Subsurface Exploration and
Geotechnical Engineering Evaluation
Water Main and Force Main
Intracoastal Crossing, Cortez Road
Bradenton, Manatee County, Florida

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December 2, 2022
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Kimley Horn & Associates, Inc.
100 Second Avenue South, Suite 105 N
St. Petersburg, FL 33701

Attention: Jordan Walker, P.E.

Subject: Subsurface Exploration and
Geotechnical Engineering Evaluation
Water Main and Force Main Intracoastal Crossing
Cortez Road, Bradenton, Manatee County, Florida

Dear Mr. Walker:

As requested and authorized, we have completed a subsurface exploration and geotechnical engineering evaluation for the subject project. The purposes of performing this exploration were to evaluate the general subsurface conditions within the utility crossing area and to provide soil parameters for use by others in design. This report documents our findings and presents our engineering recommendations.

SITE LOCATION AND SITE DESCRIPTION

The site for the proposed utility crossing is located on the south side of Cortez Road in Bradenton, Florida. The general site location is shown superimposed on the Bradenton Beach, U.S.G.S. quadrangle map presented on Figure 1B.

PROPOSED CONSTRUCTION AND GRADING

It is our understanding that the proposed construction includes a new utility pipeline (water main and force main) crossing of the intracoastal waterway. Portions may be installed by cut-and-cover methods, but most of the crossing is expected to be by horizontal directional drilling (HDD) techniques..

REVIEW OF SOIL SURVEY MAPS

Based on information obtained online from the “Web Soil Survey” as operated by the U.S. Department of Agriculture, Natural Resources Conservation Service, the on-land portion of the site is located in an area mapped as the “Canaveral fine sand, 0 to 5 percent slopes” and the “Estero muck” soil series. The general site location is shown superimposed on a soils map from the “Web Soil Survey” presented on Figure 2B.

The “Canaveral fine sand, 0 to 5 percent slopes” soil series consists of a nearly level to gently sloping, moderately well drained to somewhat poorly drained soil on narrow to broad dune-like ridges. A typical soil profile consists of fine sand to a depth of 17 inches, underlain by sand and shell fragments to a depth of 65 inches feet or more. According to the Soil Survey, the water table is within 10 to 40 inches of the natural ground surface for about 2 to 6 months of a typical year.

The NRCS describes the “Estero muck” soil series as a nearly level, very poorly drained soil in tidal mangrove swamps. A typical soil profile consists of a surface layer of black muck that is typically 6 inches thick and underlain by black to very dark gray fine sand to a depth of 14 inches below the natural ground surface. This is underlain by fine sand to a depth of 80 inches. According to the Soil Survey, these soils are flooded daily by normal high tides.

FIELD EXPLORATION PROGRAM

SPT Borings

The field exploration program included performing nine (9) Standard Penetration Test (SPT) borings. The SPT borings were advanced to a maximum depth of 50 feet below the existing ground surface on land and 100 feet below the mud-line in the water, generally using the methodology outlined in ASTM D-1586. A summary of this field procedure is included in Appendix I.

Soil samples recovered during performance of the borings were visually classified in the field and representative portions of the samples were transported to our laboratory in sealed sample jars.

The groundwater level at each of the land boring locations was measured during drilling. The borings were backfilled with cement grout upon completion.

Test Locations

The approximate locations of the borings are schematically illustrated on an aerial photograph shown on Figure 3B. These locations were determined in the field by estimating distances from existing site features and should be considered accurate only to the degree implied by the method of measurement used.

LABORATORY PROGRAM

Representative soil samples obtained during our field sampling operation were packaged and transferred to our laboratory for further visual examination and classification. The soil samples were classified using visual-manual procedures in general accordance with the Unified Soil Classification System (ASTM D-2488). The resulting soil descriptions are shown on the soil boring profiles presented on Figures 4B through 6B.

Corrosivity Tests

The laboratory testing program also included corrosivity series testing. This series of tests includes determining electrical resistivity, soil pH, sulfates content and chlorides content (FM 5-550, 5-551, 5-552 and 5-553).

The tests were performed on three (3) samples. The test results are summarized in the table below:

Sample	Borings	Depth (feet)	Soil Classification	pH	Chloride (ppm)	Sulfate (ppm)	Resistivity (ohm-cm)
C-1	CR-1	10 – 20	SP	8.16	BDL	147	4,425
C-2	CR-2	3 – 10	SP	8.24	15	189	1,370
C-3	CR-9	3 – 10	SP	7.93	BDL	48	8,610

BDL = below detection limit

Based upon Table 1.3.2-1 of the FDOT “Structures Design Guidelines” (Vol I, Sec. 1.3), sample C-1 would be classified as an “slightly aggressive” environment to concrete and “moderately aggressive” to steel. Sample C-2 would be classified as a “moderately aggressive” environment to steel and concrete, and C-3 would be classified as a “slightly aggressive” environment to steel and concrete. This assumes that the structure (pipeline) is not considered a “marine structure” (see Sec. 1.3.2.B).

GENERAL SUBSURFACE CONDITIONS

General Soil Profile

The results of the field exploration and laboratory programs are graphically summarized on the soil boring profiles presented on Figures 4B through 6B. The stratification of the boring profiles represents our interpretation of the field boring logs and the results of laboratory examinations of the recovered samples. The stratification lines represent the approximate boundary between soil types. The actual transitions may be more gradual than implied.

The results of the borings indicate the following general soil profile:

Depth Below Ground or Water Surface (feet)*		Description
From	To	
0	32 - 42	Loose to very dense fine sand (SP), fine sand with silt (SP-SM), silty fine sand (SM), and clayey fine sand (SC).
32 - 42	100	Medium dense silty fine sand (SM) and clayey fine sand; firm to very stiff clay (CL/CH) and silt (ML); with layers of hard cemented clay (CL/CH), hard cemented silt(ML) and rock.

* Below ground surface for borings on land. Below water surface for borings on water.

The above soil profile is outlined in general terms only. Please refer to Figures 4B through 6B for soil profile details.

Groundwater Level

For the borings performed on land (CR-1, CR-2 and CR-9), the groundwater level was measured in the boreholes during drilling. As shown on Figure 4B, the water level in these borings was encountered at a depth of 3 to 7 feet below the existing ground surface on the dates indicated.

At the borings performed in the water, the water level was in the range of 6 to 20 feet above the mud-line (bay bottom) at the time the borings were performed, as is depicted on the soil boring logs on Figures 5B and 6B.

Fluctuation in groundwater levels should be anticipated throughout the year primarily due to seasonal variations in rainfall, tidal fluctuation and other factors that may vary from the time the borings were conducted.

ENGINEERING EVALUATION AND RECOMMENDATIONS - CUT AND COVER

General

The results of this exploration indicate that the existing soils, as encountered in the borings, are generally suitable for supporting the proposed pipelines and associated structures. Organic soils (muck) were not encountered at the boring locations, but may be present at unexplored locations.

Plastic clays, silts, cemented silts, cemented clays and weathered limestone were encountered at depths greater than approximately 30 to 35 feet below the land surface (for the borings on land) or the water surface (for the borings on the water) at the boring locations. These soils may be difficult to excavate, but we have assumed that excavation depths for cut-and-cover installation or for HDD entry/exit pits will be shallower than 30 feet, so these soils should not be a significant concern.

The following are our recommendations for overall site preparation and foundation support which we feel are best suited for the proposed pipelines and associated structures relative to the soil conditions encountered in the borings performed to-date. The recommendations are made as a guide for the design engineer, parts of which should be incorporated into the project's specifications.

In general, the recommendations provided below are applicable to structures that are no greater than approximately 30 feet below the land surface at boring locations CR-1, CR-2 and CR-9 or below the water surface of the intracoastal waterway. If there are proposed structures at greater depth, we should review these and prepare additional recommendations, if necessary. Note that recommendations for horizontal directional drill (HDD) installation will be presented later in this report.

Pipelines and Associated Structures

<u>Excavation</u>

Based on the conditions encountered during the field exploration, we anticipate that the soils encountered from the ground surface to a depth of at least 30 feet can generally be excavated with standard earth moving equipment (i.e., front-end loaders, backhoes and excavators). Based upon the SPT N-values, the sandy soils within this depth are primarily in a loose to medium dense state (SPT N-value generally less than 30), so should not be difficult to excavate with conventional equipment, assuming that the soils are adequately dewatered. Some dense to very dense layers were encountered, however, (such as at a depth of approximately 15 to 20 feet at boring CR-2) and may also occur at other locations. In general, soils having and N-value greater than 30 may be more difficult to excavate than typical loose to medium dense soils. Note that the N-values are listed adjacent to the boring logs on Figures 4B through 6B.

The soils below the bottom of the excavations should not be disturbed by the excavation process. If soils become disturbed and difficult to compact, they should be over-excavated below the pipeline and other structures to a depth necessary to remove all disturbed soils. Over-excavated areas should be replaced with compacted backfill meeting the "Backfill Requirements" presented in a subsequent section of this report.

The excavations should be safely braced or sloped to prevent injury to personnel or damage to equipment. Temporary safe slopes in dewatered soils should be cut no steeper than 1.5 horizontal (H) to 1 vertical (V), in accordance with OSHA, 29 CFR Part 1926 Subpart P. Flatter slopes should be used if deemed necessary based on actual conditions encountered. Surcharge loads should be kept at least 5 feet from excavations. Spoil banks adjacent to excavations should be sloped no steeper than 2.0H to 1.0V. Provisions for maintaining workers' safety within and adjacent to excavations is the sole responsibility of the Contractor.

Dewatering

The control of the groundwater may be required to achieve the necessary depths of excavation and subsequent construction, backfilling and compaction requirements presented in the following sections. The actual method(s) of dewatering should be determined by the Contractor. However, regardless of the method(s) used, we suggest drawing down the groundwater table sufficiently (i.e., 2 to 3 feet) below the bottom of the excavation(s) to preclude "pumping" and/or compaction-related problems with the foundation soils. We recommend that the dewatering be accomplished in advance of the excavation.

Pipeline Bedding

Pipe bedding material should be compacted to achieve a density equivalent to 95 percent of the maximum dry density, as determined by the Modified Proctor (ASTM D-1557), to a minimum depth of 6 inches below the bottom of the pipe. Compact deeper if recommended by the pipe manufacturer.

We recommend that the bedding for the pipe be preshaped by means of a template prior to placement of the pipe to ensure that the upward reaction on the bottom of the pipe will be well distributed over the width of the bedding contact. Based on the cost involved with preshaping the bedding material and the construction time requirements, an alternative procedure may be to utilize a level bed for the pipe and require a higher pipe strength class that will adequately carry the load on a lower class of bedding. It would be prudent to perform an economic analysis of the two alternatives, or specify both design conditions within the contract documents and allow the Contractor to decide the most efficient method.

If level bedding is utilized, it will be necessary to place and compact the haunching backfill (backfill between the bedding and the springline of the pipe) to the springline of the pipe. This material should be placed in simultaneous layers on each side of the pipe and must be compacted in such a manner as to ensure an intimate contact with the sides of the pipe. Do not use blocking under the pipe to raise the pipe to grade.

The final backfill above the haunching or springline of the pipe must extend all the way to the trench walls and should be placed in level lifts not exceeding 12 inches. Each lift should be compacted to at least 95 percent of the maximum dry density, as determined by the Modified Proctor (ASTM D-1557). Care should be taken not to damage the pipe or deflect it by compacting directly above the pipe where there is insufficient cover material present. Minimum cover criteria should be in accordance with the pipe manufacturer's recommendations.

Where the utility line will traverse roadways and/or other permanent structures such as sidewalks, all backfill should be compacted to 95 percent of maximum dry density, as determined by the Modified Proctor (ASTM D-1557), from the top of the pipe to the ground surface. The design engineer may wish to specify greater compaction for the pavement subgrade, to be consistent with the pavement design requirements.

A geotechnical engineer or a designated representative from Ardaman & Associates, Inc. should observe and test all prepared and compacted areas to verify that all bedding, haunching and final backfill are prepared and compacted in accordance with the aforementioned specifications

Backfill Requirements

As a general guide to aid the Contractor regarding materials to use for fill in the excavations, we recommend using fine sand (SP) to fine sand with silt (SP-SM) that contains less than 1 percent organic matter and no greater than 12 percent by dry weight of material passing the U.S. Standard No. 200 sieve size. Soils with more than 12 percent passing the No. 200 sieve will be more difficult to compact due to their inherent nature to retain soil moisture.

Based on the soil samples obtained during our subsurface investigation, the on-site fine sand (SP) and fine sand with silt (SP-SM) soils without roots and/or organic matter appear suitable for use as structural backfill for the pipe. Material removed from below the groundwater table will be wet and will require time to dry sufficiently before compacting.

The silty fine sand (SM) could be used in some applications as structural backfill, but will be more difficult to moisture condition and compact due to its inherent nature to retain moisture. Clayey fine sand (SC), clay (CL/CH), silt (ML) and similar soils should not be used as backfill.

Resistance to Horizontal Forces on Pipeline Structures

Horizontal forces which act on structures such as thrust blocks or anchor blocks can be resisted to some extent by the earth pressures that develop in contact with the buried vertical face (buried vertical face is perpendicular and in front of the applied horizontal load) of the block structures and by shearing resistance mobilized along the base of the block structures and soil subgrade interface.

Allowable earth pressure resistance may be determined using an equivalent fluid density of 110 pounds per cubic foot (pcf) for moist soil above the water table and 70 pcf for submerged soils below the water table[1]. The passive earth pressures are developed from ground surface[2] to the bottom of the block structure.

The values presented above presume that the block structures are surrounded by well compacted sand backfill extending at least 5 feet horizontally beyond the vertical buried face. In addition, it is presumed that the block structures can withstand horizontal movements on the order of one-quarter (1/4) to three-eighths (3/8) inch before mobilizing full passive resistance. The factors of safety assumed in the above recommendations are 2.5 for passive pressure with submerged conditions, and 3.0 for passive pressure without submerged conditions.

The sliding shearing resistance mobilized along the base of the block structure may be determined by the following formula:

Allowable Shearing Resisting Force, $P = V \tan(2/3\ \phi)/S.F$

Where: P = Shearing Resistance Force (pounds)
V = Net Vertical Force (total weight of block and soil overlying the structure minus uplift forces including buoyancy forces) (pounds)
ϕ = Angle of Internal Friction of Soil = 30 degrees
S.F. = Safety Factor = 1.5

The vertical earth pressures developed by the overburden weight of soil can be calculated using the following unit weights:

- Compacted moist soil = 110 pcf
- Saturated soil = 120 pcf (buoyant unit weight of saturated soil = 58 pcf)

Vertical pressure distributions in accordance with the above do not take into account vertical forces from construction equipment, wheel loads or other surcharge loads.

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1 Equivalent fluid density (moist soil) = $K_p \gamma_m$/S.F. = 110 pcf
Equivalent fluid density (submerged soil) = $K_p (\gamma_s - \gamma_w)$/S.F. = 70 pcf

Where: K_p = effective coefficient of passive earth pressure = 3.0
S.F. = safety factor = (values given above)
γ_m = unit weight of moist soil = 110 pcf
γ_s = unit weight of saturated soils = 120 pcf
γ_w = unit weight of water = 62.4 pcf

2 Assuming there is no excavation in the vicinity of the block structure that would reduce the available passive pressure.

Foundation Support and Estimated Settlements

Permanent structures such as anchor blocks, thrust blocks, air release valves, blow offs, etc., bearing at least 18 inches below adjacent grade and at least 18 inches wide can be designed for the following maximum vertical bearing capacities:

- 1,500 psf on undisturbed natural granular soils.
- 2,000 psf on compacted natural or backfilled subgrade; this value assumes compaction of at least 95 percent of the Modified Proctor maximum density (ASTM D-1557, AASHTO T-180) to a depth of 1 foot below the structure.

Pipe settlement during and after construction should be negligible (less than 1/2 inch) provided the bedding and backfilling criteria in the above sections are satisfied. The volume of soil displaced by the pipe, compared to the weight of the pipe when full, will result in little if any net increase in bearing stress to the subsurface soils.

Uplift Resistance

Permanent structures submerged below the groundwater table will be subjected to uplift forces caused by buoyancy. The components resisting this buoyancy include: 1) the total weight of the pipe or structure divided by an appropriate factor of safety; 2) the buoyant weight of soil overlying the pipe or structure; and 3) the shearing forces that act on shear planes that radiate vertically upward from the perimeter of the pipe or the edges of the structure to the ground surface. The allowable unit shearing resistance may be determined by the following formula:

Allowable Shearing Resistance, $F=K_o\gamma_m h(2/3 \tan\phi)/S.F.$ (above water table)

Allowable Shearing Resistance, $F=K_o[\gamma_m h_w+\gamma_b(h-h_w)](2/3 \tan\phi)/S.F.$ (below water table)

where: F = unit shearing resistance (psf)
K_o = coefficient of earth pressure at rest = 0.5
γ_m = unit weight of moist soil = 110 pcf
γ_b = buoyant unit weight of soil = 58 pcf
h = vertical depth (feet) below grade at which shearing resistance is determined
h_w = vertical depth (feet) below grade to groundwater table
ϕ = angle of internal friction of the soil = 30 degrees
S.F. = safety factor = 2.0

The values given for the above parameters assume that the permanent structures are covered by clean, well-compacted, granular (sand) backfill that extends horizontally at least 2 feet beyond the structures.

Earth Pressure on Shoring and Bracing

If temporary shoring and bracing are required for any excavations, the system should be designed to resist lateral earth pressure. The design earth pressure will be a function of the flexibility of the shoring and bracing system. For a flexible system restrained laterally by braces placed as the excavation proceeds, the design earth pressure for shoring and bracing can be computed using a uniform earth pressure distribution with depth. It is recommended that soils be dewatered around the excavations. For such dewatered excavations, we recommended using the following uniform pressure distribution over the full braced height as follows:

Uniform Soil Pressure Distribution, $p = 0.65\ K_a \gamma_s H$

where: p = uniform pressure distribution for design of braced excavation
K_a = coefficient of active earth pressure = 0.33
γ_s = unit weight of saturated soils = 120 pcf
H = depth of excavation

An appropriate factor of safety should be applied for the design of the braced excavations.

Lateral pressure distributions determined in accordance with the above do not take hydrostatic pressures or surcharge loads into account. To the extent that such pressures and forces may act on the walls, they should be included in the design.

Construction equipment and excavated fill should be kept a minimum distance of 5 feet from the edge of the braced or shored excavation. Backfill material placed adjacent to (maintaining a minimum 5-foot horizontal clearance) the braced or shored excavation should have a minimum slope of 2.0H to 1.0V or flatter, if required by site specific conditions and/or to meet OSHA requirements.

Means and methods of excavation and bracing should be the responsibility of the Contractor; however, excavation and/or bracing should, at a minimum, comply with the requirements of the Occupational Safety Health Administration (OSHA).

Lateral Earth Pressures

Lateral loads acting on the embedded structure will include at-rest earth pressures as well as hydrostatic pressures and surcharge loads. The lateral earth pressure will be a function of both the depth below ground surface and the soil unit weight (submerged or moist) plus hydrostatic pressure (if applicable). The following equations can be used to determine the lateral at-rest earth pressure:

$\sigma_h = K_o \gamma_m h$ (above water table)
$\sigma_h = K_o[\gamma_m h_w + \gamma_b(h-h_w)]$ (below water table)

where: σ_h = lateral earth pressure (psf)
K_o = coefficient of at rest earth pressure (0.5) (this value assumes that the backfill is lightly compacted yet not overcompacted)
γ_m = moist unit weight of soil = 110 pcf for compacted moist soil above the water table.
γ_b = buoyant unit weight of soil = 58 pcf for compacted saturated soil below the water table.
h= vertical depth (feet) below grade at which lateral earth pressure is determined.
h_w = vertical depth (feet) below grade to groundwater table

For design, an appropriate factor of safety should be applied to the lateral earth pressure calculated using the above equation. Lateral pressure distributions determined in accordance with the above do not include hydrostatic pressures or surcharge loads. Where applicable, they should be incorporated in the design.

SOIL PARAMETERS FOR DIRECTIONAL DRILL

This section includes recommended soil parameters for use by others in the design of the utility crossings. Based upon the soil classifications and SPT "N" values, the internal friction angle, cohesion and unit weights have been estimated for the soils encountered at each of the boring locations. These are listed in the tables in Appendix II of this report.

QUALITY CONTROL

We recommend establishing a comprehensive quality control program to verify that all excavation, bedding, and backfilling is conducted in accordance with the appropriate plans and specifications. Materials testing and inspection services should be provided by Ardaman & Associates, Inc. In-situ density tests should be conducted during bedding and backfilling activities to verify that the required densities are achieved.

Backfill for the proposed pipeline should be tested at a minimum frequency of one in-place density test for each lift for each 200 lineal feet of pipe. Additional tests should be performed beneath foundations and in backfill for other associated structures. In-situ density values should be compared to laboratory Proctor moisture-density results for each of the different natural and fill soils encountered.

CLOSURE

The analyses and recommendations submitted herein are based on the data obtained from the soil borings presented on Figures 4B through 6B. This report does not reflect any variations which may occur adjacent to or between the borings. The nature and extent of the variations between the borings may not become evident until during construction. If variations then appear evident, it will be necessary to re-evaluate the recommendations presented in this report after performing on-site observations during the construction period and noting the characteristics of the variations.

This study is based on a relatively shallow exploration and is not intended to be an evaluation for sinkhole potential. This study does not include an evaluation of the environmental (ecological or hazardous/toxic material related) condition of the site and subsurface.

This report has been prepared for the exclusive use of Kimley Horn & Associates, Inc. in accordance with generally accepted geotechnical engineering practices. No other warranty, expressed or implied, is made.

We are pleased to be of assistance to you on this phase of the project. When we may be of further service to you or should you have any questions, please contact us.

Very truly yours,
ARDAMAN & ASSOCIATES, INC.
Fl. Registry No. 5950

JERRY H. KUEHN LICENSE No. 35557 STATE OF FLORIDA PROFESSIONAL ENGINEER

This document has been digitally signed and sealed by:

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APPENDIX I

Standard Penetration Test
and Laboratory Test Procedures

STANDARD PENETRATION TEST

The standard penetration test is a widely accepted test method of *in situ* testing of soils (ASTM D 1586), and Ardaman & Associates generally follows this test method. A 2-foot long, 2-inch O.D. split-barrel sampler attached to the end of a string of drilling rods is driven 18 or 24 inches into the ground by successive blows of a 140-pound hammer freely dropping 30 inches. The number of blows needed for each 6 inches of penetration is recorded. The sum of the blows required for penetration of the second and third 6-inch increments of penetration constitutes the test result or N-value. After the test, the sampler is extracted from the ground and opened to allow visual examination and classification of the retained soil sample. The N-value has been empirically correlated with various soil properties.

The tests are usually performed at 5-foot intervals. The test holes are advanced to the test elevations by rotary drilling with a cutting bit, using circulating fluid to remove the cuttings and hold the fine grains in suspension. The circulating fluid, which is a bentonitic drilling mud, is also used to keep the hole open below the water table by maintaining an excess hydrostatic pressure inside the hole. In some soil deposits, particularly highly pervious ones, flush-coupled casing must be driven to just above the testing depth to keep the hole open and/or prevent the loss of circulating fluid.

Representative split-spoon samples from the soils are brought to our laboratory in air-tight jars for further evaluation and testing, if necessary.

Soil Classifications

The soil descriptions presented on the soil boring logs are based upon the Unified Soil Classification System (USCS), which is the generally accepted method (ASTM D-2487 and D-2488) for classifying soils for engineering purposes. The following modifiers are the most commonly used in the descriptions.

For Sands:	Modifier	Fines, Sand or Gravel Content*
	with silt or with clay	5% to 12% fines
	silty or clayey	12% to 50% fines
	with gravel or with shell	15% to 50% gravel or shell

For Silts or Clays:	Modifier	Fines, Sand or Gravel Content*
	with sand	15% to 30% sand and gravel; and % sand > % gravel
	sandy	30% to 50% sand and gravel; and % sand > % gravel
	with gravel	15% to 30% sand and gravel; and % sand < % gravel
	gravelly	30% to 50% sand and gravel; and % sand < % gravel

* may be determined by laboratory testing or estimated by visual/manual procedures. Fines content is the combined silt and clay content, or the percent passing the No. 200 sieve.

The USCS also uses a set of Group Symbols, which may also be listed on the soil boring logs. The following is a summary of these.

Group Symbol	General Group Name*	Group Symbol	General Group Name*
GW	Well-graded gravel	SW	Well-graded sand
GP	Poorly graded gravel	SP	Poorly graded sand
GW-GM	Well-graded gravel with silt	SW-SM	Well-graded sand with silt
GW-GC	Well-graded gravel with clay	SW-SC	Well-graded sand with clay
GP-GM	Poorly graded gravel with silt	SP-SM	Poorly graded sand with silt
GP-GC	Poorly graded gravel with clay	SP-SC	Poorly graded sand with clay
GM	Silty gravel	SM	Silty sand
GC	Clayey gravel	SC	Clayey sand
GC-GM	Silty, clayey gravel	SC-SM	Silty, clayey sand
CL	Lean clay	ML	Silt
CL-ML	Silty clay	MH	Elastic silt
CH	Fat clay	OL or OH	Organic silt or organic clay

* Group names may also include other modifiers, per standard or local practice.

Other soil classification standards may be used, depending on the project requirements. The AASHTO classification system is commonly used for highway design purposes and the USDA soil textural classifications are commonly used for septic (on-site sewage disposal) system design purposes.

APPENDIX II

Summary of Soil Parameters

Summary of Soil Parameters						
Boring No	(see Note 1) Depth Range (ft)	Soil Classification	(see Note 4) Internal Friction Angle (degrees)	Saturated Soil Weight (pcf)	(see Note 2) Moist Soil Weight (pcf)	(see Note 3) Cohesion (psf)
CR - 1	0 - 8	SP	27	111	84	---
	8 - 10	SP	30	122	---	---
	10 - 17	SP	28	114	---	---
	17 - 22	SP	32	126	---	---
	22 -27	SP	38	132	---	---
	27 - 37	SP	31	124	---	---
	37 - 42	Assume CL/CH	--		---	
	42 - 47	CL/CH	---	132	---	3000
	47 - 50	CL/CH	---	125	---	1500
CR-2	0 - 9	SP	28	115	92	---
	9 - 12	SP	32	124	---	---
	12 - 17	SC	38	132	---	---
	17 - 22	SP	38	132	---	---
	22 - 27	SP	32	130	---	---
	27 - 32	SP-SM	30	122	---	---
	32 - 37	ML/CL/CH	---	132	---	3900
	37 - 47	CL/CH	---	120	---	1625
	47 - 50	ML/CL/CH	---	135	---	8000
CR-3	0 - 12	SC/SP-SM	28	115	--	---
	12 - 27	SP-SM/SP	30	123	---	---
	27 - 32	CL/CH	---	115	---	800
	32 - 42	CL/CH	---	132	---	3900
	42 - 57	CL/CH	---	130	---	2300
	57 - 67	CL/CH	---	120	---	1300
	67 - 82	CL/CH	---	125	---	2500
	82 - 87	CL/CH	---	135	---	7800
	87 - 92	CL/CH	---	135	---	5300
	92 - 100	CL/CH	---	135	---	8000
CR-4	0 - 6	SP	28	114	--	---
	6 - 8	SP	32	125	---	---
	8 - 17	SP	32	130	---	---
	17 - 27	SP-SM	30	122	---	--
	27 - 32	CL/CH	---	115	---	800
	32 - 42	CL/CH	---	133	---	3500
	42 - 67	CL/CH	---	125	---	1400
	67 -72	CL/CH	---	132	---	2600
	72 - 82	CL/CH	---	135	---	8000
	82 - 92	CL/CH	---	132	---	2500
	92 - 97	CL/CH	---	135	---	7900
	97 -100	CL/CH	---	132	---	4100
CR-5	0 - 2	SP	27	108	--	---
	2 - 4	SP	29	118	---	---
	4 -6	SP	32	125	---	---
	6 -12	SP	29	118	---	---
	12 - 17	SP	32	131	---	---
	17 - 27	SP	32	127	---	---
	27 - 32	SP-SM	29	118	---	---
	32 -42	ML/CL/CH	---	132	---	2500
	42 -47	CL/CH	---	135	---	7900
	47 - 52	CL/CH	---	115	---	800
	52 - 77	CL/CH	---	125	---	1600
	77 - 87	CL/CH	---	135	---	8000
	87 - 97	CL/CH	---	133	---	3500
	97 - 100	CL/CH	---	135	---	7800

Summary of Soil Parameters						
Boring No	(see Note 1) **Depth Range (ft)**	**Soil Classification**	(see Note 4) **Internal Friction Angle (degrees)**	**Saturated Soil Weight (pcf)**	(see Note 2) **Moist Soil Weight (pcf)**	(see Note 3) **Cohesion (psf)**
CR-6	0 - 7	SP	27	111	---	---
	7 - 12	SP	32	127	---	---
	12 - 17	SP	28	115	---	---
	17 - 27	SP/SP-SM	31	124	---	---
	27 - 32	CL/CH	---	125	---	1400
	32 - 37	CL/CH	---	135	---	7800
	37 - 42	CL/CH	---	135	---	5600
	42 - 47	CL/CH	---	135	---	7800
	47 - 67	CL/CH	---	130	---	2100
	67 - 92	CL/CH	---	135	---	8000
	92 - 100	CL/CH	---	135	---	4500
CR-7	0 - 6	SW/SP	27	105	---	---
	6 - 17	SP	29	119	---	---
	17 - 27	CL/CH	---	118	---	900
	27 - 32	ML	---	135	---	7800
	32 - 37	CL/CH	---	135	---	5900
	37 - 42	CL/CH	---	135	---	8000
	42 - 62	CL/CH	---	130	---	2000
	62 - 67	CL/CH	---	135	---	7800
	67 - 72	CL/CH	---	135	---	5300
	72 - 77	ML	---	135	---	7800
	77 - 82	CL/CH	---	135	---	4500
	82 - 92	CL/CH	---	132	---	3000
	92 - 97	CL/CH	---	135	---	8000
	97 - 100	CL/CH	---	131	---	2800
CR-8	0 -12	SC/SP	27	108	80	---
	12 - 32	SP	31	124	---	---
	32 - 47	ML/CL/CH	---	130	---	2250
	47 -52	CL/CH	---	135	---	7750
	52 - 57	CL/CH	---	132	---	2750
	57 - 62	CL/CH	---	135	---	10250
	62 - 72	CL/CH	---	125	---	1750
	72 - 97	CL/CH	---	133	---	3125
	97 - 100	CL/CH	---	135	---	7750
CR-9	0 - 6	SP	29	118	96	---
	6 - 8	SP	32	125	---	---
	8 - 27	SP	28	115	---	---
	27 - 37	SP	30	123	---	---
	37 - 42	SM	27	105	---	---
	42 - 50	SM/SP	33	127	---	---

Notes: pcf = pounds per cubic foot psf = pounds per square foot

(1) Depth is below land surface at MA-1, MA-2 and MA-9; and below the bay bottom at the

(2) Estimate for a drained soil above the groundwater table.

(3) No value indicates a soil that is generally considered cohesionless.

(4) The values listed above are based upon emperical correlations with the average soil conditions encountered. Appropriate saftey factors should be used with these values.

(5) The soil layers presented above are generalized and should be used for design purposes only. The above values should not be used to assess constructability of the proposed

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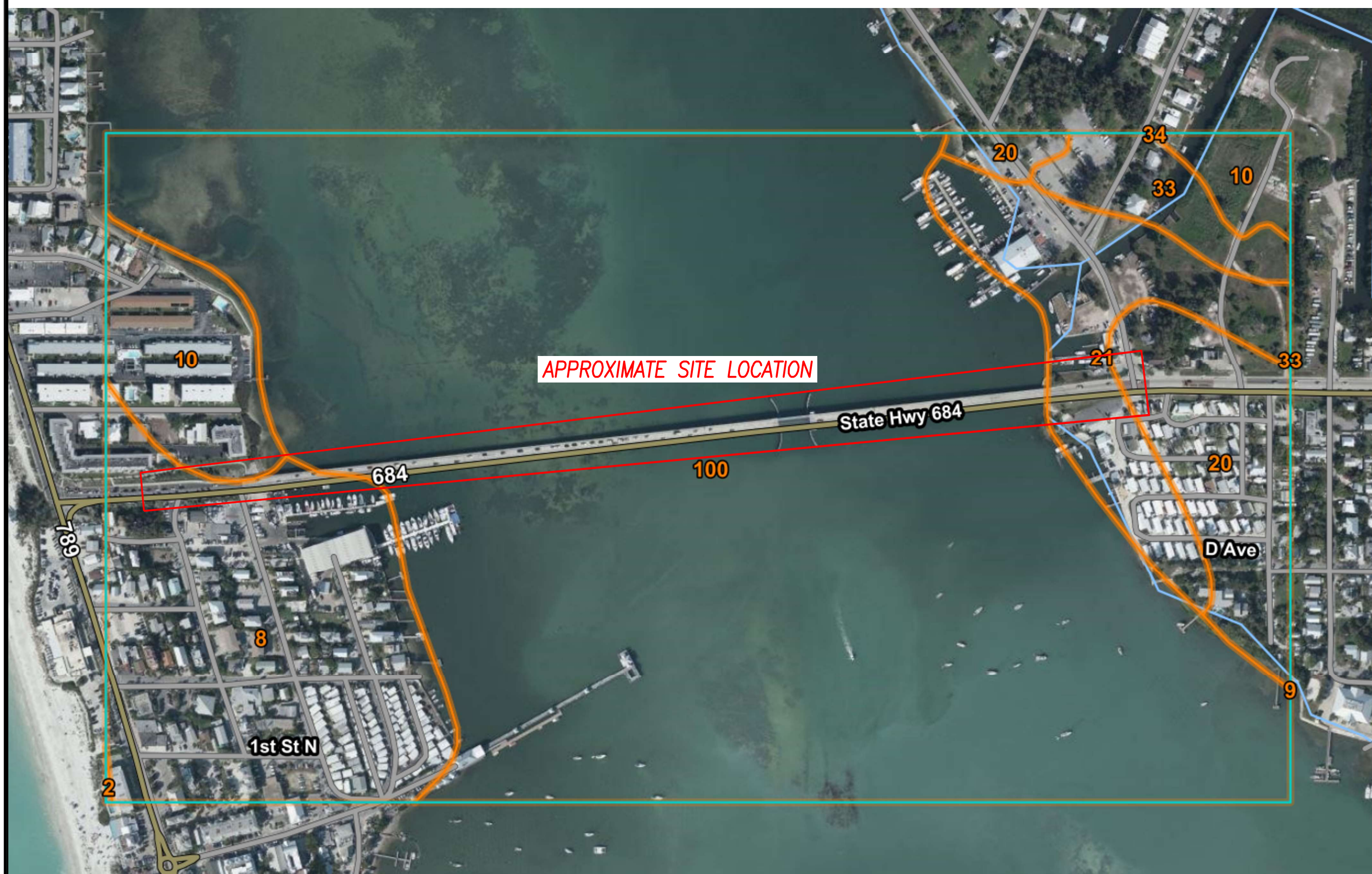
FILE NO. 21-7124	APPROVED BY:	FIGURE: 1B



Map Unit Legend

Map Unit Symbol	Map Unit Name
2	Beaches
8	Canaveral fine sand, 0 to 5 percent slopes
9	Canaveral sand, filled
10	Canaveral sand, organic substratum

Map Unit Legend

Map Unit Symbol	Map Unit Name
20	EauGallie–EauGallie wet, fine sand, 0 to 2 percent slopes
21	Estero muck, tidal, 0 to 1 percent slopes
33	Myakka fine sand, tidal
34	Okeelanta muck, tidal

Map Source: NRCS Web Soil Survey

N

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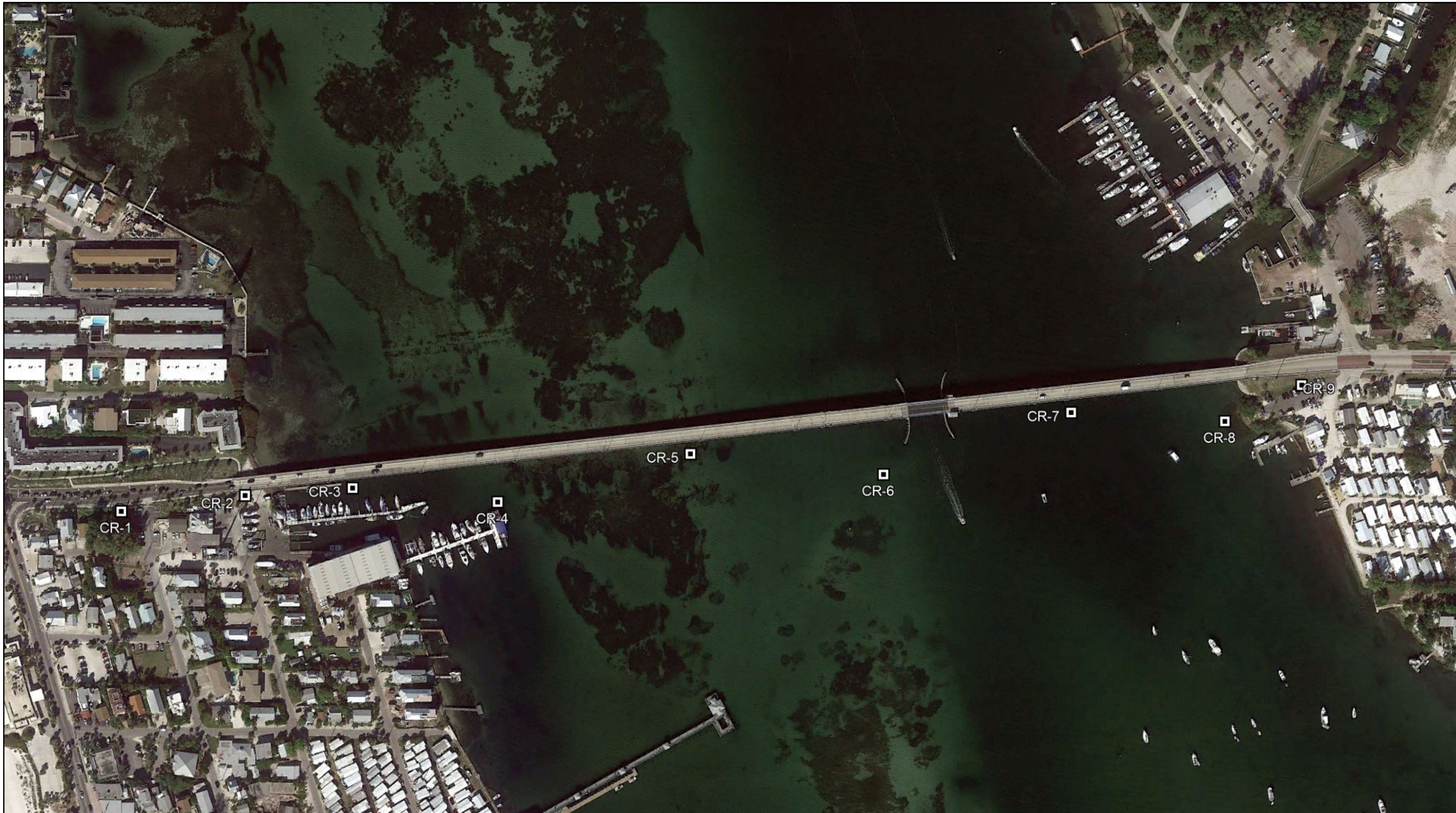

▣ TEST BORING LOCATIONS

N

Base Aerial From Google Earth Pro

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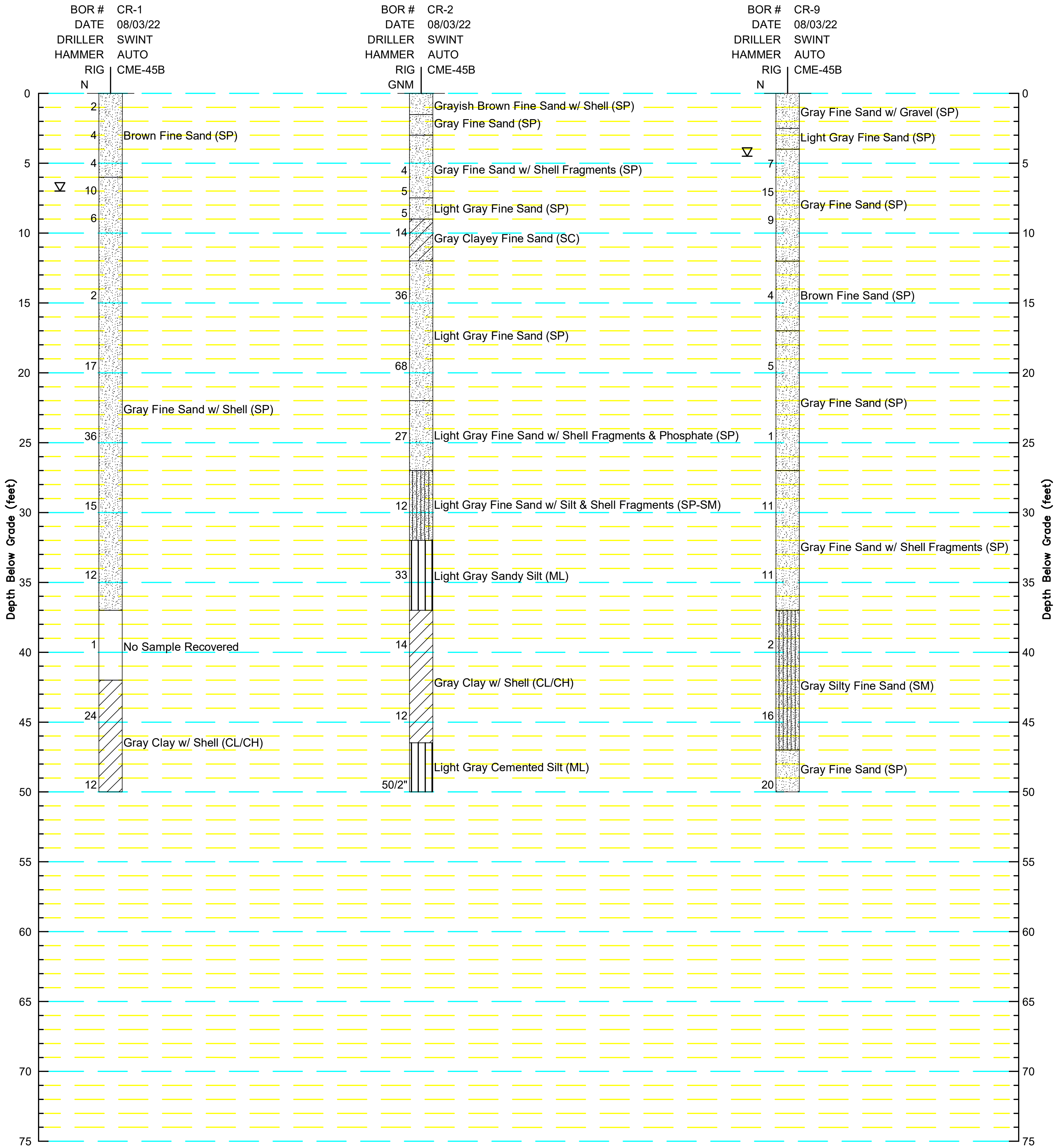
FILE NO. 21-7124	APPROVED BY:	FIGURE: 3B



LEGEND

∇ GROUNDWATER LEVEL MEASURED ON DATE DRILLED

SPT N VALUES CONVERTED TO EQUIVALENT SAFETY HAMMER

WC WATER CONTENT (%)

200 FINES PASSING THE NO. 200 SIEVE (%)

GNM GROUNDWATER NOT MEASURED

Cemented or calcareous soils were encountered within the borings. This material may behave as rock and may contain rock inclusions. Excavations into or through cemented or calcareous soils may be difficult and may require non-conventional construction techniques and specialized equipment.

GRANULAR MATERIALS–RELATIVE DENSITY	SPT (BLOWS/FOOT)
VERY LOOSE	LESS THAN 4
LOOSE	4–10
MEDIUM DENSE	10–30
DENSE	30–50
VERY DENSE	GREATER THAN 50

SILTS AND CLAYS CONSISTENCY	SPT (BLOWS/FOOT)
VERY SOFT	LESS THAN 2
SOFT	2–4
FIRM	4–8
STIFF	8–15
VERY STIFF	15–30
HARD	GREATER THAN 30

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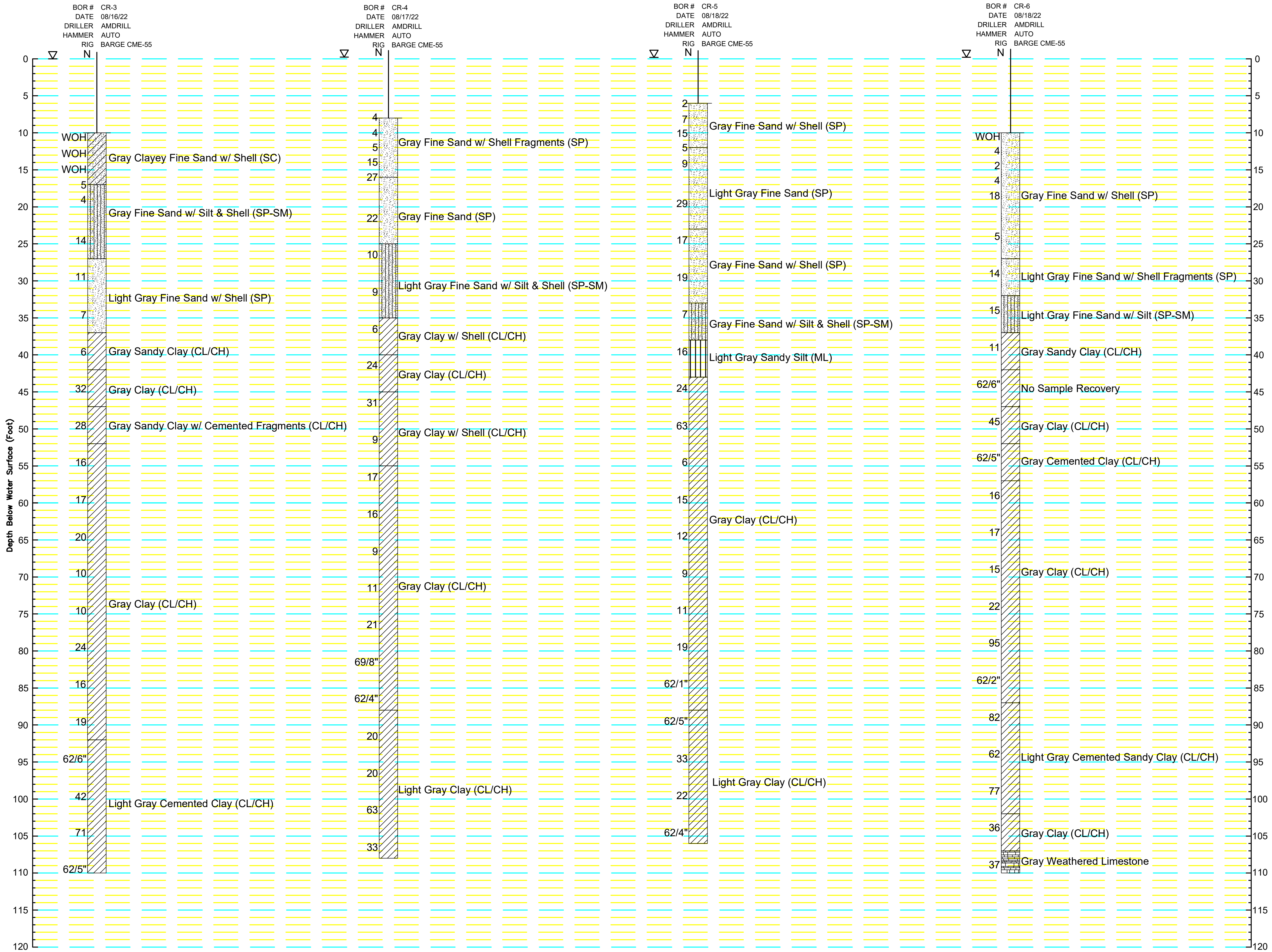

LEGEND

∇ GROUNDWATER LEVEL MEASURED ON DATE DRILLED

SPT N VALUES CONVERTED TO EQUIVALENT SAFETY HAMMER

WC WATER CONTENT (%)

200 FINES PASSING THE NO. 200 SIEVE (%)

GNM GROUNDWATER NOT MEASURED

Cemented or calcareous soils were encountered within the borings. This material may behave as rock and may contain rock inclusions. Excavations into or through cemented or calcareous soils may be difficult and may require non-conventional construction techniques and specialized equipment.

GRANULAR MATERIALS– RELATIVE DENSITY	SPT (BLOWS/FOOT)
VERY LOOSE	LESS THAN 4
LOOSE	4–10
MEDIUM DENSE	10–30
DENSE	30–50
VERY DENSE	GREATER THAN 50

SILTS AND CLAYS CONSISTENCY	SPT (BLOWS/FOOT)
VERY SOFT	LESS THAN 2
SOFT	2–4
FIRM	4–8
STIFF	8–15
VERY STIFF	15–30
HARD	GREATER THAN 30

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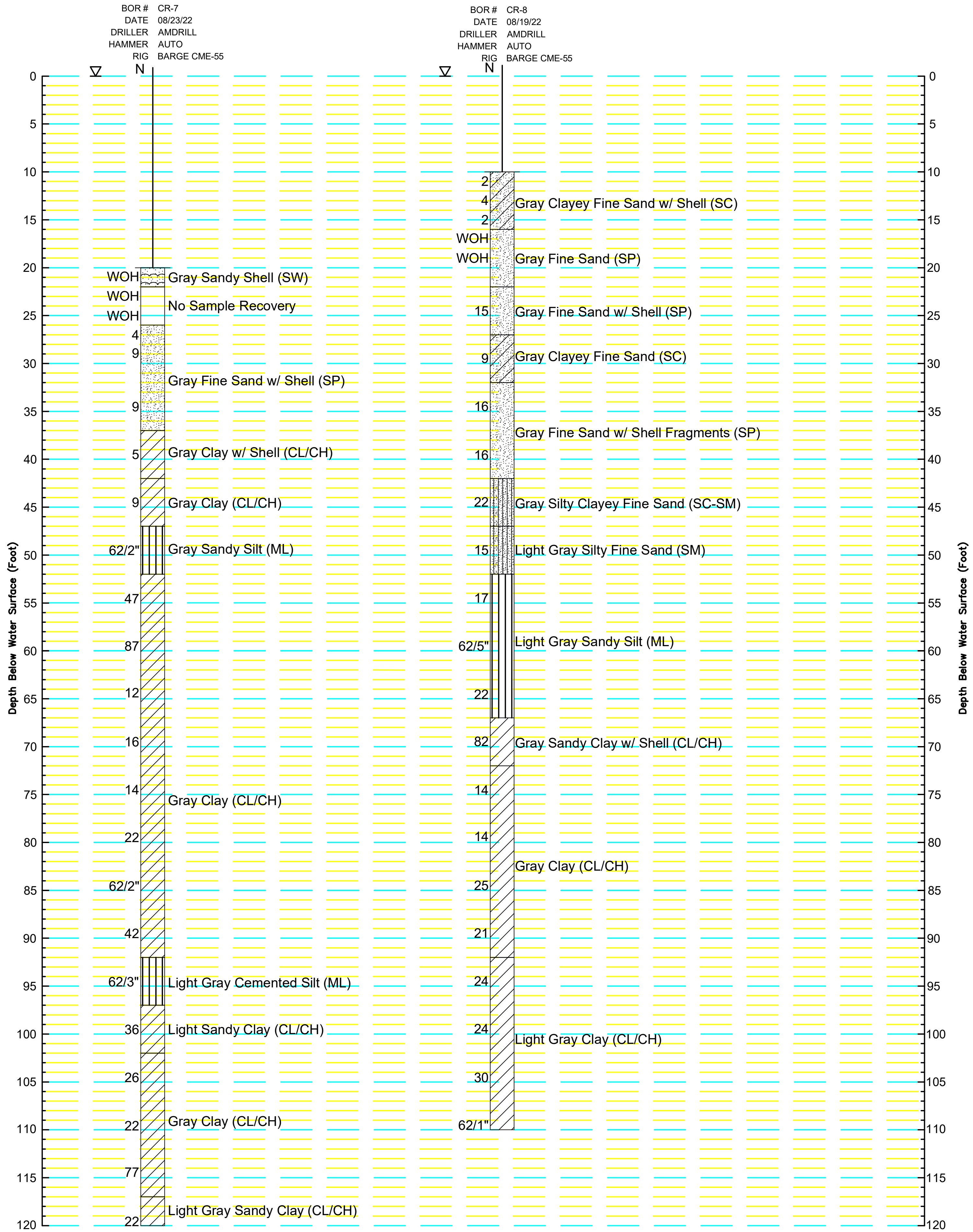

LEGEND

▽ GROUNDWATER LEVEL MEASURED ON DATE DRILLED

SPT N VALUES CONVERTED TO EQUIVALENT SAFETY HAMMER

WC WATER CONTENT (%)

200 FINES PASSING THE NO. 200 SIEVE (%)

GNM GROUNDWATER NOT MEASURED

Cemented or calcareous soils were encountered within the borings. This material may behave as rock and may contain rock inclusions. Excavations into or through cemented or calcareous soils may be difficult and may require non-conventional construction techniques and specialized equipment.

GRANULAR MATERIALS–RELATIVE DENSITY	SPT (BLOWS/FOOT)
VERY LOOSE	LESS THAN 4
LOOSE	4–10
MEDIUM DENSE	10–30
DENSE	30–50
VERY DENSE	GREATER THAN 50

SILTS AND CLAYS CONSISTENCY	SPT (BLOWS/FOOT)
VERY SOFT	LESS THAN 2
SOFT	2–4
FIRM	4–8
STIFF	8–15
VERY STIFF	15–30
HARD	GREATER THAN 30

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