Guage Readings (inches)					
	0.5 tsf Initial Load 10/5/2005	1 tsf Initial Load 10/6/2005	2 tsf Initial Load 10/7/2005	8 tsf Initial Load 10/11/2005	X tsf Load Date	X tsf Load Date
0.1	0.0100	0.0595	0.1028	0.2093		
0.2	0.0111	0.0598	0.1034	0.2100		
0.4	0.0127	0.0605	0.1044	0.2112		
0.8	0.0149	0.0617	0.1060	0.2128		
1	0.0157	0.0621	0.1065	0.2135		
2	0.0191	0.0641	0.1090	0.2162		
4	0.0237	0.0669	0.1125	0.2200		
8	0.0297	0.0709	0.1174	0.2250		
15	0.0361	0.0757	0.1230	0.2303		
30	0.0428	0.0813	0.1298	0.2358		
60	0.0474	0.0866	0.1360	0.2402		
120	0.0504	0.0908	0.1409	0.2435		
240	0.0526	0.0942	0.1448	0.2463		
480	0.0548	0.0972	0.1480	0.2485		
720	0.0557	0.0987	0.1497	0.2496		
960	0.0564	0.0997	0.1508	0.2503		
1200	0.0569	0.1004	0.1517	0.2508		
1440	0.0574	0.1011	0.1524	0.2513		
1680						
1920						
2160						
2400						
2640						
2880						

Consolidation Time Curves

Project: Mordecai Island
Schnabel Contract: 04151127.07
Boring No.: MIB-1 Depth: 20-22 ft.



Load Time Readings

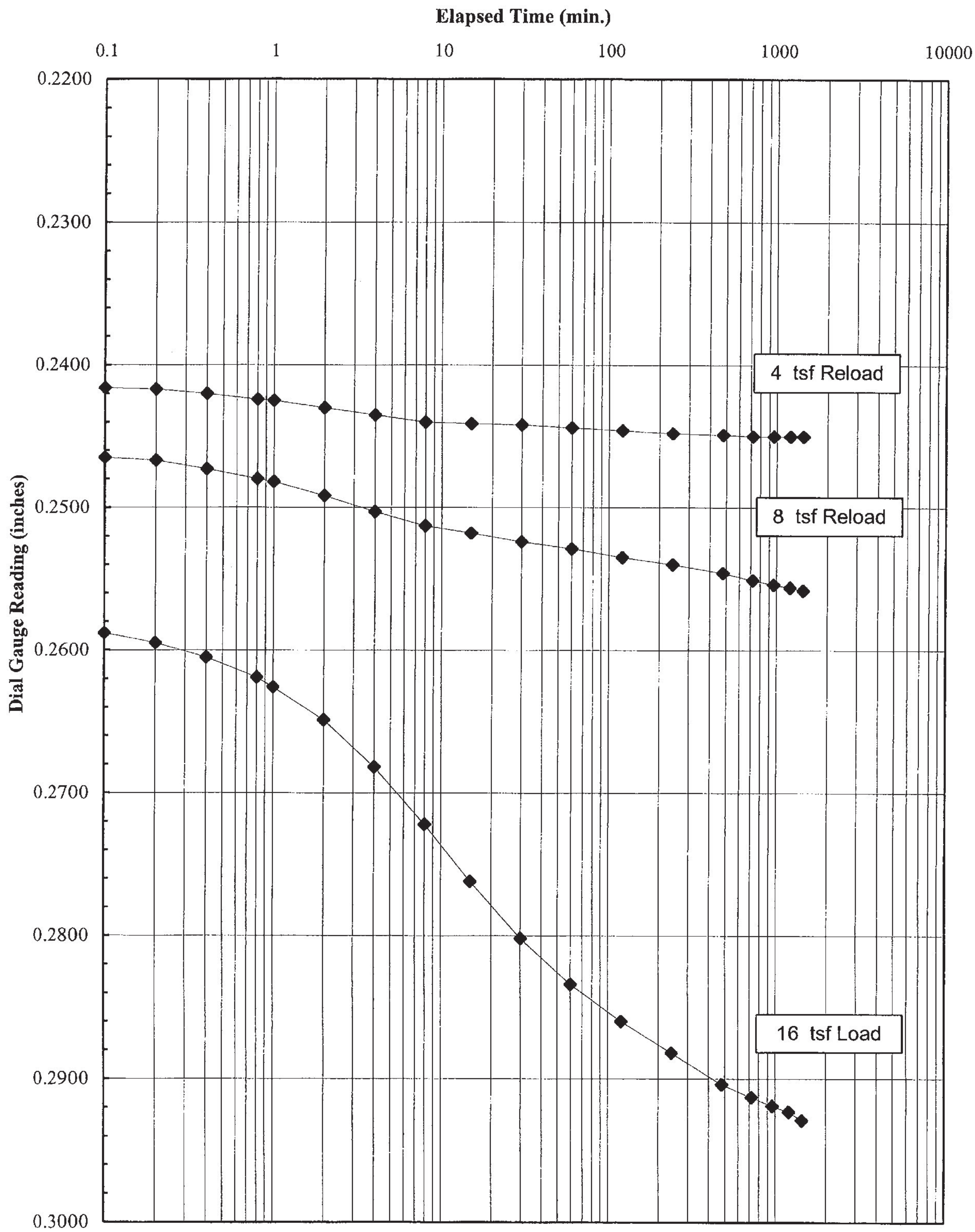
Project: Mordecai Island
Schnabel Contract: 04151127.07
Boring No.: MIB-1 Depth: 20-22 ft.

Consol. ID: G

Elapsed Time (min.)	Dial Guage Readings (inches)					
	4 tsf Reload 10/14/2005	8 tsf Reload 10/15/2005	16 tsf Reload 10/17/2005	X tsf Load Date	X tsf Load Date	X tsf Load Date
0.1	0.2416	0.2465	0.2588			
0.2	0.2417	0.2467	0.2595			
0.4	0.2420	0.2473	0.2605			
0.8	0.2424	0.2480	0.2619			
1	0.2425	0.2482	0.2626			
2	0.2430	0.2492	0.2649			
4	0.2435	0.2503	0.2682			
8	0.2440	0.2513	0.2722			
15	0.2441	0.2518	0.2762			
30	0.2442	0.2524	0.2802			
60	0.2444	0.2529	0.2834			
120	0.2446	0.2535	0.2860			
240	0.2448	0.2540	0.2882			
480	0.2449	0.2546	0.2904			
720	0.2450	0.2551	0.2913			
960	0.2450	0.2554	0.2919			
1200	0.2450	0.2556	0.2923			
1440	0.2450	0.2558	0.2929			
1680						
1920						
2160						
2400						
2640						
2880						

Consolidation Time Curves

Project: Mordecai Island
Schnabel Contract: 04151127.07
Boring No.: MIB-1 Depth: 20-22 ft.



Coefficient of Consolidation (C_v) versus Log Pressure

Schnabel Contract: 04151127.07 Date: 10/21/2005

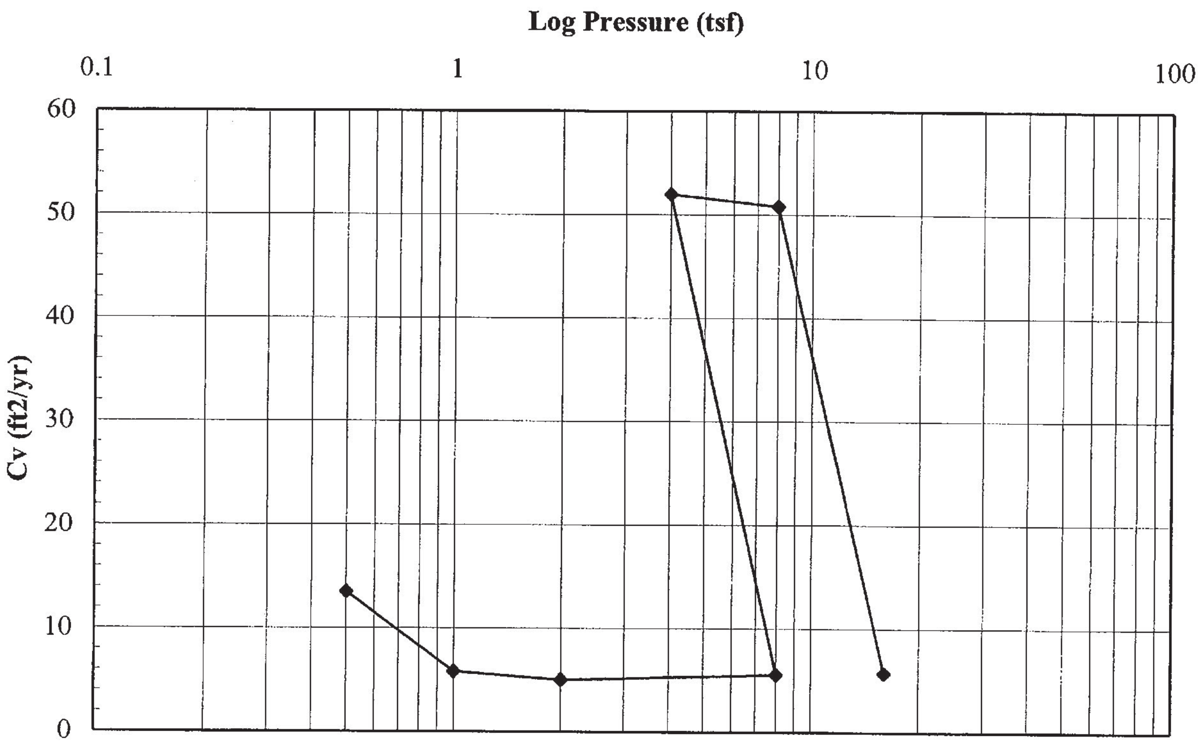
Project: Mordecai Island

Beach Haven, NJ

Boring: MIB-1

Depth, ft: 20-22

Cv Computation Method: Log Time



Applied Press., tsf	C_v ft^2/yr
0.5	13
1	6
2	5
8	6
4	51.9
8	50.8
16	5.7

Applied Press., tsf	C_v ft^2/yr

Schnabel Engineering

Unconfined Compression Test Report
ASTM D2166

Date: *10/21/2005*

Project: *Mordecai Island*
Geotechnical Laboratory Testing
Location: *Beach Haven, NJ*

Schnabel No.: *04151127.07*
Boring No.: *MIB-1*
Depth: *20-22 ft*
Elevation:

Test Specimen Data	
Initial Diameter, in:	2.869
Initial Area, in^2:	6.467
Initial Height, in:	5.819
Height/Diameter Ratio:	2.0
Moisture Content, %:	68.1
Wet Unit Weight, pcf:	99.9
Dry Unit Weight, pcf:	59.4

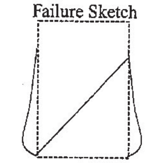
Testing Information	
Test Procedure:	ASTM D2166
Type of Specimen:	Tube Sample
Load Cell No.:	79
Strain Rate, %/min.:	1.0

Remarks:

Specimen Type: *Tube Sample*

Strength Data	
Strain at Failure, %:	12.0
Unconfined Strength (Qu), tsf:	0.28
Shear Strength (Su), tsf:	0.14

Failure Sketch



Reviewed by: *CJS*

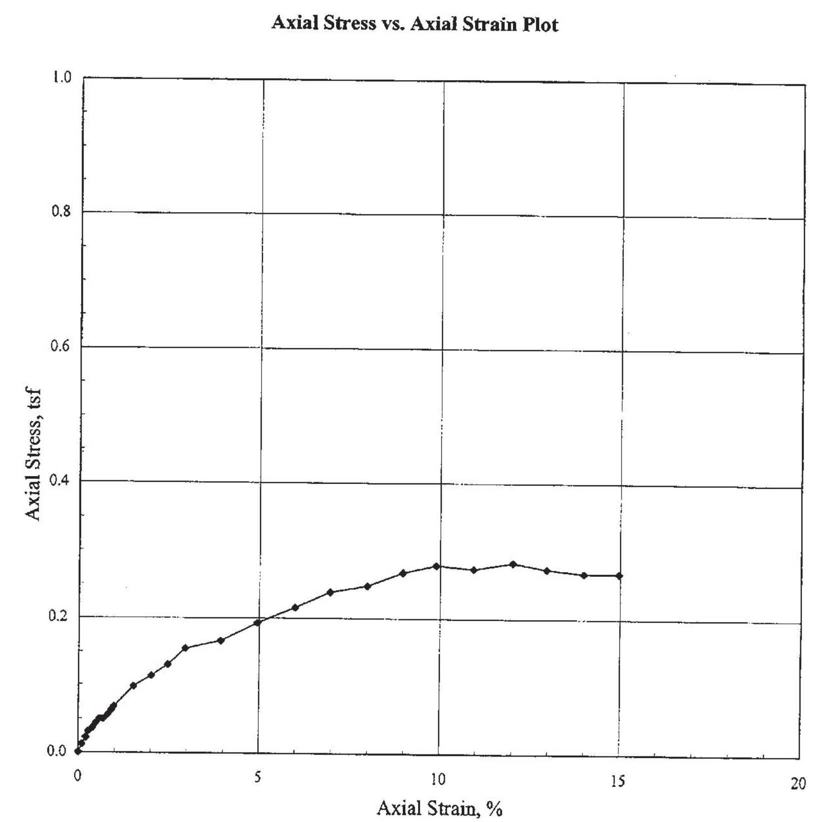
Soil Desc.: *FAT CLAY (CH), trace sand, contins mica - gray*

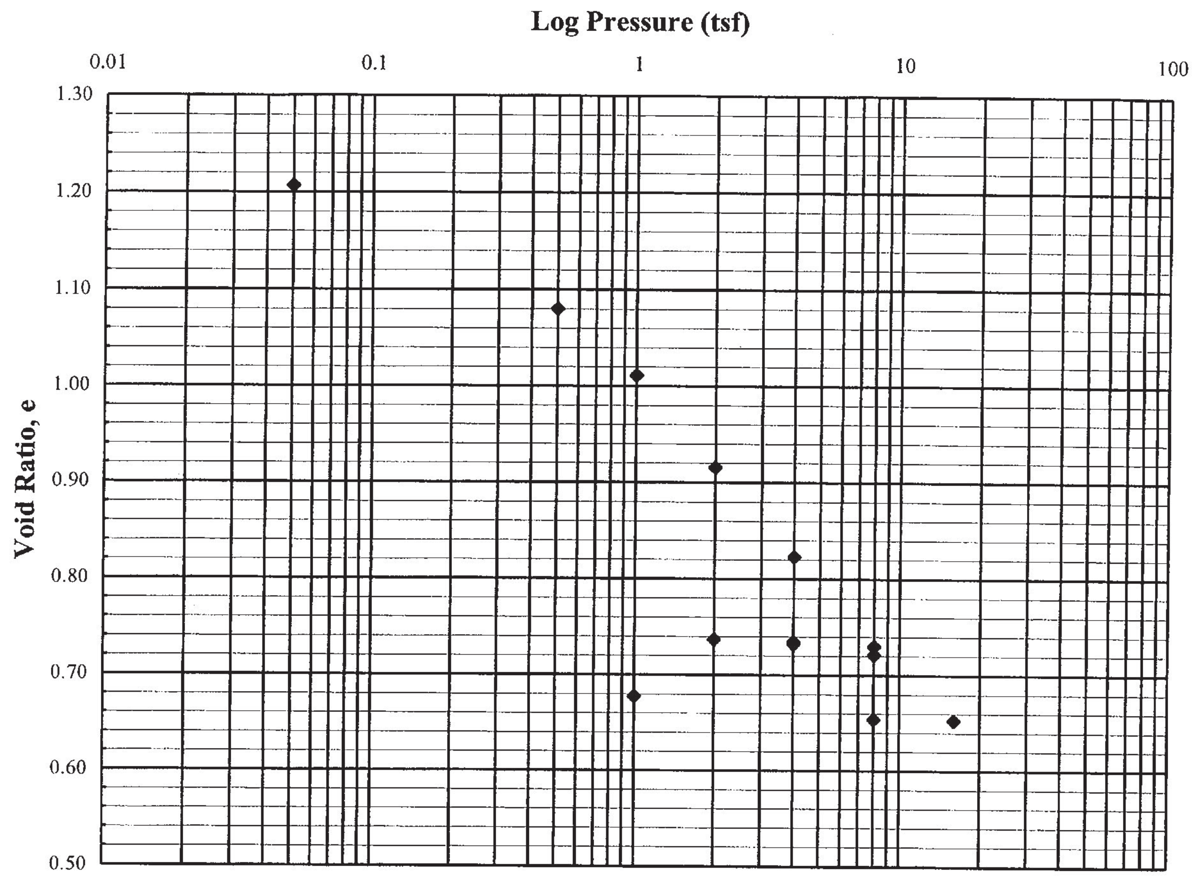
LL, %:	65	% < 200:	NA
PI, %:	36	Gs:	2.71

Test Data

Reading No.	Axial Load (lbs)	Axial Displ. (in.)	Axial Strain (%)	Corrected Area[1] (in^2)	Axial Stress (psi)	Axial Stress (tsf)
Zero	0.0	0.000	0.0	6.47	0.0	0.00
1	1.1	0.006	0.1	6.47	0.2	0.01
2	2.0	0.012	0.2	6.48	0.3	0.02
3	2.8	0.016	0.3	6.49	0.4	0.03
4	3.3	0.023	0.4	6.49	0.5	0.04
5	3.9	0.028	0.5	6.50	0.6	0.04
6	4.5	0.034	0.6	6.51	0.7	0.05
7	4.5	0.040	0.7	6.51	0.7	0.05
8	5.1	0.047	0.8	6.52	0.8	0.06
9	5.7	0.053	0.9	6.53	0.9	0.06
10	6.2	0.057	1.0	6.53	0.9	0.07
11	8.9	0.088	1.5	6.57	1.4	0.10
12	10.4	0.117	2.0	6.60	1.6	0.11
13	12.0	0.144	2.5	6.63	1.8	0.13
14	14.3	0.172	3.0	6.66	2.1	0.15
15	15.5	0.229	3.9	6.73	2.3	0.17
16	18.2	0.288	4.9	6.80	2.7	0.19
17	20.6	0.349	6.0	6.88	3.0	0.22
18	23.0	0.405	7.0	6.95	3.3	0.24
19	24.2	0.465	8.0	7.03	3.4	0.25
20	26.4	0.522	9.0	7.10	3.7	0.27
21	27.8	0.576	9.9	7.18	3.9	0.28
22	27.6	0.636	10.9	7.26	3.8	0.27
23	28.9	0.699	12.0	7.35	3.9	0.28
24	28.2	0.754	13.0	7.43	3.8	0.27
25	27.9	0.814	14.0	7.52	3.7	0.27
26	28.2	0.871	15.0	7.61	3.7	0.27

Notes: 1. Right Cylinder Area Correction Method





Probable Preconsolidation Pressure (Pp), tsf:	*0.75*
Recompression Index (Cr):	*0.0266*
Compression Index (Cc):	*0.330*

Type of Specimen: *Tube Sample*

Description: *LEAN CLAY (CL), trace sand, contains mica - gray*

LL:	*28*	**PI:**	*9*	**Gs:**	*2.75*	**P_o' (tsf):**	*0.50*
% < No. 200:	*NA*	**Test Method:**	*ASTM D2435 Method A*				
Test Condition:	*Inundated @ 0.05 tsf*						

	Initial	Final
Water Content, %	*44.4*	*26.8*
Void Ratio	*1.24*	*0.68*
Saturation, %	*98*	*100*
Dry Unit Weight, pcf	*76.5*	*102.3*

Remarks:

Project: Mordecai Island Geotechnical Laboratory Testing

Location:	*Beach Haven, NJ*		
Boring:	*MIB- 2*	**Sample No.:**	*T-1*
Depth:	*16-18 ft.*	**Elevation:**	
Date:	*12/8/05*	**Schnabel No.:**	*04151127.07*

Schnabel Engineering

Consolidation Test Report

Consolidation Test Data Sheet

Consolidometer ID: H
Schnabel Contract: *04151127.07*
Project: *Mordecai Island Geotechnical Laboratory Testing*
Boring No.: *MIB- 2*
Depth: *16-18 ft.*

Test Method: *ASTM D2435 Method A*
Test Condition: *Inundated @ 0.05 tsf*

Initial Height of Specimen (H_o), in.: 0.7504
Height of Solids (H_s), in.: 0.3346
Seating Press. (tsf): 0.02
Initial Dial Gauge Reading (D_o), in.: 0.0000
Initial Apparatus Correction (D_{co}), in.: 0.0000

					A	B	C	D		
Pressure, P (tsf)	TimeReadings Required	Date Load Applied	Time Load Applied	Time Load Effective (min.)	Final[1] Dial Reading, D_{fi} x 10^{-4} in.	Apparatus Correction[2], D_{ci} x 10^{-4} in.	Cumulative Change in Height[3], ΔH_i in.	Height of Voids[4], H_{vi} in.	Vertical Strain[5], ε_i (%)	Void Ratio[6], e_i
0.05		10/5/2005			115	-5	0.0120	0.4038	1.60	1.207
0.5	X	10/6/2005	8:45		563	20	0.0543	0.3615	7.24	1.080
1	X	10/7/2005	8:45		809	35	0.0774	0.3384	10.32	1.011
2	X	10/8/2005	8:45		1143	49	0.1094	0.3064	14.58	0.916
4	X	10/11/2005	10:40		1462	58	0.1404	0.2754	18.71	0.823
8	X	10/12/2005	10:40		1783	67	0.1716	0.2442	22.87	0.730
4		10/13/2005	10:40		1767	58	0.1709	0.2449	22.78	0.732
2		10/14/2005	9:15		1741	49	0.1692	0.2466	22.55	0.737
4	X	10/15/2005	9:15		1757	58	0.1699	0.2459	22.64	0.735
8	X	10/17/2005	9:15		1812	67	0.1745	0.2413	23.26	0.721
16	X	10/18/2005	9:15		2048	74	0.1974	0.2184	26.31	0.653
8		10/19/2005	9:15		2037	67	0.1970	0.2188	26.25	0.654
1		10/20/2005	9:15		1926	35	0.1891	0.2267	25.20	0.677

Notes:
1 "Final" based on test method; 24 hrs for Method A, end of primary for Method B.
2 Correction value, for the current pressure, from the consolidometer's calibration curve.
3 $\Delta H = D_{fi} - D_o - D_{ci} + D_{co}$ = Col. A - D_o - Col. B + D_{co}
4 $H_{vi} = (H_o - H_s) - \Delta H$
5 $\varepsilon_i = (\Delta H / H_o) \times 100$ = (Col. C / H_o) x 100
6 $e_i = H_{vi} / Hs$ = Col. D / Hs

Load Time Readings

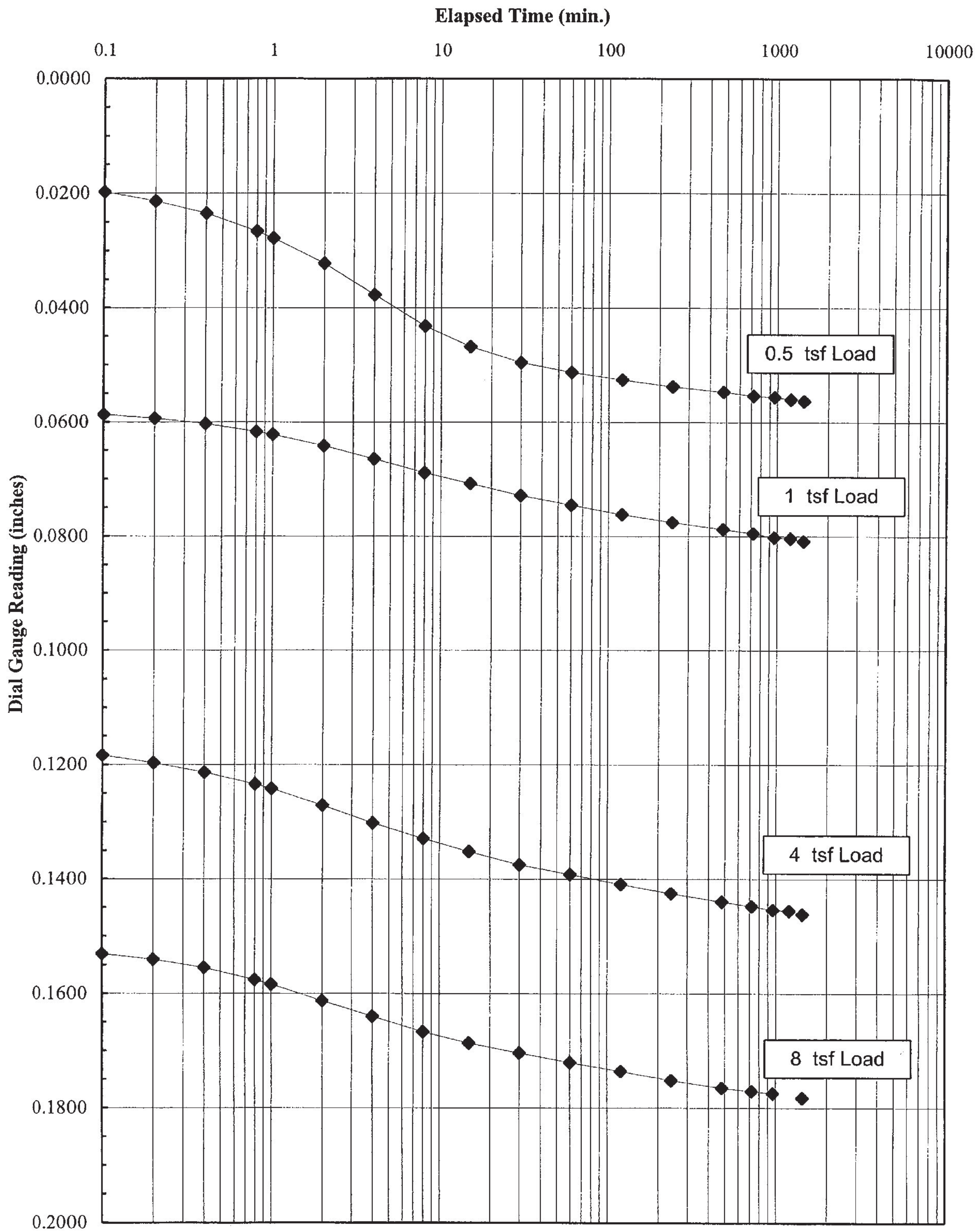
Project: Mordecai Island
Schnabel Contract: 04151127.07
Boring No.: MIB- 2 Depth: 16-18 ft.

Consol. ID: H

Elapsed Time (min.)	Dial Guage Readings (inches)					
	0.5 tsf Initial Load 10/6/2005	1 tsf Initial Load 10/7/2005	4 tsf Initial Load 10/11/2005	8 tsf Initial Load 10/12/2005	X tsf Load Date	X tsf Load Date
0.1	0.0198	0.0587	0.1184	0.1531		
0.2	0.0214	0.0594	0.1197	0.1541		
0.4	0.0235	0.0603	0.1214	0.1555		
0.8	0.0266	0.0617	0.1234	0.1576		
1	0.0278	0.0622	0.1242	0.1584		
2	0.0322	0.0642	0.1271	0.1613		
4	0.0377	0.0665	0.1302	0.1640		
8	0.0432	0.0689	0.1329	0.1667		
15	0.0468	0.0708	0.1352	0.1687		
30	0.0496	0.0729	0.1375	0.1704		
60	0.0513	0.0746	0.1392	0.1721		
120	0.0526	0.0762	0.1409	0.1736		
240	0.0538	0.0776	0.1425	0.1752		
480	0.0547	0.0788	0.1440	0.1765		
720	0.0554	0.0795	0.1448	0.1771		
960	0.0556	0.0802	0.1454	0.1775		
1200	0.0560	0.0804	0.1456			
1440	0.0563	0.0809	0.1462	0.1783		
1680						
1920						
2160						
2400						
2640						
2880						

Consolidation Time Curves

Project: Mordecai Island
Schnabel Contract: 04151127.07
Boring No.: MIB- 2 Depth: 16-18 ft.



Load Time Readings

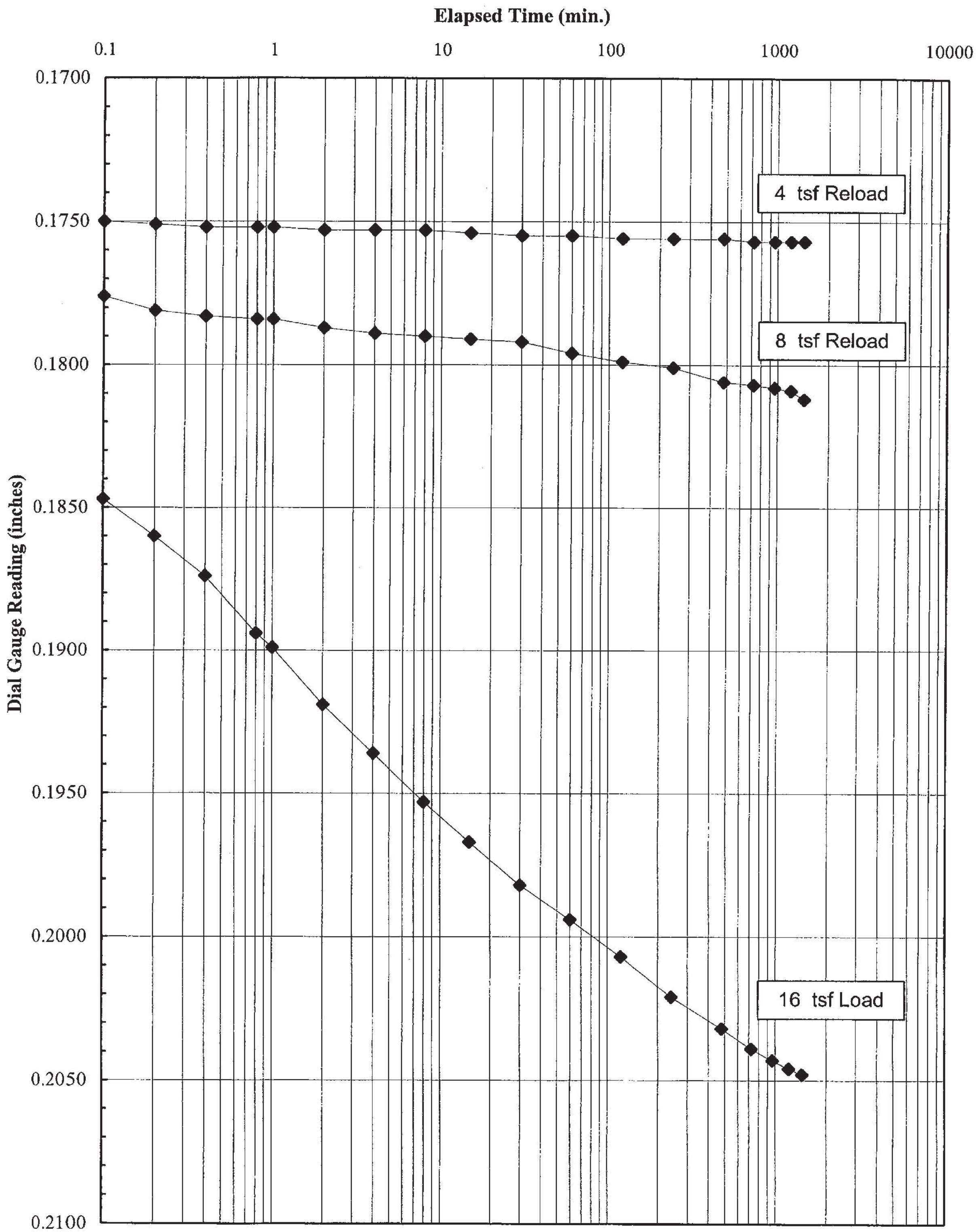
Project: Mordecai Island
Schnabel Contract: 04151127.07
Boring No.: MIB- 2 Depth: 16-18 ft.

Consol. ID: H

Elapsed Time (min.)	Dial Guage Readings (inches)					
	4 tsf Reload 10/15/2005	8 tsf Reload 10/17/2005	16 tsf Load 10/18/2005	X tsf Load Date	X tsf Load Date	X tsf Load Date
0.1	0.1750	0.1776	0.1847			
0.2	0.1751	0.1781	0.1860			
0.4	0.1752	0.1783	0.1874			
0.8	0.1752	0.1784	0.1894			
1	0.1752	0.1784	0.1899			
2	0.1753	0.1787	0.1919			
4	0.1753	0.1789	0.1936			
8	0.1753	0.1790	0.1953			
15	0.1754	0.1791	0.1967			
30	0.1755	0.1792	0.1982			
60	0.1755	0.1796	0.1994			
120	0.1756	0.1799	0.2007			
240	0.1756	0.1801	0.2021			
480	0.1756	0.1806	0.2032			
720	0.1757	0.1807	0.2039			
960	0.1757	0.1808	0.2043			
1200	0.1757	0.1809	0.2046			
1440	0.1757	0.1812	0.2048			
1680						
1920						
2160						
2400						
2640						
2880						

Consolidation Time Curves

Project: Mordecai Island
Schnabel Contract: 04151127.07
Boring No.: MIB- 2 Depth: 16-18 ft.



Coefficient of Consolidation (C_v) versus Log Pressure

Schnabel Contract: 04151127.07

Project: Mordecai Island

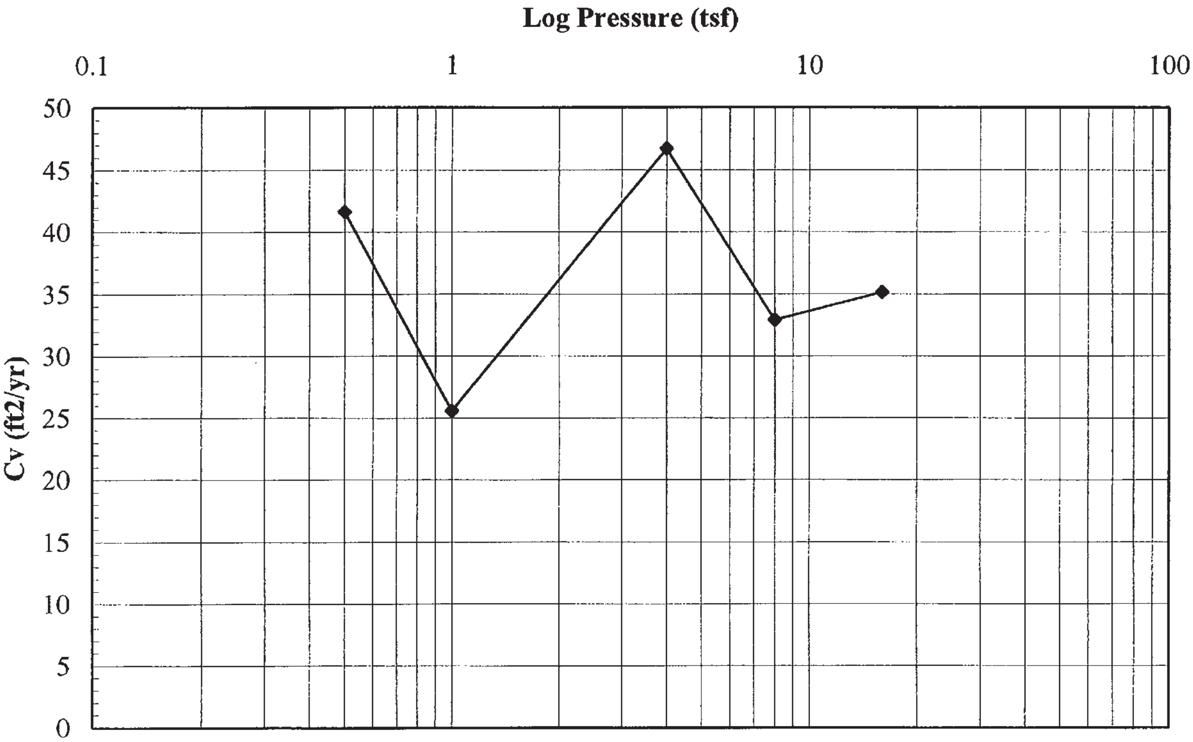
Beach Haven, NJ

Date: 10/21/2005

Boring: MIB-2

Depth, ft: 16-18

Cv Computation Method: Log Time



Applied Press., tsf	C_v ft^2/yr
0.5	42
1	26
4	47
8	33
16	35

Applied Press., tsf	C_v ft^2/yr

Schnabel Engineering

Unconfined Compression Test Report

ASTM D2166

Date: *10/21/2005*

Project: *Mordecai Island*
Geotechnical Laboratory Testing
Location: *Beach Haven, NJ*

Schnabel No.: *04151127.07*
Boring No.: *MIB-2*
Depth: *24-25.5 ft*
Elevation:

Test Specimen Data	
Initial Diameter, in:	*2.878*
Initial Area, in^2:	*6.509*
Initial Height, in:	*5.640*
Height/Diameter Ratio:	*2.0*
Moisture Content, %:	*15.5*
Wet Unit Weight, pcf:	*141.1*
Dry Unit Weight, pcf:	*122.3*

Testing Information	
Test Procedure:	*ASTM D2166*
Type of Specimen:	*Tube Sample*
Load Cell No.:	*79*
Strain Rate, %/min.:	*1.0*

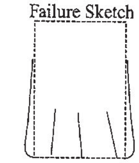
Remarks:

Specimen Type: *Tube Sample*

Strength Data	
Strain at Failure, %:	*13.9*
Unconfined Strength (Qu), tsf:	*0.28*
Shear Strength (Su), tsf:	*0.14*

Reviewed by: *CJS*

Failure Sketch



Soil Desc.: *SILT (ML), with sand, contains mica - light gray*

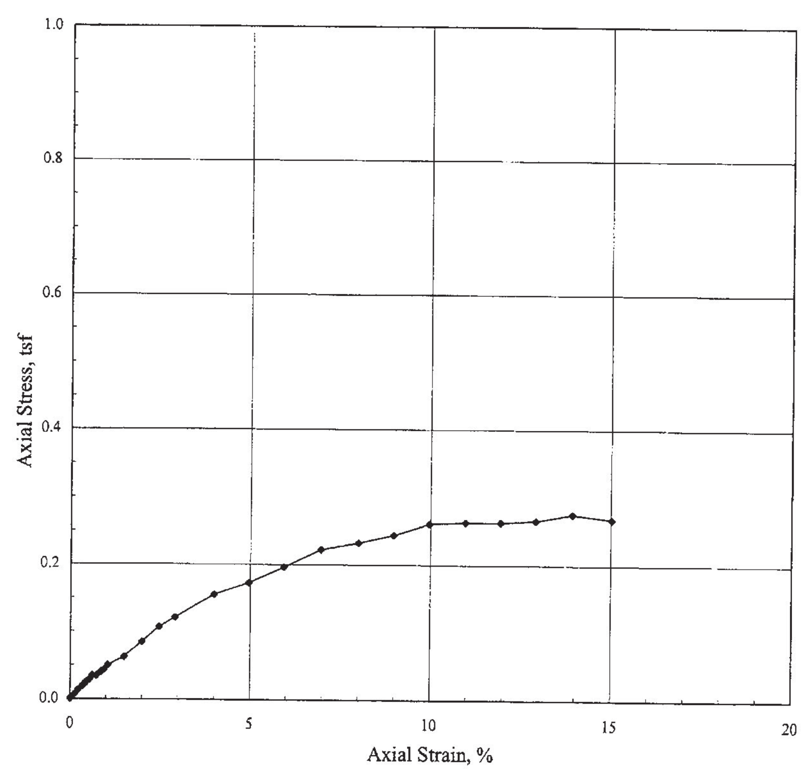
LL, %:	*15*	% < 200:	*NA*
PI, %:	*2*	Gs:	*2.75*

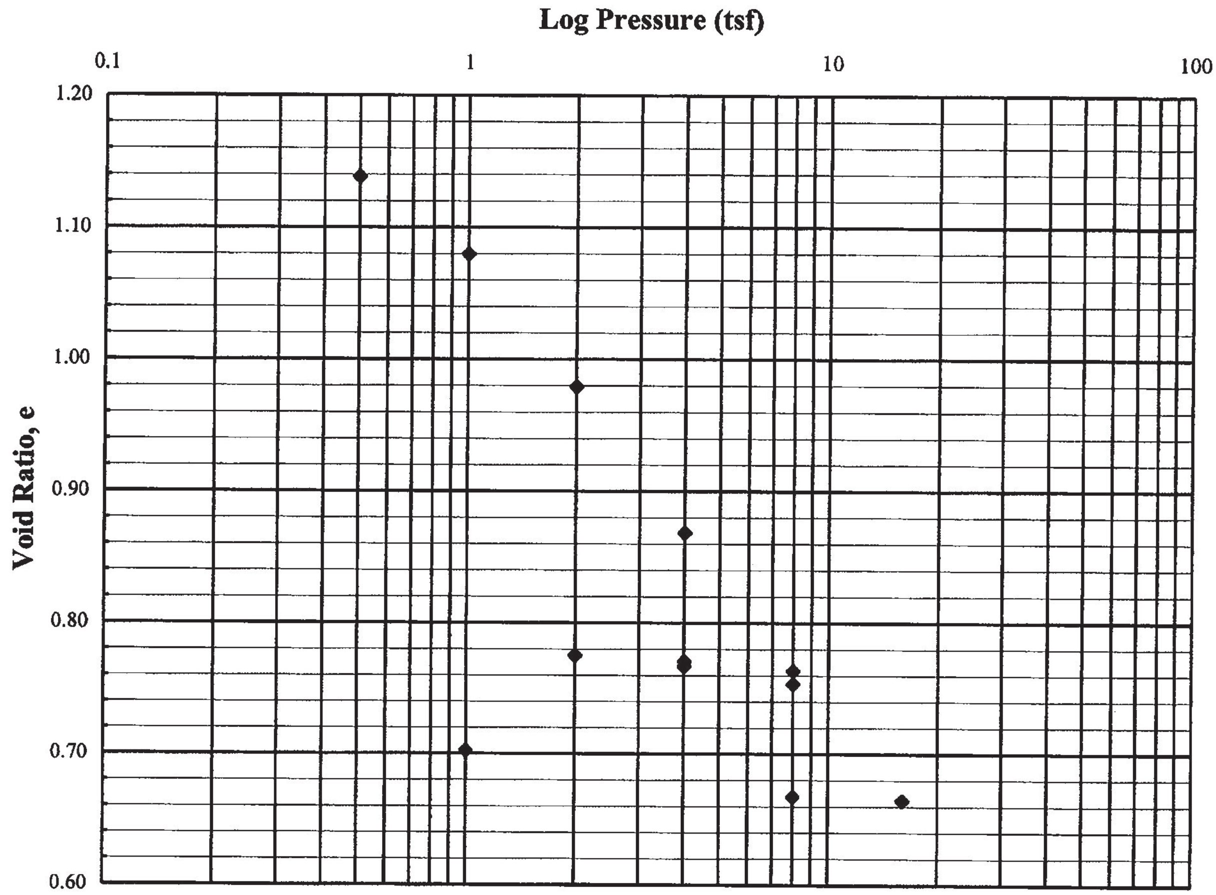
Test Data

Reading No.	Axial Load (lbs)	Axial Displ. (in.)	Axial Strain (%)	Corrected Area[1] (in^2)	Axial Stress (psi)	Axial Stress (tsf)
Zero	0.0	0.000	0.0	6.51	0.0	0.00
1	0.6	0.007	0.1	6.52	0.1	0.01
2	1.2	0.012	0.2	6.52	0.2	0.01
3	1.7	0.019	0.3	6.53	0.3	0.02
4	2.2	0.025	0.4	6.54	0.3	0.02
5	2.6	0.031	0.5	6.55	0.4	0.03
6	3.1	0.035	0.6	6.55	0.5	0.03
7	3.1	0.041	0.7	6.56	0.5	0.03
8	3.6	0.048	0.9	6.56	0.5	0.04
9	3.9	0.053	0.9	6.57	0.6	0.04
10	4.5	0.059	1.0	6.58	0.7	0.05
11	5.7	0.085	1.5	6.61	0.9	0.06
12	7.8	0.113	2.0	6.64	1.2	0.08
13	9.9	0.140	2.5	6.67	1.5	0.11
14	11.3	0.165	2.9	6.71	1.7	0.12
15	14.6	0.226	4.0	6.78	2.2	0.16
16	16.4	0.280	5.0	6.85	2.4	0.17
17	18.8	0.335	5.9	6.92	2.7	0.20
18	21.6	0.393	7.0	7.00	3.1	0.22
19	22.8	0.452	8.0	7.08	3.2	0.23
20	24.2	0.507	9.0	7.15	3.4	0.24
21	26.2	0.563	10.0	7.23	3.6	0.26
22	26.7	0.619	11.0	7.31	3.7	0.26
23	27.0	0.674	12.0	7.39	3.7	0.26
24	27.6	0.729	12.9	7.48	3.7	0.27
25	28.9	0.786	13.9	7.56	3.8	0.28
26	28.5	0.847	15.0	7.66	3.7	0.27

Notes: 1. Right Cylinder Area Correction Method

Axial Stress vs. Axial Strain Plot





Probable Preconsolidation Pressure (Pp), tsf:	*1.20*						
Type of Specimen:	*Tube Sample*						
Description:	*SILTY CLAY (CL-ML), trace sand, contains mica - gray*						
LL:	*26*	**PI:**	*6*	**Gs:**	*2.70*	**P_o' (tsf):**	*0.50*
% < No. 200:	*NA*	**Test Method:**	*ASTM D2435 Method A*				
Test Condition:	*Inundated @ 0.05 tsf*						
Remarks:							

Recompression Index (Cr):		*0.0365*
Compression Index (Cc):		*0.351*
	Initial	**Final**
Water Content, %	*42.4*	*30.0*
Void Ratio	*1.21*	*0.70*
Saturation, %	*94*	*100*
Dry Unit Weight, pcf	*76.1*	*98.9*

Project: Mordecai Island
Geotechnical Laboratory Testing

Location:	*Beach Haven, NJ*		
Boring:	*MIB-3*	**Sample No.:**	*T-1*
Depth:	*18-20 ft.*	**Elevation:**	
Date:	*12/8/05*	**Schnabel No.:**	*04151127.07*

Schnabel
Schnabel Engineering

Consolidation Test Report

Consolidation Test Data Sheet

Consolidometer ID: F

Schnabel Contract: *04151127.07*

Project: *Mordecai Island*
Geotechnical Laboratory Testing

Boring No.: *MIB-3*

Depth: *18-20 ft.*

Test Method: *ASTM D2435 Method A*

Test Condition: *Inundated @ 0.05 tsf*

Initial Height of Specimen (H_o), in.: 0.7504

Height of Solids (H_s), in.: 0.3392

Seating Press. (tsf): 0.05

Initial Dial Gauge Reading (D_o), in.: 0.0032

Initial Apparatus Correction (D_{co}), in.: 0.0000

					A	B	C	D		
Pressure, P (tsf)	TimeReadings Required	Date Load Applied	Time Load Applied	Time Load Effective (min.)	Final[1] Dial Reading, D_{fi} x 10^{-4} in.	Apparatus Correction[2], D_{ci} x 10^{-4} in.	Cumulative Change in Height[3], ΔH_i in.	Height of Voids[4], H_{vi} in.	Vertical Strain[5], ε_i (%)	Void Ratio[6], e_i
0.5	X	16/6/05	8:40		298	16	0.0250	0.3862	3.33	1.138
1	X	10/7/2005	8:40		509	28	0.0449	0.3663	5.98	1.080
2	X	10/8/2005	8:35		860	38	0.0790	0.3322	10.53	0.979
4	X	10/10/2005	8:50		1245	47	0.1166	0.2946	15.54	0.868
8	X	10/11/2005	9:55		1610	56	0.1522	0.2590	20.28	0.763
4		10/12/2005	9:55		1589	47	0.1510	0.2602	20.12	0.767
2		10/13/2005	9:50		1553	38	0.1483	0.2629	19.76	0.775
4	X	10/14/2005	9:05		1577	47	0.1498	0.2614	19.96	0.771
8	X	10/15/2006	9:05		1644	56	0.1556	0.2556	20.73	0.753
16	X	10/17/2005	9:05		1955	65	0.1858	0.2254	24.76	0.664
8		10/18/2005	9:05		1936	56	0.1848	0.2264	24.63	0.667
1		10/19/2005	9:05		1789	28	0.1729	0.2383	23.04	0.702

Notes:
1. "Final" based on test method; 24 hrs for Method A, end of primary for Method B.
2. Correction value, for the current pressure, from the consolidometer's calibration curve.
3. $\Delta H = D_{fi} - D_o - D_{ci} + D_{co}$ = Col. A - D_o - Col. B + D_{co}
4. $H_{vi} = (H_o - H_s) - \Delta H$
5. $\varepsilon_i = (\Delta H / H_o) \times 100$ = (Col. C / H_o) x 100
6. $e_i = H_{vi} / Hs$ = Col. D / Hs

Load Time Readings

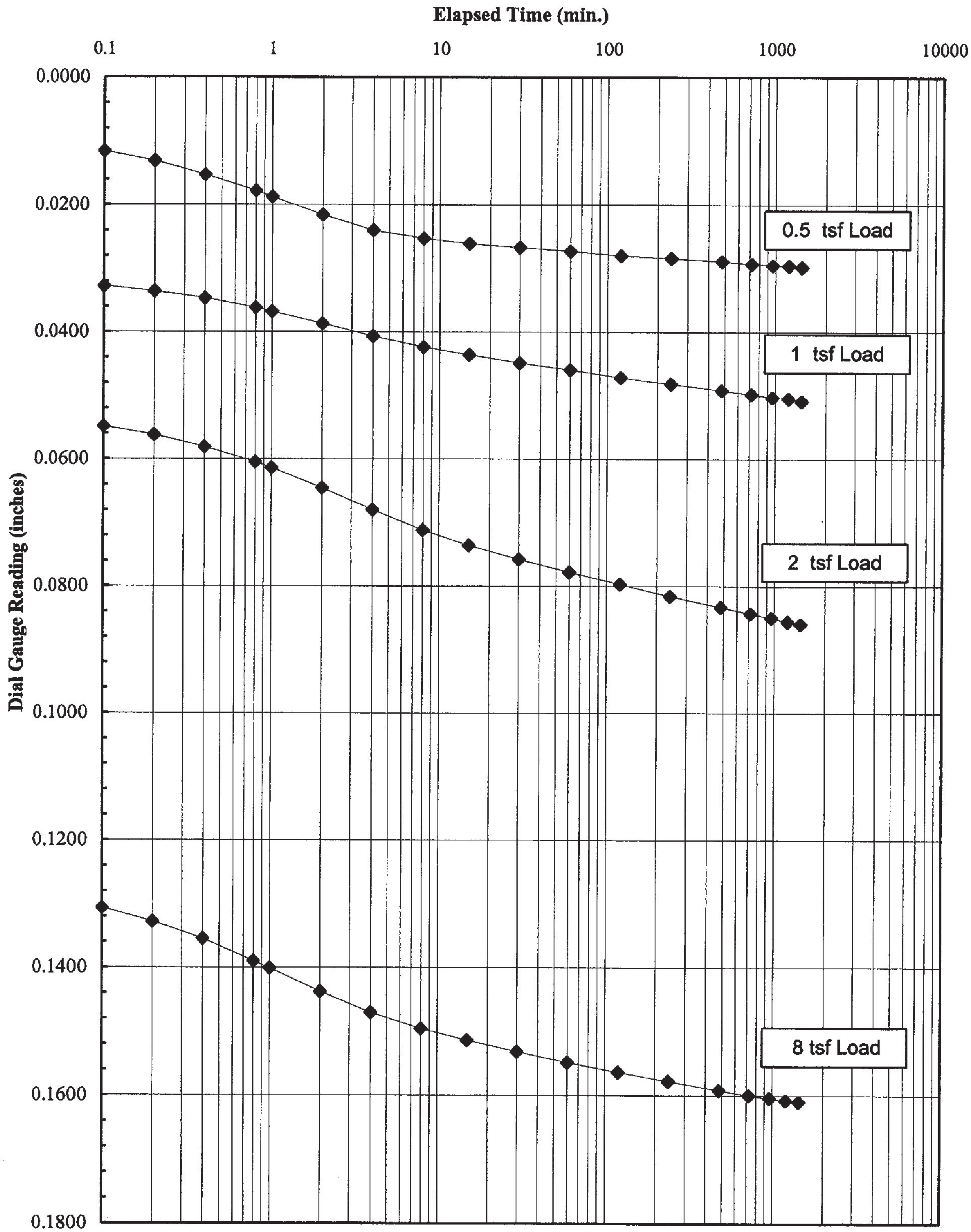
Project: Mordecai Island
Schnabel Contract: 04151127.07
Boring No.: MIB-3 Depth: 18-20 ft.

Consol. ID: F

Elapsed Time (min.)	Dial Guage Readings (inches)					
	0.5 tsf Initial Load 10/6/2005	1 tsf Initial Load 10/7/2005	2 tsf Initial Load 10/8/2005	8 tsf Initial Load 10/11/2005	X tsf Load Date	X tsf Load Date
0.1	0.0116	0.0328	0.0549	0.1307		
0.2	0.0131	0.0336	0.0562	0.1328		
0.4	0.0153	0.0347	0.0581	0.1355		
0.8	0.0178	0.0362	0.0605	0.1390		
1	0.0188	0.0368	0.0614	0.1401		
2	0.0216	0.0387	0.0646	0.1438		
4	0.0240	0.0407	0.0680	0.1471		
8	0.0253	0.0424	0.0712	0.1496		
15	0.0261	0.0436	0.0736	0.1514		
30	0.0267	0.0449	0.0758	0.1532		
60	0.0273	0.0460	0.0778	0.1549		
120	0.0280	0.0472	0.0797	0.1564		
240	0.0284	0.0482	0.0816	0.1578		
480	0.0289	0.0492	0.0833	0.1592		
720	0.0293	0.0498	0.0843	0.1600		
960	0.0295	0.0503	0.0850	0.1604		
1200	0.0296	0.0505	0.0856	0.1608		
1440	0.0298	0.0509	0.0860	0.1610		
1680						
1920						
2160						
2400						
2640						
2880						

Consolidation Time Curves

Project: Mordecai Island
Schnabel Contract: 04151127.07
Boring No.: MIB-3 Depth: 18-20 ft.



Load Time Readings

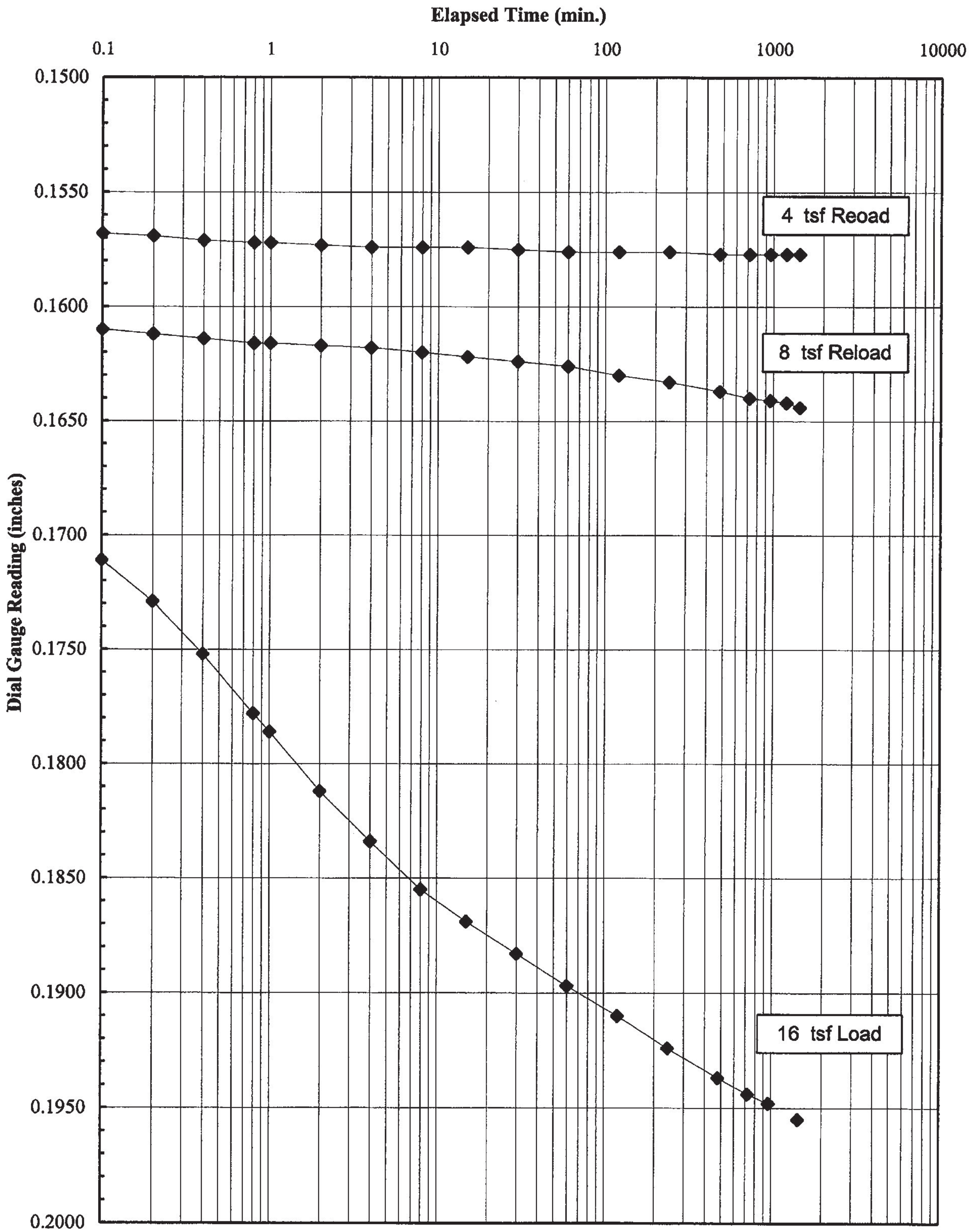
Project: Mordecai Island
Schnabel Contract: 04151127.07
Boring No.: MIB-3 Depth: 18-20 ft.

Consol. ID: F

Elapsed Time (min.)	Dial Guage Readings (inches)					
	4 tsf Reload 10/14/2005	8 tsf Reload 10/15/2005	16 tsf Load 10/17/2005	X tsf Load Date	X tsf Load Date	X tsf Load Date
0.1	0.1568	0.1610	0.1711			
0.2	0.1569	0.1612	0.1729			
0.4	0.1571	0.1614	0.1752			
0.8	0.1572	0.1616	0.1778			
1	0.1572	0.1616	0.1786			
2	0.1573	0.1617	0.1812			
4	0.1574	0.1618	0.1834			
8	0.1574	0.1620	0.1855			
15	0.1574	0.1622	0.1869			
30	0.1575	0.1624	0.1883			
60	0.1576	0.1626	0.1897			
120	0.1576	0.1630	0.1910			
240	0.1576	0.1633	0.1924			
480	0.1577	0.1637	0.1937			
720	0.1577	0.1640	0.1944			
960	0.1577	0.1641	0.1948			
1200	0.1577	0.1642				
1440	0.1577	0.1644	0.1955			
1680						
1920						
2160						
2400						
2640						
2880						

Consolidation Time Curves

Project: Mordecai Island
Schnabel Contract: 04151127.07
Boring No.: MIB-3 Depth: 18-20 ft.



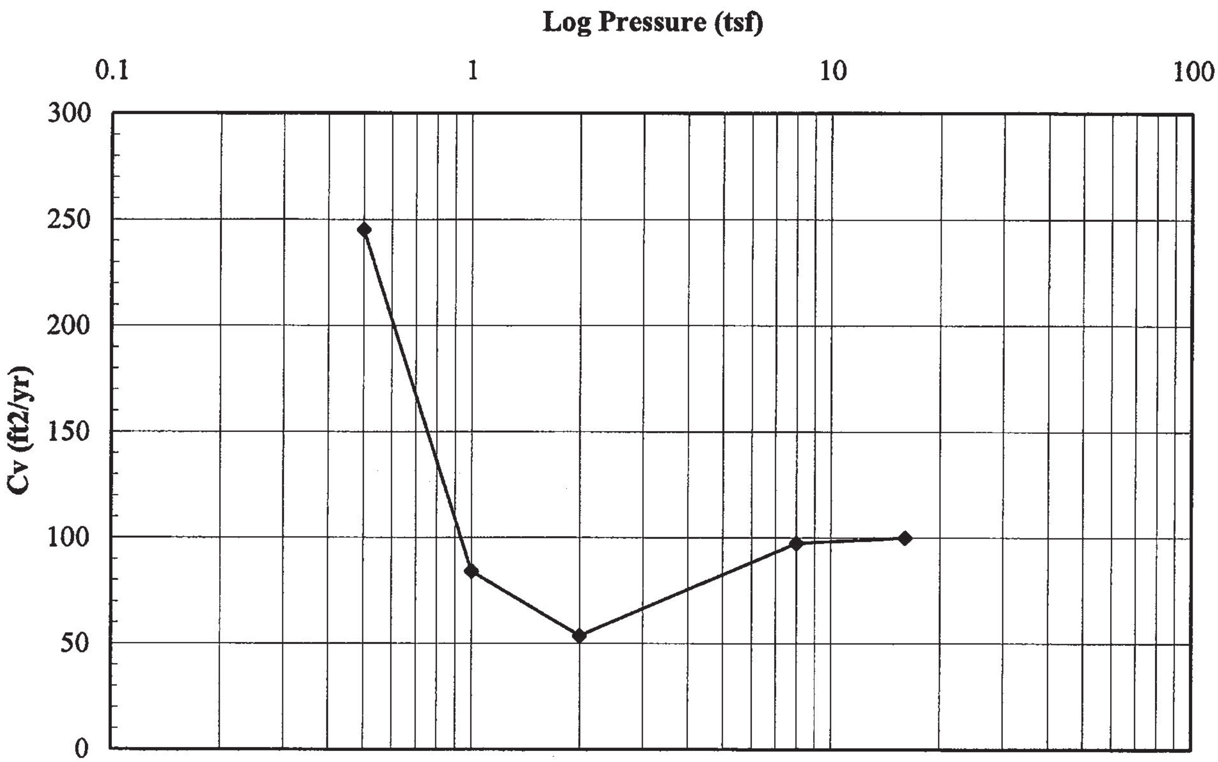
Coefficient of Consolidation (C_v) versus Log Pressure

Schnabel Contract: 04151127.07 Date: 10/21/2005

Project: Mordecai Island
Beach Haven, NJ

Boring: MIB-3 Cv Computation Method: Log Time

Depth, ft: 18-20



Applied Press., tsf	C_v ft^2/yr
0.5	245
1	84
2	54
8	97
16	100

Applied Press., tsf	C_v ft^2/yr

Section A2 – Civil Design

Table of Contents

List of Tables

List of Figures

A2-1) INTRODUCTION

Mordecai Island has experienced a significant loss of saltwater marsh due to factors such as sea level rise and erosion caused by waves and currents. The goal of this feasibility study is to develop a plan to restore habitat on Mordecai Island and slow down the rate of erosion. This Civil Appendix discusses the Civil engineering and design work performed to develop and optimize a Tentatively Selected Plan (TSP).

Erosion Control Structures

Two different erosion control structures were selected for preliminary design, the conventional rubble mound breakwater structure as well as a Wave Attenuation Device (WAD). Breakwaters are coastal engineering structures that are typically constructed parallel to the shoreline and are designed to reduce the wave energy behind the structure, see Figure A2-1. Due to the cost limitations of this project, additional technologies were evaluated, and WADs were determined to be a cost-efficient alternative that could also mitigate erosion. WADs are hollow poured concrete pyramids that are designed to dissipate wave energy while also promoting growth of biota, see Figure A2-2. Water based construction is assumed for both types of structures due to project location and accessibility.

Figure A2-1: Rubble Mound Breakwater Structure at Shooting Island in Ocean City, NJ

Figure A2-2: Wave Attenuation Devices (WADs) at Shark Island in New Iberia, LA

Living Shoreline Solutions, Inc. (LSS) designs and manufactures WADs. NAP hosted a lunch and learn with LSS where they presented several projects demonstrating that WADs not only mitigated against erosion but also naturally accreted sediment behind the structures. LSS also provided documentation and reports for several existing WAD sites. Some potential benefits of the WADs compared to the rubble mound breakwater are; easier installation, easier adaptive management, and higher growth of biota. USACE has less experience designing and implementing WADs compared to rubble mound breakwaters and as a result there is a higher risk associated with WADs. NAP identified only one USACE Mobile District project, Bayou Caddy, where WADs were implemented. The Bayou Caddy Project was part of the Mississippi Coastal Improvements Program after Hurricane Katrina. Unlike the Mordecai Feasibility Study, the Bayou Caddy Project had an authorization that did not require National Economic Development (NED) benefits. The parent contract was with an 8A company. The USACE Mobile District put out a Sources Sought notice for different shoreline technologies, then compared the submittals. Based on NAP phone conversations with the Mobile District, the WADs were successfully implemented at Bayou Caddy and the Mobile District was satisfied with their overall performance. Construction was completed in March of 2017 and a monitoring plan is in draft format. Visual observations show accretion behind the structures and biota growth on the structures. Some settlement has occurred but was in the predicted and expected range. See Attachment A2-4 in Section A2-8 for the Bioengineered Breakwater Technical Report prepared by LSS for USACE Mobile District.

Mordecai Wave Climate

In January 2019, ERDC provided an assessment of wind generated waves for Mordecai Island through their Dredge Operations Technical Support (DOTS) Program. See Attachment A2-2 in Section A2-8 for the full report. The local wind-wave climate was estimated using the STWAVE nearshore model. Ten representative wind conditions were selected. Wave energy was determined to be low at Mordecai Island for the seasonal wind simulations. The wave heights produced during the average conditions and high conditions for spring, summer, fall and winter were all under 1.0 ft. Waves larger than 1.0 ft. were generated by storm conditions of 22.4 mph winds blowing directly at the island from the west over an uninterrupted fetch. The largest wave produced was approximately 1.5 ft. The overall STWAVE nearshore model results show that wind generated waves are relatively small at Mordecai Island.

Since wind generated waves are relatively small, high erosion rates at Mordecai Island are most likely to be caused by boat wakes from the nearby NJ Intracoastal Waterway. Unlike for wind-generated waves, no detailed modeling was done to simulate the potential wave climate from vessels travelling adjacent to Mordecai Island in the NJIWW or surrounding waters. Nor did study resources allow for a high-resolution recreational boating traffic study to be done that could have tracked the number of boats that pass by Mordecai Island on an average day, boat types, speed, and traffic patterns. Instead, an online literature review was done in order to ascertain typical wave heights generated by vessels that are common to the area.

Wakes from boats have been shown to have erosive effects on shorelines located near heavy traffic areas. Wave heights generated from boat traffic are a function of the boat length, hull type, water depth, and boat speed. The best predictor of the size of a boat-generated wave is the speed at which the boat travels (Sorenson, 1973). The maximum boat wake is produced at the point just before it transitions to planing. Several reports from the online literature review were found that related boat size and speed to wave heights.

Table A2-1 is a summary of wave heights from various types of boats and speeds and was taken from a 2017 report entitled. "*Review of Boat Wake Wave Impacts on Shoreline Erosion and Potential Solutions for the Chesapeake Bay*" done by the Scientific and Technical Advisory Committee for Chesapeake Bay Program. The table is originally from a 1973 American Society of Civil Engineers Waterways Harbors and Coastal Engineering journal article entitled "*Water Waves Produced by Ships*" by Sorenson.

Table A2-1: Typical Wave Heights from Various Boat Types and Speeds

Type of Boat	Distance from Sailing Line (ft.)	Speed of Boat (knots)	Maximum Wave Height (ft.)
26 ft. Uniflight (Planing Hull)	330	10	1.33
	330	26	1.00
	490	10	1.25
	490	27	0.75
16 ft. Boston Whaler (Planning Hull)	164	10	0.75
	164	24	0.50
	490	12	0.50
	490	27	0.25
45 ft. Tugboat (Displacement Hull)	98	6	0.75
	98	10	1.50
	490	6	0.20
	490	10	1.00

In addition to the Chesapeake Bay Boat Wake Analysis, another report from NOAA in 2012 entitled "*Boat Wakes and Their Influence on Erosion in the Atlantic Intracoastal Waterway, North Carolina*" was reviewed for applicable information. The analysis was done per request of the Wilmington District of the Corps of Engineers in order to develop a prototype boat wake model that could predict wave conditions and potential seafloor erosion zones and shear stresses at Snow Cut, NC based upon input of a boat hull type, length, speed, and sailing line. Wave data and a detailed boat traffic study was collected in order to test and validate the results from the boat wake model. Two different boat lengths (23 ft. and 53 ft.) at three different speeds (3, 10, and 20 knots) were used for the model based upon typical small and large boats that are common to the area. Maximum boat-generated wave heights varied from 0.25 ft. for the 23 ft. boat travelling at 3 knots to 1.5 ft. for the 53 ft. boat travelling 10 knots.

In summary, the literature review of boat-generated wave heights indicated that for boats common to the Mordecai Island study area, typical maximum wave heights varied from 0.25 ft. to 1.5 ft. Given the results of ERDC's STWAVE wave model of wind-generated waves, boat-generated wave heights can be expected to be at the same level of magnitude or slightly larger than the wind-generated waves impacting Mordecai Island on a day to day basis.

Available Data

USACE NAP collected bathymetry and topography data at Mordecai Island and the adjacent NJ Intracoastal Waterway in January 2019 as seen in Figure A2-3 and A2-4. This was the main data set used for the feasibility study.

Figure A2-3: Bathymetry and Topography Data Points

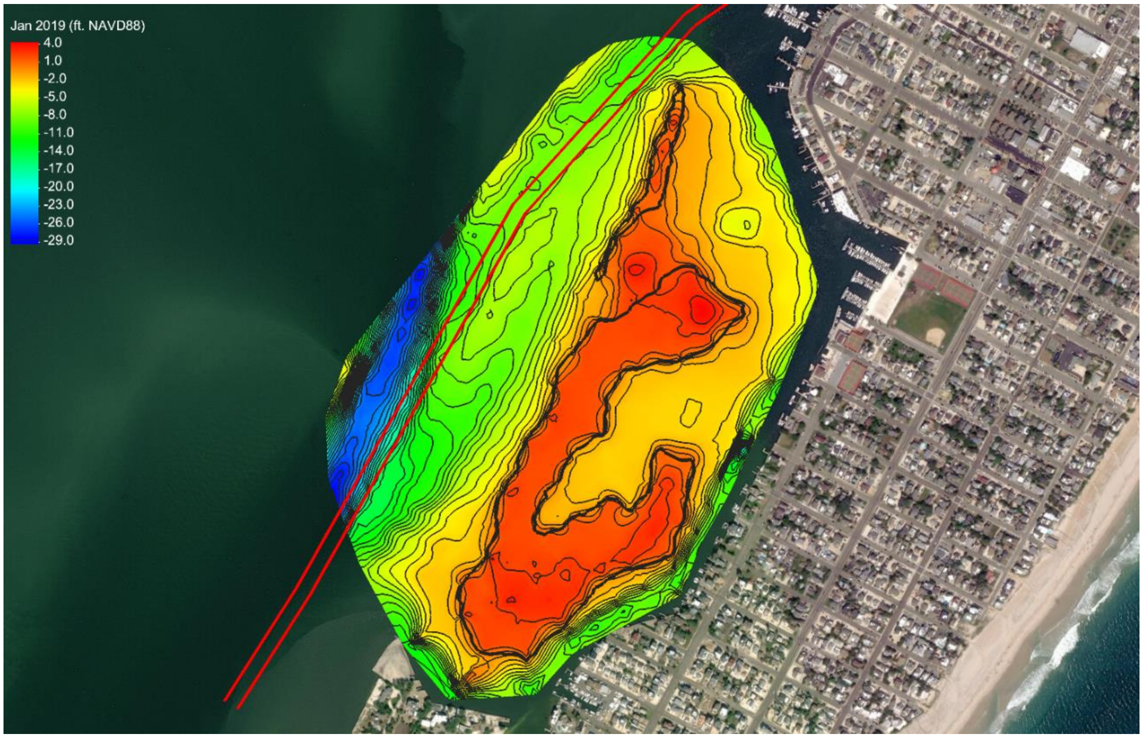


Figure A2-4: Surface Generated from Bathymetry and Topography Data

A2-2) RUBBLE MOUND DESIGN

Rubble Mound Slope

A slope of 2H:1V was chosen as the steepest allowable slope based on both economics and design. Cover layer slopes greater than 1.5H:1V are not recommended by the Corps of Engineers (pg. 7-205, SPM).

Crest Elevation

A conservative crest elevation was selected by adding wave runup to the Mean Higher High Water (MHHW) level. The largest wave height from both the STWAVE nearshore model and the boat wake literature review was 1.5 ft. and was selected to calculate wave runup. Relatively conservative assumptions went into the wave runup calculations and some structure overtopping is allowable since the project is for ecosystem restoration and not coastal storm risk management. The 1.5 ft. wave resulted in 1.2 ft. of wave runup. See Attachment A2-3 in Section A2-8 for wave runup calculations. Understanding that some overtopping is allowable, a crest elevation was selected at +2.6 ft NAVD88 using the following inputs:

Crest Elevation = +1.35 ft. NAVD88 (MHHW) + 1.2 ft. (wave runup) = +2.6 ft. NAVD88

A settlement analysis on the rubble mound structure provided a 5” to 8” range in settlement based on the available geotechnical data. In the absence of a complete geotechnical investigation, 1 ft. was selected for settlement and was added to the crest elevation as overbuild to calculate a conservative stone quantity. Settlement/overbuild calculations will be evaluated in more detail in the subsequent phases of design when a complete geotechnical investigation takes place. A geogrid composite is assumed to be placed underneath the rubble mound structure to help mitigate differential settlement and the migration of sand into the stone layer. See Section A1 – Geotechnical for settlement details and calculations.

Rubble Mound Stone Size

Eq. 7-117 from the SPM (Hudson and Jackson, 1962) was used to determine graded riprap armor stone unit weights that would be stable under a range of wave heights. This equation is intended for conditions when the crest of the structure is high enough to prevent major overtopping (some overtopping is allowable).

$$W_{50} = \frac{w_{rock}\ x\ H^3}{K_D\ (\frac{w_{rock}}{\gamma_w} - 1)^3\ cot\ \theta}$$

Due to the absence of site specific boat wake data, a 1 to 3 ft wave height range was used to evaluate stone size/weight. Relatively small stone is required to be stable against this wave range (D_{50} = 0.3 to 1 ft). To be conservative, a 3 ft wave height was selected to size the stone. An increase in stone size has a minimal effect to cost while increasing overall structure stability.

W_{rock} = 180 pcf (assumed unit weight of rock is comparable to past coastal projects in the area)

$K_{D\,=}$ 2.0 (stability coefficient, Table 7-8, SPM, breaking wave, 2 units cover layer thickness)

γ_w = 64 pcf (salt water not fresh water)

Cot θ = 2.0 (assume side slope 2H:1V)

H = 3 ft

$$W_{50} = \frac{180\ pcf\ x\ 3\ ft^3}{2.0\ (\frac{180\ pcf}{64.0\ pcf} - 1)^3\ 2.0} = 204\ lbs$$

Approximate Stone Diameter (D_{50}) :

$$\frac{204\ lbs}{1} * \frac{1\ cubic\ ft}{180\ pounds} = 1.13\ cubic\ ft$$

$$\sqrt[3]{1.13} = 1.04\ ft$$

The following are R-6 and R-7 Penn DOT riprap gradations:

Table A2-2: Penn DOT Riprap Gradations

Percent Passing		
Rock Size (Inches)	R-7	R-6
30	100*	
24		100*
18	15-50	
12	0-15	15-50
6		0-15
*Maximum Allowable Rock Size		

R-6 Riprap aligns with a D_{50} of 1 ft. and was therefore selected as the stone gradation for the rubble mound structure.

Crest Width

The following equation from the Shore Protection Manual was used to determine the rubble mound crest width:

$$B = n\, k_{\Delta} \left(\frac{w_{50}}{w_r}\right)^{\frac{1}{3}} \quad \text{(Eq. 7-120, SPM)}$$

Based on Eq. 7-120, a 3 ft. wave yields a 3 ft. crest width. The minimum crest width should equal the combined width of three armor units. The width of the crest also depends on the degree of allowable overtopping (pg. 7-233 SPM). Based on the cost limitations of the project and the fact that the project is ecosystem restoration and not coastal storm risk management, a 3 ft crest width was selected for the rubble mound structure (some overtopping is allowable).

Sill Vents

There are two types of breakwaters; gapped and continuous. Based upon historic erosion and accretion rates offshore Mordecai Island, there may not be enough sediment in the system to naturally accrete behind gapped breakwaters. A continuous breakwater was selected for the development of alternative plans due to a greater potential to protect existing and/or placed material behind the structure. Some water can transport through the breakwater voids. However, this transport may not be enough to promote water quality behind the structure. Sill vents, or lower sections of breakwater, were designed into the structure to promote intertidal flushing in order to maintain water quality. The sill vents are approximately 40 ft. long and have a crest elevation at the mean low water line to allow water to flow through the breakwater during the entire tide cycle. There is approximately 160 linear ft. of breakwater between each sill vent. The northern tip of Mordecai Island is the most vulnerable area to waves due to its proximity to the NJ Intracoastal Waterway. For this reason, there are no sill vents in the northern tip of the breakwater. The length and spacing of sill vents will be optimized in the next phase of the study.

A2-3) WAD DESIGN

For comparison purposes, WAD elevations were selected to be the same as the rubble mound crest elevations (+2.6 ft. NAVD88). LSS also recommended approximately 1 ft of overbuild for potential settlement which aligns with the USACE assumption of 1 ft of overbuild for the rubble mound breakwater. This overbuild was accounted for in the quantities and cost estimate.

In the absence of a geotechnical investigation and in order to be consistent with rubble mound breakwater assumptions, a geogrid composite was assumed to be placed underneath the WADs. The geogrid composite was also recommended to USACE by LSS.

The WADs are assumed to be aligned in a double row to more effectively attenuate wave energy. This is consistent with the Mobile District Bayou Caddy project and was recommended by LSS. Water can move between each wave attenuation structure and through the six triangular openings located on each WAD face. Since water quality is not an issue with the WADs, sill vents and gaps between the structures were not included into the design.

Three separate structure locations (Alignment A, B, and C) were evaluated for both rubble mound and WAD structures and are described in greater detail in Section A2-5. The three alignments have three different average depths resulting in three different WAD dimensions to reach the targeted crest elevation. Dimensions were scaled off existing WAD structures as well as direct input from LSS (see Table A2-3).

Table A2-3: WAD Dimensions

WAD Dimensions		
Alignment	Height (ft.)	Base Length (ft.)
A	7.6	11.5
B	7.1	11.25
C	6.6	11

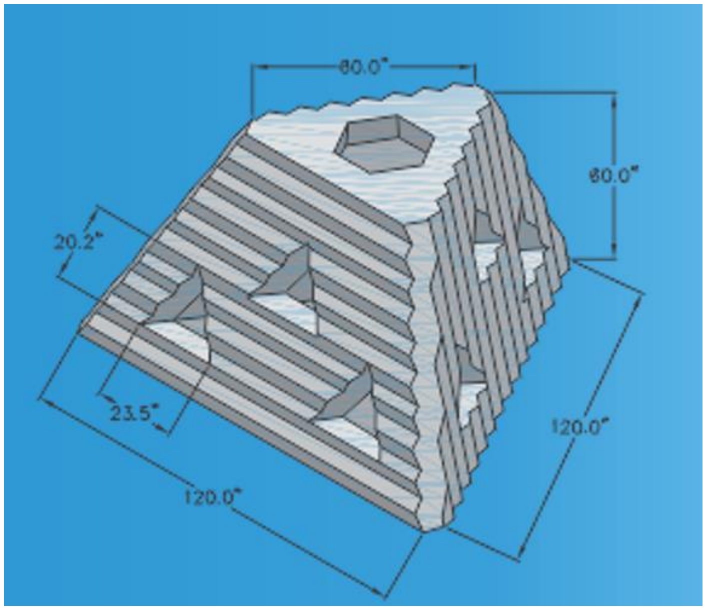


Figure A2-5: LSS WAD Design for Sunken Island, FL

A2-4) WARNING SIGNS

Warning signs with lights were assumed to be positioned approximately every 400 linear ft. of structure (rubble mound or WAD). Vessels traversing past Mordecai Island will be able to detect these signs and avoid any structures that may be submerged or hard to see. The signs reduce the overall risk of damage to vessels as well as potential damage to the structures. The warning signs are comprised of 24” by 36” reflectorized signs, Sealite 3-5NM Solar Marine Lanterns, and Sealite 1,500mm diameter navigation buoys. The signs are anchored with a chain and Sealite concrete mooring sinker.

Figure A2-6: Sealite Navigation Buoy with a Sealite Solar Marine Lantern

Figure A2-7: Sealite Concrete Mooring Sinker

A2-5) DESIGN DRAWINGS

See Attachment A2-1 in Section A2-8 for design drawings of the various alternatives (both rubble mound breakwater and WADs) investigated during this feasibility study. Note Alternative 1 is not shown in the design drawings because this is the no action plan. See sheets C-101 through C-106 for plans views and sheets C-301 through C-306 for typical section. Structures were considered at three different depths providing similar levels of erosion protection. Alignment A generally follows the 1977 shoreline and has the greatest depth which on average is approximately -4 ft NAVD88. Alignments B and C are approximately 25 ft apart in successively shallower water toward the western shore of the island. The depth of Alignment B and C on average is approximately -3.5 ft NAVD88 and -3.0’ NAVD88, respectively, and converge at the northern tip of the island as they draw closer to the channel. The northern half of Alignment C follows the same layout as the northern half of Alignment B, since a landward offset of Alignment C in this area would place the structure on the island and act as a shoreline slope revetment, a measure which was screened out prior to alternative plan formulation.

A2-6) QUANTITIES

Table A2-4: Riprap Quantities

Riprap Quantities			
Alignment	CY	CF	Tons
A	12970	350190	19085
B	10210	275670	15024
C	8780	237060	12920

Table A2-5: Geo-Composite Quantities for Rubble Mound Breakwater

Geo-composite Quantities for Rubble Mound Breakwater		
Alignment	Area (SF)[1]	Area (SY)[1]
A	127868	14210
B	112899	12540
C	105205	11690

Table A2-6: Geo-Composite Quantities for WADs

Geo-Composite Quantities for WADs		
Alignment	Area (SF)[1]	Area (SY)[1]
A	100486	11170
B	94125	10460
C	87619	9740

<u>Quantity Notes:</u>

1. *Geo-composite area assumes a 2 ft. offset on both sides of the structure plus an additional 25% for overlapping*
2. *Stone quantities were calculated directly from AutoCAD Civil 2019 by subtracting the existing ground surface from proposed riprap stone surfaces*
3. *Alignments A, B, and C on average follow the existing -4 ft, -3.5 ft, and -3 ft contours, respectively*
4. *Unit weight of riprap is assumed to be 109 pcf*

A2-7) REFERNCES

U.S. Army Corps of Engineers. Shore Protection Manual (3 Volumes). Coastal Engineering Research Center, (1973, 1977, 1983).

Bilkovic, D., M. Mitchell, J. Davis, E. Andrews, A. King, P. Mason, J. Herman, N. Tahvildari, J. Davis. 2017. Review of boat wake wave impacts on shoreline erosion and potential solutions for the Chesapeake Bay. STAC Publication Number 17-002, Edgewater, MD. 68p.

Sorensen, R. M. 1973. Water Waves Produced by Ships. Journal of Waterways, Harbors & Coastal Engineering, American Society of Civil Engineers. Issue Number WW2, New York, NY. 245p

Fonseca, M.S. and Malhotra, A. 2012 . Boat wakes and their influence on erosion in the Atlantic Intracoastal Waterway, North Carolina. NOAA Technical Memorandum NOS NCCOS # 143. 24p.

A2-8) ATTACHMENTS

Attachment A2-1: Design Drawings

Attachment A2-2: DOTS Report

Attachment A2-3: Wave Runup Calculations

Attachment A2-4: Bioengineered Breakwater Technical Report

ATTACHMENT A2-1 – DESIGN DRAWINGS

SHEET NUMBER
C-101

MORDECAI ISLAND
BEACH HAVEN, NJ

ECOSYSTEM RESTORATION FEASIBILITY STUDY AND INTEGRATED ENVIRONMENTAL ASSESSMENT

PLAN VIEW
RUBBLE MOUND BREAKWATER - ALIGNMENT A

DWN BY: SJW	DES BY: SJW	PROJ. DATE:
CKD BY:	REVIEWED BY:	SOLICITATION NUMBER:
US Army Corps of Engineers® Philadelphia District		CONTRACT NUMBER:
		PROJ. NUMBER:

SHEET NUMBER

C-102

MORDECAI ISLAND
BEACH HAVEN, NJ

ECOSYSTEM RESTORATION FEASIBILITY STUDY AND INTEGRATED ENVIRONMENTAL ASSESSMENT

PLAN VIEW
WAVE ATTENUATION STRUCTURE - ALIGNMENT A

DWN BY: SJW	DES BY: SJW	PROJ. DATE:
CKD BY:	REVIEWED BY:	SOLICITATION NUMBER:
		CONTRACT NUMBER:
		PROJ. NUMBER:

US Army Corps of Engineers ®
Philadelphia District

SHEET NUMBER

C-103

MORDECAI ISLAND
BEACH HAVEN, NJ

ECOSYSTEM RESTORATION FEASIBILITY STUDY AND INTEGRATED ENVIRONMENTAL ASSESSMENT

PLAN VIEW
RUBBLE MOUND BREAKWATER - ALIGNMENT B

DWN BY: SJW	DES BY: SJW	PROJ. DATE:
CKD BY:	REVIEWED BY:	SOLICITATION NUMBER:
		CONTRACT NUMBER:
		PROJ. NUMBER:

US Army Corps of Engineers ® Philadelphia District

SHEET NUMBER

C-104

MORDECAI ISLAND
BEACH HAVEN, NJ

ECOSYSTEM RESTORATION FEASIBILITY STUDY AND INTEGRATED ENVIRONMENTAL ASSESSMENT

PLAN VIEW
WAVE ATTENUATION STRUCTURE - ALIGNMENT B

DWN BY: SJW	DES BY: SJW	PROJ. DATE:
CKD BY:	REVIEWED BY:	SOLICITATION NUMBER:
		CONTRACT NUMBER:
		PROJ. NUMBER:

US Army Corps of Engineers® Philadelphia District

SHEET NUMBER

C-105

MORDECAI ISLAND
BEACH HAVEN, NJ

ECOSYSTEM RESTORATION FEASIBILITY STUDY AND INTEGRATED ENVIRONMENTAL ASSESSMENT

PLAN VIEW
RUBBLE MOUND BREAKWATER - ALIGNMENT C

DWN BY: SJW	DES BY: SJW	PROJ. DATE:
CKD BY:	REVIEWED BY:	SOLICITATION NUMBER:
		CONTRACT NUMBER:
		PROJ. NUMBER:

US Army Corps of Engineers ®
Philadelphia District

SHEET NUMBER
C-106

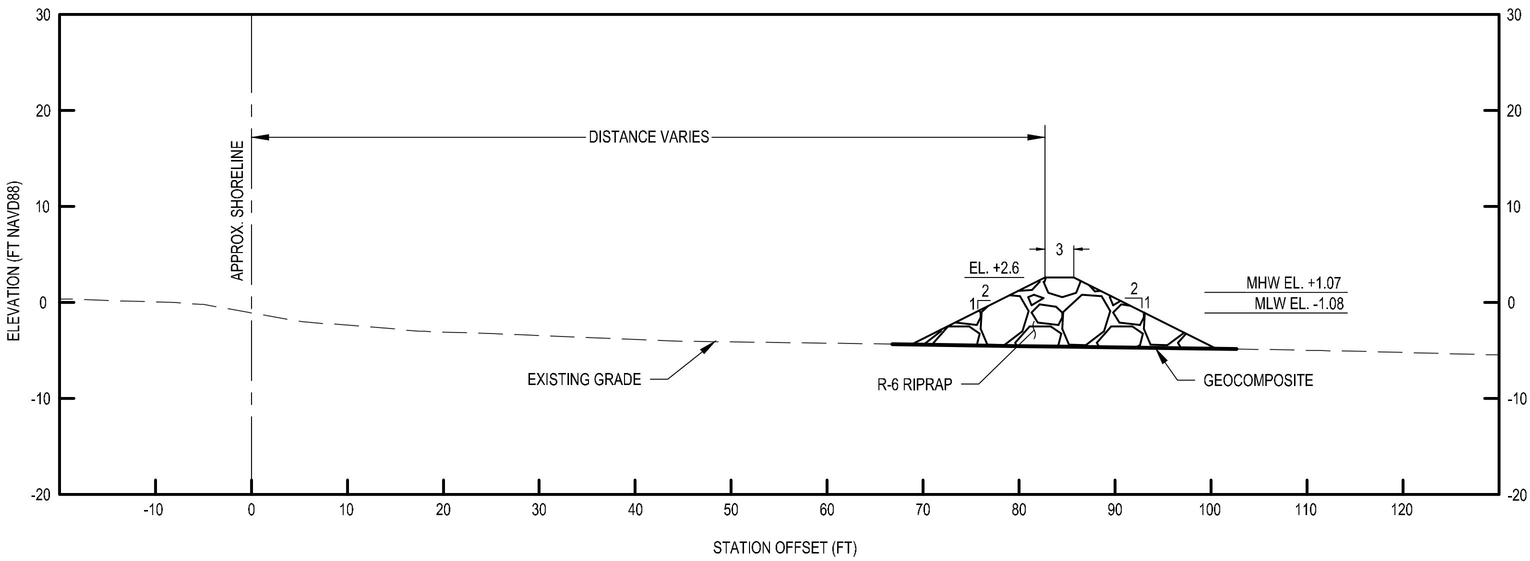
MORDECAI ISLAND
BEACH HAVEN, NJ

ECOSYSTEM RESTORATION FEASIBILITY STUDY AND INTEGRATED ENVIRONMENTAL ASSESSMENT

PLAN VIEW
WAVE ATTENUATION STRUCTURE - ALIGNMENT C

DWN BY: SJW	DES BY: SJW	PROJ. DATE:
CKD BY:	REVIEWED BY:	SOLICITATION NUMBER:
		CONTRACT NUMBER:
		PROJ. NUMBER:

US Army Corps of Engineers ® Philadelphia District



SHEET NUMBER
C-301

MORDECAI ISLAND
BEACH HAVEN, NJ

ECOSYSTEM RESTORATION FEASIBILITY STUDY AND INTEGRATED ENVIRONMENTAL ASSESSMENT

TYPICAL SECTION
RUBBLE MOUND BREAKWATER - ALIGNMENT A

DWN BY: SJW	DES BY: SJW	PROJ. DATE:
CKD BY:	REVIEWED BY:	SOLICITATION NUMBER:
US Army Corps of Engineers® Philadelphia District		CONTRACT NUMBER:
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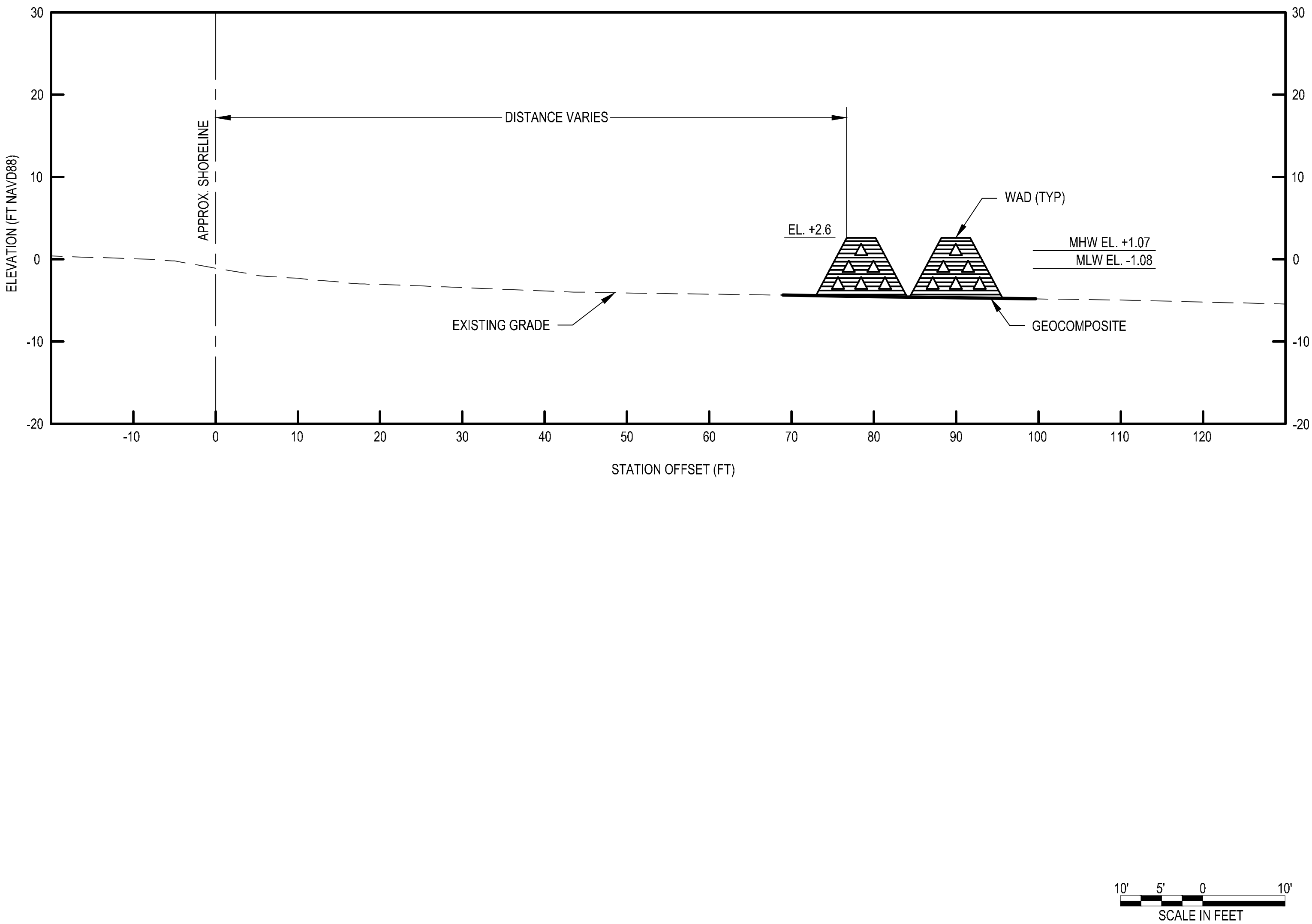


SHEET NUMBER

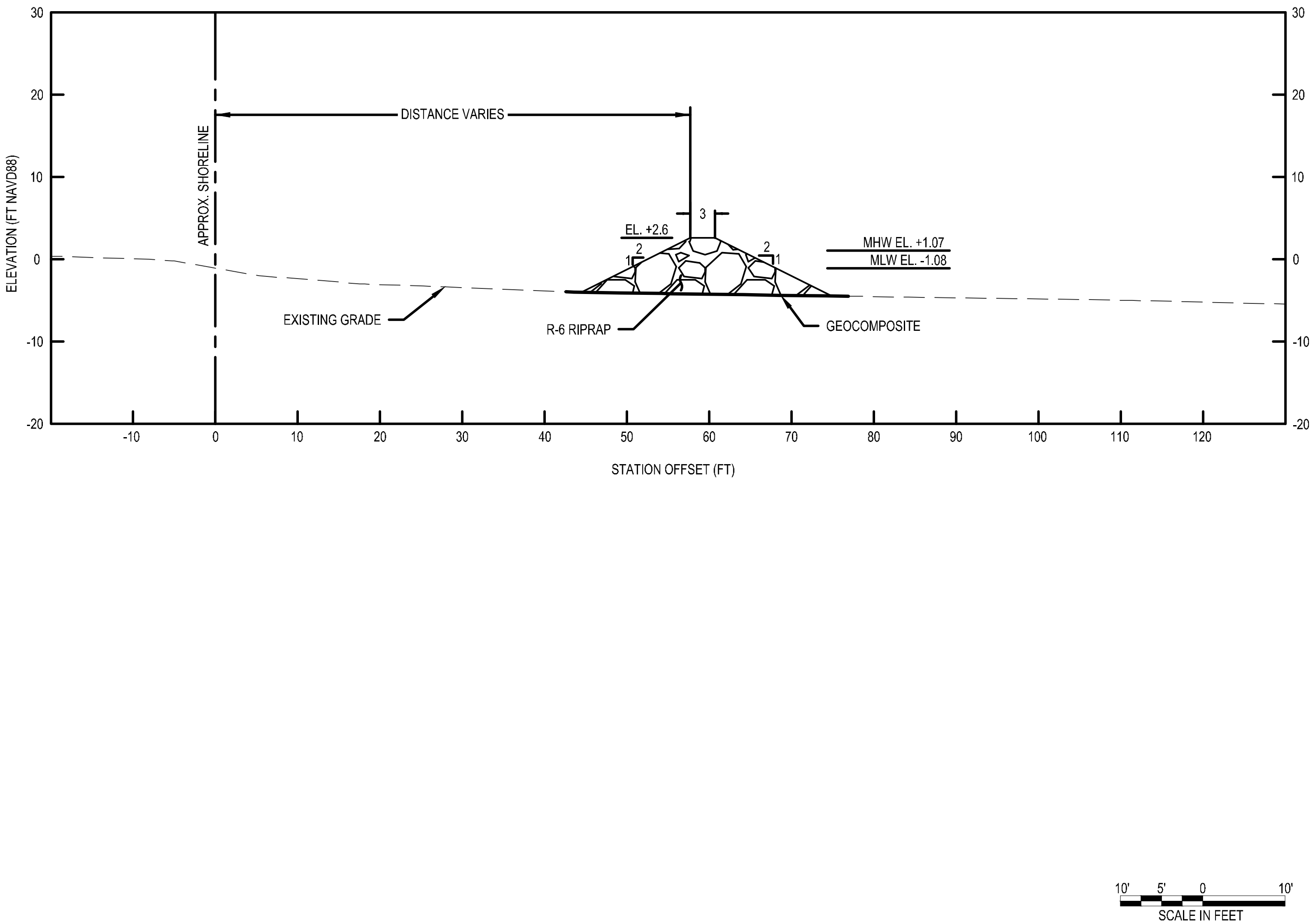
C-302

MORDECAI ISLAND
BEACH HAVEN, NJ

ECOSYSTEM RESTORATION FEASIBILITY STUDY AND INTEGRATED ENVIRONMENTAL ASSESSMENT

TYPICAL SECTION
WAVE ATTENUATION STRUCTURE - ALIGNMENT A

DWN BY: SJW	DES BY: SJW	PROJ. DATE:
CKD BY:	REVIEWED BY:	SOLICITATION NUMBER:
US Army Corps of Engineers® Philadelphia District		CONTRACT NUMBER:
		PROJ. NUMBER:



SHEET NUMBER

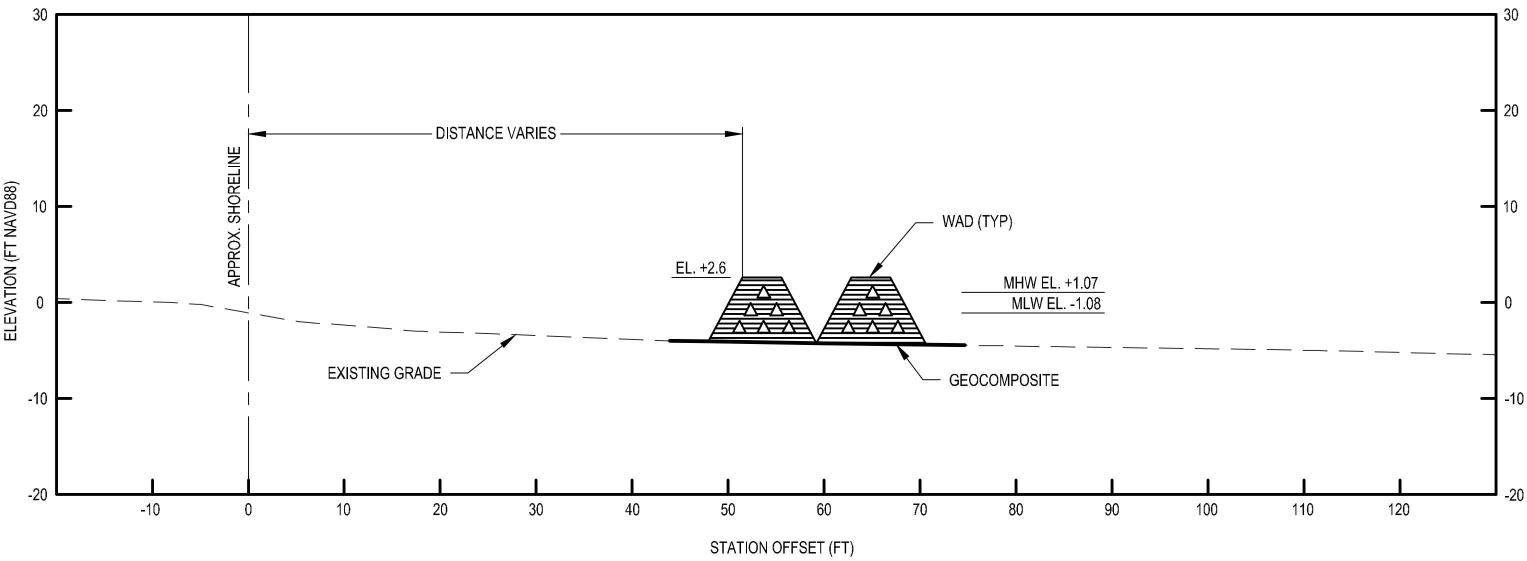
C-303

MORDECAI ISLAND
BEACH HAVEN, NJ

ECOSYSTEM RESTORATION FEASIBILITY STUDY AND INTEGRATED ENVIRONMENTAL ASSESSMENT

TYPICAL SECTION
RUBBLE MOUND BREAKWATER - ALIGNMENT B

DWN BY: SJW	DES BY: SJW	PROJ. DATE:
CKD BY:	REVIEWED BY:	SOLICITATION NUMBER:
US Army Corps of Engineers® Philadelphia District		CONTRACT NUMBER:
		PROJ. NUMBER:



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SCALE IN FEET

SHEET NUMBER
C-304

MORDECAI ISLAND
BEACH HAVEN, NJ

ECOSYSTEM RESTORATION FEASIBILITY STUDY AND INTEGRATED ENVIRONMENTAL ASSESSMENT

TYPICAL SECTION
WAVE ATTENUATION STRUCTURE - ALIGNMENT B

DWN BY: SJW	DES BY: SJW	PROJ. DATE:
CKD BY:	REVIEWED BY:	SOLICITATION NUMBER:
		CONTRACT NUMBER:
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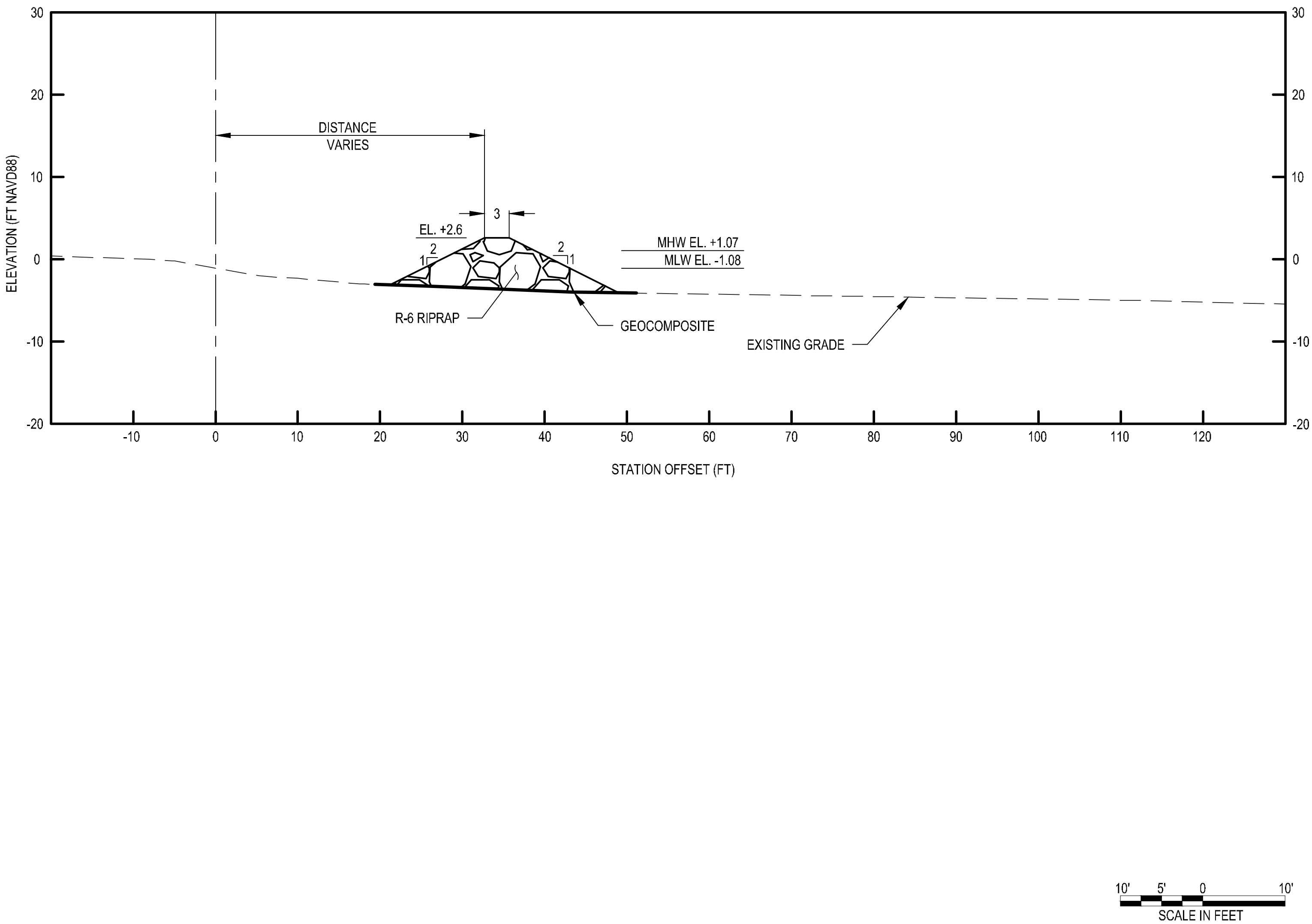
US Army Corps of Engineers ®
Philadelphia District



MORDECAI ISLAND
BEACH HAVEN, NJ

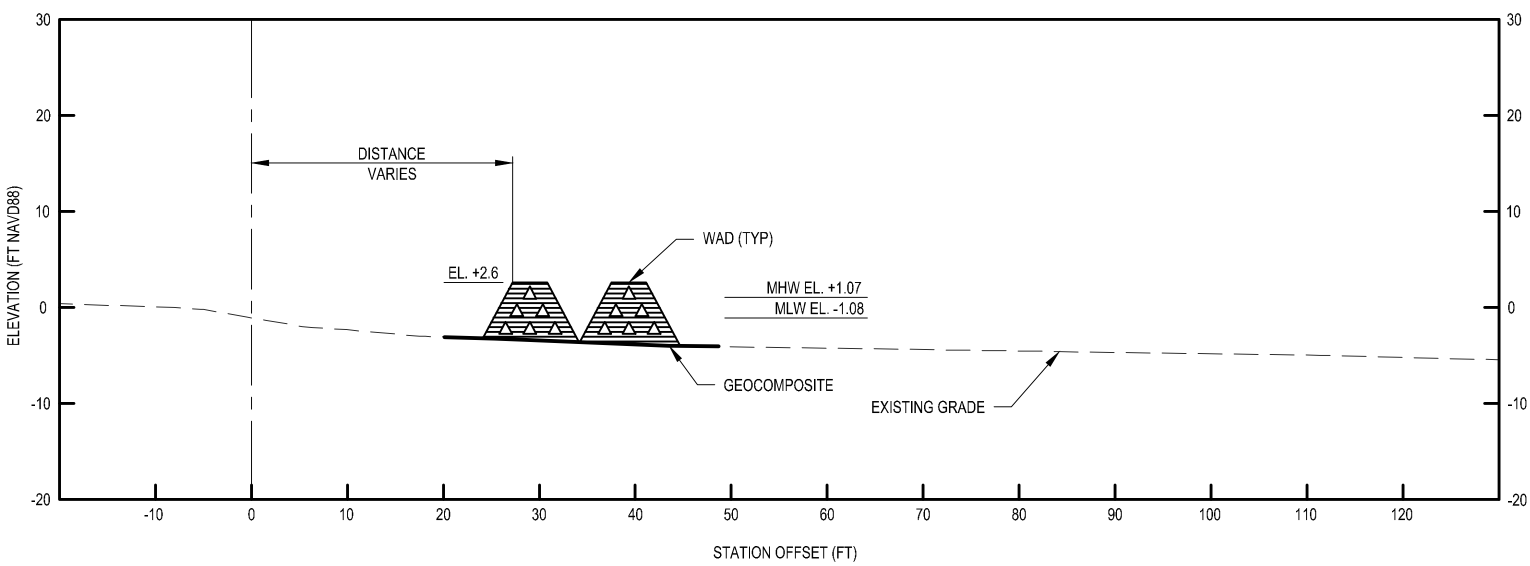
ECOSYSTEM RESTORATION FEASIBILITY STUDY AND INTEGRATED ENVIRONMENTAL ASSESSMENT

TYPICAL SECTION
RUBBLE MOUND STRUCTURE - ALIGNMENT C

SHEET NUMBER
C-305

DWN BY: SJW	DES BY: SJW	PROJ. DATE:
CKD BY:	REVIEWED BY:	SOLICITATION NUMBER:
		CONTRACT NUMBER:
		PROJ. NUMBER:

US Army Corps of Engineers® Philadelphia District



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SCALE IN FEET

SHEET NUMBER
C-306

MORDECAI ISLAND
BEACH HAVEN, NJ

ECOSYSTEM RESTORATION FEASIBILITY STUDY AND INTEGRATED ENVIRONMENTAL ASSESSMENT

TYPICAL SECTION
WAVE ATTENUATION STRUCTURE - ALIGNMENT C

DWN BY: SJW	DES BY: SJW	PROJ. DATE:
CKD BY:	REVIEWED BY:	SOLICITATION NUMBER:
		CONTRACT NUMBER:
		PROJ. NUMBER:

US Army Corps of Engineers ® Philadelphia District

ATTACHMENT A2-2 – DOTS REPORT

November 2018

DOTS: Assessment of Wind-Waves for the Mordecai Island, New Jersey Vicinity

by Mary Anderson Bryant and Brandon Boyd

INTRODUCTION: The USACE Philadelphia District (NAP) requested assistance in conducting a wave assessment for the Mordecai Island, New Jersey area. Mordecai Island is located west of Long Beach Island near Beach Haven Borough, New Jersey and adjacent to the New Jersey Intracoastal Waterway (NJIWW). Over the last five years, the Engineer Research and Development Center's Coastal and Hydraulics Laboratory and the Environmental Laboratory have assisted NAP with various aspects of the Mordecai Island Ecosystem Restoration Feasibility Study as well as the innovative placement/island creation project using NJIWW dredged material.

The entire coastline of Mordecai Island has suffered from erosion; however, the western edge, adjacent to the Federal NJIWW navigation channel, has receded at a more substantial rate on the order of 3 - 6 feet (ft) per year. Over the past 100 years, half the island has been lost through erosion. The primary causes of the significant and continuous erosion along the western shoreline of Mordecai Island are hypothesized to be waves from the bay and wakes from vessels using the adjacent Intracoastal Waterway. A better understanding of the local wave environment will help NAP evaluate features for reducing wave energy to protect this now critical environment and manage beneficial use placement areas.

MODEL SETUP: The STeady-state WAVE (STWAVE) model (Massey et al. 2011), which is a phase-averaged spectral model for wave generation, propagation and transformation, was used to estimate the local wind-wave climate in the vicinity of Mordecai Island. In order to capture the wind fetch lengths to which Mordecai Island is exposed, a STWAVE grid was developed to encompass the entire southern complex of Little Egg Harbor. The Cartesian grid resolution is 45 m and is comprised of 178 cells in the cross-shore direction (I) and 309 cells in the alongshore direction (J). The projection of the grid is Universal Transverse Mercator (UTM) Zone 18 with a vertical datum relative to NAVD88. The properties of the STWAVE domain are provided in Table 1.

Table 1. STWAVE Grid Properties

Horizontal Projection	Vertical Projection	Grid Origin (x,y) [m]	Azimuth [deg]	$\Delta x/\Delta y$ [m]	Number of Cells	
					I	J
UTM 18	NAVD88	(569753.97, 4387037.67)	148.6	45.0	178	309

The topography and bathymetry data to populate the STWAVE domain was obtained from two sources, the 2015 USGS CoNED Topobathymetric Model (1888-2014) and the 2017 USACE NCMP Topobathy Lidar DEM. The USGS CoNED model was resampled to a 10-m resolution and served as the base elevation data because of its comprehensive coverage of the entire model domain. Bathymetry and topography, including that of Mordecai Island, were then updated with the resampled 5-m USACE NCMP Lidar DEM. The STWAVE domain and inset of Mordecai Island, including savepoints along its western edge, is shown overlaid on aerial imagery in Figure 1.

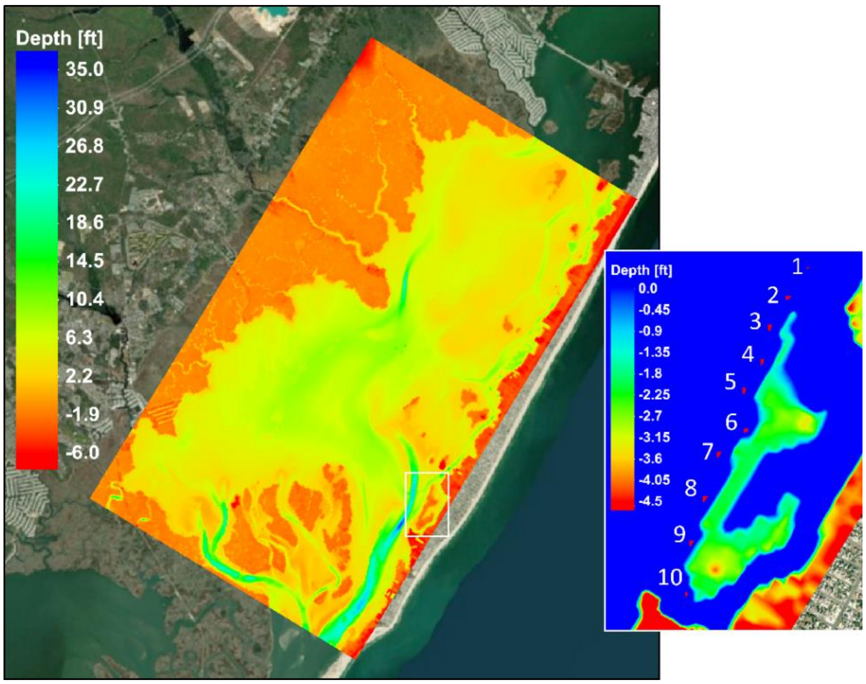


Figure 1. STWAVE domain and savepoints around Mordecai Island.

Simulations were conducted using the full-plane mode of STWAVE to allow for wave generation and transformation in a 360-degree plane. The resolved spectra was represented by 54 frequency bands, ranging from 0.2 Hz (5.0 s) to 1.0 Hz (1.0 s), and 72 angle bands, from an angle of 0 degrees to 355 degrees with respect to the x-axis. Frequency and angular resolution were 0.015 Hz and 5 degrees, respectively. Ten STWAVE savepoints were defined in order to provide estimates of zero-moment wave height (H_{m0}), peak period (T_p), and mean wave direction along the western side of Mordecai Islands. Note that STWAVE is a phase-averaged model and is not appropriate to simulate vessel wakes; one way to estimate the waves due to vessel traffic is using a phase-resolving model, such as FUNWAVE.

Wind direction and speed were obtain from the NOAA Jacques Cousteau National Estuarine Research Reserve located 11.4 miles west of Mordecai Island (Station JACNCMET, 39° 32.1'N, 74° 27.8'W; retrieved 15 Oct 2018 from http://cdmo.baruch.sc.edu/). The record used consists of 15 minute wind speed and direction from 1 Oct 2002 to 15 Oct 2018. Winds were subset by seasons for purposes of comparison, e.g. winter is December, January, and February. Wind roses showing the frequency of magnitude and direction associated with each season is provided in Figure 2.

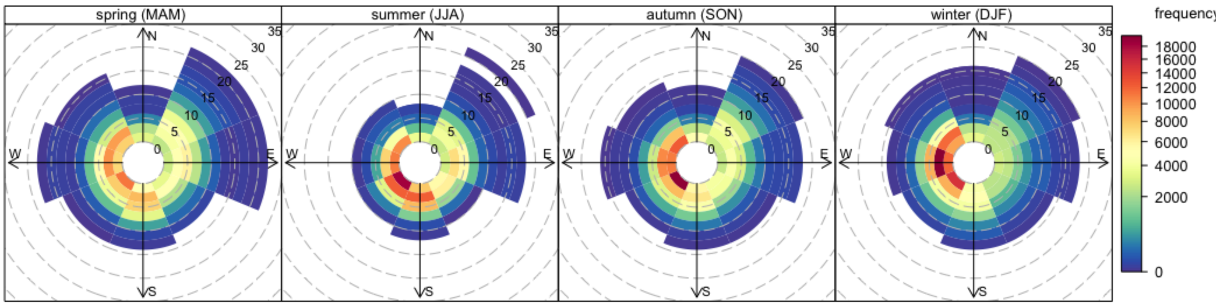


Figure 2. Seasonal wind frequency plots.

Three major hurricane landfall events were also subset to determine major storm winds and estimate their wave generation. The selected storms were hurricanes Isabel, Irene, and Sandy with landfall dates of 13 Sept 2003, 27 Aug 2011, and 29 Oct 2012, respectively. The corresponding subset included the 24 hours pre- and post-landfall. A wind rose showing the wind conditions associated with these storms is provided in Figure 3. Based on their frequency of occurrence, the seasonal and storm wind conditions in Table 2 were selected for modeling.

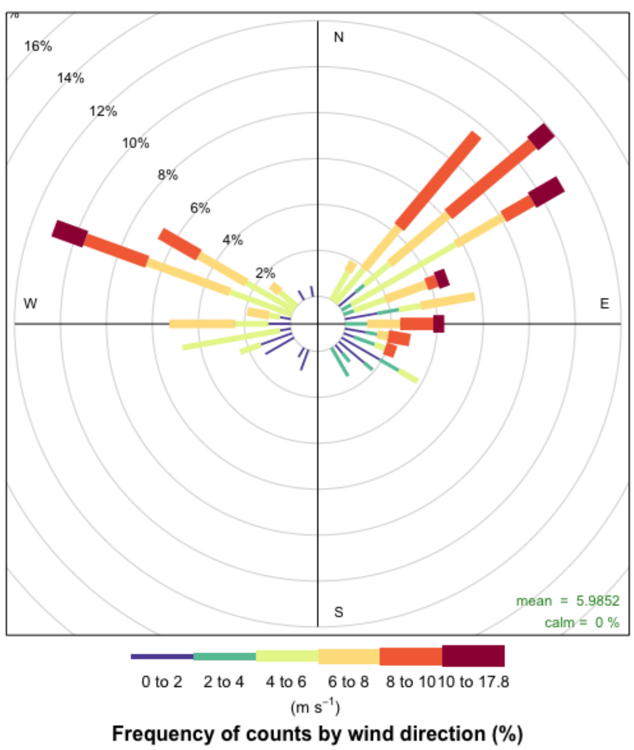


Figure 3. Wind rose of storm events.

Table 2. Modeled wind conditions.

Event	Condition Description	Wind Speed (mph)	Wind Direction (deg)
Spring	Average	4.5	220.0
	High	26.8	60.0
Summer	Average	4.5	230.0
	High	17.9	60.0
Fall	Average	4.5	230.0
	High	22.4	40.0
Winter	Average	6.7	270.0
	High	11.2	190.0
Storm	From West	22.4	290.0
	From Northeast	17.9	60.0

The USACE Sea Level Change Curve Calculator (Version 2017.55) was used to determine a 100-year sea level rise (SLR) scenario for Mordecai Island based on the ~100-year tide record at Atlantic City, NJ. An intermediate future sea level of 2.6 ft (0.8 m) NAVD88 was used for the 100-year project condition (Figure 4). SLR was represented in the model by a static water level increase across the entire domain.

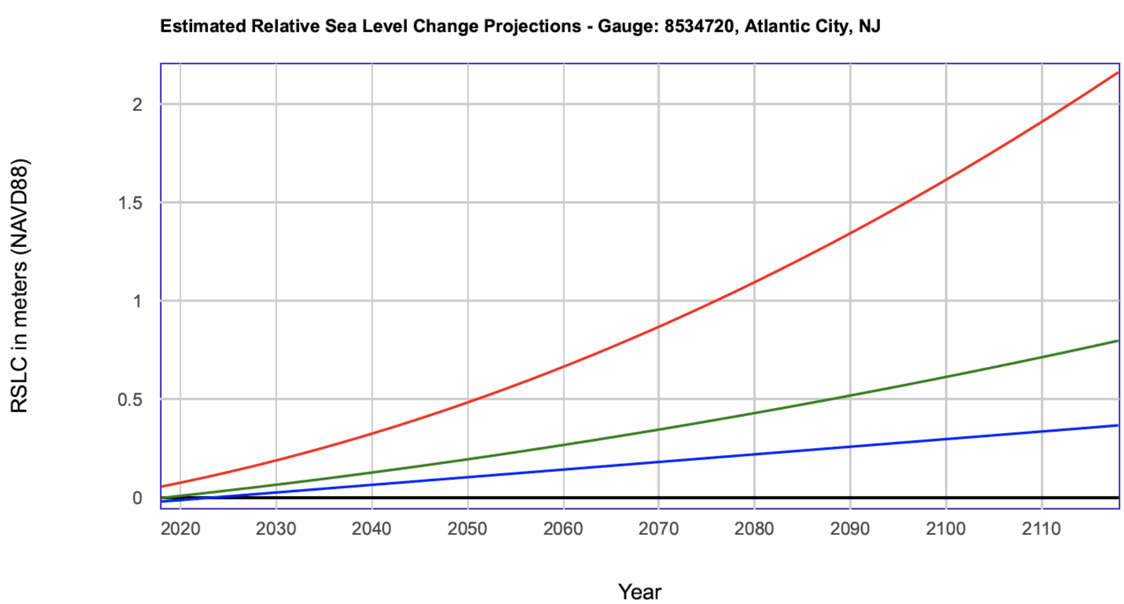


Figure 4. Relative sea level change relative to NAVD88.

ANALYSIS: Plots of the zero-moment wave height for each of the modeled wind conditions is provided in Appendix A. The average wind conditions for the Spring, Summer, and Fall produced nearly calm wave conditions characterized by wave heights less than 0.2 ft in the vicinity of Mordecai. The peak wave periods throughout the domain were very short, less than 1.5 s, and were near or at the minimum frequency resolved by the model. The average wind condition for the Winter yielded slightly larger waves than the average winds for the Spring, Summer, and Fall because of the slightly higher wind magnitude and a wind direction more directed at the island. For the Winter average wind, waves just offshore of northern Mordecai were less than 0.5 ft with smaller wave heights along the south of Mordecai due to sheltering by the islands to its west. Again, wave periods are short, around 1.5 s. It is important to note that the wave energy in sheltered areas can be underestimated because wave diffraction is not included in STWAVE.

The Spring high, Summer high, and Fall high are all wind conditions blowing out of the northeast. These northeastern winds grow the largest waves, ranging from 1.0 to 1.6 ft and periods up to 3.5 s, depending on the wind magnitude, in the southwestern bay due to the uninterrupted fetch. However, waves along the western edge of Mordecai Island are smaller due to the wind direction blowing away from the island and sheltering by island groups to its north. The wave heights offshore of Mordecai Island are less than 0.7 ft with peak periods of less than 2.0 s. The Winter high wind condition produces smaller wave heights than the Spring, Summer, and Fall high wind conditions due to its smaller wind magnitude and direction out of the south; wave heights and peak periods in the vicinity of Mordecai Island are less than 0.3 ft and 1.5 s, respectively.

The storm wind conditions generate a more energetic wave climate near Mordecai Island than the seasonal wind conditions. Whereas the storm wind condition from the northeast generates wave heights and peak periods of approximately 0.3 ft to 0.5 ft and 1.5 s, respectively, the storm wind condition from the west grows waves along an uninterrupted fetch against the western coast of Mordecai Island. This condition results in the largest waves considering all the simulations with maximum wave heights of approximately 1.5 ft and peak periods of 2.0 to 2.5 s.

The maximum zero-moment wave height, its corresponding peak period, and the savepoint at which it occurred for each modeled wind condition is provided in Table 3.

Table 3. Maximum zero-moment wave height (H_{m0}), associated peak period (T_p), and savepoint ID for modeled wind conditions.

Event	Condition Description	Maximum H_{m0} [ft]	T_p [s]	Savepoint ID
Spring	Average	0.16	1.1	3
	High	0.66	1.5	2
Summer	Average	0.16	1.2	3
	High	0.46	1.4	2
Fall	Average	0.16	1.2	3
	High	0.62	1.5	2
Winter	Average	0.36	1.4	2
	High	0.26	1.3	1
Storm	From West	1.12	2.0	7
	From Northeast	0.46	1.4	2

These wind simulations were repeated with the 100-year SLR water level of 2.6 ft. No intervention to raise the elevation of Mordecai Island or the surrounding land was undertaken. As seen in Figure 5, this water level inundated a significant amount of the domain, including almost all of Mordecai Island.

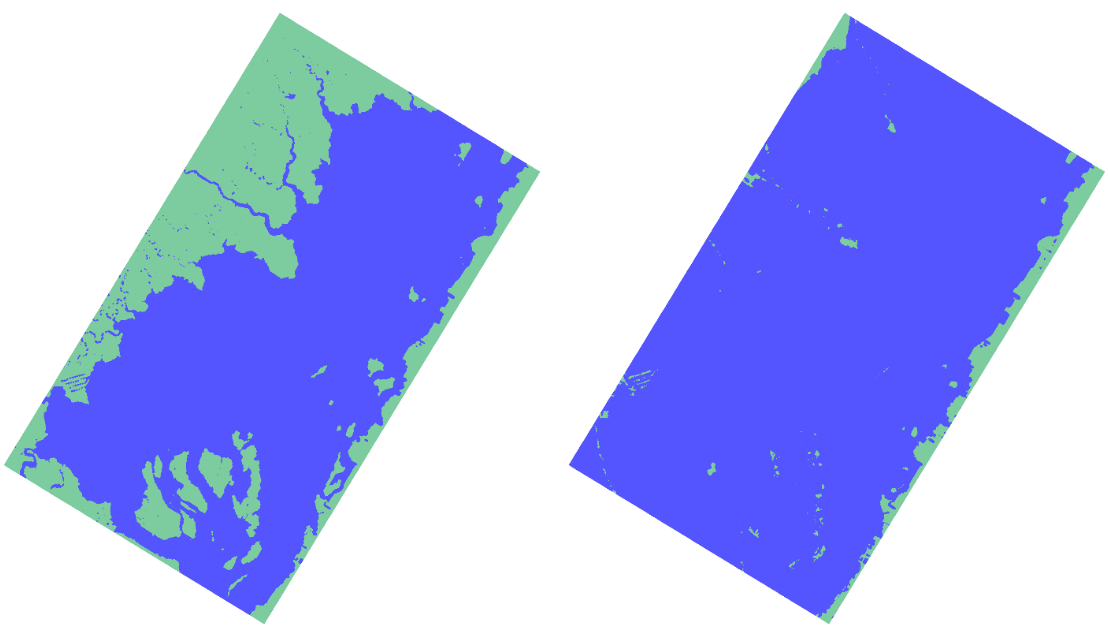


Figure 5. STWAVE domain (left) without and (right) with SLR water level adjustment.

Plots of the zero-moment wave height for each modeled wind condition with the constant SLR water level adjustment is provided in Appendix B. The largest differences in wave height between the two scenarios is associated with the seasonal high and storm wind conditions, and is generally found in areas that are now inundated due to SLR. However, offshore of and in the vicinity of Mordecai, the difference in wave height and peak period is small with the adjustment of SLR. Comparing Tables 3 and 4, the largest difference in wave height due to a static SLR water level adjustment is for the storm wind condition from the west, which was the highest wind magnitude and was oriented directly at Mordecai Island. Difference plots of wave height with and without SLR for each modeled condition are provided in Appendix C.

Table 4. Maximum zero-moment wave height (H_{m0}), associated peak period (T_p), and savepoint ID for modeled wind conditions with SLR adjustment.

Event [SLR]	Condition Description	Maximum H_{m0} [ft]	T_p [s]	Savepoint ID
Spring	Average	0.20	1.3	2
	High	0.69	1.5	1
Summer	Average	0.20	1.3	2
	High	0.49	1.3	3
Fall	Average	0.20	1.3	2
	High	0.69	1.6	3
Winter	Average	0.39	1.5	2
	High	0.32	1.3	1
Storm	From West	1.38	2.2	4
	From Northeast	0.49	1.4	3

In addition to the SLR scenario, a spring high tide simulation was considered. However, given that the tidal range is less than that of the added SLR, changes in the wave climate in the vicinity of Mordecai are expected to remain small and excluded the need for simulation.

SUMMARY: The local wind-wave climate of Mordecai Island was estimated using the STWAVE nearshore model. The 10 selected wind conditions are representative of the area considering frequency based on meteorological data from the NOAA Jacques Cousteau National Estuarine Research Reserve. The wave energy in the vicinity of Mordecai Island is found to be small for the seasonal wind simulations due to low wind magnitudes and directions generally along the north-south axis of the bay (e.g., waves are travelling roughly parallel to or away from the island, sheltering from other island

groups). The wind magnitudes for the average seasonal conditions are low, under 7 mph, and produce waves less than 0.4 ft with peak periods up to 1.5 s. The high wind condition blows out of the northeast for all of the seasons, except Winter. The maximum wave height for the Winter high condition is around 0.3 ft whereas it is about 0.7 ft for the Spring, Summer, and Fall high wind condition. The largest waves, those exceeding 1.0 ft, are generated by the storm condition of 22.4 mph blowing directly at the island from the west over an uninterrupted fetch. Except for this storm wind condition, the addition of 100-year SLR water level adjustment did not significantly alter the wave energy in the vicinity of Mordecai Island. To that end, the land used to define the STWAVE domain became inundated under the SLR scenario and full fetch extents may no longer be captured by the model.

Based on the small wave conditions estimated in this modeling effort, it seems unlikely that the severe erosion observed at Mordecai Island is solely due to wind-waves. Note that these results are extrapolated to erosion potential and no direct erosion analysis was undertaken. An assessment of vessel wakes is recommended as the contribution of transiting vessels to the wave energy impacting Mordecai Island may be significant given its close proximity to the NJIWW.

REFERENCES:

Massey, T.C., M.E. Anderson, J.M. Smith, J. Gomez, and R. Jones. 2011. *STWAVE: Steady-state spectral wave model user's manual for STWAVE, version 6.0*. ERDC/CHL SR-11-1. U.S. Army Engineering Research and Development Center, Vicksburg, MS**.**

Appendix A

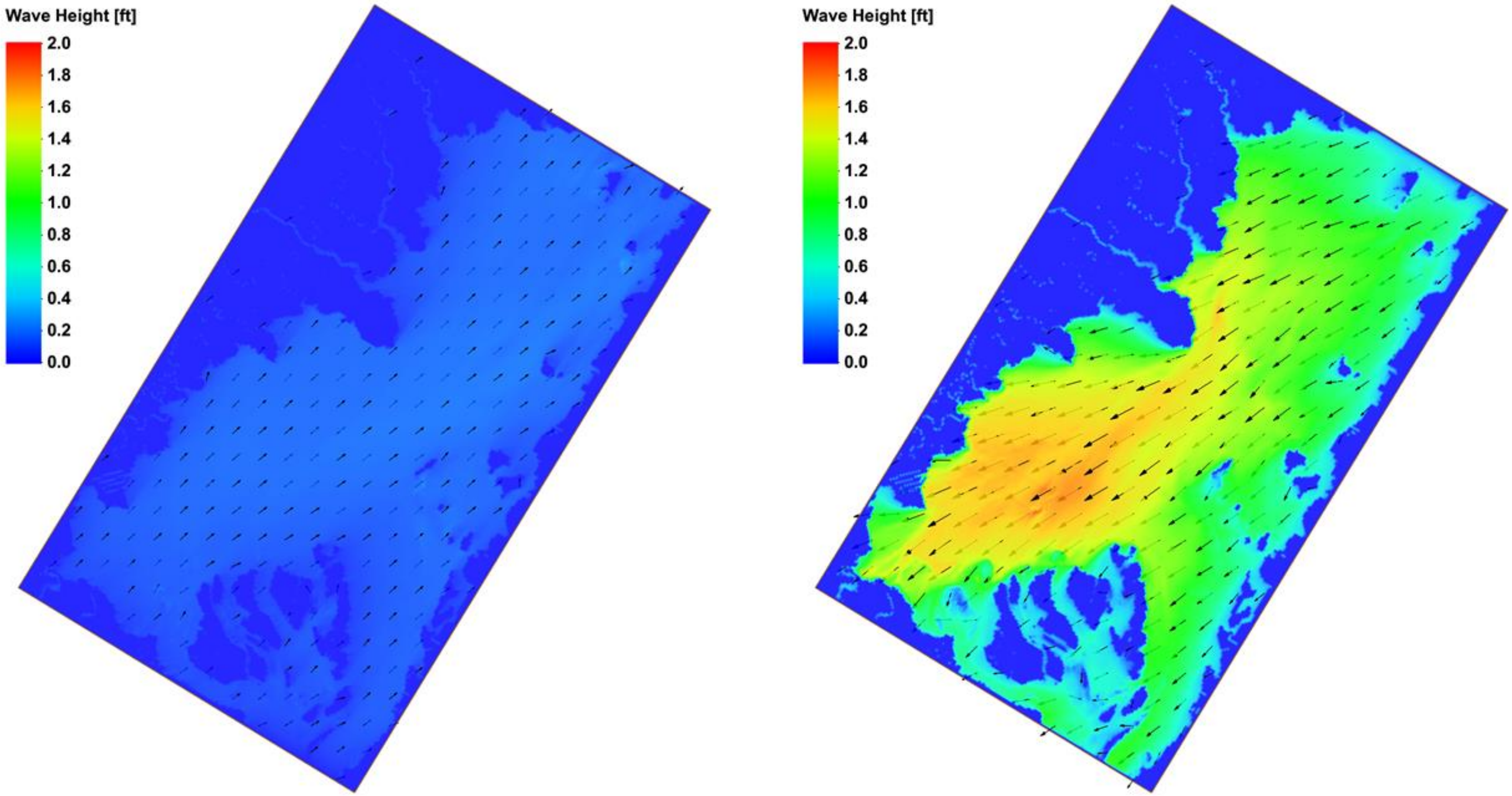


Figure 6. Wave heights for Spring (left) average and (right) high wind conditions. Vectors indicate the mean wave direction.

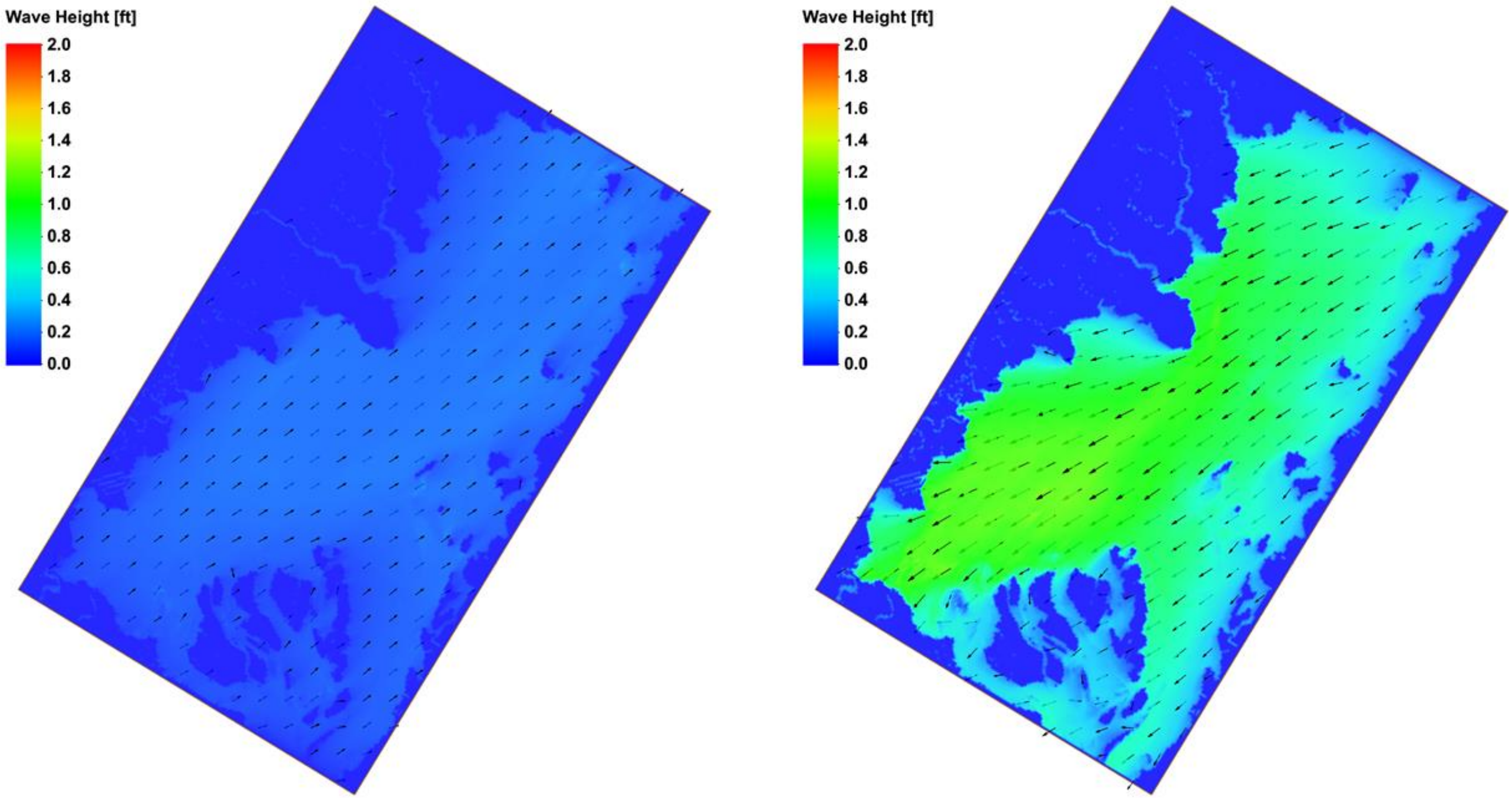


Figure 7. Wave heights for Summer (left) average and (right) high wind conditions. Vectors indicate the mean wave direction.

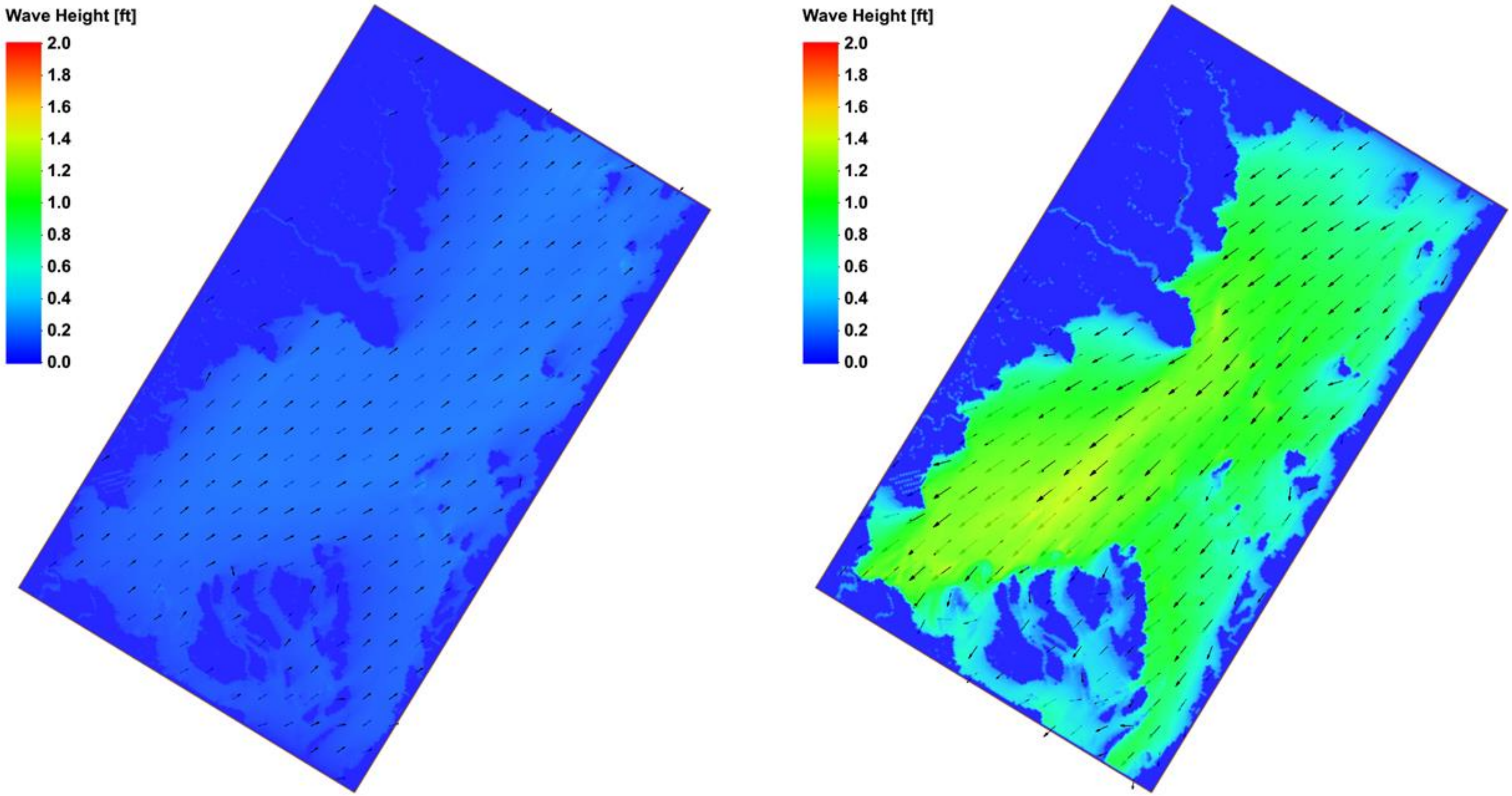


Figure 8. Wave heights for Fall (left) average and (right) high wind conditions. Vectors indicate the mean wave direction.

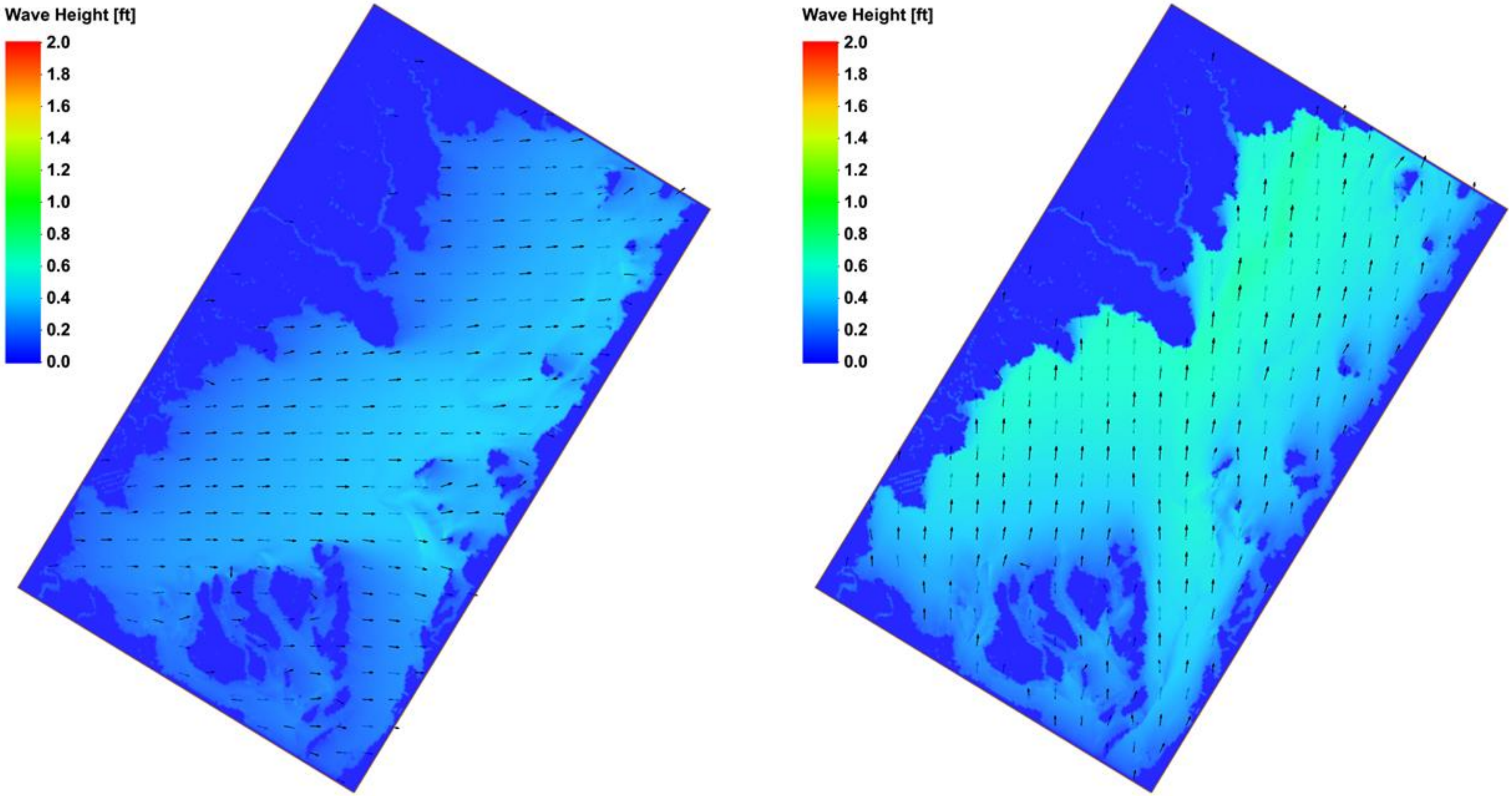


Figure 9. Wave heights for Winter (left) average and (right) high wind conditions. Vectors indicate the mean wave direction.

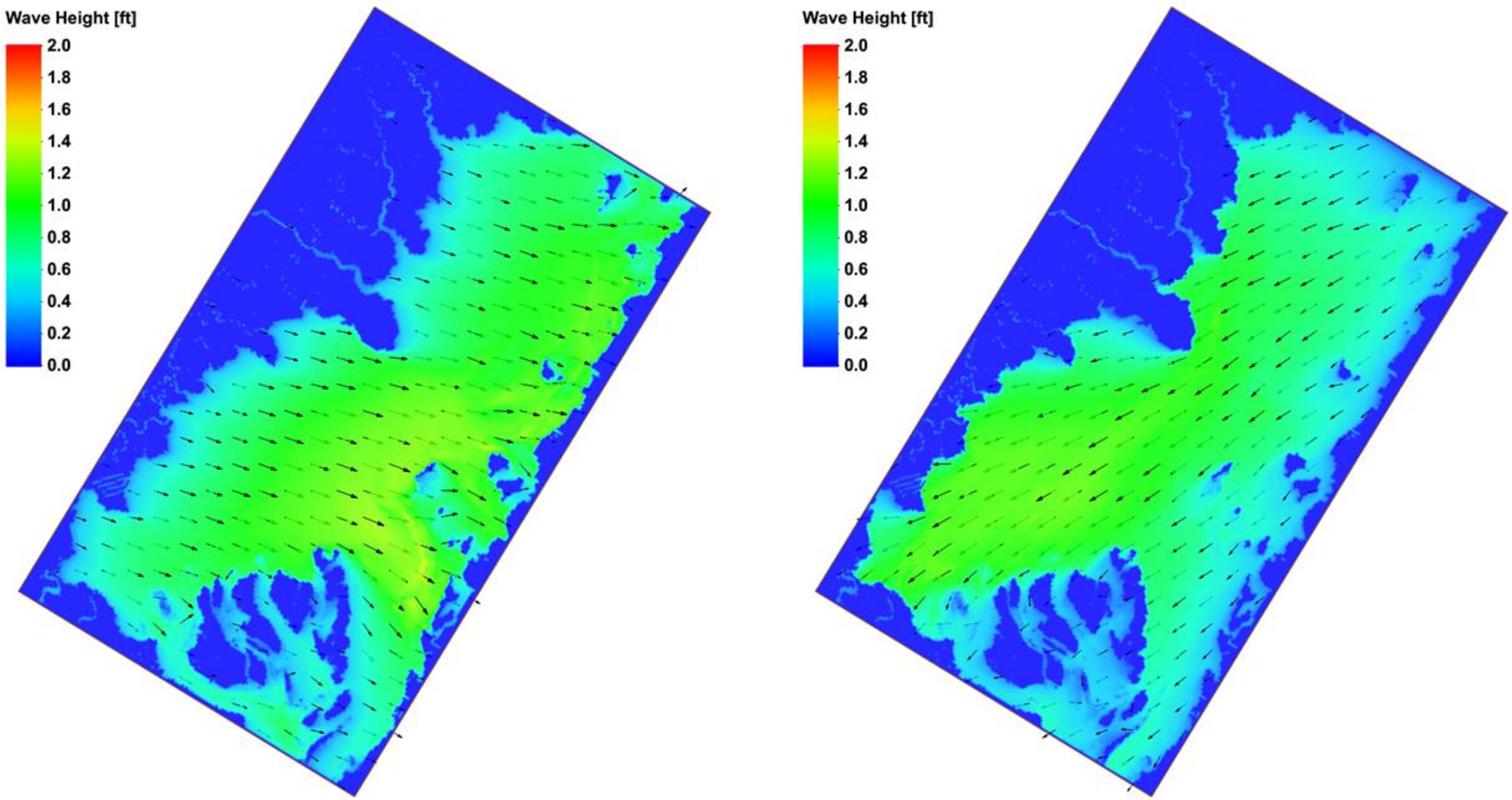


Figure 10. Wave heights for Storm (left) from west and (right) northeast wind conditions. Vectors indicate the mean wave direction.

Appendix B

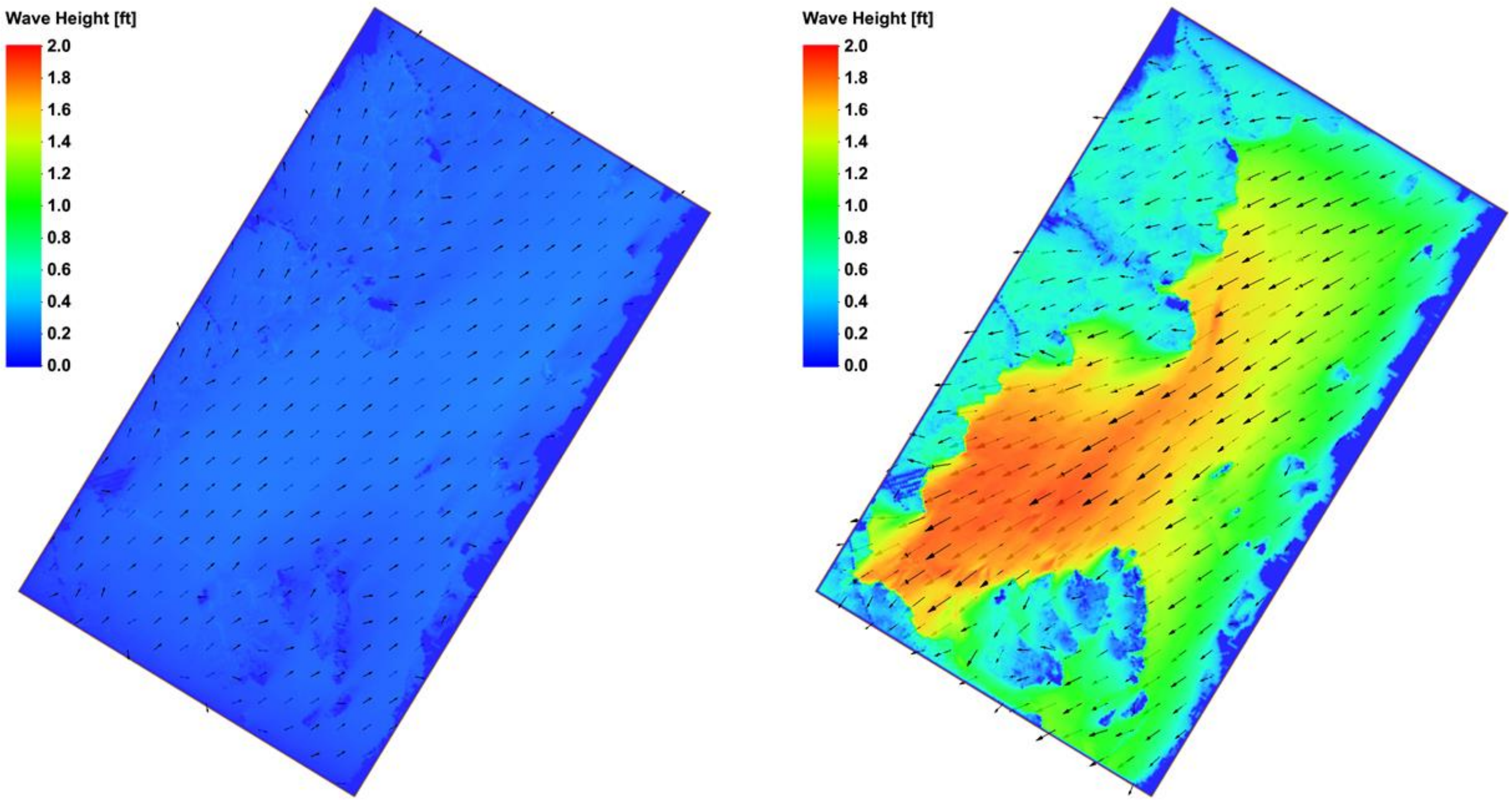


Figure 11. Wave heights for Spring (left) average and (right) high wind conditions with SLR. Vectors indicate the mean wave direction.

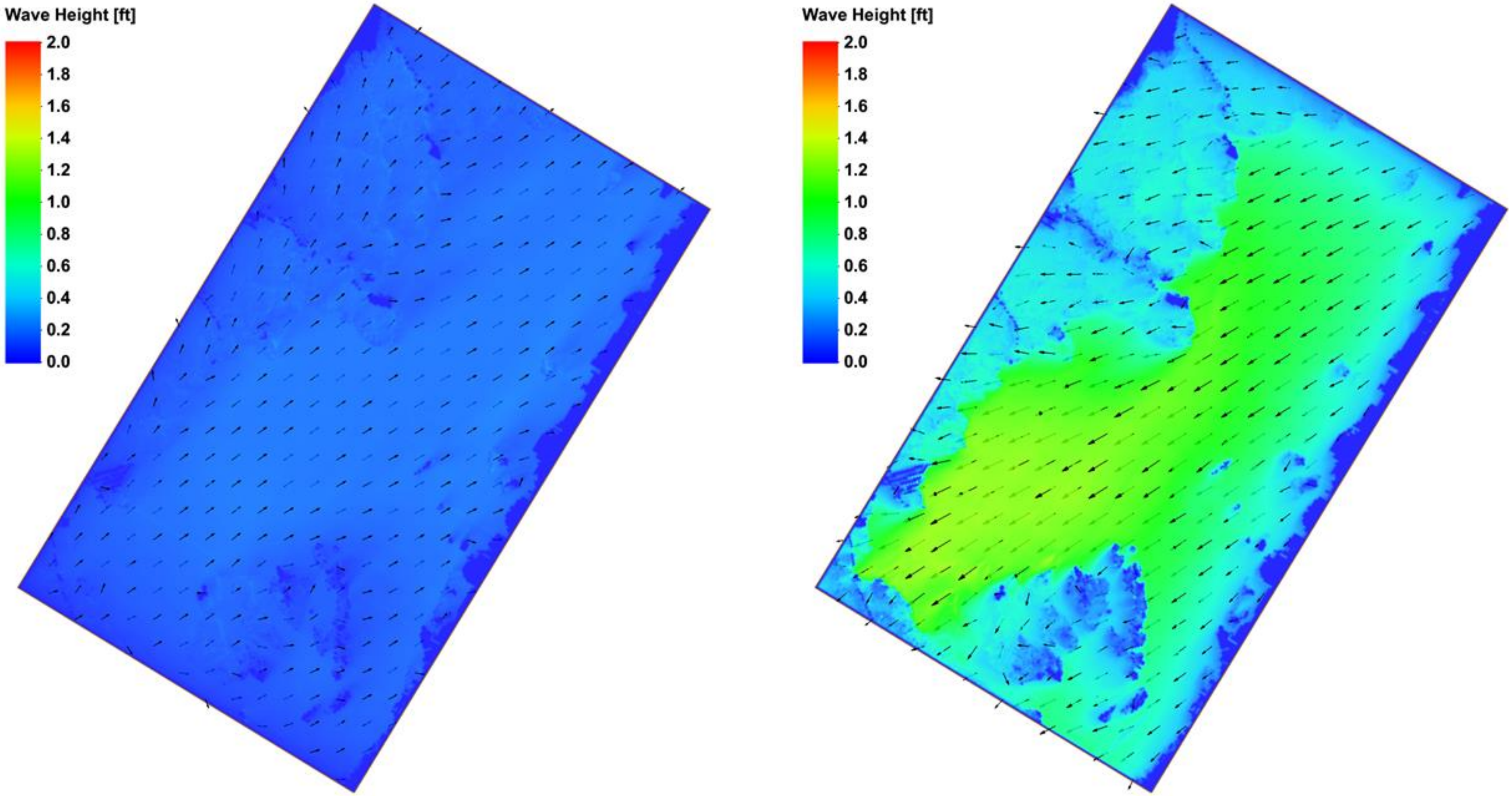


Figure 12. Wave heights for Summer (left) average and (right) high wind conditions with SLR. Vectors indicate the mean wave direction.

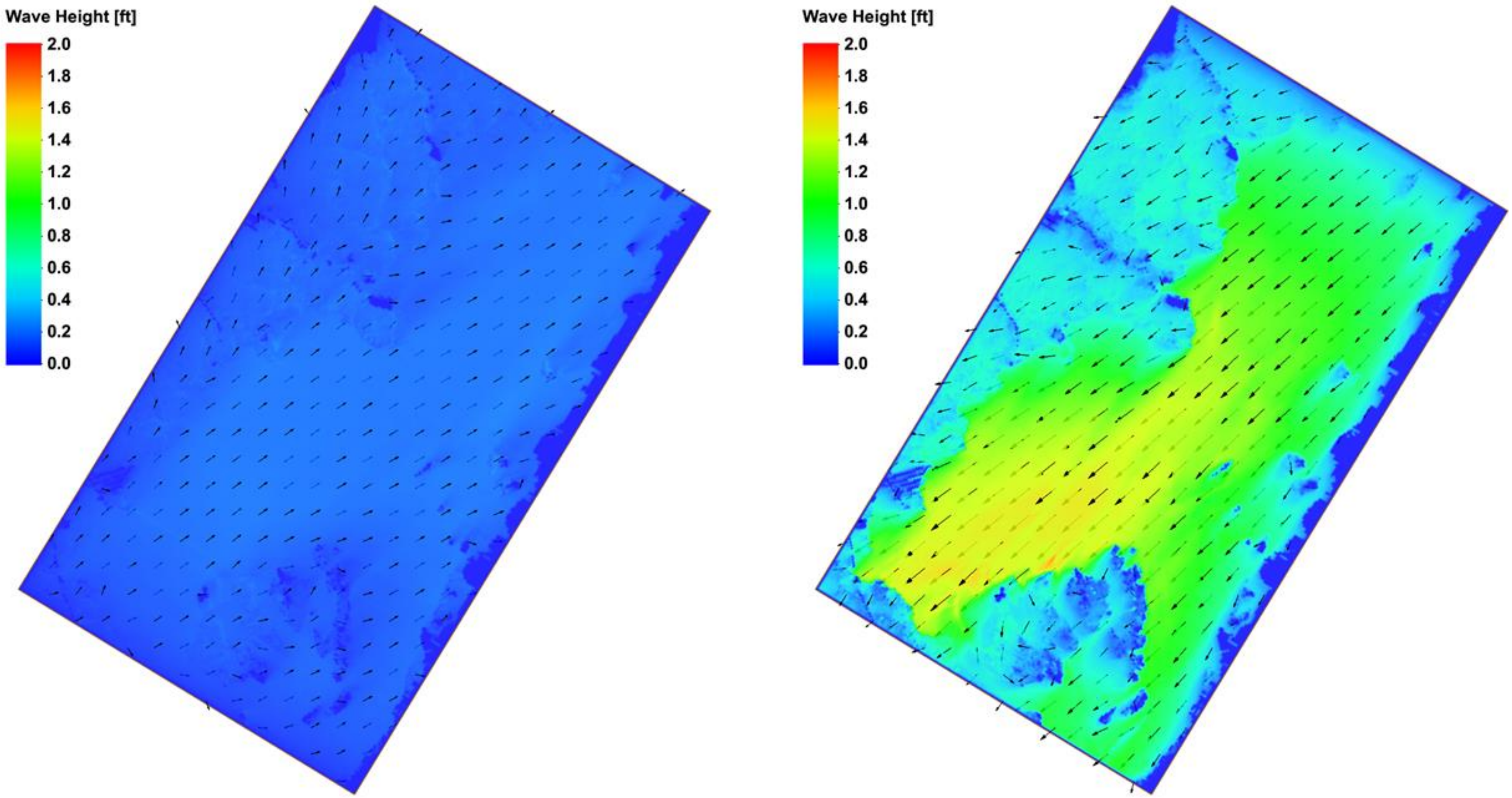


Figure 13. Wave heights for Fall (left) average and (right) high wind conditions with SLR. Vectors indicate the mean wave direction.

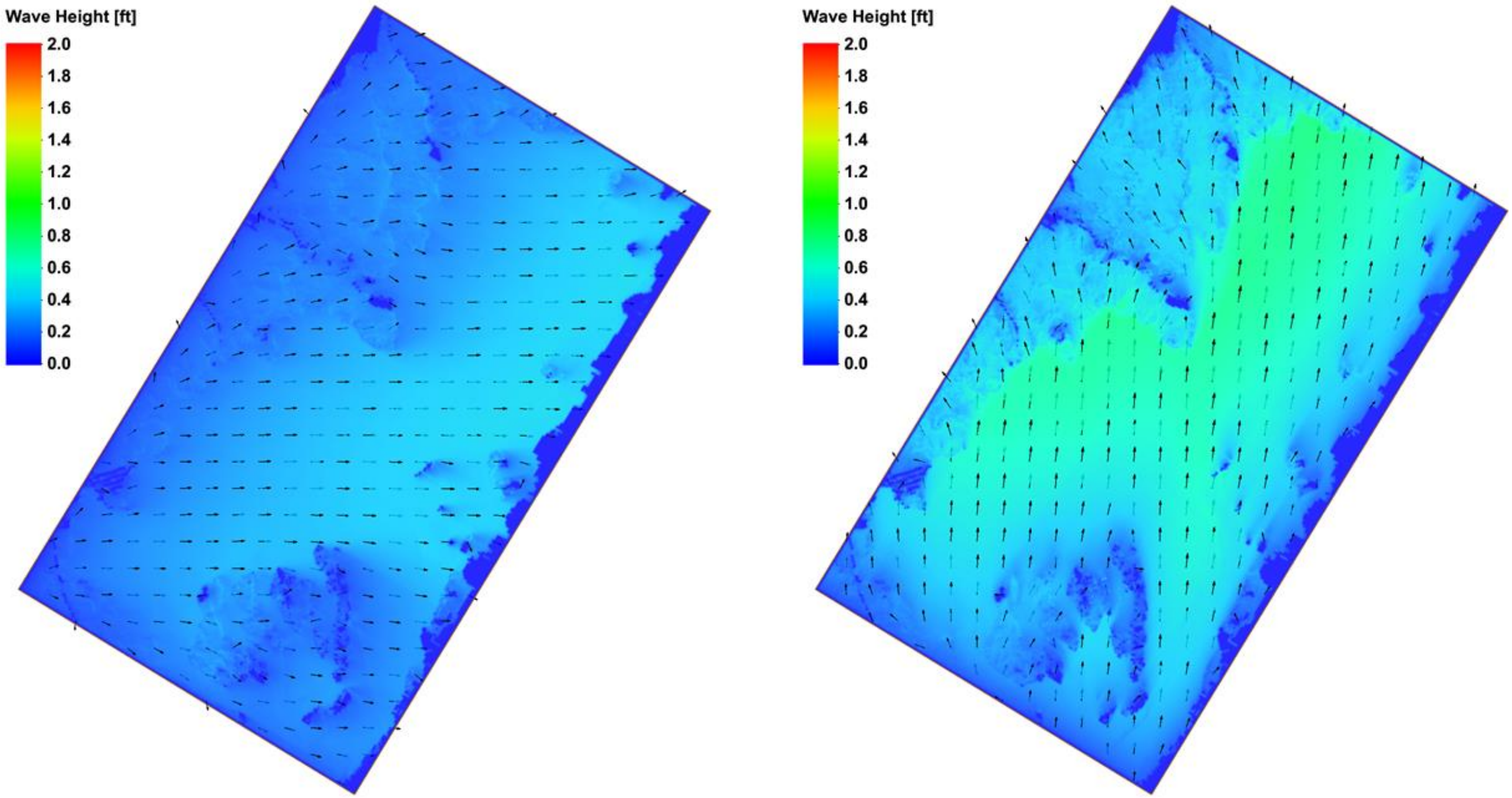


Figure 14. Wave heights for Winter (left) average and (right) high wind conditions with SLR. Vectors indicate the mean wave direction.

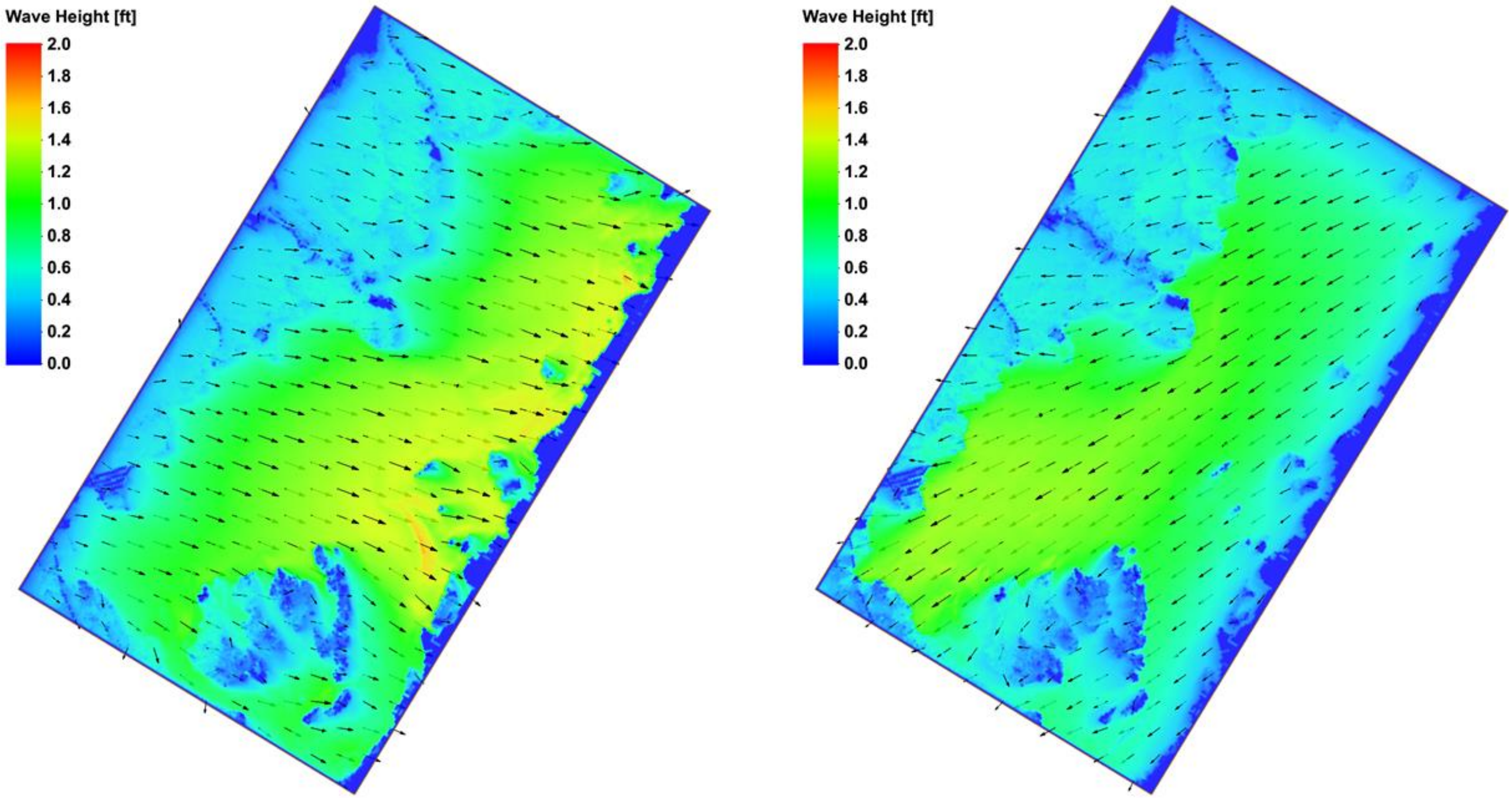


Figure 15. Wave heights for Storm (left) from west and (right) from northeast wind conditions with SLR. Vectors indicate the mean wave direction.

Appendix C

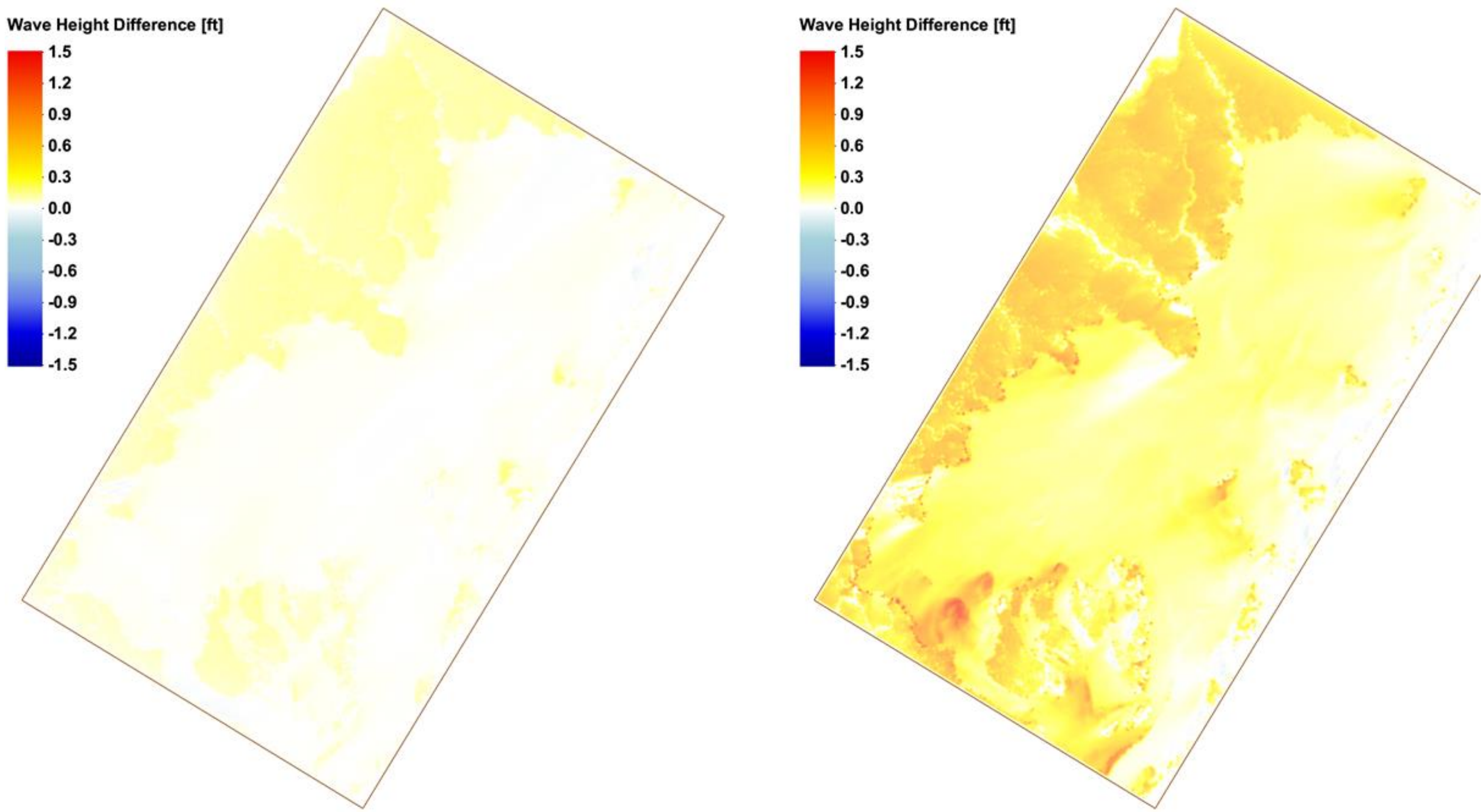
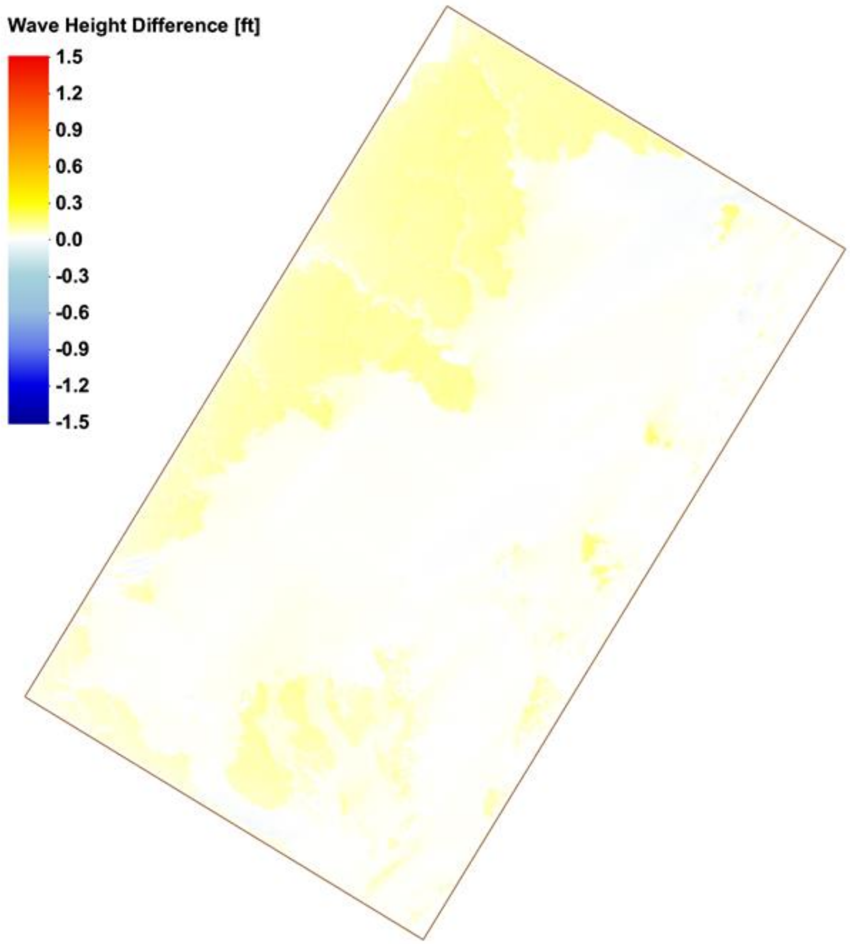


Figure 16. Difference in wave heights for Spring (left) average and (right) high with SLR adjustment. Warm colors indicate increases in wave height for the SLR simulations.



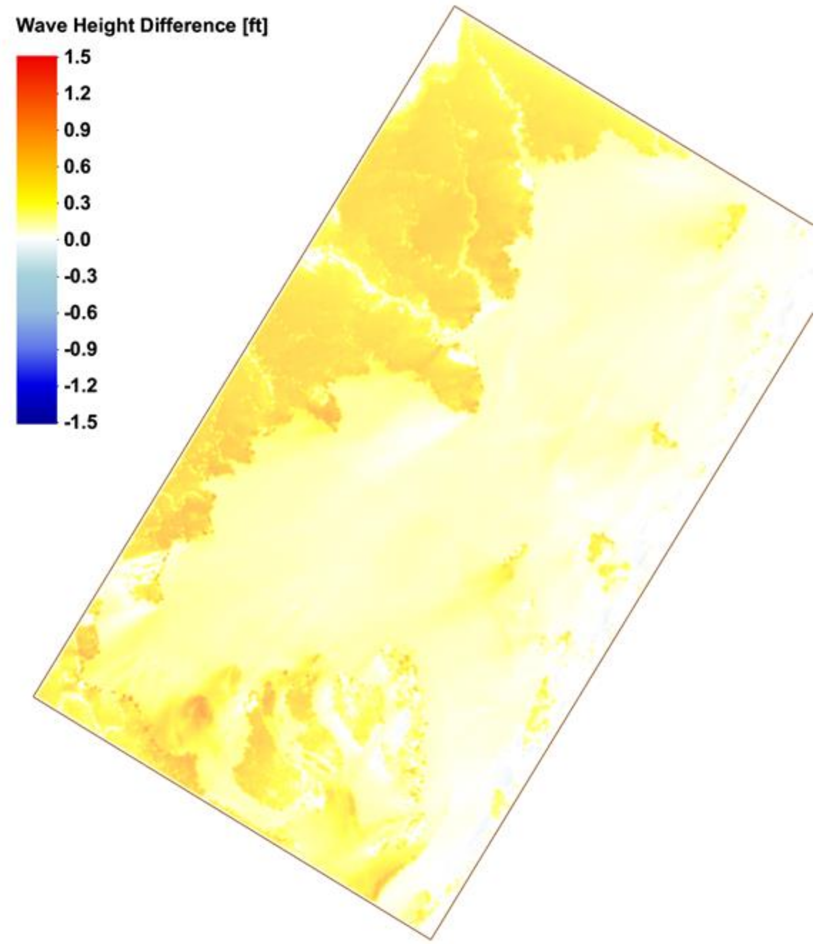


Figure 17. Difference in wave heights for Summer (left) average and (right) high with SLR adjustment. Warm colors indicate increases in wave height for the SLR simulations.

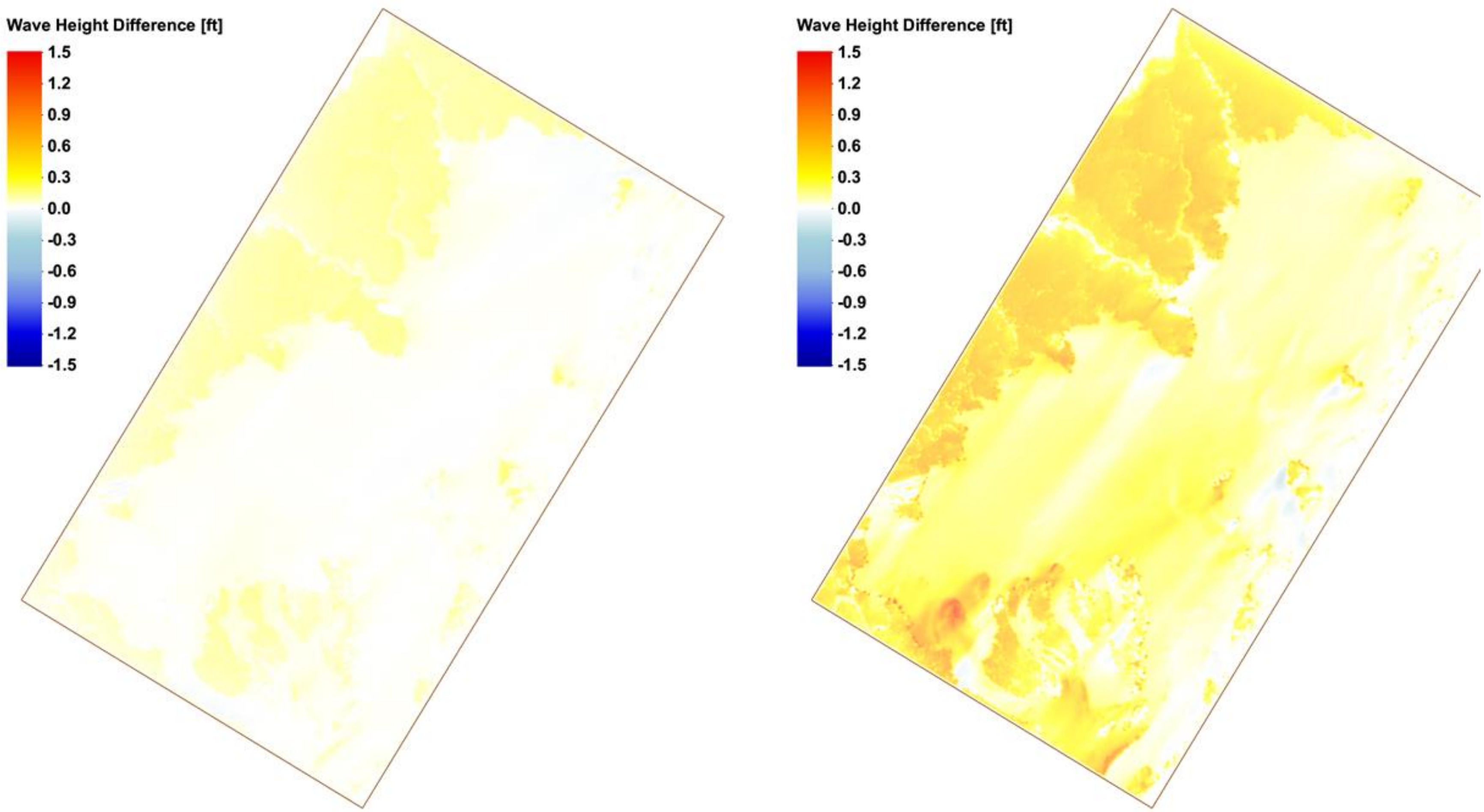


Figure 18. Difference in wave heights for Fall (left) average and (right) high with SLR adjustment. Warm colors indicate increases in wave height for the SLR simulations.

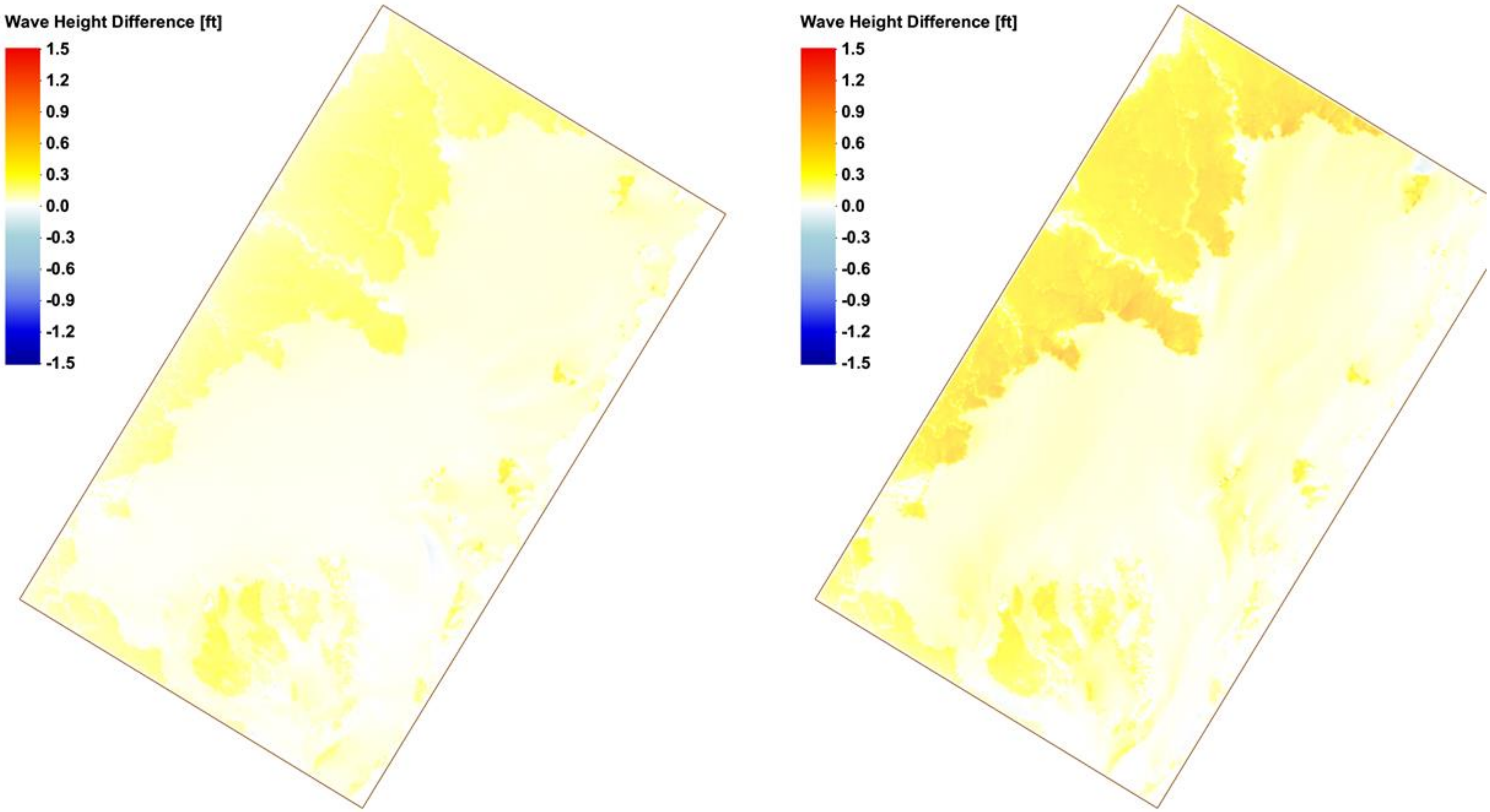


Figure 19. Difference in wave heights for Winter (left) average and (right) high with SLR adjustment. Warm colors indicate increases in wave height for the SLR simulations.

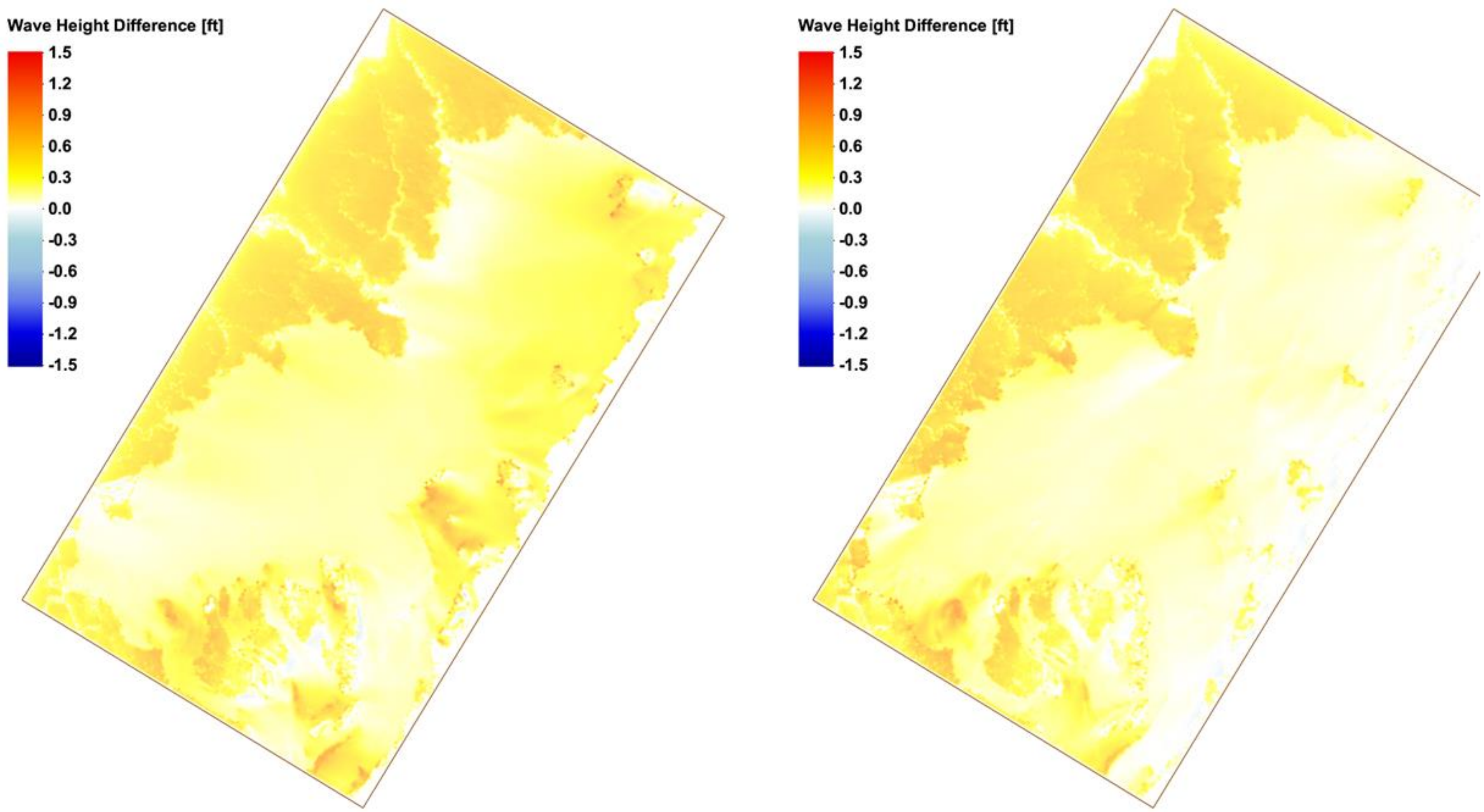
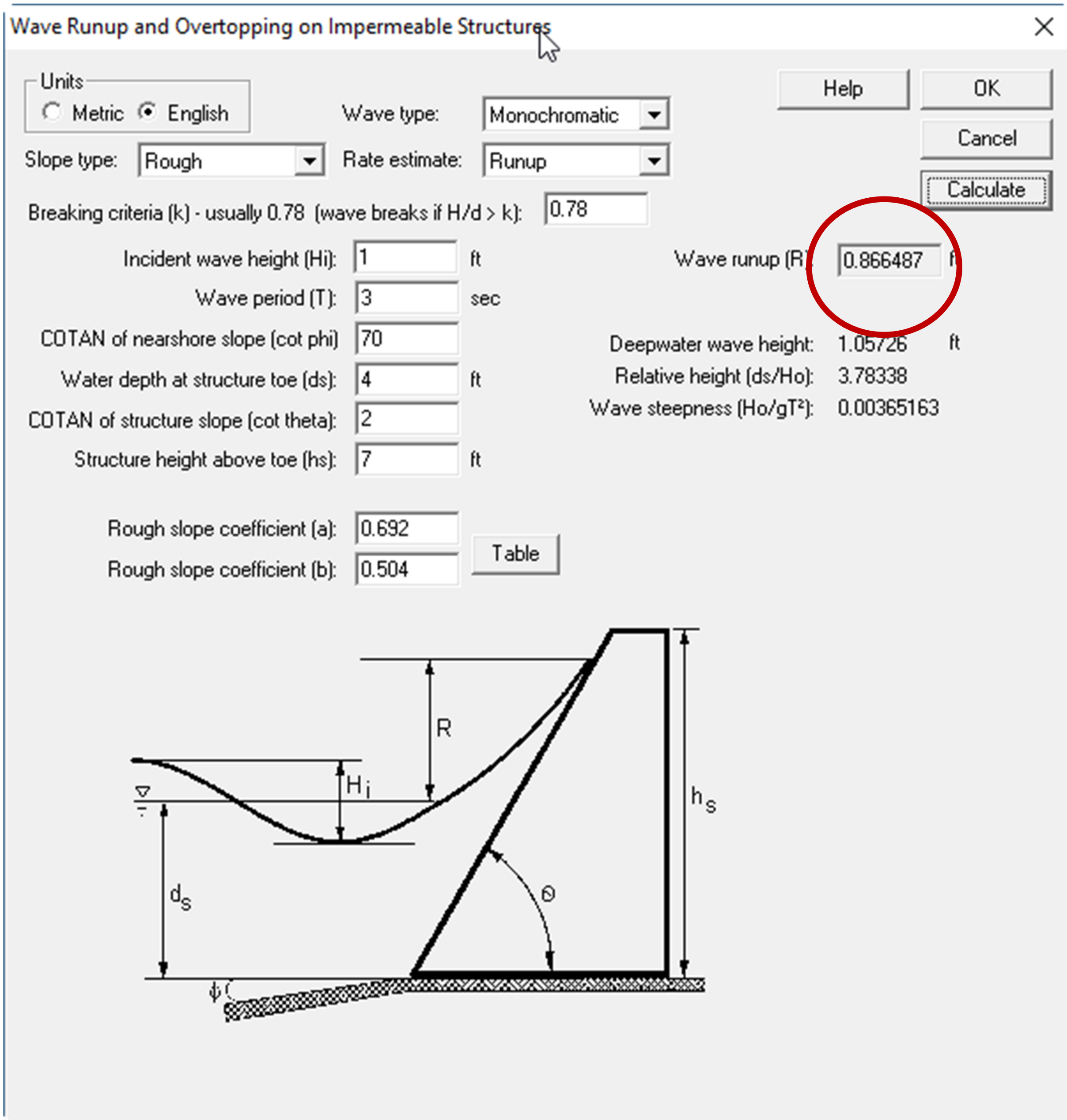


Figure 20. Difference in wave heights for Storm (left) from west and (right) from northeast with SLR adjustment. Warm colors indicate increases in wave height for the SLR simulations.

ATTACHMENT A2-3 – WAVE RUNUP CALCULATIONS

Runup from a 1.0 ft. High, 3 sec Period Wave

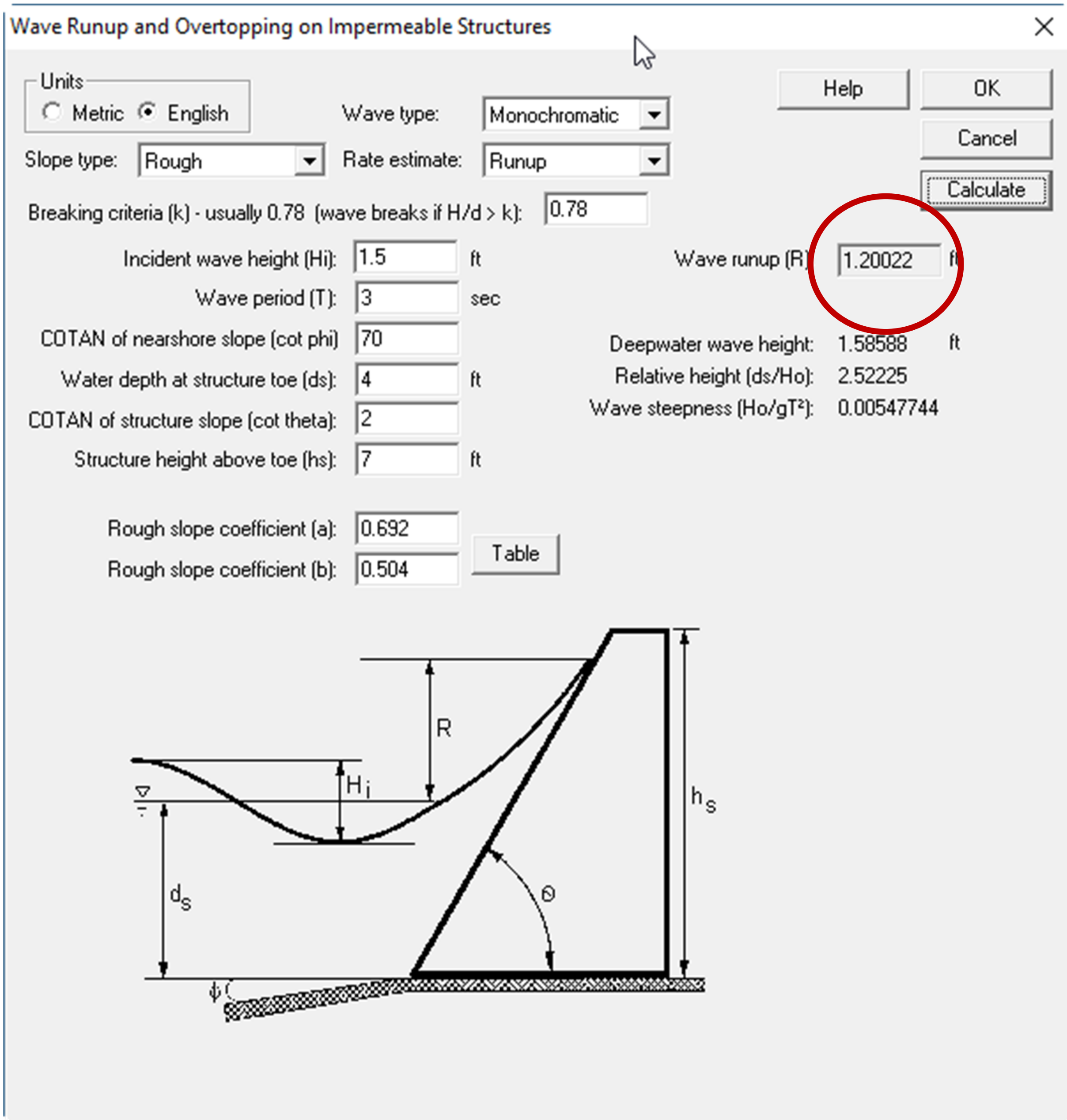
Permeable Rubble Structure with no Core



Automated Coastal Engineering System (ACES) used to calculate wave runup

Runup from a 1.5 ft. High, 3 sec Period Wave

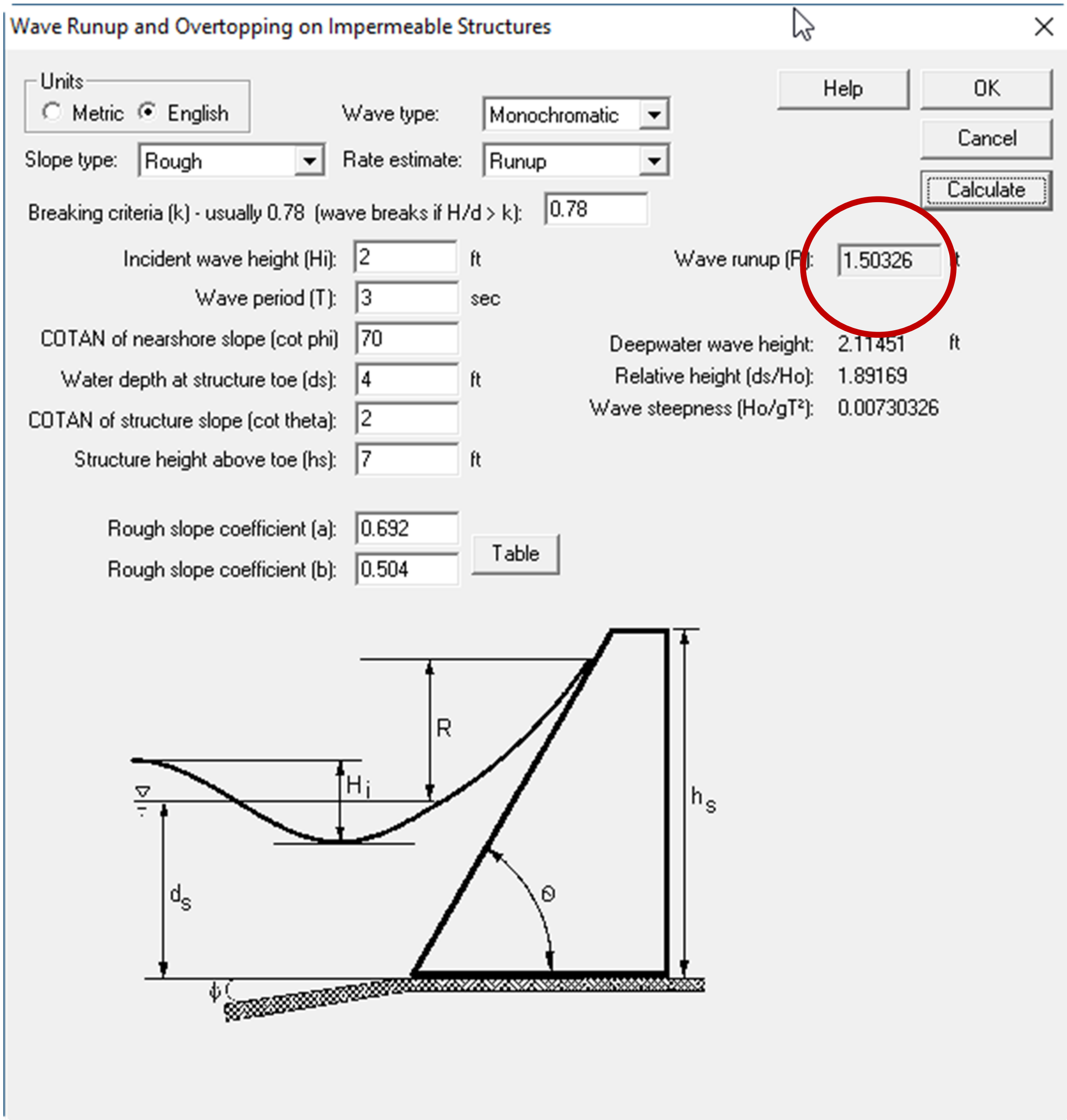
Permeable Rubble Structure with no Core



Automated Coastal Engineering System (ACES) used to calculate wave runup

Runup from a 2.0 ft. High, 3 sec Period Wave

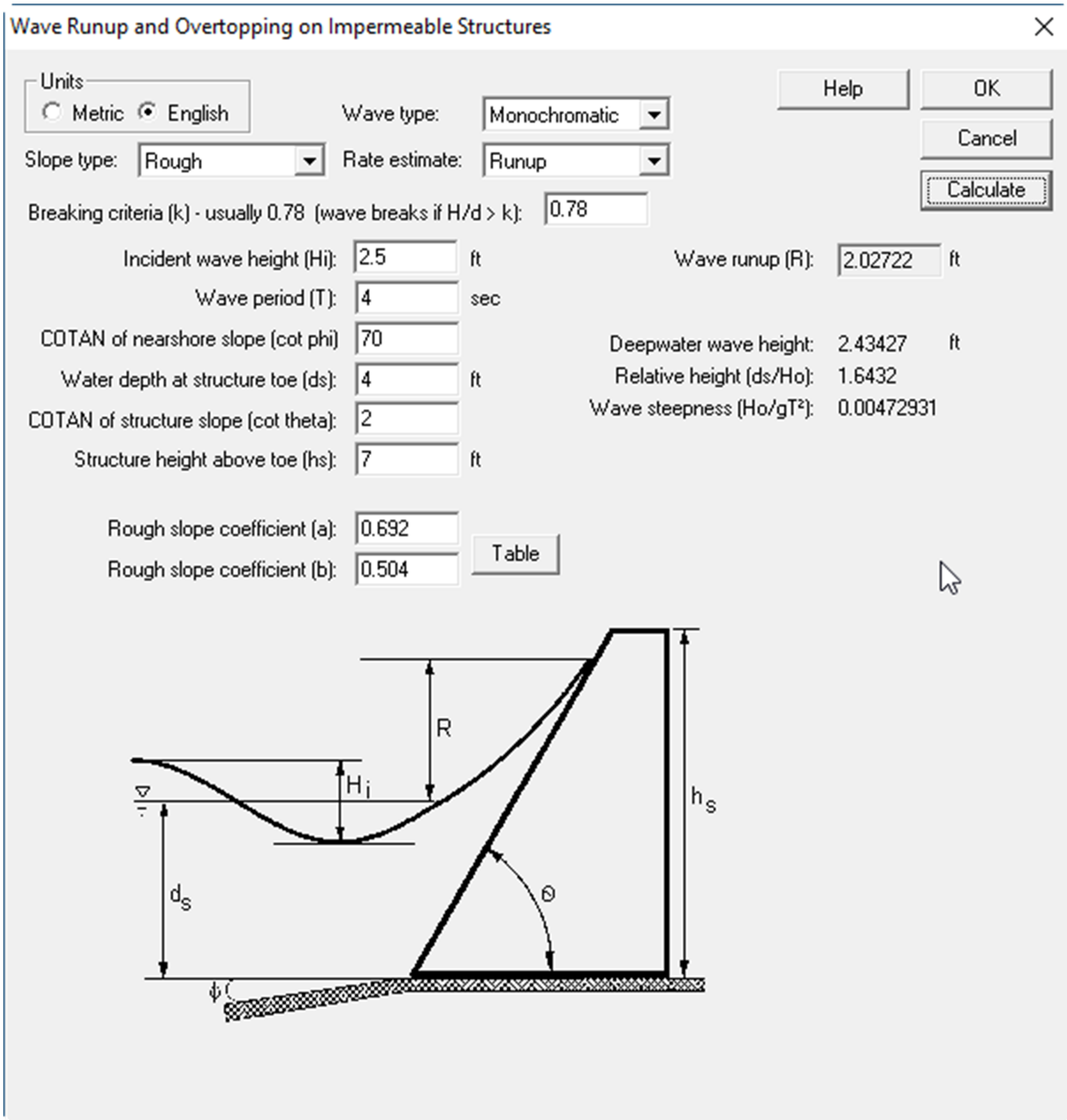
Permeable Rubble Structure with no Core



Automated Coastal Engineering System (ACES) used to calculate wave runup

Runup from a 2.5 ft. High, 4 sec Period Wave

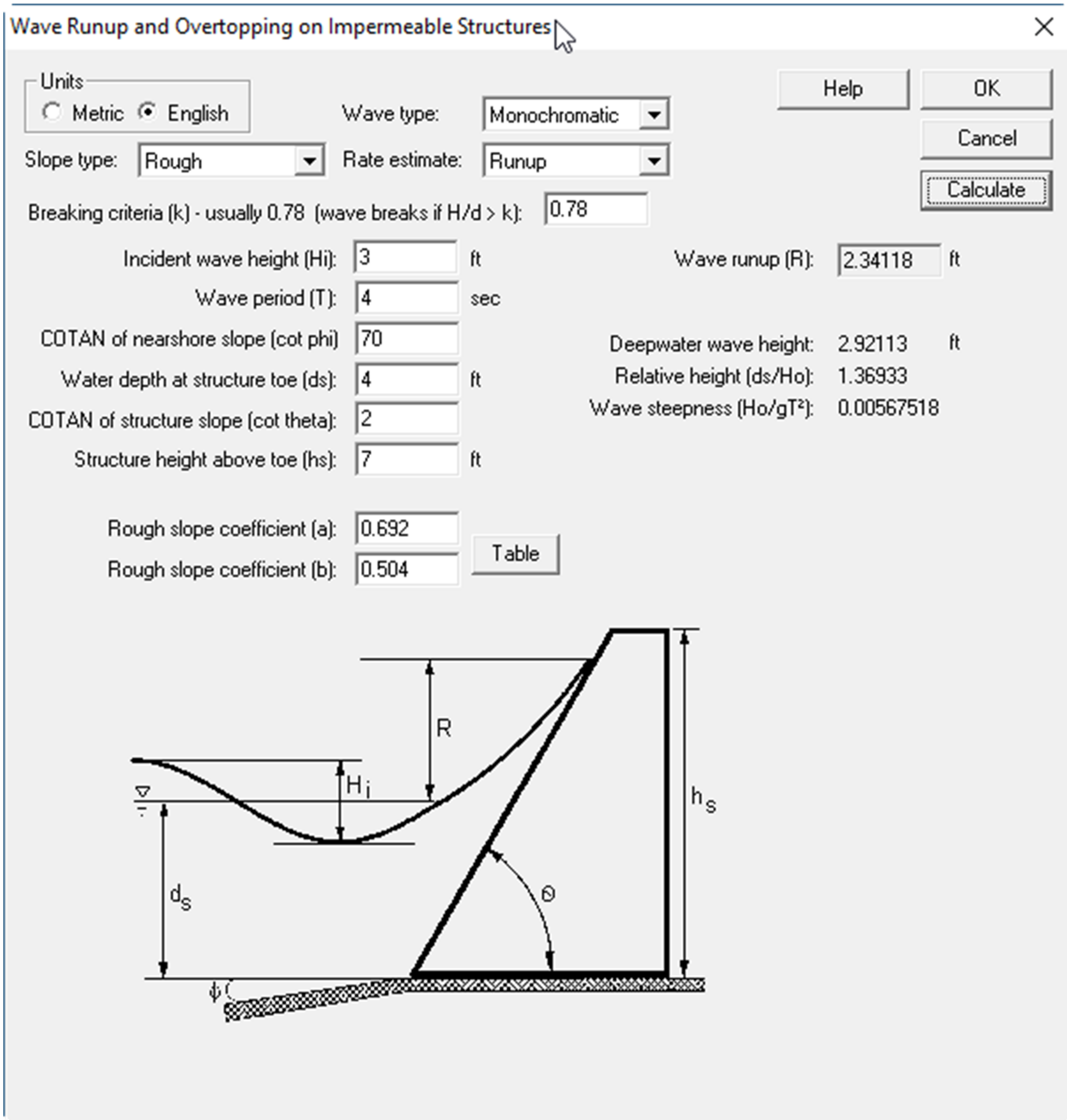
Permeable Rubble Structure with no Core



Automated Coastal Engineering System (ACES) used to calculate wave runup

Runup from a 3.0 ft. High, 4 sec Period Wave (3 sec period waves are too steep)

Permeable Rubble Structure with no Core



Automated Coastal Engineering System (ACES) used to calculate wave runup

ATTACHMENT A2-4 – BIOENGINEERED BREAKWATER TECHNICAL REPORT

Bioengineered Breakwater Technical Report

Bayou Caddy Shoreline Stabilization,
Bayou Caddy, Hancock County, Mississippi
W91278-15-D-0084 0001

Date 24 October 2016

Integris Projects, LLC
104 East Heritage Drive
Friendswood, TX 77546

TRANSMITTAL OF SHOP DRAWINGS, EQUIPMENT DATA, MATERIAL SAMPLES, OR MANUFACTURER'S CERTIFICATES OF COMPLIANCE

(Read instructions on the reverse side prior to initiating this form)

DATE: 10/27/2016

TRANSMITTAL NO.: 35 31 23.19-1

SECTION I - REQUEST FOR APPROVAL OF THE FOLLOWING ITEMS (This section will be initiated by the contractor)

TO: Irvington Site Office
7861 13th Street
Irvington, AL 35544-2899

FROM: Integris Projects, LLC
104 E HER ITAGE DR
FRIENDSWOOD, TX 77546

CONTRACT NO.: W91278-15-D-0084 0001

CHECK ONE:
- [X] THIS IS A NEW TRANSMITTAL
- [ ] THIS IS A RESUBMITTAL OF TRANSMITTAL ____

SPECIFICATION SEC. NO. (Cover only one section with each transmittal): 35 31 23.19

PROJECT TITLE AND LOCATION: 01-Main Register
Bayou Caddy Shoreline Stabilization Hancock County, Ms

CHECK ONE: THIS TRANSMITTAL IS FOR [ ] FIO [X] GA [ ] DA [ ] CR

ITEM NO. a.	DESCRIPTION OF ITEM SUBMITTED (Type size, model number/etc.) b.	MFG OR CONTR. CAT., CURVE DRAWING OR BROCHURE NO. (See instruction no. 8) c.	NO. OF COPIES d.	CONTRACT REFERENCE DOCUMENT: SPEC. PARA. NO. e.	CONTRACT REFERENCE DOCUMENT: DRAWING SHEET NO. f.	FOR CONTRACTOR USE CODE g.	VARIATION (See Instruction No. 6) h.	FOR CE USE CODE i.
1	Bioengineered Breakwater Placement Plan	PRECON SUBMTL	1			A	N	
4	Bioeng. Breakwater Technical Report	PRODUCT DATA	1			A	N	

REMARKS
In case of any discrepancies please contact:
Snehal Raut
sraut@integrisprojects.com

I certify that the above submitted items have been reviewed in detail and are correct and in the strict conformance with the contract drawings and specifications except as otherwise stated.

Sidhartha Sen

NAME AND SIGNATURE OF CONTRACTOR

SECTION II - APPROVAL ACTION

ENCLOSURES RETURNED (List by item No.)	NAME, TITLE AND SIGNATURE OF APPROVING AUTHORITY	DATE

SUBMITTAL COVER LETTER

PROJECT NO: W91278-16-D-0084 0001

Offeror: Integris Projects, LLC	**DATE:** 24 OCT 2016
Bioengineered Breakwater Technical Report for **Bayou Caddy Shoreline Stabilization, Bayou Caddy, Hancock County, Mississippi**	

Summary: This Document (Cover Letter) is to notify the submission of Bioengineered Breakwater Technical Report for Bayou Caddy Shoreline Stabilization, Bayou Caddy, Hancock County, Mississippi

Details: The work under this contract consists of shoreline restoration work at Bayou Caddy in Hancock county, Mississippi. The scope includes:

1. Construction of temporary access channel
2. Installation of Rubble Mound Breakwater
3. Installation of Bioengineered Wave attenuation devices (WAD)

The Contractor shall install approximately 1,000 linear feet of a rubble mound breakwater that will tie into land at the southern end of the Bayou Caddy site. The rubble mound breakwater will be trapezoidal in shape and will be built to a height of 4 feet NAVD88. The structure will consist of approximately 1,500 cubic yards of ALDOT Class I stone as well as approximately 3,900 cubic yards of ALDOT Class V stone. Approximately 39,000 square feet of filter fabric will be placed under the rubble mound breakwater along the entire length of the alignment. All quantities mentioned above are approximate.

The Contractor shall also install five (5) separate sections of bioengineered breakwater structures. The envelope for these sections to be placed in will be 300 feet in length, 30 feet in width and a height not to exceed 1.5 feet NAVD88. The sections will have approximately 50 feet of spacing between them and will begin 50 feet from the end of the toe of the stone dike structure.

The Contractor shall follow the alignments for both the rubble mound breakwater and the bioengineered breakwater structures as indicated in the task order plans and specifications.

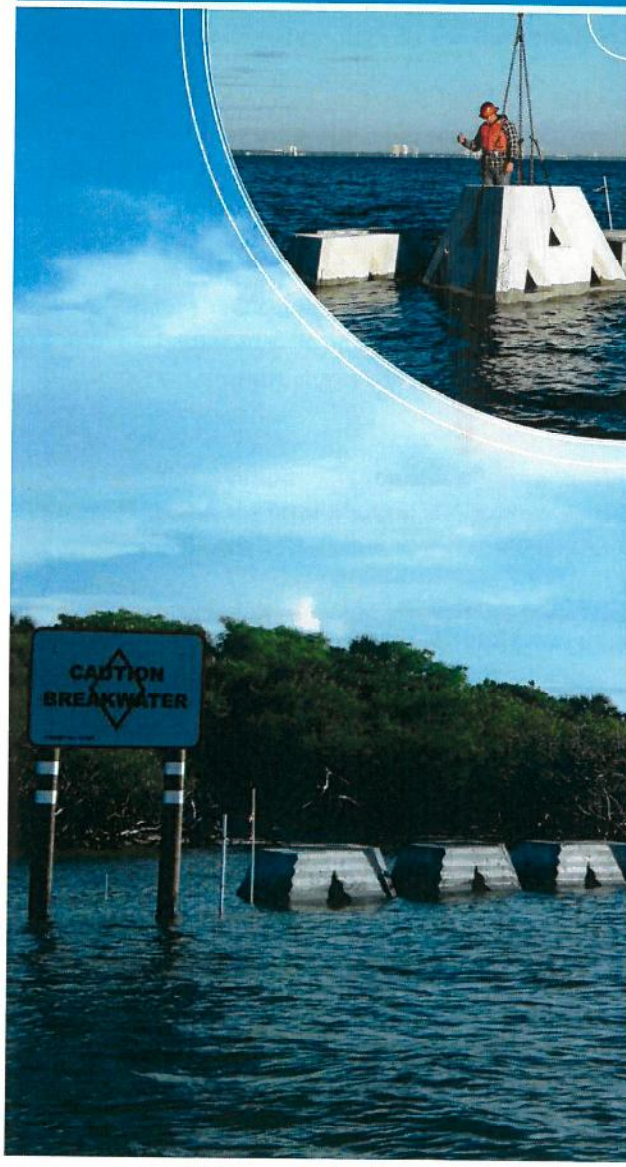
Offeror POC:

Danny Rackard
Project Manager
drackard@integrisprojects.com
713-920-7123 (w)
251-597-0849 (c)

Offeror Signature:

Technical Report of Bioengineered Breakwater WADs for Bayou Caddy Shoreline Stabilization Project. Hancock County, Mississippi

Living Shoreline Solutions Inc Technologies



October 2016

Integris Projects LLC

IN CHRIST WE TRUST

Prepared for:

USACE District, Mobile
P.O. Box 2288
109 Saint Joseph Street
Mobile, Alabama 36602

Living Shoreline Solutions Inc.

US PATENT 6,186,702 B1 WAD
www.livingshorelinesolutions.com

Technical Description

The Living Shoreline Solution's (LSS) & CDM Smith Team is well qualified to evaluate, design, fabricate, and implement a unique solution for the Bayou Caddy Shoreline Stabilization Project. LSS's Wave Attenuation Device (WAD)® LSS technologies are a proven, environmentally productive solution to reduce, eliminate or reverse coastal erosion issues around the globe. LSS has over 19 years' experience in engineering and design of its' patented technology for Infrastructural Protection, Shoreline Stabilization, Productive Oyster Barrier Reef development, Protection of Wetland Restoration Sites, Dredge Material Placement Sites, General Shoreline Protection, Barrier Island Restoration including Marsh Creation and Stabilization.

In this section, we present for your evaluation our patented technology for erosion control and how it can be tailored for each application listed above. You will find that our technical approach and capability is sound and feasible, that we have a clear understanding of the desired project outcomes and we bring an effective team for the modeling, engineering and design as well as construction and installation of the solution which, have been successfully implemented throughout the world with a verifiable, 100% success rate.

At the core of our proposed solution is the Wave Attenuation Device® (WAD®) technology, a one of a kind product which augments near shore sediment transport thereby mitigating shoreline change. Traditional methods to mitigate erosion typically rely on a hardened structure to retain sediments in place. In the Mississippi coastal zones, these approaches have failed for two reasons:

1. The near shore soils do not have the bearing capacity to sustain large hardened structures (e.g., rock or solid concrete features). Commonly, these types of structures will sink into the soft substrate within 5-10 years of construction
2. This solution shifts the erosion issue "downstream". By holding sediments in place, the littoral drift is interrupted which results in depriving the downstream location of their sediment source and increasing the rate erosion

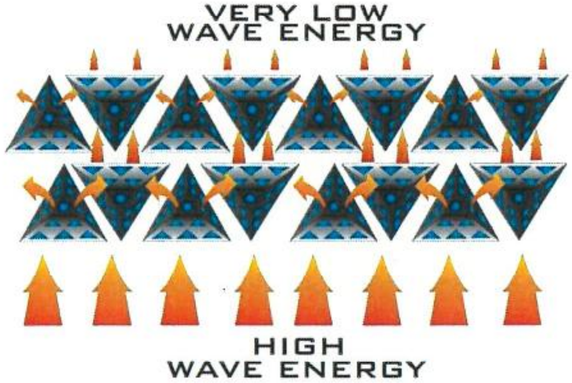
Benefits of WAD® System

- Open base requires less bearing capacity
- Alters near shore wave and current climate
- Results in accretion regularly
- Modeling and design tailored to site conditions
- System adaptable for unique conditions of each site

The WAD® system offered by LSS is a unique, proprietary, patented technology which provides improvements on traditional methods previously used on the Gulf Coast. WAD®s require less bearing capacity of the underlying sediments and will resist sinking into the near shore sediments. More significantly, WAD®s are not intended as retention structures. The WAD® technology alters the near shore wave and current climate to the point at which sediments fall out of the water column and accrete in the near shore. This process is described in greater detail in the section below. For the Bayou Caddy Coastal site, as identified in the SATOC Task Order for this project, a Site-specific WAD® has been designed and engineered through analysis of sediment conditions, bathymetric conditions, wind and wave energy, to include 20-100-year storm events to "attenuate 90% wave energy" at the site. The WAD® units will be placed in a double row, parallel to shore and perpendicular to typical wind and wave energy direction. Each WAD® unit is a three-sided pyramid shaped structure with angled sides and tapered openings that attenuate wave energy. The Bayou Caddy WAD will have a false bottom to meet the not to exceed 275 PSF load factors. (Bayou Caddy WAD load factor is 190 PSF)

As the waves impact the WAD® structure, the wave energy is deflected in the upper plane which directs any wave energy and resultant scour away from the bottom. That energy is deflected upwards and assists in impacting the horizontal component of inbound wave energy. As the wave energy enters the 6 openings on the front side of the WAD®, energy is increased to move inside the WAD® and out through the top to impact horizontal wave energy going over the WAD®. The remaining wave energy is then directed out through the back two sides of the WAD® through 12 openings that are tapered outwardly so the pressure is increased and velocity of the wave fluid is decreased significantly. This decrease in energy continues through the transmission of the wave fluid between and behind the WAD® array and allows any sediment suspended in the fluid to fall to the bottom and be deposited shoreward on the shoreline.

Unlike hardened structures, the WAD®s allows the wave fluid to transmit through the WAD® array and not be deflected or reflected where it would cause the wave energy to negatively impact the surrounding shoreline. The WAD® array and engineered design for each particular application yields 90% wave energy attenuation, stopping any further erosion and contributing to sediment accretion shoreward of the WAD® array, reversing erosion, dependent on heavy wave energy events and sediment source availability.



Wave Transmission of Bayou Caddy WAD®

Living Shoreline Solutions Inc., performs CMS-Wave modeling on all its projects sites. Per Mr. Joseph Black's USACE E-mail of July 15, 2016, LSS is providing "Documentation showing the performance of the bioengineered breakwater technology on previous projects (should suffice)." The Modeling Executive Summary below, performed by Dr. Ping Wang Ph. D. Department of Geology, USF, of a similar WAD® configuration to that of Bayou Caddy, was performed for off the coast of Mexico with more extreme conditions than can be expected at Bayou Caddy.

H_s Wave Height (feet)	4.0
T_p Peak Period (sec)	4.5
Surge Magnitude (feet)	6.2
Wind Speed (knots)	49.2

Executive Summary

This report describes the wave modeling and engineering design of a Wave Attenuation Device (WAD®) system for the protection of the 1600-m El Dorado Royale shoreline. The design is based on: 1) a literature research and a statistical analysis of meteorological and oceanographic conditions, 2) a detailed bathymetric survey of the greater study area, and 3) engineering design of the WAD® system.

Two WAD® systems with a base of 3.0 m wide will be used to accommodate the varying water depth at the project area. The taller unit is 2.5 m high, while the shorter unit is 2.0 m high. Overall, 1120 2.5-m units and 372 2.0-m units will be used, totaling 1492 WAD® units to protect the 1600-m shoreline. This Executive Summary provides an abbreviated description of the several aspects of the design and modeling of the WAD® system, as listed below:

Cross-sectional Design of WAD®s System: The cross-section design of the WAD®s is based on the surveyed beach profiles. Twenty-six surveyed beach profiles are used in the design. Overall, the WAD®s units will be approximately 50 m from the shoreline. The WAD® unit will approximately extend 0.6 m above Mean Higher High Water (MHHW). The 0.6 m above MHHW is designed to accommodate wave run-up and modest storm surge.

During rare major storms with surge of more than 0.6 m, the WAD®s units will be submerged. Substantial wave energy dissipation of over 85% is still anticipated during rare major storms. Two arrays of the WAD®s units consisting of 4 individual WAD®s will be installed at each cross-shore location. Detailed location (UTM Zone 16 in meters), local water depth (relative to Mean Sea Level in m), WAD® height, and a WAD® Identification at each WAD® installation is provided in this report. It is worth noting that the MSL is approximately 0.12 m above MLLW and 0.12 m below MHHW at the project site.

Design of WADs Array Layout and Wave Modeling: Three design alternatives are evaluated using the CMS-Wave model. The CMS-Wave model is developed by the US Army Corps of Engineer Research and Development Center (USACE-ERDC) and is broadly used in modeling nearshore wave propagation over complicated bathymetry. The CMS-Wave is suitable for the design of the WAD®s system at El Dorado Royale beach. The three design alternatives investigated here include 1) a continuous WAD®s array along the entire shoreline with no gaps, 2) a segmented WAD® array with 4-m gaps every 48 m, and 3) a segmented WAD® array with 8-m gaps every 48 m. In addition to the regular gaps, two large gaps to accommodate the present beach launch of Jet Ski boats were designed. A 4m x 4m modeling grid, roughly parallel to the regional shoreline orientation is created for the wave modeling. Based on statistical wave analysis at the National Data Buoy Center Station 42056 (Yucatan Basin), an energetic wave condition with 2-m wave height and 4.5-s peak wave period was used in the design wave modeling. The 2-m wave represents less than 2% frequency of occurrence. Based on the bathymetry conditions at the study site and the depth-limited wave breaking, waves higher than approximately 3 m are not expected without substantial storm surge. Three wave approaching angles were evaluated, including 1) a shore perpendicular incident wave, 2) a northerly approaching wave, and 3) a southerly approaching wave. The present modeling represents energetic wave conditions expected from frequent storm, e.g., storms with more than 2 occurrences annually up to 5 to 10-year storms. The top of the WAD units, designed to be about 0.6 m above MHHW, is roughly equal the elevation of the present dry beach. This should provide adequate protection again wave attacks under both normal conditions, "regular" storm wave conditions, as well as extreme storm wave conditions. The large storm surges associated with extreme storms are not incorporated in the present design of the WAD®s.

Recommended Design of WAD®s array Layout and Wave Modeling: Based on the modeling results, the **second alternative with 4-m gap every 48 m is recommended**. The detailed engineering AutoCAD drawing of the recommended WAD®s layout is provided with this report. The small gaps have limited influence on the overall wave-energy reduction, while providing convenient access to the Gulf for swimmers during calm days. Over 95% of wave-energy reduction is expected. A tapper is designed at both the northern and southern ends of the project to minimize potential impact to the adjacent beach, especially the beach to the north. The tappers are designed to limit the permanent trapping of longshore moving sand behind the WAD®s. Therefore, the beach sand that is temporarily impounded, e.g., during the northerly approaching waves can "move back" to its original locations when waves approach from the south. Based on the statistical wave analysis, there is no significant preference between northerly and southerly approaching waves. By not trapping sand with the tapper design, the WAD®s arrays will not have negative impacts to the adjacent beaches. Considerable amount of wave energy propagates through the relatively wide entrance for Jet Ski. Overlap of WAD®s array at the entrance is recommended. Detailed design of the overlap is not included in this phase of the design and can be incorporated as part of the minor design modification before or during construction.

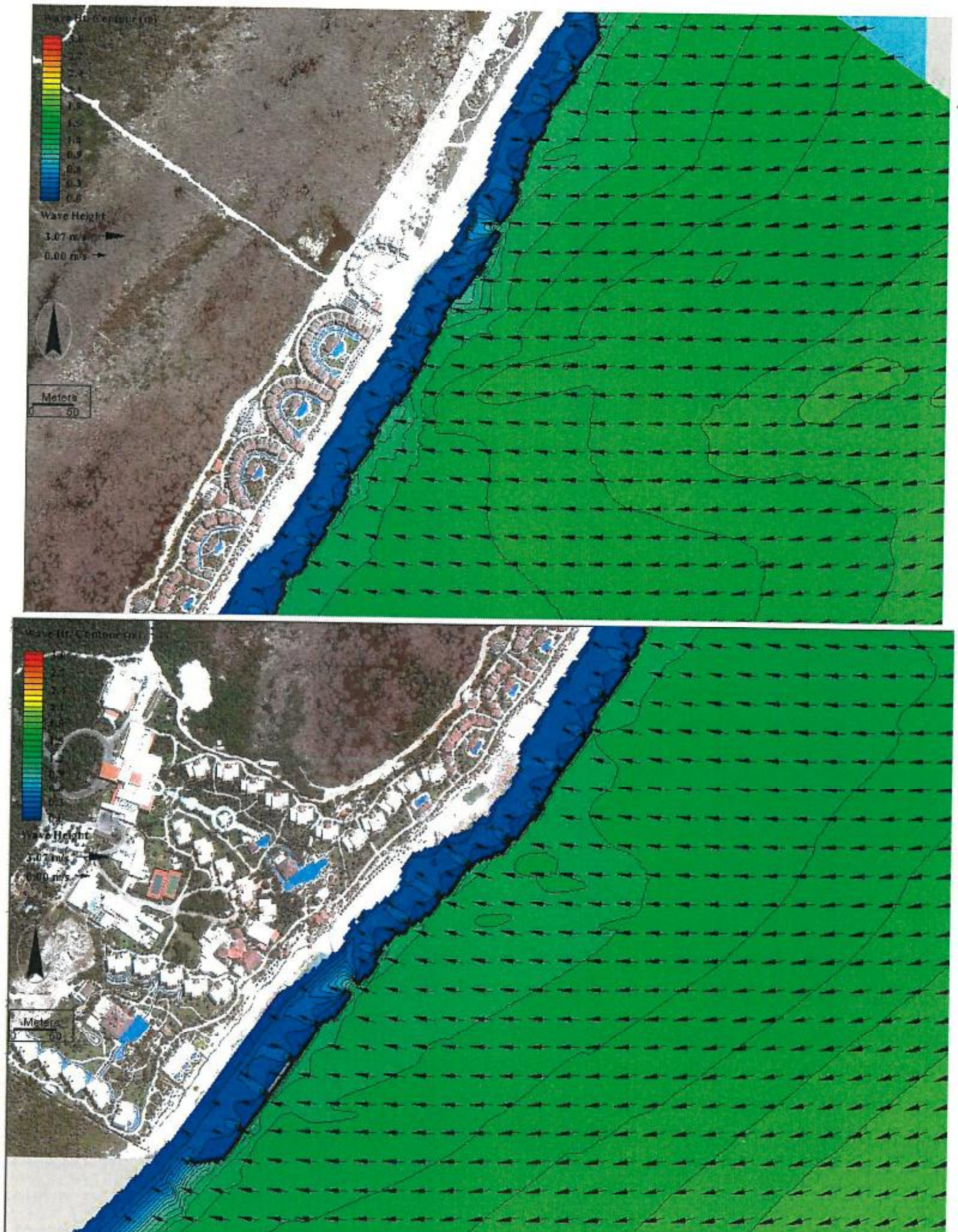


Figure 1. Design of the WAD®s layout along the shoreline. Upper: the northern half of the project. Lower: the southern half of the project. Northerly incident wave with 2-m wave height and 4.5-s wave period. 4-m gaps every 48 m of the WAD®s array in addition to Jet Ski entry. Wave height landward of the WAD®s is mostly less than 0.25 cm, as compared to over 1.2 m seaward. The wave propagation through the narrow gaps is limited to its immediate vicinity.

The general layout of the WAD®s for the Bayou Caddy site, including 1) the 4 WAD cross sectional overlapping design, 2) the approximately 2-feet extension above mean sea level, and 3) the very small gap, bears significant similarities to the El Dorado Royale design. Bayou Caddy is more shallow, with higher and longer breakwaters. Wave heights are smaller at Bayou Caddy and with the small gaps, wave propagation through the gaps is limited to the immediate vicinity. Also, distance to shore and shallow water will reduce the likelihood of any regeneration. These are key design factors influencing the performance of the WADs arrays. Therefore, similar performance as that modeled at the El Dorado site should be expected at the Bayou Caddy site despite the very different general oceanographic and morphologic characteristics of the two sites.

Actual Field Demonstration of WAD®s Attenuation

This is an executive summary of a fuller study conducted by Dr. Ping Wang Ph. D. Department of Geology, USF. The study area is located along the north shore of Pensacola Bay west of the Pensacola Three Mile Bridge The study site is open to a substantial fetch to the south and southwest. Figure 1 shows the aerial view of the study site and the locations of the wave measurements. The LSS WAD® array ®serve as a continuation of an array of rubble mound breakwaters. The east end of the structure is about 50 meters from the bridge. The WAD®s were installed in water depth of approximately 1.5 m.

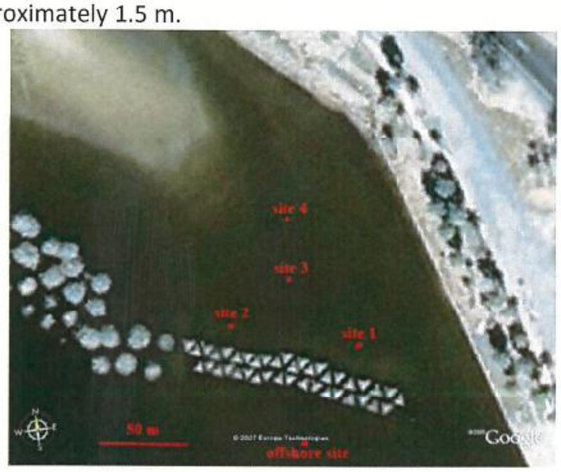


Figure 2. Aerial view of the study site and locations of the wave measurements.

The measurements were conducted on the afternoon of July 25th, 2007 during a falling tide. The water level during the measurement period was close to mean water level. The tidal range in the study area is small, roughly 0.3 m. As typical of summer conditions, land breeze with offshore direct wind dominated in the morning. The study site was calm with minimal wave activity. In the afternoon, landward directed sea breeze generated choppy waves at the study site. The data collected by this study should represent a typical summer sea breeze conditions.

The wave measurements were conducted at five different locations, including one offshore site and four sites in the protected area (Figure 2). The offshore site is roughly 30 m seaward of the structures with a water depth of 1.5 m. Site 1 in the protected area was selected to be close to the gap between the structures and the vertical seawall along the bridge, with a water depth of 1.4 m. The purpose of site 1 is to quantify the influences of wave transmission through the gap. Field observations also indicated considerable wave reflection from the vertical seawall. Wave conditions at site 1 are also influenced by the reflected waves. Site 2 is located near the western end of the structures. Based on field observation, limited wave energy was transmitted through the narrow gap between the CRI wave attenuation devices and the rubble mount breakwaters. Also, site 2 is relatively far from the seawall and should have a significantly reduced influence as compared to site 1. Site 3 is approximately one half of the length of the CRI structure landward. Site 4 is approximately one length of the CRI structure landward. The goal of sites 3 and 4 is to examine the extent of the wave sheltering by the CRI structures. It is worth noting that the wave attenuation devices and the rubble mount structures form a nearly continuous array of structure. The protected area is different from typical segmented breakwaters.

EQUIPMENT, SAMPLING SCHEME, AND DATA ANALYSIS

Two Sontek Triton wave gages were used for the field measurements. Although the Triton is capable of collecting directional information, only non-directional information, i.e., wave height and wave period, are used here. One Triton wave gage was deployed offshore and remained at the offshore site during the entire measurement from 13:30 to 18:00 eastern standard time. This gage provided a continuous offshore wave conditions for comparison. One wave gage was used to measure the wave conditions in the protected area (referred to as the nearshore gage in the following). This gage was moved from site 1 through site 4. The wave gages were mounted on a stable steel pipe which was pounded into the substrate. To better capture the high frequency wind waves, the wave gages were sampled at 4 Hz for 128 seconds. Given the high frequency wave, with a peak period of about 2 second, the 128-second sampling is adequate to provide reliable statistical values. The wave gages were installed as close to the water surface as possible to minimize signal attenuation through the water column. The wave measurements were conducted every 10 minutes. The nearshore wave gage was installed at each site for one hour to provide 6 measurements at each nearshore site. Standard frequency domain analysis was conducted to compute significant wave height and peak wave period. Although the wave gages were close to the water surface, standard depth-attenuation correction was still conducted to ensure adequate representation of high-frequency information. Wave analysis was conducted using the power spectral analysis module in MATLAB.

RESULTS AND DISCUSSION

Wave height and energy reduction caused by the WAD®s is apparent from visual observation (Figure 3). Much calmer water is observed landward of the structure. Considerable amount of wave energy entered the protected area from the gap between the structure and the seawall (Figure 5). In addition, waves reflected from the seawall are also visible in Figure 4. The reflected waves tend to be somewhat better organized than the choppy wind waves.

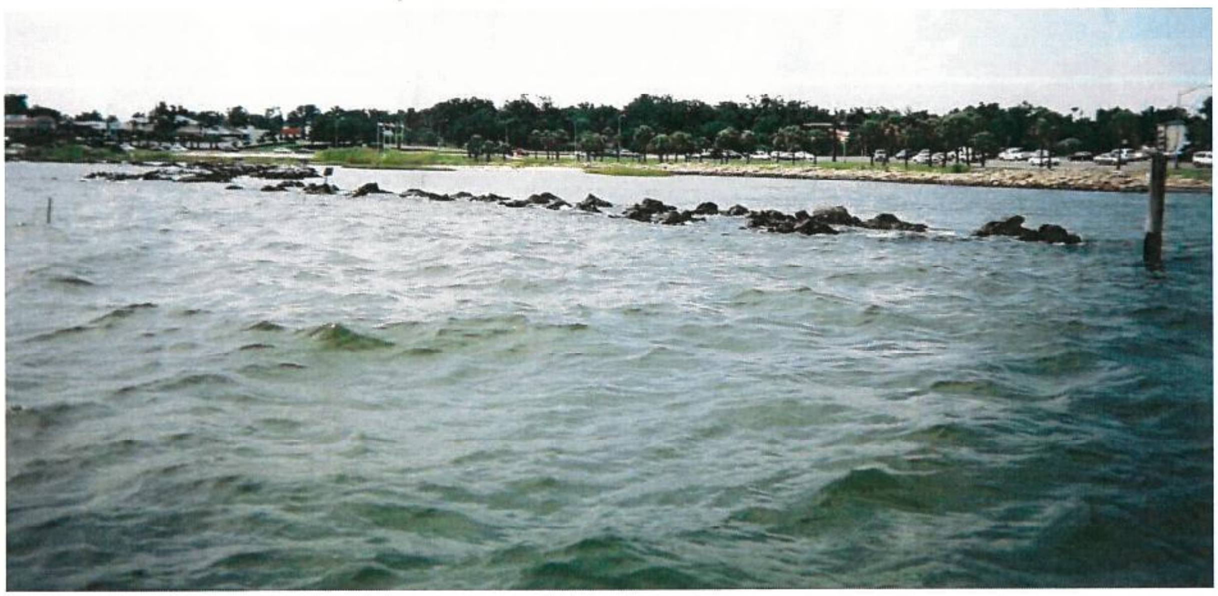


Figure 3. Visual comparison of the wave conditions seaward and landward of the structure.

Figure 4. Wave propagation through the gap between the structure and the seawall (below the bottom of the picture). Note the reflected waves propagating nearly parallel to the structure.

Measured Wave Conditions

Overall, the measured wave conditions confirm the qualitative visual observations, as expected. It is worth emphasizing again that the wave height and energy reduction described below pertains directly to the specific configuration of the Greenshores WAD® structure. Different structural configuration may result in very different patterns. Significant wave height reduction was measured at all the sites, although Sites 2, 3, and 4 showed much greater reduction than Site 1. Site 1 is influenced by the wave transmission through the gap between the seawall and the structures, and the reflected waves from the vertical seawall.

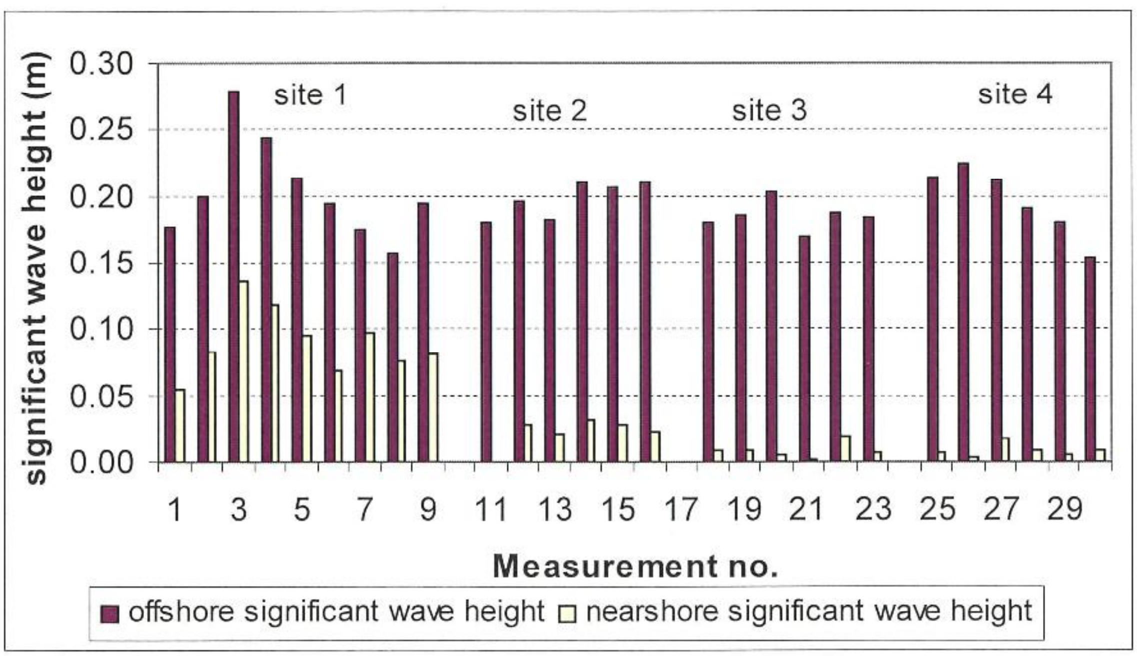


Figure 5. Summary of the measured significant wave heights at the offshore and nearshore sites.

Figure 6 summarizes the percent reduction of significant wave height and wave energy. For nearly all the measurements at the four sites, the wave height was reduced by more than 50% and wave energy was reduced by more than 80%. The relatively less wave-height and wave-energy reduction at Site 1 is caused by wave transmission through a substantial gap plus reflected wave from the vertical seawall. Three of the four sites in the protected area showed over 80% wave-height reduction and nearly 100% wave energy reduction for the particular structure configuration and incident wave conditions.

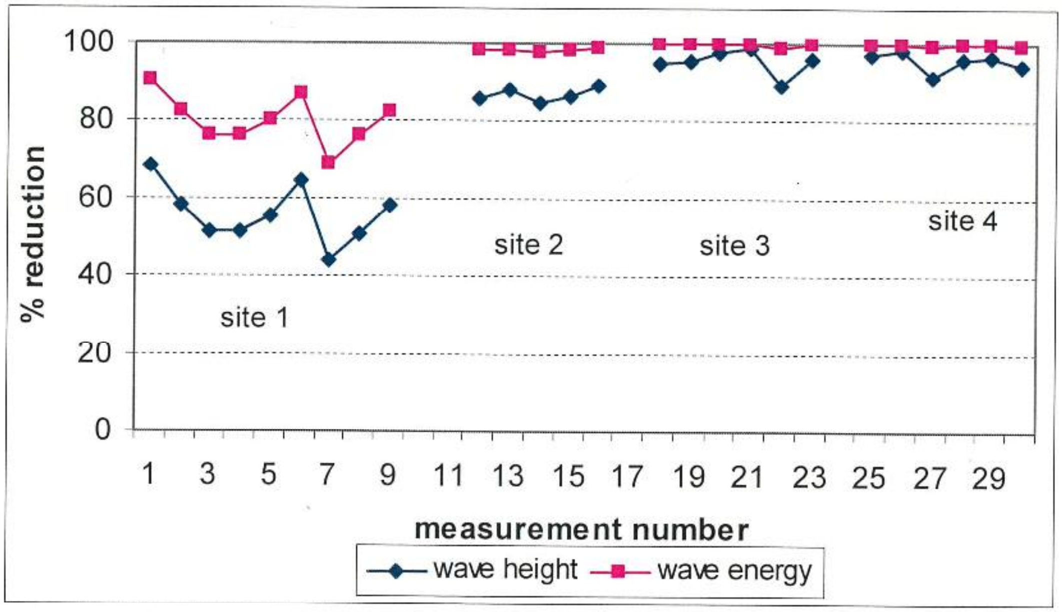


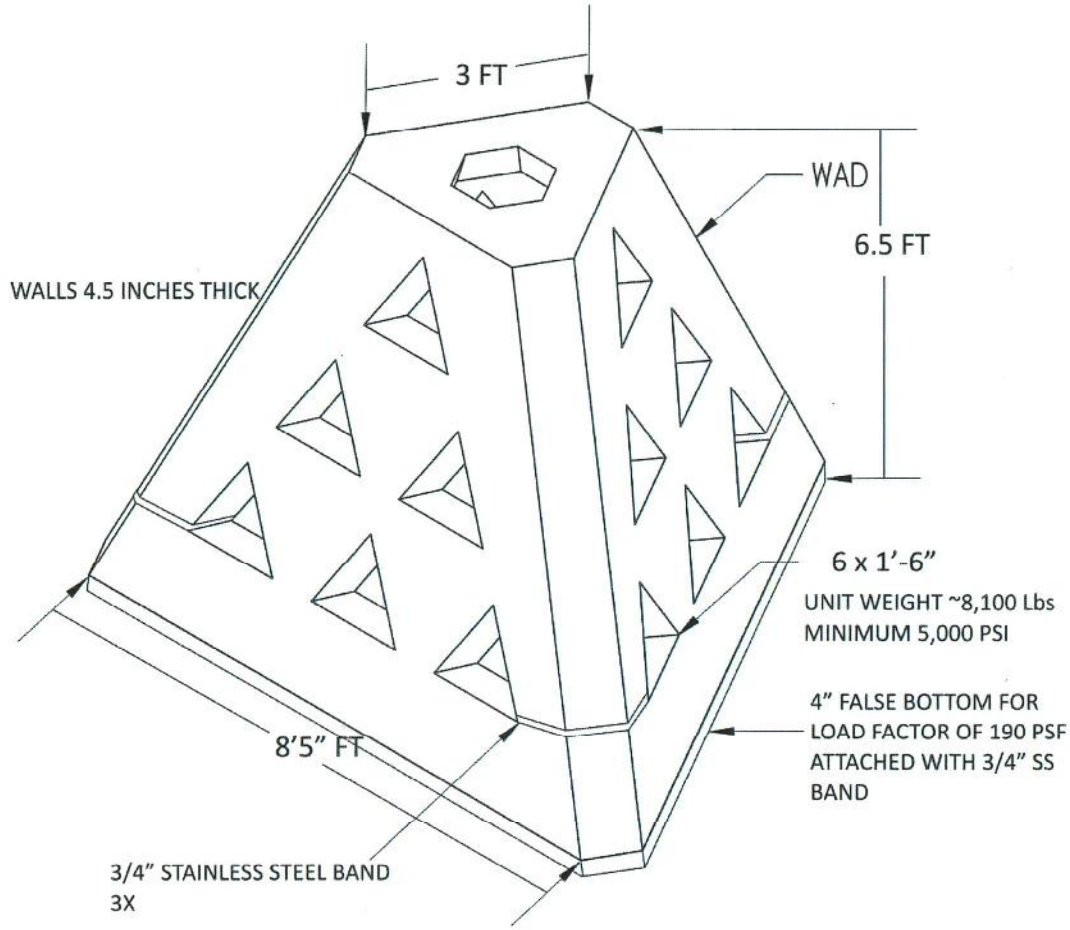
Figure 6. Summary of the percentage wave-height and wave-energy reduction.

SUMMARY

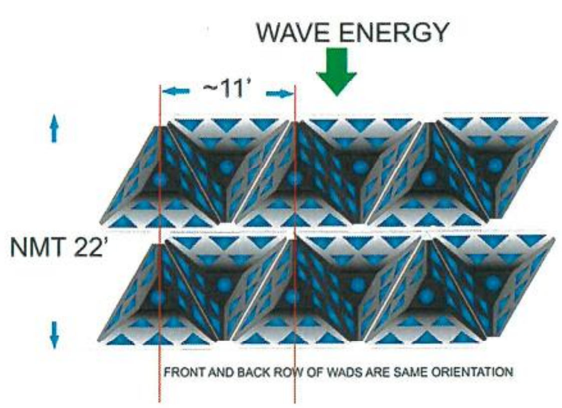
Overall, substantial wave-height and wave-energy reductions were measured at all the four sites in the protected area of the LSS WAD® array. At Site 1 with considerable wave transmission, the incident wave height was reduced by 56% on average, with an average wave-energy reduction of 80%. At Site 2 with limited wave transmission from offshore, the incident wave height was reduced by 87% on average, with an average wave-energy reduction of 98%. At Sites 3 and 4 further landward of the structure, due to the dissipation of the transmitted waves, both the incident wave height and wave energy was reduced by nearly 100%. This project was installed 2 months before Hurricane Ivan slammed the Gulf Coast.

Dimensions Bayou Caddy WAD®

The Bayou Caddy WAD® stands 6.5 feet tall with a footprint of approximately 11 feet, point to point. It is constructed using steel forms, with a designed 6,000 psi mix of marine grade concrete dosed with 3 pounds of 650 Fibermesh per cubic yard of concrete. There is no rebar in this WAD® structure. Unit weight of the WAD® is approximately 8,100 pounds, with a wall thickness of 4.5 inches. LSS utilized half of the elevation construction tolerance to maximize wave energy attenuation over a larger surge and wave profile. There is a bottom unit attached to the WAD® via stainless steel strap to minimize load factors that are driven by geotechnical considerations at the site. The stainless steel strap is type 201, warranted to meet ASTM-A666 standard, including but not limited to chemical composition requirements of carbon manganese, sulfur, silicon, chromium, and nickel. Tensile and break strengths of 95,000 psi and 3,200 psi, respectively. Base of unit is only 1800 pounds and is attached in the corners of the WAD at equal distances from each other. (Stainless strap is for lifting and placing WAD® unit in the water). WAD® sits on tongue and groove base and weight of WAD® will hold unit together.



WAD®s will be aligned front to back in identical orientation with the bases of each unit touching for increased stability. Orientation will be parallel to shore, per the engineering drawings in five, 300-foot-long sections of breakwater. Front to back construction tolerance will not exceed 22 feet, well within the 30-foot requirement.



Stability of Bayou Caddy WAD®

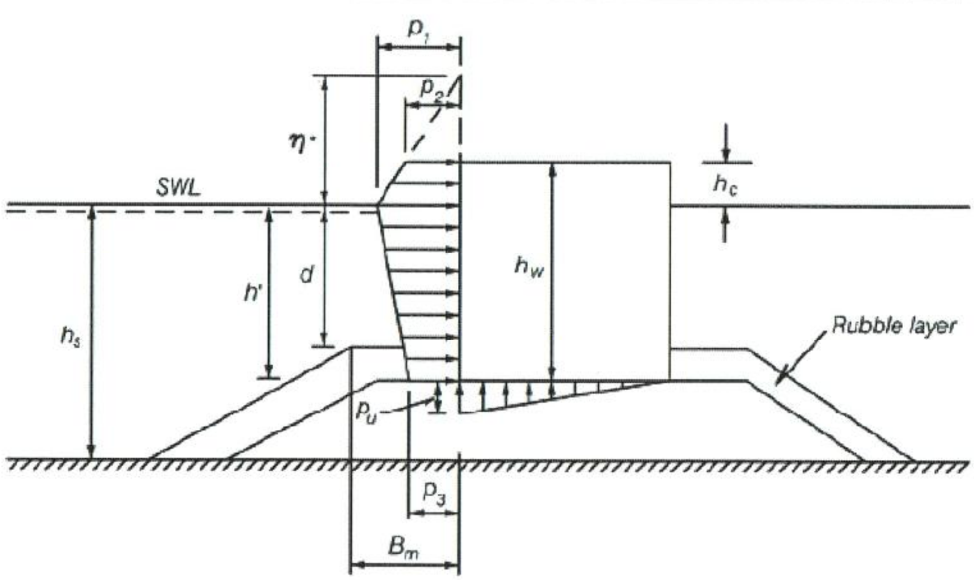
Dr. Ping Wang has provided the stability analysis and it should be noted that this analysis is done with a single WAD®. The WAD®s are installed with the front, back and side walls touching each other for increased stability. Initial prototypes with 10-foot base and 6 feet tall weighing half as much as the bayou caddy WAD® were lab tested and a wave large enough could not be generated at the facility to cause either movement or tipping. All WAD®s over the past 19 years have resisted movement and or tipping despite numerous hurricanes and tropical storms.

Wave-induced pressure on WAD®s

The most relevant analyses are by Goda (1974) and its modified version by Tanimoto and Kimura (1985) for inclined impermeable wall. No formula is available for partially permeable wall (the case of WAD®s). In theory, the forcing on partially permeable wall should be less than that on the impermeable wall. Therefore, the forcing on the WAD®s should be less than the values estimated here.

Various equations were developed by Goda to calculate the distribution of pressure induced by non-breaking waves. For our case, the waves should be mostly non-breaking with the possibility of white capping during high wind conditions. Therefore, the non-breaking wave equations are the most relevant. The pressure distribution at a vertical wall is illustrated by Goda (1974) as:

Irregular Waves (Goda 1974; Tanimoto et al. 1976) (Continued)

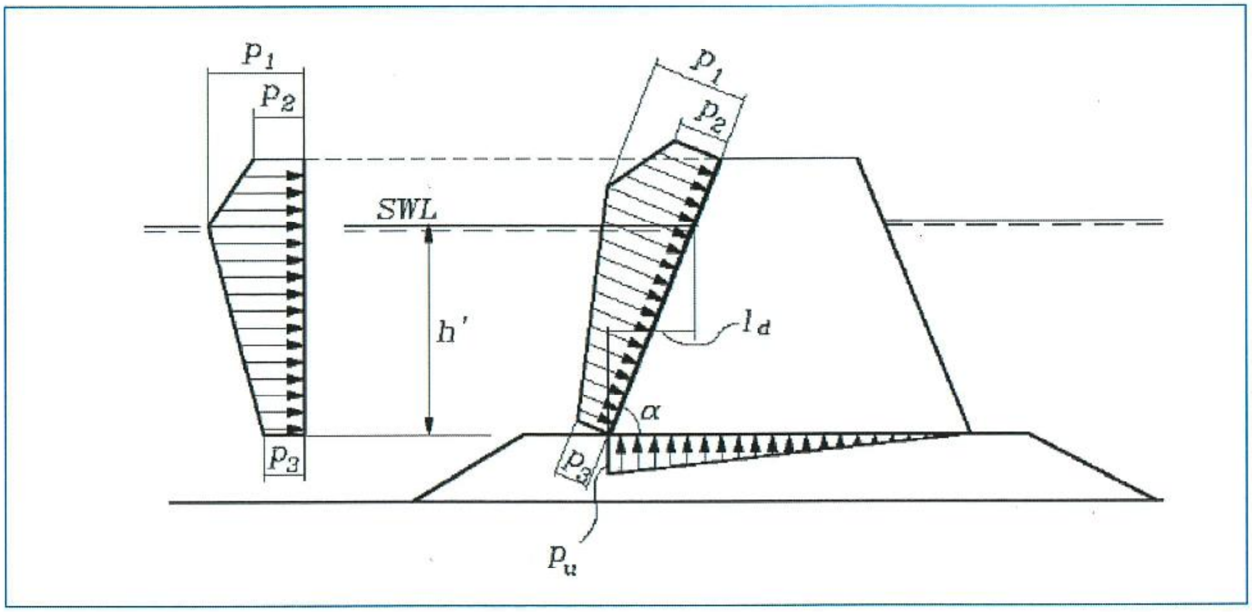


The following characteristics can be summarized from Goda pressure analysis:

1) The pressure is the highest at the mean sea level (p_1);
2) The pressure decreased both upward (p_2) and downward linearly from the mean sea level (p_3).
3) The pressure on the wall creates an upward pressure (p_u) on the bottom.

It should be noted that Goda's analyses focus on the stability of the structure to ensure that it will not be turned over by wave forcing.

Tanimoto and Kimura (1985) modified Goda (1974) illustration for an inclined wall as shown below. This modification resembles the case of WADs the closest and is therefore used here for stability and loading analysis.



The Goda (1974) equations original developed for vertical wall are largely applicable to the inclined wall case with a minor modification as illustrated in the above figure. It is worth noting that in the case of an inclined wall, the pressures p_1, p_2, and $p_3$are perpendicular to the inclined wall. The following formulae were developed to calculate the four pressure components (p_1, p_2, p_3, and p_u):

$$\eta^* = 0.75(1+cos\beta)\,\lambda_1 H_{design} \quad \text{(VI-5-147)}$$

$$p_1 = 0.5(1+cos\beta)(\lambda_1\alpha_1+\lambda_2\alpha_* cos^2\beta)\,\rho_w\, g\, H_{design} \quad \text{(VI-5-148)}$$

$$p_2 = \begin{cases} \left(1-\frac{h_c}{\eta^*}\right)p_1 & \text{for } \eta^* > h_c \\ 0 & \text{for } \eta^* \leq h_c \end{cases} \quad \text{(VI-5-149)}$$

$$p_3 = \alpha_3\, p_1 \quad \text{(VI-5-150)}$$

$$p_u = 0.5(1+cos\beta)\lambda_3\alpha_1\alpha_3\rho_w g\, H_{design} \quad \text{(VI-5-151)}$$

For Bayou Caddy WADs case, H_{design} is provided to be 1.2 m (4 ft) based on guidelines, β = 0 for normal incident wave, h_c = 2.0 m (6.6 ft) for water depth. The installation water depth is designed to be at 3.5 ft. Including 50% of the design storm surge at 6.2 ft, the resulting water depth is 6.6 ft or 2 m. The bioengineered breakwater does resist overturning and sliding when the still water level is at the crest of the structure and a wave height equal to 0.8 times the total water depth at the seaward toe of the structure. The incident wave period is 4.5 s as provided by the guideline. The installation water depth is 3.5 ft. The 0.8 times the water yielded 2.8 ft or 0.85 m. The following analysis used a wave height of 1.2 m, or 3.9 ft. This exceeded the "0.8 times water depth" requirement. Therefore, the analysis provided a larger instability potential.

The rest of the parameters are calculated as:

$$
\begin{aligned}
\alpha_* &= \alpha_2 \\
\alpha_1 &= 0.6 + 0.5\left[\frac{4\pi h_s/L}{sinh\left(4\pi h_s/L\right)}\right]^2 \\
\alpha_2 &= \text{the smallest of } \frac{h_b - d}{3h_b}\left(\frac{H_{design}}{d}\right)^2 \text{ and } \frac{2d}{H_{design}} \\
\alpha_3 &= 1 - \frac{h_w - h_c}{h_s}\left[1 - \frac{1}{cosh\left(2\pi h_s/L\right)}\right]
\end{aligned}
$$

L Wavelength at water depth h_b corresponding to that of the significant wave $T_s \simeq 1.1T_m$, where T_m is the average period.

$\lambda_1 = \lambda_2 = 1$

$$\lambda_3 = exp\left[-2.26(7.2\,\ell_d\,/L)^3\right]$$

where $\ell_d = h' \cot \alpha$ and L is the wavelength.

The parameter λ_3 accounts for the inclined wall case. The angle α is determined based on the WAD design. For the present version of WAD design cotα = 3/7.

Pressure Calculation

Input parameters are summarized below:

H_{design} = 1.2 m (incident wave height)

T_{design} = 4.5 s (incident wave period)

L_{design} = 9.3 m (incident wave length)

β = 0 (incident wave angle)

ρ_w = 1025 km/m3 (density of seawater)

g = 9.81 m/s2 (gravitational acceleration)

h_c = -0.02 m (the WAD unit will be slightly submerged under this condition)

$h_s = h' = d$ = 2.0 m (6.6 ft)

h_w = 1.98 m (height of the WAD: 6.5 ft)

$B_m = 0$ m

$\cot\alpha = 3/7$

$\lambda_1 = \lambda_2 = 1$

Calculated parameters are summarized below:

calculated parameters		
eta_*	1.8	m
p_1	10635.38	Pascal
p_2	10753.55	Pascal
p_3	2490.572	Pascal
p_u	2293.228	Pascal
α_*	0	
α_1	0.881412	
α_2	0	
α_3	0.234178	
L	18.6	
h_b	2	
λ_3	0.920764	

The computation applied here is from the Coastal Engineering Manual published by the USACE. Based on the sketch of the inclined wall case by Tanimoto and Kimura (1985), the pressure that would lead to a possible overturn of the structure is p2 because it has the longest distance to the pivot point at one of the corners of the WAD unit. Based on the calculation, p2 is slightly less than 11000 pascals, which equals roughly 1.6 psi. The submerged weight of the WAD unit is 5500 pounds. This very small wave induced pressure is far from causing instability of the WADs unit. The instability is defined here as sliding or overturning of the structure. It is highly unlikely that this small pressure forcing can overcome the friction at the bottom to cause the WAD unit to slide. This small force is also very far from overturning the 5500-pound unit. Assuming the weight of the unit can be represented in the center of the unit. To overturn the unit with a force at the top edge which is twice as far from the pivot point as the center of mass, that force needs to be 2750 pounds. The small 1.6 psi pressure would not result, by a large margin, in such a force.

Environmental Benefits Bayou Caddy WAD®

The Coastal Haven WAD® series modules were methodically designed to promote and accelerate rapid marine growth on all hard substrate surfaces, both inside the structure and out and specifically engineered to act as wave attenuation devices to protect the world's coastlines. The hydrodynamically engineered shape of the units and the designed openings were specifically developed to promote a "designed flow of micronutrient rich water" across all surfaces. Combined with the engineered light access, these characteristics lead to unsurpassed biomass development and reef productivity. This highly functional design provides the greatest amount of productive, hard substrate and spawning habitat/shelter of any commercial product available. The Coastal Haven WAD® series modules are the largest, most complex, "designed" wave attenuation systems on the market. These units provide a large marine developmental substrate and a more stable and taller profile essential for marine productivity and biomass development. These characteristics also allow these designed units to attenuate damaging wave energy thus reducing possible shoreline beach erosion. LSS has placed thousands of WADs throughout the world in similar configurations and nearshore environments to that of Bayou Caddy. Just 70 miles East of the Bayou Caddy at Dauphin Island's WAD® Breakwater, Auburn University Shellfish Laboratory performed a long-term monitoring program of Eastern Oyster density compared with pre-deployment sampling. "The WAD®s provided excellent hard substrate on which oysters have established. These oyster reefs provide various ecosystem services, including improvement of water quality and creation of a foraging area for fish. A random sample was taken after one year on the WAD® and oyster density on individual units was 205 oysters per square meter compared to 150 oysters per square meter on Alabama's MOST PRODUCTIVE REEF." (emphasis added). The report continued, "high oyster density is likely a result of the vertical relief provided by the units and due to the fact that oysters set equally well on the inside surface of the open structures as on the outside surface. Colonization of the units by oysters increases the total volume of the breakwater resulting in additional wave attenuation. Other ecosystem services provided by the breakwater include refuge habitat for aquatic animals and the use of the exposed portions of the breakwaters by colonial sea-birds (Yozzo et al. 2003). The foraging and refuge habitats are essential for locally important species such as: spotted seatrout (also known as speckled trout) *Cynoscion nebulosus*, blue crabs *Callinectes sapidus* and Gulf stone crabs *Menippe adina*, eastern oyster *Crassostrea virginica*, red drum *Sciaenops ocellatus*, southern flounder *Paralichthys lethostigma*, and various species of commercially important shrimp (brown shrimp *Farfantepenaeus aztecus*, pink shrimp *F. duorarum*, and white shrimp *Litope- naeus setiferus*."

Over the past 19 years, LSS has seen consistent above average performance in marine biomass productivity on all its breakwater systems and the same should hold true for Bayou Caddy.

Bearing Capacity of Bayou Caddy WAD®

LSS has engineered the Bayou Caddy WAD® system to be well under the maximum load factor of 275 pounds per square foot. By utilizing a 4-inch tongue and groove base for the WAD®, we were able to greatly reduce the load. With an overall, maximum a design mix weight of no more than 8,295 ponds, a known surface area of the base at 43.3 square feet. The Bayou Caddy WAD is at 191.5 PSF on dry land PSF and once the unit in positioned in place at the -3.5 contour, the load will be reduced to 126.4 PSF. With deeper water and its buoyancy, load factor will decrease.

Placement Plan of Bayou Caddy WAD®

Product Data is the same as the approved WAD® specified in this bid from Living Shoreline Solutions Inc. WAD® Units will be manufactured and assembled at Design Precast & Pipe Inc., loaded on flatbed, air-ride truck with a certified three-way chain and excavator. Units will be delivered to contractor's yard and unloaded in the same manner, for temporary storage. Prior to deployment, placement survey will be verified and PVC pipe and line placed along the designated back row alignment of the WAD® array. At deployment time, WADs® will be carefully loaded under supervision and guidance of LSS QA/QC personnel. A certified three-way chain will be hooked through the top three openings in the WAD® and carefully lifted by crane *per Crane Critical Lift Plan as required in Section 01 35 26 GOVERNMENTAL SAFETY REQUIREMENTS* and swung over the deck of a 30 X 120 barge secured along-side the storage yard. Once the barge has been loaded with ~50 WADs®, it will be pushed out on station by boat and spudded down in place. At that time the crane with certified three-way chain will be attached to a WAD® which will be lowered down in pre-determined back-row location with the flat surface parallel to shore and facing offshore into the wave energy. There it will be lowered in place and the chain unhooked by a laborer standing alongside the placed WAD®. The crane (*per Crane Critical Lift Plan as required in Section 01 35 26 GOVERNMENTAL SAFETY REQUIREMENTS*) will then boom over to the next WAD® on deck, hook that unit up in the same manner and swing that WAD® into place carefully, lowering it down as the side walls lightly contact each other and slides into place on the bottom. That unit will be unhooked and alternating again to the next WAD® which will mirror the orientation of the previous WAD® in the back row until several units have been placed in the back row. The deployment team will now start the front row which will duplicate the matching and orientation of the back row. Units will be aligned along a prepositioned guideline to mirror the designated contours. Each WAD® can be hooked up, lowered in place, unhooked and hooked up to a new WADs® one cycle every 4-5 minutes on average. Very easy to align and place the WADs® in designated arrays. (Please refer to the engineering construction drawings that are provided with this Technical Report)

Design Life of Bayou Caddy WAD®

The Design life for the Bayou Caddy WAD® is warranted at minimum, 25 years. As can be seen in stability analysis and 19 years of proven performance is totally capable of sustaining 5 percent or less damage when the still water level is equal to the crest elevation of the WAD breakwater with a wave height equal to 0.8 times the total water depth at the seaward to of the structure. The WAD® structure itself is manufactured using a minimum, 5,000 psi marine-grade concrete dosed with 3 pounds of fibermesh 650, synthetic fiber/ cubic yard of concrete, completely protecting WAD® from saltwater permeability. This translates to a highly effective system with no materials subject to corrosion in the WAD®. LSS has engineered a process to reduce concrete permeability (near 100%) increase shatter resistance, impact resistance and abrasion resistance. This concrete reaches 4,000 psi within 24 hours and 5,000-7,000 psi within 28 days. Regarding the issue of the stainless banding while transporting the WADs in place, those bands are only for lifting and placing the units. The tongue and groove mating of the WAD (main weight) and the bottom will be held in place by the weight of the WAD and cannot slip off. Additionally, the WAD units are placed toe to toe, base to base and sediment will start building up at the bases of the WAD breakwater, further stabilizing the individual units. Within a year, oyster growth will be measurable and further increase the size of attenuation surface area and "grow" the WADs together being more firmly stabilized. The Stainless Banding Material Specification sheet is included in the submittals. These WAD® structures will be totally encrusted in marine biomass in a few years and unrecognizable as an artificial structure but a natural, designed barrier reef system that will continue to protect the shorelines. To date WAD® systems have been in place throughout the world under varying conditions and have remained in place and intact without movement, even in the face of category 4-5 hurricanes, for the past 19 years.

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October 24, 2016

US Army Corps of Engineers
Mobile District
109 St. Joseph Street
Mobile, Alabama 36602

Subject: Watts' Mississippi Experience

To Whom It May Concern:

I graduated with a B.S. in Civil Engineering from Christian Brothers University, Memphis, Tennessee in 1996 and a M.S. in Water Resources Engineering from the University of Texas, Austin in 2006. I am a Water Resources Engineer, a Certified Floodplain Manger, and a Diplomate Water Resources Engineer with a P.E. in Mississippi. My Mississippi license number is 17930 with an expiration date of December 31, 2016.

I have 17 years of experience with project experience that includes green infrastructure design, water resources engineering, and civil engineering. I have worked on several coastal engineering projects including two in southern Mississippi with tidal influences, Hurricane Katrina Infrastructure Repairs for St. Michael's and Buena Vista subdivisions. Both subdivisions have outlets to the gulf. I also worked on a CPRA emergency shoreline protection project in Louisiana.

Sincerely,

Jessica L. Watts

Jessica L. Watts, P.E., CFM, D.WRE

Water Resources Engineer

CDM Smith Inc.

cc: File

Section A3 – Cost

Table of Contents

List of Tables

A3-1) Cost Estimates for Tentatively Selected Plan

Cost Estimate Development

The project cost estimate was developed in the MCACES MII cost estimating software and used the standard approaches for a feasibility estimate structure regarding labor, equipment, materials, crews, unit prices, quotes, sub-contractor markups and prime contractor markups. This philosophy was taken wherever practical within the time constraints. It was supplemented with estimating information from other sources where necessary such as from quotes, bid data, and Architect-Engineer (A-E) estimates. It is to be noted that after development of Abbreviated Risk Analysis (ARA), the costs within the Tentatively Selected Plan were further refined so some minor inconsistencies between the Cost Section and the rest of the Engineering Technical Appendix may be present.

Cost estimates for the Tentatively Selected Plan were developed at a Class 3 level of effort utilizing largely parametric unit prices from sources such as historical Government and Commercial bid data, A-E cost estimates available from design reports, the 2021 Gordian/RS Means Cost Data Books and other available historical cost data sources. For developing costs for the rubble mound construction, the standard approaches for developing a feasibility cost regarding cost elements such as labor, equipment, materials, crews, unit prices, subcontractor and prime contractor markups were used.

The intent of the cost estimate was to provide or convey a “fair and reasonable” estimate and where cost detail was provided, it depicted the local market conditions. The construction work is common to the Atlantic Coast region. The construction site is only accessible from water; however, staging area site access is easily provided through various local roads in Beach Haven, New Jersey. Water access is available through the New Jersey Intracoastal Waterway (NJIWW).

Estimate Structure

The estimate has been subdivided by feature and contains U.S. Army Corps of Engineers (USACE) feature Work Breakdown Structure (WBS) codes. Each WBS cost is subdivided into base cost, contingency and total cost.

Bid Competition

It is assumed there will not be an economically-saturated market, and that bidding competition will be present.

Contract Acquisition Strategy

There is no declared contract acquisition plan/type at this time. It is assumed that the contract acquisition strategy will be similar to past projects. The assumption is that there will be some negotiated contracts with a focus and preference for small business/8(a) along with some large, unrestricted design-bid-build contracts.

Labor Shortages

It is assumed there will be a normal labor market pulled from the New Jersey area.

Labor Rates

Labor rates were developed comparing Region 1 labor market wages with the local Davis-Bacon Wage Determination, using whichever was determined greater. Regional wage information was formulated from data gathered from approximately 5 different USACE, Philadelphia District (CENAP) construction projects in the Greater Philadelphia region and is assumed to be a fair representation of wage rates for the Beach Haven area.

Materials

Detailed cost estimates were developed for the major construction items such as rip rap and geotextile material. Material quotes were obtained in the development of this estimate and are assumed to remain consistent throughout the project. It is assumed that materials will be purchased as part of the construction contract and prices include delivery of materials.

Cost quotes are used on major construction items when available (such as the associated costs used for pump stations and vehicular and pedestrian roller and swing gates). Material price quotes were taken from previous jobs or from other historical data.

All riprap material is assumed to be contractor furnished. Specific sources for riprap material have been identified, yet are located a considerable distance away from the project site. Since the project location is in New Jersey, quarries that carry the required quantity and riprap size were unavailable. As such, quarries in Pennsylvania and Maryland were considered and material quotes from suppliers were received. Various methods of material delivery were quoted, such as marine delivery and truck delivery. The PDT assumed that truck delivery would be optimal and the material supplier quoted truck rental rates for delivery of materials to the staging area.

Quantities

Quantities for rubble mound and geotextile material were provided by CENAP Civil Section. Quantities for planting were provided by CENAP Environmental Section.

The PDT decided that for the Tentatively Selected Plan a comprehensive quantity of the alignment would be provided. The rubble mound elevation remains consistent throughout the project length, independent of location. The preliminary assumptions are that the rubble mound has a 3 ft wide crown and side slopes of 1V:2H. The existing elevations were obtained from the 2017 LIDAR raster dataset in addition to the 2017 bathymetry dataset. Since the rubble mound design elevation for the TSP was fixed, the designer calculated the area per station and multiplied it by the length. Quantities for rubble mound construction were developed by the civil designer and are provided in Section A2 – Civil Design. Cost engineering completed a review and verified the provided quantities. The Project Delivery Team (PDT) also noted that the quantities within the TSP design have considerations for rubble mound settlement and global subsidence to comply with the latest design criteria. Additionally, a Staging area was scoped and provided along with potential access points. The design parameters and quantities for the rubble mound were provided by the civil designer to meet the required design elevations for the rubble mound and costs were developed to represent each feature within the TSP.

Equipment

Rates used are based on the latest USACE EP-1110-1-8, Region I. Adjustments are made for fuel and facility capital cost of money (FCCM). Full FCCM/Cost of Money rate is the latest available. The MII program takes the EP-recommended discount, but no other adjustments have been made to the FCCM. Equipment was chosen based on historical knowledge of similar projects

Rental Rates

Judicious use of owned verses rented rates was considered based on typical contractor usage and local equipment availability. Where rental of equipment is typical, rental rates were applied (i.e. for Tugboat, marine barges, etc.).

Fuels

Fuels (e.g., gasoline and diesel fuel) for rental equipment were based on local market averages for the New Jersey area. The fuel rates were reviewed over a period of time and a composite, conservative cost was used. Due to the volatility of fuel and significant potential escalation of fuel rate, conservative costs were used in the estimates.

Crews

Major crew and productivity rates were developed and studied by senior USACE estimators familiar with the type of work. The work is typical to the New Jersey area and is well understood by CENAP cost engineers. The crews and productivity rates were checked by local CENAP estimators and comparisons with historical cost data were referenced. Crews and productivity rates were adjusted as necessary based upon those findings to reflect reasonable crew sizes and production rates. Major crews are used for placement of riprap and geotextile in marine conditions.

A 10% markup on labor for weather delay and marine conditions was selectively applied to the labor in major riprap placing detail items, and associated items that would be affected by the weather, creating unsafe or difficult conditions to operate (e.g., trying to place riprap with significant marine traffic in the channel) or would be detrimental/non-compliant to the work being performed (such as trying to place material in heavy seas). The 10% markup was to cover the common practice of paying for labor "showing up" to the job site and then being sent home due to minor weather conditions, which is part of known average weather impacts as reflected within the standard contract specifications.

Most crew work hours are assumed to be 8 hours, 5 days/week, which is typical for the project area.

Unit Prices

The unit prices were kept to a minimum since further refining of the TSP, which allowed for development of cost items. Recent pricing data and cost estimate was reviewed and compared to historical data to ensure that all costs were within reason. Historical unit price variance versus the fully developed costs are a result of differing haul distances (by truck or barge), small or large business markups, subcontracted items, designs, and estimates by others.

Relocation Costs

Relocation costs are defined as the relocation of public roads, bridges, railroads and utilities required for project purposes. In cases where potential significant impacts were known, relocation costs were included within the cost estimate. Information from the Relocations Designer showed no relocations of public roads, bridges or railroads were required in the TSP, however, it contains relocations of public utilities. The Relocations Designer did provide all utilities to be relocated for the TSP (i.e. pipe - ownership, diameter, material, product, location) and these are shown in the Engineering Appendix. Cost was developed using historical cost data and the 2021 Heavy Construction Gordian/RS Means Data Book. Relocation costs were placed in Work Breakdown Structure WBS-02 Relocations.

Mobilization

Contractor mobilization and demobilization are based on the assumption that most of the contractors will be coming from within the New Jersey area. Mobilization and Demobilization costs are based upon historical studies and detailed Government estimates with relevant historical cost pricing data, which are typically in the range of 5-10% of the construction costs. With undefined acquisition strategies and assumed individual project limits, the estimates developed detailed Mobilization and Demobilization costs, which is approximately 8% value of Cost to Prime for the Tentatively Selected Plans.

Field Office Overhead

The estimated percentages for Field Office Overhead were based upon estimating and negotiation experience, and consultation with local construction representatives. The estimates used a field office overhead rate based on the average of relevant jobs with a similar scope and magnitude. Different percentages are used when considering the scope of work for each feature. However, when reviewing historical cost pricing data, a range of 10 -25% is typically used. The field office overhead rate of 12% was used for the prime contractors, which was based on historical projects. A field office overhead rate of 5% was used for subcontractors.

Overhead Assumptions

Overhead assumptions may include costs for the superintendent, the office manager, pickup trucks, periodic travel costs, communications, temporary offices (contractor and Government), office furniture, office supplies, computers and software, as-built drawings and minor designs, tool trailers, staging setup, camp/facility/kitchen maintenance and utilities, utility service, toilets, safety equipment, security and fencing, small hand and power tools, project signs, traffic control, surveys, temporary fuel tank station, generators, compressors, lighting and minor miscellaneous items.

Home Office Overhead

The estimated percentages vary based upon consideration of 8(a), small business and unrestricted prime contractors. The rates were based upon estimating and negotiating experience, and consultation with local construction representatives. Different percentages are used when considering the contract acquisition strategy regarding small business 8(a), competitive small business and large business, high to low, respectively. For Home Office Overhead a percentage of 10% was assumed for the prime contractor while a percentage of 12% was applied to a subcontractor.

Taxes

Local taxes on supplies and materials needed for construction would be applied based on the county that contains the work. Reference the tax rate website for New Jersey: http://www.salestaxstates.com. The contracts are located in Beach Haven, New Jersey and the tax rate is 6.625%. As such, the tax rate used for this project is 6.625%.

Bond

The Bond interest rate was assumed to be 2%, applied against the prime contractor, assuming large contracts.

Real Estate Costs

Real Estate (RE) costs were developed and provided by the Realty Specialist and placed in WBS-01 Lands and Damages. The RE cost for each alternative includes land costs, acquisition costs, and 25% for contingencies.

Environmental Costs

Environmental costs were discussed within the PDT and it was determined that the project does not anticipate any additional Environmental cost outside of first construction costs since the intent of the project is to restore environmental conditions. The PDT anticipated that no additional known or identified environmental costs will be present on restoration of Mordecai Island.

Cultural Resources Costs

Cultural Resources (CR) costs were discussed within the PDT and it was determined that the project does not anticipate any Cultural Resources costs. It is estimated that Phase I & II Cultural Surveys and mitigation of resources will not be required. The PDT anticipated that no known or identified cultural resource sites will be present on Mordecai Island. At this time there is no reason to believe additional Cultural Resource sites will be found, therefore, the estimates do not include costs for any potential Culture Resources.

Pre-Construction Engineering and Design (PED)

The PED cost included such costs as USACE project management, engineering, planning, designs, investigations, studies, reviews, value engineering (VE) and engineering during construction. Historically, a rate of approximately 12% for Engineering and Design (E&D) portion, plus small percentages for other support functions, is applied against the estimated construction costs. Other USACE civil works districts have reported values ranging from 10% to 20% for E&D. Additional support functions might include project management, engineering, planning, designs, investigations, studies, reviews and VE. The percentage was calculated using PDT input and costs provided from all disciplines using their respective historical costs for this phase. A PED rate of 20.2% was applied for this project.

Supervision and Administration (S&A)

Historically, a range from 5% to 15%, depending on project size and type, has been applied against the estimated construction costs. Other USACE civil works districts report values ranging from 7.5% to 10%. Consideration is given that a portion of the Supervision and Administration (S&A) effort could be performed by contractors. An S&A rate of 8.3% was applied for this project.

Contingencies

Contingencies for the Tentatively Selected Plan were developed using the USACE Abbreviated Cost Risk Analysis (ARA) program. An ARA is a qualitative approach used by the PDT to address key risk concerns for major features of work and their impact to cost and schedule drivers such as Project Scope Growth, Acquisition Strategy, Construction Elements, Quantities, Specialty Fabrication or Equipment, Cost Estimate Assumptions and External Project Risks. The development of the ARA resulted in a composite risk contingency of 25.85%, considering all factors of the project. It should be noted Real Estate, PED and S&A costs were not included in formulating the composite risk contingency; however, the overall total project contingency that was developed in the ARA by the PDT was applied to the PED and S&A costs.

Escalation

The escalation for the structural items taken from the historical cost pricing data were based upon the latest version of the USACE Engineering Manual (EM) 1110-2-1304, "Civil Works Construction Cost Index System (CWCCIS)".

Hazardous, Toxic and Radioactive Waste (HTRW)

Phase 1 surveys have not been performed, but preliminary investigation by the Biologist indicates no issues were found along the proposed final alignments. The risk of finding HTRW on the mostly environmental island and surrounding residential areas that are along the alignment is low. At this time there is no reason to believe HTRW will be found, therefore, the estimates do not include costs for any potential HTRW.

Schedule

The project schedule for the Tentatively Selected Plan was developed based on the construction features of work. Plan Formulation/Project Management for the Mordecai Island Ecosystem Restoration study have directed that construction of Phase I of the system be assumed to begin in May of 2024 with a completed ecosystem restoration system in place by 2026. It is anticipated that there are 2 separate phases of the project, which include the construction of the rubble mound and installation of plants. There is a one year allotment for placement of dredged material. The placement of dredged material is not covered in this study, but the one year window was included in the schedule. The dredging is anticipated to be covered under Operations Division. The expected construction period for Phase I and Phase II is 8 months total. For the purposes of this study, the design is to begin in 2023 and is assumed to be complete in 2024. Construction of the Phase I Rubble Mound will commence in May of 2024 with an estimated completion in September of 2024. Maintenance Dredging for Operations Division is to commence and it is anticipated to be completed in January of 2026, or one full dredge cycle. Phase II Planting will commence in March of 2026 and is expected to be complete in April of 2026. Table A3-1 below represents the anticipated construction schedule for Mordecai Island Ecosystem Restoration.

Table A3-1: TSP – Construction Schedule

Mordecai Island Ecosystem Restoration Construction Schedule

PHASE 1 PROJECT START:	5/1/2024
Display Week:	1

Phase 1

Task	Duration - Work Days	Start Date	End Date
Submittals and Site Prep	30 days	5/1/2024	5/31/2024
Mobilization	3 days	6/1/2024	6/5/2024
Development of Work Plan	5 days	6/6/2024	6/12/2024
Relocate Power Pole for Staging Area	3 days	6/13/2024	6/17/2024
Load Buoys and Lights onto Barge	days	6/17/2024	6/17/2024
Placement of Navigational Signs and Buoys	1 days	6/17/2024	6/18/2024
Load Geotextile from Staging Area onto Barge	9 days	6/19/2024	7/1/2024
Placement of Geotextile Material	9 days	7/1/2024	7/14/2024
Load R6 Riprap onto Barge	24 days	7/14/2024	8/15/2024

Placement of R6 Riprap Rubble Mound	24 days	8/15/2024	9/16/2024
Removal of Navigational Signs and Buoys	1 days	9/16/2024	9/17/2024
Demobilization	2 days	9/17/2024	9/19/2024
Weather Days and Holidays	6 days	9/19/2024	9/29/2024

Phase 2 (To Be Completed after Maintenance Dredging, estimated contract length for Maintenance Dredging is 365 Days)

PHASE 2 PROJECT START:	3/1/2026
Display Week:	2

Phase 2

Task	Duration	Start Date	End Date
Mobilization of Planting Crew	2 days	3/2/2026	3/4/2026
Load and Transfer Plants to Island	2 days	3/5/2026	3/8/2026
Planting Spartina Patens	22 days	3/9/2026	4/8/2026
Planting Alterniflora	7 days	4/8/2026	4/19/2026
Demobilization of Planting Crew	1 days	4/19/2026	4/20/2026
Weather Days and Holidays	3 days	4/20/2026	4/23/2026

Assumptions

*Assume that all work is to be completed sequentially and all tasks are finish to start.

Cost Estimates

Tables A3-2 show the project first cost for the Tentatively Selected Plan. All costs are at December 2021 price levels.

Table A3-2: TSP – TSP – Rubble Mound

Feature	Cost	Contingency	Total
01 Lands and Damages	$54,000	$14,000	$68,000
02 Relocations	$15,000	$3,000	$18,000
06 Fish and Wildlife Facilities	$987,000	$188,000	$1,175,000
10 Breakwaters and Seawalls	$3,339,000	$931,000	$4,270,000
30 Planning, Engineering & Design	$885,000	$229,000	$1,114,000
31 Construction Management	$563,000	$146,000	$709,000
TOTAL	$5,843,000	$1,510,000	**$7,354,000**

The total baseline project cost for the comprehensive Tentatively Selected Plan is $7,354,000.

Attachments

Table A3-3: TSP – Civil Works Breakdown Structure

MORDECAI ISLAND
BEACH HAVEN, NEW JERSEY
ECOSYSTEM RESTORATION
FEASIBILITY STUDY AND INTEGRATED ENVIRONMENTAL ASSESMENT

Table 1 - Tentatively Selected Plan: Rubble Mound w/ 11 Acres of New Marsh

CIVIL WORKS BREAKDOWN STRUCTURE

Construction duration: 8 months

Price Level: Dec 2021

ACCOUNT NUMBER	DESCRIPTION OF ITEM	QTY	UOM	UNIT PRICE	ESTIMATED AMOUNT	CONTIN	TOTAL ESTIMATED AMOUNT
01.	LANDS AND DAMAGES	1	Job	LS	$54,227	$13,527	$67,754
02.	RELOCATIONS	1	Job	LS	$15,411	$2,695	$18,106
06.	FISH AND WILDLIFE FACILITIES	1	Job	LS	$987,103	$188,241	$1,175,344
10.	BREAKWATERS AND SEAWALLS	1	Job	LS	$3,338,629	$931,478	$4,270,107
12.	PORTS AND HARBORS	1	Job	LS	$0	$0	$0
30.	PLANNING, ENGINEERING, & DESIGN	1	Job	LS	$884,953	$228,760	$1,113,713
31.	CONSTRUCTION MANAGEMENT (S&A)	1	Job	LS	$562,575	$145,426	$708,001
		TOTAL PROJECT AMOUNT			$5,842,898	$1,510,126	$7,353,024
				ROUNDED	$5,843,000	$1,510,000	$7,353,000

Additional costs:

ADAPTIVE MANAGEMENT	$94,388	per year includes contingency
ENVIRONMENTAL MONITORING (Years 1 through 10)	$100,680	per year includes contingency

Notes:

1) Cost estimate assumes work will be done from the water side and a 5' minimum draft is available for a loaded barge to gain access to the shore.

2) There will be no environmental construction windows.

3) A $50K place holder has been included for real estate easements.

4) Maintenance dredging and beachfill costs for 30K CY of fill to be paid for by OPs.

Table A3-4: Mordecai Island Mii Report

Mordecai Island Nov2021

Estimated by	Jeffrey Sklencar
Designed by	Samuel Weintraub
Prepared by	Jeffrey Sklencar
Preparation Date	12/21/2021
Effective Date of Pricing	12/21/2021
Estimated Construction Time	240 Days

Description	Quantity	UOM	ContractCost	Escalation	Contingency	SIOH	ProjectCost
Project Cost Summary Report			**4,340,833.21**	**0.00**	**0.00**	**0.00**	**4,340,833.21**
Selected Plan: Rubble Mound w/ 11 Acres of New Marsh	**1.00**	**JOB**	**4,340,833.21**	**0.00**	**0.00**	**0.00**	**4,340,833.21**
02. RELOCATIONS	**1.00**	**JOB**	**15,409.13**	**0.00**	**0.00**	**0.00**	**15,409.13**
06. FISH AND WILDLIFE FACILITIES	**1.00**	**JOB**	**987,102.98**	**0.00**	**0.00**	**0.00**	**987,102.98**
10 BREAKWATERS AND SEAWALLS	**1.00**	**JOB**	**3,338,321.10**	**0.00**	**0.00**	**0.00**	**3,338,321.10**

Labor ID: EQ ID: EP20R01 Currency in US dollars TRACES MII Version 4.4