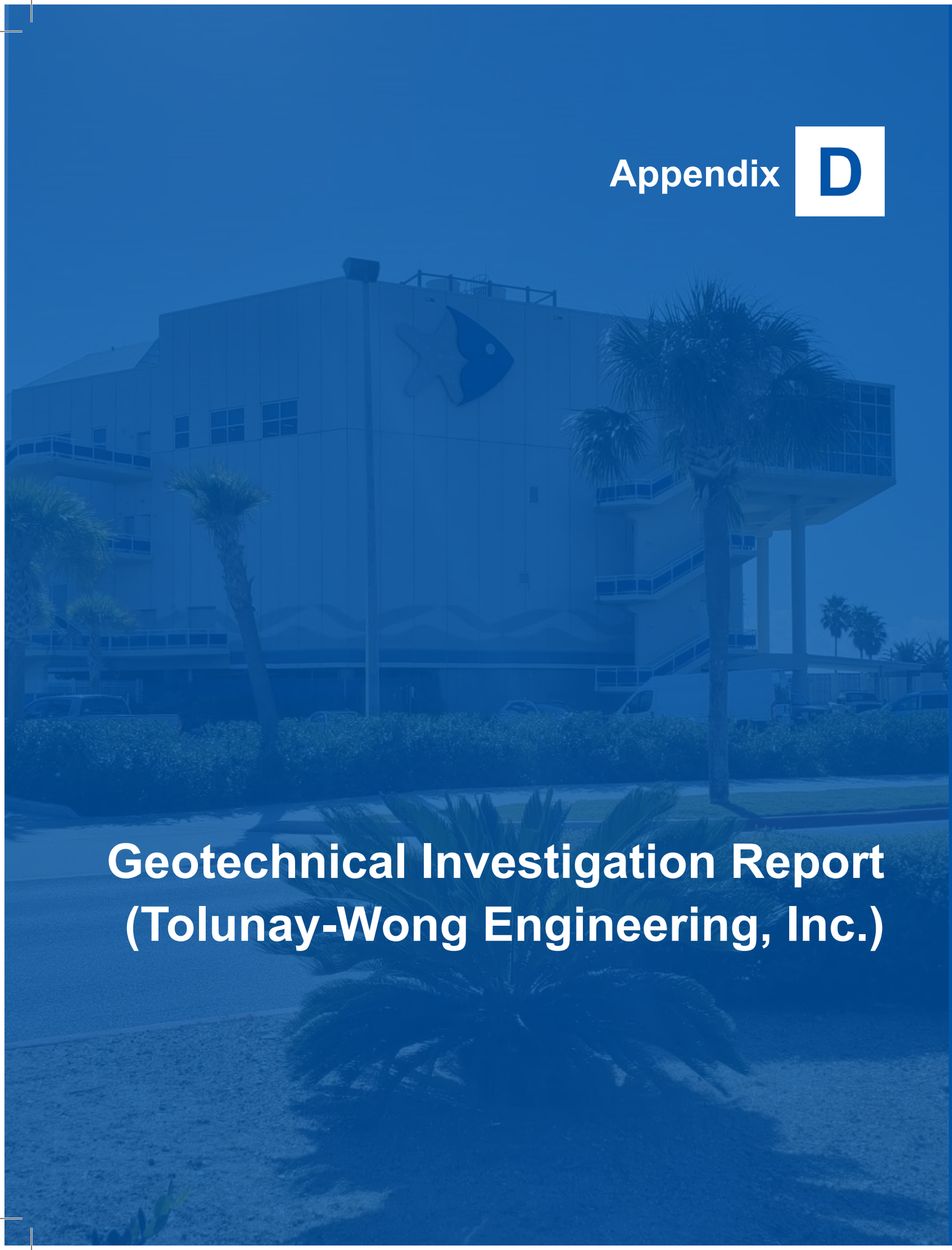
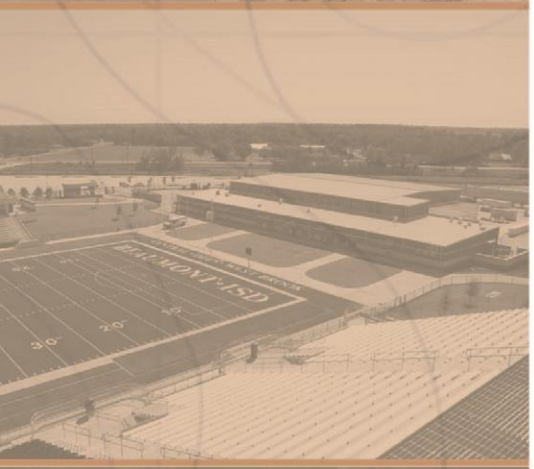


Appendix D

Geotechnical Investigation Report (Tolunay-Wong Engineering, Inc.)



Tolunay-Wong Engineers, Inc.

PRELIMINARY GEOTECHNICAL ENGINEERING STUDY
NORTH BEACH NAVIGABLE CANAL
CORPUS CHRISTI, TEXAS

Prepared for:

Lockwood, Andrews & Newnam, Inc.
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Prepared by:

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August 19, 2020

Project No. 20.53.036 / Report No. 26074

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August 19, 2020

Lockwood, Andrew & Newnam, Inc.
2925 Briarpark Drive, Suite 400
Houston, Texas 77042

Attn: Mr. Joseph Scarborough, P.E.
JEScarborough@lan-inc.com

Ref: Preliminary Geotechnical Engineering Study
North Beach Navigable Canal
Corpus Christi, Texas
TWE Project No. 20.53.036 / Report No. 26074

Dear Mr. Scarborough,

Tolunay-Wong Engineers, Inc. (TWE) is pleased to submit this report of our preliminary geotechnical engineering study for the above referenced project. This report contains a detailed description of the field program and laboratory services performed for this geotechnical engineering study as well as soil boring logs and laboratory test results.

We appreciate the opportunity to work with you on this preliminary phase of the project and we look forward to the opportunity of providing services for the final phase of the project. If you have any questions or comments regarding this report or if we can be of further assistance, please contact us.

Sincerely,

TOLUNAY-WONG ENGINEERS, INC.
TBPELS Firm Registration Number F-000124

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Appendix C: Log of Project Borings and a Key to Terms and Symbols used on Boring Logs

Appendix D: Sheet Pile Wall Global Stability Results

Appendix E: Consolidated-Undrained Triaxial Shear Tests Results

1 INTRODUCTION AND PROJECT DESCRIPTION

1.1 Introduction

This report presents the results of our preliminary geotechnical engineering study performed for the proposed North Beach Navigable Canal in Corpus Christi, Texas. Our preliminary geotechnical engineering study was conducted in accordance with TWE Proposal No. P20-C036R1, dated May 18, 2020. The study was authorized by the Subconsulting Agreement between Lockwood, Andrews, and Newnam, Inc. (LAN) and Tolunay-Wong Engineers, Inc. (TWE) and executed by Mr. Stephen A. Gilbreath, P.E. with LAN.

1.2 Project Description

We understand that a navigable canal/waterway is being proposed for construction within the North Beach area of Corpus Christi, Texas for purposes of improving drainage characteristics of the area and provide recreational opportunities. The total length of the canal/waterway will be approximately 1.25 miles and vary in width with an average depth of 10 feet. A detailed plan view provided by LAN is located in Appendix A.

2 PURPOSE AND SCOPE OF SERVICES

The purposes of our preliminary geotechnical engineering study were to investigate the general soil and groundwater conditions within the project site and to provide preliminary geotechnical design and construction recommendations for the proposed navigable canal.

Our scope of services performed for the project consisted of:

1. Drilling two (2) soil borings to depths of 50-ft within the project site to evaluate subsurface stratigraphy and groundwater conditions;

2. Performing geotechnical laboratory tests on recovered soil samples to evaluate the physical and engineering properties of the strata encountered;

3. Provide preliminary recommendations for drivability of sheet piling, global stability of canal bulkhead sheet pile wall for determination of allowable safety factor, design of anchor wall system for canal bulkhead wall, including passive resistance on the anchor wall and location of anchor wall behind main wall;

4. Provide preliminary design profile and soil parameters for bulkhead wall analysis;

5. Provide guidance for erosion susceptibility/characteristics of soils near the mudline of the sheet piles based on cross section provided by LAN;

6. Provide preliminary geotechnical design recommendations for flexible (asphalt) and rigid (concrete) pavement sections including subgrade preparation and required component thicknesses; and,

7. Provide geotechnical recommendations including subgrade preparation, excavation considerations, fill and backfill placement, and overall quality control monitoring, inspection and testing services.

Our scope of services did not include any environmental assessments for the presence or absence of wetlands or of hazardous or toxic materials within or on the soil, air or water within this project site. Any statements in this report or on the boring logs regarding odors, colors or unusual or suspicious items or conditions are strictly for the information of the Client. A geological fault study was also beyond the scope of our services associated with this geotechnical engineering study.

3 FIELD PROGRAM

3.1 Soil Borings

TWE conducted an exploration of subsurface soil and groundwater conditions at the project site on July 7, 2020 by drilling and sampling 2 soil borings to depths of 50-ft below grade. The soil boring locations are presented on TWE Drawing No. 20.53.036-1 and 20.53.036-2 in Appendix B of this report. Drilling and sampling of the soil borings were performed using truck-mounted drilling equipment. Our field personnel coordinated the field activities and logged the boreholes. The boring locations were staked at the site by TWE personnel. The latitude and longitude for each boring location were determined by TWE using a hand operated GPS unit and are presented on the boring logs. The borings were backfilled with soil cuttings and bentonite chips.

3.2 Drilling Methods

Field operations were performed in general accordance with the *Standard Practice for Soil Investigation and Sampling by Auger Borings [American Society for Testing and Materials (ASTM) D 1452]*. Typically, borings are dry-augered using a flight auger to advance the boreholes until groundwater is encountered or until the boreholes become unstable and/or collapse. At that point, soil borings are completed using wash-rotary drilling techniques. Samples were obtained at intervals of 3-ft from existing ground surface to a depth of 10-ft and at intervals of 5-ft thereafter until the boring completion depths of 50-ft were reached.

3.3 Soil Sampling

Fine-grained, cohesive soil samples were recovered from the soil borings by hydraulically pushing 3-in diameter, thin-walled Shelby tubes a distance of about 24-in. The field sampling procedures were conducted in general accordance with the *Standard Practice for Thin-Walled Tube Sampling of Soils (ASTM D 1587)*. Our geotechnician visually classified the recovered soils and obtained field strength measurements using a pocket penetrometer. A factor of 0.67 is typically applied to the penetrometer measurement to estimate the undrained shear strength of the Gulf Coast cohesive soils. The samples were extruded in the field, wrapped in foil, placed in moisture sealed containers and protected from disturbance prior to transport to the laboratory.

Cohesionless and semi-cohesionless samples were collected with the standard penetration test (SPT) sampler driven 18-in by blows from a 140-lb hammer falling 30-in in accordance with the *Standard Test Method for Standard Penetration Test (SPT) and Spilt-Barrel Sampling of Soils (ASTM D 1586)*. The number of blows required to advance the sampler three (3) consecutive 6-in depths are recorded for each corresponding sample on the boring logs. The N-value, in blows per foot, is obtained from SPTs by adding the last two (2) blow count numbers. The compactness of cohesionless and semi-cohesionless samples are inferred from the N-value. The samples obtained from the split-barrel sampler were visually classified, placed in moisture sealed containers and transported to our laboratory.

The recovered soil sample depths with corresponding pocket penetrometer measurements and SPT blowcounts are presented on the boring logs in Appendix C.

3.4 Boring Logs

Our interpretations of general subsurface soil and groundwater conditions at the soil boring locations are included on the boring logs. Our interpretations of the soil types throughout the boring depths and the locations of strata changes were based on visual classifications during field sampling and laboratory testing in accordance with *Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System) (ASTM D 2487)* and *Standard Practice for Description and Identification of Soils (Visual-Manual Procedure) (ASTM D 2488).*

The boring logs include the type and interval depth for each sample along with its corresponding pocket penetrometer measurements and SPT blow counts. The boring logs and a key to terms and symbols used on boring logs are presented in Appendix C.

3.5 Groundwater Measurements

Groundwater level measurements were attempted in the open boreholes during dry-auger drilling. Water level readings were attempted in the open boreholes when groundwater was first encountered and after a ten (10) to fifteen (15) minute time period. The groundwater observations are presented on the boring logs and are summarized in Section 5.5 of this report entitled *"Groundwater Observations."*

4 LABORATORY SERVICES

A laboratory testing program was conducted on selected samples to assist in classification and evaluation of the physical and engineering properties of the soils encountered in the project borings. Laboratory tests were performed in general accordance with *ASTM International* standards to measure physical and engineering properties of the recovered samples. The types and brief descriptions of the laboratory tests performed are presented in Table 4-1 below.

Table 4-1: Laboratory Testing Program

Test Description	Test Method
Amount of Material in Soils Finer than No. 200 Sieve	ASTM D 1140
Water (Moisture) Content of Soil	ASTM D 2216
Liquid Limit, Plastic Limit and Plasticity Index of Soils	ASTM D 4318
Density (Unit Weight) of Soil Specimens	ASTM D 2937
Unconsolidated-Undrained Triaxial Compressive Strength (UU)	ASTM D 2850
Consolidated-Undrained Triaxial Compression w/ Pore Water Pressure	ASTM D 4767

Amount of Materials in Soils Finer than No. 200 (75-µm) Sieve (ASTM D 1140)

This test method determines the amount of materials in soils finer than the No. 200 (75-µm) sieve by washing. The loss in weight resulting from the wash treatment is presented as a percentage of the original sample and is reported as the percentage of silt and clay particles in the sample.

Water (Moisture) Content of Soil by Mass (ASTM D 2216)

This test method determines water (moisture) content by mass of soil where the reduction in mass by drying is due to loss of water. The water (moisture) content of soil, expressed as a percentage, is defined as the ratio of the mass of water to the mass of soil solids. Moisture content may provide an indication of cohesive soil shear strength and compressibility when compared to Atterberg Limits.

Liquid Limit, Plastic Limit and Plasticity Index of Soils (ASTM D 4318)

This test method determines the liquid limit, plastic limit and the plasticity index of soils. These tests, also known as Atterberg limits, are used from soil classification purposes. They also provide an indication of the volume change potential of a soil when considered in conjunction with the natural moisture content. The liquid limit and plastic limit establish boundaries of consistency for plastic soils. The plasticity index is the difference between the liquid limit and plastic limit.

Unconsolidated Undrained Triaxial Compressive Strength of Cohesive Soil (ASTM D 2850)

This test method determines the compressive strength of cohesive soil when subjected to strain-controlled axial load as the sample is subjected to a confining stress. The confining stress generally is that stress the sample is subjected to in the in-situ state. The test method provides an approximate value of shear strength of cohesive materials in terms of confined unconsolidated undrained (UU) stresses.

Consolidated-Undrained Triaxial Compression w/ Pore Water Pressure (ASTM D 4767)

This test method determines the strength and stress-strain relationships of a cylindrical specimen of an intact saturated cohesive soil. Samples are isotropically consolidated and sheared in compression without drainage at a constant rate of axial deformation (strain controlled).

Standard geotechnical laboratory test results and soil properties encountered in the project borings are presented on the logs of borings in Appendix C of this report. Results of consolidated-undrained triaxial compression tests performed on the selected cohesive soil samples obtained for this study are included in Appendix E.

5 SITE AND SUBSURFACE CONDITIONS

5.1 General

Our interpretations of soil and groundwater conditions within the project site are based on information obtained at the soil boring locations only. This information has been used as the basis for our preliminary conclusions and recommendations included in this report. Due to the widely spaced locations of the soil borings, subsurface conditions may vary significantly at areas not explored by the soil borings. Significant variations at areas not explored by the soil borings will require reassessment of our recommendations.

5.2 Site Description and Surface Conditions

The general site location for the project is shown on the attached Option 3 Drainage Map provided by LAN as shown in Appendix A of this report. The North Beach Navigable Canal spans from its southern end near Breakwater Ave. north to its northern end near Beach Ave. The current configuration has the canal approximately 1.25 miles in length with various widths and an average depth of ten feet. The site was occupied by existing city streets and commercial, residential, and public buildings at the time of the field exploration. Areas where soil borings were conducted was covered by natural vegetation and gravel.

5.3 Subsurface Conditions

Subsurface soil conditions encountered in the project boring B-1 consisted of stiff sandy silty clay (CL-ML) to a depth of 2.5-ft which was underlain by loose silty sand (SM) to a depth of 6.5-ft. Below this depth, loose to medium dense poorly graded sand with silt (SP-SM) was then encountered to a depth of 23-ft. The sands were underlain by very soft to firm lean clay with sand (CL) and fat clay (CH) that extended to a depth of approximately 43-ft. Very loose to loose clayey sand (SC) were then encountered and continued to the termination depth of 50-ft.

The initial soil stratum encountered in soil boring B-2 consisted of very stiff sandy silty clay (CL-ML) that extended to a depth of 2.5-ft below the natural ground surface. Below this stratum, loose to very loose intermittent layers of poorly graded sand with silt (SP-SM), poorly graded sand (SP), and clayey sand (SC) were encountered to approximately 23.5-ft. below existing grade. Very soft cohesive soils consisting of lean clay (CL) and fat clay (CH) were then encountered to the termination depth of B-2 at 50-ft below existing grade. The boring logs presenting detailed soil layer classifications and tabulated field and laboratory test results are provided in Appendix C of this report.

5.4 Subsurface Soil Properties

In-situ moisture contents of selected cohesive clay samples ranged from 6% to 87%. Results of Atterberg Limits tests on selected clay samples indicated liquid limits (LL) ranging from 22 to 103 with plasticity indices (PI) ranging from 1 to 74. The amount of materials finer than the No. 200 sieve on the selected samples ranged from 60% to 99%. In-situ moisture contents of selected semi-cohesionless and cohesionless sand samples ranged from 18% to 38%. The amount of materials finer than the No. 200 sieve on the selected samples tested for grain size distribution ranged from

3% to 38%. Atterberg Limits testing indicated liquid limits of non-plastic to 30 and plasticity indices of non-plastic to 11 for selected semi-cohesionless and cohesionless sand samples.

Undrained shear strengths derived from field pocket penetrometer readings ranged from 0.25-tsf to 0.75-tsf. Undrained shear strengths derived from laboratory unconsolidated-undrained triaxial shear (UU) strength testing ranged from 0.65-tsf to 0.94-tsf with corresponding total unit weights of 62-pcf to 98-pcf. Shear strength of cohesive soils inferred from SPT blow counts generally were similar. Drained shear strengths indicated by laboratory consolidated-undrained triaxial shear (CU) strength testing are presented on the Triaxial Shear Test Reports in Appendix E.

Tabulated laboratory test results at the recovered sample depths are presented on the boring logs in Appendix C.

5.5 Groundwater Observations

Groundwater observations show groundwater was encountered at both soil boring locations. At soil boring B-1, groundwater was encountered at a depth of about 4.5-ft during dry-auger drilling and, after a 15-minute waiting period, the groundwater level was at a depth of about 3.8-ft. At soil boring B-2, groundwater was encountered at a depth of 4.0-ft during dry-auger drilling and, after a 15-minute waiting period, the groundwater level was at a depth of about 4.8-ft.

Groundwater levels would be expected to fluctuate with climatic, seasonal, and tidal variations and should be verified before construction. If accurate determination of the static groundwater level is desired, more permanent standpipe piezometers should be used. Installation of more permanent piezometers to evaluate the long-term groundwater condition was not included within the current scope of services.

5.6 Soil Shrink/Swell Potential

The tendency for a soil to shrink and swell with change in moisture content is a function of clay content and type which are generally reflected in soil consistency as defined by the Atterberg Limits. A generalized relationship between shrink/swell potential and the soil plasticity index is shown in Table 5-1 below.

Table 5-1: Relationship Between Plasticity Index and Shrink/Swell Potential

Plasticity Index Range	Shrink/Swell Potential
0 – 15	Low
15 – 25	Medium
25 – 35	High
> 35	Very High

The amount of expansion that will actually occur with increase in moisture content is inversely related to the overburden pressure. Therefore, the larger the overburden pressure, the smaller the amount of expansion. Near-surface soils are thus susceptible to shrink/swell behavior because they experience low amounts of overburden. Based on the soil boring data, the shallow clay soils at this site have low potential for shrink/swell movements.

5.7 Soil Erosion Susceptibility

The cohesionless granular soils (sands, silty sands, clayey sands) which were encountered above depths of 23-ft to 28-ft in the borings for this study are primarily fine-grained sands, some with abundant fine seashell fragments. These materials will be prone to erosion by becoming part of the water column when subjected to wave action as well as large water velocities below the water surface due to turbulent flow (eddies, jets, etc.). As a result, our analyses of sheet pile walls were based on compete erosion of the sands to a depth of 10-ft below the top of the walls. Protection methods against erosion could consider installation of hardscape (cast-in-place concrete or precast concrete reticulated block) at the intersection of the wall and sand along the inner face of the wall.

6 PRELIMINARY SHEET PILE RETENTION SYSTEM

We understand that the North Beach Navigable Canal will include construction of sheet pile bulkhead walls (anchored or unanchored) on either side of the canal. Furthermore, it is our understanding that steel, concrete, and vinyl sheet piles are being considered for the walls. As part of design, we performed analyses to evaluate the internal (rotational) stability of the sheet pile walls. Minimum allowable pile embedment depths were determined that would result in acceptable factors of safety against internal (rotational) stability. The analyses also provided maximum bending moments and maximum scaled deflections. Global (deep-seated) stability analyses of the walls and their corresponding design soil profiles were performed to verify the embedment depths for the design soil profiles would satisfy acceptable factors of safety against failure.

Based on the cross-sections provided by LAN, presented in Appendix A, the top of sheet pile will be approximately -1.5-ft from the top of the concrete walkway located behind the sheet wall. We understand that the water level in front of the sheet pile wall is expected to be -3.5-ft on average from the top of the concrete walkway/bulkhead. In the case of a catastrophic event occurring that would cause complete erosion of the soil along the sheet pile wall, the depth of soil in front of the sheet pile wall of -10-ft below the water surface was used for the analyses. This replicates a worst case scenario and will determine the factors of safety of the walls against global (deep-seated) stability and internal (rotational) stability failure.

6.1 Design Soil Parameters

Soil design parameters were developed for both soil borings due to the distance between the two locations and variations in the properties of subsurface soil stratigraphy. Undrained design soil parameters for short-term (end of construction) analyses were developed based on the field and laboratory undrained shear strength measurements and based on our experience. Long-term (drained) design soil parameters were developed based on consolidated-undrained (C-U) triaxial shear tests performed, published correlations with soil index properties and based on our experience. The design soil parameters are presented in Table 6-1 and Table 6-2 on the following page.

Table 6-1 Recommended Geotechnical Soil Design Parameters Soil Boring B-1

Soil Layer	Soil Description	Depth Range (ft)	γ (pcf)	γ' (pcf)	Undrained Parameters (Short-Term)									Drained Parameters (Long-Term)						
					c (psf)	φ (°)	δ (°)	a (psf)	K_a	K_p	K_o	e50	k (pci)	c' (psf)	φ' (°)	δ (°)	a (psf)	K_a	K_p	K_o
1	Stiff Clay	0 - 2.5	120	58	1,000	0	0	450	1.00	1.00	1.00	0.007	200	0	24	12	0	0.42	2.37	0.59
2	Loose Sand	2.5 - 6.5	110	48	0	27	14	0	0.38	2.66	0.55	-	20	0	27	14	0	0.38	2.66	0.55
3	Medium Dense Sand	6.5 - 18	115	53	0	27	14	0	0.38	2.66	0.55	-	60	0	27	14	0	0.38	2.66	0.55
4	Loose Sand	18 - 23	110	48	0	27	14	0	0.38	2.66	0.55	-	20	0	27	14	0	0.38	2.66	0.55
5	Very Soft Clay	23 - 33	105	43	200	0	0	150	1.00	1.00	1.00	0.02	30	0	18	9	0	0.53	1.89	0.69
6	Firm Clay	33 - 38	117	55	650	0	0	350	1.00	1.00	1.00	0.02	100	0	27	14	0	0.38	2.66	0.55
7	Soft Clay	38 - 43	110	48	400	0	0	250	1.00	1.00	1.00	0.02	30	0	18	9	0	0.53	1.89	0.69
8	Loose Sand	43 - 48	110	48	0	27	14	0	0.38	2.66	0.55	-	20	0	27	14	0	0.38	2.66	0.55
9	Very Loose Sand	48 - 50	105	43	0	27	14	0	0.38	2.66	0.55	-	20	0	27	14	0	0.38	2.66	0.55

Legend:
γ = Total Unit Weight
γ' = Submerged Unit Weight
c = Cohesion
φ = Friction Angle
δ = Angle of Wall Friction (Steel Sheet Pile)
a = Adhesion
K_a = Active Earth Pressure Coefficient
K_p = Passive Earth Pressure Coefficient
K_o = At-Rest Earth Pressure Coefficient
k = Soil Modulus Parameter (Undrained Conditions Only)
e50 = Soil Strain Parameter for Clay Soils (50% undrained strength)

Notes:
1) Approximate depths are from existing ground surface at the soil boring locations.
2) Plasticity index (PI) was used to provide a correlation of effective friction angle for clay soils.

Table 6-2 Recommended Geotechnical Soil Design Parameters Soil Boring B-2

Soil Layer	Soil Description	Depth Range (ft)	γ (pcf)	γ' (pcf)	Undrained Parameters (Short-Term)									Drained Parameters (Long-Term)						
					c (psf)	φ (°)	δ (°)	a (psf)	K_a	K_p	K_o	e50	k (pci)	c' (psf)	φ' (°)	δ (°)	a (psf)	K_a	K_p	K_o
1	Very Stiff Clay	0 - 2.5	120	58	2,800	0	0	650	1.00	1.00	1.00	0.007	200	0	27	14	0	0.38	2.66	0.55
2	Very Loose Sand	2.5 - 5	105	43	0	27	14	0	0.38	2.66	0.55	-	20	0	27	14	0	0.38	2.66	0.55
3	Loose Sand	5 - 13.5	110	48	0	27	14	0	0.38	2.66	0.55	-	20	0	27	14	0	0.38	2.66	0.55
4	Very Loose to Loose Sand	13.5 - 28.5	105	43	0	27	14	0	0.38	2.66	0.55	-	20	0	27	14	0	0.38	2.66	0.55
5	Very Soft Clay	28.5 - 38	105	43	250	0	0	150	1.00	1.00	1.00	0.02	30	0	26	13	0	0.39	2.56	0.56
6	Very Soft Clay	38 - 43	105	43	940	0	0	425	1.00	1.00	1.00	0.01	200	100	31	16	0	0.32	3.12	0.48
7	Very Soft Clay	43 - 47	105	43	940	0	0	425	1.00	1.00	1.00	0.01	200	100	31	16	0	0.32	3.12	0.48
8	Firm Clay	47 - 50	115	53	940	0	0	425	1.00	1.00	1.00	0.01	200	100	31	16	0	0.32	3.12	0.48

Legend:
γ = Total Unit Weight
γ' = Submerged Unit Weight
c = Cohesion
φ = Friction Angle
δ = Angle of Wall Friction (Steel Sheet Pile)
a = Adhesion
K_a = Active Earth Pressure Coefficient
K_p = Passive Earth Pressure Coefficient
K_o = At-Rest Earth Pressure Coefficient
k = Soil Modulus Parameter (Undrained Conditions Only)
e50 = Soil Strain Parameter for Clay Soils (50% undrained strength)

Notes:
1) Approximate depths are from existing ground surface at the soil boring locations.
2) Plasticity index (PI) was used to provide a correlation of effective friction angle for clay soils.

6.2 Internal (Rotational) Stability Analyses

The objectives of our analyses were to determine the minimum required sheet pile lengths (design embedment), maximum bending moment in the sheet pile sections and the loading that the anchoring system will experience per foot section of the sheet pile wall. Soil design parameters were determined by analyzing the soil stratigraphy of borings that were conducted at boring locations presented on TWE Drawing No. 20.53.036-1 and 20.53.036-2 in Appendix B of this report.

We analyzed the proposed sheet pile wall sections for internal (rotational) stability using the computer program CWALSHT developed by the U.S. Army Corps of Engineers (USACE) at the Engineering Research & Development Center in Vicksburg, Mississippi. CWALSHT uses classical methods of sheet pile analysis based on limit equilibrium methods in accordance with USACE EM 1110-2-2503 (Design of Sheet Pile Wall). The results of the rotational stability analyses for each location are located in Table 6-3 and Table 6-4 below. Factors of safety of 1.0 and 1.5 were used for active and passive pressure, respectively.

Table 6-3 : Minimum Sheet Pile Wall Design Parameters Soil Boring B-1

Minimum Design Parameter	Anchored	Cantilever
Embedment Depth	12-ft	23-ft
Overall Height	22-ft	33-ft
Maximum Bending Moment	60 kip-in	325 kip-in
Maximum Scaled Deflection	3.0370 x 10^8 lb-in^3	1.9079 x 10^10 lb-in^3
Anchor Load	1,650 lb/ft	---

Table 6-4 : Minimum Sheet Pile Wall Design Parameters Soil Boring B-2

Minimum Design Parameter	Anchored	Cantilever
Embedment Depth	13-ft	22-ft
Overall Height	23-ft	32-ft
Maximum Bending Moment	85 kip-in	410 kip-in
Maximum Scaled Deflection	3.3016 x 10^8 lb-in^3	1.9300 x 10^10 lb-in^3
Anchor Load	1,680 lb/ft	---

The following formulas are used to find the minimum required section modulus and top of wall deflection for steel, vinyl, and concrete sheet pile sections:

$S = \frac{M_m}{\sigma_a}$

$\Delta_{TOW} = \frac{\Delta_{ms}}{EI}$

Where;

$S = Minimum\ Required\ Section\ Modulus\ for\ a\ Sheet\ Pile\ Section\ in\ in^3$

$M_m = Maximum\ Bending\ Momement\ in\ kip - in$

$\sigma_a = Allowable\ Yield\ Stress\ of\ \ Material\ in\ kip/in^2\ *$

$\Delta_{TOW} = Top\ of\ Wall\ Deflection\ in\ inches$

$\Delta_{ms} = Maximum\ Scaled\ Deflection\ in\ lb - in^3$

$E = Modulus\ of\ Elasticity\ for\ Material\ in\ lbs/in^2$

$I = Moment\ of\ Inertia\ for\ Sheet\ Pile\ Section\ in\ in^4$

* It should be noted that the allowable stress for steel, concrete and vinyl require a factor of reduction before application these reductions are as follows;

$\sigma_a\ of\ concrete = 0.6 * f'_c$ (minimum f'_c of 5000 psi)
$\sigma_a\ of\ vinyl\ and\ steel = 0.5 * f_y$

6.3 Sheet Pile Anchor System

We performed analyses for passive resistance and active resistance to determine the net passive resistance acting on the sheet pile wall anchoring system for both boring locations. The passive resistance and active resistance were calculated using the long-term (drained) design soil parameters for each soil boring location. The design soil parameters can be found in Table 6-1 and Table 6-2 in section 6.1, "*Design Soil Parameters"* above. Our analysis assumed that the connection of the anchoring system to the sheet pile wall will be located at the sheet pile cap (top of sheet pile wall). The results of the analysis are provided in Table 6-5 and Table 6-6 below.

Table 6-5 Net Passive Resistance Soil Boring B-1

Kp = 2.66, Ka = 0.375		Groundwater level = 4 ft below existing grade			
Depth Below Grade (ft)	Unit Weight of Soil (PCF)	Effective Overburden Pressure (PSF)	Factored Passive Resistance (PSF)	Active Resistance (PSF)	Net Passive Resistance (PSF)
0	120	0	0	0	0
2.5	120	300	399	113	287
6.5	110	584	777	219	558
15	115	1031	1371	387	985

Note: Net passive resistance is based on passive resistance factored by 2 against failure.

Table 6-6 Net Passive Resistance Soil Boring B-2					
Kp = 2.66, Ka = 0.375		Groundwater level = 4 ft below existing grade			
Depth Below Grade (ft)	Unit Weight of Soil (PCF)	Effective Overburden Pressure (PSF)	Factored Passive Resistance (PSF)	Active Resistance (PSF)	Net Passive Resistance (PSF)
0	120	0	0	0	0
2.5	120	300	399	113	287
5	105	500	665	188	478
10	110	738	982	277	705
15	105	969	1288	363	925
Note: Net passive resistance is based on passive resistance factored by 2 against failure.					

Anchor Location

In consideration of sheet pile anchor placement relative to the sheet pile wall a distance sufficient to utilize the full factored net passive resistance was calculated. Based on our analysis of the active earth pressure wedge behind the sheet pile wall and the passive pressure wedge in front of the anchor, a minimum distance of thirty (30) linear feet from the top of the sheet pile wall should be maintained for all anchoring systems. A minimum depth of embedment of 2.5-ft should be observed for all anchors used for this sheet pile wall; however, the embedment depth should provide sufficient overburden pressure and net passive resistance to resist uplift pressure and the anchor loads as presented in Table 6-3 and Table 6-4 of section 6.2 "*Internal (Rotational) Stability"*. Preferably, the top of the anchoring system should be above the observed groundwater levels to avoid excessive uplift pressure.

Anchor System Installation

Anchors meeting or exceeding the stated criteria in section 6.2, "*Internal (Rotational) Stability),* and in the above section may consist of either concrete dead man, steel member, or sheet pile wall attached to the pile with tie rods, or tiebacks with grouted anchors (soil nails), or helical piles, or various configurations of steel or concrete piles.

Anchor forces, soil pressures and water loads are affected by the method of construction and construction practices. The sequence of tightening tie rods should be specified to prevent overstresses in isolated sections of the sheet pile wall. Anchors and tie rods should be placed and tightened in a uniform manner so that no overstresses may occur. Backfilling above the anchor elevation should be carefully controlled to prevent bending of the tie rods. The backfill material should be controlled, and the thickness of compacted layers should be limited to ensure proper compaction and drainage of the back fill material.

6.4 Global Stability Analyses

We performed global stability analyses of the sheet pile wall section using the computer program Slide 7.0 (Rocscience 2018) to determine the adequacy of the sheet pile embedment obtained from the internal (rotational) stability criteria. Slide is a 2D limit equilibrium slope stability program for evaluating the safety factor, or probability of failure, of circular or non-circular failure surfaces in soil or rock slopes. Slide analyzes the stability of slip surfaces using vertical slice limit equilibrium methods.

Global stability analysis was performed using Spencer's (1967) method for short-term conditions, using undrained (total stress) parameters, and long-term conditions using drained (effective stress) parameters. Spencer's (1967) method satisfies both force and moment equilibriums. The results of our global stability evaluations are presented in Appendix D of this report. According to the guidance provided in U.S. Army Corps of Engineers (USACE) Engineer Manual for Slope Stability (EM 1110-2-1902), the minimum required factor of safety considered appropriate for short-term (undrained) and long-term (drained) stability analysis are 1.3 and 1.5, respectively.

Based on the results of our analyses, the global stability factor of safety for short-term and long-term conditions, for the sheet pile embedment depth and design soil parameters considered, exceed the minimum required factors of safety.

6.5 Sheet Pile Installation/Drivability

The most common methods of installing sheet pile walls include driving, jetting and trenching. The type of sheet piling will often govern the method of installation. There are several types of driving hammers that are available for sheet pile installation and can be broken down into two separate categories; impact and vibratory hammers. Vibratory hammers are generally the faster method of pile installation depending on the soil stratigraphy however if a penetration rate of 1-ft per minute or less is experienced the vibratory hammer should be discontinued and an impact hammer implemented. The prolonged use of a vibratory hammer in hard soil conditions can cause damage to pile interlocks. The selection of the type or size of the hammer is based on the soil in which the pile is driven, size of pile, and depth of penetration. When impact hammers are used the hammer should be appropriately sized and a protective cap utilized to prevent excessive damage to the pile. To ensure that piles are placed and driven to the correct alignment, a guide structure or templates should be used. At least two templates should be used in driving each pile or pair of piles.

Jetting is usually used to penetrate strata of dense cohesionless soils. Jetting should be performed on both sides of the piling simultaneously and discontinued during the last 5-ft to 10-ft of pile penetration. Adequate steps must be taken to ensure the control, treatment, and disposal of runoff water. Sheet piling should not be driven more than 1/8 inch per foot out of plumb either in the plane of the wall or perpendicular to the plane of the wall. Due to soil stratigraphy and intended application of the sheet pile wall trenching is not a recommended method of installation for the sheet pile sections. Since the anticipated subsurface soil conditions largely include very loose to loose sands and very soft to soft clays, jetting should not be required for this site.

Additionally, we do not expect unusual difficult sheet pile placement for the subsurface conditions encountered in the borings for this project. Although we expect that vibratory placement of sheet piles can be accomplished for these subsurface conditions, we recommend a sheet pile contractor be contacted to confirm this conclusion. If additional detailed information regarding drivability for steel sheet piling, prestressed concrete sheet piling, or vinyl sheet piling is desired, please refer to *U.S Army Corps of Engineers, EM1110-2-2504, March 31, 1994, Design of Sheet Pile Walls* publication.

7 PRELIMINARY PAVEMENT RECOMMENDATIONS

7.1 Discussion

Preliminary pavement recommendations for improvement of existing roadways at North Beach are provided below. Possible other improvements may include replacement of existing utilities and addition of new traffic control lights.

7.2 Design Review

The methods used in our pavement analysis can be found in the *AASHTO, Guide for Design of Pavement Structures*. Traffic conditions provided the City of Corpus Christi for local non-residential traffic were used for design purposes using a 30-year design life. An annual traffic growth rate of 0.2% was used in accordance with city requirements.

7.2.1 Flexible Pavement Design

The primary design requirements needed for flexible pavement design according to the Pavement Design Guide include the following:

- Material Layer Coefficient;
- Soil Resilient Modulus, psi;
- Serviceability Indices;
- Drainage Coefficient;
- Overall Standard Deviation;
- Reliability, %; and,
- Design Traffic, 18-kip Equivalent Single Axle Load (ESAL)
- Design Average Daily Traffic (ADT)
- Design % Truck

The design values used for our analyses are presented in Table 7-1 on the following page for 30-yr design life.

Table 7-1 Flexible Pavement Design Values for 30 year Design		
Description		**Value**
Design ADT and % Truck(1)	Average Daily Traffic (ADT)	n/a
	% Truck	n/a
Material Coefficients	Hot Mix Asphalt Concrete (HMAC), Type D	0.44
	HMAC, Type B	0.40
	Crushed Limestone (Type A, Grade 2 or better) [CLB]	0.14
	Crushed Concrete (CC)	0.12
	Lime Stabilized Subgrade (LSS)	0.08
	Compacted Subgrade	0.035
Serviceability Indices	Initial	4.2
	Terminal	2.5
Soil Resilient Modulus		3,100-psi
Drainage Coefficient		1.0
Overall Standard Deviation		0.45
Reliability		80%
Design Traffic, 18-kip Equivalent Single Axle Load (ESAL)		1,000,000
Structural Number Required		4.52

(1) The Average Daily Traffic and the % truck were provided and determined by using the traffic data parameters regarding local non-residential traffic provided by City of Corpus Christi.

Table 7-2 Recommended Minimum Typical Flexible Pavement Thicknesses for 30 Year Design						
Pavement Option	**HMAC, Type D**	**HMAC, Type B**	**CLB**	**CC**	**CS**	**SN**
A	3.0-in	3.0-in	8.0-in	---	12.0-in	4.60
B	2.5-in	3.0-in	8.0-in*	---	8.0-in	4.76
C	3.0-in	3.0-in	---	9.0-in	12.0-in	4.56
D	2.5-in	3.0-in	---	8.0-in*	8.0-in	4.57

(*) A layer of geogrid (Tensar TX-5 or equivalent) installed at the bottom of the crushed limestone or crushed concrete base.

HMAC = Hot Mix Asphalt Concrete
CLB = Crushed Limestone Base (Type A, Grade 1-2)
CC = Crushed Concrete (Type D, Grade 1-2)
CS = Compacted Subgrade
SN = Structural Number

It should be noted that the upper 2.5-ft of soil at both boring locations exhibit low plasticity properties similar to a lime stabilized subgrade and can be treated as such. Thus, low plasticity subgrade materials were used for the pavement analyses. Sufficient monitoring and testing should be done to observe that these properties are homogenous throughout the segments of roadway that may be reconstructed.

Existing roadways consisting of one of the above sections or similar may meet the City of Corpus Christi criteria for a local non-residential roadway with a 30-year lifespan at 80% reliability. Field explorations such as coring can be implemented in order to determine the viability of the North Beach roadway segments. As it stands now there is little to no drainage systems in place along the two referenced roadways and consideration for flood mitigation and drainage should be taken.

7.2.2 Rigid Pavement Design

The primary design requirements needed for rigid pavement design according to the AASHTO Guide include the following:

- 28-day Concrete Modulus of Rupture, psi;
- 28-day Concrete Elastic Modulus, psi;
- Effective Modulus of Subgrade Reaction, pci (k-value);
- Serviceability Indices;
- Load Transfer Coefficient;
- Drainage Coefficient;
- Overall Standard Deviation;
- Reliability, %; and,
- Design Traffic, 18-kip Equivalent Single Axle Load (ESAL)

The design values used for our analyses are presented in Table 7-3 below for 30-yr design life.

Table 7-3
Rigid Pavement Design Values for 30 Year Design

Description		Value
28-day Concrete Modulus of Rupture		620-psi
28-day Concrete Elastic Modulus		3,860,000-psi
Effective Modulus of Subgrade Reaction		110-pci
Serviceability Indices	Initial	4.5
	Terminal	2.5
Load Transfer Coefficient	Continuously Reinforced	2.6
	Plain	3.2
Drainage Coefficient		1.0
Overall Standard Deviation		0.39
Reliability		80%
Design Traffic, 18-kip Equivalent Single Axle Load (ESAL)		1,000,000

Table 7-4 below provides the recommended minimum typical rigid pavement sections derived from our analysis using the AASHTO Pavement Design Guide.

Table 7-4 Recommended Minimum Typical Rigid Pavement Thicknesses for 30 Year Design				
Pavement Option	**PCC**	**BB**	**CLB**	**CS**
Continuously Reinforced	6.0-in	1.0-in	7.0-in	12.0-in
Plain	7.0-in	1.0-in	7.0-in	12.0-in.

PCC = Portland Cement Concrete
BB = Bond Breaker (HMAC, Type B or D)
CLB = Crushed Limestone Flexible Base (TxDOT, Item 247, Type A, Grade 1-2)
CS = Compacted Subgrade

7.3 Pavement Section Material

Hot Mix Asphalt Concrete (HMAC)

HMAC should conform to Item 340, "Dense-Graded Hot-Mix Asphalt" of the Texas Department of Transportation (TxDOT) 2004 Standard Specifications for Construction and Maintenance of Highways, Streets and Bridges. The HMAC should provide a minimum tensile strength (dry) of 85 to 200 psi when tested in accordance with TxDOT Test Method Tex-226-F, and should be compacted at 92% to 96% of the theoretical density as determined from the asphaltic mixture design prepared in accordance with TxDOT Test Method Tex-207-F "Determining Density of Compacted Bituminous Mixtures".

Portland Cement Concrete (PCC)

PCC should be provided in accordance with TxDOT Item 421 "Hydraulic Cement Concrete", 2014. Concrete should be designed to meet a minimum average flexural strength (modulus of rupture) of at least 620-psi at 28-days or a minimum average compressive strength of 4,500-psi at 28-days. Reinforcing steel consisting of deformed steel rebar should be used in accordance with TxDOT Item 440 "Reinforcing Steel."

The first few loads of concrete should be checked for slump, air and temperature on start-up production days to check for concrete conformance and consistency. Concrete should be sampled and strength test specimens [two (2) specimens per test] prepared on the initial day of production and for each 400-yd^2 or fraction thereof of concrete pavement thereafter. At least one (1) set of strength test specimens should be prepared for each production day. Slump, air and temperature

tests should be performed each time strength test specimens are made. Concrete temperature should also be monitored to ensure that concrete is consistently within the temperature requirements.

Crushed Limestone Base (CLB)

CLB should conform to City of Corpus Christi Standard Construction Specification (COCC) Section 025223 "Flexible Base" and should be moisture conditioned to -2% to +2% of the optimum moistures and compacted to at least 98 percent of the maximum dry densities determined by Modified Proctor (ASTM D 1557) and Standard Proctor (ASTM D 698) for flexible pavement sections and rigid pavement sections, respectively.

Crushed Concrete (CC)

CC should conform to TxDOT Item 247, Type D, Grade 1-2 and should be compacted in the same manner to CLB and can be substituted at the same thickness as CLB.

Compacted Subgrade (CS)

After completion of necessary stripping and clearing, the exposed soil subgrade should be carefully evaluated by probing and testing. Any unsuitable material (shell, gravel, and organic material, wet, soft or loose soil) still in place should be removed. The exposed soil subgrade should be further evaluated by proofrolling with a heavy pneumatic tired roller, loaded dump truck or similar equipment weighing at least 20-tons to ensure that soft or loose material does not exist beneath the exposed soils. Proofrolling procedures should be observed routinely by a qualified representative of TWE. Any undesirable material revealed should be removed and replaced in a controlled manner with soils similar in classification or select fill.

Once final subgrade elevation is achieved and prior to placement of crushed limestone base, or crushed concrete material, the exposed surface of the pavement subgrade soil should be scarified to a depth of 12-in. and compacted in two, 6-in lifts, each to a minimum 95% of the maximum dry density as determined by Standard Proctor (ASTM D 698) at a moisture content within the range of 3% above optimum. Crushed limestone base, or crushed concrete material should be promptly placed on the compacted, tested, and accepted subgrade.

7.4 Pavement Maintenance

Periodic maintenance of the roadway should be performed over the life of the pavement structure. Maintaining the roadway to prevent infiltration of water into the crushed limestone base material and subgrade soils is essential. Allowing water to infiltrate these materials will result in high maintenance costs and premature failures.

8 PRELIMINARY EARTHWORK CONSIDERATIONS

8.1 Site, Subgrade Preparation, and Fill Requirements

Soils for backfilling behind the sheet pile walls and site filling above or below critical structures (sheet pile anchors or streets) should be placed in controlled and compacted lifts per the recommendations below in Section 8.2 of this report.

Areas designated to receive fill at the site should be stripped of all surface vegetation, loose topsoil and major root systems. Any subgrade to receive fill soils, pavements, or flatwork should be proof rolled with at least a 20-ton pneumatic roller, loaded dump truck, or equivalent, to detect weak areas. Such weak areas should be removed and replaced with soils exhibiting similar classification, moisture content, and density as the adjacent in-place soils.

The exposed subgrades to receive fill as well as subsequent fill materials should be compacted as indicated below in Table 8.1.

Table 8-1: Compaction Requirements

Subgrade/Fill Type	Required Compaction Level	Required Moisture Level
Subgrades for General Fill	92%-95% of ASTM D 698	-2% to +3% of optimum
Subgrades for Wall Backfill	90%-92% of ASTM D 698	-2% to +3% of optimum
Subgrades for Select Fill	95%+ of ASTM D 698	-2% to +3% of optimum
General Site Fills	92%-95% of ASTM D 698	-2% to +3% of optimum
Fills behind Sheet Pile Walls	90%-92% of ASTM D 698	-2% to +3% of optimum
Select Fills	95%+ of ASTM D 698	-2% to +3% of optimum

General site fill for this project should consist of a clean clayey sands (SC), silty sands (SM), low plasticity clays (CL), high plasticity clays (CH) or any combination of these materials with a liquid limit of less than 50 and a plasticity index between 20 and 30. The general fill should be placed in thin lifts, not exceeding 8-in. loose measure, and compacted as indicated above in Table 8.1.

Backfill material for placement behind sheet pile walls for this project should consist of a clean poorly graded sands (SP), well graded sands (SW), clayey sands (SC), or silty sands (SM) or any combination of these materials with a liquid limit of less than 30 and a plasticity index between 0 and 15. The backfill materials should be placed in thin lifts, not exceeding 8-in. loose measure, and compacted as indicated above in Table 8.1.

Select fill for this project should consist of a clean low-plasticity sandy clay (CL) or clayey sand (SC) material with a liquid limit of less than 40 and a plasticity index between 7 and 20. The select fill should be placed in thin lifts, not exceeding 8-in. loose measure, and compacted as indicated above in Table 8.1.

Prior to any filling operations, samples of the proposed borrow materials should be obtained for soil classification and laboratory moisture-density testing. The tests will provide a basis for evaluation of fill compaction by in-place density testing. A qualified soil technician should perform sufficient in-place density tests during the earthwork operations to verify that proper levels of compaction are being attained.

8.2 Drainage

The performance of the sheet pile wall, foundation systems, and site pavement/flatwork will not only be dependent upon the quality of construction but also upon the stability of the moisture content of the near surface soils. Therefore, we highly recommend that site drainage be developed so that ponding of surface runoff near structures or pavements/flatwork does not occur. Accumulations of water near structures or pavements/flatwork could cause significant moisture variations in the soils adjacent to the foundations and pavements/flatwork thus increasing the potential for structural distress.

9 LIMITATIONS AND DESIGN REVIEW

9.1 Limitations

This revised report has been prepared for the exclusive use of Lockwood, Andrews and Newnam, Inc. and the project team for specific application to the design of the proposed North Beach Navigable Canal located in Corpus Christi, Texas. Our report has been prepared in accordance with the generally accepted geotechnical engineering practice common to the local area. No other warranty, express or implied, is made.

The analyses and recommendations contained in this report are based on the data obtained from the referenced subsurface explorations within the project site. The soil boring indicates subsurface conditions only at the specific location, time and depth penetrated. The soil borings do not necessarily reflect strata variations that could exist at other locations within the project site. The validity of our recommendations is based in part on assumptions about the stratigraphy made by the Geotechnical Engineer. Such assumptions may be confirmed only during construction of the project. Our recommendations presented in this report must be reevaluated if subsurface conditions during the construction phase are different from those described in this report.

If any changes in the nature, design or location of the project are planned, the conclusions and recommendations contained in this revised report should not be considered valid unless the changes are reviewed, and the conclusions modified or verified in writing by TWE. TWE is not responsible for any claims, damages or liability associated with interpretation or reuse of the subsurface data or engineering analyses without the expressed written authorization of TWE.

9.2 Design Review

Review of the design and construction drawings as well as the specifications should be performed by TWE before release. The review is aimed at determining if the geotechnical design and construction recommendations contained in this revised report have been properly interpreted. Design review is not within the authorized scope of work for this study.

9.3 Construction Monitoring

Construction surveillance is recommended and has been assumed in preparing our recommendations. These field services are required to check for changes in conditions that may result in modifications to our recommendations. The quality of the construction practices will affect performance and should be monitored. TWE would be pleased to provide construction monitoring, testing and inspection services for the project.

APPENDIX A

PROJECT INFORMATION OF LOCKWOOD, ANDREWS AND NEWNAM
NORTH BEACH NAVIGABLE CANAL CROSS SECTIONS

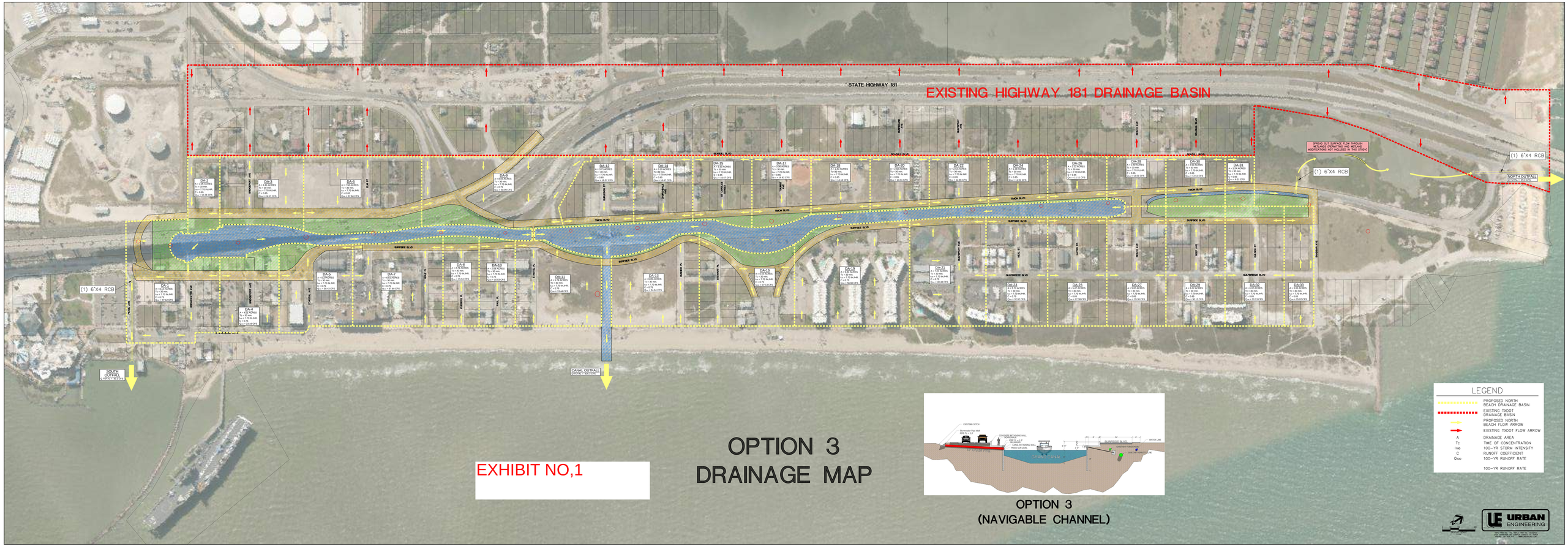

EXHIBIT NO,1

OPTION 3 DRAINAGE MAP

OPTION 3
(NAVIGABLE CHANNEL)

APPENDIX B

SOIL BORING LOCATION PLAN
TWE DRAWING NO. 20.53.036-1 AND 20.53.036-2

VICINITY MAP

SOIL BORING COORDINATES

BORING	DEPTH	LATITUDE	LONGITUDE
B-1	50'	27° 49' 01.6" N	97° 23' 30.9" W

LEGEND

SYMBOL	DESCRIPTION
⊕	BORING LOCATION

Tolunay-Wong Engineers, Inc.

TWE Final Boring Location Plan
North Beach Navigable Canal
Corpus Christi, Texas

Drawn	M.K.M	06/30/20
Checked	J.B	
Approved	D.R.R.	
Scale	N.T.S.	

TWE DRAWING NO. 20.53.036-1

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VICINITY MAP

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SOIL BORING COORDINATES

BORING	DEPTH	LATITUDE	LONGITUDE
B-2	50'	27° 49' 46.8" N	97° 23' 02.7" W

	LEGEND
SYMBOL	DESCRIPTION
⊕	BORING LOCATION

Tolunay-Wong Engineers, Inc.

TWE Final Boring Location Plan
North Beach Navigable Canal
Corpus Christi, Texas

Drawn	M.K.M	06/30/20
Checked	J.B	
Approved	D.R.R.	
Scale	N.T.S.	

TWE DRAWING NO. 20.53.036-2

APPENDIX C

LOGS OF PROJECT BORINGS AND A KEY TO
TERMS AND SYMBOLS USED ON BORING LOGS

LOG OF BORING B-1

PROJECT: North Beach Navigable Canal
Coprus Christi, Texas

CLIENT: Lockwood, Andrews & Newnam, Inc.
Corpus Christi, Texas

COORDINATES: N 27° 49' 01.6"
W 97° 23' 30.9"
SURFACE ELEVATION:
DRILLING METHOD:
Dry Augered: 0.0-ft. to 6.0-ft.
Wash Bored: 6.0-ft. to 50.0-ft.

DEPTH (ft)	SAMPLE TYPE	SYMBOL/USCS	MATERIAL DESCRIPTION	(P) POCKET PEN (tsf) (T) TORVANE (tsf)	STD. PENETRATION TEST (blows/ft)	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (pcf)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	COMPRESSIVE STRENGTH (tsf)	FAILURE STRAIN (%)	CONFINING PRESSURE (psi)	PASSING #200 SIEVE (%)	OTHER TESTS PERFORMED
0			Stiff dark gray SANDY SILTY CLAY (CL-ML) with shells		7/6" 8/6" 5/6"			23	2					
			Loose tan and gray SILTY SAND (SM) with shells		1/6" 1/6" 5/6"	18		NP	NP				29	
5			-color changes to tan with abundant shells		2/6" 4/6" 5/6"									
			Medium dense tan POORLY GRADED SAND with SILT (SP- SM) with abundant shells		3/6" 9/6" 10/6"	19							7	
10			Medium dense gray POORLY GRADED SAND with SILT (SP-SM)		2/6" 5/6" 6/6"	22							9	
15					4/6" 7/6" 9/6"									
20			-becomes loose		3/6" 2/6" 3/6"	24							5	
25			Very soft tan and gray LEAN CLAY with SAND (CL)		W.O.H									
30			-color changes to gray		W.O.H	43		45	30				73	
35			-becomes firm	(P) 0.75		30	90			0.65	7.8	(15)	72	(1)

COMPLETION DEPTH: 50 ft
DATE BORING STARTED: 7-6-20
DATE BORING COMPLETED: 7-6-20
LOGGER: J.Gonzales
PROJECT NO.: 20.53.036

REMARKS: Free water was encountered at a depth of 4.5-ft. below existing grade during dry auger drilling. After a 15 minute waiting period, water was at a depth of 3.8-ft. At completion of drilling and sampling, the open bore hole was back filled with soil cuttings and bentonite pellets.

LOG OF BORING B-1

PROJECT: North Beach Navigable Canal
Coprus Christi, Texas

CLIENT: Lockwood, Andrews & Newnam, Inc.
Corpus Christi, Texas

COORDINATES: N 27° 49' 01.6"
W 97° 23' 30.9"
SURFACE ELEVATION:
DRILLING METHOD:
Dry Augered: 0.0-ft. to 6.0-ft.
Wash Bored: 6.0-ft. to 50.0-ft.

DEPTH (ft)	SAMPLE TYPE	SYMBOL/USCS	MATERIAL DESCRIPTION	(P) POCKET PEN (tsf) (T) TORVANE (tsf)	STD. PENETRATION TEST (blows/ft)	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (pcf)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	COMPRESSIVE STRENGTH (tsf)	FAILURE STRAIN (%)	CONFINING PRESSURE (psi)	PASSING #200 SIEVE (%)	OTHER TESTS PERFORMED
35														
40			Soft gray FAT CLAY (CH) with iron oxide stains	(P) 0.5		14	62	93	68				97	(2)
45			Loose tan and gray CLAYEY SAND (SC) with sand seams		3/6" 3/6" 5/6"									
			-becomes very loose		1/6" 2/6" 1/6"	25							16	
50			Bottom @ 50'											
55														
60														
65														
70														

COMPLETION DEPTH: 50 ft
DATE BORING STARTED: 7-6-20
DATE BORING COMPLETED: 7-6-20
LOGGER: J.Gonzales
PROJECT NO.: 20.53.036

REMARKS: Free water was encountered at a depth of 4.5-ft. below existing grade during dry auger drilling. After a 15 minute waiting period, water was at a depth of 3.8-ft. At completion of drilling and sampling, the open bore hole was back filled with soil cuttings and bentonite pellets.

LOG OF BORING B-2

PROJECT: North Beach Navigable Canal
Coprus Christi, Texas

CLIENT: Lockwood, Andrews & Newnam, Inc.
Corpus Christi, Texas

COORDINATES: N 27° 49' 46.8"
W 97° 23' 02.7"
SURFACE ELEVATION:
DRILLING METHOD:
Dry Augered: 0.0-ft. to 6.0-ft.
Wash Bored: 6.0-ft. to 50.0-ft.

DEPTH (ft)	SAMPLE TYPE	SYMBOL/USCS	MATERIAL DESCRIPTION	(P) POCKET PEN (tsf) (T) TORVANE (tsf)	STD. PENETRATION TEST (blows/ft)	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (pcf)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	COMPRESSIVE STRENGTH (tsf)	FAILURE STRAIN (%)	CONFINING PRESSURE (psi)	PASSING #200 SIEVE (%)	OTHER TESTS PERFORMED
0			Very stiff dark gray SANDY SILTY CLAY (CL-ML) with shells		10/6" 13/6" 10/6"	6		22	1				60	
5			Very loose tan POORLY GRADED SAND with SILT (SP-SM) with shells -becomes loose -with abundant shells		2/6" 1/6" 1/6" 1/6" 3/6" 2/6" 3/6" 3/6" 2/6"	19		NP	NP				10	
10			Loose gray POORLY GRADED SAND (SP)		2/6" 3/6" 3/6"	26							3	
15			Very loose gray CLAYEY SAND (SC) with sand seams		1/6" W.O.H	36		NP	NP				15	
20			-becomes loose		3/6" 4/6" 6/6"									
25			-becomes very loose		W.O.H	34		30	11				38	
30			Very soft gray LEAN CLAY (CL) with silt seams and shell fragments		W.O.H W.O.H									
35														

COMPLETION DEPTH: 50 ft
DATE BORING STARTED: 7-7-2020
DATE BORING COMPLETED: 7-7-2020
LOGGER: J.Gonzales
PROJECT NO.: 20.53.036

REMARKS: Free water was encountred at a depth of 4.0-ft. below existing grade during dry auger drilling. After a 15 minute waiting period, water was at a depth of 4.8-ft. At completion of drilling and sampling, the open bore hole was back filled with soil cuttings and bentonite pellets.

LOG OF BORING B-2

PROJECT: North Beach Navigable Canal
Coprus Christi, Texas

CLIENT: Lockwood, Andrews & Newnam, Inc.
Corpus Christi, Texas

COORDINATES: N 27° 49' 46.8"
W 97° 23' 02.7"
SURFACE ELEVATION:
DRILLING METHOD:
Dry Augered: 0.0-ft. to 6.0-ft.
Wash Bored: 6.0-ft. to 50.0-ft.

DEPTH (ft)	SAMPLE TYPE	SYMBOL/USCS	MATERIAL DESCRIPTION	(P) POCKET PEN (tsf) (T) TORVANE (tsf)	STD. PENETRATION TEST (blows/ft)	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (pcf)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	COMPRESSIVE STRENGTH (tsf)	FAILURE STRAIN (%)	CONFINING PRESSURE (psi)	PASSING #200 SIEVE (%)	OTHER TESTS PERFORMED
35														
				(P) 0.25		87	98	39	21				87	(2)
40														
			Very soft FAT CLAY (CH) with sand seams	(P) 0.5		64		103	74				99	
45														
			-becomes firm with silt seams	(P) 0.5		51	69			0.94	2.9	(21)	93	(1)
50			Bottom @ 50'											
55														
60														
65														
70														

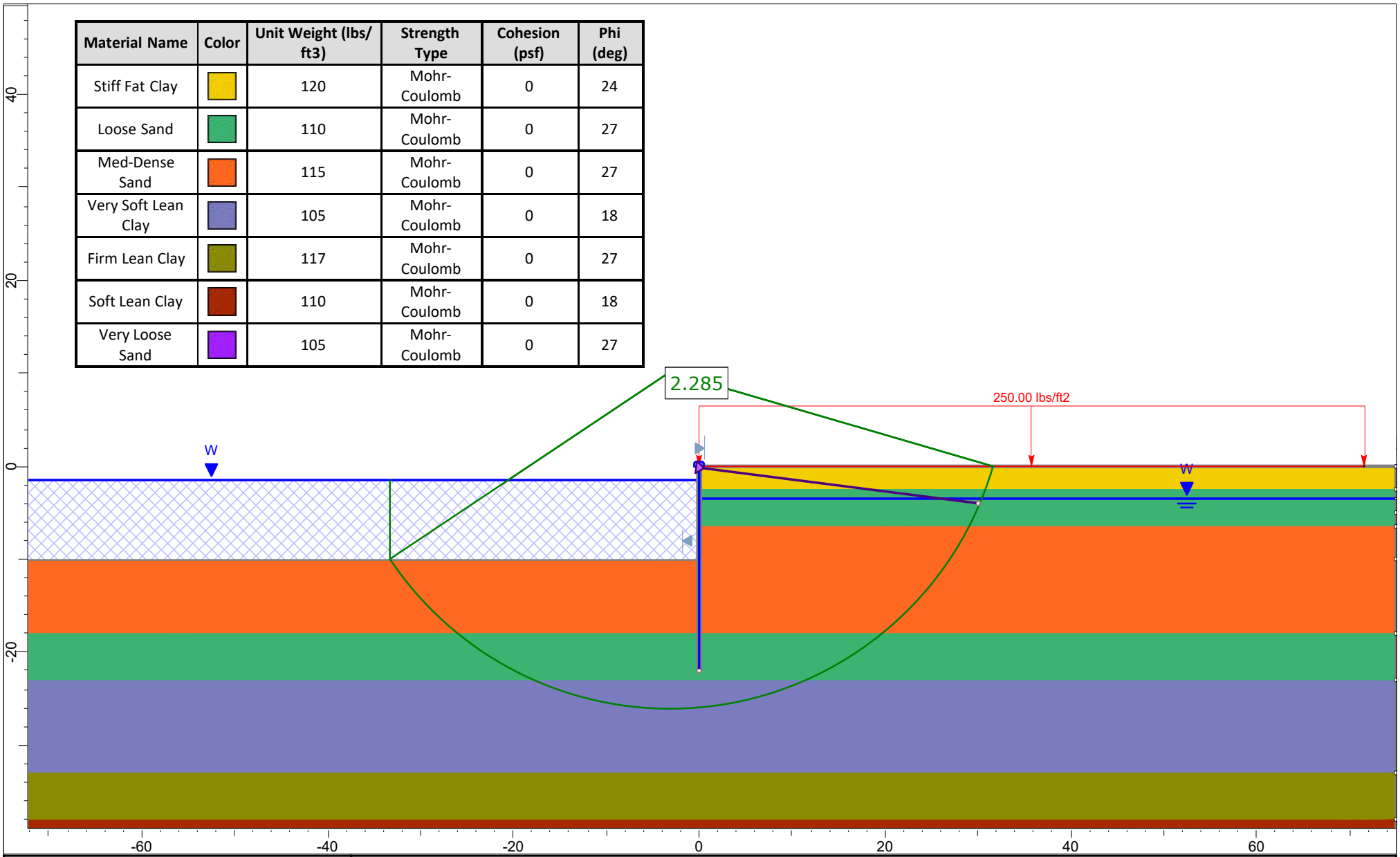
COMPLETION DEPTH: 50 ft
DATE BORING STARTED: 7-7-2020
DATE BORING COMPLETED: 7-7-2020
LOGGER: J.Gonzales
PROJECT NO.: 20.53.036

REMARKS: Free water was encountred at a depth of 4.0-ft. below existing grade during dry auger drilling. After a 15 minute waiting period, water was at a depth of 4.8-ft. At completion of drilling and sampling, the open bore hole was back filled with soil cuttings and bentonite pellets.

APPENDIX D

SHEET PILE WALL GLOBAL STABILITY RESULTS

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Stiff Fat Clay		120	Mohr-Coulomb	0	24
Loose Sand		110	Mohr-Coulomb	0	27
Med-Dense Sand		115	Mohr-Coulomb	0	27
Very Soft Lean Clay		105	Mohr-Coulomb	0	18
Firm Lean Clay		117	Mohr-Coulomb	0	27
Soft Lean Clay		110	Mohr-Coulomb	0	18
Very Loose Sand		105	Mohr-Coulomb	0	27



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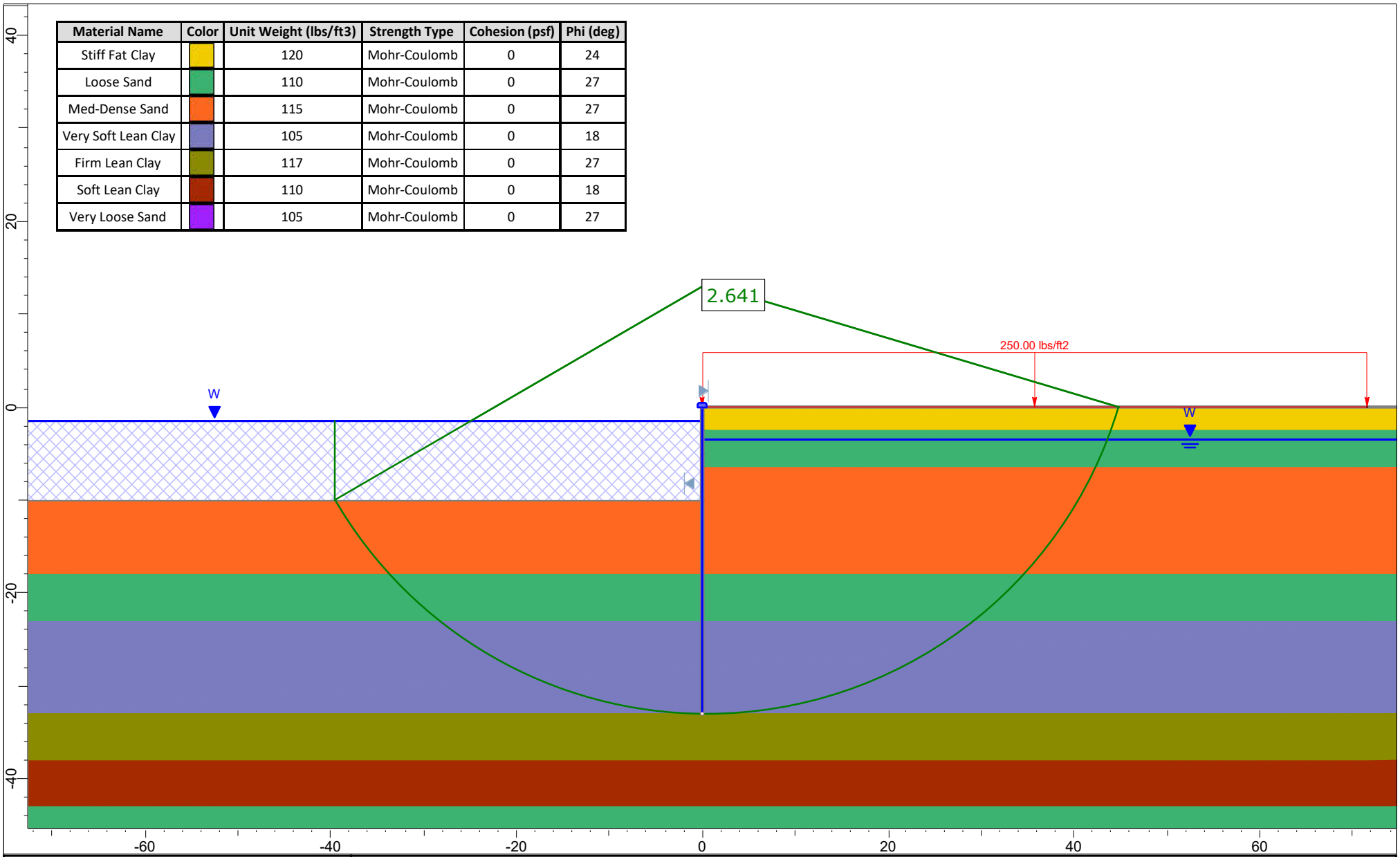
SLIDEINTERPRET 9.007

Project	North Beach Navigable Canal		
Group	Group 1	*Scenario*	Master Scenario
Drawn By	J. Buchen	*Company*	Tolunay-Wong Engineers, Inc.
Date	8/14/2020	*File Name*	B-1 Long-Term Anchored

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Stiff Fat Clay		120	Mohr-Coulomb	0	24
Loose Sand		110	Mohr-Coulomb	0	27
Med-Dense Sand		115	Mohr-Coulomb	0	27
Very Soft Lean Clay		105	Mohr-Coulomb	0	18
Firm Lean Clay		117	Mohr-Coulomb	0	27
Soft Lean Clay		110	Mohr-Coulomb	0	18
Very Loose Sand		105	Mohr-Coulomb	0	27



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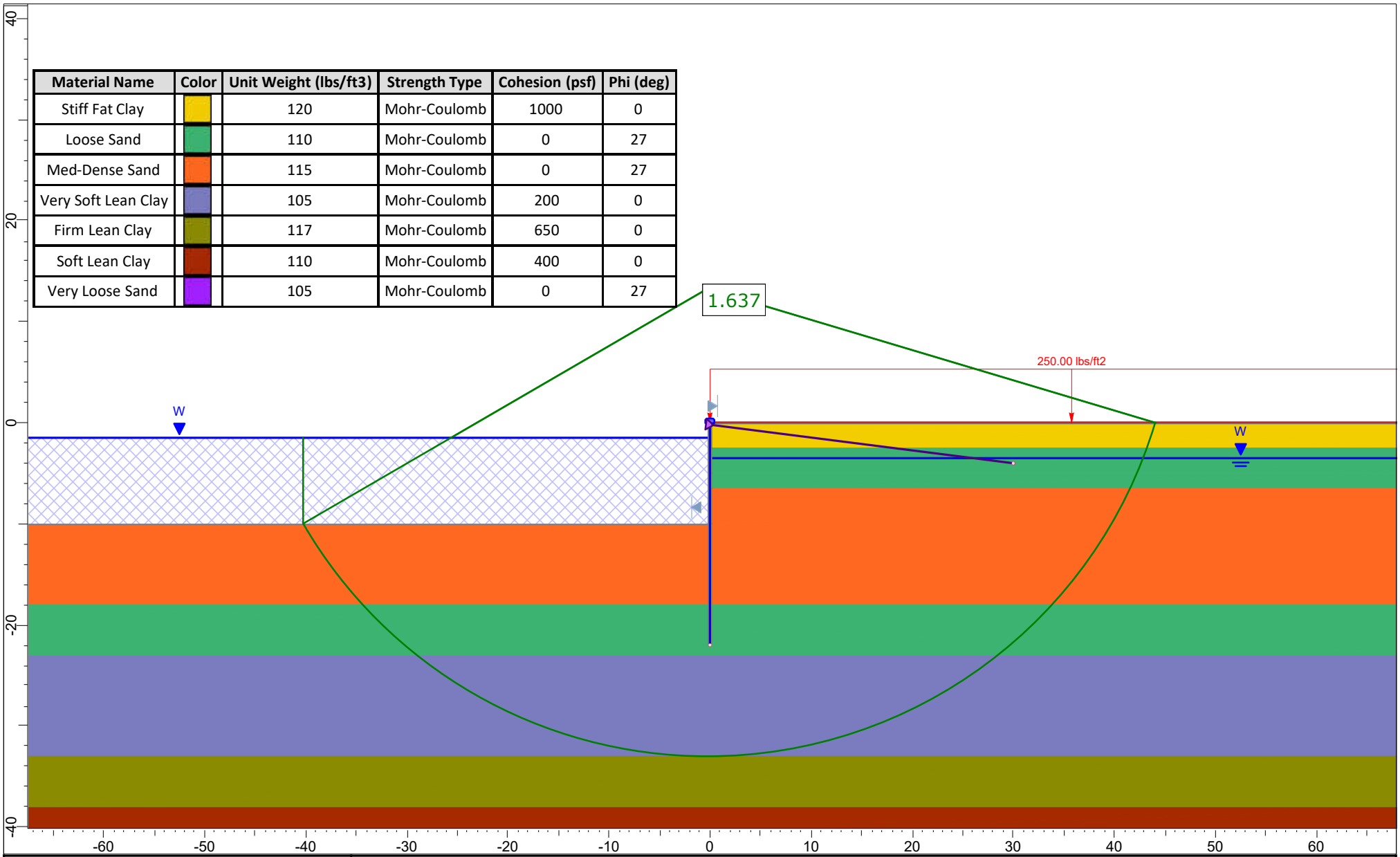
SLIDEINTERPRET 9.007

Project	North Beach Navigable Canal		
Group	Group 1	*Scenario*	Master Scenario
Drawn By	J. Buchen	*Company*	Tolunay-Wong Engineers, Inc.
Date	8/14/2020	*File Name*	B-1 Long-Term Cantilever

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Stiff Fat Clay		120	Mohr-Coulomb	1000	0
Loose Sand		110	Mohr-Coulomb	0	27
Med-Dense Sand		115	Mohr-Coulomb	0	27
Very Soft Lean Clay		105	Mohr-Coulomb	200	0
Firm Lean Clay		117	Mohr-Coulomb	650	0
Soft Lean Clay		110	Mohr-Coulomb	400	0
Very Loose Sand		105	Mohr-Coulomb	0	27



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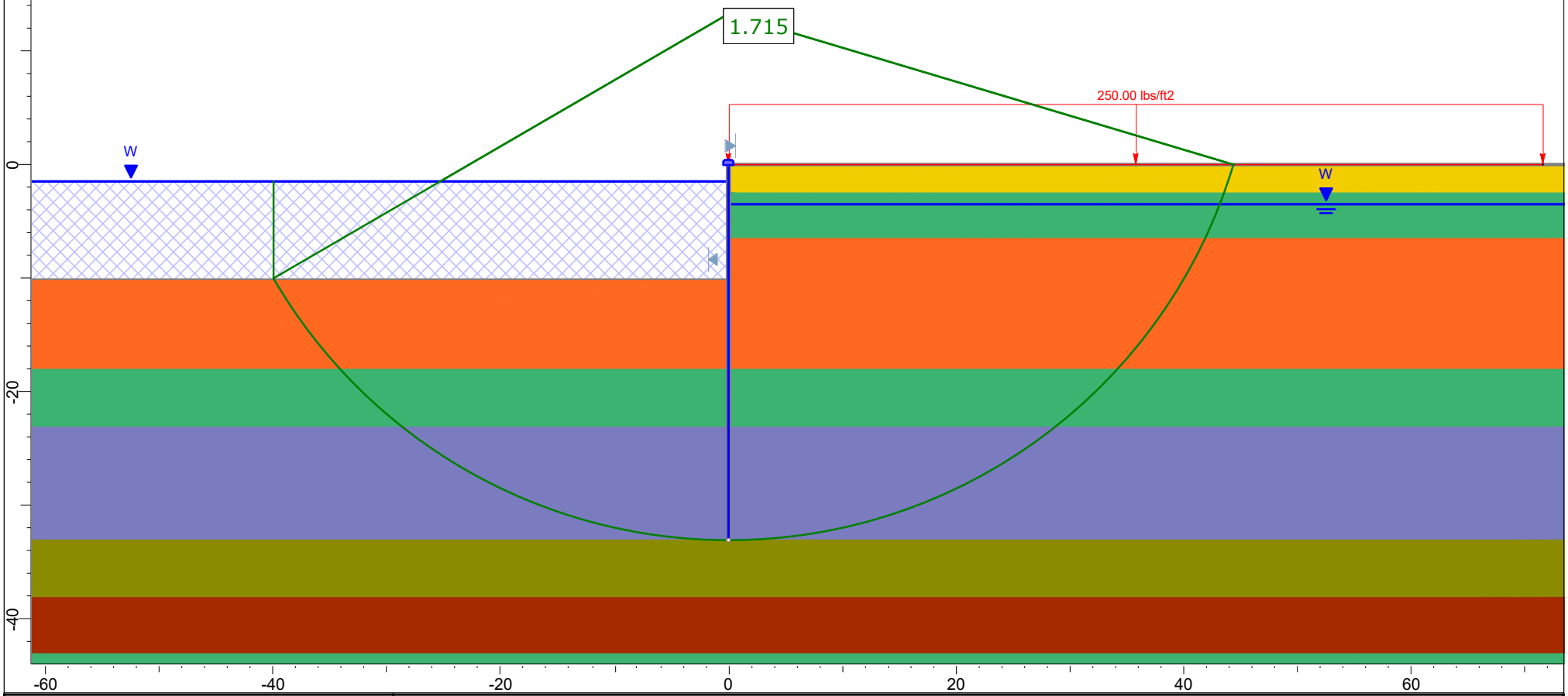
SLIDEINTERPRET 9.007

Project	North Beach Navigable Canal		
Group	Group 1	*Scenario*	Master Scenario
Drawn By	J. Buchen	*Company*	Tolunay-Wong Engineers, Inc.
Date	8/14/2020	*File Name*	B-1 Short-Term Anchored

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Stiff Fat Clay		120	Mohr-Coulomb	1000	0
Loose Sand		110	Mohr-Coulomb	0	27
Med-Dense Sand		115	Mohr-Coulomb	0	27
Very Soft Lean Clay		105	Mohr-Coulomb	200	0
Firm Lean Clay		117	Mohr-Coulomb	650	0
Soft Lean Clay		110	Mohr-Coulomb	400	0
Very Loose Sand		105	Mohr-Coulomb	0	27



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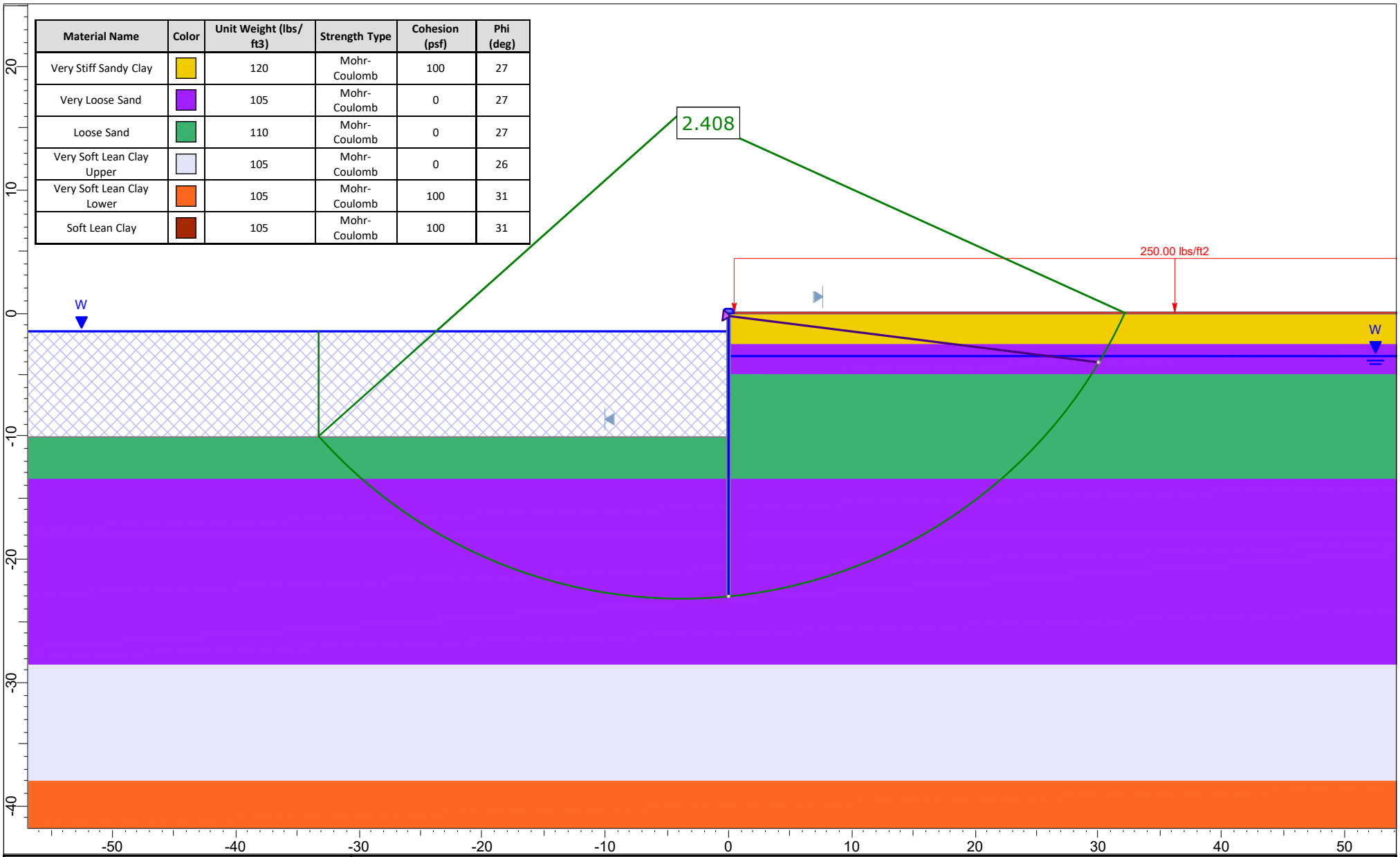
SLIDEINTERPRET 9.007

Project	North Beach Navigable Canal		
Group	Group 1	Scenario	Master Scenario
Drawn By	J. Buchen	Company	Tolunay-Wong Engineers, Inc.
Date	8/14/2020	File Name	B-1 Short-Term Cantilever

Material Name	Color	Unit Weight (lbs/ ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Very Stiff Sandy Clay		120	Mohr-Coulomb	100	27
Very Loose Sand		105	Mohr-Coulomb	0	27
Loose Sand		110	Mohr-Coulomb	0	27
Very Soft Lean Clay Upper		105	Mohr-Coulomb	0	26
Very Soft Lean Clay Lower		105	Mohr-Coulomb	100	31
Soft Lean Clay		105	Mohr-Coulomb	100	31



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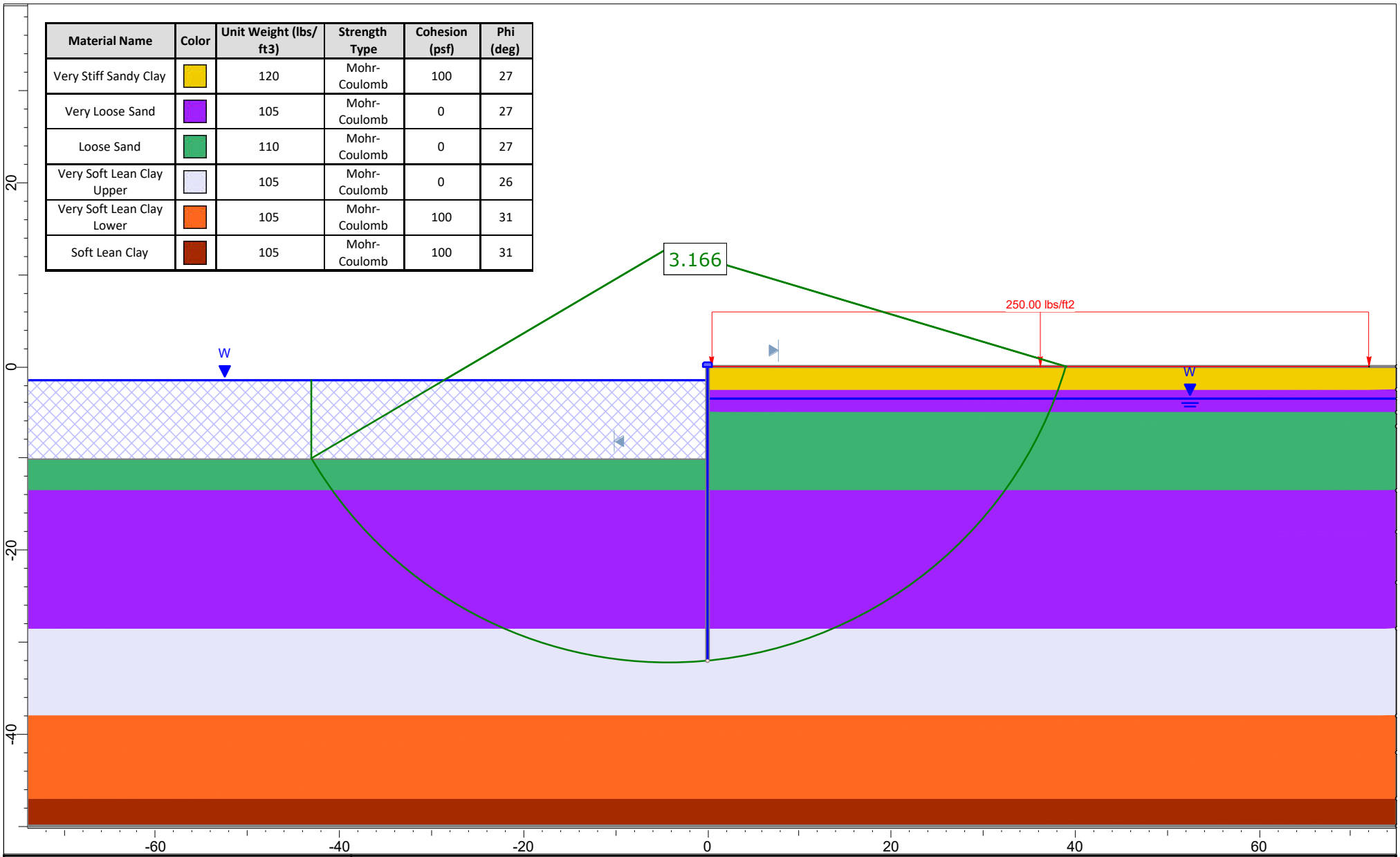
SLIDEINTERPRET 9.007

Project	North Beach Navigable Canal		
Group	Group 1	*Scenario*	Master Scenario
Drawn By	J. Buchen	*Company*	Tolunay-Wong Engineers, Inc.
Date	8/14/2020	*File Name*	B-2 Long-Term Anchored

Material Name	Color	Unit Weight (lbs/ ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Very Stiff Sandy Clay		120	Mohr-Coulomb	100	27
Very Loose Sand		105	Mohr-Coulomb	0	27
Loose Sand		110	Mohr-Coulomb	0	27
Very Soft Lean Clay Upper		105	Mohr-Coulomb	0	26
Very Soft Lean Clay Lower		105	Mohr-Coulomb	100	31
Soft Lean Clay		105	Mohr-Coulomb	100	31



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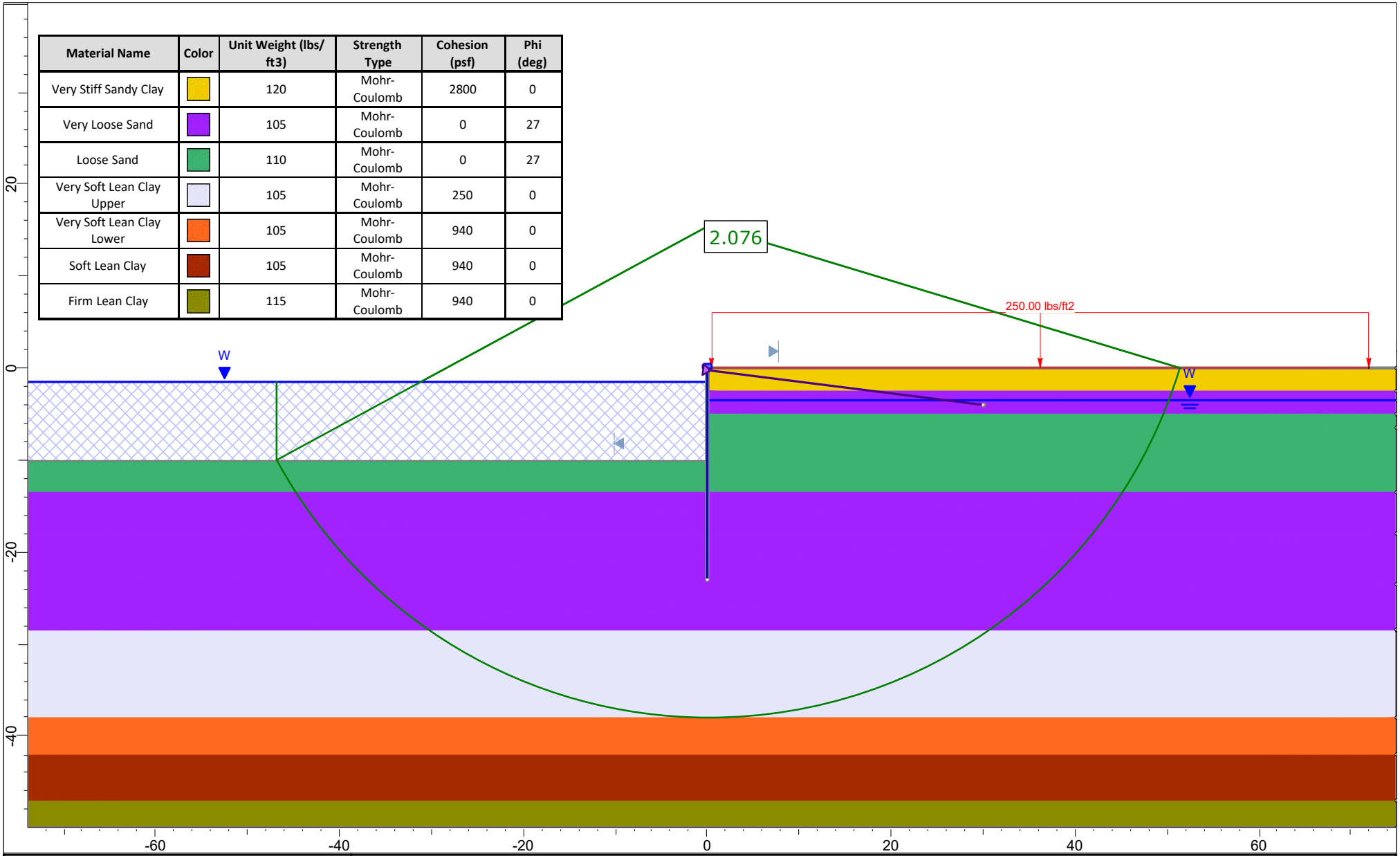
SLIDEINTERPRET 9.007

Project	North Beach Navigable Canal		
Group	Group 1	*Scenario*	Master Scenario
Drawn By	J. Buchen	*Company*	Tolunay-Wong Engineers, Inc.
Date	8/14/2020	*File Name*	B-2 Long-Term Cantilever

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Very Stiff Sandy Clay		120	Mohr-Coulomb	2800	0
Very Loose Sand		105	Mohr-Coulomb	0	27
Loose Sand		110	Mohr-Coulomb	0	27
Very Soft Lean Clay Upper		105	Mohr-Coulomb	250	0
Very Soft Lean Clay Lower		105	Mohr-Coulomb	940	0
Soft Lean Clay		105	Mohr-Coulomb	940	0
Firm Lean Clay		115	Mohr-Coulomb	940	0



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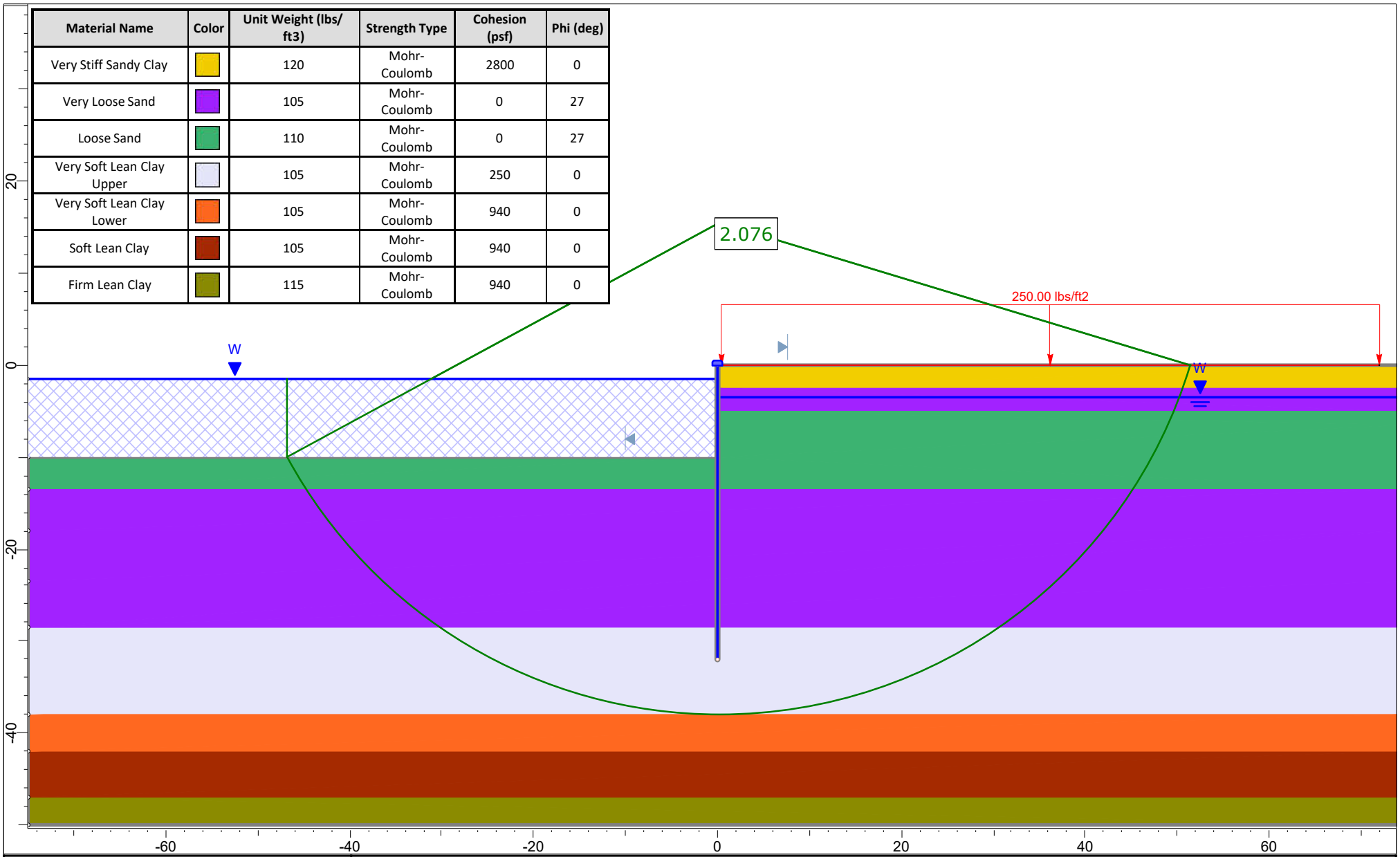
SLIDEINTERPRET 9.007

Project	North Beach Navigable Canal		
Group	Group 1	*Scenario*	Master Scenario
Drawn By	J. Buchen	*Company*	Tolunay-Wong Engineers, Inc.
Date	8/14/2020	*File Name*	B-2 Short-Term 2 Anchored

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Very Stiff Sandy Clay		120	Mohr-Coulomb	2800	0
Very Loose Sand		105	Mohr-Coulomb	0	27
Loose Sand		110	Mohr-Coulomb	0	27
Very Soft Lean Clay Upper		105	Mohr-Coulomb	250	0
Very Soft Lean Clay Lower		105	Mohr-Coulomb	940	0
Soft Lean Clay		105	Mohr-Coulomb	940	0
Firm Lean Clay		115	Mohr-Coulomb	940	0



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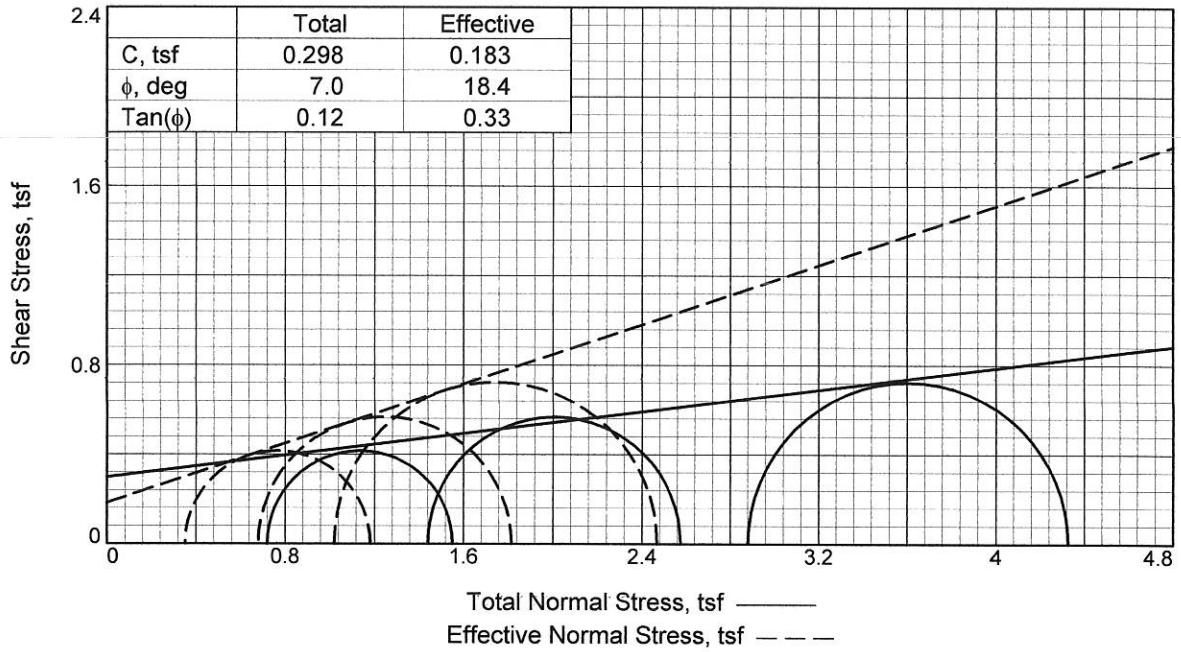
SLIDEINTERPRET 9.007

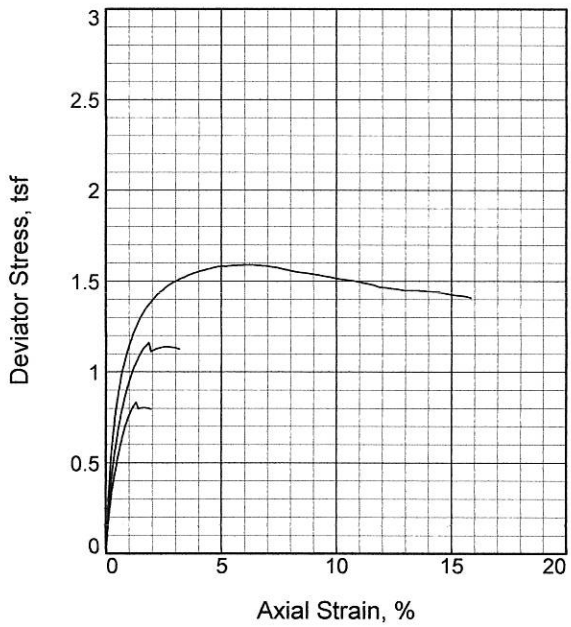
Project	North Beach Navigable Canal		
Group	Group 1	*Scenario*	Master Scenario
Drawn By	J. Buchen	*Company*	Tolunay-Wong Engineers, Inc.
Date	8/14/2020	*File Name*	B-2 Short-Term 2 Cantilever

APPENDIX E

CONSOLIDATED-UNDRAINED TRIAXIAL SHEAR TEST RESULTS



	Total	Effective
C, tsf	0.298	0.183
ϕ, deg	7.0	18.4
Tan(ϕ)	0.12	0.33



Sample No.		1	2	3
Initial	Water Content, %	62.4	62.4	62.4
	Dry Density, pcf	61.6	61.6	61.6
	Saturation, %	97.1	97.1	97.1
	Void Ratio	1.7365	1.7365	1.7365
	Diameter, in.	2.84	2.84	2.84
	Height, in.	5.73	5.73	5.73
At Test	Water Content, %	51.2	51.2	51.2
	Dry Density, pcf	70.8	70.8	70.8
	Saturation, %	100.0	100.0	100.0
	Void Ratio	1.3817	1.3817	1.3817
	Diameter, in.	2.64	2.67	2.72
	Height, in.	5.78	5.67	5.43
Strain rate, %/min.		0.0080	0.0080	0.0080
Eff. Cell Pressure, psi		10.00	20.00	40.00
Fail. Stress, tsf		0.83	1.13	1.45
Excess Pore Pr., tsf		0.37	0.76	1.86
Strain, %		1.3	2.9	13.7
Ult. Stress, tsf		0.83	1.16	1.59
Excess Pore Pr., tsf		0.37	0.69	1.66
Strain, %		1.3	1.8	6.1
$\bar{\sigma}_1$ Failure, tsf		1.18	1.81	2.47
$\bar{\sigma}_3$ Failure, tsf		0.35	0.68	1.02

Type of Test:
CU with Pore Pressures
Sample Type: Undisturbed
Description: Gray Fat Clay

LL= 93 **PL=** 25 **PI=** 68
Assumed Specific Gravity= 2.7
Remarks: Test Method: ASTM D4767
Failure Type: Bulge
Failure Criteria: Effective Stress Ratio

Figure _____

Client: CCOC

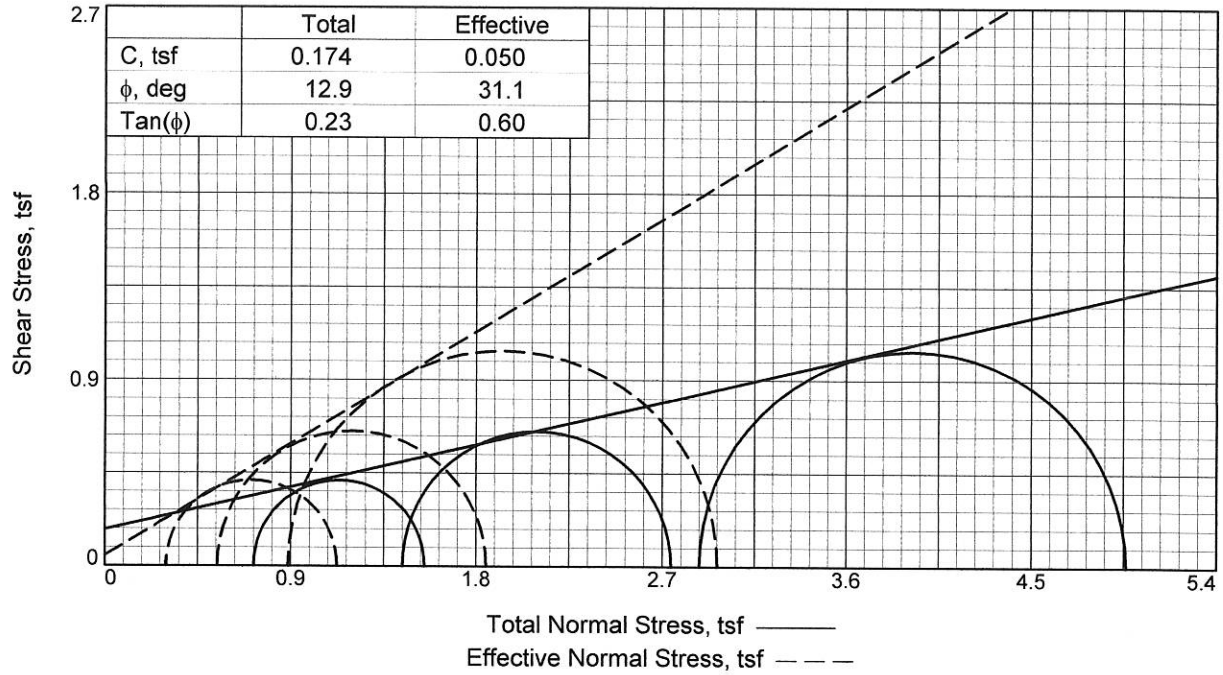
Project: North Beach Drainage Project

Source of Sample: B-1 **Depth:** 38-40

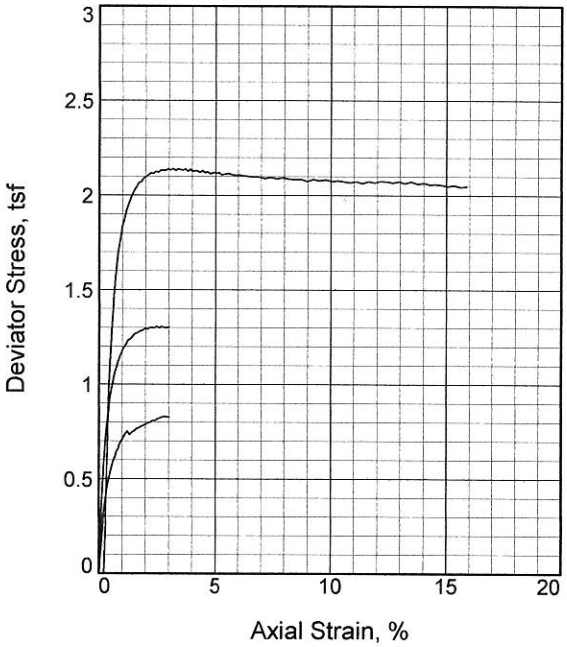
Proj. No.: 20.53.036 **Date Sampled:** 07/20/20

TRIAXIAL SHEAR TEST REPORT
Tolunay-Wong Engineers, Inc.
Houston, Texas

Tested By: EC **Checked By:** PH



	Total	Effective
C, tsf	0.174	0.050
ϕ, deg	12.9	31.1
Tan(ϕ)	0.23	0.60



Sample No.		1	2	3
Initial	Water Content, %	34.4	34.4	34.4
	Dry Density, pcf	88.1	88.1	88.1
	Saturation, %	99.7	99.7	99.7
	Void Ratio	0.9477	0.9477	0.9477
	Diameter, in.	2.83	2.83	2.83
	Height, in.	5.73	5.73	5.73
At Test	Water Content, %	27.1	27.1	27.1
	Dry Density, pcf	98.4	98.4	98.4
	Saturation, %	100.0	100.0	100.0
	Void Ratio	0.7441	0.7441	0.7441
	Diameter, in.	2.68	2.71	2.75
	Height, in.	5.71	5.57	5.42
Strain rate, %/min.		0.0080	0.0080	0.0080
Eff. Cell Pressure, psi		10.00	20.00	40.00
Fail. Stress, tsf		0.83	1.30	2.08
Excess Pore Pr., tsf		0.42	0.90	1.99
Strain, %		3.0	3.0	9.8
Ult. Stress, tsf		0.83	1.30	2.14
Excess Pore Pr., tsf		0.42	0.87	1.86
Strain, %		2.8	2.5	3.2
$\bar{\sigma}_1$ Failure, tsf		1.12	1.85	2.97
$\bar{\sigma}_3$ Failure, tsf		0.30	0.54	0.89

Type of Test:
CU with Pore Pressures

Sample Type: Undisturbed

Description: Gray SANDY LEAN CLAY

LL= 39 **PL=** 18 **PI=** 21

Assumed Specific Gravity= 2.75

Remarks: Test method: ASTM D4767
Failure type: Bulge
Failure criteria: Effective Stress Ratio

Figure _____

Client: CCOC

Project: North Beach Drainage Project

Source of Sample: B-2 **Depth:** 38-40

Proj. No.: 20.53.036 **Date Sampled:** 7/14/20

TRIAXIAL SHEAR TEST REPORT
Tolunay-Wong Engineers, Inc.
Houston, Texas