I-526/S-97 (Long Point Road)/Wando Port Interchange Improvement

Mt. Pleasant, South Carolina

GEOTECHNICAL BASELINE REPORT

Prepared By

CDM Smith

For:

South Carolina Department of Transportation

Original Issue: November 5, 2024

Revised September 9, 2025

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MT. PLEASANT, SOUTH CAROLINA

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Table of Contents

List of Figures

List of Tables

Appendices

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Section 1 - Introduction

Project Description

CDM Smith is providing design services to the South Carolina Department of Transportation (SCDOT) related to the proposed construction on I-526 of a new partial interchange and/or modification of an existing interchange at Long Point Road in Charleston County, South Carolina. The study area will extend approximately 4,500 feet west and 3,000 feet east of I-526 along S-97 Long Point Road and approximately one mile north and south of S-97 Long Point Road along I-526 as noted in **Figure 1-1** below:

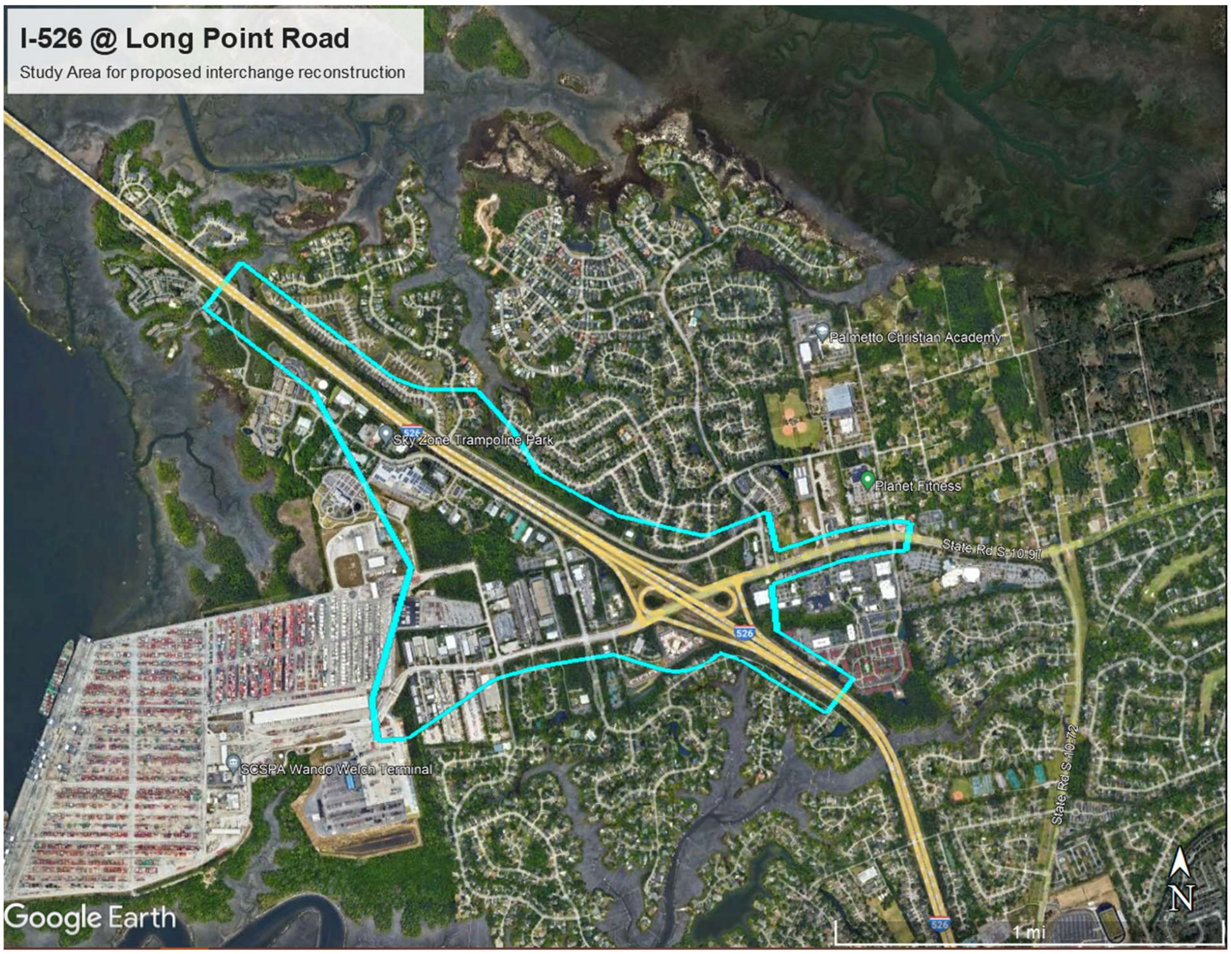


Figure 1-1 Site Limits

Proposed Construction

The proposed I-526 improvements begin approximately 1.3 miles to the northwest of the interchange with Long Point Road and continue past Long Point Road approximately 0.5 miles to the southeast. The improvements include the adjustment/relocation of 8 existing alignments (Relocated Line 1 through 6, Relocated Long Point Road, and Relocated Wando Park Boulevard) and three new alignments (Line A and Line B, which convene to create the Wando Port

Connector). **Figure 1-2** shows the proposed alignments and **Figure 1-3** shows the proposed wall and bridge locations.

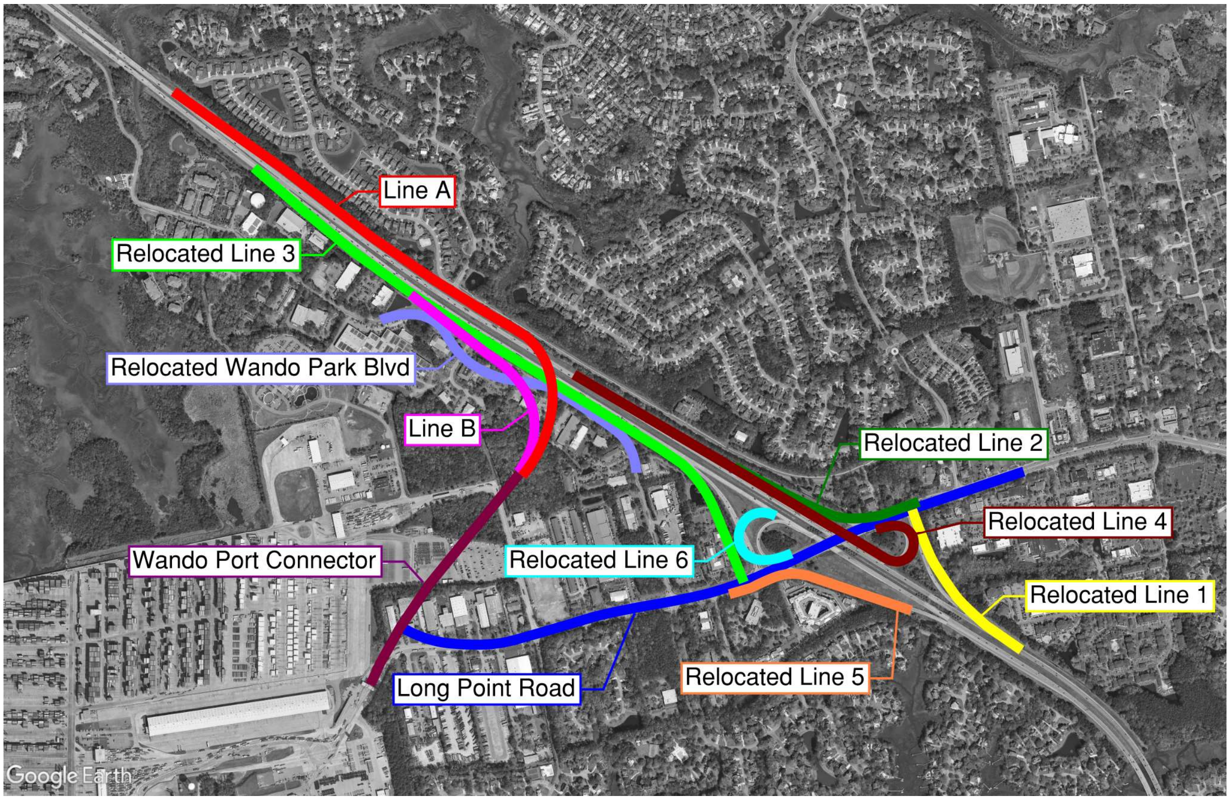


Figure 1-2 Alignment Reference

The Relocated Lines 1 through Line 6 are access ramps to and from I-526 at the intersection with Long Point Road. New MSE walls are proposed along Relocated Line 2, Relocated Line 3, and Relocated Line 4 as grade separation between adjacent alignments. Wall heights vary up to 25 feet in these locations. For Relocated Line 1, a new bridge is over an existing wetland area. The proposed Relocated Line 1 bridge is 365 feet in length. The relocation of Line 4 has a proposed bridge over the existing Long Point Road with a bridge length of 247.8 feet.

The existing Wando Park Blvd will be relocated to make room for the new Line A approach wall and bridge. Long Point Road will have new tie-ins to the access ramps of Lines 1-6 and will have a new section of alignment to tie in with the Wando Port Connector.

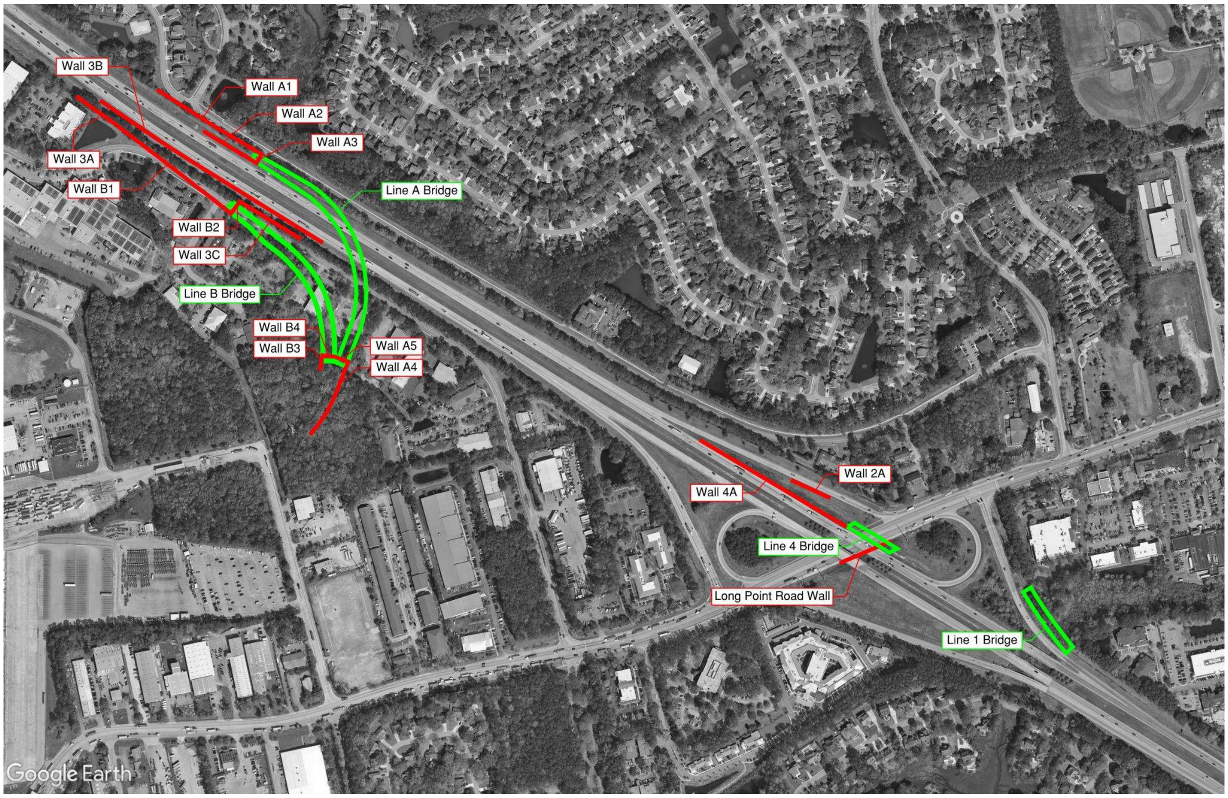


Figure 1-3 Walls and Bridges

The three new alignments will provide direct access on and off I-526 from the South Carolina Ports Authority Wando Welch Terminal, currently located at the end of Long Point Road. Both Line A and Line B require new bridges (approximately 1,260 and 930 feet in length, respectively) over the existing I-526 and Relocated Wando Park Blvd. MSE walls are proposed at the approaches for Lines A and B. Wall heights range up to 25 feet in these locations.

Elevation Datum

Survey information was provided by CDM Smith licensed professional surveyors. The vertical datum used in this project is the 1988 North American Vertical Datum (NAVD88).

Objective and Scope

The objective of this work was to conduct a geotechnical subsurface investigation and Baseline Report related to the title project, in general accordance with the requirements of SCDOT *Geotechnical Design Manual (GDM), 2022*. This report presents the subsurface information collected during the geotechnical subsurface investigation.

Report Limitations

The Geotechnical Baseline Report (GBR) has been prepared for the I-526 Lowcountry Corridor East Improvement Project Phase II in Charleston County, South Carolina, as understood at this time and described in this report. This baseline represents a contractual definition of what ground conditions are assumed to be encountered with respect to the differing site condition (DSC) clause. The provisions in this baseline are not a warranty that the baseline conditions will in fact, be encountered. During construction of the bridge and roadway features, this report will be the basis used by the Owner, when interpreting the DSC clause.

The Contractor is responsible to read and understand the GBR and all other Contract Documents in their entirety, consider them when developing their project approach, construction means and methods, equipment selection, and in planning and bidding all other elements of the work. Bidders should have on staff or retain the services from a qualified engineering geologist/geotechnical engineer to help evaluate and interpret this document and related geologic/geotechnical documents.

In establishing these baselines, the Engineer considered available data and past construction experience in similar ground conditions. Ground behavior will be influenced by the construction sequence and methods employed by the Contractor, as well as the Contractor's equipment and workmanship. It has been assumed that the level of workmanship will be consistent with what can be reasonably expected from an experienced and qualified Contractor.

This report has been prepared in accordance with generally accepted engineering practices and the SCDOT *GDM*. The data provided in this report are not intended for the preparation of construction plan documents. No other warranty, express or implied, is made.

Section 2 - Field Exploration

General

An initial geotechnical subsurface investigation was conducted by S&ME between March 1st and May 9th, 2023. An additional phase of subsurface investigation was performed by S&ME between June 17th and June 24th, 2025. The investigations included soil borings, soundings, and laboratory testing. The findings of the geotechnical explorations were provided in a report titled *Geotechnical Subsurface Data Report (GSDR Rev2), I-526 Long Point Road Interchange, Charleston, South Carolina, S&ME Project No. 200424A*, Dated August 26, 2025.

Boring and Sampling

Field tests included Standard Penetration Test (SPT) borings, Cone Penetration Test (CPT) soundings, Seismic CPT soundings, flat Dilatometer Test (DMT) soundings, and hand auger borings with Dynamic Cone Penetration (DCP) tests. The location of field testing can be found in the GSDR.

- Forty-six (46) Standard Penetration Test borings
- Thirty-five (35) Cone Penetration Test soundings
- Seven (7) Flat Dilatometer Test soundings
- Sixteen (16) hand auger and dynamic cone penetrometer testing

Standard Penetration Test Borings

The 46 SPT borings were drilled using truck-mounted and All-Terrain Vehicle (ATV) mounted drilling equipment using rotary wash drilling methods and in general accordance with ASTM D 1586. Detailed descriptions of the SPT boring methods and individual boring logs are included in the Geotechnical Subsurface Data report provided by S&ME Inc.. Borings were backfilled with soil cuttings, bentonite chips, and grout upon completion. Where pavement was cored, the top foot of the pavement was patched with concrete or asphalt.

Cone Penetration Test Soundings

Thirty-five CPT Soundings were advanced using truck-mounted and ATV-mounted rigs. Seismic shear wave velocity measurements were taken at 3 of the 35 locations. A description of the CPT methods and individual logs are presented in the GSDR.

Flat Dilatometer Soundings

Seven DMT Soundings were advanced using the CPT rig. A description of the DMT boring methods and individual logs are presented in the GSDR.

Hand Auger Borings

Sixteen manual hand auger borings were drilled for collecting bulk samples and performing Kessler DCP tests at the proposed pavement subgrade. Detailed descriptions of the boring methods and individual boring logs are presented in the GSDR.

Laboratory Testing

Representative soil samples were identified by S&ME and confirmed by CDM Smith for laboratory testing. A summary of the laboratory testing performed for this project is provided in **Table 2-1** and results are included in the GSDR.

Table 2-1 Laboratory Testing Table

Test Type	Quantity
Natural Moisture Content Tests (ASTM D 2216)	335
Atterberg Limits Tests (ASTM D 4318)	335
Sieve Analysis Tests (ASTM D 6913)	335
Consolidated Undrained Triaxial Compression Tests (ASTM D4767)	2
Standard Proctor Compaction Tests (ASTM D 698)	2
Corrosion Series-pH, Sulfates and Chlorides, and Resistivity (ASTM G51, ASTM D4327, ASTM G187)	14
One-Dimensional Consolidation Test (ASTM D2435)	5

Section 3 - Site and Subsurface Conditions

Regional Geology

The project site is located in the Lower Coastal Plain physiographic province of South Carolina. The site is located within the Wando Formation of the late Pleistocene over the Ashley Formation and Cooper Group referred to as Cooper Marl.

According to "Geology of the Cainhoy, Charleston, Fort Moultrie, and North Charleston Quadrangles" (Weems and Lemon, 1993), near-surface soils consist primarily of Coastal Plain sediments (Pleistocene soils) of Quaternary age consisting of slightly weathered beds of sand, clayey sand, and clay sediments deposited during three late Pleistocene high-sea-level pulses. The accumulations during these pulses are separated by clay beds which were deposited in lagoon-like environments present during the regressions. These soils are underlain by the Tertiary age soils of the Marks Head and Ashley Formations.

The Coastal Plain sediments and Cooper Marl are underlain by Mesozoic/Paleozoic basement rock (granite, schist, and gneiss). At the project site, the thickness of the Coastal Plain sediments is estimated to be about 2,400 to 2,600 feet. **Figure 3-1** depicts the mapping in this area.

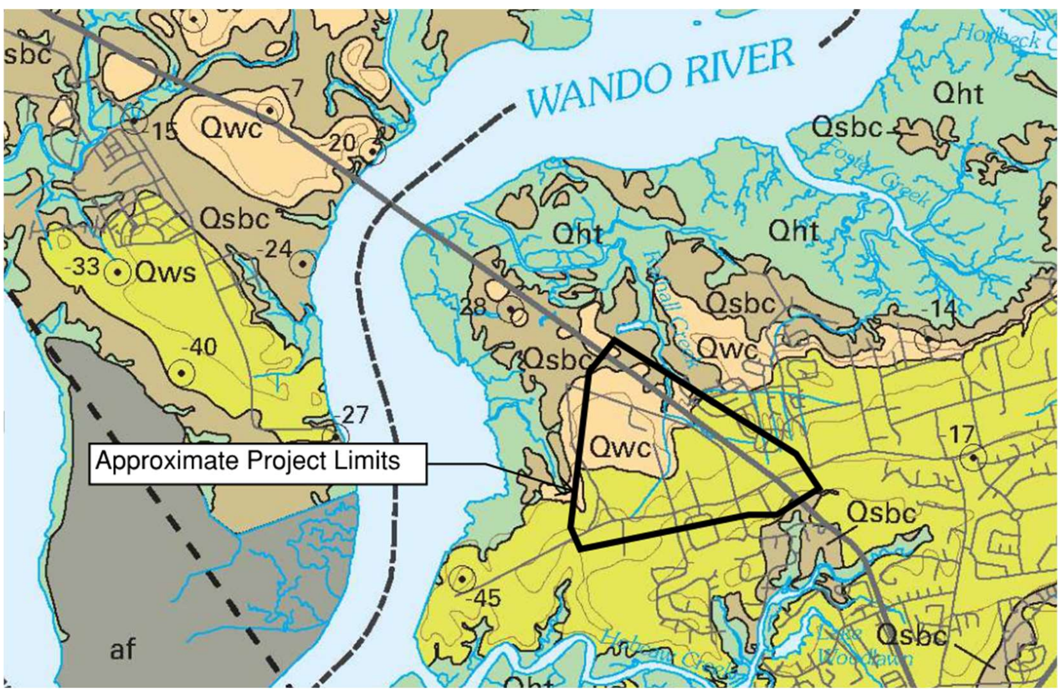


Figure 3-1 Clipping of Surficial Geologic Map of the Charleston Region, Berkeley, Charleston, Colleton, Dorchester, and Georgetown Counties, South Carolina (Weems, et. al. 2014).

Geologic References:
Weems, R., Lemon, E., *Geology of the Cainhoy, Charleston, Fort Moultrie, and North Charleston Quadrangles, Charleston and Berkeley Counties, South Carolina*, 1993.
Weems, R., Lewis, C., Lemon, E., *Surficial Geologic Map of the Charleston Region, Berkeley, Charleston, Colleton, Dorchester, and Georgetown Counties, South Carolina, 2014.*

Soil samples recovered from SPT, or hand auger borings were visually classified in the field and select samples were sent for additional laboratory testing. Boring logs, CPT, and DMT logs are presented in the GSDR. Typically, the subsurface across the site consisted of seven generalized strata for this preliminary analysis. **Figure 3-2** is a clipping from a nearby cross section (A'-B) of the quadrangle (Weems and Lemon, 1993). Later geologic mapping further delineates the layering in our project Area (Weems, et. al., 2014).

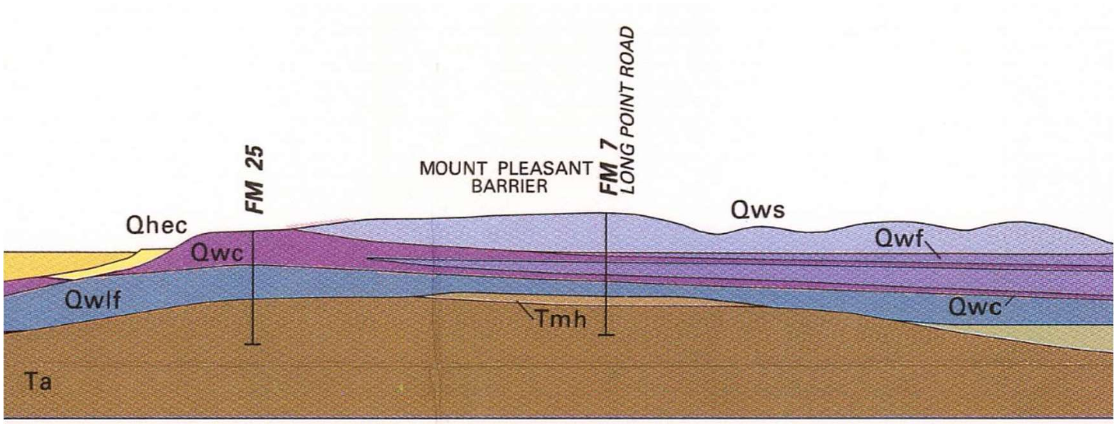


Figure 3-2 Clipping of soil profile A'-B from *Geology of the Cainhoy, Charleston, Fort Moultrie, and North Charleston Quadrangles* (Weems, and Lemon, 1993).

The alignment begins to the northwest in Marsh and Lagoon deposits. These are more recent deposits of Holocene and Upper Pleistocene age of the Quaternary period (2.58 mya to present day). The Holocene tidal-marsh deposits **(Qht)** are described as clayey sand and clay rich in organics and are noted as up to 10 feet thick. Continuing south, we encounter the clayey sand and clay facies **(Qsbc)** that are described as silty to sandy clay that occur beneath about 10 feet in elevation and is noted as up to 10 feet thick. These soils weren't explicitly encountered in our exploration, however, likely exist in areas along the project site.

The Pleistocene sediments include barrier sand facies **(Qws)** consisting of gray to brown quartz sand. In the upper 7 feet, the unit may be yellow and orange and fine-grained, and this unit has been locally mapped up to 25 feet thick.

Beneath the barrier sand unit lies three minor but distinct late-Pleistocene high-sea-level pulses of clayey sand and clay facies **(Qwc)** which consists of a mottled pale grayish orange to gray clay. This unit is reportedly up to 35 feet thick and is sometimes covered by less than 3 feet of modern swamp deposits. Interbedded between layers of the clayey sand and clay faces are lenses of fossiliferous shelf-sand **(Qwf)** which are fine to medium-grained, phosphatic, bioturbated sands and have been observed up to 20 feet thick.

Below the upper clay and sand lens deposits is another fossiliferous shelf-sand **(Qwlf)** from the early Pleistocene age which is lithologically similar to the younger shelf sands. These older sands are fine to medium-grained, phosphatic, bioturbated sands. The Ashley Formation of the Cooper Group **(Ta)** is light olive-brown, fine-grained, and phosphatic of upper Oligocene, Tertiary age. The formation consists of relatively homogeneous calcareous, micro fossiliferous, silty and sandy clays. Locally known as Cooper Marl, the thickness of this unit is mapped up to 125 feet. This material is well-researched by borings and load tests as a part of nearby bridge projects.

Subsurface Conditions

The soils encountered within the project site can be generalized into seven different strata. See **Table 3-1** for general properties.

Table 3-1 Soil Stratification Table

Geology	Uppermost Elevation of Top of Layer	USCS Classification	Typical SPT-N Values (bpf)	Comments
Pleistocene	+30	SC, SM, SC-SM, SC	0- 75	"Upper Sand" (Qws)
	+16	CL, CH, MH, ML, SC	0 - 20	"Upper Clay" (Qwc)
	+3	SC, SM, SP-SM, SC-SM	0 - 35	"Sand Lens" (Qwf)
	-2	CL, CH, MH,SC	0 - 14	"Lower Clay" (Qwc)
	-6	SP, SC, SM, SP-SM, SP-SC, SC-SM	0 - 90	"Lower Sand" (Qwlf)
Pre-Pleistocene	-24	SC, CL, ML, CH, MH	8	"Young Marl" (Tmh)
	-23	SC, CL, ML, CH, SM	6-100	"Cooper Marl" (Ta)

The seven strata were generally observed throughout the site and in each site group and are described in the following sections. **Figures 3-3 through 3-8** present Index Properties for each site group and **Figures 3-10 through 3-19** present generalized subsurface profiles for the alignments.

A preliminary seismic hazard analysis was performed using the CPT sounding data and an acceleration design response spectrum (ADRS) Curve received from SCDOT. The ADRS curve was developed by SCDOT using the shear wave velocity data from sounding SCPT-11. The analysis included a preliminary screening of the sites susceptibility to liquefaction and cyclic softening following the procedures in Chapter 13 of the SCDOT *GDM*. The ADRS curve is presented in **Appendix A**.

Upper Sand

The upper sand layer encountered is predominantly recent to Pleistocene-age barrier sand. The upper sand layer ranged from 6 to 32 feet thick with an average thickness of 12.5 feet.

This layer generally consisted of very loose to medium dense poorly graded sands (SP), sands with varying amounts of clay and silt (SC, SM, SC-SM, SC-SM). There was also a relatively thin layer of lean or fat clay (CL or CH) observed in some borings.

Figure 3-3 displays the index properties: Atterberg Limits, or Limit states (LL and PL), Moisture content (%w), and percent passing #200 sieve (#200). As shown in the figure, the variability of the Upper Sand is relatively consistent. More than 50% of the samples tested were non-plastic. The majority of the tested samples contained less than 50 percent fines (by weight), indicating predominately sand materials.

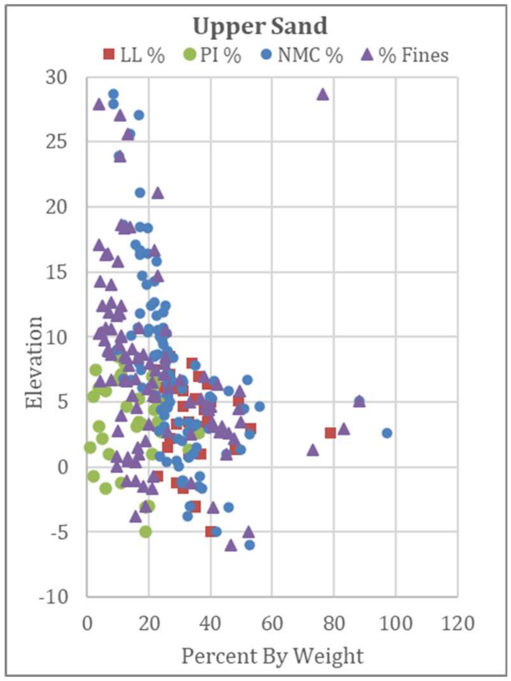


Figure 3-3 Upper Sand – Index Properties (%) with Elevation

For this layer, the Liquid Limit ranged from 22 to 81 percent with an average of 35.5 percent, and the Plasticity Index ranged from 1 to 58 percent with an average of 15.5 percent. Much of the Upper Sand layer is below the groundwater table. SPT N-values for the Upper Sand varied from 1 to 75 blows per foot. Portions of the Upper Sand are very loose to loose and are susceptible to cyclic liquefaction.

Upper Clay

The upper clay layer encountered is predominantly marine deposits of late-Pleistocene sandy clay and clay. The upper clay layer ranged from 1.5 to 26 feet thick with an average thickness of 9.7 feet. This layer is predominantly of high plasticity clay (CH) with some lean clay (CL).

Figure 3-4 displays the index properties: Atterberg Limits, or Limit states (LL and PL), Moisture content (%w), and percent passing #200 sieve (#200). As shown in the figure, the variability of the Upper Clay is relatively consistent.

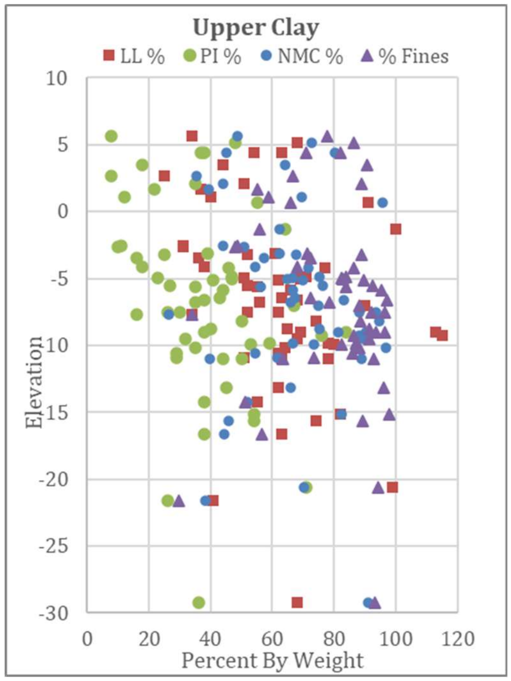


Figure 3-4 Upper Clay – Index Properties (%) with Elevation

For this layer, the Liquid Limit ranged from 25 to 115 percent with an average of 62.5 percent, and the Plasticity Index ranged from 8 to 84 percent with an average of 38 percent. The natural moisture content in many samples was near or above the Liquid Limit and the corresponding liquidity index was an average of 1.2 indicating soils have a near-liquid consistency. SPT N-values for the Upper Clay varied from 0 to 20 blows per foot. Although cohesive soils are not susceptible to cyclic liquefaction, an earthquake event may cause cyclic softening in the soft deposits.

Sand Lens

The late Pleistocene Sand Lens encountered ranged from 2 to 18 feet thick with an average thickness of about 6.5 feet. This layer generally consisted of very loose to medium-dense sands with varying amounts of clay and silt (SC, SM, SP-SM, SC-SM).

Figure 3-5 displays the index properties: Atterberg Limits, or Limit states (LL and PL), Moisture content (%w), and percent passing #200 sieve (#200). As shown in Figure 3-6, the variability of the Sand Lens is relatively consistent. The tested samples generally contained less than 40 percent fines (by weight), indicating predominately sand materials. Approximately 50% of the samples tested were determined to be non-plastic.

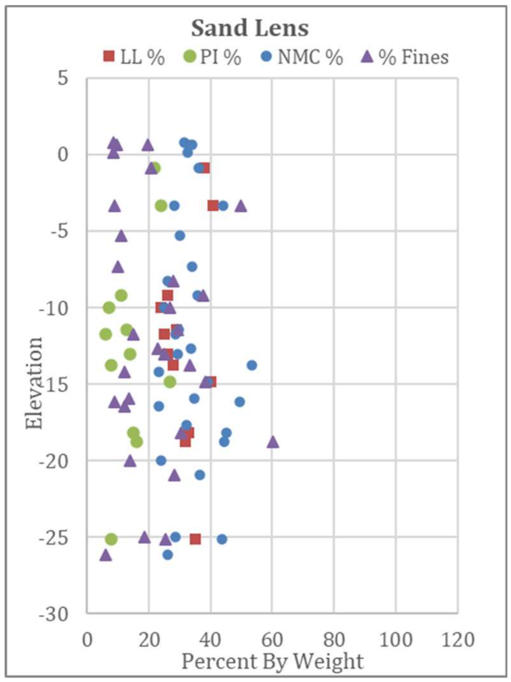


Figure 3-5 Sand Lens – Index Properties (%) with Elevation

For this layer and samples determined as plastic, the Liquid Limit ranged from 24 to 40 percent with an average of 31.5 percent, and the Plasticity Index ranged from 7 to 27 percent with an average of 14.5 percent. The Sand Lens layer is saturated below the groundwater table and is generally loose. SPT N-values for the Sand Lens varied from 0 to 35 blows per foot. Based on the preliminary seismic hazard analysis, these soils are susceptible to cyclic liquefaction.

Lower Clay

The Lower Clay is similar geologically to the upper clay encountered and is predominantly marine deposits of late-Pleistocene sandy clay and clay. The Lower Clay layer ranged from 2 to 26 feet thick with an average thickness of 10 feet.

This layer is predominantly of high plasticity clay (CH) with some lean clay (CL) and high plasticity silt (MH). **Figure 3-6** displays the index properties: Atterberg Limits, or Limit states (LL and PL), Moisture content (%w), and percent passing #200 sieve (#200). As shown in the figure, Lower Clay is relatively consistent.

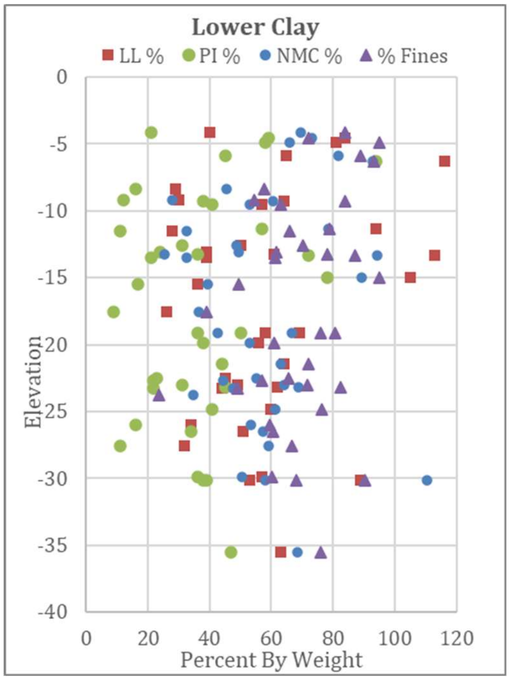


Figure 3-6 Lower Clay – Index Properties (%) with Elevation

For this layer, the Liquid Limit ranged from 26 to 116 percent with an average of 56 percent, and the Plasticity Index ranged from 9 to 94 percent with an average of 35 percent. The natural moisture content in many samples was near or above the Liquid Limit and the corresponding liquidity index was an average of 1.3indicating soils have a near-liquid consistency. SPT N-values for the Lower Clay varied from 0 to 14 blows per foot. Although cohesive soils generally do not experience cyclic liquefaction, an earthquake event will cause cyclic softening of the very soft and soft soils, thereby reducing the soil strength.

Lower Sand

The Lower Sand is a fossiliferous shelf-sand likely from the early Pleistocene age and lithologically similar to the Upper Sand and Sand Lens layers. The Lower Sand encountered ranged from 2 to 27 feet thick with an average thickness of about 11.5 feet.

This layer generally consisted of very loose to medium-dense sands with varying amounts of clay and silt (SP, SC, SM, SP-SM, SP-SC, SC-SM).

Figure 3-7 displays the index properties: Atterberg Limits, or Limit states (LL and PL), Moisture content (%w), and percent passing #200 sieve (#200). As shown in Figure 3-8, the variability of the Sand Lens is relatively consistent. The tested samples generally contained less than 50 percent fines (by weight), indicating predominately sand materials. Approximately 50% of the samples tested were determined to be non-plastic.

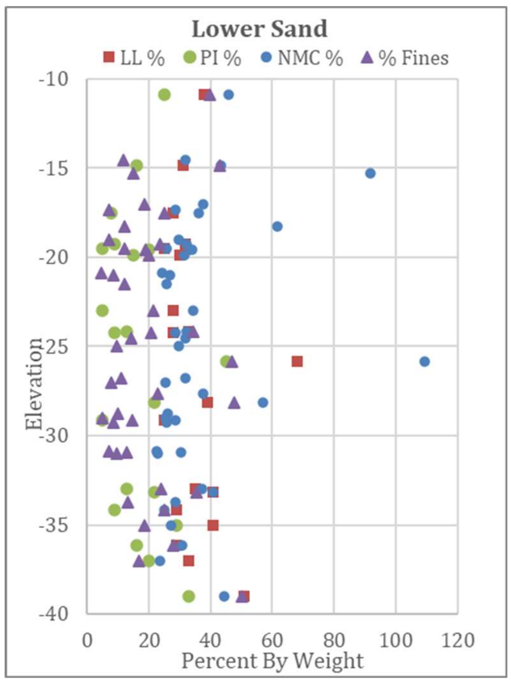


Figure 3-7 Lower Sand – Index Properties (%) with Elevation

For this layer and samples determined as plastic, the Liquid Limit ranged from 25 to 68 percent with an average of 36.5 percent, and the Plasticity Index ranged from 5 to 45 percent with an average of 18 percent. Note that the Liquid Limits are often exceeded by the natural moisture content. SPT N-values for the Lower Sand varied from 0 to 90 blows per foot. Based on the preliminary seismic hazard analysis, the very loose to loose soils, low fines content soils, and low plasticity soils will be susceptible to cyclic liquefaction.

Young Marl

The Young Marl encountered ranged from 3 to 9 feet thick with an average thickness of about 5.5 feet. This layer was observed as thin and was encountered in few borings with SPT sampling, therefore laboratory test results are sparse. The Young Marl was identified in 16 of the CPT soundings. The identification of this layer is primarily from the gradual increase of the porewater pressure from CPT soundings as compared to the rapid increase seen in the older Cooper Marl. The sample tested was high plasticity silt (MH). Due to the age of the deposit (Pre-Pleistocene) and the generally clay-like behavior, the Cooper Marl is not considered susceptible to cyclic liquefaction or cyclic softening.

Figure 3-8 displays the index properties: Atterberg Limits, or Limit states (LL and PL), Moisture content (%w), and percent passing #200 sieve (#200). As shown in Figure 3-8, the variability of the Sand Lens is consistent.

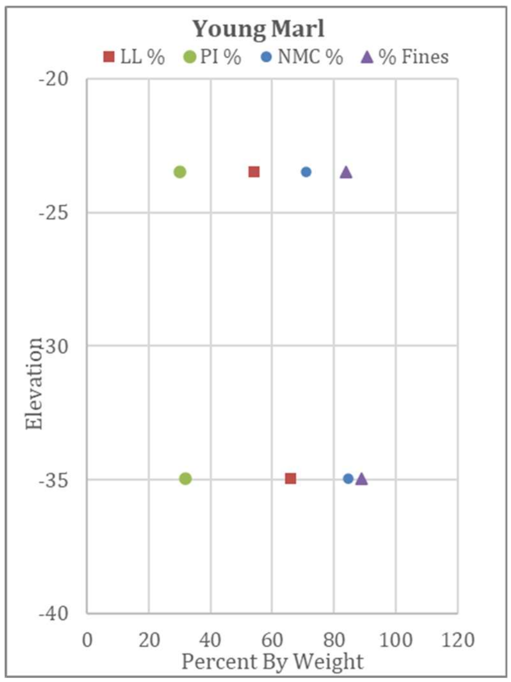


Figure 3-8 Young Marl – Index Properties (%) with Elevation

For this layer, the Liquid Limit for the sample tested was 66 percent, and the Plasticity Index was 32.

Cooper Marl

The deepest geologic layer encountered in the Ashley Group is the Cooper Marl. The Cooper Marl was encountered at depths ranging from 27 ft-bgs to 67 ft-bgs and was not fully penetrated in any test boring. Generally, this layer consisted of loose to medium dense, clayey, and silty sand (SC, SC-SM); and soft to firm, sandy elastic silt, sandy silt, and sandy lean, lean, and fat clay (ML, MH, CL, CH). An identifying characteristic of the Marl is the spike in pore water pressure measurements at the Marl interface. This pore pressure spike can be observed in the CPT sounding logs, shown in the appendices. Due to the age of the deposit (Pre-Pleistocene) and the generally clay-like behavior, the Cooper Marl is not considered susceptible to cyclic liquefaction or cyclic softening.

Figure 3-9 shows the variability of the marl index properties with depth. The fines contents range from 12 to 92%, and the moisture content (%w) is below the liquid limit (LL) and above the plastic limit (PL). For this layer, the Liquid Limit ranged from 21 to 102 percent with an average of 55.5 percent, and the Plasticity Index ranged from 1 to 65 percent with an average of 30 percent. SPT N-values for the Upper Sand varied from 5 to 100 blows per foot.

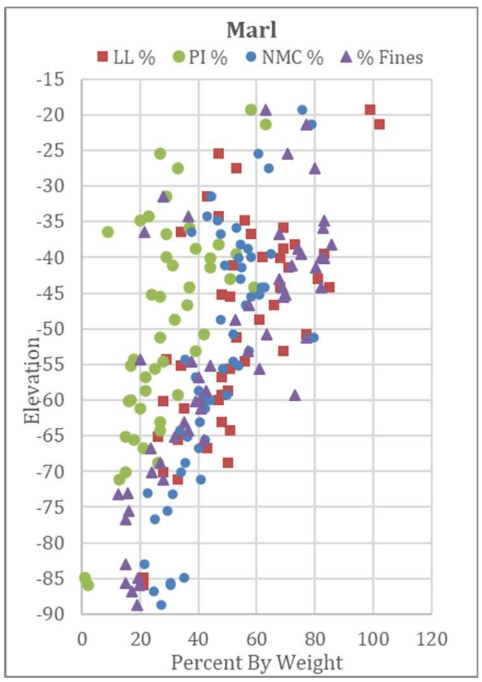
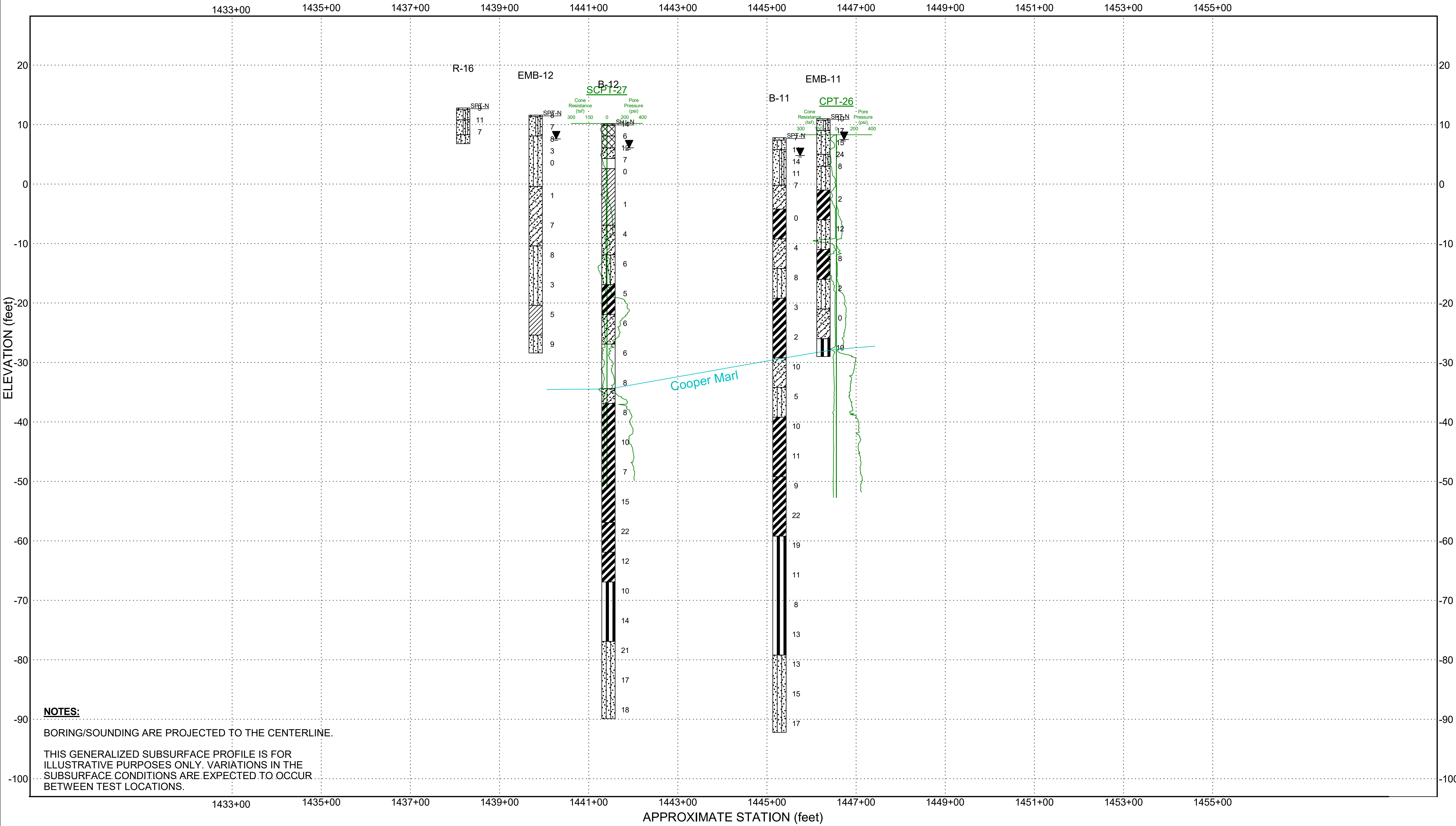


Figure 3-9 Cooper Marl – Index Properties (%) with Elevation

Groundwater Conditions

Groundwater was measured in many of the SPT Borings and estimated in some of the CPT sounding. Groundwater was measured at the time of drilling and after 24 hours at depths corresponding to elevations between 2.3 and 32.7 (NAVD 88), with an average water surface elevation of about 12.2 ft (NAVD88). Seasonal groundwater variations of 6 feet have been observed in a well approximately one mile southeast of the project site.



NOTES:

BORING/SOUNDING ARE PROJECTED TO THE CENTERLINE.

THIS GENERALIZED SUBSURFACE PROFILE IS FOR ILLUSTRATIVE PURPOSES ONLY. VARIATIONS IN THE SUBSURFACE CONDITIONS ARE EXPECTED TO OCCUR BETWEEN TEST LOCATIONS.

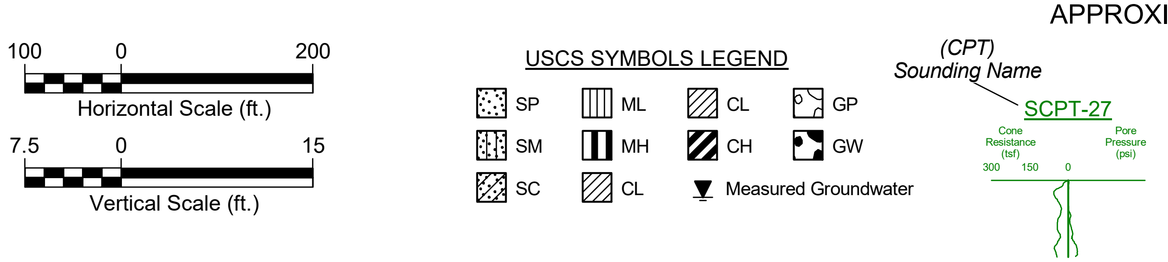
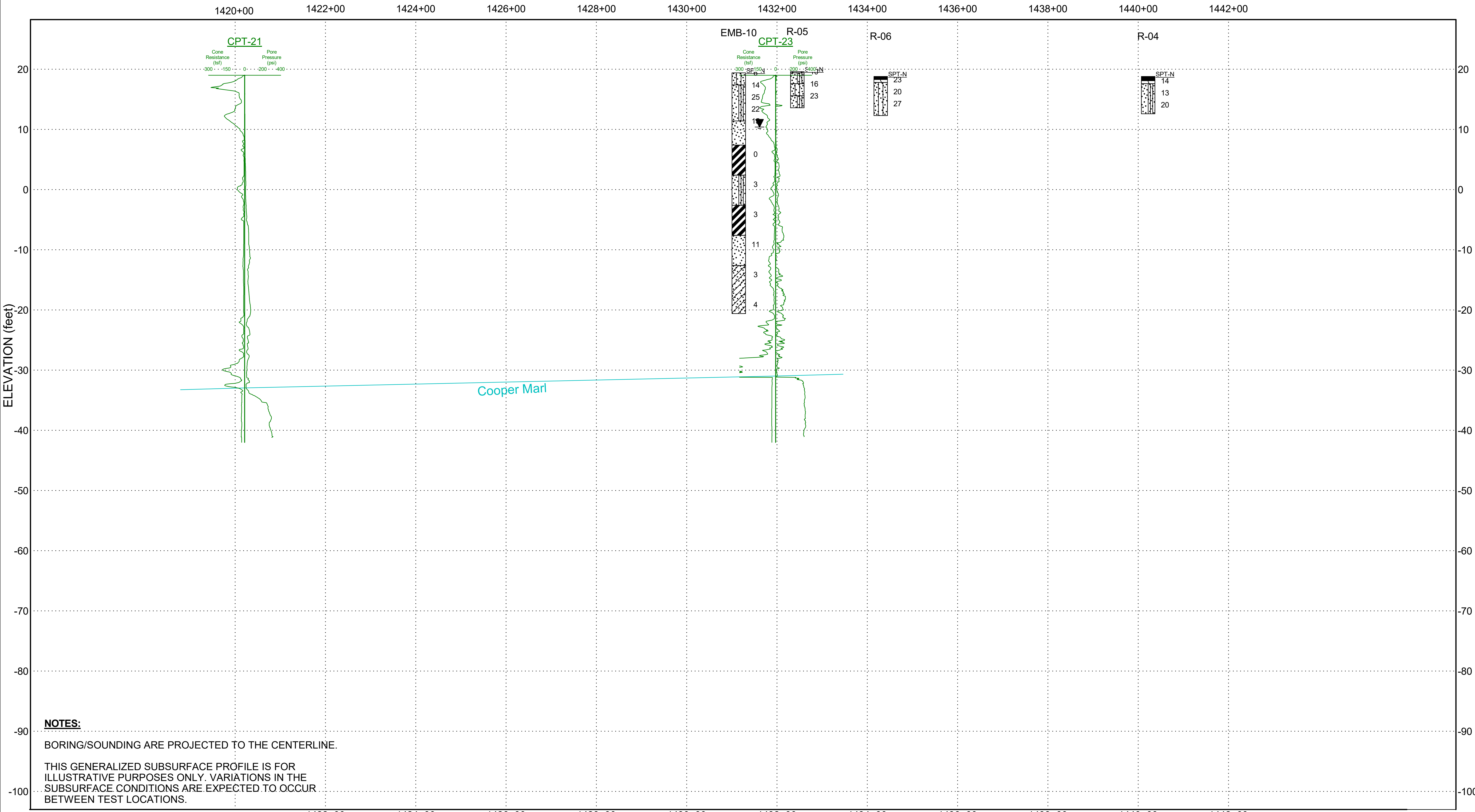


FIGURE 3-10
GENERALIZED SUBSURFACE PROFILE - RELOCATED LINE 1
I-526 LOWCOUNTRY CORRIDOR PHASE 2
CHARLESTON, SOUTH CAROLINA

DRAWN BY: DCP
DATE: 08.14.2023

REVIEWED BY: JM
DATE: 09.18.2023



NOTES:

BORING/SOUNDING ARE PROJECTED TO THE CENTERLINE.

THIS GENERALIZED SUBSURFACE PROFILE IS FOR ILLUSTRATIVE PURPOSES ONLY. VARIATIONS IN THE SUBSURFACE CONDITIONS ARE EXPECTED TO OCCUR BETWEEN TEST LOCATIONS.

100 0 200
Horizontal Scale (ft.)

7.5 0 15
Vertical Scale (ft.)

USCS SYMBOLS LEGEND

SP ML CL GP
SM MH CH GW
SC CL Measured Groundwater

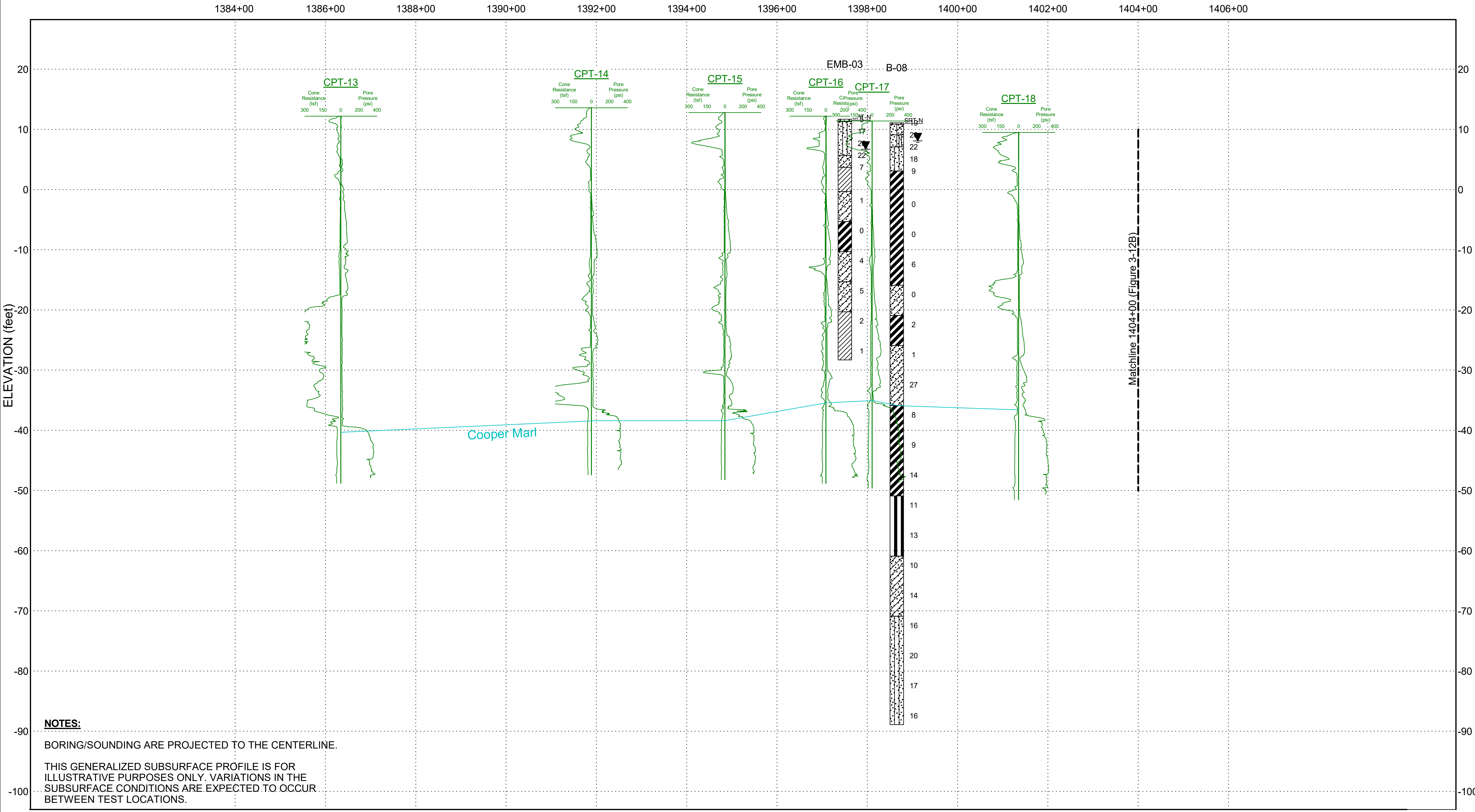
(CPT)
Sounding Name
SCPT-27

(SPT)
Boring Name
R-16

FIGURE 3-11
GENERALIZED SUBSURFACE PROFILE - RELOCATED LINE 2
I-526 LOWCOUNTRY CORRIDOR PHASE 2
CHARLESTON, SOUTH CAROLINA

DRAWN BY: DCP
DATE: 08.14.2023

REVIEWED BY: JM
DATE: 09.18.2023



NOTES:

BORING/SOUNDING ARE PROJECTED TO THE CENTERLINE.

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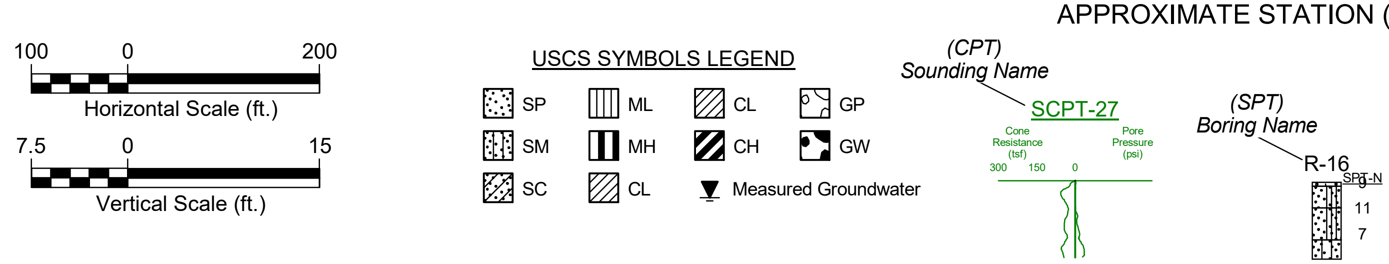
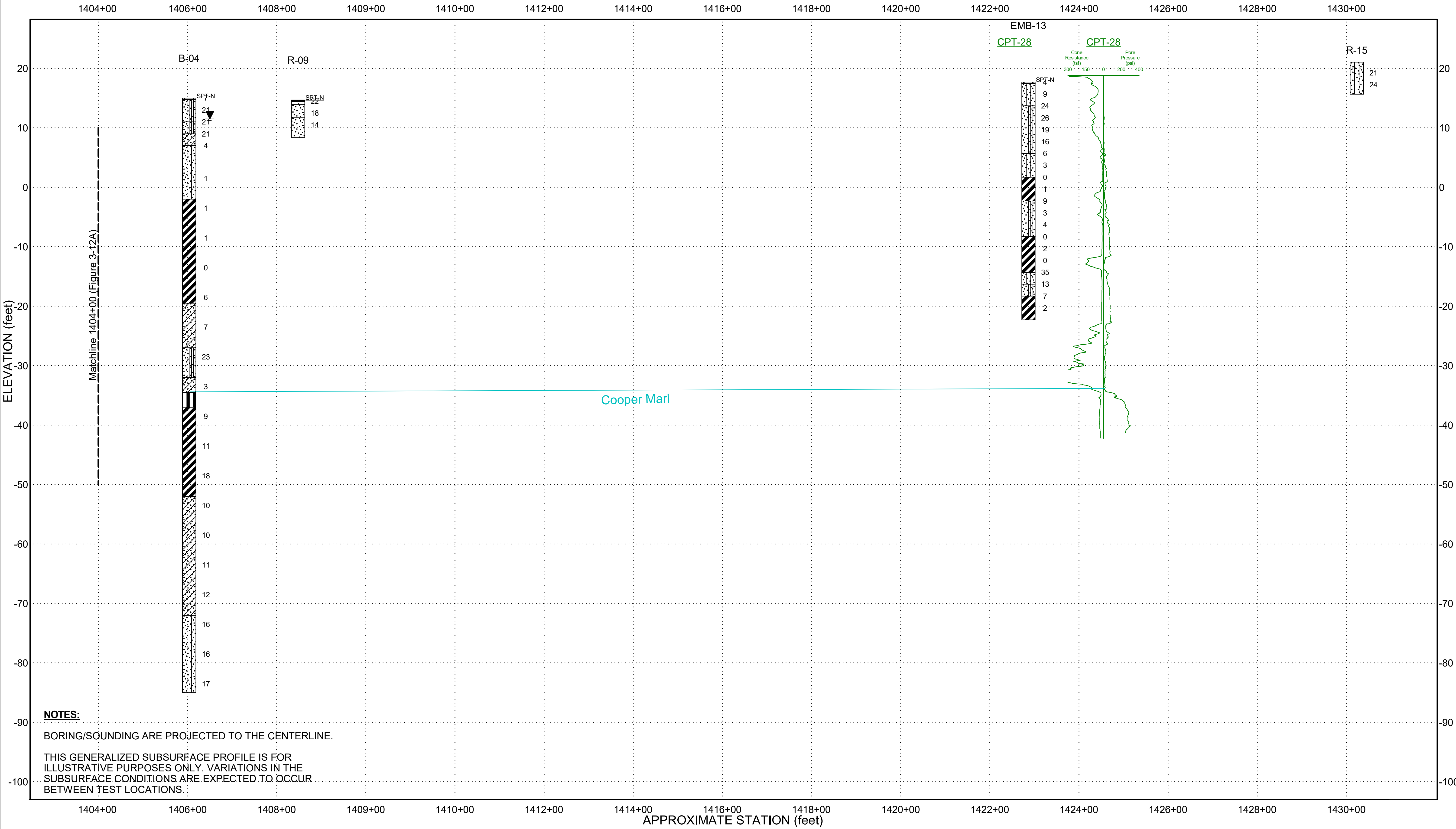


FIGURE 3-12A
GENERALIZED SUBSURFACE PROFILE - RELOCATED LINE 3
I-526 LOWCOUNTRY CORRIDOR PHASE 2
CHARLESTON, SOUTH CAROLINA

DRAWN BY: DCP
DATE: 08.14.2023

REVIEWED BY: JM
DATE: 09.18.2023

CDM SPT 526 LONGPOINT BH LOGS.GPJ - 8/14/23



NOTES:

BORING/SOUNDING ARE PROJECTED TO THE CENTERLINE.

THIS GENERALIZED SUBSURFACE PROFILE IS FOR ILLUSTRATIVE PURPOSES ONLY. VARIATIONS IN THE SUBSURFACE CONDITIONS ARE EXPECTED TO OCCUR BETWEEN TEST LOCATIONS.

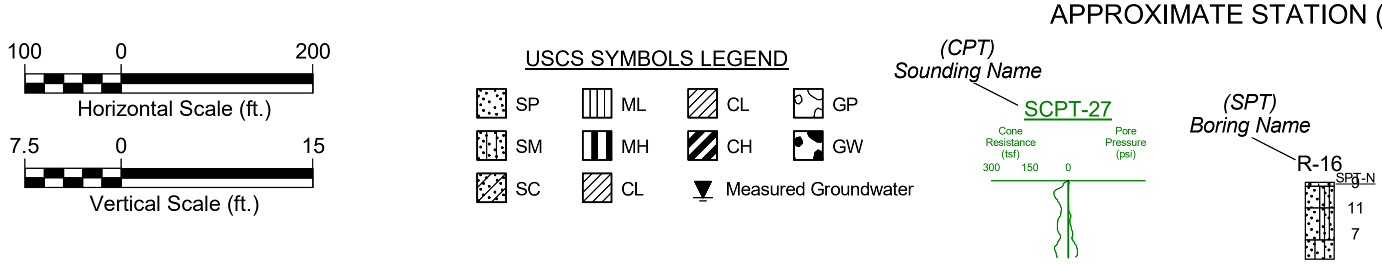
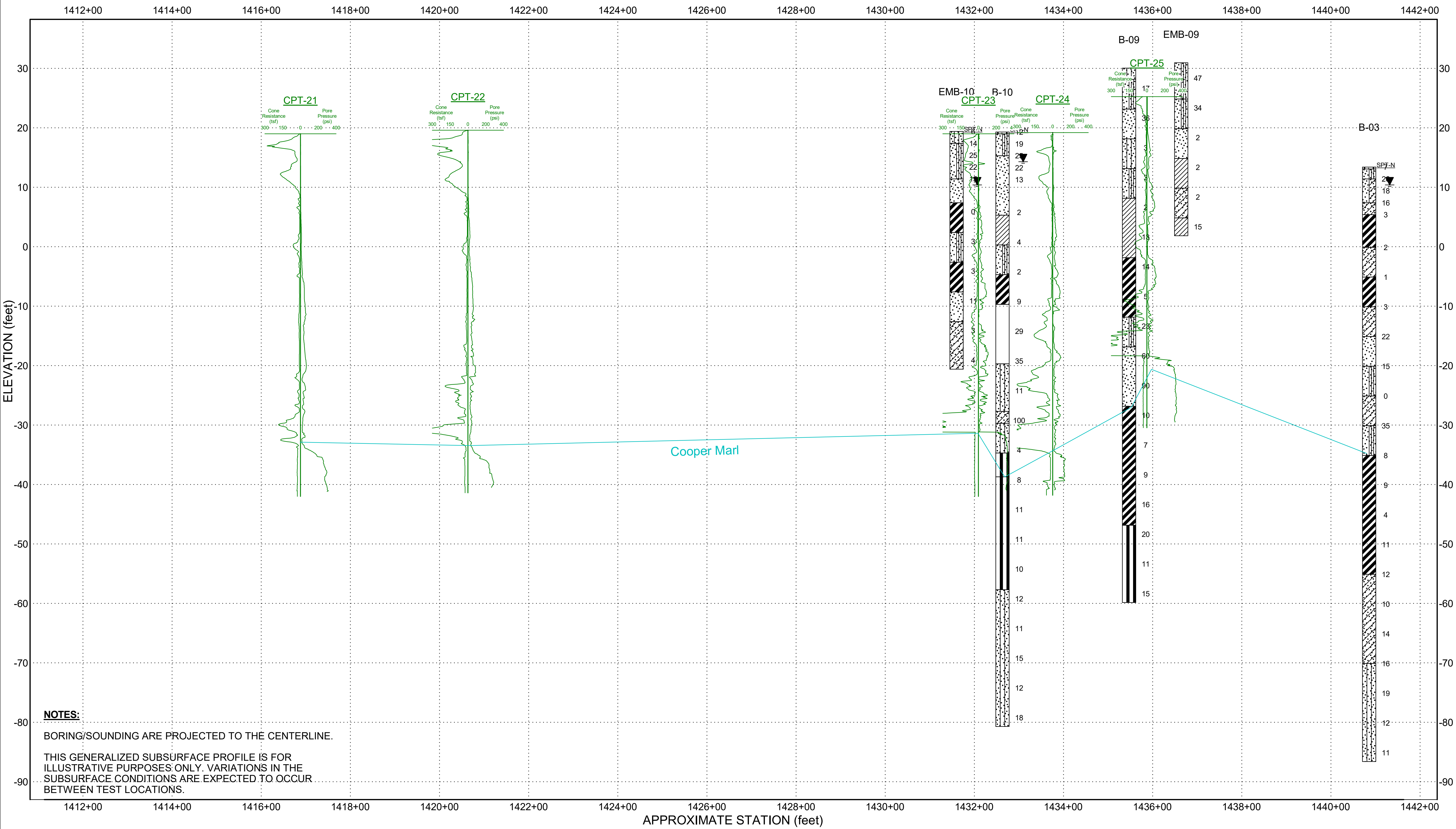


FIGURE 3-12B
GENERALIZED SUBSURFACE PROFILE - RELOCATED LINE 3
I-526 LOWCOUNTRY CORRIDOR PHASE 2
CHARLESTON, SOUTH CAROLINA

DRAWN BY: DCP
DATE: 08.14.2023

REVIEWED BY: JM
DATE: 09.18.2023

CDM SPT 526 LONGPOINT BH LOGS.GPJ - 8/14/23



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100 0 200
Horizontal Scale (ft.)

7.5 0 15
Vertical Scale (ft.)

USCS SYMBOLS LEGEND

SP, ML, CL, GP, SM, MH, CH, GW, SC, CL, Measured Groundwater

(CPT) Sounding Name
SCPT-27
Cone Resistance (tsf) 300 150 0
Pore Pressure (psi)

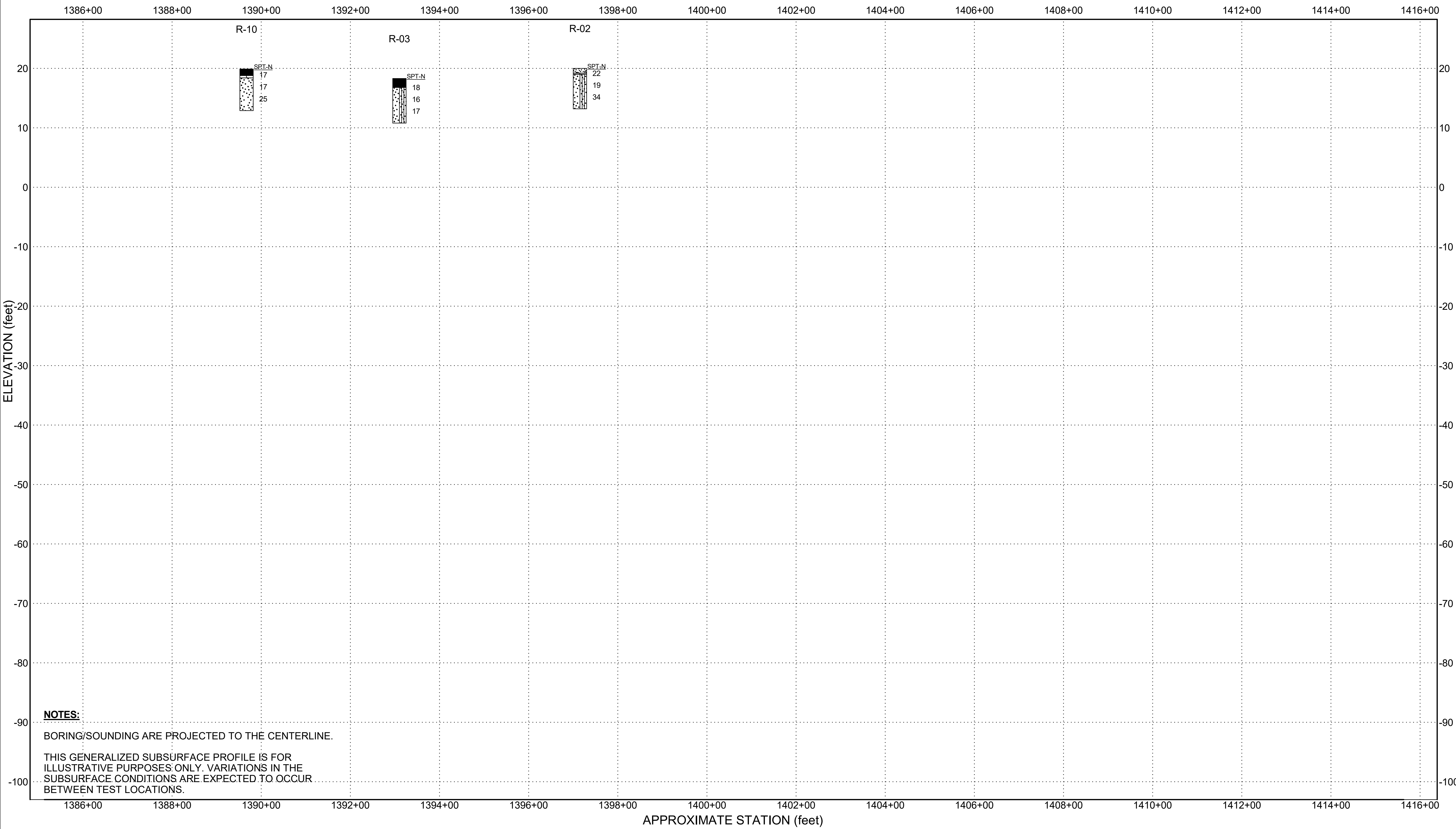
(SPT) Boring Name
R-16
SPT-N
9
11
7

FIGURE 3-13
GENERALIZED SUBSURFACE PROFILE - RELOCATED LINE 4
I-526 LOWCOUNTRY CORRIDOR PHASE 2
CHARLESTON, SOUTH CAROLINA

DRAWN BY: DCP
DATE: 08.14.2023

REVIEWED BY: JM
DATE: 09.18.2023

CDM SPT 526 LONGPOINT BH LOGS.GPJ - 8/14/23



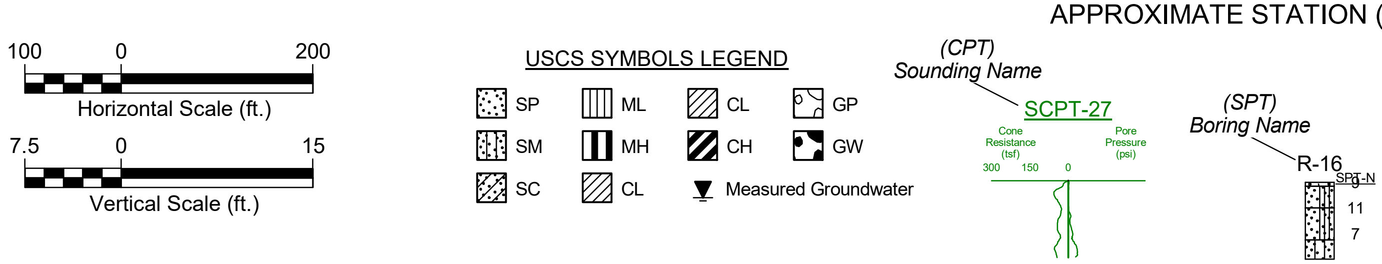
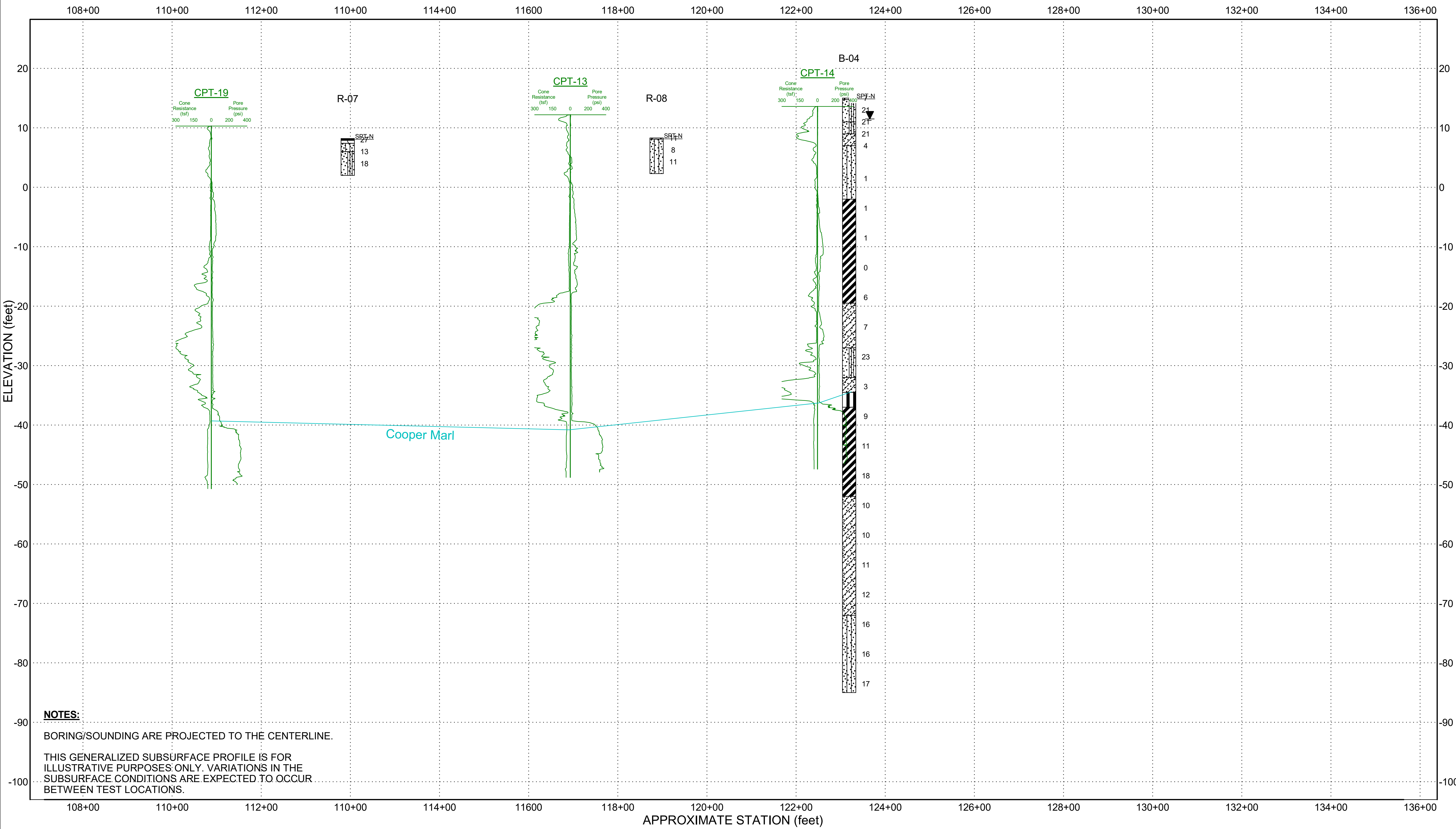


FIGURE 3-14
GENERALIZED SUBSURFACE PROFILE - RELOCATED LINE 6
I-526 LOWCOUNTRY CORRIDOR PHASE 2
CHARLESTON, SOUTH CAROLINA

DRAWN BY: DCP
DATE: 08.14.2023

REVIEWED BY: JM
DATE: 09.18.2023



NOTES:

BORING/SOUNDING ARE PROJECTED TO THE CENTERLINE.

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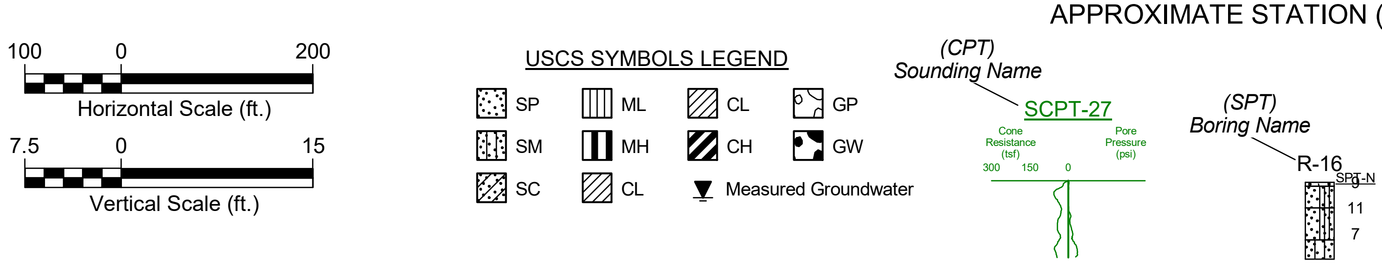
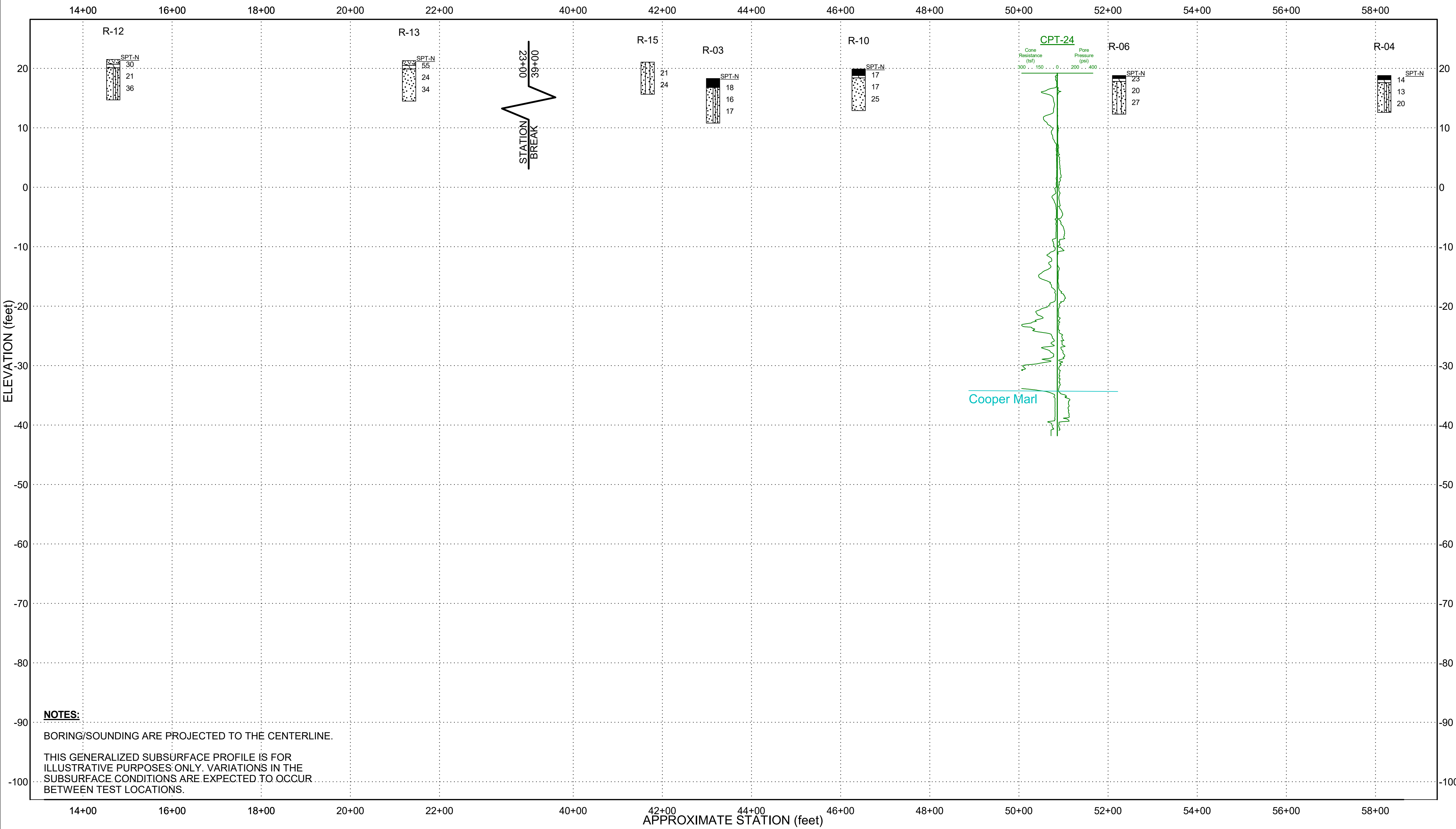


FIGURE 3-15
GENERALIZED SUBSURFACE PROFILE - RELOCATED WANDO PARK BLVD
I-526 LOWCOUNTRY CORRIDOR PHASE 2
CHARLESTON, SOUTH CAROLINA

DRAWN BY: DCP
DATE: 08.14.2023

REVIEWED BY: JM
DATE: 09.18.2023

CDM SPT 526 LONGPOINT BH LOGS.GPJ - 8/14/23



NOTES:

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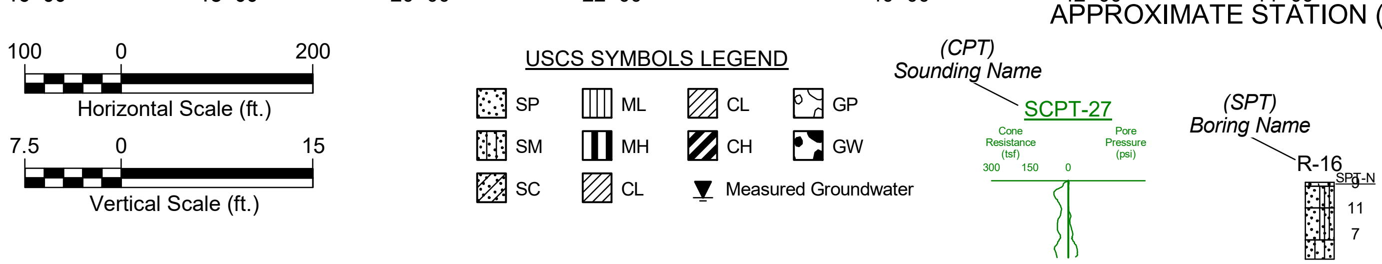
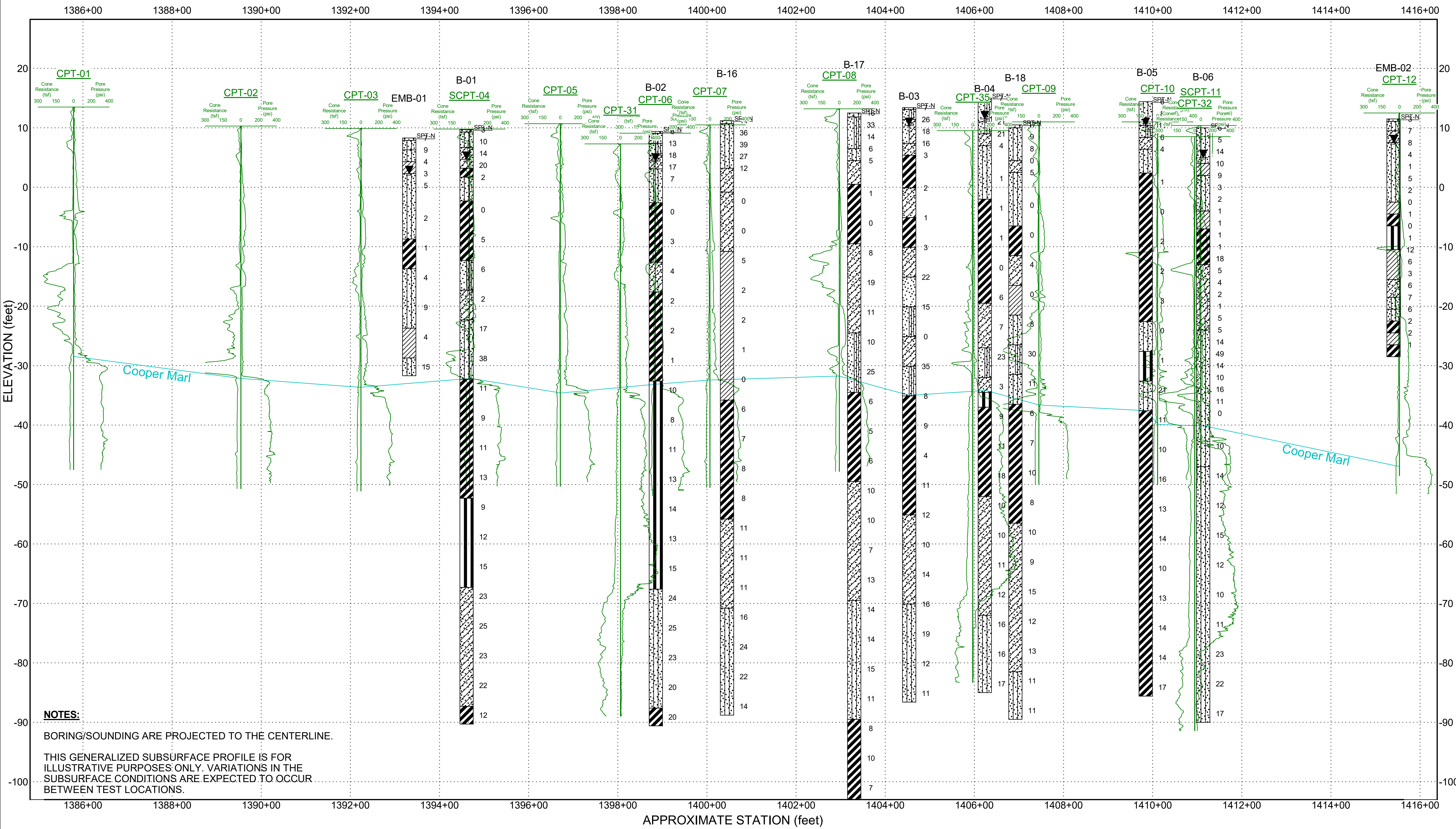


FIGURE 3-16
GENERALIZED SUBSURFACE PROFILE - RELOCATED LONG POINT ROAD
I-526 LOWCOUNTRY CORRIDOR PHASE 2
CHARLESTON, SOUTH CAROLINA

DRAWN BY: DCP
DATE: 08.14.2023

REVIEWED BY: JM
DATE: 09.18.2023



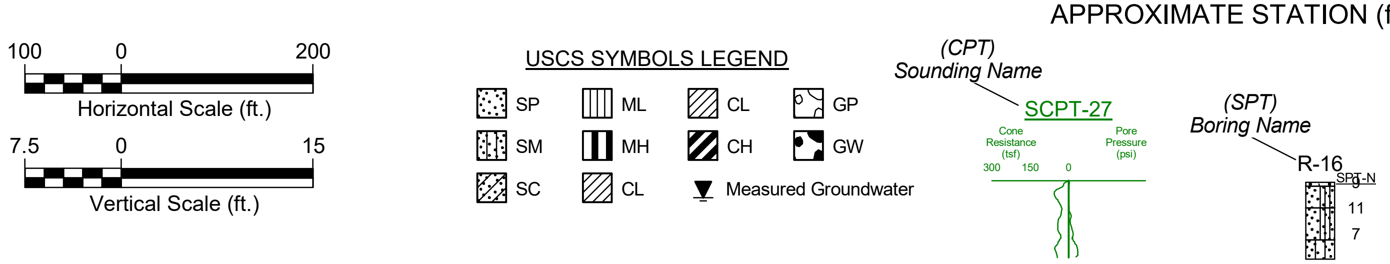
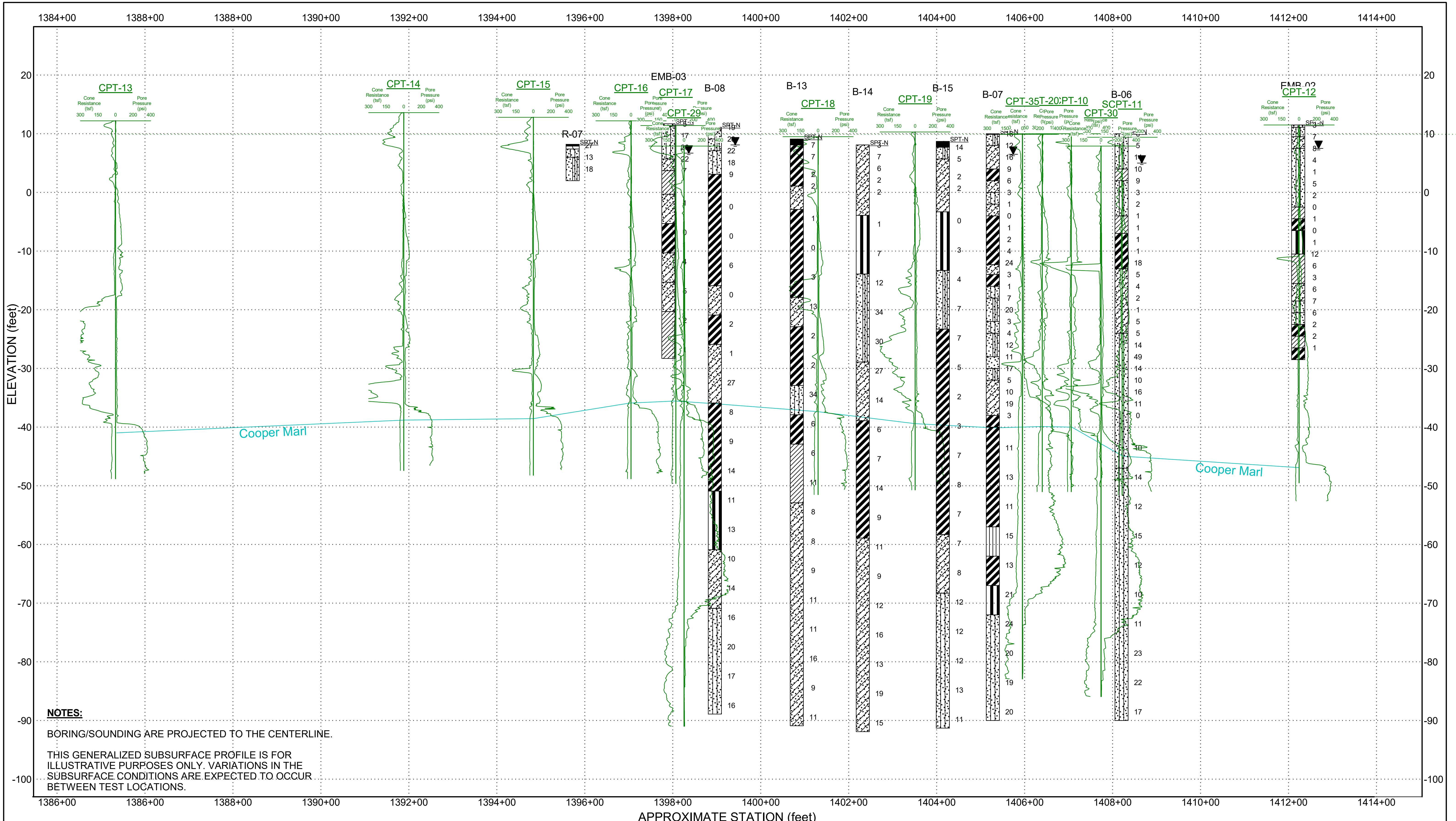


FIGURE 3-17
GENERALIZED SUBSURFACE PROFILE - LINE A
I-526 LOWCOUNTRY CORRIDOR PHASE 2
CHARLESTON, SOUTH CAROLINA

DRAWN BY: DCP
DATE: 08.29.2025

REVIEWED BY: KA
DATE: 09.8.2025



NOTES:

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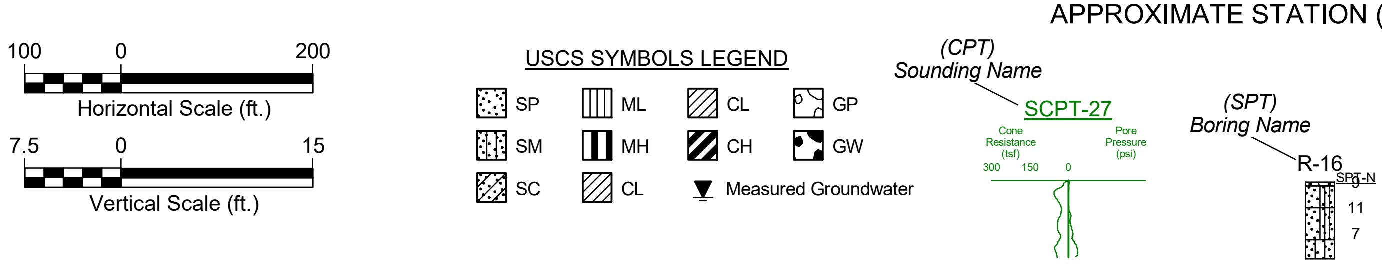
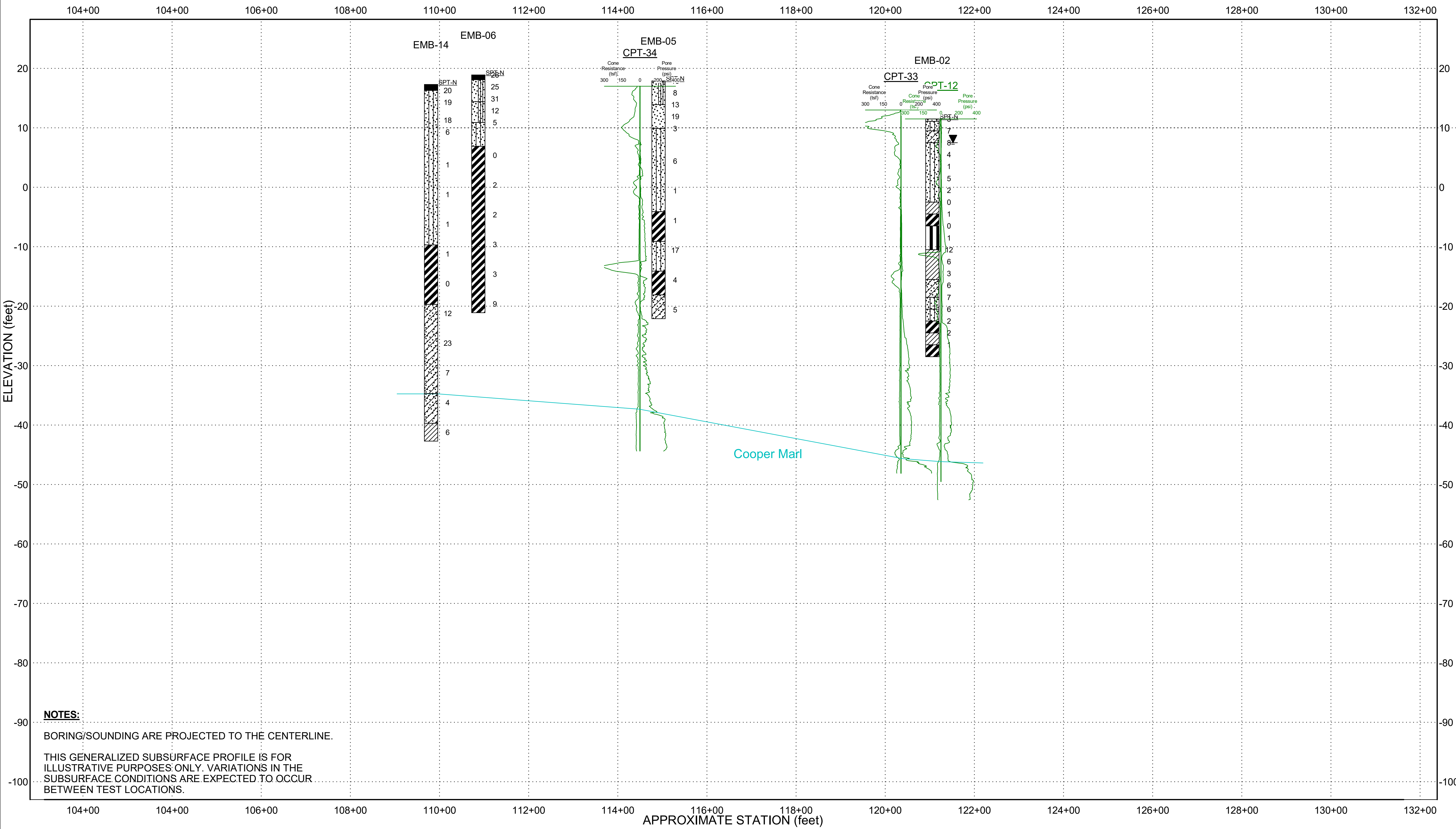


FIGURE 3-18
GENERALIZED SUBSURFACE PROFILE - LINE B
I-526 LOWCOUNTRY CORRIDOR PHASE 2
CHARLESTON, SOUTH CAROLINA

DRAWN BY: DCP
DATE: 08.29.2023

REVIEWED BY: KA
DATE: 09.8.2023

CDM SPT 526 LONGPOINT BH LOGS.GPJ - 8/14/23



NOTES:

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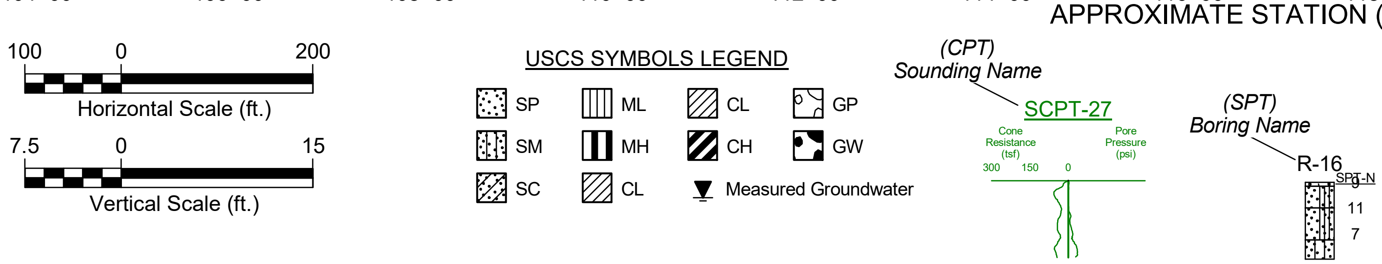


FIGURE 3-19
GENERALIZED SUBSURFACE PROFILE - WANDO PORT CONNECTOR
I-526 LOWCOUNTRY CORRIDOR PHASE 2
CHARLESTON, SOUTH CAROLINA

DRAWN BY: DCP
DATE: 08.29.2025

REVIEWED BY: KA
DATE: 09.8.2025

Section 4 - General Geotechnical Considerations

General

The general considerations provided within are based on preliminary analyses performed based on the preliminary roadway and bridge plans, borings/soundings performed as part of the site exploration, laboratory test data, and our experience with the subsurface conditions in the region. Preliminary analyses were performed for embankments, retaining walls, and bridges. Final design recommendations should be based on detailed analyses of the final design conditions and plans.

Settlement

Settlement analyses were performed utilizing the program *Settle3* by RocScience at critical cross sections typically at locations of maximum embankment/wall height sections. Primary consolidation parameters were conservatively selected based on the collected data.

The settlement at critical cross sections were compared with select Service Limit State performance limits listed in SCDOT *GDM* tables 10-12 through 10-15 for embankments and 10-16 and 10-17 for earth retaining structures. The primary performance limits considered were EV-01B (3") and EV-05B (0.5") for embankments, and RV-01A (NL – 12" to demarcate single/multi-stage construction), RV-01B (2"), and RV-04B (0.15 L_{Reinf}) for earth retaining structures. A design life of 20 years (7,300 days) was considered relative to the pavement deflection performance limits.

Stability

Global stability analyses were performed using GeoStudio's SLOPE/W for critical cross-sections. The analyses were performed using a traditional allowable stress design, or ASD, approach to calculate factors of safety (FS) against global stability failure. The inverse of the factor of safety was then used to determine the stability Resistance Factor (RF), or ratio of demand to capacity.

Typically, critical cross-sections were determined to be at locations of maximum embankment or wall height. Where MSE walls are proposed, an assumed reinforcement length of 0.7 times the wall height was assumed. No local stability or MSE Wall reinforcement design calculations were performed. The long term required resistance factor required for slope stability in the *GDM* is 0.75.

Pseudo-static analyses were performed for FEE (Functional Evaluation Earthquake) and SEE (Structural Evaluation Earthquake) peak ground accelerations (PGA= 0.23g for FEE, and PGA=0.45g for SEE) and applying a wave scattering factor. Where pseudo-static analyses showed a Factor of Safety (FOS) less than 1.0 (Resistance Factor = 1.0), the yield acceleration was determined and reported. The analyses were performed using peak soil strengths similar to the Service Limit State. It should be noted that there is potential liquefaction and shear strength loss due to cyclic softening. Due to the instability in the

Extreme Event presented in the following sections, it can be assumed that the instability issues will still be present using reduced strengths.

Bridge Foundations

For this preliminary analysis, actual capacity and resistance will be derived through a combination of side friction and end bearing within the Cooper Marl, depending on foundation type. The resistance factors are subject to the construction and/or sequencing and have not been included.

Driven Pile Foundations

Steel H-piles are assumed as the primary end bent foundation; however, some success has been shown driving concrete piles that are 24 inches or smaller. Larger concrete piles have been difficult to drive to adequate depths within the Cooper Marl. Some structures are shown in bridge concept plans as flat slab structures with prestressed concrete pile bents. These will likely be 18- or 24-inch concrete piles.

Potential for predrilling at this site is very low. The following sections provide discussion regarding driven pile foundations along the entire site.

Axial Resistance

The Strength limit state will likely govern the tip elevations of the bridge piles. Non-displacement, driven piles will develop the required driving resistance through both skin friction and end bearing in the Cooper Marl formation. Embankment fills near the bridge ends will cause large downdrag loads on the foundations and foundations depths shall accommodate for this additional loading. Construction sequencing or other common techniques may negate some of this downdrag loading, and result in shorter piles.

Lateral Resistance

For the Strength and Extreme Event limit states, we anticipate that driven piles would develop the required lateral stability within the Pre-Pleistocene and Cooper Marl formations. SSL should be conducted to determine whether the tip elevations should be extended beyond the requirements of the strength limit state. No scour has been included when evaluating the pile stability. Seismic bridge abutment backwall passive pressure should be considered in the lateral analysis of the end bent piles, in accordance with Chapter 14 of the SCDOT *GDM*.

Drivability

Piles are typically driven with diesel pile hammers. We anticipate that non-displacement piles (Steel H-piles) will extend into the Marl. Pile points or stingers may be required fo facilitate penetration and reduce potential for tip damage while driving. If prestressed concrete piles are selected, the hammer selection must ensure that driving stresses are not exceeded prior to reaching required tip elevations.

The contractor should anticipate that driving resistances will not be met on initial drive and should anticipate a waiting period of about 14 days or more for substantial pile setup within the Cooper Marl. PDA testing may be performed on the initial drives and restrikes to verify capacity and monitor driving stresses of the bridge piles.

Depending on the required driving resistance of the pile system, a medium to large pile hammer may be required. Pile driving has been shown to have potential for causing damage to nearby structures. Depending on the location of the bridge piles relative to nearby structures, vibration monitoring will be required and should influence the hammer selection. The selected driving system shall conform to SCDOT criteria. The vibration monitoring plan should be developed in accordance with Chapter 24 of the *GDM*.

Drilled Shaft Foundations

For drilled shafts, we assumed permanent casing with a minimum socket of one diameter into the Cooper Marl to create an adequate seal. Design methodology does not include contribution to skin friction within the casing depth for drilled shafts. We anticipate that the drilled shaft sizes would range between 60 and 144 inches in diameter. Specific drilled shaft design issues are discussed in the following sections.

Axial Resistance

The Strength limit state is expected to govern the geotechnical design of drilled shafts, however, the Extreme Event limit may govern a number of the foundations. As is typical in this part of the state, drilled shafts will likely be installed with construction casing and constructed in the dry. We expect that the primary resistance for the shaft will be derived in the Cooper Marl, especially considering that the skin friction is neglected in the cased sections of the shaft. It is possible, however, on a few of the bridges that do not cross water, that casing could be forgone, with SCDOT approval. This option may provide added axial capacity and limit the depth of the drilled shafts.

Lateral Resistance

For the Strength and Extreme Event limit states, the drilled shafts will develop lateral stability within the Cooper Marl formation. We expect that the lateral loads on several of the bridges to be very high and lateral stability to be relatively deep. Where applicable, the 100-year and 500-year flood elevations and associated scour depths, should be used in determination of the lateral stability of the drilled shafts.

Constructability

Depending on the final selected geometry of the drilled shaft bents, and potential drilled shaft footings, temporary shoring may be required. The soil parameters for the temporary shoring will need to be developed for each bridge location in accordance with Chapters 21 and 22 of the *GDM*.

Corrosion and Deterioration Potential

Select soil samples were tested according to ASTM D4327 for Sulfates and Chlorides, ASTM G187 for Resistivity of soil, and ASTM G51 for acidity (pH). Results are indicated in the following table. Values that exceed recommended thresholds are noted in red.

Table 4-1 Laboratory Corrosion Testing Summary

Boring Number	Sample (depth)	As received Resistivity (Ohm-cm)	Minimum Resistivity (Ohm-cm)	Sulfates (mg/kg)	Chlorides (mg/kg)	pH
B-01	4-6	5,293	4,757	37.1	12.8	6.8
	6-8	5,628	4,690	54.4	15.8	6.8
	38.5-40	1,474	1,407	562.4	13.9	5.6
	43.5-45	1,005	938	297.7	21.2	7.4
B-04	4-6	38,860	19,430	25.9	12.1	6.4
	73.5-75	630	603	314	14.8	7.2
B-06	4-6	4,757	4,489	30.7	16.4	8
	6-8	871	804	17.1	12.7	7.7
	16-18	442	429	1,655.1	19.1	5.4
	22-24	804	670	184.5	17	6.4
B-13	3-5	1,943	1,675	96.7	38.7	7.3
	6-8	2,412	2,278	139	59.3	6.5
B-16	2-4	2,345	1,742	212.1	31.3	6.6
	6-8	5,427	3,886	96.8	24.6	6.2

Per Table 7-34 in the *GDM*, testing results for soil are **Aggressive** if any of the following conditions exist:

- Resistivity less than 2,000 ohm-cm
- pH less than 5.5 (or between 5.5 and 8.5 in organic soils)
- Sulfate concentrations greater than 1,000 ppm (or mg/kg)
- Chloride concentrations greater than 500 ppm (or mg/kg)

As indicated in Table 4-1, resistivity values indicate the potential for corrosion and/or deterioration near all borings that were tested during our exploration. A single test indicates a vulnerability to sulfate attack and a pH less than 5.5in Boring B-06. Based on this information, the site is considered as **Aggressive** should consider additional testing and subsequent mitigations for corrosion of deep foundation elements.

Existing Pavements

Sixteen (16) shallow pavement borings with pavement cores and shallow dynamic cone penetrometer tests (ASTM D6951) were conducted within the existing pavement for the interchange project. Some pavement cores were not performed or collected, but borings were conducted adjacent to the roadway for purposes of obtaining representative in-situ CBR values. See Table 4-2 for summary of pavement and subgrade tests.

Table 4-2 Pavement and Subgrade Testing Summary

Boring ID	Pavement Thickness (inches)	Pavement Type	Average Subgrade In-Situ CBR Value
R01	-	-	5.7
R02	12	Concrete	11.7
R03	17	Asphalt	12.3
R04	9.5	Asphalt	1.4
R05	-	-	4.2
R06	7	Asphalt	3.0
R07	3.5	Asphalt	15.7
R08	2.5	Asphalt	13.5
R09	4.5	Asphalt	6.0
R10	14.5	Asphalt	4.2
R11	14	Concrete	15.2
R12	11	Concrete	5.4
R13	10.5	Concrete	8.2
R14	12.5	Concrete	3.7
R15	6	Asphalt	13.3
R16	-	-	6.4

Relocated Line 1

Relocated Line 1 is the I-526 Westbound off-ramp to State Road S-10-97 (Long Point Road). The relocated alignment begins at Station 1436+00 at the tie-in to Long Point Rd and continues approximately 1,575 feet to Station 1451+75 where it will tie in with I-526 Westbound. The stationing increase is counter to the flow of traffic. Within the alignment, a new proposed bridge from Station 1441+25 to 1444+00, approximately 375 feet in length, will transverse an existing wetlands area.

Relocated Line 1 Roadway Embankment

The roadway embankment for Relocated Line 1 is expected to require up to 8 feet of fill and minimal cut for ditches to achieve final grades. South of Bridge 1, maximum embankment slopes will be 6H:1V, and north of the bridge embankments will have maximum slopes anticipated are 2H:1V. The critical cross section was selected at approximate Station 1441+00 was used for settlement and global stability analyses.

Settlement

Settlement analysis was conducted at representative cross sections along the Relocated Line 1. Based on our preliminary analyses, a waiting period of about 3 months is anticipated to reach the end of primary consolidation. However, near the bridge, long term settlements may be of concern and not meet SCDOT performance limits over the pavement design life. Ground improvements or waiting periods may be required to meet the performance limits.

Global Stability

The results of the global stability analysis at station 1441+00 indicate the embankment meet the required resistance factors listed in Chapter 10 of the SDCOT *GDM* for both Strength and Extreme Event limits states.

Relocated Line 1 Bridge Foundations

The proposed Relocated Line 1 Bridge will be 12 spans (13 bent), approximately 46 feet wide, and 365 feet long. Bridge concept plans include a flat slab structure with concrete piles supporting end bents and interior bents. Bridge foundations are discussed in an earlier section of this report.

Relocated Line 2

Relocated Line 2 is the on-ramp to I-526 Westbound from State Road S-10-97 (Long Point Road). The relocated alignment begins at Station 1422+50 at the tie-in to I-526 Westbound and continues southeast approximately 1,700 feet to Station 1439+50 where it ties in with Long Point Road. The stationing increase is counter to the flow of traffic. Within the alignment, a new retaining wall is (Wall 2A) proposed from approximately Station 1428+75 to 1431+00 to separate Relocated Line 2 grade from the Relocated Line 4 embankment.

Relocated Line 2 Roadway Embankment

The roadway embankment for Relocated Line 2 is expected to require up to 10 feet of fill and minimal cut for ditches to achieve final grades. Maximum slopes anticipated are 2H:1V The critical cross section was selected at approximate (Line B) Station 1430+00 was used for settlement and global stability analyses.

Settlement

At the representative cross-section at (Relocated Line 4) Station 1430+00, approximately 10 feet of new fill is proposed. It is anticipated that 4.4 inches of settlement will occur within 3 months of embankment construction. We estimate that primary consolidation will complete in about 6 months after full height of the embankment is placed. Total settlement over the design life is expected to be about 6.0 inches. We expect that post-construction performance limits for pavements (EV-01B) will be satisfied.

Global Stability

The results of the global stability analysis at Station 1430+00 indicate the embankment meet the required resistance factors listed in Chapter 10 of the SDCOT *GDM* for Strength limit state, but not for the Extreme Event limit state. It is likely that reinforcement or ground improvements will be required to satisfy resistance factor requirements during the SEE.

Retaining Wall 2A

Wall 2A has a maximum height of approximately 5 feet and will separate the Relocated Line 4 grade from Relocated Line 2. The wall is proposed to be an MSE wall. For this preliminary analysis, we have assumed a reinforcement length of 0.7H with a minimum length of 8 feet. The critical cross section was selected at approximate Station 1430+00 was used for settlement and global stability analyses.

Settlement

At the representative cross-section at Station 1430+00, approximately 15 feet of new fill is proposed (10 feet of fill below Wall A and 5 feet retained by Wall A). It is anticipated that about 7 inches of settlement will occur within 3 months of construction at the wall face. It is estimated that primary consolidation will complete about 6 months after full height of the wall is placed Total settlement over the design life is expected to be about 9 inches. Some performance limits were not achieved; Ground improvements, waiting periods, or other methods may be required to meet the performance limits.

Global Stability

The results of the global stability analysis at Station 1430+00 indicate Wall 2A meet the required resistance factors listed in Chapter 10 of the SDCOT *GDM* for the Strength limit state but not the Extreme Event limit state. It is likely that additional reinforcement or ground improvements will be required to satisfy resistance factor requirements during the SEE.

Relocated Line 3

Relocated Line 3 is the I-526 Eastbound off-ramp to State Road S-10-97(Long Point Road). The Relocated alignment begins at Station 1373+00 at the tie-in to I-526 Eastbound and continues southeast approximately 5625 feet to Station 1430+25 where it ties in with Long Point Road. Within the alignment, new retaining walls are proposed from approximate Stations 1390+00 to 1391+00 (Wall 3A), 1390+00 to 1403+00 (Wall 3B), and from 1398+50 to 1402+25 (Wall 3C).

Roadway Embankment

The roadway embankment for Relocated Line 3 is expected to require up to 12 feet of fill and minimal cut for ditches to achieve final grades. Maximum slopes anticipated are 2H:1V The critical cross section was selected at approximate Station 1403+00 was used for settlement and global stability analyses.

Settlement

At the representative cross-section at Station 1403+00, approximately 12 feet of new fill is proposed. It is anticipated that about 7 inches of settlement will occur within 3 months of embankment construction and reach the end of primary consolidation. Total settlement over the design life is expected to be about 7.3 inches, which meets the post-construction pavement performance limits (EV-01B).

Global Stability

The results of the global stability analysis at Station 1403+00 indicate the resistance factors are not met for the Strength or Extreme Event limit states. Some reinforcement or ground improvements will likely be required to meet stability requirements.

Retaining Walls 3A, 3B, and 3C

The retaining walls 3A, 3B, and 3C are located near the northern start of Relocated Line 3. These walls support the elevated roadway of Relocated Line 3 and adjacent Line B. Wall 3A has a maximum height of about 5 feet. Walls 3B and 3C reach maximum heights of about 18 and 25 feet, respectively. The walls are proposed to be MSE walls. For this preliminary analysis, we have assumed a reinforcement length of 0.7H with a minimum length of 8 feet. Walls 3B and 3C were evaluated at approximate Station 1398+50 for settlement and global stability analyses.

Retaining Wall Settlement

At the representative cross-section at Station 1398+50, approximately 25 feet of new fill is proposed. It is anticipated that about 8 inches of settlement at the wall face and about 15 inches of settlement within the footprint of reinforcement will occur within 3 months of construction. It is estimated that primary consolidation will complete about 1 year after full height of the wall is placed. Total settlement over the design life is expected to be about 10 and 21 inches at the wall face and within the footprint of reinforcement, respectively. This exceeds the acceptable post-construction pavement settlement limits (RV-01B) for earth retention structures over the design life. Installation of wick drains, a surcharge program, construction staging, or other methods may be required to reduce these post-construction settlements.

Retaining Wall Global Stability

The results of the global stability analysis at Station 1398+50 indicate that the Strength and Extreme Event limits states do not meet the required resistance factors for static or seismic stability. Relocated Line 3 at this location is an elevated roadway supported by MSE retaining walls on both sides. The primary failure at the critical cross-section will engage the reinforcement of the opposite retaining wall. Our preliminary analysis used assumed values for MSE Wall reinforcement and spacing. Additional layers of reinforcement, or higher-strength reinforcements, ground modification, or other methods may be necessary to meet stability requirements. Additionally, the use of a surcharge, waiting period, or construction staging may also allow for the upper clay layer to gain strength and improve stability.

Relocated Line 4

Relocated Line 4 is the I-526 westbound on-ramp from State Road S-10-97 (Long Point Road). The Relocated alignment begins at Station 1406+50 at the tie-in to I-526 westbound and continues southeast approximately 3900 feet to Station 1445+50 where it ties in with Long Point Road. The stationing increase is counter to the flow of traffic. Within the alignment, a new retaining wall is proposed starting from approximately Station 1424+00 to separate Relocated Line 4 from I-526 westbound. A proposed bridge from Station 1432+50 to 1435+00 approximately 250 feet in length will transverse an existing wetlands area along the alignment.

Roadway Embankment

The roadway embankment for Relocated Line 4 is expected to require up to 14 feet of fill and minimal cut for ditches to achieve final grades. Maximum slopes anticipated are

2H:1V The critical cross section was selected at approximate Station 1430+00 was used for settlement and global stability analyses.

Settlement

At the representative cross-section at Station 1430+00, approximately 14 feet of new fill is proposed. It is anticipated that about 7 inches of settlement will occur within 3 months of embankment construction. We estimate that primary consolidation will be complete after about 1 year after full height of the embankment is placed. Total settlement over the design life is expected to be about 10 inches. This settlement exceeds the acceptable limits of 3 inches (EV-01B) for embankments. Installation of wick drains, a surcharge program, or other methods may be required to reduce these post-construction settlements.

Retaining Wall 4

Wall 4 has a maximum height of approximately 5 feet and will separate the Relocated Line 4 grade from I-526. The wall is proposed to be an MSE wall. For this preliminary analysis, we have assumed a reinforcement length of 0.7H with a minimum length of 8 feet. The critical cross section was selected at approximate Station 1430+00 was used for settlement and global stability analyses.

Settlement

At the representative cross-section, it is anticipated that less than one inch of settlement at the wall face will occur within 3 months of construction. We estimate that primary consolidation will complete about 7 months after full height of the wall is placed. Total settlement over the design life is expected to be about 2.0 inches, which meets SCDOT settlement performance limits.

Global Stability

The results of the global stability analysis at Station 1430+00 indicate Wall 4 meet the required resistance factors listed in Chapter 10 of the SDCOT *GDM* for the Strength limit state, but not for the Extreme Event limit state. Some reinforcement or ground improvements will likely be required to meet stability requirements.

Relocated Line 4 Bridge

The proposed Relocated Line 4 Bridge is planned as a 3 span (4 bent) structure, approximately 38 feet wide, and 250 feet long. Bridge concept plans show steel pile end bents with drilled shaft interior bents. Abutments are shown as a concrete sloped spill-through abutment on one side, and the other side includes a pedestrian walkway with a short wall. Bridge foundations are discussed in an earlier section of this report.

Relocated Line 5

Relocated Line 5 is the I-526 Eastbound on-ramp from State Road S-10-97 (Long Point Road). The Relocated alignment begins at Station 39+50 at the tie-in to I-526 Eastbound and continues southeast approximately 1,650 feet to Station 56+00 where it ties in with Long Point Road.

Roadway Embankment

The roadway embankment for Relocated Line 5 is expected to require up to 2 feet of fill and minimal cut for ditches to achieve final grades. Maximum slopes anticipated are 4H:1V. Due to the embankment height, no stability analyses were performed. Settlements are anticipated to be elastic and to be less than the performance limits for embankments.

Relocated Line 6

Relocated Line 6 is the I-526 Eastbound on-ramp from State Road S-10-97 (Long Point Road). The Relocated alignment begins at Station 1417+50 at the tie-in to I-526 Eastbound and continues southeast approximately 1,025 feet to Station 1427+75 where it ties in with Long Point Road.

Roadway Embankment

The roadway embankment for Relocated Line 6 is expected to require up to 12 feet of fill and minimal cut for ditches to achieve final grades. Maximum slopes anticipated are 2H:1V. The critical cross section was selected at approximate Station 1425+00 was used for settlement and global stability analyses.

Settlement

At the representative cross-section at Station 1425+00, approximately 12 feet of fill is proposed. It is anticipated that about 5 inches of settlement will occur within 3 months of embankment construction. We estimate that primary consolidation will complete about 5 months after full embankment has been placed. Total settlement over the design life is expected to be about 7 inches.

Global Stability

The results of the global stability analysis at Station 1425+00 indicate the embankment will not meet the required resistance factors listed in Chapter 10 of the SDCOT *GDM* in the Strength or Extreme Event limit states. Reinforcement or some ground improvements may be required to meet resistance factors for stability of the embankment.

Relocated Wando Park Blvd

Relocated (or realigned) Wando Park Blvd is the road parallel to I-526. The Relocated alignment begins at Station 10+00.00 and continues southeast approximately 2,775 feet to Station 37+75 on Wando Park Blvd.

Roadway Embankment

The roadway embankment for Relocated Wando Park Blvd is expected to require up to 6 feet of fill and minimal cut for ditches to achieve final grades. Maximum slopes anticipated are 2H:1V The critical cross section was selected at approximate Station 120+00 was used for settlement and global stability analyses.

Settlement

At the representative cross-section at Station 120+00, approximately 6 feet of new fill is proposed. It is anticipated that about 5 inches of settlement will occur within 3 months of embankment construction. We estimate that primary consolidation will complete within this 3-month period following construction. Total settlement over the design life is expected to be about 7 inches, which meets performance limits for pavement and embankments.

Global Stability

The results of the global stability analysis at Station 120+00 indicate the embankment meet the required resistance factors listed in Chapter 10 of the SDCOT *GDM* for Strength and Extreme Event limit states.

Relocated Long Point Road

Relocated Long Point Road is a part of State Road S-10-97 (Long Point Road). The Relocated alignment begins at Station 10+75 and continues southeast approximately 750 feet to Station 18+25 where it connects with Wando Port Connector and ties in with Long Point Road.

Roadway Embankment

The roadway embankment for Relocated Long Point Road is expected to have minimal new fill and cut to achieve final grades. Maximum slopes anticipated are 2H:1V. Where Long Point Road passes under the existing I-526, the proposed widening will require a cut into the existing I-526 bridge east embankment. The cut will require a tie-back reinforcement. The critical cross section was selected at approximate Station 52+50 was used for global stability analyses.

Global Stability

The proposed cut wall at Station 52+50 is approximately 11 feet tall. For tie-back reinforcement, we assumed an anchor length of 25 feet installed two-thirds the height (7.25 feet) up the wall face. Results of the global stability that with a reinforcement length of 25 feet the wall meet the required resistance factors listed in Chapter 10 of the SDCOT *GDM* for the Strength limit state, but not for the Extreme Event limit state. Further reinforcement or ground improvements may be required to meet required resistance factors in the SEE.

New Line A

Relocated New Line A is the on-ramp to I-526 westbound. The Relocated alignment begins at approximate Station 1364+50 and continues southeast approximately 5,000 feet to Station 1414+50 where it crosses Relocated Line 3 and Relocated Wando Park Blvd. The stationing increase is counter to the flow of traffic. Within the alignment, new retaining walls are proposed from approximate Stations 1392+00 to 1398+00, 1395+00 to 1398+00, 1398+00, 1410+50 to 1414+50, and 1410+50. Based on the alignment profile, the proposed sloped embankments will have a maximum fill height of about 6 feet and MSE walls will have a maximum height of about 17 feet. Within the alignment, a new proposed bridge from Station 1397+75 to 1410+50 approximately 1,275 feet in length will transverse an existing wetlands area.

Roadway Embankment

The roadway embankment Line A is expected to require up to 6 feet of fill and 5 feet of cut for ditches to achieve final grades. Maximum slopes anticipated are 2H:1V. The critical cross section was selected at approximate Station 1375+00 for settlement and global stability analyses.

Settlement

At a representative cross-section at Station 1375+00, approximately 6 feet of new fill is proposed. It is anticipated that about 9 inches of settlement will occur within 3 months of embankment construction. Primary consolidation is estimated to complete after about 6 months following placement of the full embankment. Total settlement over the design life is expected to be about 11 inches. This settlement exceeds the acceptable performance limit of 3 inches (EV-01B) for earth embankments over the design life. Installation of wick drains, a surcharge program, or other methods may be required to reduce these post-construction settlements.

Global Stability

The results of the global stability analysis at Station 1375+00 indicate the embankment meets the required resistance factors listed in Chapter 10 of the SDCOT *GDM* for the Strength limit state. Extreme Event limit states do not meet required resistance factors, indicating that reinforcement or some other ground improvement will be required in order to satisfy SCDOT requirements.

Retaining Walls A1, A2, and A3

Retaining walls A1, A2, and A3 are located at the northern end of the Line A Bridge. Wall A1 has a maximum wall height of about 14 feet. Wall A2 has a maximum height of approximately 18 feet and will separate the Line A grade from I-526. Wall A3 has a maximum height of about 10 feet and will be the Line A Bridge Abutment wall. The walls are proposed to be MSE walls. For this preliminary analysis, we have assumed a reinforcement length of 0.7H with a minimum length of 8 feet. Walls A1 and A2 were evaluated at approximate Station 1398+00 for settlement and global stability analyses.

Settlement

At the representative cross-section at Station 1398+00 approximately 18 feet of new fill is proposed behind Wall A2 it is anticipated that about 4 inches of settlement at the wall face and about 7 inches of settlement within the footprint of the reinforcement will occur within 3 months of construction. Primary consolidation is estimated to complete about 1 year after placing the full height of the wall. Total settlement over the design life is expected to be about 6 inches at the wall face and 10 inches within the footprint of the reinforcement. Wall A2 is immediately adjacent to Wall A3 and the Line A north abutment. This settlement exceeds the acceptable limits of 2 inches (RV-01B) for earth retention structures and 0.5 inches (EV-05B) at bridge abutment transitions over the design life. Installation of wick drains, a surcharge program, or other methods may be required to accelerate settlement.

Global Stability

The results of the global stability analysis at Station 1398+00 indicate Walls A1 and A2 meet the required resistance factors listed in Chapter 10 of the SDCOT *GDM* for the strength limit state. The resistance factors for the extreme event limit state were not met for some of the wall sections. Additional reinforcement or other ground improvements should be considered to meet SCDOT requirements.

Retaining Walls A4 and A5

Retaining Walls A4 and A5 are located at the southern end of Line A Bridge. Wall A4 has a maximum wall height of about 15 feet. Wall A5 has a maximum height of about 8 feet and is the Line A bridge south abutment wall. The walls are proposed to be MSE walls. For this preliminary analysis, we have assumed a reinforcement length of 0.7H with a minimum length of 8 feet. Wall A4 was selected for global stability analysis at the critical cross section at approximate (Line B) Station 1407+75 was used for settlement and global stability analyses.

Settlement

At a representative cross-section at Station 1407+75, approximately 15 feet of fill is proposed. It is anticipated that about 7 inches of settlement at the wall face and approximately 14 inches within the footprint of reinforcement will occur within 3 months of construction. It is estimated that 3 months after construction of the full wall height, the settlement will have reached the end of primary consolidation. Total settlement over the design life is expected to be about 8 inches at the wall face and 16 inches within the footprint of the reinforcement. Wall A4 is immediately adjacent to Wall A5 and the Line A bridge southern abutment wall. This settlement exceeds the acceptable limits of 0.5 inches (EV-05B) at bridge abutment transitions over the design life. Installation of wick drains, a surcharge program, or other methods may be required to reduce these post-construction term settlements.

Global Stability

The results of the global stability analysis at Station 1047+75 indicate Wall A4 meet the required resistance factors listed in Chapter 10 of the SDCOT *GDM* in the Strength limit state, but not the Extreme Event limit state. Some additional reinforcement or ground improvements should be considered in order to meet SCDOT requirements.

Line A Bridge Foundations

The proposed Line A Bridge will be approximately 10 spans (11 bents), approximately 50 feet wide, and 1,275 feet long. Bridge concept plans show steel pile end bents and drilled shaft interior bents. Bridge foundations are discussed in an earlier section of this report. Abutments are supported with MSE walls at the end bents.

New Line B

Relocated New Line B is the off-ramp from I-526 westbound connected to Relocated Line 3. The Relocated alignment begins at approximate Station 1391+00 and continues southeast approximately 2,075 feet to Station 1411+75 where it intersects Relocated Wando Park Blvd and ties in with WP Connector. Within the alignment, a new proposed bridge from Station 1398+50 to 1407+75 approximately 925 feet in length will traverse an existing wetlands area. Also, within the alignment, new retaining walls are proposed from approximate Stations 1391+25 to 1398+50 (Wall B1), 1398+50 (Wall B2), 1407+75 to 1408+50 (Wall B3), and 1407+75 (Wall B4).

Retaining Walls

Retaining walls B1 and B2 are located at the northern end of the Line B Bridge. Wall B1 has a maximum wall height of about 24 feet. Wall B2 has a maximum height of approximately 18 feet and is the Line B Bridge north abutment wall and ties into Wall 3C. Wall B2 was not analyzed as Walls 3B and 3C were the critical sections at the Line B North abutment. Wall B3 has a maximum height of about nine feet and will be the Line B Bridge south abutment wall. Wall B4 has a maximum height of about six feet. The walls are proposed to be MSE walls. For this preliminary analysis, we have assumed a reinforcement length of 0.7H with a minimum length of 8 feet. Wall B4 was evaluated at approximate Station 1407+75 for settlement and global stability analyses.

Settlement

At the representative cross-section at Station 1407+75, approximately 9 feet of new fill is proposed. It is anticipated that about 4 inches of settlement at the wall face and about 14 inches at the center of the Line A/B embankment will occur within 3 months of construction. Primary consolidation is estimated to complete after 3-months of placement of the full wall height. Total settlement over the design life is expected to be about 5 inches at the wall face and 16 inches at the center of the Line A/B embankment. Wall A4 is immediately adjacent to Wall A5 and the Line A South abutment wall. This settlement exceeds the acceptable limits of 0.5 inches (EV-05B) at bridge abutment transitions over the design life. Installation of wick drains, a surcharge program, or other methods may be required to reduce these post-construction settlements.

Global Stability

The results of the global stability analysis at Station 1407+75 indicate Wall B3 meet the required resistance factors listed in Chapter 10 of the SDCOT *GDM* in the Strength limit state, but not the Extreme Event limit state. Additional reinforcement or ground improvements should be considered in order to meet the SCDOT requirements.

Line B Bridge Foundations

The proposed Line B Bridge will be 7 spans (6 bents), approximately 60 feet wide, and 925 feet long. Bridge concept plans show steel pile end bents and drilled shaft interior bents. Bridge foundations are discussed in an earlier section of this report. Abutments are supported with MSE walls at the end bents.

New Wando Port Connector

Relocated Wando Port Connector is a part of State Road S-10-97 (Long Point Road). The relocated alignment begins construction from Station 100+00.00 and continues north approximately 2,500 feet to Station 125+00 at the confluence of Lines A and B.

Roadway Embankment

The roadway embankment Wando Port Connector is expected to require up to 11 feet of fill and minimal cut for ditches to achieve final grades. Maximum slopes anticipated are 2H:1V. The critical cross section was selected at approximate Station 122+25 and is used for settlement and global stability analyses.

Settlement

At the representative cross-section at Station 122+25, approximately 11 feet of new fill is proposed. It is anticipated that about 9 inches of settlement will occur within 3 months of embankment construction. Primary consolidation is estimated to conclude after about 3 months after full height of embankment is placed. Total settlement over the design life is expected to be about 11 inches.

Global Stability

The results of the global stability analysis at Station 122+25 indicate the embankment meets the required resistance factors listed in Chapter 10 of the SDCOT *GDM* for the Strength limit state, but not for the Extreme Event limit state. Additional reinforcement or ground improvements should be considered in order to meet the SCDOT requirements.

Design and Construction Considerations

Strength Limit

Global Stability

From the analyzed cross sections, the embankments and the earth retaining structures meet the requirements of the *GDM* in the static conditions.

Deep foundations

Depending on the construction sequencing and methods at the bridge abutments, downdrag loads and the appropriate load factors should be considered as these would likely control the design tip elevations.

Service Limit

Settlement

The estimated settlements presented in this report were typically evaluated where the most fill was anticipated for each alignment. In many of the cross sections analyzed, settlement occurring after an estimated 3-month construction period and through the design life are expected to be greater than the performance limits listed in SCDOT *GDM* tables 10-12 through 10-15 for embankments and 10-16 and 10-17 for earth retaining structures.

To mitigate the potential settlement issues for embankments, additional waiting periods (between 6 and 12 months) may be required to allow for primary consolidation to complete prior to the construction of pavements. To accelerate the consolidation of compressible clay-like soils encountered, prefabricated vertical drains (PVDs) or wick drains could be used to reduce the waiting period. Use of additional surcharge to be removed during final grading could be used to minimize the post-construction settlements. Additionally ground improvements such as aggregate piers or column supported embankments could be used at bridge abutment locations where the performance limits are more stringent.

Extreme Event

Cyclic Liquefaction and Softening

A preliminary liquefaction analysis was performed using the CPT sounding data and an ADRS Curve received from SCDOT. Analysis shows that liquefaction is likely a widespread issue for the site. Cyclic liquefaction is likely to occur in the Upper Sand and Sand Lens layers across the site for both FEE and SEE design events. Liquefaction in these layers can occur down to Elevation -46 (NAVD88). The Pre-Pleistocene sediments (Young Marl and Cooper Marl) are considered to not experience liquefaction due to aging effects. Liquefaction is likely to induce vertical and horizontal displacements of embankments and earth retaining structures that exceed the performance limits of Chapter 10 in the SCDOT *GDM*. To mitigate/limit the effects of cyclic liquefaction, Earthquake Drains could be used to reduce the pore pressure ratio to a level that prevents or limits the potential for liquefaction. Other more extreme ground improvements may be considered to reduce settlements and earthquake movements such as soil mixing or aggregate columns.

Global Stability

Under pseudo-static loading, the requirements for global stability were not met. As mentioned previously, strength parameters used in the pseudo-static analysis were similar to the strengths at the Service limit. Newmark charts indicate that seismic displacements up to about nine inches are expected; however, given that the analyses indicate instability with peak soil strengths, we conclude that utilizing residual strengths of soils will result in greater instability after an earthquake. Based on our preliminary seismic hazard analysis of CPT soundings, large lateral displacements were calculated. This indicates that post-earthquake displacements are excessively high and ground modifications to limit pore pressure generation (or increase soil strength) during the seismic event must take place to reduce displacements.

APPENDIX A

ACCELERATION DESIGN RESPONSE SPECTRUM CURVES

3-Point Acceleration Design Response Spectrum

SCDOT v3.2 - 06/01/2023

Project ID:	P041314			Latitude:	32.8420
Route:	I-526	County:	10 - Charleston	Longitude:	79.8707
Project:	I-526 & Long Point Road Interchange				

Designer:	N. Harman - Support
Date:	7/13/2023

Design EQ	PGA	S_{DS}	S_{D1}	M_W	R	PGV	D_{a5-95}	T 'o
	g	g	g	-	km	inches/sec	sec	sec
FEE	0.23	0.48	0.23	7.30	25.40	8.70	23.85	0.17
SEE	0.45	0.95	0.75	7.30	26.13	28.62	23.96	0.20

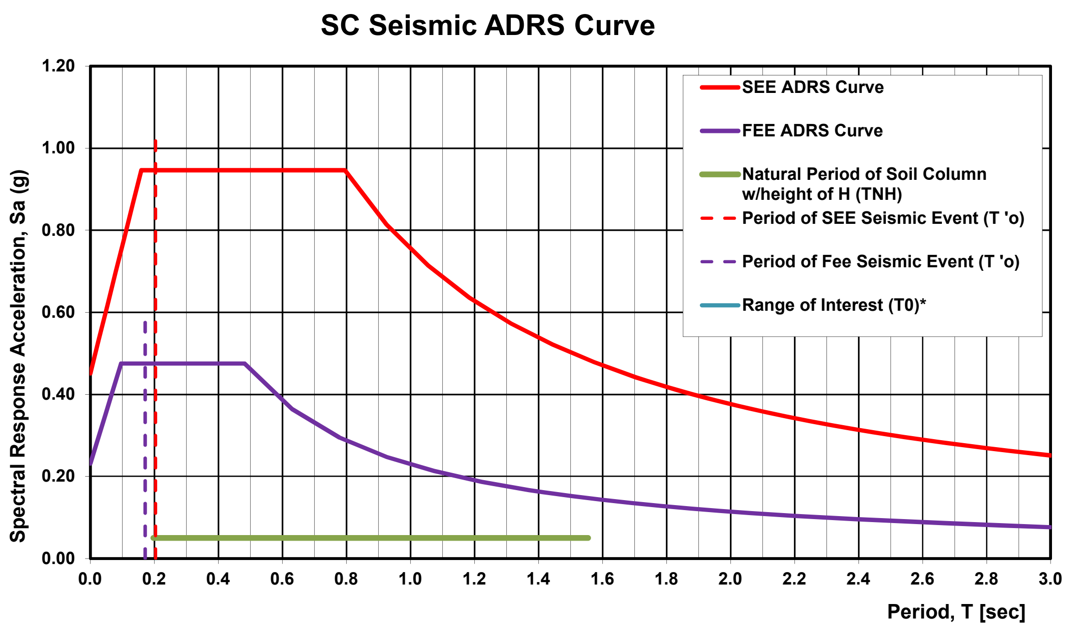
Damping:	5%
Geologic Condition:	Geologically Realistic (Q = 100)*
	SCCP
ADRS Location within Soil Column:	At Ground Surface

South Carolina Coastal Plain

*Same Geologic Condition as used in SCENARIO_PC (2006)

Fundamental Period of Structure, T_0^*	Range of Interest**		$V^*_{s,H}$	H	T_{NH}	
	sec				sec	
sec	$0.5*T_0$	$2.0*T_0$	ft/sec	ft	$(4*H)/V^*_{s,H}$	$(6*H)/V^*_{s,H}$
0.00	0.00	0.00	1234.34	320.00	0.20	1.56
0.00	0.00	0.00				

H = B-C Boundary



	FEE Data T	FEE Data S_a		SEE Data T	SEE Data S_a
	0.00	0.231		0.00	0.451
	0.02	0.272		0.03	0.534
	0.03	0.313		0.05	0.616
	0.05	0.353		0.08	0.699
	0.06	0.394		0.11	0.781
	0.08	0.435		0.13	0.864
To	0.10	0.475	To	0.16	0.946
	0.13	0.475		0.21	0.946
	0.16	0.475		0.27	0.946
	0.19	0.475		0.32	0.946
	0.22	0.475		0.37	0.946
	0.26	0.475		0.42	0.946
	0.29	0.475		0.48	0.946
	0.32	0.475		0.53	0.946
	0.35	0.475		0.58	0.946
	0.39	0.475		0.64	0.946
	0.42	0.475		0.69	0.946
	0.45	0.475		0.74	0.946
Ts	0.48	0.475	Ts	0.80	0.946
	0.63	0.364		0.93	0.814
	0.78	0.294		1.06	0.714
	0.93	0.247		1.18	0.636
	1.07	0.213		1.31	0.573
	1.22	0.187		1.44	0.522
	1.37	0.167		1.57	0.479
	1.52	0.151		1.70	0.442
	1.67	0.137		1.83	0.411
	1.81	0.126		1.96	0.384
	1.96	0.117		2.09	0.360
	2.11	0.108		2.22	0.339
	2.26	0.101		2.35	0.320
	2.41	0.095		2.48	0.304
	2.56	0.090		2.61	0.288
	2.70	0.085		2.74	0.275
	2.85	0.080		2.87	0.262
	3.00	0.076		3.00	0.251

**The SEOR is encouraged to check the fundamental period of the structure versus the period of the seismic event and the period of the site. According to LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations, FHWA-NHI-11-032, GEC No. 3 "So, the damage potential of an earthquake ground motion increases when the predominant period of the earthquake motion is close to the resonant period of the site and when the resonant period of the site is close to the fundamental period of the structure. The damage potential of an earthquake ground motion is greatest when all three of the predominat or fundamental periods coincide."