GEOTECHNICAL STUDY
Houston Ship Channel Expansion Channel Improvement Project
Harris and Chambers Counties, Texas

SUBMITTED TO

HDR Engineering, Inc.

4828 Loop Central Drive, Suite 800

Houston, Texas 77081

BY

HVJ ASSOCIATES, INC.

HOUSTON, TEXAS

DECEMBER 4, 2020

REPORT NO. HG1910092.2.1 - DES

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December 4, 2020

Mr. Neil McLellan, PE
HDR Engineering, Inc.
4828 Loop Central Drive
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Houston, Texas 77081

Re: Geotechnical Study
Houston Ship Channel Expansion Channel Improvement Project
Harris and Chambers Counties, Texas
Owner: Port of Houston Authority
HVJ Report No. HG1910092.2.1 – DES

Dear Mr. McLellan:

Submitted herein is the draft design report of our geotechnical study for the above referenced project. This report presents the results of our analyses performed for the proposed structures based on available cross sections. The study was conducted in general accordance with our proposal number HG1910092.2.1 dated September 12, 2019 (revised October 25, 2019) and is subject to the limitations presented in this report. We appreciate the opportunity of working with you on this project. Please read the entire report and notify us if there are questions concerning this report or if we may be of further assistance.

Sincerely,

HVJ ASSOCIATES, INC.
Firm License No. F-000646

DRAFT

Michael Hasen, PE
Executive Vice President

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Senior Project Manager

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Copies submitted: 1 (electronic)

This document was released under the authority of Michael Hasen, PE 57498 on November 25, 2020 for interim review. This document is not to be used for the purposes of bidding, design, or construction.

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1 EXECUTIVE SUMMARY

HVJ Associates, Inc. was retained by HDR Engineering, Inc. (HDR) to perform a geotechnical study for the Houston Ship Channel Expansion Channel Improvement Project in Harris and Chambers Counties, Texas. The overall project involves widening and deepening (where applicable) to the Houston Ship Channel (HSC) and its two tributaries, Bayport Ship Channel (BSC) and Barbours Cut Channel (BCC). The dredge material will be used to construct placement areas and beneficial use sites. The work is separated into eleven sections and the description of each location addressed in this study is provided in Section 2.1.

This design report presents our recommendations and results of the slope stability analyses of the Bayou Reach channel, Bayport Ship Channel, Barbour Cut channel bulkheads, Atkinsons Island Cell M12, Beltway 8 and E2 Clinton Placement Area side slopes. In addition, displacement of soft clay soils at Cell M12 and consolidation settlement analysis of foundation soils at Cell M12, Beltway 8 and E2 Clinton placement areas are included in this report.

1. <u>HSC – Bayou Reach</u>: We evaluated the proposed dredge template comprising 3H:1V slopes to El. -50.5 feet. The global stability analyses indicate that the proposed 3H:1V template has an adequate factor of safety. The results of 2.5H:1V slope stability analyses confirm that the Red Side up to Station 804+00 needs to remain at 3H:1V. A 2.5H:1V side slope meets the required factors of safety on the Red Side beyond Station 804+00 and on the entire Green Side.

   Very dense sand, cemented sand, and hard clay soils were encountered within the proposed dredge depths at many of the borings drilled for this study. These soils may impact dredge production rates, the extent of the impact depends on the dredging equipment used. Cemented sand and hard clay were encountered in borings ECP-403D, ECP-406D, ECP-410D, ECP-414D, ECP-415D, and ECP-419D below about El. -41 feet which may prove difficult for many dredges to excavate.

   There are several facilities located immediately adjacent to the channel slopes and the vertical grade beyond the limits of the cross sections appears to be substantially higher based on the aerial images. The scope of this study did not include sufficient survey data or geotechnical borings to allow us to evaluate these facilities. These locations must be investigated to assure that dredging will not cause damage.

2. <u>BCC – Spilmans Island</u>: We evaluated the proposed dredge template comprising 3H:1V slopes to El. -61 feet with a cutoff wall at an offset of about 250 feet from the proposed channel toe. Our analyses also included a pipeline protection wall near the turning basin. The global stability analyses indicate that the proposed cutoff wall should extend to at least El. -58 feet MLLW for global stability and the pipeline protection wall should extend to at least El. -90 feet. The stability analyses meet or exceed the required minimum factor of safety, however interior stability berms will be needed.

3. <u>BCC Site 1:</u> We evaluated the proposed dredge template to El. -61.0 feet with varying bulkhead alignments at Site 1. The global stability analyses indicate that the proposed bulkhead should have a minimum tip elevation of -60 feet MLLW for global stability.

4. BCC Site 2: We evaluated a 3H:1V slope extending to El. -61, this slope did not meet the required factor of safety. A 20-foot wide shelf at about El. -8 feet MLLW and a 4H:1V slope from the shelf to the top of bank are required to achieve the required safety factor for the Rapid Drawdown case.

5. Cell M12: The estimated site capacity is 2,520,000 cut cubic yards if no marsh is required. If site capacity is limited to about 1,630,000 cut cubic yards about 200 acres of marsh can be created if the clay ball fill elevation is properly managed. The stability analyses indicate that the proposed cross sections do not meet the minimum required factor of safety. In order to achieve a stable cross section, the exterior slope must be 3H:1V for the bay side dike (Baseline A) and 3.5H:1V for the shore side dike (Baseline B).

6. E2 Clinton Placement Area: The estimated site capacity is 1,482,550 cut cubic yards. The global stability analyses indicate the proposed cross sections are stable with dike elevation of +55 feet MLLW, a crest width of 15 feet, a 4:1 exterior slope and 3:1 interior slope except where very soft clay material from past dredging is encountered in the dike foundation. Dike foundation replacement with fill is needed at the following locations.
   - Section 1 – Extends 70 feet from the centerline to the exterior and the full dike width from the centerline to the interior. Limits are from Station 1+00 to 19+00 with bottom elevation at El. +20 feet from Station 1+00 to 15+50 and El. +25 feet from Station 15+50 to 19+00.
   - Section 2 – Extends 60 feet from the centerline to the exterior and the full dike width from the centerline to the interior. Limits are from Station 48+50 to 65+00 with bottom elevation at El. +20 feet from Station 54+50 to 65+00 and El. +24 feet from Station 48+50 to 54+50.
   - Section 3 – Extends 60 feet from the centerline to the exterior and the full dike width from the centerline to the interior. Limits are from Station 73+00 to 75+50 with bottom at El. +22.

   Borrow material for the dike and foundation replacement will encounter previously placed hydraulic fill that will need to be dried and stabilized with an estimated 4% to 8% of lime in order to be suitable for use as fill.

7. Beltway 8 Placement Area: The estimated site capacity is 1,854,400 cut cubic yards. All cross sections meet the required factors of safety. The current cross sections – 3H:1V at the pipeline crossing near Station 74+00 and 6H:1V at the pipeline crossings near Station 95+00, 97+00, and 100+00 – have adequate factors of safety with a weak, organic layer included near the ground surface due to restrictions on grubbing.

8. Bayport Ship Channel, San Jacinto College: We evaluated the proposed dredge template comprising 3H:1V slopes to -48.5 feet and box cut to El. -50.5 feet in the land cut adjacent to the San Jacinto College (SJC) site. The widening in the vicinity of SJC has been reduced in order to allow a sloped channel cut without bulkhead. The global stability analyses indicate that the proposed template has an adequate factor of safety.

Please note that this executive summary does not fully relate our findings. These findings are only presented through our full report.

2 INTRODUCTION

2.1 Project Description

HVJ Associates, Inc. was retained by HDR Engineering, Inc. (HDR) to perform a geotechnical study for the Houston Ship Channel Expansion Channel Improvement Project in Harris and Chambers Counties, Texas. The overall project involves widening and deepening (where applicable) to the Houston Ship Channel (HSC) and its two tributaries, Bayport Ship Channel (BSC) and Barbours Cut Channel (BCC). The dredge material will be used to construct placement areas and beneficial use sites. The work is separated into eleven sections and the description of each location addressed in this study is provided below.

1. **Houston Ship Channel Widening - Bay Reach** – No geotechnical design work in the Bay Reach was performed for this study.

2. **Houston Ship Channel Widening - Bayou Reach** – The Bayou Reach of the HSC extends from Station 00+05 near Morgans Point to Station 1266+48 at the Turning Basin. The Turning Basin stations range from 00+00 to 30+95. Planned modifications to the channel in the Bayou Reach are described below.

    - No widening or deepening is planned from Morgans Point (Station 00+05) to Boggy Bayou (Station 684+03).
    - Between Boggy Bayou to Greens Bayou (Station 833+05) the channel will be deepened from -41.5 to -49.5 feet MLLW plus 1 foot overdredge allowance for a total project depth of -50.5 feet MLLW. The channel will be widened from 300 feet to 530 feet from Station 684+03 to 823+35.
    - Between Greens Bayou and Sims Bayou (Station 1110+78) the channel will not be widened but will be deepened to a total project depth of -49.5 feet as discussed above, the deepening will extend to Hunting Bayou at Station 930+00 which is not the full length. No widening or deepening will occur between Stations 930+00 to 1110+78.

3. **Bayport Ship Channel Widening** – The 4.1-mile long BSC will be widened generally to 455 feet, the land cut by 105 feet to the north and the bay cut by 55 feet to the north. It was envisioned that the proposed channel slope may encroach into the school site at the western end near the Bayport Turning Basin and a sheet pile bulkhead would be required to protect and secure the shoreline in the vicinity of the school site.

    Based on the recent information provided to us, we understand that the channel will be tapered down as it approaches the Bayport Turning Basin to avoid impacts to the San Jacinto College Maritime Campus (SJC) school site and the bulkhead is no longer required. Engineering analysis performed for this report includes slope stability analysis for the land cut portion immediately adjacent to the SJC site.

4. **Barbours Cut Ship Channel Widening** – The BCC is 300 feet wide and 1.6-mile long. The channel mouth will be widened and the channel will be widened to the north by 155 feet towards Spilmans Island Placement Area. A bulkhead will be required to provide adequate global stability for the north slope of the channel and the adjacent Spilmans Island

Placement Area. A bulkhead will also be required at the eastern end at Morgan's Point due to the BCC flare dredging encroaching on the existing container terminal.

5. **Lower Bay Bird Islands** – No geotechnical design work for the bird islands was performed for this study.

6. **Bird Island Marsh** – No geotechnical design work for the Bird Island Marsh was performed for this study.

7. **Atkinson Island, Cell M11** – No geotechnical design work for Cell M11 was performed for this study.

8. **Atkinson Island, Cell M12 –** M12 has about 11,600 feet of new dike with 20-foot wide crest at an elevation of about +8.00 feet MLLW. The bay side dike will be constructed using hydraulic dredge fill with exterior slope of 2.5H:1V protected with stone riprap and the interior slope will be 3H:1V. The shore side dike will be constructed using hydraulic dredge fill with 3H:1V side slopes.

9. **Oyster Reef** – No geotechnical design work for the Oyster Reef was performed for this study.

10. **BSC Shoaling Attenuation Feature –** No geotechnical design work was performed for the Shoaling Attenuation Feature for this study.

11. **Bayou Reach Placement Areas** – The placement areas E2 Clinton and Beltway 8 tracts are currently undeveloped parcels. The E2 Clinton tract has about 8,000 feet of new dike with 15-foot wide crest at an elevation of about +55.00 feet MLLW. Beltway 8 tract has about 16,800 feet of new dike with 10 to 15-foot wide crest at an elevation of about +32 feet MLLW.

2.2 Geotechnical Study Program

This report is the second report on the project, and includes the geotechnical design analyses included in our scope of work with the HDR. The outline of the analyses performed is listed below:

- Global stability of Houston Ship Channel Bayou Reach, Bayport Ship Channel near SJC site and Barbours Cut Ship Channel slopes, and for the Cell M12, E2 Clinton and Beltway 8 placement areas.
- Displacement of soft clay soils at Cell M12.
- Settlement analysis of fill and foundation soils at Cell M12, E2 Clinton, and Beltway 8 placement areas.
- Analysis to estimate the site capacity of Cell M12 for future maintenance dredging.

We prepared a companion data report (HVJ Report No. HG1910092.2.1 – DATA) which presents the boring logs, plan of borings, and summary of the subsurface conditions at the sites. For a detailed discussion of the geotechnical data please see that report. The plan of borings is included on Plate 1.

3 LABORATORY TESTING

3.1 General
One dimensional consolidation and consolidated undrained triaxial compression tests test results at the sites addressed in this study are summarized in this section. For a presentation of the lab test data please see the companion data report.

3.2 Consolidated Undrained Triaxial Test
Consolidated undrained (CU) triaxial tests were performed in accordance with ASTM D4767. A soil specimen is fully saturated in a triaxial cell and isotropically consolidated while allowing drainage to occur. Once the sample is consolidated, the drainage valve is closed and the sample is sheared in compression at a constant rate of axial deformation. This testing provides shear strength parameters for total stress and effective stress global stability analysis. The test results are summarized in Table 3-1.

Table 3-1 – Consolidated Undrained Test Results

Boring No.	Depth, Feet	Soil Type (USCS)	c', psf	ϕ', degrees	c_{cu}, psf	ϕ_{cu}, degrees
HSC – Bayou Reach						
ECP-402D	8-10	Lean Clay w/ Sand (CL)	345	30.8	331	41.7
ECP-405D	24-26	Lean Clay w/ Sand (CL)	322	15.5	348	10.3
ECP-420D	14-16	Fat Clay (CH)	343	27.2	382	16.5
Bayport Ship Channel – SJC Site						
ECP-207	28-30	Fat Clay (CH)	360	21.9	475	15.1
ECP-208	12-14	Lean Clay (CL)	201	26.7	273	24.2
Barbours Cut Channel – Spilmans Island						
ECP-314	10-12	Lean Clay w/ Sand (CL)	244.8	18.6	345.6	12.5
ECP-315	8-10	Fat Clay w/ Sand (CH)	93	29.2	218	20.1
L-05*	13-15	Lean Clay (CL)	100	25	NA	NA
L-07*	18-20	Fat Clay (CH)	300	22	NA	NA
S-01*	24-26	Fat Clay (CH)	400	18	NA	NA
Barbours Cut Channel – Site 1						
ECP-317	6-8	Lean Clay (CL)	187.2	25.8	316.8	16.0
ECP-317	28-30	Fat Clay (CH)	158.4	22.6	115.2	17.7
ECP-319	16-18	Fat Clay (CH)	316.8	23.6	288	15.1
Barbours Cut Channel – Site 2						
ECP-321	33-35	Fat Clay (CH)	86.4	23.2	259.2	10.8

Boring No.	Depth, Feet	Soil Type (USCS)	c', psf	ϕ', degrees	c_{cu}, psf	ϕ_{cu}, degrees
HSC Bayou Reach						
ECP-426D	16-18	Lean Clay (CL)	247	21.4	264	17.8
E2 Clinton Placement Area						
ECP-2003	10-12	Lean Clay w/ Sand (CL)	806.4	16.2	720	16.4
ECP-2004	6-8	Fat Clay (CH)	187.2	23.4	302.4	14.1
ECP-2006	8-10	Lean Clay (CL)	230.4	27.6	432	21.4
Beltway 8 Placement Area						
ECP-2020	6-8	Lean Clay	370	17.7	510	16.6
ECP-2031	12-14	Fat Clay	190	16.7	310	13.1
ECP-2043	6-8	Fat Clay	740	16.3	630	16.5
Cell M12						
ECP-1046	14-16	Sandy Lean Clay (CL)	21	24.0	25	10.3
ECP-1046	16-18	Fat Clay (CH)	222	15.2	248	5.4

* Performed for a previous study
Where:
c': Consolidated Drained Cohesion
c_{cu}: Consolidated Undrained Cohesion
ϕ': Consolidated Drained Friction Angle
ϕ_{cu}: Consolidated Undrained Friction Angle

Note that we performed CU triaxial tests using a multi-stage test on single samples. The ASTM Test Method is based on shearing three separate samples each consolidated to a different overburden pressure. In the multi-stage test, a single sample will be consolidated to three different consolidation pressures and sheared at the end of each consolidation step. The initial two shear steps were to a low strain approaching a peak failure stress at that level. Shearing after the final step proceeded to failure per the ASTM method. We used multi-stage method due to the limited availability of multiple test specimens in an individual boring.

3.3 Consolidation Test Results

One-dimensional consolidation tests were performed in accordance with ASTM D2435. In the test, a cylindrical soil specimen is restrained laterally and axially drained while subjected to applied vertical loadings. Seating stress is applied to the sample, then inundated. Once the sample stabilizes (does not change in height), the sample is loaded incrementally to obtain the virgin compression and rebound curves. The stress where the sample changes from rebound compression to virgin compression is referred to as the preconsolidation pressure, which represents the stress at which the soil has previously been subject to loading and unloading increments, measurements are made of the change in the specimen height and the data is used to determine the relationship between applied stress and void ratio. Table 3-2 presents the consolidation test results:

Table 3-2 – Consolidation Test Results

Boring	Depth, Feet	Soil Description	e_0	C_c	Cr	σ_p, psf	OCR
E2 Clinton Placement Area							
ECP-2003	6-8	Lean Clay w/ Sand (CL)	0.556	0.103	0.018	3,800	4.68
ECP-2005	28-30	Lean Clay w/ Sand (CL)	0.460	0.100	0.013	6,650	3.22
ECP-2006	14-16	Lean Clay (CL)	0.559	0.102	0.028	6,000	4.36
ECP-2008	14-16	Fat Clay w/ Sand (CH)	0.774	0.170	0.096	20,000	16.52
Beltway 8 Placement Area							
ECP-2018	6-8	Sandy Lean Clay (CL)	0.524	0.118	0.037	6,600	7.50
ECP-2026	18-20	Fat Clay (CH)	0.993	0.250	0.126	10,200	4.25
ECP-2037	14-16	Fat Clay (CH)	0.894	0.207	0.126	11,600	6.17
Cell M12							
ECP-1044	23-25	Sandy Fat Clay (CH)	1.142	0.419	0.042	640	0.49

Where:
e_0: Initial Void Ratio
C_c: Compression Index
σ_p: Preconsolidation Pressure, psf
OCR: Overconsolidation Ratio with effective overburden pressure at the sample depth.

4 ANALYSIS APPROACH

4.1 Slope Stability Analysis

Stability analyses of the proposed channel and dike side slopes were conducted using 2019 version of slope stability program SLOPE/W by Spencer's method for circular rotational failure and block failure. Block failure evaluates non-circular failure surfaces and is particularly helpful in evaluating the potential for translational failures. The program calculates the factor of safety against slope failure using a two-dimensional limiting equilibrium method.

For the channel slopes, the recommended minimum factors of safety are 1.3, 1.5, and 1.1 to 1.3 for short term (end-of-construction), long term, and rapid drawdown conditions, respectively (Ref., US Army Corps of Engineers EM 1110-2-1902 Slope Stability, Chapter 3, Table 3-1).

According to US Army Corps of Engineers EM 1110-2-5025 Dredging and Dredged Material Management, Chapter 4, Table 4-8, the recommended minimum factors of safety (FS) are 1.3 for End-of-Construction (Short Term) and Steady Seepage (Long-Term) and 1.0 for rapid drawdown but limits these safety factors to dikes less than 30 feet high and it refers to EM 1110-2-1902 for the dikes with height more than 30 feet. We recommend these safety factors for Cell M12 and Beltway 8 Placement Areas. For E2 Clinton Placement Area, the recommended safety factors 1.3 and 1.5 for short term (end-of-construction) and long term conditions, respectively since the proposed dike height exceeds 30 feet.

The factor of safety represents the calculated resisting forces and moments divided by the calculated driving forces and moments of the various potential failure surfaces analyzed. These forces and moments are based on the estimated unit weights and shear strengths of the various soils in the slope profile. Accordingly, a factor of safety of 1.0 indicates impending failure. The larger the factor of safety is above 1.0, the lower the risk is that the slope will fail. As a practical matter, and in consideration of the variables and unknowns involved, the risk cannot be reduced to zero. The goal is to reduce the risk of slope failure to a reasonable and acceptable level, with due consideration of the consequences of failure.

In general, the soil parameters are determined based on the stratigraphy and material properties determined from borings located in the vicinity of the cross section.

Short Term: The short term case models the initial undrained condition of the soil. For this analysis, torvane, unconfined compression and unconsolidated undrained soil parameters are predominantly used.

Long Term. The long-term design case represents steady state piezometric and stress conditions. When a slope is constructed, altered stress conditions create changes within the slope and the undrained strength of the soils is mobilized. With time, the soil pore pressures adjust to the imposed stress and piezometric conditions, and the bank soils rely on their available strength for long-term stability. Drained or effective shear strength parameters (from Consolidated Undrained Tests and engineering judgment) were used in this analysis.

Rapid Drawdown. The rapid drawdown design case represents the rapid lowering of water level and associated stress conditions. Note that the analysis does not account for damage due to the erosive force of water that overtops a dike or island. When the water level is lowered in a short duration of time, it destabilizes the slope due to the development of excess pore pressures in the embankment consisting of low permeability materials (e.g. clay) and removal of stabilizing force on the upstream face of the slope due to water. The program SLOPE/W utilizes the Duncan et al.'s (1992) staged rapid drawdown method to evaluate slope stability after rapid drawdown. This is a 3-stage process:

The first stage involves the stability analysis of the embankment before drawdown when the water level is high and the pore water pressure in the soils is at steady state condition. Both the effective normal stress and the shear stress along the slip surface are used to determine the undrained shear strength of the soils that do not drain freely.

The second stage involves the stability analysis of the embankment after drawdown when the water level is low and the pore water pressure in the soils is in steady state condition. The effective normal stress obtained from stage two, together with the effective strength parameters are used to compute the drained strength along the slip surface. Both the drained and undrained strength at the slice base along the slip surface are compared and the smaller strength is chosen as the computed shear strength to be used.

The third stage involves stability analysis using the computed shear strength and final drawdown water level. The computed factor of safety from the first and second stages are ignored, and only the factor of safety computed from the third stage analysis is used to represent the stability after rapid drawdown.

The Rapid Drawdown strength parameters in clay were determined from Consolidated Undrained Triaxial Compression tests with pore pressure measurements. Rapid drawdown strengths were based on total stress parameters.

4.2 Displacement Method

Nearshore placement areas and beneficial use sites are usually constructed using the displacement method. Wherever possible, these facilities are constructed on firm, incompressible soil. However, the foundation soils at most boring locations are weak and compressible. In these foundation conditions base failure of the embankment is a critical aspect of the design.

The displacement approach offers the advantage of replacing the weak soils with firmer dredge fill, but has the disadvantage that a substantial amount of material is required for the displacement. General practice has been to displace any soft soils along the dike alignment to the top of underlying firm soils. The objective of construction by displacement method would be to concentrate the load such that bearing capacity failure of any very soft clay bottom soils would occur. The dredge material would then sink into the very soft clay improving the foundation conditions beneath the structure. This process of fill sinking into the bay bottom is referred to as displacement. By displacement, we mean that existing bay bottom would be replaced by fill during construction. This would increase the required volume of material needed to construct the structure, since a portion of the fill sinks below the existing bottom. The figure below and further discussion of the displacement method is presented in Section 4.6 of USACE Engineering Manual EM 1110-2-5025, Dredging and Dredge Material Management.

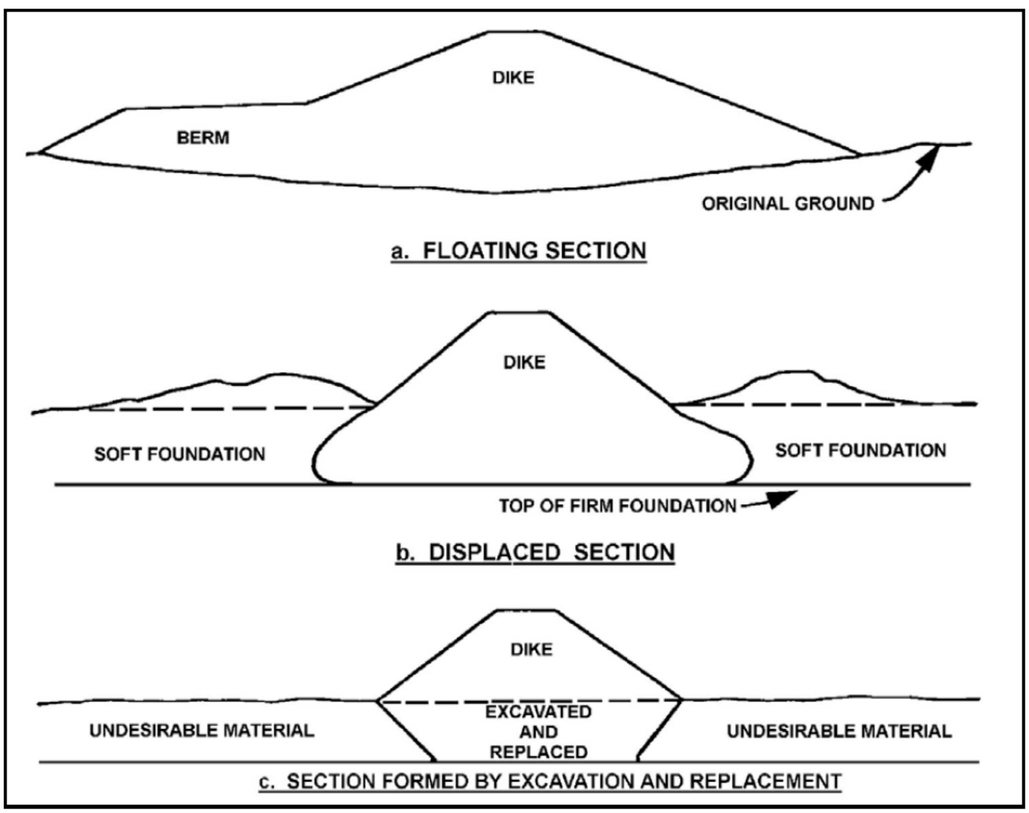


Figure 4-52. Basic Methods of Forming Dike Sections for Stability

The volume of fill for this construction method is dependent on the bottom conditions and the fill material used. Displacement occurring to the full depth of the very soft clay material is optimal for the design, once displacement begins it will proceed to this depth due to the physics involved. Stone and new work clay are efficient at producing displacement because they can be stacked high and narrow enough to cause the initial displacement to begin. Sand fill tends to spread widely and naturally creates a "floating" fill situation. This can be avoided by pre-excavating the very soft clay from beneath the structure, the process is sometimes referred to as "mucking". The material excavated is placed in hopper barges for transport.

Sandy reaches of dike have been known to "squat" after placement. By "squatting" we mean that a portion of the dike cross section settles, sometimes by as much as several feet. The reason for this is that the very soft soil did not displace. For example, if a flotation channel is excavated near a sand dike the original very soft clay bay bottom beneath the sand fill is exposed and squeezes into the flotation channel. This effectively completes the displacement process, and after the initial settlement very little additional settlement occurs. Usually when the dike squats it has been shaped to the final template, so after the squat there is a deficit of material to bring the dike back up to template grades. This can be mitigated by excavating flotation channels prior to final shaping of the dike.

Due to the variable thickness of the soft soil, fill crest elevation, and bottom elevation the weight of the fill above the mudline will vary. As a result, the pressure due to the embankment and the associated amount of soft foundation displaced will also vary. Undrained shear strength of very soft clay needed to prevent additional displacement was evaluated using bearing capacity theory based on the effective unit weight of the dike material. Above the water surface the wet unit weight was used taken as 110 pcf. Between the water surface and bay bottom the effective unit weight was calculated by subtracting the unit weight of water from the dike fill wet unit weight. The unit weight of water was taken as 63 pcf since the bay water is brackish, giving an effective unit weight of 47 pcf for the dike fill. Below the mudline the effective unit weight was determined by subtracting the wet unit weight of the very soft clay from the wet unit weight of the dike fill. The unit weight of the very soft clay was generally taken as 97 pcf, for an effective unit weight of the dike fill of 13 pcf.

Based on the analyses the very soft clay needs to have an undrained shear strength ranging from about 150 psf to about 200 psf to prevent additional displacement. Also, any sand layer will prevent displacement unless it is thin and has a very soft clay layer beneath it. The boring logs at Cell M12 were examined to determine the depth of displacement based on the required undrained shear strength and stratigraphy.

4.3 Settlement

The overall settlement will consist of two parts: settlement within the dike or hydraulically placed fill and settlement within the foundation soils.

4.3.1 Settlement of Coarse-Grained Hydraulically Placed Fill

As fill is hydraulically placed the clay balls consisting of firm to very stiff clay and the sand fraction of the dredged material are deposited near the discharge point. This material is used to construct dikes for placement sites constructed at marine locations. It is also part of the fill at both marine and upland placement areas where new work dredge material is deposited.

This material forms a lattice, honey comb structure with large void spaces between them near the discharge point. The void spaces shrink during construction as a result of filling by smaller soil particles and also as a result of compression of the clay balls by the overlying material; equipment used in shaping the embankment during construction helps to accelerate this process. This phenomenon occurs mostly during construction but we've observed it to continue for some time after construction. Settlement of fill post construction is estimated to be 2.5% of the thickness of the fill (including displacement) after one year and 5% of the thickness of the fill after five years based on prior experience. Settlement is assumed to be complete after five years.

4.3.2 Consolidation of Fine-Grained Hydraulically Placed Fill

Fine-grained fill settlement is governed by the interaction of two processes – self-weight consolidation of the fill and desiccation of the fill. During dredging, the soil is deposited and undergoes self-weight consolidation. After dredging, the fill continues to consolidate under its self-weight. If the fill surface is above the site water level, desiccation will occur which will cause additional settlement of the fill surface. Desiccation refers to the drying of the soil near the fill surface. This is discussed in in U.S. Army Corps of Engineers Engineer Manual EM 1110-2-5025, Dredging and Dredge Material Management. Figure 4-23 from the EM shown below illustrates the process.

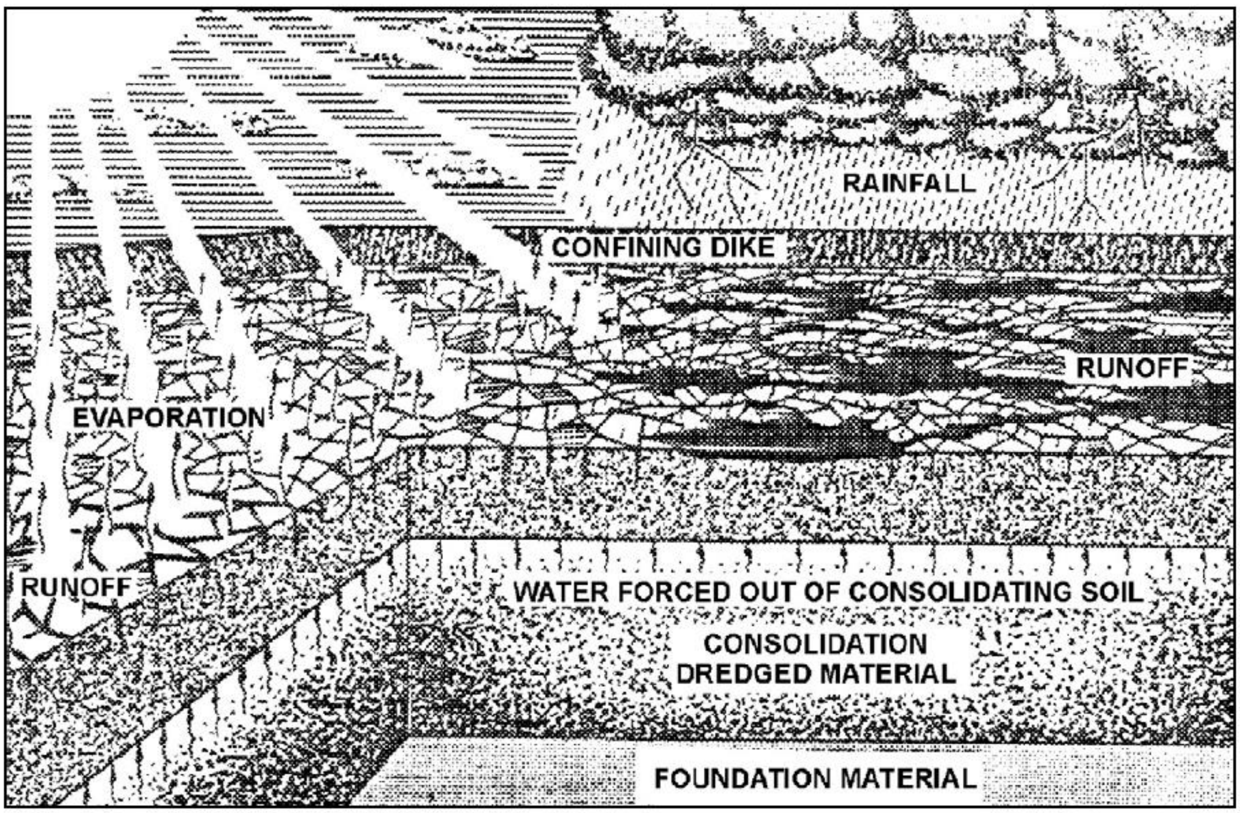


Figure 4-23. Conceptual Diagram of Dredged Material Consolidation and Dewatering Processes

Self-Weight Consolidation. Self-weight consolidation refers to the process of fine-grained fill coming to equilibrium under its own weight. When a soil is hydraulically dredged it is completely disturbed and mixed with a large amount of water for transport to the disposal site. Once the soil-water mixture is deposited in the cell, the soil sediments out of the solution.

After sedimentation the soil still contains too much water. This extra water is squeezed out of the soil by the weight of the soil. This process is referred to as self-weight consolidation, and is also referred to as "primary consolidation" in geotechnical literature.

In order for the fill surface elevation to stabilize self-weight consolidation must be substantially complete. The thickness of the fill, the properties of the fill material, and the permeability of the foundation soil are factors that control the time to complete self-weight consolidation. For a particular site and fill material, the time to complete self-weight consolidation is determined primarily by the fill thickness.

<u>Desiccation</u>. Desiccation refers to the drying of the surface due to exposure to the sun. Desiccation begins after the site is drained, once the rate of water seepage from the fill becomes less than the evaporation rate. A crust of stiffer soil is formed by desiccation.

Desiccation causes fine-grained fill surface settlement in two ways. First, as the fill dries the volume of the crust soil is greatly reduced because the water content goes down. Second, as drying causes groundwater level lowering in the fill the effective self-weight of the fill is increased, this leads to additional self-weight consolidation. Surface settlement due to drying of the crust occurs immediately during the desiccation period. Surface settlement due to additional self-weight consolidation occurs slowly over time after desiccation is complete.

The final crust thickness is controlled by several factors. At an upland site, where the groundwater level is controlled by desiccation and site drainage, the final crust thickness is related to the fill permeability and the evaporation rate. For clay maintenance material in the Galveston Bay area eight to twelve inches represents a typical equilibrium crust thickness.

<u>Analysis Method</u>. We used a computer program developed by the U.S. Army Corps of Engineers called Primary Consolidation, Secondary Compression, and Desiccation of Dredged Fill (PSDDF) to evaluate fine-grained fill settlement. The PSDDF program evaluates time rate of consolidation due to self-weight consolidation, crust formation due to desiccation, and additional consolidation due to the surcharge created by the crust. The program evaluates consolidation using the finite strain consolidation model and it evaluates crust formation using an empirical desiccation model. The finite strain consolidation model differs from the typical small-strain consolidation theory routinely used in geotechnical practice in its ability to account for the following:

1) self-weight consolidation,
2) permeability varying with void ratio,
3) non-linear void ratio-effective stress relationship, and
4) large strains.

The large strain capability is particularly important since strains in excess of 50% are common in these calculations.

In order to use the program, the layer thickness, rate of placement of dredge fill material, and material properties must be known. The program allows input of an incompressible foundation, a compressible foundation, and multiple dredge fill events. The incompressible foundation is primarily a drainage boundary, and permeability characteristics are required. The compressible foundation and dredge fill material require permeability-void ratio and effective stress-void ratio relationships as input for the finite strain consolidation calculations. In addition, an initial void ratio for the fill material is required along with desiccation properties of the fill. Precipitation and evaporation data for the site area are also required as input to the desiccation model.

Compressibility Properties. Input data for the analysis will be obtained from self-weight and oedometer consolidation tests. For this study data from previous testing of maintenance material placed in marsh cells at the Atkinson Island Beneficial Use site will be used for the analysis.

Desiccation Properties. Climatic conditions existing at the site control the effectiveness of evaporative drying and the consolidation and permeability characteristics of the dredged material. Because of the complex nature of desiccation, an empirical model is used to estimate the settlement caused by desiccation in PSDDF. The three most important parameters required by the desiccation model in PSDDF are the void ratio at the saturation and desiccation limits, and the depth to which second-stage drying occurs.

PSDDF models desiccation as a two-stage process. First-stage drying ends and second-stage drying begins when the void ratio decreases to the void ratio corresponding to the saturation limit. During the first stage, the free water table remains at or near the surface of the dredged material though widely spaced and shallow surface cracks will likely develop. Since any non-saturated surface film will be negligible, the dredged fill remains saturated and buoyant when the void ratio is at or above the saturation limit; therefore, the term "saturation limit." During the first-stage drying, the dredged fill surface settles because of the effects of primary consolidation, secondary compression and desiccation.

As the dredged material begins to lose saturation, the free water table drops below the surface and the material develops negative pore-water pressures. The negative pore-water pressures shrink the material to a hard crust having a lower permeability and reduced evaporative rates. The evaporative rate in second-stage drying depends not only on the water conductivity of the unsaturated crust but also the crust thickness. When desiccation progresses to the limiting depth, evaporation from the dredged material effectively ceases. At this point the void ratio is at the desiccation limit and the thickness of the dried crust is equal to the depth of second-stage drying.

The void ratio at the saturation limit and desiccation limit are taken as 4.25 and 2.50, respectively, based on relationships presented in the references discussed above and results of our laboratory tests.

Evaporation and Precipitation Data. Evaporation and precipitation data obtained from the National Oceanic and Aeronautic Administration for the Galveston, Texas area are presented below.

Table 4-1 – Evaporation and Precipitation Data

Month	Evaporation, Feet	Precipitation, Feet
January	0.28	0.30
February	0.24	0.29
March	0.41	0.22
April	0.55	0.27
May	0.68	0.37
June	0.59	0.26
July	0.74	0.38
August	0.75	0.37
September	0.60	0.35

Month	Evaporation, Feet	Precipitation, Feet
October	0.46	0.27
November	0.27	0.29
December	0.26	0.31

4.3.3 Settlement of Mechanically Constructed Upland Site Dike Fill

At the E2 Clinton and Beltway 8 Placement Areas the dikes will be constructed by borrowing material from the site interior. The borrow material will be placed and compacted mechanically. Settlement of this fill depends on the compaction control during fill placement. Post construction settlement is estimated to be 0.5% of the thickness after one year and 1.0% of the thickness after five years.

4.3.4 Consolidation of Foundation Soils

Beneath the fill, consolidation settlement of the foundation soils will control the settlement. This settlement will occur over time after construction. We performed consolidation settlement calculations to estimate settlement that will occur beneath the fill and to estimate the future crest elevation that will remain after settlement. Consolidation settlement analyses were performed using RocScience SETTLE3 software based on parameters determined from consolidation testing and engineering judgment.

4.3.5 Total Settlement

The total settlement of the dike or site fill is the combination of the foundation settlement and the fill settlement. These settlement components are detailed in the discussion of each site. Note that foundation consolidation settlement is included in the PSDDF analysis for consolidation of fine-grained fill. Therefore an independent foundation consolidation analysis was not performed.

4.4 Retention

All new work material dredged in Segment 4 of the Houston Ship Channel will be discharged into the E2 Clinton or Beltway 8 Placement Areas. New work material dredged from Barbours Cut Channel will be deposited in Cell M12. There are three basic materials that need to be considered: very soft to soft clay/silt, firm to stiff clay, and loose to dense sand. As a result of degradation during dredging and transportation through dredging pipes the volume of material discharged is different than the in situ volume. The volume of material deposited depends on the material type and the length of dredging pipe.

It is necessary to develop a means of estimating the volume of material that will be discharged into the placement areas. This relationship is expressed by the Retention Factor (fill/cut ratio) that is defined as the ratio of the volume discharged to the in situ volume of the material.

<u>Firm to Stiff Clay</u>

A portion of the firm to very stiff clay forms "clay balls" which are the material of interest for evaluating the Retention Factor (fill/cut ratio). The clay balls lose mass as they are transported through the dredge pipe. An evaluation of Retention Factor (fill/cut ratio) was made for the Gorini Marsh project, the dike of the Gorini Marsh is located adjacent to the north end of Placement Area (PA) 15, see Figure 4-1. The material used to construct the Gorini Marsh dike was mined from the Houston Ship Channel near the marsh location. Since construction of the Gorini Marsh Atkinson Island Cells M4 and M5/M6 have been constructed using the Gorini Marsh dike as their western limits.

The south portion of the Gorini Marsh dike is clayey and considered representative of the dikes that may typically be constructed from firm to stiff clay cut material. An evaluation of this dike shows a Retention Factor (fill/cut ratio) of about 0.84 when displacement losses of the fill are included and maintenance material in the cut is removed. This Retention Factor (fill/cut ratio) includes transportation mass losses over a distance ranging from 6,500 to 8,500 feet.

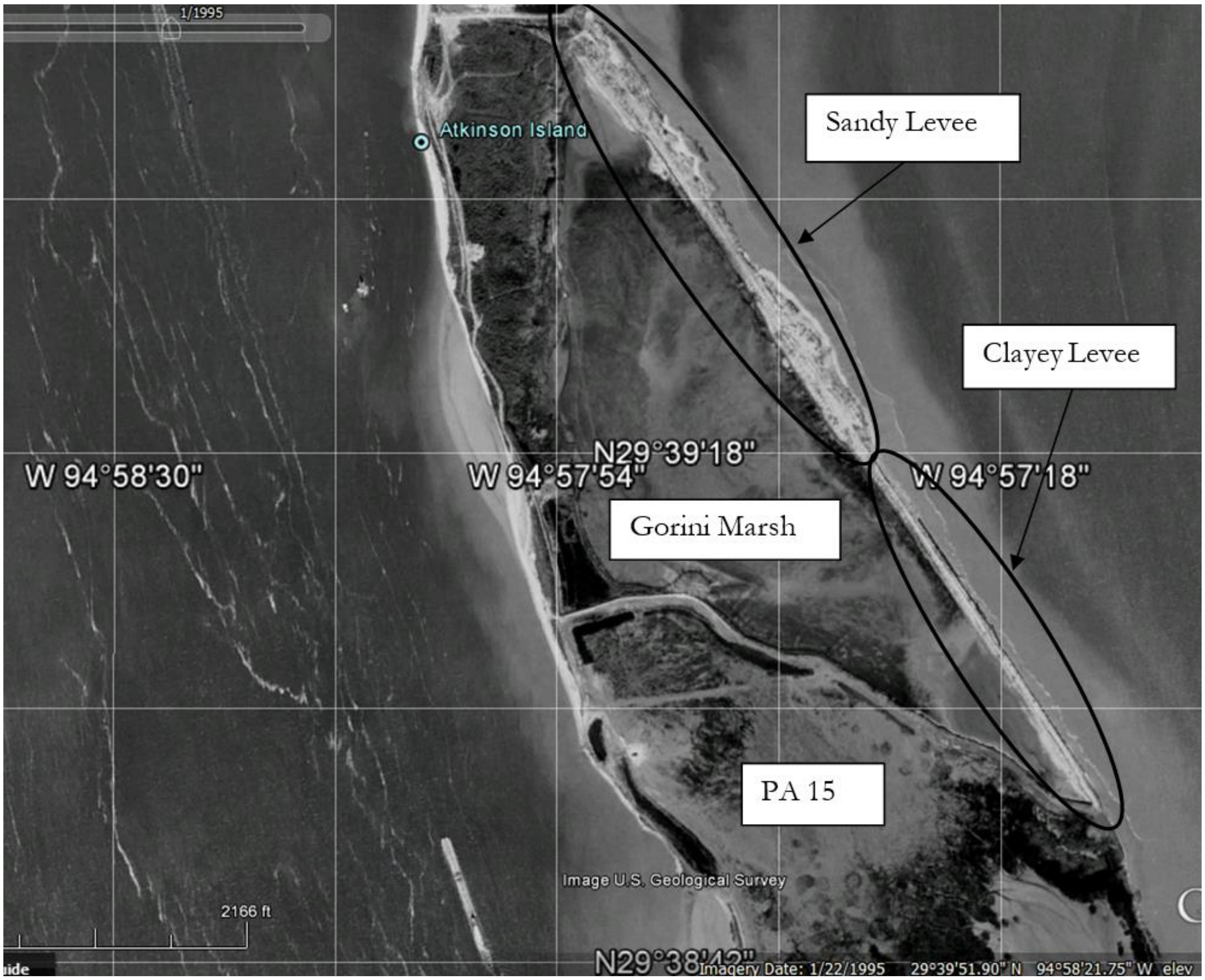


Figure 4-1 – Gorini Marsh circa 1995 (courtesy Google Earth)

As part of the HSC 45-Foot Project, the Galveston District commissioned a study by Waterways Experiment Station on the degradation of clay balls due to agitation over time. Model studies were performed by Dr. Paul Gilbert on Beaumont Clay along the Houston Ship Channel, (Disintegration of Clay During Hydraulic Transport Through a Dredge Pipe, Ph.D. Dissertation, University of Delaware, Summer 1996). Several samples were tested, and the clay ball mass degradation was measured as a function of agitation time. Based on these results, projections of the fractional clay ball mass remaining (mass retention) as a function of time in the dredge pipe were made for 16 samples from the Lower Galveston Bay portion of the project. Pump distances were estimated from exposure times based on an assumed soil/water mixture velocity in the dredge pipe of 15 feet per second. The study suggests that materials with Plasticity Index (PI) less than 25 easily disintegrate while those with PI greater than 35 have greater resistance to disintegration during hydraulic transport through dredge pipes. The following table presents the data from the study.

Table 4-2 – Clay Ball Mass Degradation

Sample No.	Station No.	Plasticity Index	Retention Factor = Fill/Cut Ratio				
			8,500'	20,000'	25,000'	30,000'	35,000'
93-12/20	108+400	67	0.97	0.94	0.93	0.91	0.90
93-13/20	106+600	61	0.99	0.98	0.97	0.97	0.96
93-11/20	111+600	59	0.96	0.90	0.87	0.86	0.84
93-14/15	103+400	58	0.92	0.82	0.78	0.73	0.70
93-13/15	106+600	49	0.82	0.67	0.61	0.58	0.54
93-15/15	101+600	49	0.84	0.64	0.55	0.45	0.35
93-05/07	127+000	48	0.96	0.91	0.88	0.86	0.84
93-12/15	108+400	48	0.89	0.78	0.73	0.69	0.66
93-12/15	108+400	48	0.93	0.86	0.76	0.81	0.78
93-11/15	111+600	39	0.89	0.78	0.74	0.70	0.66
93-08/17	118+400	37	0.78	0.59	0.50	0.47	0.43
93-07/16	121+600	29	0.70	0.45	0.35	0.29	0.22
93-02/08	134+000	25	0.85	0.71	0.65	0.60	0.56
93-04/08	130+500	24	0.25	0.06	0.04	0.00	0.00
93-09/11	116+600	23	0.30	0.08	0.04	0.01	0.00
93-06/10	123+000	19	0.55	0.25	0.18	0.11	0.09

Under the Unified Soil Classification System (USCS, ASTM D2487) clays are classified as low plasticity (lean clay, CL) or high plasticity (fat clay, CH). The boundary between CL and CH clays is a Liquid Limit of 50. For the purposes of this evaluation we will apply the study data for PI greater than 30 to fat clays (CH) and the study data for PI less than 30 to lean clays (CL).

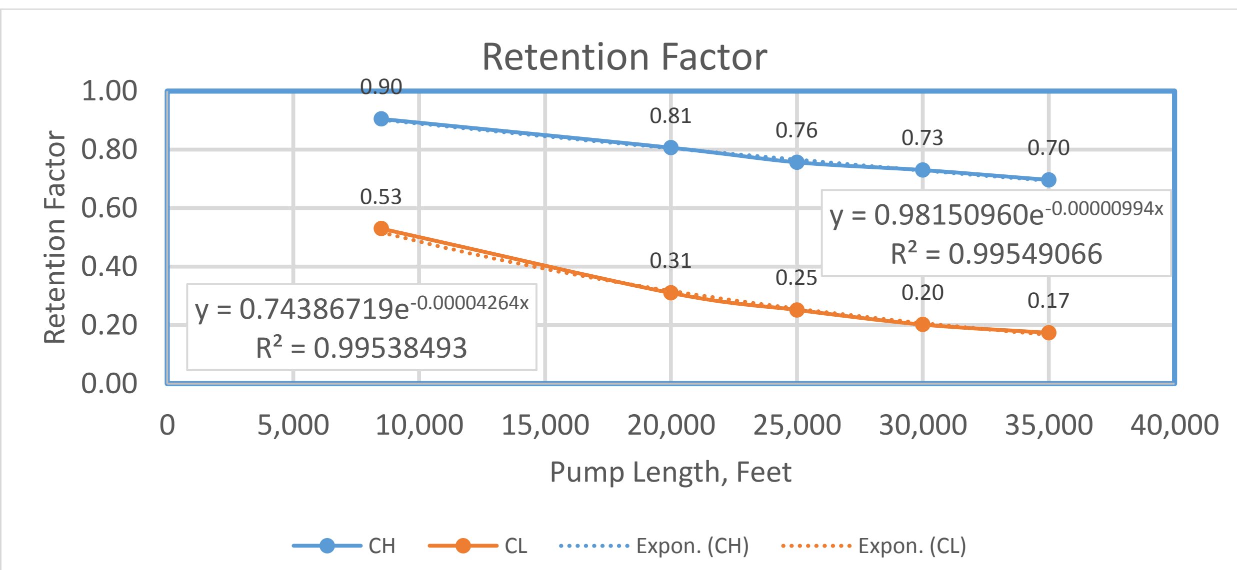


Figure 4-2 – Retention Factor for Fat Clay and Lean Clay

In order to develop an average retention factor we determined the mean for all samples with Plasticity Index (PI) greater than 30 and all samples with a PI less than 30 for each pump distance included in the study. Based on these means we developed relationships for Retention Factor. These factors produce results consistent with the experience at the Gorini Marsh discussed previously.

The portion considered retained will create coarse-grained clay ball fill which is will have a unit weight of about 100 pcf. The portion of the firm to stiff clay that is not considered retained mixes with the discharge water and is deposited through sedimentation in the site interior as very soft to soft clay material and becomes fine-grained fill.

Loose to Dense Sand
Sand typically does not suffer transportation losses during dredging, and usually has a loose consistency after discharge. Typically, a fill to cut ratio of 1.0 is assumed for the sand. An approximate unit weight of 110 pcf should be used for sand deposited inside a placement area.

Very Soft to Soft Clay and Silt
After dredging and transport the soft clay and silt material mixes with the discharge water and is deposited through sedimentation in the site interior. About 10% to 15% of this material is typically sand. The density of the remaining material can be determined based on settling column tests and the SETTLE program which is part of the USACE ADDAMS software as described in Section 4.5.

Conclusion and Recommendation
We recommend the following Retention Factors (fill/cut ratios) for firm to stiff clay:

$$For\ CH\ Clay, RF = 0.982e^{-0.00000994 \times Pump\ Length}$$
$$For\ CL\ Clay, RF = 0.744e^{-0.0000426 \times Pump\ Length}$$

In Excel the formula is =EXP(-0.00000994*K29)*0.982 for CH and =EXP(-0.0000426*K29)*0.744 for CL where K29 is the cell containing the pipeline length.

A summary of Retention Factors for discharge into a placement area for all material types based on pump distance is presented below.

Table 4-3 – Retention Factor Based on Material Type

Cut Material	Retention Factor = Fill/Cut Ratio			
	Pump < 10,000 feet	**Pump 10,000 to 20,000 feet**	**Pump 20,000 to 30,000 feet**	**Pump 30,000 to 40,000 feet**
Firm to Stiff Lean Clay (CL)*	0.53	0.40	0.25	0.17
Firm to Stiff Fat Clay (CH)*	0.9	0.84	0.76	0.70
Very Soft to Soft Clay/Silt*	0.1	0.1	0.1	0.1
Sand, Clayey Sand, Silty Sand	1.0	1.0	1.0	1.0

* Remainder of this material is suspended in the discharge water and deposits in the placement area through sedimentation.

4.5 Site Capacity

In order to establish the capacity of a placement area the fill volume immediately after dredging must be determined. We evaluated the initial storage volume based on procedures discussed in U.S. Army Corps of Engineers Engineer Manual EM 1110-2-5025, Dredging and Dredge Material Management. Figure 4-13 from the EM is shown below illustrating that the coarse-grained fraction comprised of sand and clay ball material accumulates near the discharge point and the fine-grained material flows into the remainder of the site as a slurried clay.

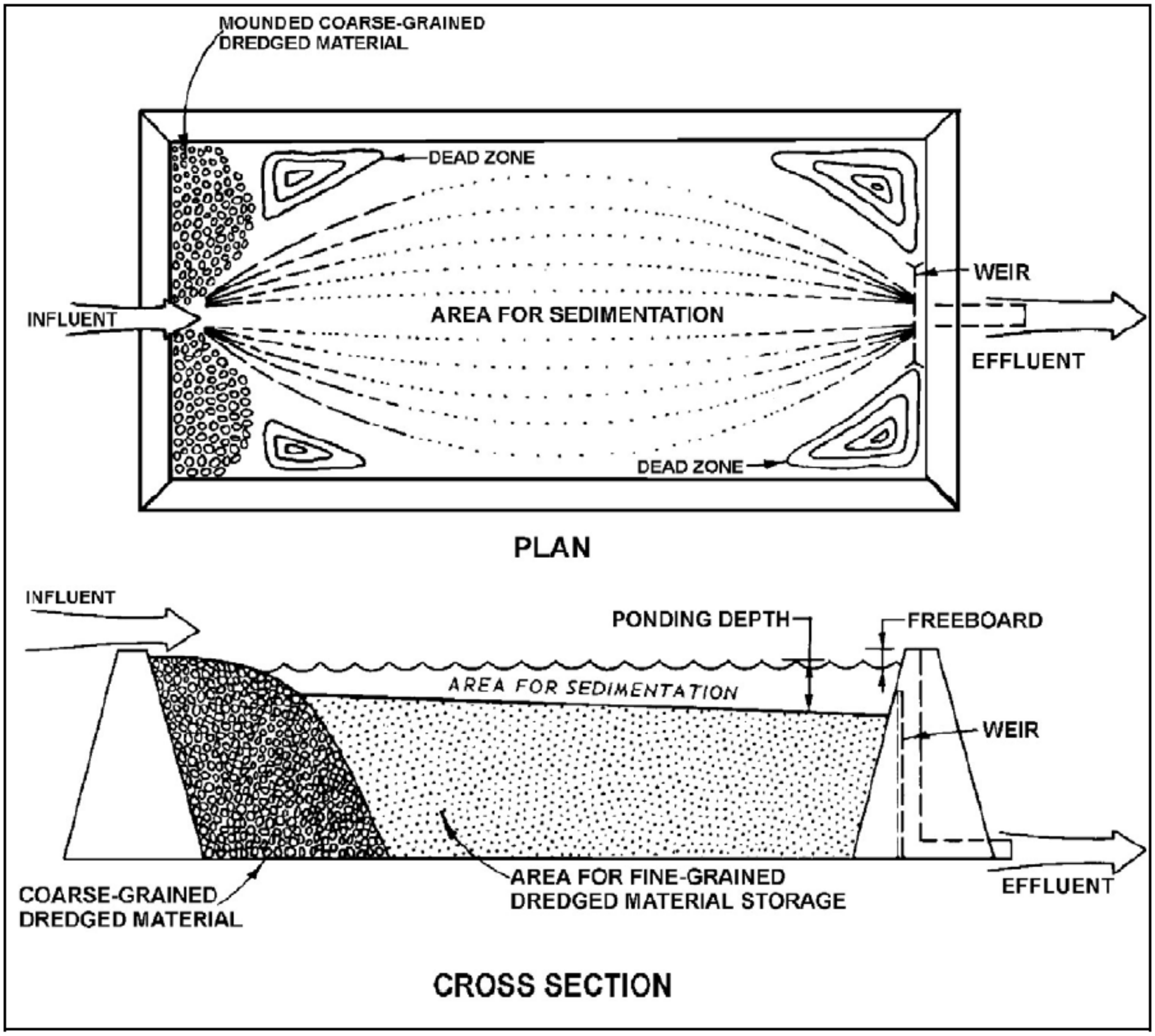


Figure 4-13. Conceptual Diagram of a Dredged Material Containment Area

These procedures use the settling column test results to estimate the fine-grained fill void ratio after dredging depending on the duration of dredging. The program SETTLE was used to develop the relationship between fill concentration and time as described in Technical Note EEDP-06-18, Documentation of the SETTLE Module for ADDAMS, dated December, 1992.

The SETTLE program is based on interpreting a relationship between concentration and time based on the initial solids concentration of the test and the rate that the interface between water and soil moves down during the test. Settling column test results for new work dredging in the Houston Ship Channel Bay reach were used for the analysis. The program calculates the following relationship from lab data on fine-grained material:

$$C = R\,t^{s}$$

where: C = concentration, grams per liter,
R and s = correlation coefficients, and
t = time after start of dredging, days.

In addition to the material properties, the other data required for the analysis are the area of the site, the volume of in situ material to be dredged, the percent coarse material in the in situ soil, and the duration of dredging. The site area is established during the design and volume of in situ material to be dredged is estimated. The percent of coarse material in the in situ soil is determined based on laboratory test results for the sand portion and retention analysis as described in Section 4.4 for the clay ball portion. The duration of dredging is estimated based on anticipated performance of the construction equipment.

4.6 Dredgability

The ability to excavate underwater with respect to known or assumed equipment, methods, and in situ material characteristics is referred to as dredgability. In the Houston Ship Channel and adjacent channels all dredging is in soil of varying types and consistency/density. The dredgability of the soils depends on the equipment used. Suction cutterhead dredges are most commonly used for channel dredging project in the Houston Ship Channel and adjacent channels. The discussion in this section is limited to suction cutterhead dredges.

There are two types of dredging performed – maintenance and new work. Maintenance dredging is removal of material that has accumulated within the limits of a previously dredged channel. New work dredging is excavation to widen and/or deepen an existing channel. Both types of dredging are discussed below.

The following discussion is intended to address issues that might arise during construction. Our recommendations are intended for use as guidelines in dealing with particular soil conditions. The recommendations contained herein are not intended to dictate construction methods or sequences. Instead they are provided solely to assist designers in identifying potential construction problems related to dredging plans and specifications, based upon findings derived from sampling.

Prospective contractors for the project must evaluate potential dredging problems on the basis of their review of the contract documents, their own knowledge of and experience in the local area, and on the basis of similar projects in other localities, taking into account their own proposed methods and procedures.

4.6.1 Maintenance Dredging

The typical material encountered in maintenance dredging is very soft clay with about 10% to 15% sand. This material is dredgable by even small diameter suction, cutterhead dredges. Stronger material may be encountered in areas where bank sloughing has occurred. This may happen in localized areas due to construction activities or vessel impacts. Some sloughing should be expected after a widening or deepening project when soil that was loosened during the new work dredging sloughs into the channel over time.

4.6.2 New Work Dredging

Natural soils that must be excavated to widen or deepen the Houston Ship Channel and adjacent channels typically consist of very soft to soft clay, firm to hard clay, and sand. Maintenance material may also be dredged. Smaller dredges that easily excavate maintenance material may have difficulty excavating new work material.

Larger dredges will generally have less difficulty excavating new work material. However, two dredges of the same size may have different production rates due to myriad factors such as the age and condition of the dredge, details of the cutterhead design, and operating procedures.

Very soft to soft clay new work material is different than maintenance material in that it is denser and stronger. It has been deposited over thousands of years in comparison to maintenance material which has typically been deposited over very short times ranging from 1 to 10 years. This material is usually encountered above firm to hard clay or medium dense to very dense sand and dredges that can excavate these soils are able to excavate the weaker clays.

Firm to Hard Clay was deposited before the last ice age and at that time was emergent ground above sea level. During this time the clay became much stronger. Consistency descriptors used on the boring logs for the project are presented in the table below.

Table 4-4 – Clay Soil Consistency

Consistency	Undrained Shear Strength (tsf)	Penetration Resistance "N" * Blows/Foot
Very Soft	0 - 0.125	0 - 2
Soft	0.125 - 0.25	2 - 4
Firm	0.25 - 0.5	4 - 8
Stiff	0.5 - 1.0	8 - 16
Very Stiff	1.0 - 2.0	16 - 32
Hard	> 2.0	> 32

*"N" refers to the number of blows required to penetrate the final 12 inches per ASTM D-1586

Firm to very stiff clay is common in new work dredging in the area and local experience with production rates should be indicative of what to expect. Hard clays can be encountered but are usually below the dredge template. Where hard clays are encountered within the dredge template a significantly reduced production rate should be expected.

Sand layers in the area were also usually deposited before the last ice age and became denser during this time. Density descriptors used on the boring logs for this project are presented in the table below.

Table 4-5 – Density of Sands

Descriptive Term	Penetration Resistance "N" * Blows/Foot
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	> 50

*"N" refers to the number of blows required to penetrate the final 12 inches per ASTM D-1586

Very loose to dense sand is common in new work dredging in the area and local experience with production rates should be indicative of what to expect. Very dense sand can be encountered and a significantly reduced production rate should be expected.

Cemented sand have been encountered in the Houston area. Cemented sands are indicated by high SPT N values in sand layers. Sampler blow counts in excess of 50 for 6 inches are typical, which correspond to N values in excess of 100. They are bound together and behave as weak rock. Suction cutterhead dredges will have difficulty excavating cemented sands.

5 HSC BAYOU REACH

5.1 Generalized Soil Conditions

Borings ECP-401D thru ECP-425D excluding ECP-404, ECP-411D and ECP-423D were drilled adjacent to existing channel to a depth of -60 feet MLLW. The depth of water at the boring locations varied between 7 and 37 feet. In general, very soft to hard clays and very loose to very dense sands were observed in the borings.

5.2 Dredgability

The following discussion is intended to address issues that might arise during construction. Our recommendations are intended for use as guidelines in dealing with particular soil conditions. The recommendations contained herein are not intended to dictate construction methods or sequences. Instead they are provided solely to assist designers in identifying potential construction problems related to dredging plans and specifications, based upon findings derived from sampling.

Prospective contractors for the project must evaluate potential dredging problems on the basis of their review of the contract documents, their own knowledge of and experience in the local area, and on the basis of similar projects in other localities, taking into account their own proposed methods and procedures.

Dredgability is discussed in Section 4.6. The following table presents borings which encountered very dense sand or hard clay above El. -50. In the table "N" refers to the number of blows required to penetrate the final 12 inches per ASTM D-1586.

Table 5-1 – Very Dense or Cemented Sand/Hard Clay, HSC Segment 4

Boring	Elevation (MLLW), Feet	Soil Type	"N" Value	Undrained Shear Strength, tsf
ECP-401D	-34	Sand	56	--
	-40	Sand	72	--
	-42	Sand	88	--
	-46	Clay	--	2.24
	-48	Clay	36	--
ECP-403D	-35	Clay	46	--
	-39	Clay	38	--
	-43	Cemented Sand	79/4.5"	--
	-48	Cemented Sand	75/5"	--
ECP-405D	-41	Clay	38	--
	-45	Clay	34	--
ECP-406D	-38	Clay	43	--
	-42	Clay	38	--
	-44	Clay	--	3.20
	-46	Clay	47	--
ECP-407D	-45	Clay	33	--
ECP-408D	-39	Clay	36	--
ECP-409D	-38	Sand	51	--
	-46	Clay	34	--
ECP-410D	-23	Clay	--	2.04
	-33	Clay	33	--
	-45	Clay	50/5.88"	--
ECP-412D	-40	Clay	45	--
	-44	Sand	62	--
	-50	Sand	61	--
ECP-413D	-36	Clay	36	--
	-49	Clay	39	--
ECP-414D	-35	Clay	--	2.23
	-41	Cemented Silt	50/4"	--
	-50	Silt	65	--
ECP-415D	-45	Clay	37	--
	-49	Clay	68/1"	--
ECP-418D	-35	Clay	55	--
	-37	Clay	46	--
	-43	Clay	36	--
ECP-419D	-43	Cemented Sand	50/4.5"	--
	-45	Clay	50/4.25"	--
	-47	Clay	50/3"	--
	-49	Clay	50/3.2"	--
ECP-420D	-28	Clay	--	2.53
	-30	Clay	64	--
	-40	Clay	39	--
	-44	Clay	40	--
ECP-421	-43	Clay	--	2.00

5.3 Slope Stability

Analyses were performed for the short term and long term cases. Since the slopes are submerged rapid drawdown analyses were not performed. We have performed the analyses at HSC centerline Stations 702+00, 720+00, 742+00, 760+00, 800+00, 803+26, 808+00, 830+00, 876+00 and 918+00. These locations were chosen based on relative strength of the soils as revealed by the boring logs and the variation in the channel cross section.

The cross sections, soil parameters used in the slope stability analyses, and the slope stability outputs are presented in Appendix A. We assumed the water level at El. 0.0 feet MLLW. The results are summarized in the table below, green side is to the south of the channel and red side is to the north.

Table 5-2 – HSC Slope Stability Results – Proposed 3H:1V Template

Station	Factor of Safety	
	Short Term	Long Term
702+00 Green Side	4.45	2.30
720+00 Red Side	1.48	1.55
742+00 Red Side	1.97	1.53
760+00 Red Side	2.40	2.48
800+00 Red Side	1.30	1.68
803+26 Red Side	1.31	1.70
808+00 Green Side	6.10	1.60
830+00 Green Side	1.65	1.77
876+00 Green Side	2.22	2.01
918+00 Green Side	1.94	1.94

The stability analyses meet or exceed the required minimum factors of safety. In addition to the proposed 3H:1V template, we evaluated the feasibility of 2.5H:1V slope along the alignment. The results are summarized in the table below.

Table 5-3 – HSC Slope Stability Results – 2.5H:1V Template

Station	Factor of Safety	
	Short Term	Long Term
702+00 Green Side	4.16	2.44
720+00 Red Side	1.34	**1.34***
742+00 Red Side	1.74	**1.32***
760+00 Red Side	4.50	2.24
800+00 Red Side	**1.17***	**1.44***
808+00 Green Side	2.34	1.60
830+00 Green Side	1.66	1.75
876+00 Green Side	2.00	2.31

Station	Factor of Safety	
	Short Term	Long Term
918+00 Green Side	1.94	1.94

* Does not meet the minimum required factor of safety.

The stability analyses does not meet the required minimum factors of safety at Stations 720+00, 742+00 and 800+00 Red Side.

5.4 Recommendations

Based on the soil parameters and water level discussed earlier, we evaluated the proposed dredge template comprising 3H:1V slopes to El. -50.5 feet. The global stability analyses indicate that the proposed 3H:1V template has an adequate factor of safety.

The results of 2.5H:1V slope stability analyses confirm that the Red Side up to Station 804+00 needs to remain at 3H:1V. A 2.5H:1V side slope meets the required factors of safety on the Red Side beyond Station 804+00 and on the entire Green Side.

There are several facilities located immediately adjacent to the channel slopes and the vertical grade beyond the limits of the cross sections appears to be substantially higher based on the aerial images. The scope of this study did not include sufficient survey data or geotechnical borings to allow us to evaluate these facilities. At the locations listed below the global slope stability factor of safety is less than on the nearby analyses listed in Section 5.2. These locations, and any other similar locations, must be investigated to assure that dredging will not cause damage. Additional survey and geotechnical information are required for comprehensive stability analysis.

Table 5-4 – Adjacent Facilities Requiring Additional Analysis

Approximate Station
686+00 Red
724+00 Red/Green
726+00 Green
742+00 to 756+00 Red
780+00 Red
824+00 to 826+00 Red
890+00 Red

We understand that the existing foundations for the Beltway 8 bridge are within the dredge template. The bridge is being reconstructed, and the foundations for the new bridge are outside the dredge template. We recommend that the top of cut remain at least 100 feet away from the existing bridge pylons. If the planned cuts are closer than 100 feet an evaluation of the impact on the foundation capacity should be performed.

6 BAYPORT SHIP CHANNEL

In general the stability analysis for Bayport Ship Channel is being performed by others. In the original project scope the design of a bulkhead to protect the existing San Jacinto Junior College (SJC) site was included in the scope of this study. Based on recent information provided to us, we understand that the channel will be tapered as it approaches the Bayport Turning Basin to avoid the need for a bulkhead at the SJC school site.

6.1 Generalized Soil Conditions

Two borings (ECP-205A and ECP-206A) were drilled by others to 60 feet or deeper below the base of the channel adjacent to the SJC site. The depth of water at the boring locations varied between El. -38 and -41 feet MLLW. Firm to very stiff clays were encountered to about El. -55 feet with loose to medium dense sands below to at least El. -80 feet.

We have reviewed the available borings from the previous studies and the land borings performed at the SJC site for this study. In general, firm to very stiff clays were observed at the surface underlain by thick medium dense to very dense sand or silt layer. Firm to very stiff clays were encountered below the sand or silt layer.

6.2 Slope Stability

Analyses were performed for the channel section adjacent the San Jacinto College site at BSC centerline Stations 42+20, 40+00 and 37+00. These locations were chosen based on relative strength of the soils as revealed by the boring logs and the variation in the channel cross section.

The cross sections, soil parameters used in the slope stability analyses, and the slope stability outputs are presented in Appendix B. We assumed the water level at EL. +2.0 feet MLLW for the short and long term cases. In the Rapid Drawdown analysis, a drawdown of the water level was assumed from El. +12.49 to El. -3.69 feet MLLW to reflect the impact of hurricane surge on the slopes. The results are summarized in the table below, the Red Side is the north side of the channel.

Table 6-1 – BSC Slope Stability Results

Station	Factor of Safety		
	Short Term	Long Term	Rapid Drawdown
42+20 Red Side	2.03	2.03	2.02
40+00 Red Side	2.96	2.36	2.16
37+00 Red Side	2.98	2.28	2.05

The stability analyses meet or exceed the required minimum factor of safety discussed in Section 4.1.

6.3 Recommendations

Based on the soil parameters and water level discussed earlier, we evaluated the proposed dredge template comprising 3H:1V slope. The global stability analyses indicate that the proposed template has an adequate factor of safety.

7 BARBOURS CUT SHIP CHANNEL

7.1 Dredgability

The following discussion is intended to address issues that might arise during construction. Our recommendations are intended for use as guidelines in dealing with particular soil conditions. The recommendations contained herein are not intended to dictate construction methods or sequences. Instead they are provided solely to assist designers in identifying potential construction problems related to dredging plans and specifications, based upon findings derived from sampling.

Prospective contractors for the project must evaluate potential dredging problems on the basis of their review of the contract documents, their own knowledge of and experience in the local area, and on the basis of similar projects in other localities, taking into account their own proposed methods and procedures.

Dredgability is discussed in Section 4.6. The only boring performed for this study near the channel is ECP-316. Additional borings were previously performed for HVJ Report No. HG1010561 dated October 17, 2013. Borings L-05 to L-08 from that study are within the material to be dredged. The material encountered in the borings was firm to very stiff clay. No "N" values were obtained. The undrained shear strengths measured on samples from the borings were all less than 2 tsf which indicates that no Hard Clays were encountered.

We should note that borings S-01 to S-04 drilled for the previous study did have "N" values in the same layer. These borings were drilled in material that was dredged during a previous widening of the Barbours Cut channel. The undrained shear strength measurements were similar to ECP-316 and L-05 to L-08 and were all less than 2 tsf. The SPT "N" values exceeded 32 at some test locations which would indicate a consistency of "hard" as discussed in Section 4.6. For geotechnical engineering purposes the undrained shear strength measured in laboratory tests is considered the more reliable indicator of clay consistency.

In the borings reviewed we observed soil condition consistent with new work dredging along the Houston Ship Channel.

7.2 Spilmans Island

7.2.1 General

The following are excerpts from HVJ Report 93-249G dated August 29, 1994 which addressed geotechnical conditions at Spilmans Island Placement Area

> *Spilmans Island is located between the Houston Ship Channel (HSC) stations 20+00 and 130+00 on the west side of the HSC. The island is bounded on the north and east sides by the HSC, on the south by Barbours Cut, and on the west side by the Lower San Jacinto Bay. Figures 7-1 and 7-2 show the Spilmans Island area in 1921 and between 1950 and 1995, respectively. As can be seen, placed dredge material has filled a portion of the San Jacinto Bay to the south resulting in the connection of the island to the mainland. At the time of preparation of this report, Spilmans Island had a usable disposal area of 925 acres.*
>
> ***Site Description***
>
> *Levees with slopes of 3 horizontal to 1 vertical (3H:1V) have been constructed along the perimeter of Spilmans Island to contain the dredge material. Levees were raised to elevation +20.0 feet MLT in 1979. In 1989, levees were raised to elevation +25.0 feet MLT, and in March 1993, attempts were made to raise to elevation +32.0 feet. However, a slope failure occurred at about station 192+50 adjacent to the Barbours Cut Turning Basin, and levee raises were limited to elevation +28.0 feet. Signs of a foundation type slope failure were also evident at station 104+50 at the northeastern corner of the island. At other levee locations, elevations presently vary between 30.0 and 32.0 feet MLT, and levees appear to be generally in a good condition.*

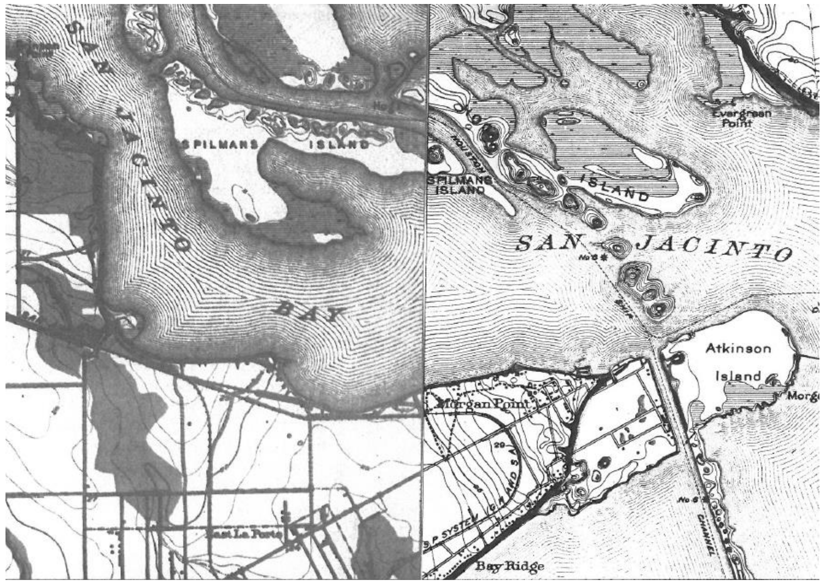


Figure 7-1 – Spilmans Island – 1921 USGS Topographic Maps

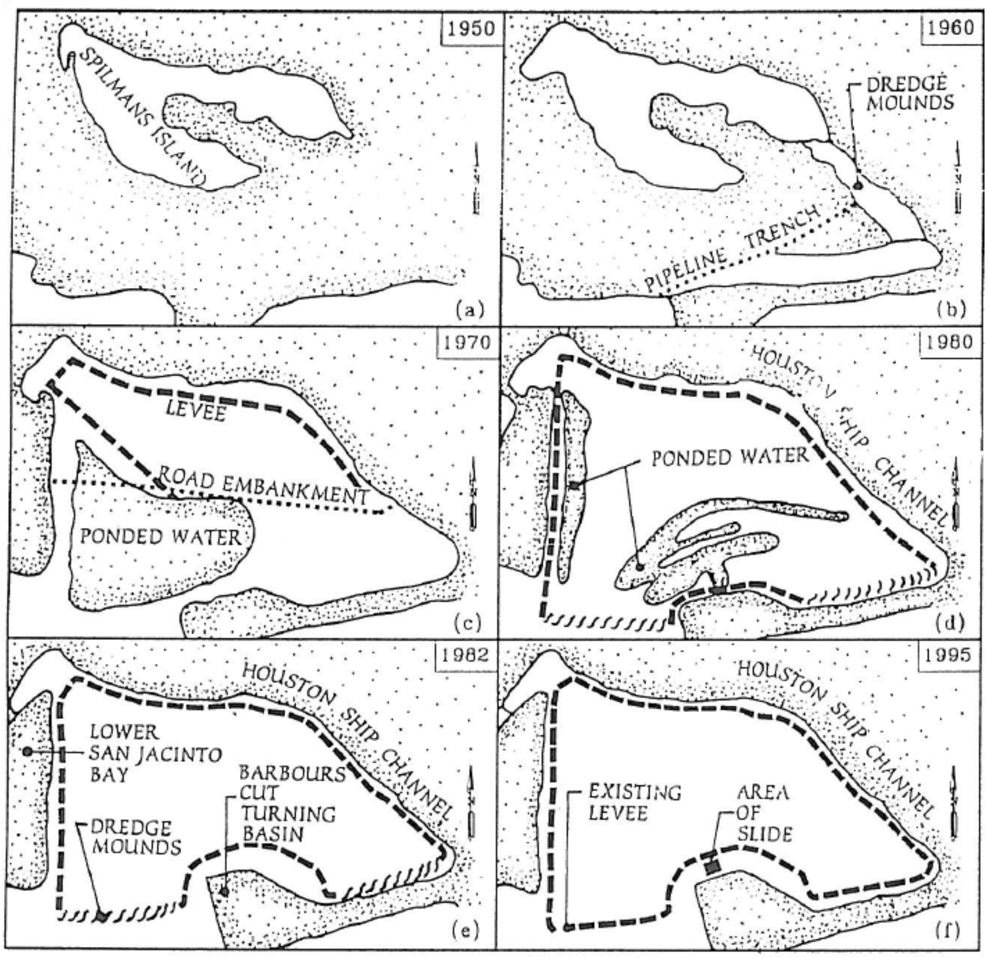


Figure 7-2 – Spilmans Island Filling History, 1950 to 1995

In recent years the Barbours Cut Ship Channel immediately south of Spilmans Island has been widened to north on two occasions. Existing dock structures were encountered during the most recent widening which occurred in about 2015.

The proposed channel widening for this project will impact the Spilmans Island Placement Area. A cutoff wall will be required to provide adequate global stability for the north slope of the channel and to mitigate encroachment of channel slope into the Spilmans Island containment dike. We understand that the berm located north of the proposed cutoff wall will also be realigned. In addition to the cutoff wall adjacent to the Spilmans Island, a pipeline protection wall is required to retain ground so that the existing pipelines can remain at their current elevations between about channel centerline Sta. 59+00 and Sta. 65+00.

For the cutoff and pipeline protection wall designs a future channel cut elevation of -61 feet was considered to allow for future deepening of the channel beyond that planned for this project. The proposed cross sections were evaluated, results of the stability analysis are presented in the following sections.

7.2.2 Generalized Soil Conditions

Soil Borings: We have reviewed the available borings from the previous studies (land borings, L-01 thru L-08 and marine borings S-01 thru S-04) and the supplemental land borings ECP-309 thru ECP-316 performed for this study. In general, very soft to hard clays were predominantly encountered with intermittent very loose to very dense sand layers of varying thickness in borings L-01, L-03, L-04, L-06, L-08, S-01, S-03, S-04 and ECP-309 thru ECP-316. The groundwater depth varied between 5 and 14 feet below the existing grade at the time of drilling.

Vane Shear (VST) and Piezocone Penetrometer Testing (PCPT): Soft clay layers were observed in borings taken from the dike crest which imposed substantial impacts to the design of the proposed channel slopes. We further investigated the strength of this weak material by conducting a VST and PCPT testing program. We performed PCPT and VST tests near borings L-02, L-03 and L-04 to a depth of 60 feet each below the existing grade. The PCPT and VST data was analyzed to determine the shear strength of the soft clay material and its approximate elevation. The PCPT data was primarily used to determine the stratigraphy at the test locations, while the VST data was used to assess clay soil shear strength. The VST test determines clay undrained shear strength in situ by measuring the torque required to turn a vane of know size inserted into the soil at the selected test depth. In lab testing by unconfined compression or unconsolidated-undrained triaxial test soil disturbance during sampling and sample transport affects the results reducing the measured undrained shear strength. VST testing dramatically reduces the amount of soil disturbance and its impact on measured undrained shear strength.

For PCPT data the shear strength is estimated based on a cone factor, N_k that relates the tip resistance measured by the PCPT to shear strength. In order to determine the appropriate cone factor for this project we compared the PCPT tip resistance to mobilized undrained shear strength from the Field Vane testing.

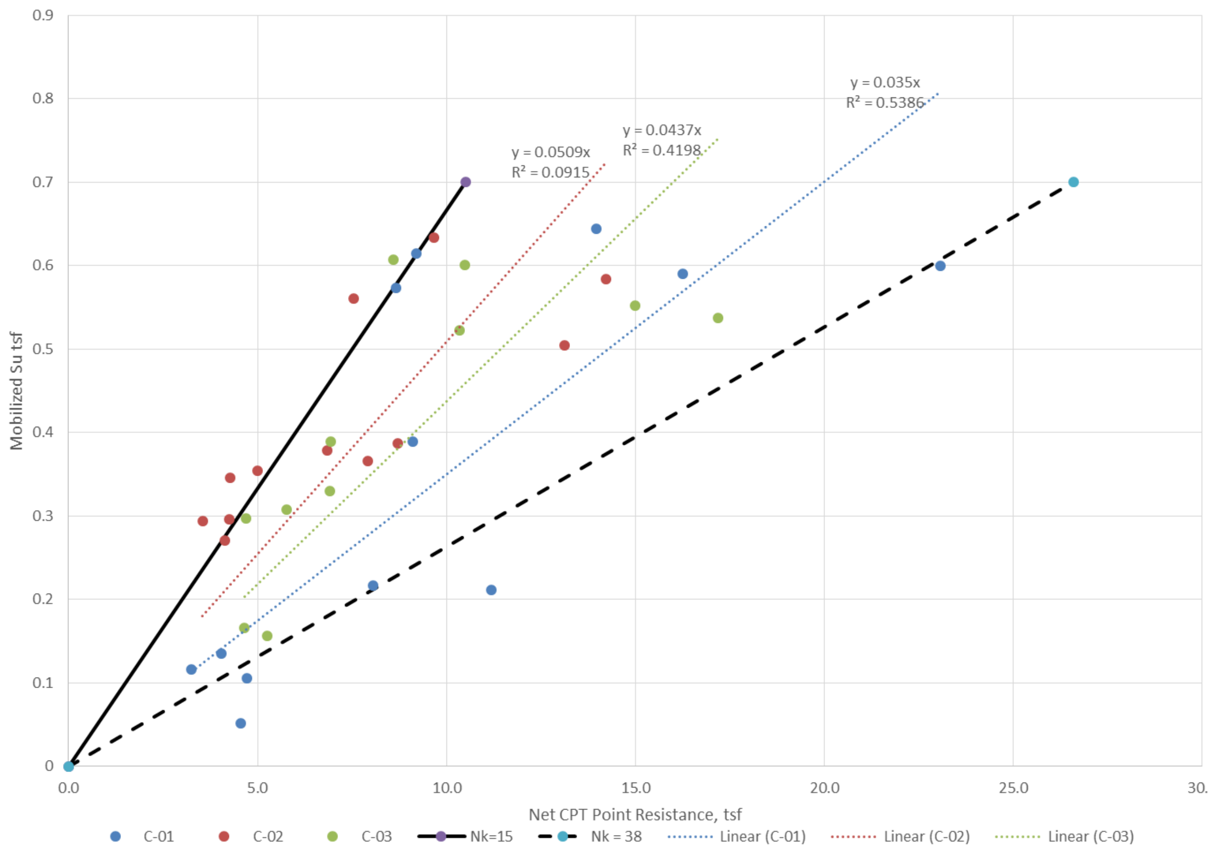


Figure 7-3: Determination of Nk (FV Mobilized Strength)

In Figure 7-3 the Net Point Resistance was calculated by subtracting the effective overburden pressure from the measured PCPT Tip Resistance. Based on the available data represented in Figure 7-3 an upper and a lower bound N_k was obtained as 15 and 38 respectively. The upper and lower bound N_k provided a range of undrained shear strength values for the subsurface soils. An N_k value of about 15 is often considered reasonable, we can see from Figure 7-3 that field vane data shows that much higher N_k values are representative of some of the soils encountered for this investigation.

7.2.3 Design Cross Section

Slope stability analyses were performed for the End of Construction, Long-Term, and Rapid Drawdown conditions. Based on the cross-sections provided to us and the soils information obtained from subsurface investigation conducted by us, we have analyzed the slope stability at channel stations 34+00, 44+00, 56+00, and 64+00. The cross sections, soil parameters used in the slope stability analyses, and the slope stability outputs are presented in Appendix C. We assumed the water level at El. +1.0 feet MLLW for the short and long term cases in the channel. In the Rapid Drawdown analysis, a drawdown of the water level was assumed from El. +12.49 to El. -3.69 feet MLLW to reflect the impact of hurricane surge on the slopes. The water level was assumed at existing grade in the site interior whereas it was assumed 2 feet below the future dike height allowing 2 feet of freeboard in the Long Term and Rapid Drawdown analyses. The slope stability analyses outputs are presented in Appendix C and the results are summarized in the table below.

Table 7-1 – BCC Slope Stability Results at Spilmans Island – Bulkhead at 250 feet from Toe

Station	Factor of Safety					
	Short Term - Exterior		Short Term – Interior		Long Term	Rapid Drawdown
	Global	Local	Circular	Block		
34+00	1.56	2.28	**1.20***	**0.89***	1.56	1.52
44+00	1.51	2.36	2.28	2.33	1.56	1.48
56+00	1.57	3.93	3.90	4.25	1.57	1.48
64+00	1.54	3.75	8.84	9.98	1.60	1.49

* Does not meet the minimum required factor of safety.

The stability analyses at all locations assume a cutoff wall with tip elevation at -58 feet MLLW located at 250 feet from the proposed channel slope toe. The analyses at Station 64+00 assume a pipeline protection wall with tip elevation at -90 feet MLLW located at 70 feet from the proposed channel slope toe. At Station 56+00 a level bench is included in front of the cutoff wall. This bench should extend from the end of the pipeline protection wall on the west to Station 48+00 on the east.

The stability analyses meet or exceed the required minimum factor of safety except for short term interior stability at Station 34+00. An offset of 100 feet tot eh cutoff wall is needed, a typical section showing the recommended berm configuration is presented in Figure 7-4. The stability analysis results of the recommended slope are summarized in the table below.

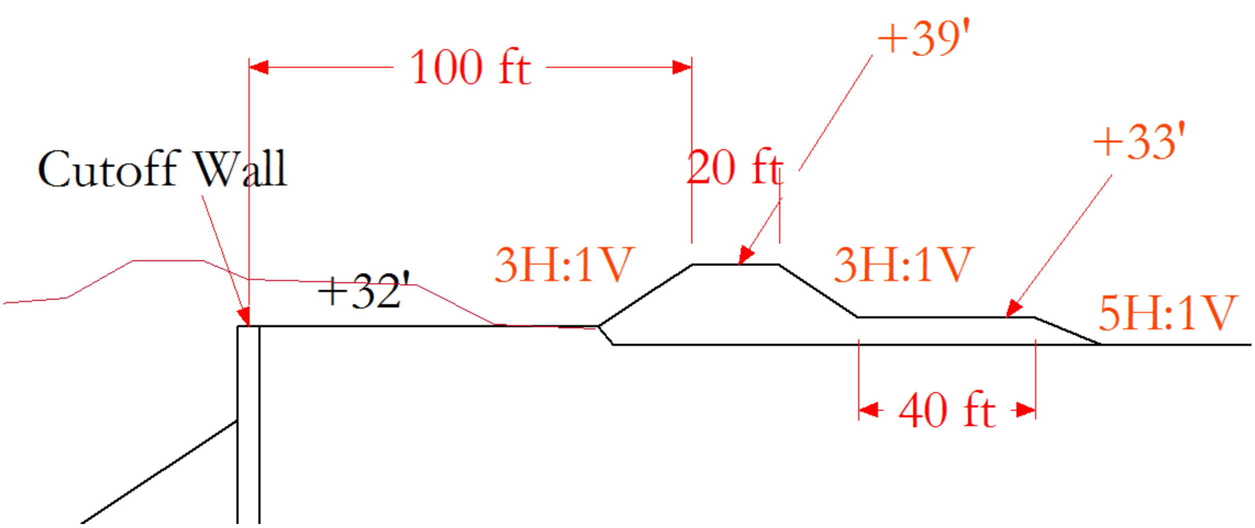


Figure 7-4: Recommended Configuration for Realigned Berm at +39' MLLW

Table 7-2 – BCC Slope Stability Results at Spilmans Island – Realigned Berm at El. +39' Recommended Configuration

Station	Factor of Safety					
	Short Term - Exterior		Short Term – Interior		Long Term	Rapid Drawdown
	Global	Local	Circular	Block		
34+00 with int. berm at El. +33' and 5H:1V Slope	1.58	2.28	1.88	1.31	1.56	1.52

In addition to the proposed realigned berm at El. +39 feet MLLW, we evaluated the berm configuration and its offset needed to build the top of berm to El. +45 feet MLLW. A typical section showing the recommended berm configuration is presented in Figure 7-5. The results of the recommended slope are summarized in the following table.

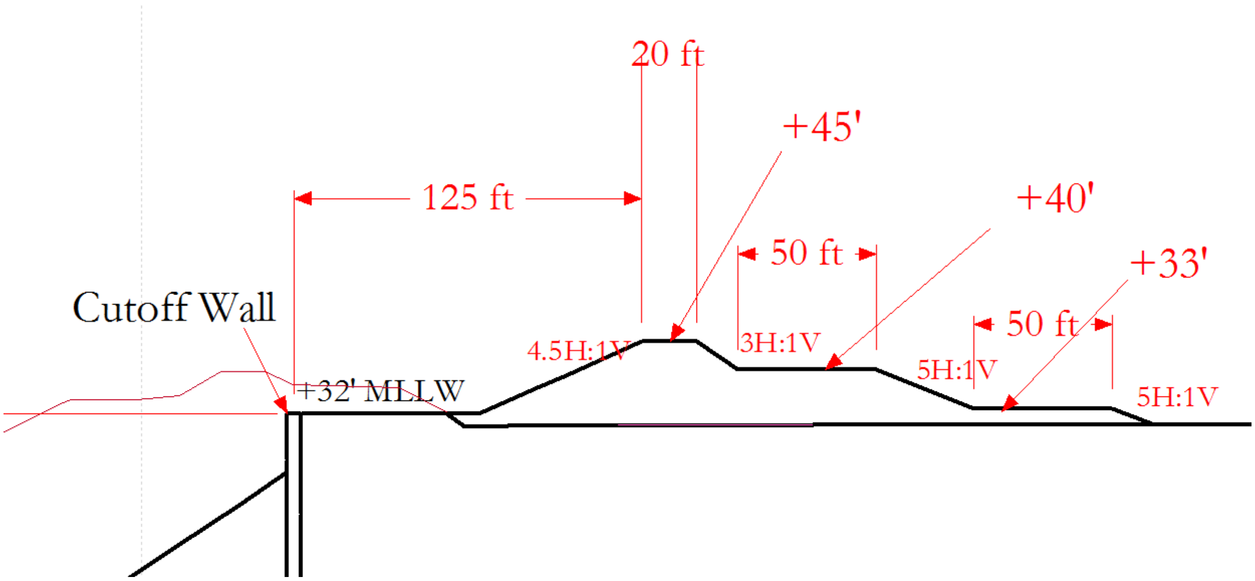


Figure 7-5: Recommended Configuration for Realigned Berm at +45' MLLW

Table 7-3 – BCC Slope Stability Results at Spilmans Island – Realigned Berm at El. +45' Recommended Configuration

Station	Factor of Safety					
	Short Term - Exterior		Short Term – Interior		Long Term	Rapid Drawdown
	Circular	Block	Circular	Block		
34+00	1.34	1.32	1.32	1.31	1.61	1.54
44+00	1.33	1.34	1.66	1.73	1.57	1.48
56+00	1.47	1.55	2.67	2.83	1.56	1.48
64+00	1.46	1.52	3.52	3.77	1.59	1.49

7.2.4 Bulkhead Design

We understand that the cutoff wall and pipeline protection wall will be constructed as "Combi-walls" which comprise open-ended steel pipe piles installed at intervals with steel sheet piles in between the pipe piles. The pipe piles provide the main structural capacity of the wall while the sheet piles provide continuity and transmit the soil loads to the wall. The sheet piles do not provide significant resistance to lateral loading in the Combi-wall system. Therefore, they do not need to be installed to the full depth of the pipe piles. We recommend that the depth of the sheet piles be based on the lateral load analysis of the wall, and the sheet piles can terminate below the point where lateral loading stops. The sheet piles can stop at a point 25% of the height of the wall or 5 feet below the bottom of the active/at rest soil pressure load on the wall. The bottom of the active/at rest soil pressure on the wall should be slightly below the elevation of the ground surface in front of the wall. Soil parameters for the bulkhead design are presented in Appendix C. In cohesionless soil layers wall friction on steel can be taken as 5° less than the angle of internal friction shown in Appendix C. In cohesive soil layers wall adhesion can be taken as one-half the undrained shear strength shown in Appendix C.

Soils in front of the pipeline protection wall are expected to be stiff to very stiff clay which is not highly erodible. We recommend that 5 feet of scour be assumed in front of the pipeline protection wall. The cutoff wall is set back about 70 feet from the shoreline for the ultimate assumed dredging to El. -61 and about 100 feet for the current project dredging to El. -51. If the shore protection recommendations discussed in Section 7.1.7 are implemented no allowance for scour needs to be included in the cutoff wall design. These conditions should be considered as "Usual" loading for the purpose of determining wall design factors of safety in accordance with EM 1110-2-250 Design of Sheet Pile Walls. To the extent that deeper scour is considered in the design such conditions can be considered as "Extreme" loading.

In addition to supporting lateral loads from the retained height of soil, the bulkhead is also providing lateral support to the slope required to achieve the design factor of safety for global stability. In the global stability analyses discussed in Section 7.1.3 the bulkheads were assumed to be very strong with the result that the critical failure surfaces do not pass through the bulkheads. To assess the slope load on the bulkhead we ran additional global stability analyses assuming that the bulkhead had a shear strength equivalent to an angle of internal friction of 45°. Where a critical slip surface passes through the bulkhead the forces acting on that slice indicate the slope loading placed on the bulkhead. These loads are summarized in Table 7-4, note that these loads are unfactored.

Table 7-4 – Slope Loading on Bulkhead, Spilmans Island

Station	Load Case	Downdrag, lb/foot of bulkhead	Shear Force, lb/foot of bulkhead	Shear Force Angle	Shear Force El., Feet MLLW
34+00, Cutoff	Long Term	948	1,702	29.26°	+17.73
	Short Term	5,713	5,080	27.13°	+1.84
44+00, Cutoff	Long Term	6,813	5,200	33.94°	-1.16
	Short Term	22,780	11,381	6.20°	-28.36
56+00, Cutoff	Long Term	5,263	5,423	27.00°	-0.36
	Short Term	23,643	13,078	4.24°	-30.06
64+00, Cutoff	Long Term	-102	2,069	29.89°	+22.19
	Short Term*	--	--	--	--
64+00, PPW	Long Term	13,151	23,436	25.06°	-48.00
	Short Term*	--	--	--	--

*No slope loading, critical slip circle passes below the bottom of the bulkhead

Active earth pressures may be used if the wall deflection is about 1% of the wall height, if the deflections are restricted at-rest pressures should be considered. As long as the wall is not protecting movement sensitive structures deflections of up to 2% of the wall height can be acceptable. Wall movements under extreme loading conditions should be limited to no more than 6 inches.

Soils in the Houston area are not generally corrosive. We have not made any specific tests of soil corrosivity for this study. The water in the Barbours Cur Channel is brackish, and corrosion consistent with a salt water environment should be expected.

7.2.5 Bulkhead Construction

Based on the construction cross sections provided to us, we understand that installation of the cutoff wall will require a large crane and surcharge loading of the crane during construction is a concern. For the analysis information on a Manitowoc 2250 Lattice Boom crawler crane was provide by HDR as shown in Appendix C. Based on this information we assumed a surcharge loading of 1,400 psf over a width 36 feet in the stability analysis. The crane will be supported on a prepared bench as shown in the cross sections presented in Appendix C. We considered a bench located along the current dike crest north of proposed cutoff wall location and a bench located along the dike slope south of the proposed cutoff wall in our analyses. In order to construct the pipeline protection wall a flotation channel will be needed for access of marine construction equipment, this flotation channel was included in the analyses at Station 64+00. The slope stability analyses outputs are presented in Appendix C and the results are summarized in the following table.

Table 7-5 – BCC Slope Stability Results at Spilmans Island – Wall Construction

Station	Crane North of Cutoff Wall/Along Crest (Above)			Crane South of Cutoff Wall/Along Slope (Below)
	Short Term - Exterior	Short Term – Interior Circular	Short Term – Interior Block	Short Term - Exterior
34+00	1.60	1.58	1.40	2.03
44+00	1.73	1.44	**1.23***	2.08
55+00	**1.03***	3.62	3.43	1.76
62+00	**0.94***	4.23	2.95	1.66

* Does not meet the minimum required.

We recommend only allowing the Manitowoc 2250 Lattice Boom crawler crane or similar to be located along the dike slope south of the cutoff wall location. Analyses for the crane located north the cutoff wall along the dike crest show the factor of safety is below acceptable levels on three cross sections, and near or below 1.0 at two of the sections.

We further evaluated the proposed cross sections north of the cutoff wall to estimate the allowable construction surcharge with adequate safety factor. In addition, analysis was performed to estimate the allowable construction surcharge adjacent to the 2H:1V floatation channel slope at Station 62+00. The results of our analyses are presented in Appendix C and are summarized in the table below. . The Contractor should use adequate matting to support cranes used to construct the bulkheads.

Table 7-6 – Wall Construction at Spilmans Island – Allowable Construction Surcharge

Station	Short Term - Exterior	Short Term – Interior Circular	Short Term – Interior Block	Construction Surcharge, psf
Crane North of Cutoff Wall/Along Crest (Above)				
34+00	1.60	1.58	1.40	1,400 (proposed)
44+00	1.79	1.66	1.32	1,200
55+00	1.31	4.81	6.80	700
62+00	1.32	5.93	7.10	500
North of 2H:1V Floatation Channel Slope near Pipeline Protection Wall				
62+00	1.32	NA	NA	900

7.2.6 Dike Fill

The proposed construction will require mechanical excavation of a substantial amount of existing soil from the exterior of the current dike. This material will be placed as fill elsewhere. Where such material is placed within the dike the recommendations in this section should be followed.

The excavation may encounter hydraulically placed fill from prior use of the site as a placement area. This material will require substantial drying and mixing with stabilizing agents such as lime to be used as dike fill. Borrow soils that are natural are stronger. These soils will not require stabilization for use as dike fill.

There are two types of fill that can be considered for dike construction – compacted and semi-compacted. Compacted fill is placed in loose lifts of 6 to 9 inches, compaction is controlled based on field density testing, and moisture is controlled within a relatively narrow range. Semicompacted fill is placed in loose lifts of 12 inches, compacted based on controlled movement of hauling equipment or limited passes of compaction equipment, and is placed at in situ moisture, although very wet fill is will require drying and/or treatment with a stabilizing agent such as lime. Strength of compacted fill is higher than semicompacted fill, therefore a dike constructed of compacted fill will require less volume that for semicompacted fill. However, the compacted fill is more expensive to produce and place, and constructability issues related to drying wet borrow material can prove challenging.

In the Houston area drying fill material in the winter months is essentially impossible due to periodic cold fronts that can bring substantial rainfall every 3 to 5 days and low temperatures. The ability of construction equipment to simply move about the site can be challenging due to wet conditions. In the summer drying can be accomplished, however, afternoon thunderstorms are common which impact drying operations. Due to these constructability issues we believe that dike fill material should be considered semicompacted fill.

Dike fill should be semicompacted fill compacted based on controlled movement of hauling equipment or limited passes of compaction equipment, and is placed at in situ moisture, although very wet fill is will require drying and/or treatment with a stabilizing agent such as lime. Fill material should be placed in loose lifts not exceeding twelve inches in thickness and should be compacted to 95 percent of Standard Proctor maximum dry density as determined by ASTM D698 without a moisture requirement. If former hydraulic fill material is encountered it should be placed outside the dike cross section. If it must be used as dike fill it needs to be dried and treated with lime prior to use. For estimating purposes assume 8% lime by dry weight. The actual percentage of lime used should be determine based on testing the borrow material during construction.

We assumed that the dike fill material will have an undrained shear strength of 600 psf for end of construction analyses and will have drained friction angle of 23° for long term analysis with drained cohesion of 100 psf. These are consistent with semicompacted fill.

7.2.7 Shore Protection

The post-dredging shoreline will be close to the location of the current dike crest. We reviewed Borings L-01, ECP-313, L-02, ECP-314, L-03, ECP-315, and L-04 to assess the potential soil conditions at the new shoreline. The boring locations are shown on Plate 1G. The elevations of interest are between about El. -4 and +6 feet MLLW. In all borings except ECP-315 firm to stiff clay soils were encountered. Sands that are susceptible to erosion were encountered in ECP-315. Soil conditions are variable over short distances and sand layers at the waterline may be encountered at other locations along the new shoreline.

Previous widening projects at Barbours Cut installed erosion protection comprised of 3-foot thick 55 to 1500 pound stone rip rap from El. -2 to +8 feet MLT which is equivalent to about El. -3.3 to +6.7 feet MLLW. The riprap was installed on a geotextile fabric underlayment. We recommend similar erosion protection for the new shoreline for this project from El. -5 to +7 feet MLLW. The gradation of the stone is shown in the table below.

Table 7-7 – Shore Protection Stone Gradation – Spilmans Island

Design Stone Dimensions*	Percent of Stone by Weight Less than Design Stone Size (%)	Weight of Design Stone Size (pounds)
8.5	0	<55
13 – 17.5	15	200 – 520
17 – 20	50	500 – 760
22 – 25	100	1,000 – 1,500

* Dimensions will depend on the specific gravity of the stone and the weight of the stone governs the gradation.

7.2.8 Recommendations

Based on the soil parameters and water level discussed earlier, we evaluated the proposed dredge template comprising 3H:1V slopes to El. -61 feet with a cutoff wall at an offset of about 250 feet from the proposed channel toe. Our analyses also included a pipeline protection wall near the turning basin. The stability analyses for a dike crest elevation of +39 feet MLLW meet or exceed the required minimum factor of safety except for short term interior stability at Station 34+00. We recommend interior berm is offset at least 100 feet between the cutoff wall and top of berm and the cross section is as shown in Figure 7-4. The global stability analyses indicate that the proposed cutoff wall will need to extend to at least -58 feet MLLW for global stability. The proposed pipe protection wall will need to extend to at least -90 feet MLLW. Note that these elevations are based solely on global stability and do not consider the length of wall needed based on structural design.

In addition to the proposed realigned berm at El. +39 feet MLLW, we evaluated the berm configuration and its offset needed to build the top of berm to El. +45 feet MLLW. Our analysis indicates that an offset of 125 feet is required between the cutoff wall and top of berm and the cross section is as shown as shown in Figure 7-5.

The bulkhead should be designed based on the soil properties given in Appendix C and should include the slope loads shown in Table 7-4. Downdrag does not need to be considered in the design. We recommend that 5 feet of scour be assumed in front of the pipeline protection wall. If the shore protection recommendations discussed in Section 7.1.7 are implemented no allowance for scour needs to be included in the cutoff wall design. The walls should be designed to resist corrosion consistent with a salt water environment.

Fill material should be placed in loose lifts not exceeding twelve inches in thickness and should be compacted to 95 percent of Standard Proctor maximum dry density as determined by ASTM D698 without a moisture requirement. If former hydraulic fill material is encountered it should be placed outside the dike cross section. If it must be used as dike fill it needs to be dried and treated with lime prior to use. For estimating purposes assume 8% lime by dry weight.

Based on the construction cross sections provided to us, we understand that installation of pipeline protection wall and cutoff wall will require partial excavation of the channel slope and Spilmans Island dike for crane access. The cutoff wall will be installed using a crane operated from the bench created north of the cutoff wall by excavating the dike or from the bench created south of the cutoff wall. We evaluated the proposed cross sections and the results of our analyses indicates that installing the cutoff wall from the north side will not have adequate safety factor. The allowable construction surcharge with adequate safety factor for installing the cutoff wall from the north side and adjacent a 2H:1V flotation channel slope near the pipeline protection wall are presented in Table 7-6.

Previous widening projects at Barbours Cut installed erosion protection comprised of 3-foot thick 55 to 1500 pound stone rip rap from -3.3 feet MLT to El. +6.7 feet MLLW. The riprap was installed on a geotextile fabric underlayment. We recommend similar erosion protection for the new shoreline for this project.

7.3 Axial Capacity Curves for Pipe Piles at Spilmans Island

7.3.1 General

Based on the information provided to us, we understand that the proposed Pipeline Protection Wall and the Cutoff Wall on the east corner of the Spilmans Island consists of king piles and battered piles of dimensions shown in the table below. Boring ECP-316 was drilled in the vicinity of the Pipeline Protection Wall and boring ECP-312 was performed near the proposed Cutoff Wall at Spilmans Island east corner. These borings were primarily utilized to develop capacity curves for the pipe piles.

Table 7-8 – Pipe Pile Details at Spilmans Island

Wall	Pile Type	Batter	Dimensions	Pile Top Elevation, Feet	Pile Tip Elevation, Feet
Pipeline Protection Wall	King Pile	NA	60” x 1.0” WT	-6	-101
	Battered Pile	6H:12V	30” x 0.75” WT	-2	-132

Wall	Pile Type	Batter	Dimensions	Pile Top Elevation, Feet	Pile Tip Elevation, Feet
Cutoff Wall at East End	King Pile	NA	48" x 1.0" WT and 30" x 0.75" WT	+32	-72
	Sheet Pile	NA	NZ 38	+32	-72
	Battered Pile	5H:12V	24" x 0.75" WT	+25	-55
Cutoff Wall at West End	King Pile	NA	48" x 1.0" WT	+25	-72
	Sheet Pile	NA	NZ 38	+25	-72

7.3.2 Axial Capacity

Allowable compressive and tensile capacity curves were developed for steel piles based on USACE method with the use of APILE computer program. For the Cutoff Wall pipe piles, skin friction contributed at the top 10 feet from the existing grade was ignored to account for construction disturbances. We ignored the skin friction resistance to the future channel dredge elevation at the Pipeline Protection Wall. The driven pile capacity curves for allowable axial capacity under compression and tension are presented in Appendix D. In order to determine the allowable compressive capacity a factor of safety must be applied to the total ultimate capacity. Allowable axial tensile capacity can be calculated by applying a factor of safety to the ultimate skin friction capacity. Factors of safety should be determined based on USACE EM1110-2-2906 Design of Pile Foundations. In order to rely on factors of safety based on capacity verified by pile driving analyzer a minimum of 3 piles or 5% of the total piles driven for each wall should be tested, whichever is greater.

The soils at the cut off wall location are soft to firm clay and sand to about El. 0 to -14 feet MLLW. If significant additional load is applied to the soils then settlement may occur which would lead to downdrag forces. The current plan is to degrade the dike to El. +32 feet MLLW for 100 feet from the cutoff wall into the site interior which will remove a significant load from the soils at the cutoff wall location. Since the soil loading near the wall is being reduced no settlement that would lead to downdrag is expected. Therefore, we do not recommend including downdrag loads in the cutoff wall design.

The soils at the pipeline protection wall are primarily stiff to very stiff clay, these soils are not susceptible to the kind of settlement that would lead to significant downdrag loads. The current plan is to degrade the ground near the wall to about El. -5 feet MLLW which will remove a significant load from the soils at the pipeline protection wall location. Since the soil loading is being reduced no settlement that would lead to downdrag is expected. Therefore, we do not recommend including downdrag loads in the cutoff wall design.

7.3.3 Pile Driving Vibrations

An existing pipeline is located close the pipeline protection wall, and the cutoff wall will be installed in an existing dike that contains both mechanically placed and hydraulically placed fill. The vibration due to pile driving may affect both the pipeline and the fill.

The sensitivity of the pipeline to vibrations should be assessed prior to beginning pile driving and vibration monitoring equipment should be used to assess whether damaging vibrations are occurring or may occur. The Contractor should be prepared to alter the installation methods to reduce vibrations to tolerable levels.

We recommend that the dike slope be monitored during pile driving to assess whether movement is occurring that would indicate weakening of vibration sensitive soils. If significant movement is detected then pile driving should immediately cease and an investigation of measures required to safely continue pile installation should be performed by the Contractor.

7.3.4 Pile Construction Recommendations

Methods and effects of pile installation are important considerations in the choice and design of pile foundation systems. Piles normally experience their largest stresses during installation. Pile and soil properties, embedment length and driving equipment are a few of the variables that must be considered.

1. We note that difficult driving may be encountered below about El -65 feet due to N values of 50/1" and 50/1.5" at the East Corner Cutoff Wall.

2. Adequate cushioning material should be provided between the pile driver and the pile head. A six to twelve-inch thick cushion of softwood is usually adequate for piles that are over 50 feet long. Cushioning material condition should be carefully observed and the cushion must be changed if excessive compression occurs or at least every three piles.

3. Based on our experience, piles can usually be safely driven to about 100 blows per foot. Consistent blow counts above 100 blows per foot are not advisable.

4. The hammer, cushion and pile should be designed such that installation to design specifications can be realized with no damage to the pile.

5. The pile driving cap should fit loosely around the top of the pile so that torsional stresses do not develop in the pile. The cap should, however, be able to control the alignment of the pile.

6. Prior to driving, the pile should be properly aligned and held with fixed leads. The pile should not be realigned once driving has begun.

7. Clays and some silty soils tend to undergo a reduction in strength during pile driving and regain strength after pile installation. This phenomenon is usually referred to as freeze or set-up. The number and duration of delays in the driving program should be minimized so as to control the effect of set-up and pile heaving. Pilot holes will also minimize this effect.

7.4 Morgans Point Sites 1 and 2

7.4.1 General

Morgans Point is the area located along the west bank of the Houston Ship Channel immediately south of the Barbours Cut Ship Channel as shown in the figure below which shows the site conditions in 2002. There is a large dock with pilings referred to as the LASH Dock that makes up the easternmost facility in the Barbour Cut Container Terminal. East of the terminal there is a narrow strip of land that contained private docks and facilities. Since this photo was taken this area has been incorporated into the terminal and the previous private facilities have been removed. We understand that removal of the LASH Dock is also planned.

The proposed dredging impacts to Site 1 at the northern end and Site 2 at the southern end of Morgans Point are addressed in this study.

Figure 7-6 – Morgans Point, circa 2002

Site 1: For a channel dredge cut elevation of El. -61.00 feet, a bulkhead is required at the eastern end at Morgan's Point due to the BCC flare dredging encroaching on the existing terminal. The proposed channel dredge cut elevation without bulkhead is El. -51 feet MLLW. For the bulkhead design a future channel cut elevation of -61 feet was considered to allow for future deepening of the channel beyond that planned for this project.

Two alternatives with different turning points, referred to as Options 2 and 3 were evaluated. We have reviewed the available borings in the vicinity and borings ECP-317, ECP-318 and ECP-319 were performed as part of the present study. We analyzed the proposed cross sections to determine the minimum embedment length of the bulkhead required to satisfy global stability. . The results of both options were discussed with HDR. It was determined that channel slope in front of the bulkhead for Option 2 does not achieve the minimum required safety factor. The results of Option 2 slope stability analyses are not included in this report.
Option 3 involves channel toe at about 140 to 200 feet from the bulkhead. The soil profile primarily from boring ECP-318 was utilized to evaluate the proposed cross section with bulkhead at an offset of about 140 feet from the channel toe. Soil profiles from borings ECP-317 and ECP-319 were utilized to analyze the proposed cross section with bulkhead at about 220 feet from the channel toe. We understand that the structural design of the bulkhead is being performed by HDR which will include assessing required bulkhead length to satisfy structural requirements. Soil parameters for the bulkhead design are presented in Appendix C.

Site 2: The proposed HSC channel slopes will encroach into the Port of Houston property located south of Ballester Road near the Barbours Cut Channel. The proposed channel dredge cut elevation analyzed is El. -61 feet MLLW at this location. Due to the property encroachment a future channel cut elevation of -61 feet was considered to allow for future deepening of the channel beyond that planned for this project. We analyzed the proposed cross sections based on borings ECP-320 and ECP-321 performed at this location. Results of the stability analysis are presented in the following sections.

7.4.2 Generalized Soil Conditions

Site 1: Borings ECP-317, ECP-318 and ECP-319 were drilled adjacent to the proposed bulkhead to a depth of 100 to 150 feet below the existing grade. Water was observed at the surface at boring ECP-317 and it varied between 3 and 8 feet at the remaining locations. Boring B-1 was performed by others at the northeast corner of the container terminal for the proposed communications tower. In addition, marine borings ECP-307A and ECP-308A were performed in the vicinity. In general, the land borings revealed the presence of soft to stiff clays at the surface underlain by 12-foot thick very loose to medium dense sands. The sand layer was followed by soft to hard clays with occasional sand or silt layers to the boring termination depth but was not encountered in boring ECP-319.

Site 2: Land borings ECP-320 and ECP-321 were drilled to a depth of 100 feet below the existing grade. Also, marine boring ECP-304B was performed to a depth of 52 feet below the mudline. Fill material was observed at the surface followed by a thick layer of very loose to medium dense silt. The silt layer was underlain by alternating layers of very soft to stiff clays and very loose to dens sands.

7.4.3 Design Cross Section

Slope stability analyses were performed for the End of Construction, Long-Term and Rapid Drawdown conditions. At Site 1, a surcharge of 1,000 psf was assumed to reflect the future tower that will be constructed in the container terminal behind the bulkhead. We have analyzed the slope stability using the cross sections provided at Site 1 and Station 19+00 at Site 2. The cross sections, soil parameters used in the slope stability analyses, and the slope stability outputs are presented in Appendix C. We assumed the water level at El. +1.0 feet MLLW in the bay for the short and long term cases. In the Rapid Drawdown analysis, a drawdown of the water level was assumed from El.

+12.49 to El. -3.69 feet MLLW to reflect the impact of hurricane surge on the slopes. The results are summarized in the table below.

Table 7-9 – BCC Slope Stability Results – Proposed Slopes at Sites 1 and 2

Location	Boring	Factor of Safety		
		Short Term	**Long Term**	**Rapid Drawdown**
Site 1 (Option 3)	ECP-317	1.80	1.63	1.36
	ECP-318	2.23	1.59	1.32
	ECP-319	2.60	2.08	1.75
Site 2 (No Bulkhead)	19+00 (ECP-321)	1.57	1.59	**0.99***

* Does not meet the minimum required factor of safety.

At Site 1, the stability analyses meet or exceed the required minimum factors of safety discussed in Section 4.1. At Site 2, a 20-foot wide shelf at about El. -8 feet is needed to achieve the required safety factor for the Rapid Drawdown case. The results of the recommended section at Site 2 are summarized in the table below.

Table 7-10 – BCC Slope Stability Results – Recommended Section at Site 2

Location	Station (Boring)	Factor of Safety		
		Short Term	**Long Term**	**Rapid Drawdown**
Site 2 (No Bulkhead)	19+00 (ECP-321)	1.63	1.67	1.30

7.4.4 Bulkhead Design

We understand that the bulkhead will be constructed as a "Combi-walls" which comprise open-ended steel pipe piles installed at intervals with steel sheet piles in between the pipe piles. The pipe piles provide the main structural capacity of the wall while the sheet piles provide continuity and transmit the soil loads to the wall. Therefore, they do not need to be installed to the full depth of the pipe piles. We recommend that the depth of the sheet piles be based on the lateral load analysis of the wall, and the sheet piles can terminate below the point where lateral loading stops. The sheet piles can stop at a point 25% of the height of the wall or 5 feet below the bottom of the active/at rest soil pressure load on the wall. The bottom of the active/at rest soil pressure on the wall should be slightly below the elevation of the ground surface in front of the wall. Soil parameters for the bulkhead design are presented in Appendix C. In cohesionless soil layers wall friction on steel can be taken as 5° less than the angle of internal friction shown in Appendix C. In cohesive soil layers wall adhesion can be taken as one-half the undrained shear strength shown in Appendix C.

The soil conditions in Boring ECP-318 include erodible fill and sand. This material will be exposed in front of the bulkhead during future dredging to El. -61 as shown in the sections in Appendix C. Unless erosion protection is provided to El. -25 feet MLLW lateral support to the bulkhead may be affected. We recommend that lateral support is neglected above El. -23 in the bulkhead design.

In addition to supporting lateral loads from the retained height of soil, the bulkhead is also providing lateral support to the slope required to achieve the design factor of safety for global stability. In the global stability analyses discussed in Section 7.3.3 the bulkheads were assumed to be very strong with the result that the critical failure surfaces do not pass through the bulkheads. To assess the slope load on the bulkhead we ran additional global stability analyses assuming that the bulkhead had a shear strength equivalent to an angle of internal friction of 45°. Where a critical slip surface passes through the bulkhead the forces acting on that slice indicate the slope loading placed on the bulkhead. These loads are summarized in Table 7-11, note that these loads are unfactored.

Table 7-11 – Slope Loading on Bulkhead, Morgans Point Site 1

Section	Load Case	Downdrag, lb/foot of bulkhead	Shear Force, lb/foot of bulkhead	Shear Force Angle	Shear Force El., Feet MLLW
ECP-317	Long Term	12,704	10,393	8.82	-28.97
	Short Term	7,034	11,551	-12.50	-32.60
	RDD	4,412	6,183	1.89	-18.60
ECP-318	Long Term	3,951	4,092	27.87	-22.98
	Short Term	3,864	3,561	17.05	-22.20
	RDD	6,267	5,596	15.13	-22.34
ECP-319	Long Term	2,534	3,585	36.56	-22.51
	Short Term	1,563	3,767	19.60	-19.05
	RDD	1,715	2,113	21.26	-8.78

Active earth pressures may be used if the wall deflection is about 1% of the wall height, if the deflections are restricted at-rest pressures should be considered. As long as the wall is not protecting movement sensitive structures deflections of up to 2% of the wall height can be acceptable. Wall movements under extreme loading conditions should be limited to no more than 3 inches.

Soils in the Houston area are not generally corrosive. We have not made any specific tests of soil corrosivity for this study. The water in the Houston Ship Channel is brackish, and corrosion consistent with a salt water environment should be expected.

7.4.5 Bulkhead Construction

Morgans Point Site 1 has significant existing area around the bulkhead location therefore global stability should not be an issue for heavy equipment such as cranes. The Contractor should use adequate matting to support cranes used to construct the bulkhead. A setback of at least 30 feet from the existing shoreline should be maintained, if heavy construction equipment must approach closer to the shoreline a detailed global stability analysis should be performed to confirm adequate global stability.

7.4.6 Shore Protection

We reviewed Borings ECP-317, ECP-318, and ECP-319 to assess the potential soil conditions at the new shoreline. The boring locations are shown on Plate 1H. The elevations of interest are between about El. -4 and +6 feet MLLW. In borings except ECP-317 and ECP-318 erodible fill, sand, and very soft clay were encountered to between El. -18 and -23 feet MLLW. Soil conditions are variable over short distances and sand layers at the waterline may be encountered at other locations along the new shoreline.

Previous widening projects at Barbours Cut Ship Channel installed erosion protection comprised of 3-foot thick 55 to 1500 pound stone rip rap from El. -2 to +8 feet MLT which is equivalent to about El. -3.3 to +6.7 feet MLLW along the north shore. The riprap was installed on a geotextile fabric underlayment. We recommend similar erosion protection for the new shoreline from El. -5 to +7 feet MLLW. The gradation of the stone is shown in the table below. Note that we are assuming the erosion environment at Morgans Point will be similar to Barbours Cut Channel, and if significant differences are expected the gradation may need to be revised.

Table 7-12 – Shore Protection Stone Gradation – Morgans Point

Design Stone Dimensions*	Percent of Stone by Weight Less than Design Stone Size (%)	Weight of Design Stone Size (pounds)
8.5	0	<55
13 – 17.5	15	200 – 520
17 – 20	50	500 – 760
22 – 25	100	1,000 – 1,500

* Dimensions will depend on the specific gravity of the stone and the weight of the stone governs the gradation.

7.4.7 Recommendations

Site 1: Based on the soil parameters and water level discussed earlier, we evaluated the proposed dredge template to El. -61.0 feet with varying bulkhead alignments at Site 1. The global stability analyses indicate that the proposed bulkhead should have a minimum tip elevation of -60 feet MLLW for global stability.

The bulkhead should be designed based on the soil properties given in Appendix C and should include the slope loads shown in Table 7-11. We recommend that lateral support is neglected above El. -23 in the bulkhead design due to the erodible nature of the soils. The walls should be designed to resist corrosion consistent with a salt water environment.

Previous widening projects at Barbours Cut installed erosion protection comprised of 3-foot thick 55 to 1500 pound stone rip rap from -3.3 feet MLT to El. +6.7 feet MLLW. The riprap was installed on a geotextile fabric underlayment. We recommend similar erosion protection for the new shoreline for this project.

Site 2: Based on the soil parameters and water level discussed earlier, we evaluated the proposed dredge template comprising 3H:1V slopes to -61.0 feet. A 20-foot wide shelf at about El. -8 feet MLLW and a 4H:1V slope from the shelf to the top of bank are required to achieve the required safety factor for the Rapid Drawdown case at Site 2 for a future dredging to El. -61.

7.5 Axial Capacity Curves for Pipe Piles at Morgans Point

7.5.1 General

Based on the information provided to us, we understand that the proposed bulkhead consists of sheet and battered piles of dimensions shown in the table below. Boring ECP-318 drilled in the vicinity was utilized to develop capacity curves for the piles.

Table 7-13 – Bulkhead Details at Morgans Point

Wall	Pile Type	Batter	Dimensions	Pile Top Elevation, Feet	Pile Tip Elevation, Feet
Bulkhead	Sheet Pile	NA	NZ 38	+3.0	-60.0
	Battered Pile	4H:12V	HP 14x102	+3.0	-75.0

7.5.2 Axial Capacity

Allowable compressive and tensile capacity curves were developed for steel piles based on USACE method with the use of APILE computer program. We ignored the skin friction resistance to the bottom of the sand layer that is susceptible to erosion. The driven pile capacity curves for allowable axial capacity under compression and tension are presented in Appendix K. In order to determine the allowable compressive capacity a factor of safety must be applied to the total ultimate capacity. Allowable axial tensile capacity can be calculated by applying a factor of safety to the ultimate skin friction capacity. Factors of safety should be determined based on USACE EM1110-2-2906 Design of Pile Foundations. In order to rely on factors of safety based on capacity verified by pile driving analyzer a minimum of 3 piles or 5% of the total piles driven for each wall should be tested, whichever is greater.

7.5.3 Pile Construction Recommendations

Methods and effects of pile installation are important considerations in the choice and design of pile foundation systems. Piles normally experience their largest stresses during installation. Pile and soil properties, embedment length and driving equipment are a few of the variables that must be considered.

1. Adequate cushioning material should be provided between the pile driver and the pile head. A six to twelve-inch thick cushion of softwood is usually adequate for piles that are over 50 feet long. Cushioning material condition should be carefully observed and the cushion must be changed if excessive compression occurs or at least every three piles.

2. Based on our experience, piles can usually be safely driven to about 100 blows per foot. Consistent blow counts above 100 blows per foot are not advisable.

3. The hammer, cushion and pile should be designed such that installation to design specifications can be realized with no damage to the pile.

4. The pile driving cap should fit loosely around the top of the pile so that torsional stresses do not develop in the pile. The cap should, however, be able to control the alignment of the pile.

5. Prior to driving, the pile should be properly aligned and held with fixed leads. The pile should not be realigned once driving has begun.

6. Clays and some silty soils tend to undergo a reduction in strength during pile driving and regain strength after pile installation. This phenomenon is usually referred to as freeze or set-up. The number and duration of delays in the driving program should be minimized so as to control the effect of set-up and pile heaving. Pilot holes will also minimize this effect.

8 ATKINSON ISLAND BENEFICIAL USE SITE – CELL M12

8.1 General

Atkinson Island has been created primarily through the placement of dredge material. Dredge mounds were visible as early as the USGS 1921 topographic maps as shown in Figure 7-1 along with an emergent portion of Atkinson Island that has since subsided into Galveston Bay. The configuration of the placement area in about 2002 is shown in Figure 8-1.

Figure 8-1 – Atkinson Island Beneficial Use Site, circa 2002

Atkinsons Island was originally comprised of Placement Areas 14, 15, and 16. As part of preparation for the 45-Foot Project the Gorini Demonstration Marsh was constructed in 1993 between Placement Areas 15 and 16 (the Gorini Marsh is shown in Figure 4-1). As part of the 45-Foot Project four beneficial use marsh cells were constructed. Subsequently additional beneficial use marsh cells have been constructed south of the original cells adjacent to Placement Areas 14 and 15. For this project beneficial use marsh cell M12 is planned east of Placement Area 16 and north of the original beneficial use marsh cells.

8.2 Generalized Soil Conditions

Borings ECP-1042 thru ECP-1053 were performed along the proposed dike alignment and borings ECP-1054, ECP-1055 and ECP-1056 were drilled in the site interior. The depth of water at the boring locations varied between 2.5 and 5 feet. In general, very loose to dense sands were predominantly encountered with occasional very soft to soft clays from the mudline to a depth of 12 to 33 feet. Very soft clays were observed from the mudline to a depth of 6 feet in borings ECP-1047, ECP-1050 and ECP-1052.

8.3 Displacement

We estimated the foundation displacement based on the cross sections shown in Appendix E and the soil properties at each boring location using the procedures discussed in Section 4.2. The estimated displacement elevations are shown in the following table.

Table 8-1 – Displacement – Cell M12

Boring	Northing, Feet	Easting, Feet	Mudline Elevation, Feet (MLLW)	Displacement Elevation, Feet (MLLW)
Cell M12				
ECP-1042	13,816,201.67	3,253,229.88	-3.56	-6
ECP-1043	13,817,184.55	3,252,928.98	-4.23	-7
ECP-1044	13,817,847.56	3,252,321.44	-3.80	-7
ECP-1045	13,818,386.48	3,251,597.49	-3.96	-6
ECP-1046	13,818,750.81	3,250,558.72	-3.72	-8
ECP-1047	13,819,188.38	3,249,638.16	-3.59	-10
ECP-1048	13,819,279.74	3,248,700.98	-2.80	-7
ECP-1049	13,819,013.68	3,247,738.00	-2.41	-5
ECP-1050	13,818,693.28	3,247,942.84	-2.04	-4
ECP-1051	13,817,873.87	3,248,342.81	-1.95	-4
ECP-1052	13,816,834.62	3,248,381.83	-1.24	-5
ECP-1053	13,816,179.01	3,248,572.83	-1.74	-6
ECP-1054*	13,818,380.56	3,248,792.36	-3.64	-6
ECP-1055*	13,817,597.33	3,250,134.71	-3.09	-5
ECP-1056*	13,817,132.36	3,251,841.01	-3.58	-6

* Interior Boring

8.4 Slope Stability

Analyses were performed for the short term, rapid drawdown and long term cases using the procedures described in Section 4.1. Analyses were performed for the proposed bay side dike with 20-foot wide crest at El. +8.0 feet MLLW with 3H:1V interior slope and 2.5H:1V exterior slope with rock shore protection. The shore side dike with 20-foot wide crest at an elevation of +8.0 feet MLLW with 3H:1V side slopes was also analyzed. Analyses were performed using the soil profiles from borings ECP-1050, ECP-1053, ECP-1048, ECP-1047, ECP-1046 and ECP-1044 and the estimated displacement discussed above. These locations were chosen based on relative strength of the soils. We assumed the exterior water level at El. 0.0 feet for the short term and long term cases. In the Rapid Drawdown analysis, a drawdown of the water level was assumed from El. +10.0 to El. 0.0 feet MLLW to reflect the impact of hurricane surge on the slopes. We note that the water levels were assumed for the short term, rapid drawdown and long term cases.

The slope stability analyses outputs are presented in Appendix E and the results are summarized in the following table.

Table 8-2 – Atkinson Cell M12 Slope Stability Results

Station (Boring)	Factor of Safety				
	Short Term - Exterior		Short Term – Interior*	Long Term	Rapid Drawdown
	Circular	Block	Circular	Circular	Circular
5+00 Baseline B (ECP-1050)	2.69	2.82	2.68	**1.26****	1.09
30+00 Baseline B (ECP-1053)	2.12	1.37	2.15	**1.24****	1.10
20+00, Baseline A (ECP-1048)	1.87	1.33	2.00	**1.25****	1.16
30+00, Baseline A (ECP-1047)	1.36	**1.27****	1.33	**1.24****	1.16
40+00, Baseline A (ECP-1046)	1.34	1.35	1.36	**1.24****	1.16
60+00, Baseline A (ECP-1044)	1.39	1.30	1.48	**1.26****	1.16

*Block analysis for the short term interior case was not performed as the exterior block analysis results are at equal or steeper slope.
**Does not meet the minimum required safety factor.

The stability analyses indicate that the proposed cross sections does not meet the minimum required factor of safety for long term condition at all locations. In addition, the safety factor is lower than minimum required for short term condition at ECP-1047.

In order to achieve a stable cross section, the exterior slope must be 3H:1V for the bay side dike and 3.5H:1V for the shore side dike. The results of the stability analyses with recommended cross sections are summarized in the following table.

Table 8-3 – Atkinson Cell M12 Slope Stability Results – Recommended Section

Station (Boring)	Factor of Safety				
	Short Term - Exterior		Short Term – Interior*	Long Term	Rapid Drawdown
	Circular	Block	Circular	Circular	Circular
5+00 Baseline B (ECP-1050)	2.80	2.97	2.69	1.36	1.26
30+00 Baseline B (ECP-1053)	2.32	1.61	2.12	1.30	1.20
20+00, Baseline A (ECP-1048)	2.01	1.38	1.99	1.34	1.35
30+00, Baseline A (ECP-1047)	1.38	1.30	1.34	1.32	1.33

Station (Boring)	Factor of Safety				
	Short Term - Exterior		Short Term – Interior*	Long Term	Rapid Drawdown
	Circular	Block	Circular	Circular	Circular
40+00, Baseline A (ECP-1046)	1.39	1.39	1.37	1.33	1.31
60+00, Baseline A (ECP-1044)	1.41	1.35	1.48	1.31	1.30

The stability analyses meet or exceed the required minimum factor of safety discussed in Section 4.1.

8.5 Site Capacity

The site capacity analysis was performed based on assessing the character of the material to be dredged from the channel. Based on the proposed cut elevations and soil properties observed in the boring logs we determined the following distribution of material, note that this is an estimate for site capacity estimating purposes only. For specific soil conditions refer to the borings performed for this study.

Table 8-4 – Soil Type in Cut – M12 PA

Soil Type	Distribution in Cut
Sand	1.5%
Firm to Stiff CL Clay	11.5%
Firm to Stiff CH Clay	87.0%
Very Soft to Soft Clay	--

In the above table the designation of CL and CH clay correspond to those used in the discussion of Retention in Section 4.4. See Section 7.1 for a discussion of the borings used to evaluate dredge material. The retention factors for Firm to Stiff CL and CH Clay are 53% and 90% based on an assumed pump distance of 9,000 feet. Based on the retention factors and distribution of soils in the cut we estimate that 86% of the cut material will be coarse-grained fill. This includes all of the sand and the retained portions of the firm to stiff CL and CH clay. The remaining 14% will be slurried fine-grained fill.

The M12 Placement Area has an area of 250 acres inside the dike and an average depth of 11 feet. The SETTLE analysis results are presented in Appendix M. Table 8-5 presents the capacity of the site for various scenarios, average elevations shown are after settlement discussed in Section 8.6.

Table 8-5 – M12 Site Capacity/Elevation

Cut Capacity, Cu. Yds.	Coarse-Grained (Clay Ball) Fill				Fine-Grained Fill	
	Upland		Marsh			
	Area, Acres	Average Elev., Feet	Area, Acres	Average Elev., Feet	Area, Acres	Average Elev., Feet
2,520,000	167	+5.1	--	--	83	+3.6 (upland)
1,630,000	52	+5.1	111	+1.1	87	+1.1 (marsh)

The 2,520,000 cubic yard case represents the estimated maximum capacity of Cell M12, and will not result in any site interior within the marsh target range. It assumes that the coarse fill is placed at an initial average elevation of +6.0. The duration of dredging is about 168 days based on a 30-inch dredge operating 17 hours per day, 7 days per week.

The 1,630,000 cubic yard case represents the estimated capacity of Cell M12 with the most area at the marsh elevation. It assumes that the coarse fill is placed at an initial average elevation of +6.0 in the upland section and +1.6 in the marsh section. Managing the elevation of the coarse fill in the marsh area to an elevation of +1.6 during dredging will be a challenge since the cell will be flooded during that time, and the coarse grained fill tends to stack at the discharge point rather than spread. The duration of dredging is about 109 days based on a 30-inch dredge operating 17 hours per day, 7 days per week.

8.6 Settlement

8.6.1 Dike

We performed consolidation settlement calculations based on the procedures discussed in Section 4.3. The analysis indicates that a consolidation soil settlement between 0.1 and 6.7 inches could occur due to the fill weight below the base of foundation displacement from the construction. The settlement occurs over a period of time. The results of our analyses are presented in the figure below and in Appendix E.

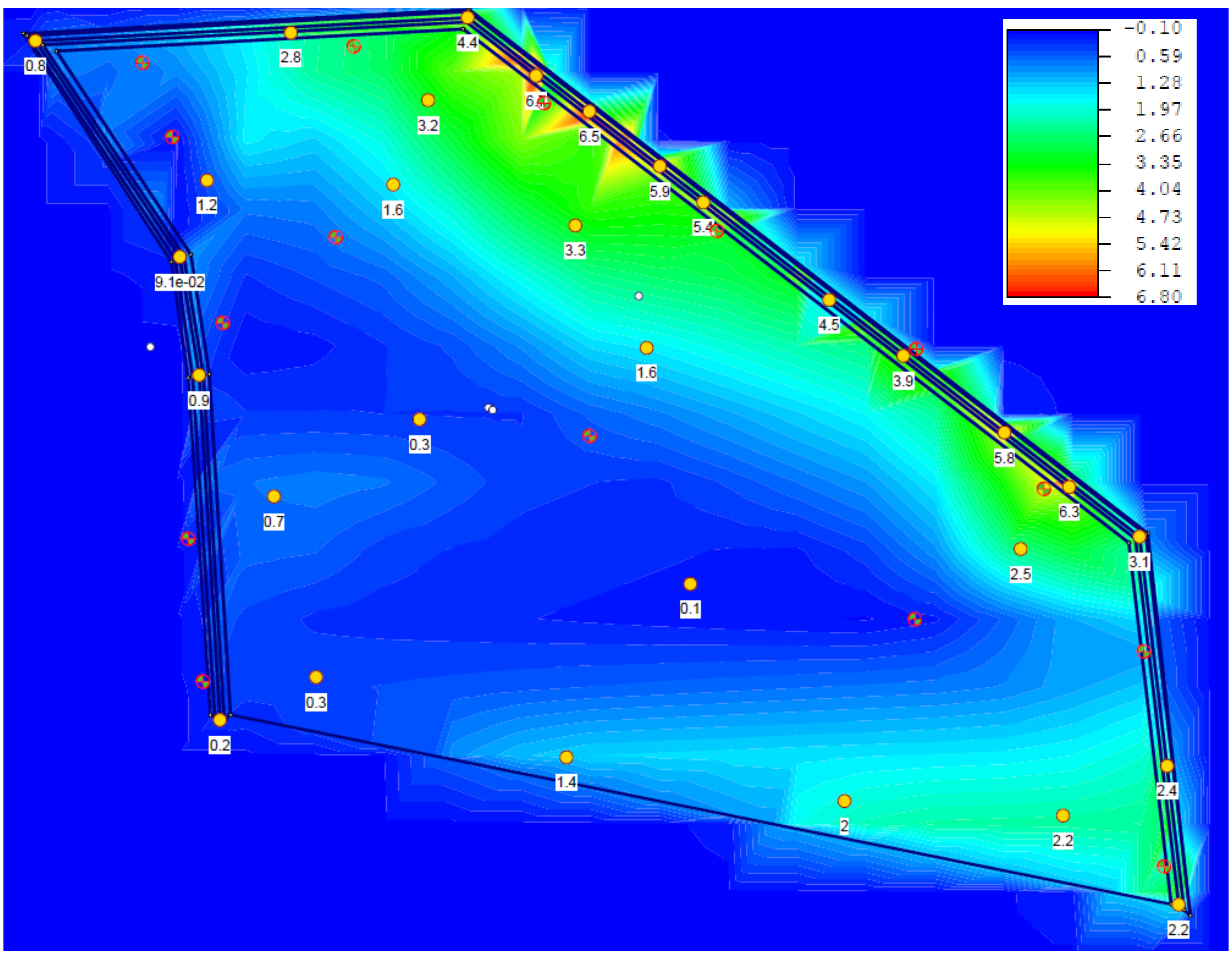


Figure 8-2 – 50-Year Consolidation Settlement, Cell M12

The following table shows the estimated maximum settlement vs. time of the dike.

Table 8-6 – Settlement vs. Time at M12 Dike

Time, Years	Estimated Settlement, Inches		
	Dike Fill	Foundation Consolidation	Total
1	5.4	4.1	9.5
5	10.8	5.7	16.5
20	10.8	6.3	17.1
50	10.8	6.7	17.5

8.6.2 Coarse Fill

We performed consolidation settlement calculations based on the procedures discussed in Section 4.3. The analysis indicates that a consolidation soil settlement between 2.2 and 3.1 inches could occur due to the fill weight in the site interior. The settlement occurs over a period of time. The results of our analyses are presented in Figure 8-2 and Appendix E.

The following table shows the estimated maximum settlement vs. time of the coarse grained fill. The two settlements of upland and marsh correspond to those shown in Table 8-5 above.

Table 8-7 – Settlement vs. Time of M12 Coarse Fill

Time, Years	Estimated Center Settlement, Inches					
	Coarse Fill		Foundation Consolidation		Total	
	Upland	Marsh	Upland	Marsh	Upland	Marsh
1	2.7	1.3	3.4	2.3	6.1	3.6
5	5.4	2.7	4.4	3.0	9.8	5.7
20	5.4	2.7	4.8	3.3	10.2	6.0
50	5.4	2.7	5.0	3.3	10.4	6.0

8.6.3 Fine Fill

After filling the fine-grained fill will settle over a period of about 2 to 3 years as shown in the figure below. The analysis of fine-grained fill settlement was performed using PSDDF as discussed in Section 4.3.2. PSDDF uses changes in void ratio to predict settlement. The most important aspects of making PSDDF settlement estimates is determining the height of fill and the duration of the placement. Section 9.5 discusses the duration of filling.

The height of fill is determined from the volume of solids which is based on the void ratio of the cut soils. This averages about 0.59 for the firm to stiff clay. Based on the cut soil distribution discussed in Section 8.5 the volume of solids for the fine-grained fill can be determined. Based on an assumed initial void ratio of 6.0 during discharge and the area of the site occupied by the fine-grained fill the height of fill input in PSDDF is 7.6 and 11.6 feet for the 1,630,000 and 2,560,000 cut cubic yard cases, respectively. The analysis results are presented in Appendix M.

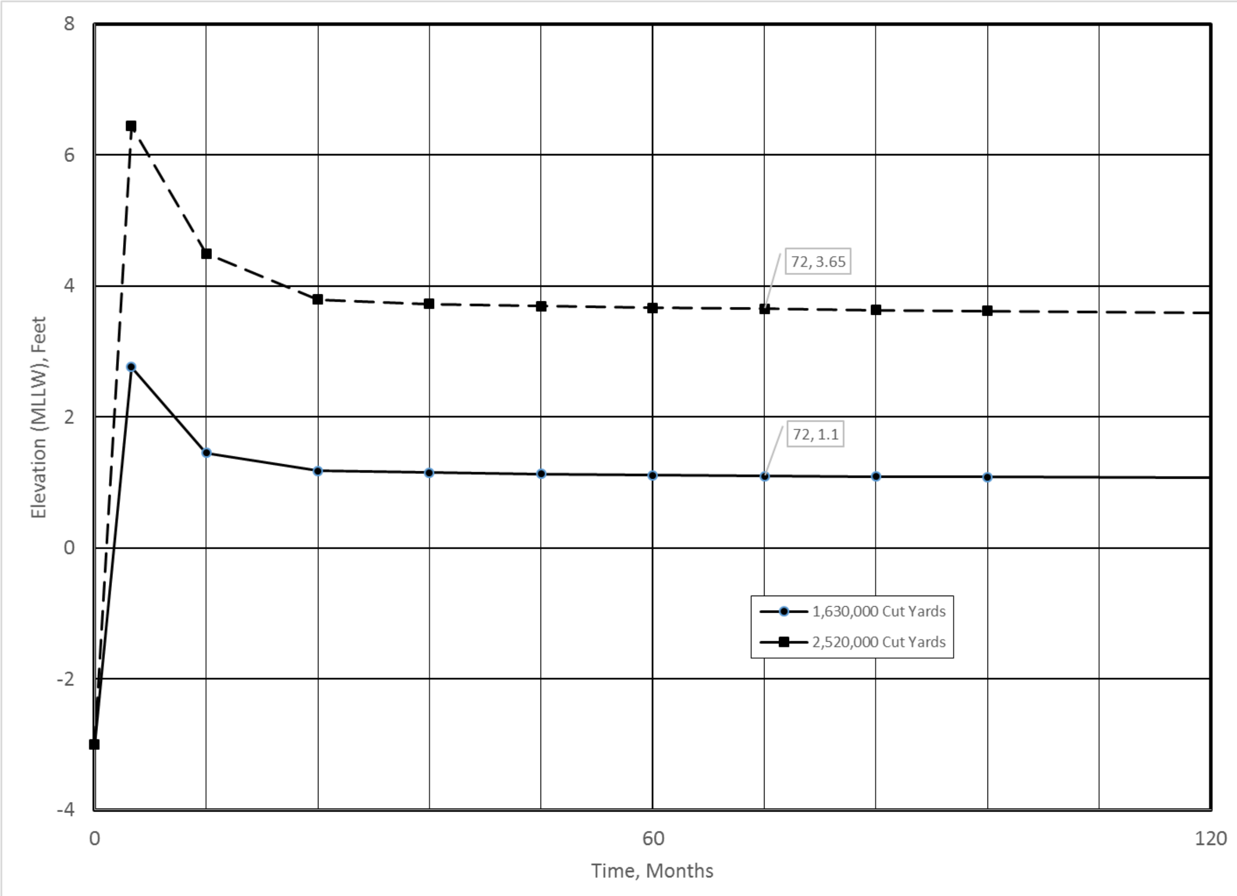


Figure 8-3 – Fine-Grained Fill Settlement – M12 PA

8.7 Axial Capacity Curves for Access Bridge at Cell M12

8.7.1 General

Based on the information provided to us, we understand that an access bridge will be constructed at the outfall near Station 73+00. Boring ECP-1043 was drilled in the vicinity of the access bridge to a depth of 30 feet below the mudline. We understand that the bridge will be supported using HP14x89 or W8x40 piles (Structural steel piles).

8.7.2 Axial Capacity

Allowable compressive and tensile capacity curves were developed for steel piles based on USACE method with the use of APILE computer program. We ignored the skin friction resistance contributed at the top 10 feet from the mudline. The driven pile capacity curves for allowable axial capacity under compression and tension are presented in Appendix L. In order to determine the allowable compressive capacity a factor of safety must be applied to the total ultimate capacity. Allowable axial tensile capacity can be calculated by applying a factor of safety to the ultimate skin friction capacity. Factors of safety should be determined based on USACE EM1110-2-2906 Design of Pile Foundations. In order to rely on factors of safety based on capacity verified by pile driving analyzer a minimum of 2 piles or 5% of the total piles driven for each structure should be tested, whichever is greater.

8.7.3 Pile Construction Recommendations

Methods and effects of pile installation are important considerations in the choice and design of pile foundation systems. Piles normally experience their largest stresses during installation. Pile and soil properties, embedment length and driving equipment are a few of the variables that must be considered.

1. Adequate cushioning material should be provided between the pile driver and the pile head. A six to twelve-inch thick cushion of softwood is usually adequate for piles that are over 50 feet long. Cushioning material condition should be carefully observed and the cushion must be changed if excessive compression occurs or at least every three piles.

2. Based on our experience, piles can usually be safely driven to about 100 blows per foot. Consistent blow counts above 100 blows per foot are not advisable.

3. The hammer, cushion and pile should be designed such that installation to design specifications can be realized with no damage to the pile.

4. The pile driving cap should fit loosely around the top of the pile so that torsional stresses do not develop in the pile. The cap should, however, be able to control the alignment of the pile.

5. Prior to driving, the pile should be properly aligned and held with fixed leads. The pile should not be realigned once driving has begun.

6. Clays and some silty soils tend to undergo a reduction in strength during pile driving and regain strength after pile installation. This phenomenon is usually referred to as freeze or set-up. The number and duration of delays in the driving program should be minimized so as to control the effect of set-up and pile heaving. Pilot holes will also minimize this effect.

8.8 Weir Structure Design Recommendations

8.8.1 General

Based on the available information, we understand that a weir structure will be constructed at each outfall and it will be founded on the dike fill. Design recommendations for the weir structure are provided in the following sections.

8.8.2 Allowable Bearing Capacity

Using a factor of safety of 3, a net allowable bearing capacity of 600 pounds per square foot can be used for the weir structure installed within the dike fill. A subgrade reaction modulus (k) of 25 pounds per cubic inch can be used for the structural design.

8.8.3 Lateral Pressure

The soil pressure exerted on the walls is mainly a function of the type of fill and its method of placement. For the dike fill material, an equivalent fluid pressure of 43 psf/ft and 81 psf/ft can be used for above and below water level, respectively.

8.9 Recommendations

The stability analyses indicate that the proposed cross sections do not meet the minimum required factor of safety. In order to achieve a stable cross section, the exterior slope must be 3H:1V for the bay side dike (Baseline A) and 3.5H:1V for the shore side dike (Baseline B). Dike settlement will be about 9.5 inches after 1 year and 16.5 inches after 5 years. The dike crest elevation will be about +6.6 feet five years after construction.

The estimated site capacity is 2,518,700 cut cubic yards. The settlement of coarse-grained clay ball fill is estimated to be 14 inches and of the fine grained fill to be about 1.6 feet.

9 E2 CLINTON PLACEMENT AREA

9.1 General

The E2 Clinton Placement Area has been used previously for placement of dredge material, but has not been used for many years. Figures 9-1 to 9-3 show aerial photos of the E2 Clinton site in 1944, 1953, and 1978. In 1944 the site appears to have been partially filled, the northern portion starting about 200 feet south of boring ECP-2002 does not appear to be filled. In 1953 the site appears to have been at least partially filled after 1944. In 1978 the site appears to have been dormant for a period of time since the last filling, but a pond has been excavated in the northern end of the site west of borings ECP-2002 and ECP-2003. The site has been leased for grazing in recent years.

For the current project dikes will be constructed to El. +55 feet MLLW around the site perimeter.

Figure 9-1 – E2 Clinton Site - 1944

Figure 9-2 – E2 Clinton Site - 1953

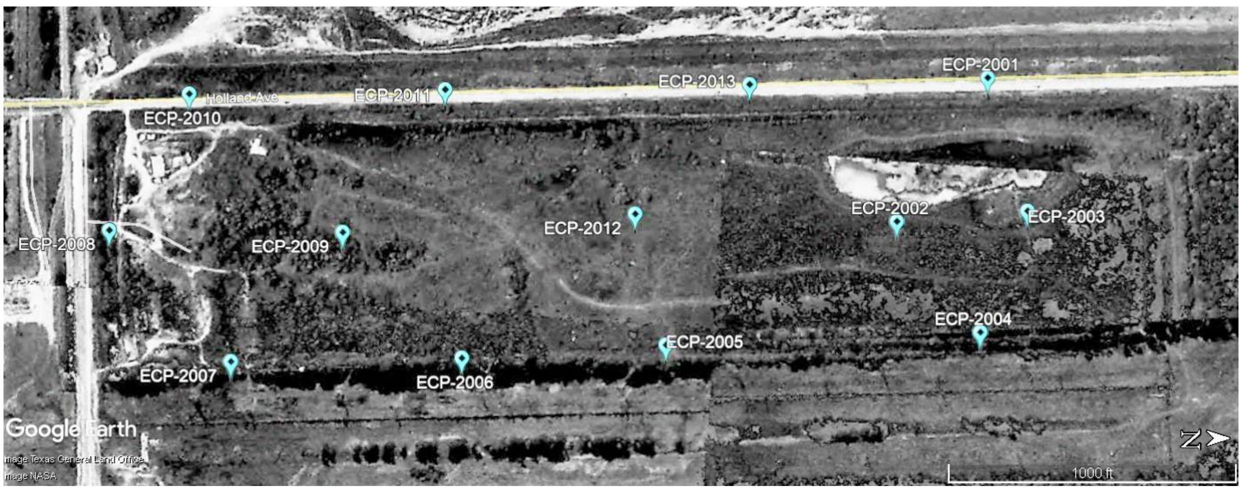


Figure 9-3 – E2 Clinton Site - 1978

9.2 Generalized Soil Conditions

Borings ECP-2001 thru ECP-2013 were drilled at this location to a depth of 40 to 70 feet below the existing grade. Borings ECP-2002, ECP-2009 and ECP-2012 were performed in the site interior and the remaining borings were drilled along the proposed dike alignment.

Very soft to very stiff clays were predominantly observed from the surface to the boring termination depths at most of the locations. Very loose to medium dense sands or silts were encountered in borings ECP-2003, ECP-2007, ECP-2008, ECP-2010, ECP-2011 and ECP-2012 at various depths.

Groundwater was encountered between El. +16 and +28 feet MLLW and between 3 and 14 feet during drilling. Note that groundwater levels encountered during drilling are approximate and are often different than static water levels measured in piezometers (no piezometers were installed for this study). Often piezometer data shows groundwater levels at shallower depth or higher elevation than the levels encountered during drilling, but not always. The borings generally encountered groundwater at El. +20 to +26 feet and depth of 3 to 10 feet during drilling. Two borings in the northwest corner of the site encountered groundwater deeper at El. +16 to +17 and depth of 13 to 14 feet during drilling.

Very soft clay and very loose silty sand soils were encountered in ECP-2005, ECP-2006, ECP-2009, ECP-2010, ECP-2011, and ECP-2012 above about El. +20 feet MLLW. These soils are consistent with properties often observed for hydraulically placed interior fill at placement areas.

In order to investigate the extent of hydraulically placed interior fill at the dike location a program of 31 piezocone penetrometer (PCPT) were performed to depths ranging from 15 to 40 feet. Three of these tests were also used to confirm the soil conditions in the northwest corner of the site.

We note that samples obtained at boring ECP-2005 between 6 and 10 feet depth, ECP-2010 between 6 and 10 feet, and at ECP-2011 between 6 and 8 feet contain gasoline odor. We understand that potential environmental contamination of soil and groundwater is being addressed by others.

9.3 Dike Fill

The proposed construction will require borrow to about El. +12 feet MLLW in the site interior in order to provide fill material for dike construction. In addition, replacement of dike foundation soils with fill is needed to address global stability concerns for portions of the dike alignment as discussed in Sections 9.4 and 9.6. Both the borrow area and dike foundation replacement will require dewatering during construction.

A significant portion of the planned borrow material is hydraulically placed fill from prior use of the site as a placement area. Although the site has not been used for many years the former hydraulic fill is a combination of very soft clay and very loose silty sand. This material will require substantial drying and mixing with stabilizing agents such as lime to be used as dike or key trench fill. We estimate that this material will be encountered above about El. +20 feet MLLW over the borrow area in the middle of the site south of about a line connecting Station 46+50 on the east dike to Station 19+00 on the west dike. This material will also be encountered in foundation replacement areas recommended for dike construction as discussed in Section 9.5.

Borrow soils that are natural are stronger but much of it is below the water table. These soils will not require stabilization for use as dike fill.

There are two types of fill that can be considered for dike construction – compacted and semi-compacted. Compacted fill is placed in loose lifts of 6 to 9 inches, compaction is controlled based on field density testing, and moisture is controlled within a relatively narrow range. Semicompacted fill is placed in loose lifts of 12 inches, compacted based on controlled movement of hauling equipment or limited passes of compaction equipment, and is placed at in situ moisture, although very wet fill is will require drying and/or treatment with a stabilizing agent such as lime. Strength of compacted fill is higher than semicompacted fill, therefore a dike constructed of compacted fill will require less volume that for semicompacted fill. However, the compacted fill is more expensive to produce and place, and constructability issues related to drying wet borrow material can prove challenging.

In the Houston area drying fill material in the winter months is essentially impossible due to periodic cold fronts that can bring substantial rainfall every 3 to 5 days and low temperatures. The ability of construction equipment to simply move about the site can be challenging due to wet conditions. In the summer drying can be accomplished, however, afternoon thunderstorms are common which impact drying operations. Due to these constructability issues we believe that dike fill material should be considered semicompacted fill.

Dike fill should be semicompacted fill compacted based on controlled movement of hauling equipment or limited passes of compaction equipment, and is placed at in situ moisture, although very wet fill is will require drying and/or treatment with a stabilizing agent such as lime. Fill material should be placed in loose lifts not exceeding twelve inches in thickness and should be compacted to 95 percent of Standard Proctor maximum dry density as determined by ASTM D698 without a moisture requirement. The former hydraulic fill material should be dried and treated with lime prior to use as dike or key trench fill. The Plasticity Index of the former hydraulic fill varies widely from 8 to 78 in the samples from our borings. About a third of the tests reflect low plasticity (PI < 20) material that will require relatively lower amounts of lime to achieve drying, for estimating purposes assume 4% by weight. The reminder are high plasticity (PI between 45 and 78) and will require significantly more lime to achieve drying, for estimating purposes assume 8% by dry weight. The actual percentage of lime used should be determine based on testing the borrow material during construction.

We assumed that the dike fill material will have an undrained shear strength of 600 psf for end of construction analyses and will have drained friction angle of 23° for long term analysis with drained cohesion of 100 psf. These are consistent with semicompacted fill.

9.4 Slope Stability

Analyses were performed for the cases – Interior Dike Slope at End of Construction (Int/EOC), Exterior Dike Slope at End of Construction (Ext/EOC), and Exterior Dike Slope at Long Term (Ext/LT). We have performed the analyses at dike centerline Stations 0+50, 20+00, 34+00, 45+50, 54+50, 62+68 and 78+00. These locations were chosen based on relative strength of the soils as revealed by the boring logs and the variation in the existing grade cross section.

For the long term exterior stability analysis, we assumed the interior was filled with dredge fill with 2 foot minimum freeboard and 1 foot of ponding depth. Freeboard is the vertical distance between the water surface and the top of dike crest. Ponding is the vertical distance between the water surface and top of interior fill. Water level in the exterior side for all cases and for short term interior stability analysis was assumed at the ground water depth observed in the boring. For interior analysis the borrow excavation was assumed to be dewatered.

During our analyses we determined that the original proposed exterior slope of 3H:1V did not meet the required long term factor of safety. The factor of safety for a 3H:1V exterior slope is about 1.27 which is lower than the required factor of safety of 1.50 as discussed in Section 4.1. In order to achieve the required long term factor of safety an exterior slope of 4H:1V is required. The design cross sections and slope stability analyses outputs are presented in Appendix F and the results are summarized in the table below using an exterior slope of 4H:1V and interior slope of 3H:1V.

Table 9-1 – E2 Clinton PA Slope Stability Results – 4H:1V Exterior, 3H:1V Interior Slopes

Boring	Station	Factor of Safety				
		Ext/EOC		Int/EOC		Ext/LT
		Circular	Block	Circular	Block	Circular
ECP-2010	0+50	1.31	**1.03***	**1.07***	**0.80***	**1.19***
ECP-2013	20+00	2.47	2.53	2.06	2.15	1.57

Boring	Station	Factor of Safety				
		Ext/EOC		Int/EOC		Ext/LT
		Circular	Block	Circular	Block	Circular
ECP-2003	34+00	2.20	2.31	1.86	1.94	1.57
ECP-2004	45+50	1.95	1.97	1.63	1.68	1.50
ECP-2005	54+50	**1.05***	**0.85***	**0.82***	**0.64***	1.54
ECP-2006	62+68	1.56	1.39	**1.28***	**1.19***	1.64
ECP-2008	78+00	2.66	2.50	2.09	2.08	1.60

*Does not meet the minimum required safety factor.

The stability analyses meet or exceed the required minimum factor of safety discussed in Section 4.1 except at Stations 0+50, 54+50 and 62+68. The borings at these locations encountered previously placed hydraulic fill. Dike foundation needs to be replaced to the limits presented in the table below.

Table 9-2 – E2 Clinton PA Recommended Fill Key

Boring	Station	Key Bottom Elevation, Feet (MLLW)	Width from the Dike Centerline, Feet	
			Interior	Exterior
ECP-2010	0+50	+20	Full dike width	70
ECP-2005	54+50	+20	Full dike width	60
ECP-2006	62+68	+23	32.5	Not required

The results of the stability analyses with the recommended fill key at these locations are summarized in the table below.

Table 9-3 – E2 Clinton PA Slope Stability Results With Fill Key

Boring	Station	Factor of Safety				
		Ext/EOC		Int/EOC		Ext/LT
		Circular	Block	Circular	Block	Circular
ECP-2010	0+50	1.53	1.50	1.47	1.49	1.53
ECP-2005	54+50	1.40	1.34	1.46	1.54	1.59
ECP-2006	62+68	1.56	1.38	1.34	1.34	1.64

All factors of safety meet or exceed the required minimum values as discussed in Section 4.1.

9.5 Site Capacity

The site capacity analysis was performed based on assessing the character of the material to be dredged from the channel. Based on the proposed cut elevations and soil properties observed in the boring logs we determined the following distribution of material, note that this is an estimate for site capacity estimating purposes only. For specific soil conditions refer to the borings performed for this study.

Table 9-4 – Soil Type in Cut – E2C PA

Soil Type	Distribution in Cut
Sand	24.1%
Firm to Stiff CL Clay	28.7%
Firm to Stiff CH Clay	23.7%
Very Soft to Soft Clay	23.6%

In the above table the designation of CL and CH clay correspond to those used in the discussion of Retention in Section 4.4. The retention factors for Firm to Stiff CL and CH Clay are 23% and 66% based on an assumed pump distance of 17,000 to 38,000 feet. Based on the retention factors and distribution of soils in the cut we estimate that 56% of the cut material will coarse-grained fill. This includes all of the sand and the retained portions of the firm to stiff CL and CH clay. The remaining 44% will be slurried fine-grained fill.

The E2C Placement Area has an area of 50.9 acres inside the dike and an average depth of 35.8 feet. We assumed 2 feet of freeboard and 1 foot of ponding for the analysis. The duration of dredging is about 99 days based on a 30-inch dredge operating 17 hours per day, 7 days per week.

The SETTLE analysis results are presented in Appendix H. They indicate that the cut yard capacity of the E2C placement area is 1,482,550 cubic yards. Coarse fill will occupy about 15 acres of the site interior near the discharge points and fine grained fill will occupy the remainder of the site.

9.6 Settlement

9.6.1 Dike

We performed consolidation settlement calculations based on the procedures discussed in Section 4.3. The analysis indicates that a consolidation soil settlement between 0.1 and 5.6 inches could occur due to the fill weight from the construction. The settlement is highest along the north dike. The settlement occurs over a period of time. The results of our analyses are presented in the figure below and Appendix F.

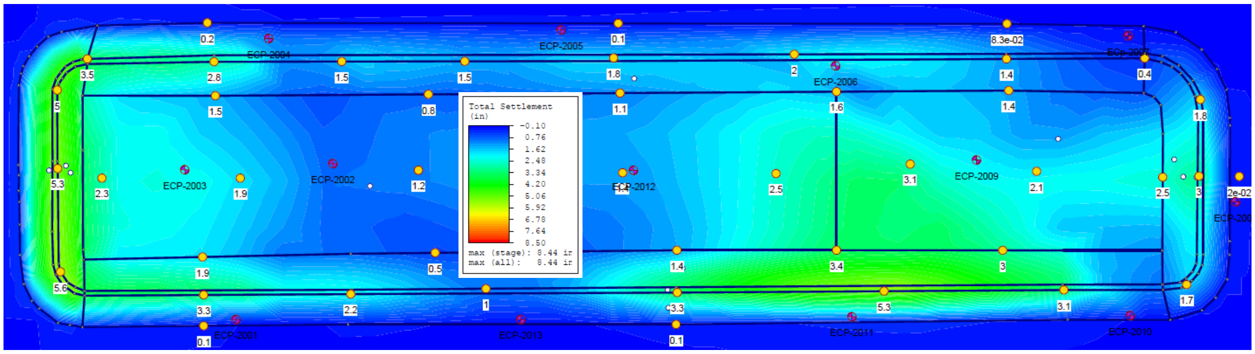


Figure 9-4 – 50-Year Consolidation Settlement

The following table shows the estimated maximum settlement vs. time of the dike.

Table 9-5 – Settlement vs. Time at E2C Dike

Time, Years	Estimated Settlement, Inches		
	Dike Fill	Foundation Consolidation	Total
1	1.8	1.7	3.5
5	3.9	3.0	6.9
20	3.9	4.6	8.5
50	3.9	5.6	9.5

9.6.2 Coarse Fill

We performed consolidation settlement calculations based on the procedures discussed in Section 4.3. The analysis indicates that a consolidation soil settlement between 2.2 and 3.1 inches could occur due to the fill weight in the site interior. The settlement occurs over a period of time. The results of our analyses are presented in Figure 9-4 and Appendix F.

The following table shows the estimated maximum settlement vs. time of the coarse grained fill.

Table 9-6 – Settlement vs. Time of E2C Coarse Fill

Time, Years	Estimated Center Settlement, Inches		
	Coarse Fill	Foundation Consolidation	Total
1	10.2	2.2	12.4
5	20.4	2.7	23.1
20	20.4	2.9	23.3
50	20.4	3.4	23.8

9.6.3 Fine Fill

After filling the fine-grained fill will settle an average of about 10 feet over a period of about 5 to 7 years as shown in the figure below. The analysis of fine-grained fill settlement was performed using PSDDF as discussed in Section 4.3.2. PSDDF uses changes in void ratio to predict settlement. The most important aspects of making PSDDF settlement estimates is determining the height of fill and the duration of the placement. As discussed in Section 9.5 the duration of filling is expected to be about 99 days assuming a 30-inch dredge is used.

The height of fill is determined from the volume of solids which is based on the void ratio of the cut soils. This averages about 1.92 for the very soft to soft clay and 0.59 for the firm to stiff clay. Based on the cut soil distribution discussed in Section 9.5 this gives a volume of solids of about 308,000 cubic yards for the fine-grained fill. Based on an assumed initial void ratio of 5.2 during discharge and the area of the site occupied by the fine-grained fill the height of fill input in PSDDF is 33.7 feet. The analysis results are presented in Appendix H.

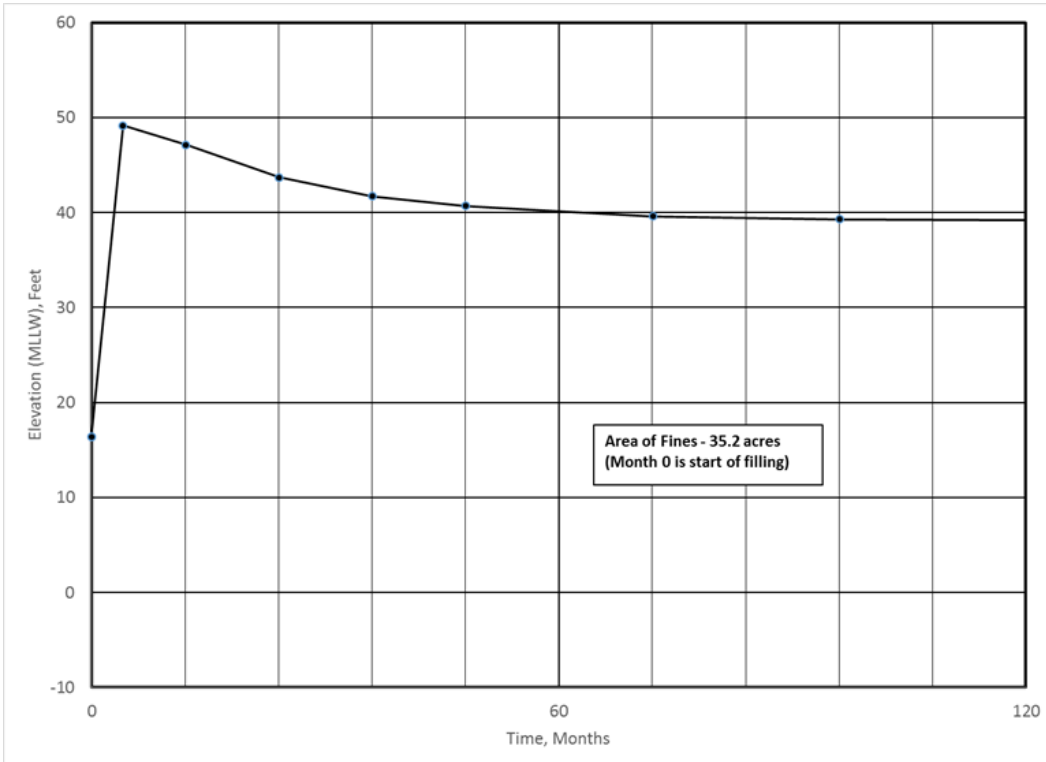


Figure 9-5 – Fine-Grained Fill Settlement – E2C PA

9.7 Weir Box/Access Bridge Foundation Recommendations

9.7.1 General

Based on the drainage outfall drawings provided, we understand that an access bridge will be constructed at the proposed weir box near Station 35+76. Boring ECP-2003 was drilled in the vicinity of the access bridge to a depth of 70 feet below the existing grade. We understand that the bridge will be supported using HP14x89 or W8x40 piles (Structural steel piles).

9.7.2 Axial Capacity

Allowable compressive and tensile capacity curves were developed for steel piles based on API RP 2A method with the use of APILE computer program. Skin friction contributed at the top 10 feet from the existing grade was ignored to account for construction disturbances associated with the outfall installation. Groundwater was assumed at the existing grade level. The driven pile capacity curves for allowable axial capacity under compression and tension are presented in Appendix G. In order to determine the allowable compressive capacity a factor of safety must be applied to the total ultimate capacity. Allowable axial tensile capacity can be calculated by applying a factor of safety to the ultimate skin friction capacity. Factors of safety should be determined based on USACE EM1110-2-2906 Design of Pile Foundations. In order to rely on factors of safety based on capacity verified by pile driving analyzer a minimum of 2 piles or 5% of the total piles driven for each structure should be tested, whichever is greater.

The soils at weir box location are firm to very stiff clay. Consolidation settlement of about 2.3 inches is estimated after 50 years at the interior dike toe near boring ECP-2003. The firm soils above about El. +20 will settle more than the deeper stiff to very stiff soils and downdrag loading could occur in the firm soils. We recommend that downdrag be included in the design for the firm soils about El. +20 feet MLLW. The skin friction due to downdrag can be taken as one –half of the undrained shear strength which is 400 psf.

Soils in the Houston area are not generally corrosive. We have not made any specific tests of soil corrosivity for this study. However, hydraulic placement of fill in the site interior will introduce brackish water into the site that will drain through the weir box. Corrosion consistent with a salt water environment should be expected for structural members exposed to the site interior fill and discharge water.

9.7.3 Pile Driving Vibrations

Existing structures are located on adjacent property near the northern boundary of the site. The vibration due to pile driving may affect these structures.

The sensitivity of the structures to vibrations should be assessed prior to beginning pile driving and vibration monitoring equipment should be used to assess whether damaging vibrations are occurring or may occur. The Contractor should be prepared to alter the installation methods to reduce vibrations to tolerable levels.

9.7.4 Pile Construction Recommendations

Methods and effects of pile installation are important considerations in the choice and design of pile foundation systems. Piles normally experience their largest stresses during installation. Pile and soil properties, embedment length and driving equipment are a few of the variables that must be considered.

1. We note that difficult driving may be encountered below about El. -25 feet due to an N value of 76.

2. Adequate cushioning material should be provided between the pile driver and the pile head. A six to twelve-inch thick cushion of softwood is usually adequate for piles that are over 50 feet long. Cushioning material condition should be carefully observed and the cushion must be changed if excessive compression occurs or at least every three piles.

3. Based on our experience, piles can usually be safely driven to about 100 blows per foot. Consistent blow counts above 100 blows per foot are not advisable.

4. The hammer, cushion and pile should be designed such that installation to design specifications can be realized with no damage to the pile.

5. The pile driving cap should fit loosely around the top of the pile so that torsional stresses do not develop in the pile. The cap should, however, be able to control the alignment of the pile.

6. Prior to driving, the pile should be properly aligned and held with fixed leads. The pile should not be realigned once driving has begun.

7. Clays and some silty soils tend to undergo a reduction in strength during pile driving and regain strength after pile installation. This phenomenon is usually referred to as freeze or set-

up. The number and duration of delays in the driving program should be minimized so as to control the effect of set-up and pile heaving. Pilot holes will also minimize this effect.

9.8 Recommendations

The global stability analyses indicate the proposed cross sections are stable with dike elevation of +55 feet MLLW, a crest width of 15 feet, a 4:1 exterior slope and 3:1 interior slope except at Stations 0+50, 54+50 and 62+68. Dike foundation replacement with fill is needed at the following locations.

1. Section 1 – Extends 70 feet from the centerline to the exterior and the full dike width from the centerline to the interior. Limits are from Station 1+00 to 19+00 with bottom elevation as follows:
    a. Station 1+00 to 15+50 – El. +20
    b. Station 15+50 to 19+00 – El. +25
2. Section 2 – Extends 60 feet from the centerline to the exterior and the full dike width from the centerline to the interior. Limits are from Station 48+50 to 65+00 with bottom elevation as follows:
    a. Station 48+50 to 54+50 – El. +24
    b. Station 54+50 to 65+00 – El. +20
3. Section 3 – Extends 60 feet from the centerline to the exterior and the full dike width from the centerline to the interior. Limits are from Station 73+00 to 75+50 with bottom at El. +22.

The estimated site capacity is 1,482,550 cut cubic yards. Settlement at the center of the dike due to foundation consolidation and fill settlement will be about 3.5 inches after 1 year and 7 inches after 5 years which will reduce the general elevation of the dike crest to about +54.4 feet MLLW. The settlement of coarse-grained clay ball fill is estimated to be 24 inches and of the fine grained fill to be about 10 feet.

Dike fill should be semicompacted fill compacted based on controlled movement of hauling equipment or limited passes of compaction equipment, and is placed at in situ moisture, although very wet fill is will require drying and/or treatment with a stabilizing agent such as lime. Fill material should be placed in loose lifts not exceeding twelve inches in thickness and should be compacted to 95 percent of Standard Proctor maximum dry density as determined by ASTM D698 without a moisture requirement. Borrow material for the dike and foundation replacement will encounter previously placed hydraulic fill that will need to be dried and stabilized with an estimated 4% to 8% of lime in order to be suitable for use as fill.

10 BELTWAY 8 PLACEMENT AREA

10.1 General

The Beltway 8 Placement Area site was previously used partially as part of a military ordnance depot on the northern portion of the site and partially for dredge material placement on the southern portion of the site. The former San Jacinto Ordnance Depot was under military control between 1942 and 1960 for storing and out-loading ammunition, producing anhydrous ammonia, and demilitarizing conventional munitions. According to historical documentation, chemical weapons (e.g., phosgene- and mustard-gas-filled bombs) were also managed at this installation in 1946. The site may also have been used for burial of both conventional and chemical ordnance.

Environmental studies have been done by others that concluded in part that there is a potential for buried munitions and explosives of concern within the depot limits.

The southern portion of the site has been used historically for dredge material placement, evidence of such placement can be seen in Figure 10-1. Based on review of aerial photos the southern portion of the site does not appear to have been used for dredge material placement since at least 1978.

Figure 10-1 – Beltway 8 Placement Area, circa 1953

10.2 Generalized Soil Conditions

Borings ECP-2014 thru ECP-2044 were drilled at this location to a depth of 40 below the existing grade. In general, firm to hard cohesive soils were observed throughout the boring depths with occasional loose to medium dense silt below 28 feet. At most locations the groundwater depth varied between 18 and 39 feet. As borings ECP-2023 and ECP-2024 drilled at northwest corner of the site groundwater was encountered at 11 to 12 feet. We note that groundwater was not observed in borings ECP-2027, ECP-2028, ECP-2033 and ECP-2038.

Borings ECP-2014 and ECP-2044 performed in the southwestern corner of the site are in an area where the ground surface elevation is at or near El. +30. These borings generally encountered very loose to loose silty sand or silt in the upper 10 to 12 feet with groundwater encountered at 6 to 8 feet. A surficial layer of 2 feet thick very loose silt or sand was observed in borings ECP-2015, ECP-2017, ECP-2021, ECP-2019 and ECP-2039 and a layer 4 feet thick was encountered at ECP-2031. At ECP-2015 a layer of loose to medium dense sandy silt and silty sand was encountered between 6 and 12 feet.

10.3 Dike Fill

The proposed construction will require borrow to about El. +20 feet MLLW in the site interior in order to provide fill material for dike construction. Dike fill should be semicompacted fill compacted based on controlled movement of hauling equipment or limited passes of compaction equipment, and is placed at in situ moisture, although very wet fill is will require drying and/or treatment with a stabilizing agent such as lime. Fill material should be placed in loose lifts not exceeding twelve inches in thickness and should be compacted to 95 percent of Standard Proctor maximum dry density as determined by ASTM D698 without a moisture requirement. We assumed that the dike fill material will have an undrained shear strength of 600 psf for end of construction analyses and will have drained friction angle of 23° for long term analysis with drained cohesion of 100 psf. These are consistent with semi-compacted fill as discussed in Section 9.3.

10.4 Slope Stability

Analyses were performed for the cases – Interior Dike Slope at End of Construction (Int/EOC), Exterior Dike Slope at End of Construction (Ext/EOC), and Exterior Dike Slope at Long Term (Ext/LT). We have performed the analyses at dike centerline Stations 10+00, 35+00, 50+00, 70+00, 74+00, 87+18, 92+00, 100+00 and 120+00, and 155+00. These locations were chosen based on relative strength of the soils as revealed by the boring logs and the variation in the existing grade cross section.

Based on the available information we understand that the proposed dike alignment will cross the Enterprise pipeline at three locations (Approx. Sta. 74+00, 95+00, 97+00 and 100+00) and following constraints are imposed:

1. Enterprise has said that the presence of a dike over their pipeline is acceptable
2. Clearing will be allowed to remove vegetation, but grubbing will not be allowed

Since no grubbing will be allowed there will be a layer near the surface with a significant fraction of organics. We evaluated global stability near the crossings assuming that this near surface layer has a reduced shear strength of 500 psf or angle of internal friction of 22°.

For the long term exterior stability analysis, we assumed the interior was filled with dredge fill with 2 foot minimum freeboard and 1 foot of ponding depth. Freeboard is the vertical distance between the water surface and the top of dike crest. Ponding is the vertical distance between the water surface and top of interior fill. Water level in the exterior side for all cases and for short term interior stability analysis was assumed at the ground water depth observed in the boring. For interior analysis the borrow excavation was assumed to be dewatered.

The slope stability analyses outputs are presented in Appendix I and the results are summarized in the following table.

Table 10-1 – Beltway 8 Placement Area Slope Stability Results

Station (Boring)	Factor of Safety					
	Short Term - Exterior		Short Term – Interior		Long Term	
	Circular	Block	Circular	Block	Circular	Block*
10+00 (ECP-2026)	5.87	4.16	5.64	5.74	2.01	--
35+00 (ECP-2031)	2.70	2.46	2.53	2.44	1.39	--
50+00 (ECP-2035)	6.25	6.46	6.03	6.31	2.10	--
70+00 (ECP-2037)	14.26	13.27	5.07	3.77	3.97	--
74+00 (ECP-2039, Pipeline Crossing)	4.23	3.16	3.71	3.03	1.82	--
87+18 (ECP-2039)	6.05	5.76	6.22	6.22	2.24	--
92+00 (ECP-2040 and ECP-2041, Pipeline Crossing)	4.48	4.19	5.17	4.96	1.82	1.45
100+00 (ECP-2043, Pipeline Crossing)	4.76	4.25	5.50	5.29	1.85	1.45
120+00 (ECP-2014)	4.02	4.08	1.38	1.39	2.62	--
155+00 (ECP-2022)	6.96	7.50	6.30	6.67	2.28	--

* Long term block failure was only analyzed at selected pipeline crossing locations with weak organic layers

The stability analyses meet or exceed the required minimum factor of safety discussed in Section 4.1

The current cross sections – 3H:1V at the crossing near Station 74+00 and 6H:1V at the crossings near Station 95+00, 97+00, and 100+00 – have adequate factors of safety with the weak, organic layer included. No modification to the cross sections are needed at the pipeline crossings.

Note that the dike will impose a permanent surcharge load on the pipeline at each crossing. The surcharge loading is based on the dike height and can be calculated assuming a dike fill unit weight of about 115 pcf. The dike height at the pipeline crossings ranges from about 10 to 14 feet, giving surcharge loads of 1,150 to 1,610 psf. Also, settlement of about up to about 10 inches is estimated at the pipeline locations due to dike construction as discussed in Section 10.6. The pipeline ability to sustain these settlements, permanent surcharge loads, and temporary surcharge loads due to construction equipment should be confirmed.

10.5 Site Capacity

The site capacity analysis was performed based on assessing the character of the material to be dredged from the channel. Based on the proposed cut elevations and soil properties observed in the boring logs we determined the following distribution of material, note that this is an estimate for site capacity estimating purposes only. For specific soil conditions refer to the borings performed for this study.

Table 10-2 – Soil Type in Cut – BW8 PA

Soil Type	Distribution in Cut
Sand	9.7%
Firm to Stiff CL Clay	35.8%
Firm to Stiff CH Clay	4.5%
Very Soft to Soft Clay	50.0%

In the above table the designation of CL and CH clay correspond to those used in the discussion of Retention in Section 4.4. The retention factors for Firm to Stiff CL and CH Clay are 75% and 93% based on an assumed pump distance of 2,000 to 6,000 feet. Based on the retention factors and distribution of soils in the cut we estimate that 47% of the cut material will be coarse-grained fill. This includes all of the sand and the retained portions of the firm to stiff CL and CH clay. The remaining 53% will be slurried fine-grained fill.

The BW8 Placement Area has an area of 315 acres inside the dike and an average depth of 10 feet. We assumed 2 feet of freeboard and 1 foot of ponding for the analysis. The duration of dredging is about 124 days based on a 30-inch dredge operating 17 hours per day, 7 days per week.

The SETTLE analysis results are presented in Appendix N. They indicate that the cut yard capacity of the BW8 placement area is 1,854,400 cubic yards. Coarse fill will occupy about 67 acres of the site interior near the discharge point and fine grained fill will occupy the remainder of the site.

10.6 Settlement

10.6.1 Dike

We performed consolidation settlement calculations based on the procedures discussed in Section 4.3. The analysis indicates that a consolidation soil settlement between 0.8 and 10 inches could occur due to the fill weight from the construction. The settlement is highest along the south dike. The settlement occurs over a period of time. The results of our analyses are presented in Figure 10-1 and Appendix I.

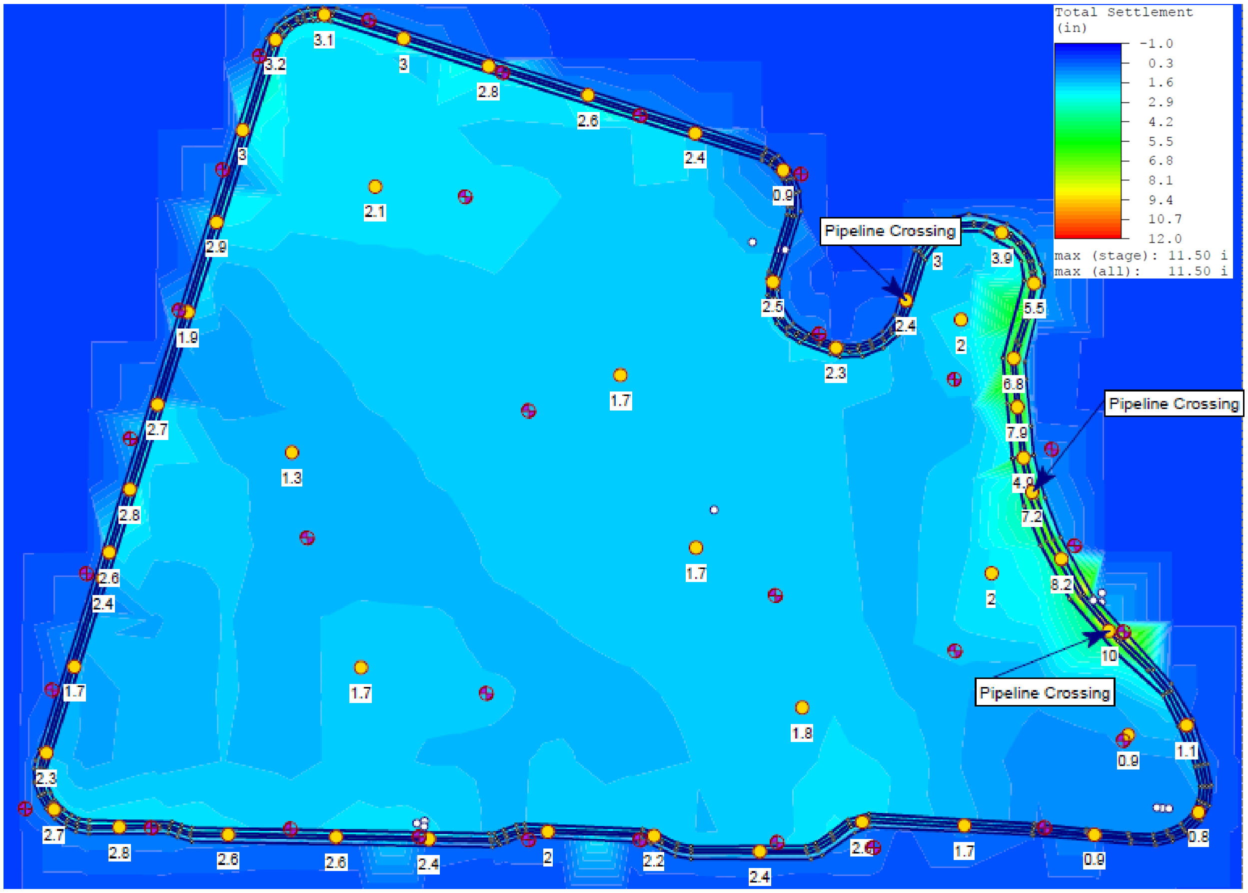


Figure 10-2 – 50-Year Consolidation Settlement

The following table shows the estimated maximum settlement vs. time of the dike.

Table 10-3 – Settlement vs. Time at BW8 Dike

Time, Years	Estimated Settlement, Inches		
	Dike Fill	Foundation Consolidation	Total
1	0.9	10.0	10.9
5	1.9	10.0	11.9
20	1.9	10.0	11.9
50	1.9	10.0	11.9

10.6.2 Coarse Fill

We performed consolidation settlement calculations based on the procedures discussed in Section 4.3. The analysis indicates that a consolidation soil settlement between 1.7 and 2.0 inches could occur due to the fill weight in the site interior. The settlement occurs over a period of time. The results of our analyses are presented in Figure 10-2 and Appendix I.

The following table shows the estimated maximum settlement vs. time of the coarse grained fill.

Table 10-4 – Settlement vs. Time of BW8 Coarse Fill

Time, Years	Estimated Center Settlement, Inches		
	Coarse Fill	Foundation Consolidation	Total
1	2.1	2.0	4.1
5	4.2	2.0	6.2
20	4.2	2.0	6.2
50	4.2	2.0	6.2

10.6.3 Fine Fill

After filling the fine-grained fill will settle an average of about 2.1 feet over a period of about 2 to 3 years as shown in the figure below. The analysis of fine-grained fill settlement was performed using PSDDF as discussed in Section 4.3.2. PSDDF uses changes in void ratio to predict settlement. The most important aspects of making PSDDF settlement estimates is determining the height of fill and the duration of the placement. As discussed in Section 10.5 the duration of filling is expected to be about 124 days assuming a 30-inch dredge is used.

The height of fill is determined from the volume of solids which is based on the void ratio of the cut soils. This averages about 1.92 for the very soft to soft clay and 0.59 for the firm to stiff clay. Based on the cut soil distribution discussed in Section 10.5 this gives a volume of solids of about 462,230 cubic yards for the fine-grained fill. Based on an assumed initial void ratio of 6.0 during discharge and the area of the site occupied by the fine-grained fill the height of fill input in PSDDF is 8.1 feet. The analysis results are presented in Appendix N.

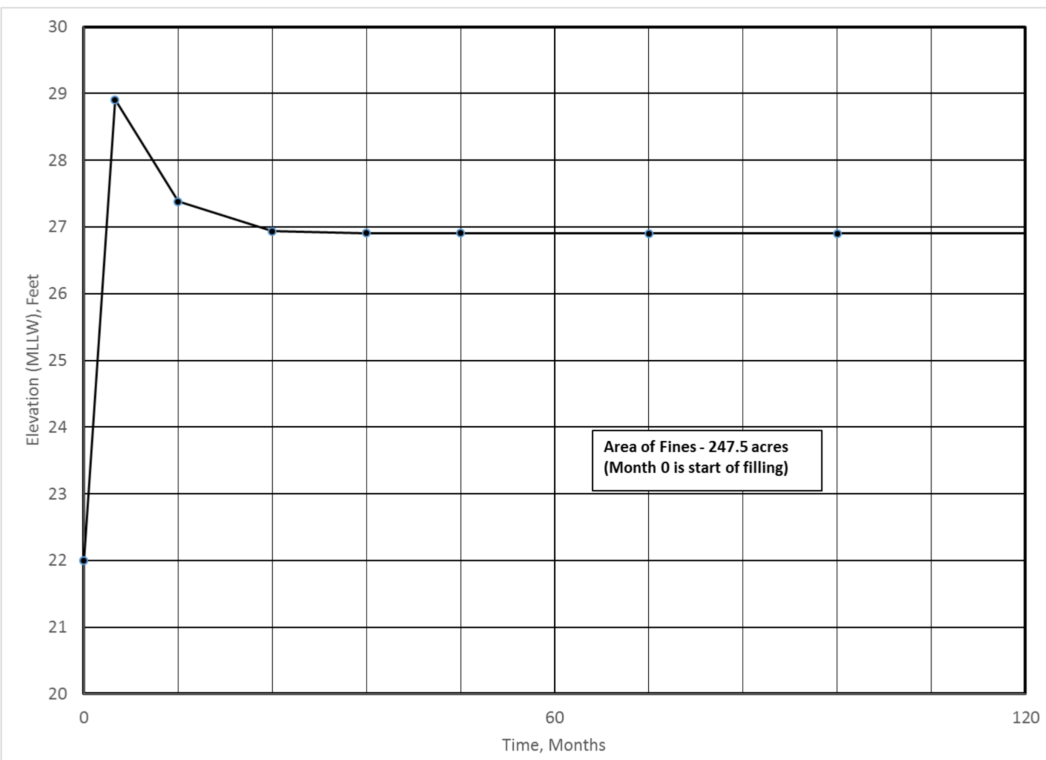


Figure 10-3 – Fine-Grained Fill Settlement – BW8 PA

10.7 Drop Structure Foundation Recommendations

10.7.1 General

Based on the information provided, we understand that a drop structure will be constructed near Boring ECP-2040. The boring was drilled to a depth of 40 feet below the existing grade. We understand that the structure will be supported using HP14x89 or W8x40 (Structural steel piles).

10.7.2 Axial Capacity

Allowable compressive and tensile capacity curves were developed for steel piles based on USACE method with the use of APILE computer program. Skin friction contributed at the top 10 feet from the existing grade was ignored to account for construction disturbances. The driven pile capacity curves for allowable axial capacity under compression and tension are presented in Appendix J. In order to determine the allowable compressive capacity a factor of safety must be applied to the total ultimate capacity. Allowable axial tensile capacity can be calculated by applying a factor of safety to the ultimate skin friction capacity. Factors of safety should be determined based on USACE EM1110-2-2906 Design of Pile Foundations. In order to rely on factors of safety based on capacity verified by pile driving analyzer a minimum of 2 piles or 5% of the total piles driven for each structure should be tested, whichever is greater.

The soils at drop structure location are firm to very stiff clay. Consolidation settlement of about 5 inches is estimated after 1 year along the dike near boring ECP-2040. The soils in about the upper 10 feet will settle more than the deeper stiff to very stiff soils and downdrag loading could occur. We recommend that downdrag be included in the design for the soils about 10 feet below the bottom of the dike fill. The skin friction due to downdrag can be taken as one–half of the undrained shear strength which is 450 psf.

Soils in the Houston area are not generally corrosive. We have not made any specific tests of soil corrosivity for this study. However, hydraulic placement of fill in the site interior will introduce brackish water into the site that will drain through the drop structure. Corrosion consistent with a salt water environment should be expected for structural members exposed to the site interior fill and discharge water.

10.7.3 Pile Driving Vibrations

The vibration due to pile driving may affect structures located close to the drop structure. We are not aware of any existing structures that may be damaged, but this should be confirmed prior to construction.

The sensitivity of structures to vibrations should be assessed prior to beginning pile driving and vibration monitoring equipment should be used to assess whether damaging vibrations are occurring or may occur. The Contractor should be prepared to alter the installation methods to reduce vibrations to tolerable levels.

10.7.4 Pile Construction Recommendations

Methods and effects of pile installation are important considerations in the choice and design of pile foundation systems. Piles normally experience their largest stresses during installation. Pile and soil properties, embedment length and driving equipment are a few of the variables that must be considered.

1. Adequate cushioning material should be provided between the pile driver and the pile head. A six to twelve-inch thick cushion of softwood is usually adequate for piles that are over 50 feet long. Cushioning material condition should be carefully observed and the cushion must be changed if excessive compression occurs or at least every three piles.

2. Based on our experience, piles can usually be safely driven to about 100 blows per foot. Consistent blow counts above 100 blows per foot are not advisable.

3. The hammer, cushion and pile should be designed such that installation to design specifications can be realized with no damage to the pile.

4. The pile driving cap should fit loosely around the top of the pile so that torsional stresses do not develop in the pile. The cap should, however, be able to control the alignment of the pile.

5. Prior to driving, the pile should be properly aligned and held with fixed leads. The pile should not be realigned once driving has begun.

6. Clays and some silty soils tend to undergo a reduction in strength during pile driving and regain strength after pile installation. This phenomenon is usually referred to as freeze or set-up. The number and duration of delays in the driving program should be minimized so as to control the effect of set-up and pile heaving. Pilot holes will also minimize this effect.

10.8 Recommendations

All cross sections meet the required factors of safety. The current cross sections – 3H:1V at the pipeline crossing near Station 74+00 and 6H:1V at the pipeline crossings near Station 95+00, 97+00, and 100+00 – have adequate factors of safety with a weak, organic layer included near the4 ground surface. No modification to the cross sections are needed at the pipeline crossings.

Note that the dike will impose a permanent surcharge load on the pipeline at each crossing. The surcharge loading is based on the dike height and can be calculated assuming a dike fill unit weight of about 115 pcf. The dike height at the pipeline crossings ranges from about 10 to 14 feet, giving surcharge loads of 1,150 to 1,610 psf. Also, settlement of about up to about 10 inches is estimated at the pipeline locations due to dike construction. The pipeline ability to sustain these settlements, permanent surcharge loads, and temporary surcharge loads due to construction equipment should be confirmed.

The estimated site capacity is 1,854,400 cut cubic yards. Settlement at the center of the dike due to foundation consolidation and fill settlement will be about 5 to 12 inches which will reduce the general elevation of the dike crest to about +31.5 to +31.0 feet MLLW. The settlement of coarse-grained clay ball fill is estimated to be 6 inches and of the fine grained fill to be about 2.1 feet.

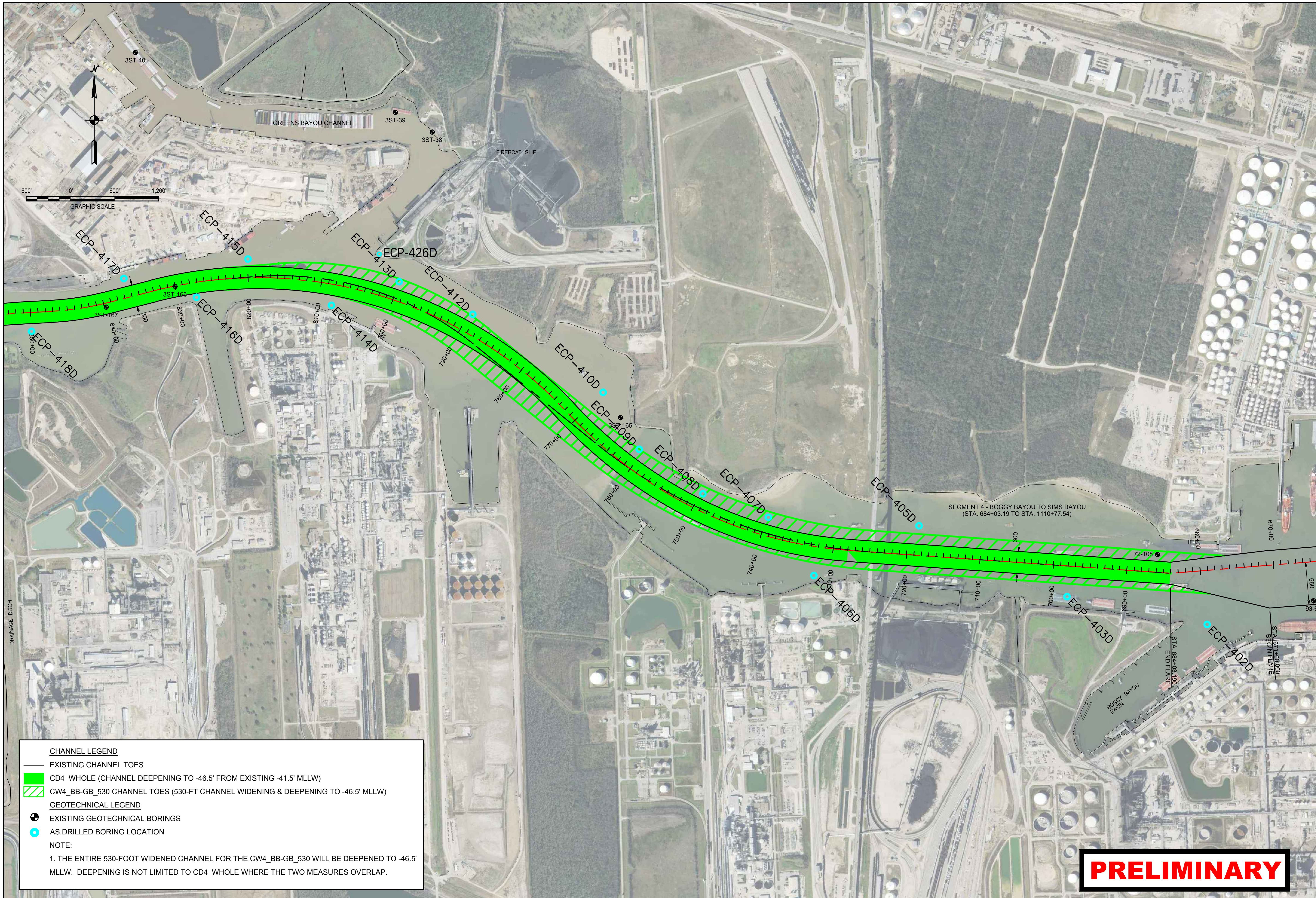
Dike fill should be semicompacted fill compacted based on controlled movement of hauling equipment or limited passes of compaction equipment, and is placed at in situ moisture, although very wet fill is will require drying and/or treatment with a stabilizing agent such as lime. Fill material should be placed in loose lifts not exceeding twelve inches in thickness and should be compacted to 95 percent of Standard Proctor maximum dry density as determined by ASTM D698 without a moisture requirement.

11 LIMITATIONS

This investigation was performed for the exclusive use HDR Engineering, Inc. for specific application to Houston Ship Channel Expansion Channel Improvement Project in Harris and Chambers Counties, Texas. HVJ Associates, Inc. has endeavored to comply with generally accepted geotechnical engineering practice common in the local area. HVJ Associates, Inc. makes no warranty, express or implied. The analyses and recommendations contained in this report are based on data obtained from subsurface exploration, laboratory testing, the project information provided to us and our experience with similar soils and site conditions. The methods used indicate subsurface conditions only at the specific locations where samples were obtained, only at the time they were obtained, and only to the depths penetrated. Samples cannot be relied on to accurately reflect the strata variations that usually exist between sampling locations. Should any subsurface conditions other than those described in our boring logs be encountered, HVJ Associates should be immediately notified so that further investigation and supplemental recommendations can be provided.

ILLUSTRATIONS

PLAN OF BORINGS



CHANNEL LEGEND

EXISTING CHANNEL TOES

CD4_WHOLE (CHANNEL DEEPENING TO -46.5' FROM EXISTING -41.5' MLLW)

CW4_BB-GB_530 CHANNEL TOES (530-FT CHANNEL WIDENING & DEEPENING TO -46.5' MLLW)

GEOTECHNICAL LEGEND

EXISTING GEOTECHNICAL BORINGS

AS DRILLED BORING LOCATION

NOTE:

1. THE ENTIRE 530-FOOT WIDENED CHANNEL FOR THE CW4_BB-GB_530 WILL BE DEEPENED TO -46.5' MLLW. DEEPENING IS NOT LIMITED TO CD4_WHOLE WHERE THE TWO MEASURES OVERLAP.

PRELIMINARY

Rev.	Description	Date	By

Drawn by: SAH
Designed by: --
Checked by: ---
Submitted by:
Approved by:
Date: 02/03/2020
Rev. 11/20
Scale: AS SHOWN
Approval Recommended:

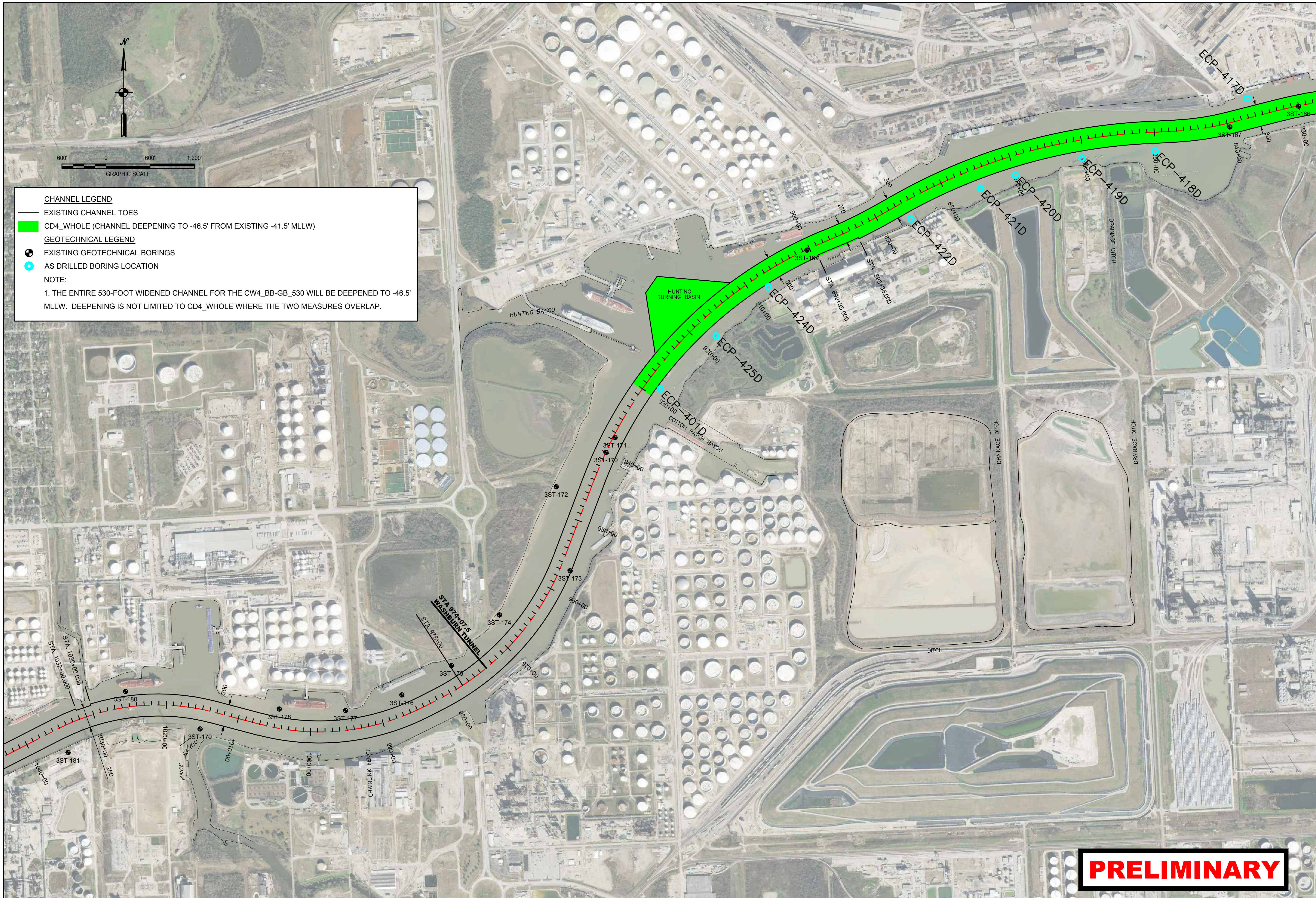
Turner Collie & Braden Inc.
GAHAGAN & BRYANT ASSOCIATES, INC.
5444 WESTHEIMER RD. SUITE 200
HOUSTON, TX 77056

HOUSTON SHIP CHANNEL EXPANSION CHANNEL IMPROVEMENT PROJECT (HSC ECIP)

GEOTECHNICAL BORING LOCATIONS
SEGMENT 4 BOGGY BAYOU TO SIMS BAYOU

Plate No.:

PLATE 1A



Rev.	Description	Date	By

HOUSTON SHIP CHANNEL
EXPANSION CHANNEL IMPROVEMENT
PROJECT (HSC ECIP)

GEOTECHNICAL BORING LOCATIONS
SEGMENT 4 BOGGY BAYOU TO SIMS BAYOU

Turner Collie & Braden Inc.
GAHAGAN & BRYANT ASSOCIATES, INC.
5444 WESTHEIMER RD. SUITE 200
HOUSTON, TX 77056

Drawn by:	SAH	Date: 02/03/2020	Rev. 11/20
Designed by:	--	Scale: AS SHOWN	
Checked by:	---		
Submitted by:		Approval Recommended:	
Approved by:			

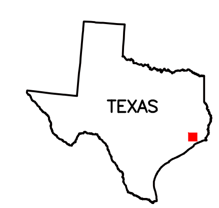
Plate No.:

PLATE 1B

Approximate Boring Location (By Others for the Current Study)

Approximate Boring Location for the Current Study

Approximate Boring Location (From the Previous Studies)



MAP LOCATION

HVJ ASSOCIATES
6201 S. DAIRY ASHFORD RD.
HOUSTON, TX 77072
PH: 281-933-7388
FAX: 281-933-7293
TEXAS FIRM NO. F-000646

SCALE: 1" = 60'		
DATE: 5/4/2020		
DRAWN BY: AR	PROJ. CHK: NK	APPRV. BY: MH

BOREHOLE LAYOUT
HOUSTON SHIP CHANNEL EXPANSION
CHANNFL IMPROVEMENT
BAYPORT SHIP CHANNEL - SJC SITE

PROJECT NO.: HG1910092.2.1	FILENAME: POB	PLATE 1C

12/19/2017
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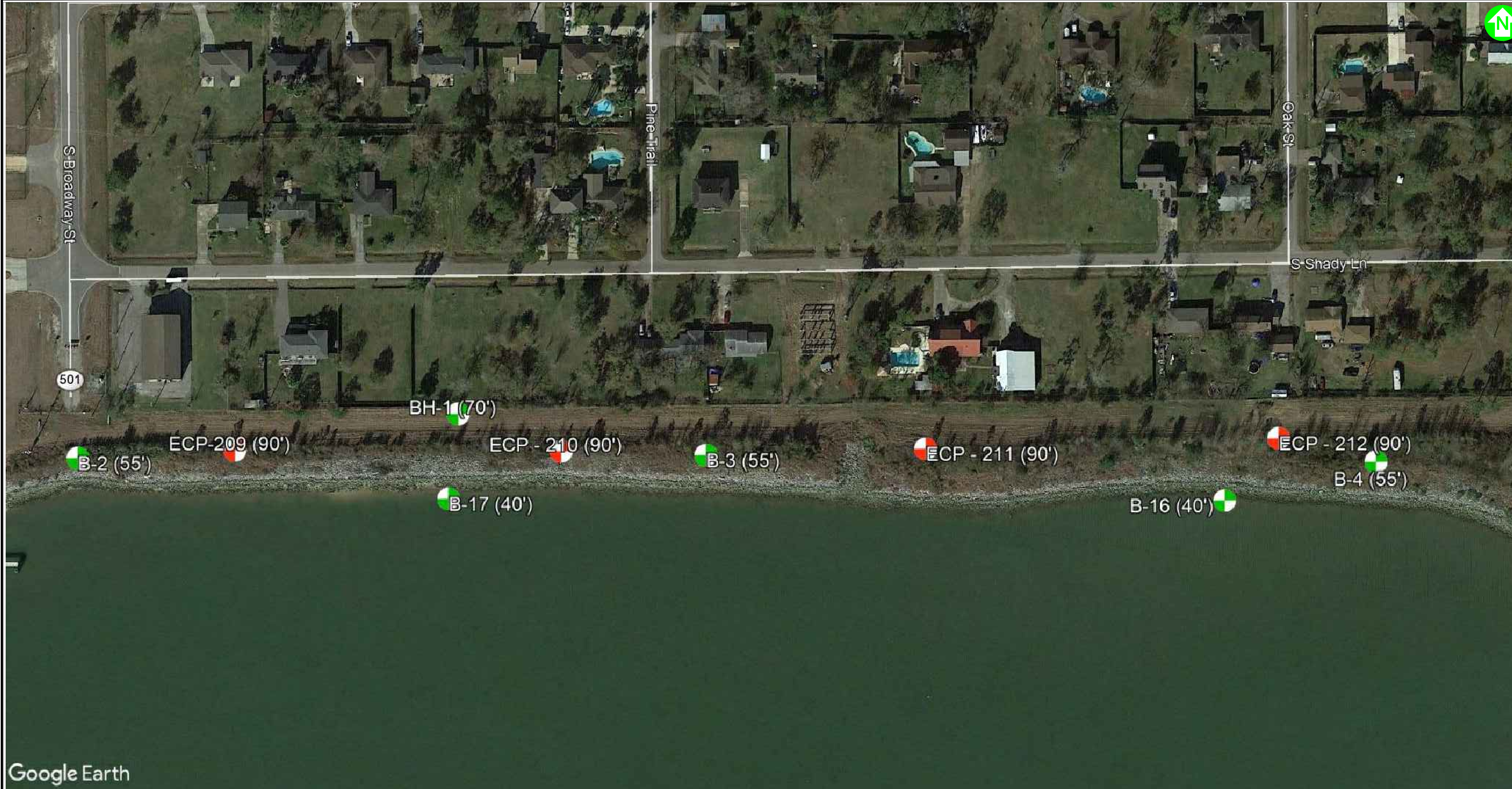
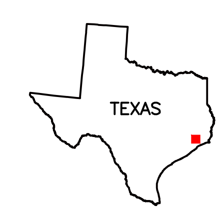


Approximate Boring Location for the Current Study

Approximate Boring Location (From the Previous Studies)



MAP LOCATION

HVJ ASSOCIATES
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SCALE: 1" = 125'

DATE: 5/4/2020

DRAWN BY:	PROJ. CHK:	APPRV. BY:
AR	NK	MH

BOREHOLE LAYOUT
HOUSTON SHIP CHANNEL EXPANSION
CHANNEL IMPROVEMENT
BAYPORT SHIP CHANNEL - EAST OF SJC

PROJECT NO.:	FILENAME:	PLATE
HG1910092.2.1	POB	1D

12/19/2017
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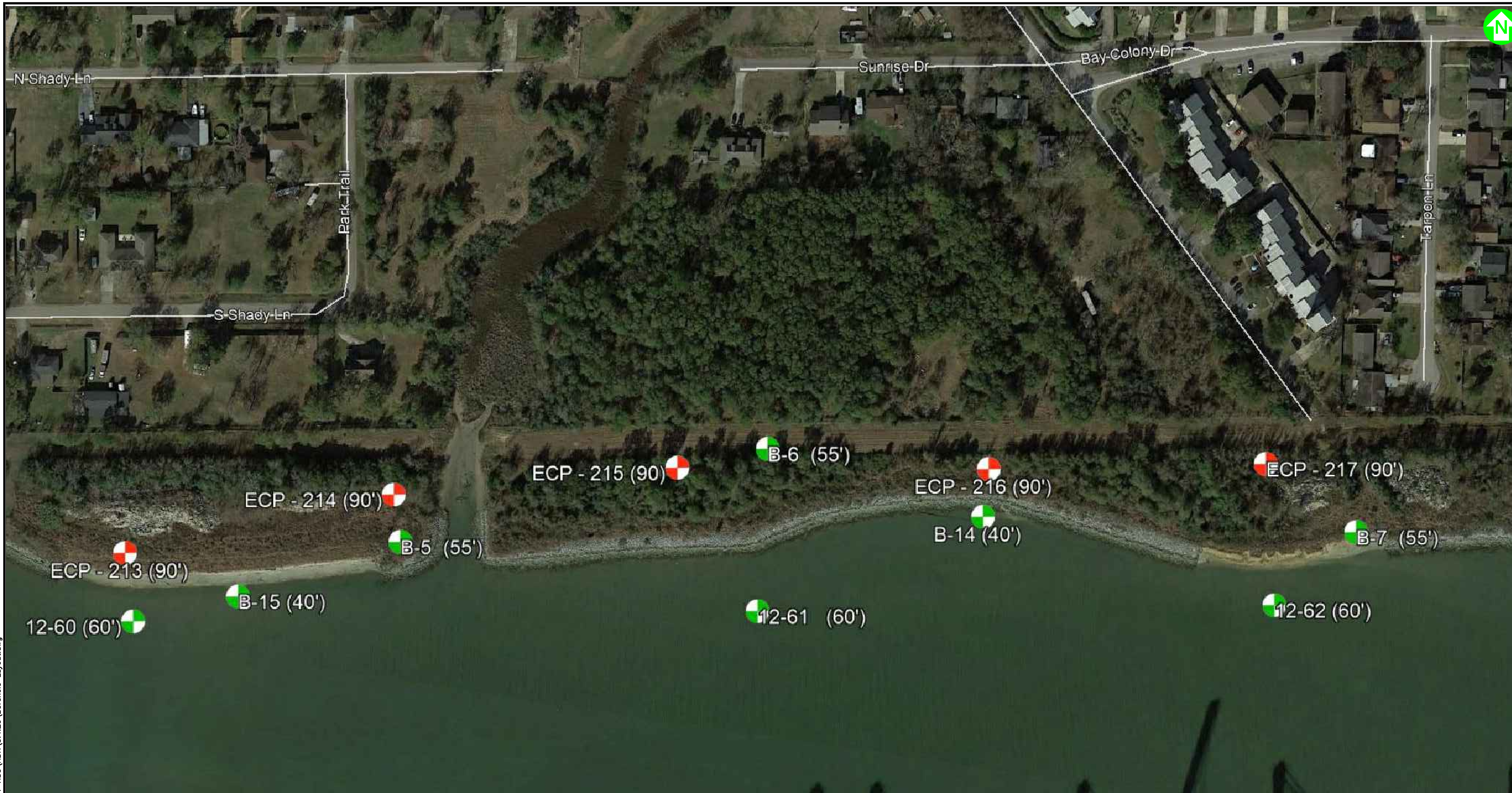
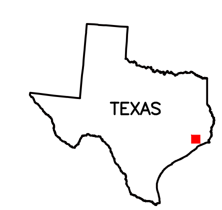


Approximate Boring Location for the Current Study

Approximate Boring Location (From the Previous Studies)



MAP LOCATION

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PH: 281-933-7388
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TEXAS FIRM NO. F-000646

SCALE: 1" = 150'		
DATE: 5/4/2020		
DRAWN BY: AR	PROJ. CHK: NK	APPRV. BY: MH

BOREHOLE LAYOUT
HOUSTON SHIP CHANNEL EXPANSION
CHANNEL IMPROVEMENT
BAYPORT SHIP CHANNEL - EAST OF SJC

PROJECT NO.: HG1910092.2.1	FILENAME: POB	PLATE 1E

12/19/2017
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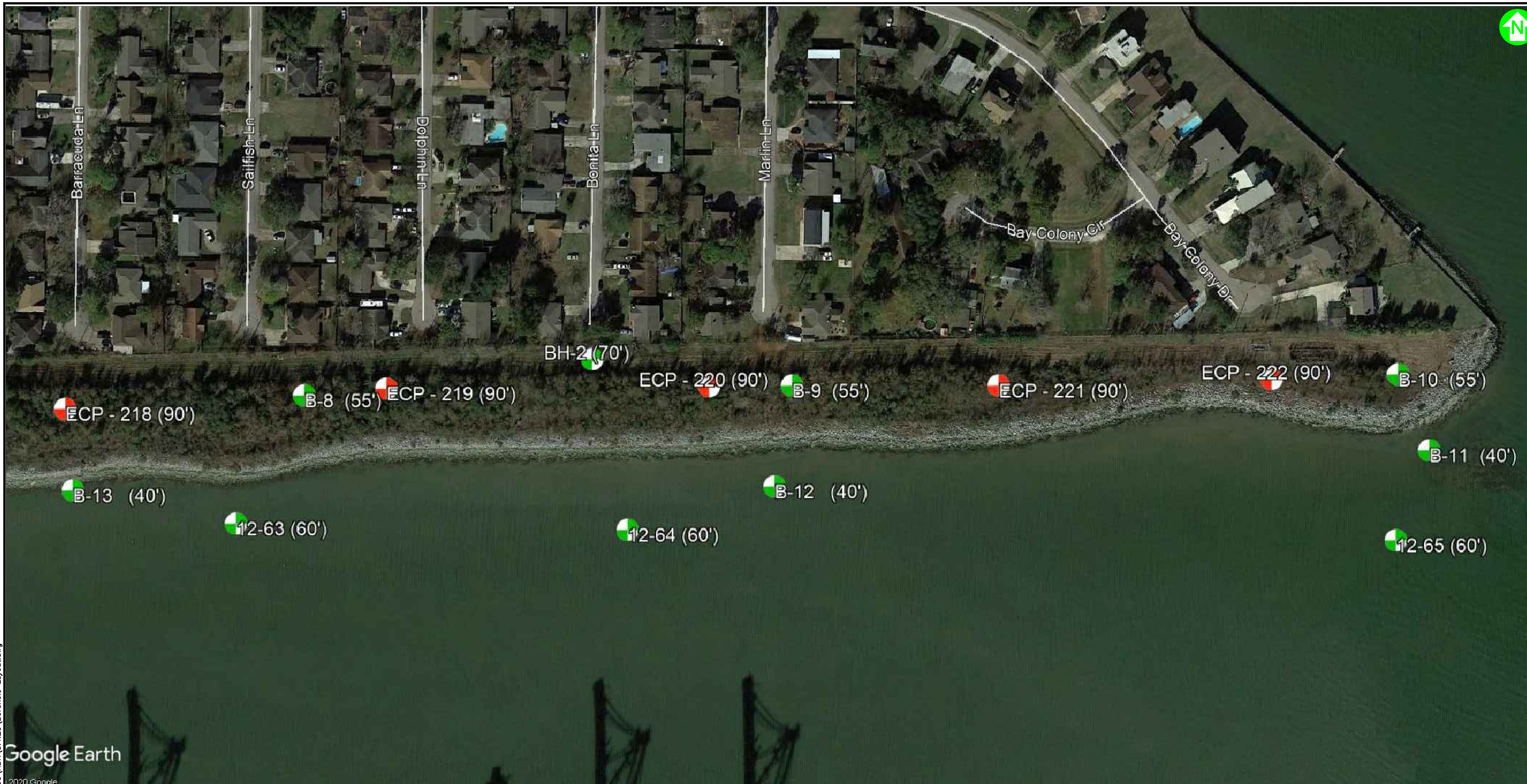
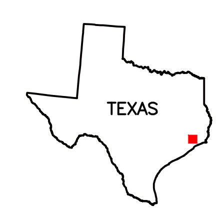


Approximate Boring Location for the Current Study

Approximate Boring Location (From the Previous Studies)



MAP LOCATION

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HOUSTON, TX 77072
PH: 281-933-7388
FAX: 281-933-7293
TEXAS FIRM NO. F-000646

SCALE: 1" = 150'		
DATE: 5/4/2020		
DRAWN BY: AR	PROJ. CHK: NK	APPRV. BY: MH

BOREHOLE LAYOUT
HOUSTON SHIP CHANNEL EXPANSION
CHANNEL IMPROVEMENT
BAYPORT SHIP CHANNEL - EAST OF SJC

PROJECT NO.:	FILENAME:	
HG1910092.2.1	POB	PLATE 1F

12/19/2017
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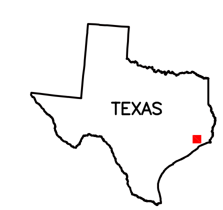
Approximate Boring Location for the Current Study

Approximate PCPT/VST Location for the Current Study

Approximate Boring Location (From the Previous Studies)



MAP LOCATION

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SCALE: 1" = 250'
DATE: 5/4/2020
DRAWN BY: AR
PROJ. CHK: NK
APPRV. BY: MH

BOREHOLE LAYOUT
HOUSTON SHIP CHANNEL EXPANSION
CHANNEL IMPROVEMENT
BARBOURS CUT SHIP CHANNEL -SPILMANS ISLAND

PROJECT NO.: HG1910092.2.1
FILENAME: POB
PLATE 1G

12/19/2017
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