Fort Ord Regional Trail & Greenway
Retaining Wall No. 1
Del Rey Oaks, California

(Post Mile MON 0.921)

Foundation Report for Retaining Wall No. 1

Report Status – Final

April 2023

McMILLEN
JACOBS
ASSOCIATES

April 7, 2023

Ms. Lindsey Van Parys
GHD
2200 21st Street
Sacramento, CA 95818

Subject: Final Foundation Report for Retaining Wall No 1
Transportation Agency for Monterey County
Fort Ord Regional Trail & Greenway
Del Rey Oaks, California

Dear Lindsey,

We are pleased to submit the attached Final Foundation Report for the Transportation Agency of Monterey County's (TAMC) Retaining Wall No. 1 at the bridge crossing for the planned Fort Ord Regional Trail & Greenway (FORTAG) in Del Rey Oaks, California.

We appreciate the opportunity to serve GHD and TAMC on this project. Please contact us if you have any questions about this report.

Sincerely,

McMILLEN JACOBS ASSOCIATES

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1.0 Introduction

McMillen Jacobs Associates (MJA) has prepared this foundation report for the Transportation Agency of Monterey County's (TAMC) planned Retaining Wall No. 1 connected to State Route (SR) 218 Undercrossing Bridge (bridge) of the Fort Ord Regional Trail & Greenway (FORTAG) Project in Del Rey Oaks, California (Figure 1).

This report summarizes findings of a geotechnical investigation and presents geotechnical design recommendations for Retaining Wall No. 1 near SR 218 Post Mile Mon 0.921 (the project area). The recommendations presented herein are based on the Retaining Wall No. 1 General Plan Drawing dated 4/7/23 (General Plan) from Cornerstone Structural Engineering Group, 95% submittal Civil and Structural Plans from GHD, Caltrans comments received during a design team conference call on March 17, 2023, and our interpretation of the geotechnical findings for the project area that are summarized in Sections 3.0 and 5.0 herein. In addition to this report, we have provided a Geotechnical Design Report (GDR) for the project and a Geotechnical Foundation Report that provides recommendations for the SR 218 Undercrossing Bridge under two separate and independent reports (see MJA 2023a and 2023b).

2.0 Project Description

The project consists of a planned SR 218 undercrossing bridge and a retaining wall connected to the bridge that extends about 130 feet northeast from SR 218 (Figure 1). A partial summary of project components is provided in Table 1. A separate discussion of the ground conditions at the planned SR 218 undercrossing was included in the Structure Selection Fact Sheet for the project (MJA, 2021a). Initially, the trail was planned to undercross SR 218 within a structure to be installed by trenchless or tunneling means. The geotechnical investigation that was initially completed for the project was scoped based on that initial plan (MJA, 2021b). Subsequent to the completion of borings B-4 and B-5b, it was determined by the design team that conflicts with existing utilities and elevation requirements for the undercrossing would prevent the installation of an undercrossing structure by trenchless or tunneling means, and that an undercrossing bridge would be required (see Section 5.3.1).

The scope of the initial geotechnical investigation for a trenchless option included 40-foot-deep exploration borings (B-4 and B-5b as described in Section 5.3.2) at the planned crossing. Boring B-4 was drilled near what is now the southeast corner of the planned bridge and encountered elastic silt and peat below a depth of 34.5 feet. This type of soil would not have had a significant impact on the originally planned design for trenchless/tunnel installation of an SR 218 undercrossing structure; however, it does have an impact on the current design of a deep foundation support for the planned bridge (i.e., driven piles). Consequently, it was necessary to perform additional deeper subsurface explorations to define the ground conditions in the project area for the purpose of designing driven-pile support for the planned bridge. Two cone penetration tests (CPTs) were completed on the south side of the bridge at the locations illustrated in Figure 1. The CPTs were pushed to refusal as indicated in Appendix D.

Four CPTs were planned, one at each abutment corner of the bridge, but only two CPTs could be advanced on the south side of the bridge. The two planned CPTs on the north side of the bridge encountered sand consistent with utility backfill in the pre-CPT hand auger holes excavated to clear the upper 5 to 6 feet prior to CPT advance. For Safety, the CPT contractor's policy is to not advance CPTs in material that is consistent with trench backfill materials.

Retaining Wall No. 1 will be approximately 136.3 feet long and approximately 3 to 15 feet high. Portions of the retaining wall will be designed as a tiered retaining wall system with an upgradient upper retaining wall (Retaining Wall No. 2) located at least 11 feet from Retaining Wall No. 1. Geotechnical investigations and related recommendations for the design of Retaining Wall No. 2 are not part of this scope of work, which only addresses Caltrans requirements. Elevations and coordinates referred to in this report are based on the 1988 North American Vertical Datum (NAVD 88) and the 1984 Whole Geodetic System (WGS 84), respectively.

Table 1. Summary of the Project Components

Component	SR 218 Stations[1]	Bike Trail Stations		Length (feet)	Maximum Height (feet)
		Begin	End		
Retaining Wall No.1	104+88	202+00	203+30	136.3	12

[1] SR 218 and bike trail stationing based on project drawings by Cornerstone (2023).

3.0 Geotechnical Investigation

The location of completed borings and cone penetration tests (CPTs) for the project are mapped in Figure 1. Boring log legends and the logs of the borings are provided in Appendix A and B, respectively, and the CPT results are provided in Appendix D. Table 2 summarizes information from the borings and CPTs. The detail descriptions of the field explorations are provided in the GDR prepared by MJA (2023a).

Table 2. Partial Summary of Borehole/CPT Data

Borehole/ CPT No. (1)	Nearby Project Component	Northing/Easting (Latitude/Longitude) (2)		Ground Surface Elevation, ft (3)	Depth, ft	Completion Date
B-4	Bridge	36.593614	-121.836221	84.5	40	8/2021
B-5a(5)	Bridge; Retaining Wall No. 1	36.594007	-121.836316	95.5	2	8/2021
B-5b	Bridge; Retaining Wall No. 1	36.593986	-121.836339	95.5	40	8/2021
CPT-1A(4)	Bridge	36.593750	-121.836247	91.5	2.4	2/2023
CPT-1B	Bridge	36.593807	-121.836247	91.5	63.4	2/2023
CPT-2	Bridge	36.593714	-121.836211	88.5	66.5	2/2023

(1) Locations mapped in Figure 1. Logs and results provided in Appendices B (borings) and D (CPT results).

(2) From Google Earth.

(3) Based on topographic survey by Whitson (2020).

(4) CPT rig shifted when refusal was encountered on suspected concrete and had to be abandoned and relocated to 1B.

(5) Refusal on concrete. Abandoned and relocated to 5B.

4.0 Laboratory Testing Program

Moisture content, unit weight, Atterberg limits, grain size, unconfined compression, soil corrosion, and direct shear tests were performed on ground samples retrieved from project borings. The results of the tests are summarized in the boring logs provided in Appendix B, and in laboratory test results provided in Appendix C.

5.0 Geotechnical Conditions

5.1 Geology

The project area is located within the Coast Ranges Geomorphic Province, which is characterized by northwest-trending mountain ranges and valleys that run subparallel to the region's fault zones. The region's fault zones are summarized in Section 0. The Coast Ranges generally consist of Mesozoic and Cenozoic sedimentary strata overlain by alluvium. Geology maps of the region have been completed by several authors (e.g., Hartwell et al., 2016; Dibblee and Minch, 2007; Clark et al., 1997; Dupre 1990; Dibblee et al., 1974), including those provided in Figures 2.1 and 2.2.

The planned SR 218 Undercrossing Bridge and the portion of Retaining Wall No.1 that will be connected to the bridge are mapped to be underlain at the ground surface by Holocene-age alluvial deposits of unconsolidated gravel, sand, silt, and clay that were likely deposited by the nearby Laguna Del Rey Creek. The remaining portion of the Retaining Wall No.1 is mapped to be underlain at the ground surface by Holocene-age alluvial fan deposits and highly weathered Miocene-age Monterey Formation. The Monterey Formation includes calcareous to siliceous claystone, siltstone, and sandstone; porcelanite; chert; diatomite; and bentonite.

Debris flows are a common form of slope failure in Monterey County; however, no evidence of landslides or debris flow instability was observed in the project area during our geotechnical investigation, and there are no known landslides or debris flow instabilities recorded for the project area in the U.S. Geological Survey's Landslide Inventory database.[1]

Peat was logged at a depth of 37 feet in boring B-4. Peat is a soil type that contains a high percentage of organic matter. The peat encountered in boring B-4 is most likely from organic matter that accumulated in historical meanders of Laguna Del Rey Creek. The CPTs also confirmed the presence of a very soft layer between about 40 and 55 feet below ground surface affirming the presence and thickness of these deposits.

5.2 Surface Conditions

Land use near the project area consists of roadways, commercial and residential properties, and recreational and preservation areas. Surface conditions at the planned bridge and retaining wall consist of a paved roadway and vegetation (see Figures 1, 3, and 4). The vegetation along the planned retaining wall predominantly consists of trees and shrubs. Overhead power lines run parallel to the northwest shoulder of SR 218.

Based on the topographic survey map by Whitson Engineers (2020) provided in Figure 4, SR 218 embankments at the location of the bridge are steeper on the northeast side and flatter on the southwest side. As indicated in Figure 1, a retaining wall will be constructed along the slope located northeast of the bridge. This slope has a gradient of less than 1.5H:1V and heights between 5 and 12 feet.

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[1] https://usgs.maps.arcgis.com/apps/webappviewer/index.html?id=ae120962f459434b8c904b456c82669d

5.2.1 Near Surface Soil Mapping

Near surface soils in the project area are mapped and described in Figure 3. The planned bridge and retaining wall are mapped to be in Rindge muck. As indicated in Figure 3, Rindge muck is classified by the U.S. Soil Conservation Service as Peat. Areas mapped as Rindge muck have a seasonal high-water table between 0 and 6 feet below ground surface. Risk of corrosion in Rindge muck is high in uncoated steel and moderate in concrete. CPT-1B confirms the presence of the peat layer encountered in B-4 at a depth between 40 and 55 feet based on the cone and sleeve resistance with a CPT interpretation of sensitive fine-grained materials. CPT-2 encountered slightly stronger clays at these depths which may reflect a transition out of the sensitive peaty clays.

5.3 Subsurface Conditions

5.3.1 Existing Subsurface Utilities

The approximate location of existing utilities mapped in the project area are shown in Figures 4 and 5.1, and include the following:

- 4" X 2 AT&T lines (to be relocated)
- 2" AT&T line (to be relocated)
- 8" W (water pipeline, to be relocated)
- 16" W (water pipeline, to be relocated)
- 4" G (gas pipeline, to be relocated)
- 16" SD (storm drain pipeline, to be removed)
- 12" SS (sanitary sewer pipeline, to remain)

There may be other utilities in the area that are not shown in these figures. We have no firsthand information as to the size and shape of the excavations that were performed to install utilities in the project area (e.g., with vertical and/or side-sloped sidewalls), nor for the material that was used to backfill the excavations—including materials used as a foundation for and below the utility, for embedment used immediately around the utility, and for backfill above the utility and below the pavement surface.

Overhead utilities are also present at the site on the north side of SR 218 above the road shoulder.

5.3.2 Exploration Borings and CPTs

The location of exploration borings and CPTs completed for the project are mapped in Figure 1, and a subsurface profile across the slope and through the retaining wall of the project is provided in Figure 5. Boring log legends and the logs of the borings are provided in Appendix A and Appendix B, respectively, and the CPT results are provided in Appendix D. A partial summary of information from the boring logs and CPTs is provided in Table 3 and Table 4, respectively.

Table 3. Partial Summary of Information from Project Boring Logs

Boring(1)	Elev (ft)(2)	BGS Depth(3) (ft)			Bedrock or USCS(4) Group Symbol	SPT(5) (N)	Qu(5) (ksf)	Notes(6)
		Total	to GW (Seep) / Level	Interval (ft)				
B-4	84.5	40	(10)/31.0	0–16	SM/SC	5, 7	-	fill in upper 5’
				16–17.5	ML	4	-	$W_c = 43$, $\gamma_d = 74$ pcf
				17.5–23.5	SM/SC	9	-	
				23.5–25.5	CL/CH	-	-	
				25.5–34.5	SP-SM	12, 20	-	
				34.5–37	MH	-	-	diatomite/bentonite (?)
				37–40	MH & PT	4	-	LL =112, PI = 42
B-5a	95.5	2	NE	0–2	SP	-	-	refusal in fill on concrete & metal
B-5b	95.5	40	26.0	0–3	SM	-	-	fill
				3–12	SP-SM/SP-SC	4	-	concrete in fill from 10.5 to 12 feet that was eventually bypassed
				12–32	SM/SC	3, 4, 4, 5	-	$W_c = 78$, $\gamma_d = 52$ pcf (tuffaceous)
				32–40	MH & Bedrock	26, 27	0.4	Monterey Formation (?)

(1) Drilled in August 2021. See Figure 1 for mapped boring locations. See logs and lab test results in Appendices B and C.

(2) Ground surface elevation from Whitson (2020).

(3) BGS = below ground surface. GW = groundwater. NE = not encountered. Groundwater seepage depth during drilling and groundwater level depth measured in boring at time of backfilling, not necessarily the static groundwater level depth.

(4) Unified Soil Classification System (USCS) and group symbol defined in Appendix A.

(5) N = greatest ASTM D1586 Standard Penetration Test Blow Count for interval. Qu = unconfined compressive strength.

(6) W_c = moisture content. γ_d = dry density.

Table 4. Partial Summary of Information from CPTs

CPTs (1)	Elevation (ft) (2)	BGS Depth (3) (ft)		Soil Behavior Type (SBT) (4)	Notes(5)
		Total	to GW		
CPT-1A	90.0	2.38	NE	sands, sand mixtures	CPT rig shifted when refusal was encountered on suspected concrete and had to be abandoned
CPT-1B	91.5	63.4	17.6	sand mixtures, silt mixtures, sands, clays	Refusal on concrete at ~11 feet, punched through concrete and advanced to refusal N60 value ranging from 1 to 70 bpf Sensitive fine-grained layers
CPT-2	88.5	66.5	14.6	sand mixtures, sands, silt mixtures, clays	N60 value ranging from 1 to 66 bpf

(1) Performed in February 2023. See Figure 1 for mapped CPT locations. See CPT results in Appendix D.
(2) Ground surface elevation from Whitson (2020).
(3) BGS = Below ground surface. GW = Groundwater. NE = not encountered.
(4) SBT scatter plots provided in Appendix D.
(5) N60 = SPT N value at 60% energy calculated from q_t/N ratios assigned to each SBT zone using Robertson and Wride (1998).
bpf = blow per foot.

5.4 Groundwater Level

The depth to groundwater was measured and logged in project borings during and immediately after their drilling (see logs of borings in Appendix B) and is summarized in Table 5. The groundwater level estimated from project CPTs is also summarized in Table 5, and is based on the shallowest pore pressure dissipation tests results performed in and during the CPTs.

The project CPTs were performed in February 2023, and therefore the groundwater level estimated in the project CPTs is during a rainy, winter-time season as compared to a dry summer-time season like that when the project borings were completed in August 2021. The depth to the groundwater level at the project site during project construction will vary relative to changes in seasons (i.e., rainfall), elevation, topography, and the proximity of drainageways, water bodies, and dewatering activities (e.g., wells). The depth to groundwater typically shallows during the rainy season as it collects in areas of low elevation and basinal topography (e.g., Laguna Del Rey and the Frog Pond) and near drainageways (e.g., Laguna Del Rey Creek). Areas of shallow perched groundwater (i.e., groundwater located above the elevation of static groundwater levels) may exist in the project area, including that which could be trapped within porous and permeable import materials (e.g., drain rock) that were used to backfill existing parallel or crossing utility excavations.

Based on groundwater elevations encountered in the project borings and CPTs, a groundwater table elevation of +74 feet was considered for design purposes.

Table 5. Measured Groundwater Level in Project Borings

Boring [1]	Ground Surface Elevation (ft) [2]	Groundwater Level or Piezometric Elevation [3]		Date Measured (mo/yr)	Notes
		Depth BGS (feet)	Elevation (feet)		
B-4	84.5	31.0	53.5	8/2021	
B-5a	95.5	NE	NE	8/2021	refusal in fill on concrete & metal at depth of 2 ft
B-5b	95.5	26.0	69.5	8/2021	
CPT-1A	90.0	2.4	NE	2/2023	CPT rig shifted when refusal was encountered on suspected concrete and had to be abandoned
CPT-1B	91.5	17.6	74.0	2/2023	Suspected concrete encountered at ~10 feet, punched out of the way to continue advance
CPT-2	88.5	14.6	74.0	2/2023	

(1) See map of boring and CPT locations in Figure 1, and logs of borings and CPT results in Appendix B and Appendix D, respectively.

(2) Ground surface elevations from Whitson (2020).

(3) NE = not encountered.

5.5 Corrosion Evaluation

The corrosion potential was evaluated using the results of limited laboratory testing on samples obtained during the subsurface exploration. One soil sample was collected for corrosion testing during our subsurface investigation. The result of the corrosivity test is reported in Appendix C and summarized in Table 6. Based on the criteria provided in Caltrans corrosion guidelines (Caltrans, 2021a), the sample of material tested does not meet the definition of a corrosive environment.

Table 6. Soil Corrosion Test Summary

Boring ID	Elevation (feet)	Minimum Resistivity (Ohm-Cm)	pH	Chloride Content (ppm)	Sulfate Content (ppm)
B-4	74	1,908	7.1	29	417

5.6 Seismic Information

5.6.1 Site Seismic Parameters

Based on the Vs30 map prepared by Branum et al. (2016), the average shear-wave velocity for the upper 30 meters (98 ft) of ground (V_{S30}) at the project site is approximately 1,150 ft/sec (Figure 6). However, Vs30 value at the project site determined using the project CPT data and All Soils method provided in PEER Guidelines (Wair et al., 2012) was approximately 720 ft/sec, which is significantly lower than the mapped Vs30 value. Therefore, Vs30 of 720 ft/sec, which is consistent with a seismic Site Class D (see Table 7) was used for the design purposes to be conservative.

Table 7. Seismic Site Classification

Seismic Site Class(1)	Average Shear Wave Velocity for the Upper 30 Meters of Ground (V_{S30}) (1)	Generic Description(1)
A	> 5,000	Hard rock
B	> 3,000 to 5,000	Medium hard rock
BC	> 2,100 to 3,000	Soft rock
C	> 1,450 to 2,100	Very dense sand or hard clay
CD	> 1,000 to 1,450	Dense sand or very stiff clay
D	**> 700 to 1,000**	**Medium dense sand or stiff clay**
DE	> 500 to 700	Loose sand or medium stiff clay
E	> 500 ft/s	Very loose sand or soft clay
(1) Modified from ASCE 7-22 Table 20.2.1.		

5.6.2 Ground Motion Parameters

Design ground motion parameters for the project are provided in Table 8 and Appendix E. These parameters were determined based on Caltrans' Design Acceleration Response Spectrum Module (version 3.0.2) (Caltrans, 2020a).

Table 8. Caltrans-Based Ground Motion Parameters

Project Component ID	Site Parameters			Design Ground Motion Parameters(1) (Return Period = 975 years)		
	Location		Shear-Wave Velocity V_{S30} (m/sec)	Horizontal Peak Ground Acceleration	Mean Earthquake Moment Magnitude	Mean Site-to-Fault Source Distance (km)
	Latitude (degrees)	Longitude (degrees)				
STA 201+75	36.5937	-121.8362	360	0.49g	6.75	26.4

(1) Based on Caltrans web tool ARS Online (Version 3.0.2): https://arsonline.dot.ca.gov/.

5.6.3 Fault Rupture

Major plate boundary faults and lesser-known smaller faults near the project area are shown in the U.S. Geological Survey's Fact Sheet 2016-3020 provided in Figure 7. The nearest Alquist-Priolo Earthquake Fault Zone to the project area is for the San Andreas Fault, located approximately 25 miles to the north and east (see Figure 7 and CGS 2018). Figure 7 shows that the Reliz Fault (Fault No. 27 in Figure 7) is located several miles to the northwest of the project area, and that the Monterey Bay-Tularcitos Fault (Fault No. 29 in Figure 7) is located 1½ miles southwest of the project area. Neither the Reliz Fault nor the Monterey Bay-Tularcitos Fault are associated with an Alquist-Priolo Earthquake Fault Zone (CGS 2018).

As is shown in Figure 2.1, the Chupines Fault and the Seaside Fault have been mapped as concealed faults (i.e., fault traces that have been covered by younger unfaulted material, and therefore not presently visible at the ground surface) that have been inferred by some mappers (e.g., Hartwell et. al., 2016) to occur between the Reliz Fault and the Monterey Bay-Tularcitos Fault, and near the project area. The inferred location of the Seaside Fault is mapped in Figure 2.1 to be more than 1,000 feet northeast of the project area. The inferred location of the Chupines Fault is mapped in Figure 2.1 to be less than 1,000 feet from the project area. However, the location of the Chupines Fault, if it indeed exists near the project area, is concealed by manmade fills and alluvium.

Clark et al. (2000) argues for possible Holocene activity of the western offshore extension of the Chupines Fault in Monterey Bay based on (1) assertions that the Chupines Fault cuts Holocene deposits and the sea floor in the bay (McCulloch and Greene, 1989), and (2) the location of historic offshore earthquake epicenters in proximity of the general fault trend (e.g., see Figure 8). However, the U.S. Geological Survey identifies the Chupines Fault only as Quaternary; one with displacement within the last 1.6 million years (Bryant 2001). The California Geological Survey and the State of California does not classify the Chupines Fault as Holocene-active, and the Chupines Fault is not associated with an Alquist-Priolo Earthquake Fault Zone (see Bryant 1985, and CGS 2018).

5.6.4 Liquefaction

Liquefaction develops when cyclically induced ground stresses increase pore water pressure within the soil to sufficient levels that the soil loses shear strength and liquefies. Construction vibrations and ground shaking can cause liquefaction. Liquefied soils densify (settle) as pore pressures decrease to static levels and soil particles reconfigure into a denser packing. The extent or degree of liquefaction depends on (1) the distribution of cohesionless sediments (gravels, sands, and very low-plasticity silts) within the deposit, (2) a sufficiently high-water table for the sediments to be saturated, and (3) age of the deposits since the sediments become more resistant with age (Idriss, 2008). The most susceptible soils of the project area are fills, and alluvial and marine deposits.

A liquefaction potential map of the project area from Dupre (1990) is provided in Figure 9. The area of high susceptibility liquefaction appears to coincide with the area mapped as Rindge muck soil, as described in Section 5.2 and in Figure 3. Dupre (1990) map shows that the bridge is located an area mapped as having high liquefaction susceptibility. No liquefaction-related ground effects from historical earthquakes have been mapped specifically in the project area; however, ground settlement from liquefaction during earthquakes in the region has been mapped to have occurred about 1.5 miles northwest in Laguna Del Rey (Youd and Hoose, 1978; Tinsley et al., 1990).

The borings drilled in the vicinity of the project area only provide subsurface information to 40 feet below ground surface. In order to perform a detailed analysis to determine liquefaction potential in the project area, CPTs were pushed to refusal (see final depths in Table 2). A numerical analysis of liquefaction triggering was performed with data from the CPT using CLiq v.3.0 (GEOLOGISMIKI, 2007). Earthquake magnitude, peak ground acceleration at the project site, and assumed groundwater depth below the surface from recent CPT data during an earthquake are summarized in Table 9.

Table 9. Summary of Liquefaction Input Parameters

Exploration(1)	Assumed Groundwater Depth during Earthquake (ft)	Earthquake Magnitude (M)	Peak Ground Acceleration, PGA
CPT-1	17.5	6.81	0.5g
CPT-2	14.5		

(1) See Table 2 for summary of CPT.

Based on liquefaction evaluation guidelines provided in Caltrans Geotechnical Manual (Caltrans, 2020c), the procedure from Youd and Idriss (2001) was used for liquefaction analysis. The data obtained during the CPTs were correlated with lab testing results from project borings for fines content and relative density (SPT "N" blow counts) measurements. The CPT-based analysis results are provided in Appendix F and summarized in Table 10. The factor of safety against liquefaction is plotted in Figures 5.. The CPT based analysis results of both CPTs determined that potentially 5 inches of settlement could occur during a 6.81 magnitude earthquake.

Table 10. Summary of CPT-based analysis results

CPT	Approximated Liquefaction Elevation (feet)	Layer Thickness (feet)	Estimated Seismic-induced Settlement (inches)
CPT-1B	74 to 72	2	5.13
	67.5 to 65.5	2	
	60 to 51	9	
	39.5 to 28.5	11	
CPT-2	74 to 70.5	3.5	4.95
	60.5 to 51.5	9	
	35.5 to 32.5	3	
	28.5 to 25.5	3	

5.6.5 Liquefaction-Induced Lateral Spreading

Lateral spreading is caused by the accumulation of incremental displacements that develop within liquefied soil under cyclic loading. Depending on the number and amplitude of stress pulses, lateral spreading can produce displacements that range from a few inches to tens of feet. As indicated in Section 5.6.4, no liquefaction-related ground effects from historical earthquakes have been mapped in the project area. The groundwater elevation at the project site during the modeled earthquake used for the liquefaction analysis is assumed to be the elevation encountered in the recent CPTs at approximately El. +74 feet, which is below the bottom of the free face elevation at the retaining wall location. Multiple liquefiable soil layers a few feet thick were encountered within a depth of 1.2H from the bottom of the wall toe (where H is the height of the retaining wall), and therefore multiple failure surfaces were considered within this depth range, per Figure 2 in Caltrans Memo to Designers (MTD) 20-15 (2017).

In this case, the top of the slope is assumed to be at +103 feet and the toe of the slope is assumed to be at +80 feet as illustrated in Figure 5. As explained above, failure surfaces within liquifiable layers were considered down to El. +52.4 (1.2H below the bottom of the wall toe). Based on the height of the wall and Caltrans (2020b), the potential for lateral spreading displacement at the retaining wall location can be significant. According to Caltrans MTD 20-15, pseudo-static slope stability analysis was performed using reduced strength parameters for liquefiable layers provided in Table 11. Residual shear strength (S_r) values used in the analysis for liquefiable layers were calculated from the results of the CPT investigation following Equation 82 from Boulanger and Idriss (2014). S_r values were also calculated following Equation 1 in Caltrans MTD 20-15 for the SPT-based investigation but were slightly higher than the CPT-based approach. As such, the CPT-based S_r values were used out of conservatism. The search limits for the critical failure surface were limited to extend laterally no more a distance of 4H from the back of the retaining wall, and vertically to no more than 1.2H depth from the bottom of the wall toe, per Figure 2 in Caltrans MTD 20-15.

Table 11. Soil Parameters for Pseudo-Static Slope Stability Analysis

Elevation (feet)	Soil Type	Unit Weight, γ (pcf)	Shear Strength Parameters	
			Friction Angle, Φ (degrees)	Cohesion, c (psf); Undrained Shear Strength, S_u (psf); Residual Shear Strength, S_r (psf)
100.0–74.0	Sand	100	30	c = 0
74.0–70.5	Sand (Liquefied Layer)	105	$\Phi_u = 0$	$S_r = 232$
				$S_r = 145$
				$S_r = 59$
70.5–67.5	Clay	97	$\Phi_u = 0$	$S_u = 650$
67.5–65.5	Sand (Liquefied Layer)	115	$\Phi_u = 0$	$S_r = 331$
				$S_r = 217$
				$S_r = 103$
65.5–60.5	Clay	105	$\Phi_u = 0$	$S_u = 500$
60.5–51.5	Sand (Liquefied Layer)	115	$\Phi_u = 0$	$S_r = 475$
				$S_r = 336$
				$S_r = 196$
51.5–39.5	Clay	95	$\Phi_u = 0$	$S_u = 400$
39.5–28.5	Sand (Liquefied Layer)	115	$\Phi_u = 0$	$S_r = 827$
				$S_r = 635$
				$S_r = 444$
Below 28.5	Sand	125	35	c = 0

The methodology followed the procedure outlined in the example problem in the Caltrans Geotechnical Design Manual (Caltrans, 2020b). Pseudo-static slope stability analyses were performed with the software program Slide 2 (Rocscience, 2022) using General Limit Equilibrium (GLE) / Morganstern-Price (MP) Method and Spencer Method. The first step was running a pseudo-static slope stability analysis with no horizontal seismic coefficient applied. Results showed the Factor of Safety of greater than 1.0, meaning that down to a depth of 1.2H below the wall toe the slope is not susceptible to liquefaction-induced flow failure. The next step is identifying whether any liquefaction-induced slope failure is likely under seismic loading. This consisted of running a pseudo-static slope stability analysis with a horizontal seismic coefficient (k_h) of 0.25g, half of the design peak horizontal ground motion per guidelines provided in Caltrans Geotechnical Manual for Geotechnical Seismic Design for Earth Retaining Systems (Caltrans, 2021b). Results showed the Factor of Safety was less than 1.0 and a slope failure is likely under seismic loading, therefore further analysis was performed as follows. The results for the stability analyses from Slide 2 program are summarized in Table 12 and presented in Appendix G.

Table 12. Summary of Liquefaction-induced Lateral Spreading Assessment Result

Scenario	Method	Failure Type	Liquefiable Layer Elevation (ft)	Support Resistance, R_{tot} (kips)	Factor of Safety	Required Factor of Safety
Liquefied Condition with $k_h = 0$	GLE/MP	Block Failure	72.25	0	1.07	1.0
	Spencer				1.14	
Liquefied Condition with $k_h = 0$	GLE/MP	Block Failure	66.5	0	1.14	
	Spencer				1.18	
Liquefied Condition with $k_h = 0$	GLE/MP	Block Failure	56	0	1.14	
	Spencer				1.11	
Liquefied Condition with $k_h = 0.25g$	GLE/MP	Block Failure	72.25	0	0.91	
	Spencer				0.99	
Liquefied Condition with $k_h = 0.25g$	GLE/MP	Block Failure	66.5	0	0.92	
	Spencer				0.99	
Liquefied Condition with $k_h = 0.25g$	GLE/MP	Block Failure	56	0	1.03	
	Spencer				1.02	

Furthering this analysis, a pseudo-static slope stability analysis was performed to identify the yield acceleration (the horizontal seismic coefficient which produces a Factor of Safety equal to 1.0). Yield acceleration results are summarized in Table 13 and provided in Appendix G. The lowest yield acceleration was found to occur with a failure surface within the second liquefiable soil layer (between El. +74 feet and +70.5 feet, with the failure plane analyzed at +72.25 feet). Therefore, a failure plane at El. +72.25 feet within this liquefiable layer and the associated yield acceleration were used for soldier pile design against lateral spreading.

Table 13. Yield Acceleration

Scenario	Method	Failure Type	Liquefiable Layer Elevation (ft)	Support Resistance, R_{tot} (kips)	Yield Acceleration (k_y)
Liquefied Condition k_y	GLE/MP	Block Failure	72.25	0	0.03
	Spencer				0.08
Liquefied Condition k_y	GLE/MP	Block Failure	66.5	0	0.05
	Spencer				0.07

6.0 Geotechnical Recommendations

Recommendations provided herein are intended for design and construction of Retaining Wall No. 1 in a safe and economic manner, and to ensure the completed project's useful long-term function. Contractors constructing the project are responsible for:

- Reviewing the project GDR and this foundation report
- Supplementing findings of the project GDR with their own investigations
- Interpreting findings from the project GDR and their own investigations
- Selecting and implementing appropriate construction means, methods, and monitoring

Contractors should be required to successfully construct the project design in a safe manner and such that no existing structure, improvement, or utility becomes damaged during or because of the work required to construct the project. Retaining Wall No. 1 will be constructed to support native and backfill soil that mainly consists of very loose to medium dense sandy materials. Initially, a hollow bar soil nail wall was evaluated as an earth retaining system for Retaining Wall No. 1. However, Caltrans design guide commentary (email from Cornerstone dated June 22, 2022) does not favor the use of hollow bar soil nails for retaining wall support. Therefore, the design team elected to use a Caltrans-preferred soldier pile and lagging system for support of Retaining Wall No. 1.

The General Plan (Cornerstone, 2023) shows the soldier piles as W24 beams embedded in a 36-inch-diameter drilled hole that is backfilled with concrete. The horizontal span (center to center) between soldier piles varies from 5.8 to 8.2 feet, with a maximum height above finished grade of approximately 10 feet. Timber lagging is shown between the soldier piles of Retaining Wall No.1 on the General Plan. The total length of wall is approximately 136.3 feet, with Station 9+79 of the wall being at the west side of the wall (i.e., the connecting point between SR 218 Undercrossing Bridge and Retaining Wall No. 1). Finished grade at the wall is shown to be backfilled on the downslope side of the retaining wall from the bridge abutment to approximately 90 feet up station, with a maximum slope of 1.5H:1.0V per the 95% submittal Civil Plans (GHD, 2022). Cut and fill quantities appear to be minimal above Retaining Wall No. 1, based on the existing and final topographic contours of the 95% submittal Civil Plans (GHD, 2022). Retaining Wall No. 2 is to be constructed above Retaining Wall No.1, with the center of the wall being approximately 11 feet from the edge of Retaining Wall No. 1. The recommendations provided herein are based on boring B-5b and extrapolation of the CPT data (see Figure 1).

6.1 Ground Material Properties, Models, and Loading

Ground material parameters were determined based on the available geologic data, in situ and laboratory test data, and the data from CPTs. Foundation material parameters are shown in Table 14. The water table is assumed to be at +74 feet elevation.

Table 14. Foundation Material Parameters for Design

Elevation (ft)	Soil Type	Unit Weight γ (pcf)	Friction Angle φ' (degrees)	Cohesion s_u (psf)	Active Earth Pressure Coefficient, k_a [(1)]	Passive Earth Pressure Coefficient, k_p [(1, 2)]	Interface Friction Angle, δ	Arching Factor [(3)]
85.0–70.5	Sand	100	30	0	0.30	4.85	15	3
70.5–60.5	Clay	100	0	500	0.37	3.43	12	1
60.5–51.5	Sand	115	30	0	0.30	4.85	15	3
51.5–39.5	Clay	95	0	400	0.45	2.59	0	1
below 39.5	Sand	125	35	0	0.25	6.74	17	3

(1) Earth pressure coefficients are calculated per Section 3-11-5 of AASHTO (2017)

(2) The passive pressure should not exceed 2,000 psf.

(3) Arching factor of 3 for Sand under the assumption that the pile spacing is less than 3 times pile diameter.

Table 15 presents the material parameters for the backfill material, with the material assumed to be free draining with the water table below the retained material. Although the amount of fill to be placed above Retaining Wall No. 1 is minimal, the backfill above the wall has been assumed to significantly increase the average unit weight of the retained material.

Table 15. Preliminary Retained Material Parameters for Design

Material Type	Unit Weight, γ (pcf)	Friction Angle, φ' (degrees)	Cohesion, s_u (psf)	Active Earth Pressure Coefficient, k_a	Interface Friction Angle, δ
Sand	120	30	0	0.3	15.0

Active and passive pressure coefficients for the retained and foundation materials were calculated as specified in the AASHTO LRFD Bridge Design Specifications (AASHTO, 2017). The interface friction angle (δ) between the sands and the retaining wall lagging and soldier piles is assumed to be 50 percent of the soil friction angle. Earth pressures for clay materials were calculated with the assumption that stiff and soft clay has a friction angle of 25 and 20 degrees, respectively, with no cohesion for conservatism. Lateral earth pressures for assessing local stability of Retaining Wall No. 1 are shown in Figure 10. The active and passive pressures shown are from AASHTO (2017), which are taken from Broms (1964 a,b). Surcharge loading from Retaining Wall No. 2 above Retaining Wall No. 1 can be conservatively treated as an infinite line load. In determining passive earth pressure, adjusted pile width should be used to take soil arching effect between soldier piles into account. The adjusted pile width is calculated as the product of the pile width (b; see Figure 10) and the arching factor provided Table 14 (f; see Figure 10).

Dynamic earth pressure from seismic shaking (Pe) is 30H, expressed as pounds per square foot, and should be applied as a triangular distribution over a height of H as illustrated in Figure 10. The resultant should be applied at a distance of 0.3H from the bottom of the retained soil layer (see Figure 10). In addition to the static and dynamic earth pressure provided above, horizontal inertial force due to seismic loading of the wall mass (k_hW_w) and dynamic forces caused by permanent surcharge loads located above the wall ($k_hW_{surcharge}$) should also be considered for seismic design of the retaining wall. Seismic horizontal acceleration coefficient (k_h)value of 0.28g can be used.

6.2 Retaining Wall No. 1 Design Considerations

Due to the minimal quantities of cut and fill expected for the retained material, settlement of the retained and foundation material will be minimal. Drag loads imparted on the soldier piles due to the fill materials will be negligible.

6.2.1 Liquefaction-Induced Settlement and Downdrag

Tip elevations for retaining wall soldier piles are provided in the Foundation Plan prepared by Cornerstone (2023). Tip elevations determined based on lateral earth pressure load were within liquefiable soil layer. Therefore, soldier piles were extended below the liquefiable layer with minimum one foot embedment into the non-liquefiable layer. Estimated liquefaction-induced settlement and downdrag loads for each pile are summarized in Table 16.

Downdrag is the phenomenon in which the pile foundation is subjected to negative/downward skin friction as a result of downward movement/settlement of the ground surrounding the pile. Post-liquefaction residual shear strengths using methods by Boulanger and Idriss (2014) were used for the liquefiable soils, and the full shear strengths were used for the nonliquefiable soil layers to calculate the downdrag loading in accordance with Caltrans liquefaction-induced downdrag manual (Caltrans, 2020d). Axial pile resistance calculated using CPT-based procedures (Eslami and Fellenius Method) presented in the FHWA Design and Construction of Driven Piles Manual (FHWA, 2016) is approximately 275 kips which is greater than the estimated downdrag load.

Since the pile tip elevation is located above the lowest liquefaction zone, the piles are anticipated to have approximately 2.5 inches of settlement due to the lowest liquefaction zone between Elevation 39.5 feet and 28.5 feet, which is acceptable according to the structural designer.

Table 16. Liquefaction Potential at Retaining Wall No. 1

Pile Tip Elevation (feet)	Liquefaction Elevation (feet)	Estimated Liquefaction-induced Settlement (1) (inches)	Downdrag Zone Bottom Elevation (2) (feet)	Estimated Downdrag Load (3) (kips/pile)
50	Elev. 74.0 to 70.5 Elev. 67.5 to 65.5 Elev. 59.5 to 51.5 Elev. 39.5 to 28.5	2.5	Elev. 52.5	175

(1) Estimated liquefaction-induced settlement of pile.

(2) Estimated in accordance with Caltrans liquefaction-induced downdrag manual (Caltrans, 2020d).

(3) Downdrag loads calculated for 36-inch drilled hole for the soldier piles

6.2.2 Liquefaction-Induced Lateral Spreading Mitigation

Figure 11 represents the interactive curve that shows the resisting force from soldier piles prepared by Cornerstone and the displacement response of the sliding mass calculated by MJA for El. +72.25 feet failure plane. The liquefaction-induced lateral spreading displacement was estimated per Caltrans MTD

20-15 (Caltrans, 2017) using the method of Bray and Tavasarou (2007) for Newmark's rigid body type sliding displacement with the parameters provided below:

$(k_h)_y$	-	the coefficient of the yield accelerations determined from Slide2
HPGA	-	horizontal peak ground acceleration for the design ground motion (0.5g)
M_w	-	the moment magnitude of the associated design earthquake (6.81)

Yield acceleration used to estimate the displacement was determined using GLE / Morganstern-Price Method since it has more conservative result compared to Spencer Method (Table 13).

Based on the interactive curve presented in Figure 11 which takes into account the constructed soldier pile retaining wall, there will be approximately 17 inches of displacement along elevation +72.25 feet failure plane. Retaining wall No.1 located northeast of the bridge is constructed to support the pedestrian and bike trail across the existing slope. According to the pseudo-static slope stability analysis performed on the existing slope prior to the retaining wall construction (see Appendix H), there is a potential for liquefaction-induced lateral spreading of the existing slope with an estimated lateral displacement of 24 inches along elevation +72.25 feet failure plane. Based on engineering judgement and discussions with Caltrans, relative ground displacement of 17 inches under the liquefaction condition appears reasonable for the retaining wall across existing slope that is predisposed to preexisting liquefaction-induced lateral spreading potential in its present condition.

6.2.3 Global Stability Analysis

Global stability analysis was performed to determine the minimum pile embedment depth and/or pile tip elevations. The soil parameters listed in Table 14 and Table 15 were used for the analysis. The maximum height of the wall is anticipated to be approximately 10 feet. Slide2 (Rocscience, 2022) was used to evaluate the embedment depth required for global stability of the slope. The global stability analyses were performed for both circular and block failure analysis using Bishop Simplified Method, Janbu Simplified Method, and Spencer Method. Slide2 outputs for the stability analyses are provided in Appendix I. Based on the results, the proposed retaining walls tip elevation provided in General Plan prepared by Cornerstone (2023) meet the minimum factor of safety of 1.5. The results of the global stability analysis are summarized in Table 17.

Table 17. Stability Analysis Results

Wall Type	Method	Failure Type	Factor of Safety	Required Minimum Factor of Safety
Solder pile wall	Bishop simplified	Circular	2.14	1.5
	Janbu simplified		1.97	
	Spencer		2.16	
	Bishop simplified	Block	1.57	
	Janbu simplified		1.53	
	Spencer		1.70	

6.3 Retaining Wall No. 1 Construction Considerations

Lift heights for the construction of Retaining Wall No. 1 should be determined by the Contractor's means and methods for maintaining a stable slope. Gaps between lagging and the retained material should be backpacked to ensure intimate contact. Lagging should be spaced adequately to prevent the fallout of retained material while still allowing adequate drainage between vertically adjacent lagging boards. The drainage system used should be adequate to prevent the buildup of hydrostatic forces against the retaining wall.

Weak sedimentary rock may be encountered during installation of the soldier piles, which is designated as the Monterey Formation (see Section 5.0). Backfill used in the retained material and at the base of the wall overlaying the foundation material should conform to the earthwork requirements specified in the GDR (MJA, 2023a). Overhead utilities are also present at the site on the north side of SR 218 above the road shoulder.

Because of the inability to obtain subsurface geotechnical information, the assessment for liquefaction-induced lateral spreading potential was performed by conservatively assuming that the liquefiable layer encountered in the project borings and CPTs extend laterally under the retaining wall and under the existing slope behind private properties. Additional field explorations (i.e., borings or CPTs) should be performed prior to construction to confirm or refute the reasonableness of this conservative assumption and further inform the liquefaction-induced lateral spreading potential.

6.4 Notes for Specifications

Difficult pile installation is not expected based on our current investigation data. Subsurface conditions on along the retaining wall used for soldier pile design were projected from the data obtained from our investigations, and there is a potential for encountering Monterey Formation at a shallower depth along the retaining wall bridge. According to Monterey Formation interpreted to have been encountered in our project boring B-5b, it is classified as very stiff soil or soft rock hardness, which is not anticipated to result in difficult driving conditions. However, project boring B-5b was terminated at Elev. +55.5 feet, and because ground conditions from the CPTs and boring B-4 on the south side of SR 218 were extrapolated north across highway and along the retaining wall, the contractor should perform confirming investigations along the retaining wall to confirm the in-situ ground conditions.

6.5 Notes for Construction

We recommend that a geotechnical engineer perform inspections and testing during the following stages of construction:

- Placement of compacted backfill
- Installation of soldier piles
- When any unusual subsurface conditions are encountered

7.0 Limitations

This report has been prepared for the exclusive use of GHD, and TAMC for the planned Retaining Wall No. 1 of the Canyon Del Rey/SR218 Segment of Fort Ord Regional Trail & Greenway (FORTAG) project in Del Rey Oaks California, as described herein. Project details referred to herein are from information provided in the FORTAG Undercrossing at SR 218 drawings prepared by Cornerstone Structural Engineering Group (2023) and 95% submittal drawings prepared by GHD (2022). We understand that there will be a planned Retaining Wall No. 2 for the project; however, performing a geotechnical investigation and providing related design recommendations for Retaining Wall No. 2 are not part of our scope of work.

Subsurface conditions at and between locations of subsurface exploration for the project (borings and CPTs) may vary over time from those encountered and logged in the explorations as provided herein (see Appendices B and D). Subsurface conditions along the retaining wall and under the existing slope were projected from data obtained from project borings and CPTs. If the ground conditions that are exposed during construction differ from those indicated in logs of project explorations as provided herein, then McMillen Jacobs Associates is to be retained to evaluate the exposed ground conditions and to provide written confirmation or modifications to the recommendations provided in this report. Studies of the absence, existence, and effects of artificial contamination (e.g., from leaking underground storage tanks) and natural environmental conditions (e.g., from naturally occurring asbestos) on project construction, if any, are outside of our expertise and are not part of our scope of services. Any reference in this report to related data is solely provided as a value-added service. Additionally, the corrosion recommendation provided herein is from limited data, and therefore, a soil corrosion engineer should be retained to evaluate soil corrosivity relative to design of the project.

The geotechnical recommendations provided in this report have been formulated in a manner consistent with the level of care and skill ordinarily exercised by members of the geotechnical profession currently practicing in the area under similar project constraints for this type of project.

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>
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>
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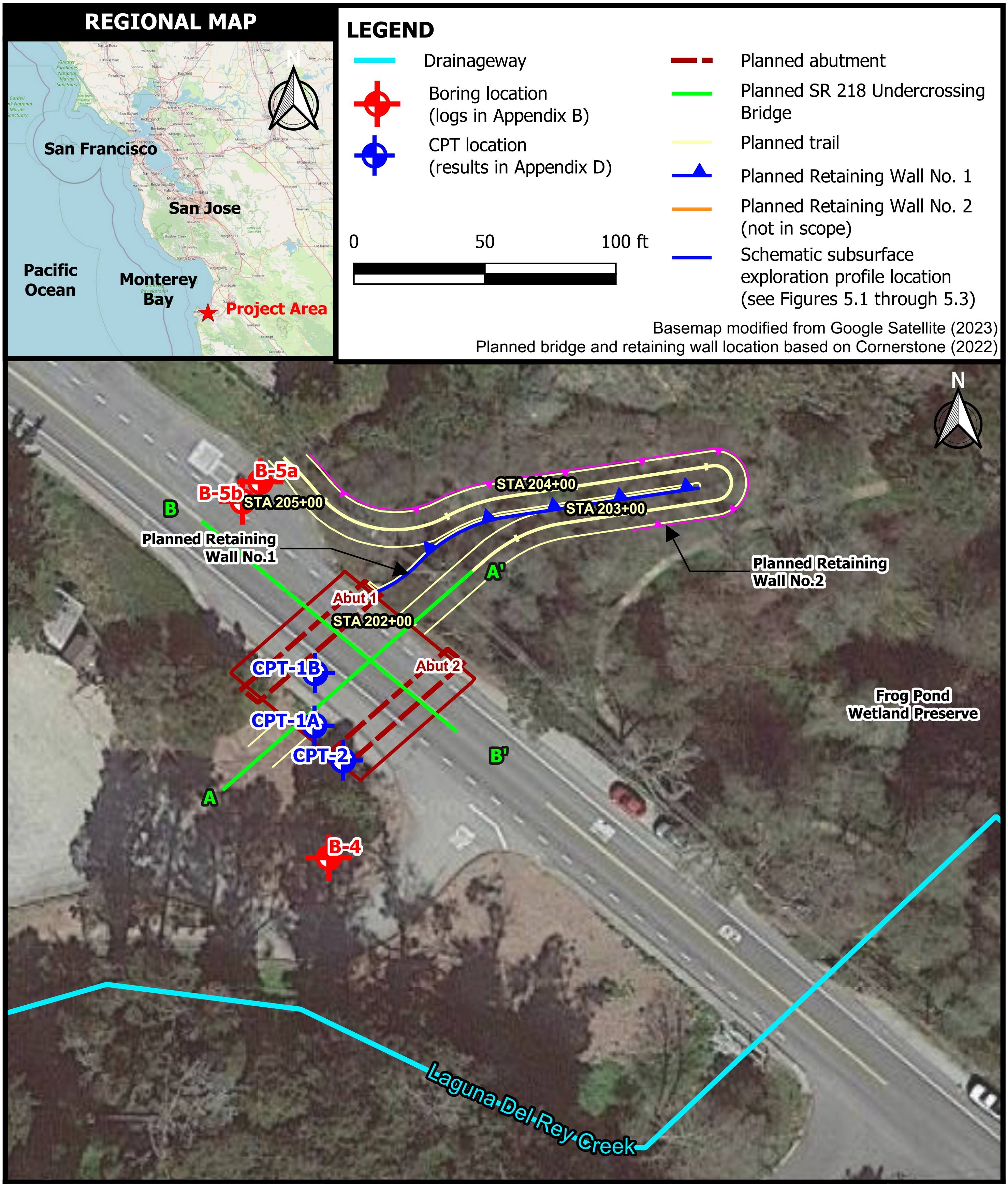
Figures



McMILLEN JACOBS ASSOCIATES

File No. 6231.0

April 2023

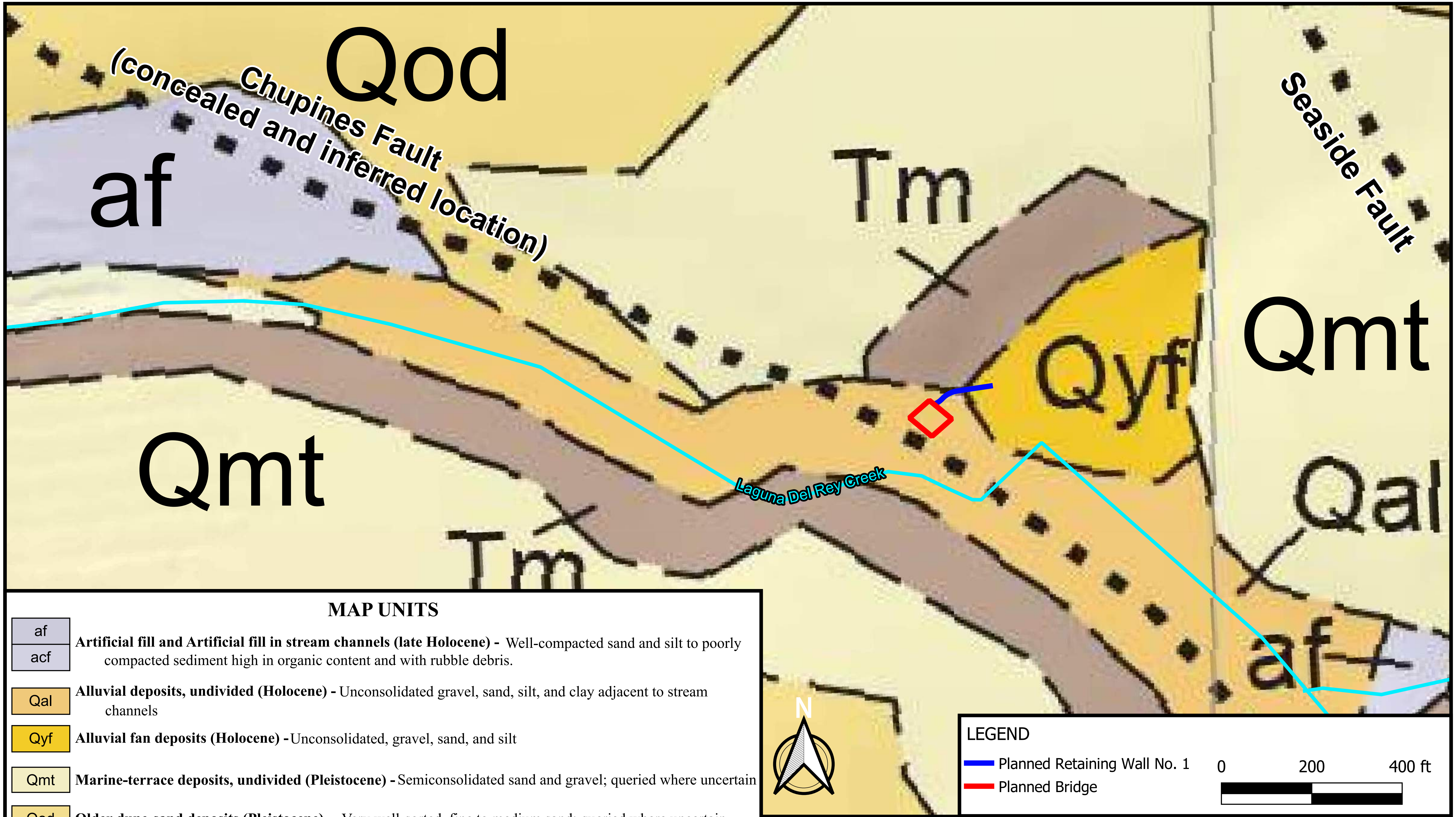
GHD

Transportation Agency for Monterey County
FORTAG - Canyon Del Rey/SR218 Segment
Del Rey Oaks, California

Project Area Map

Figure

1



MAP UNITS

af acf	**Artificial fill and Artificial fill in stream channels (late Holocene) -** Well-compacted sand and silt to poorly compacted sediment high in organic content and with rubble debris.
Qal	**Alluvial deposits, undivided (Holocene) -** Unconsolidated gravel, sand, silt, and clay adjacent to stream channels
Qyf	**Alluvial fan deposits (Holocene) -** Unconsolidated, gravel, sand, and silt
Qmt	**Marine-terrace deposits, undivided (Pleistocene) -** Semiconsolidated sand and gravel; queried where uncertain
Qod	**Older dune-sand deposits (Pleistocene) -** Very well-sorted, fine to medium sand; queried where uncertain
Tm	**Monterey Formation (Miocene) -** Pale-orange to white, porcelaneous shale interbedded with chert, mudstone, calcareous claystone, siltstone, and sandstone; some diatomaceous and tuffaceous/bentonitic volcanic ash

Map and descriptions modified from Hartwell et al., (2016)

LEGEND

🟦	Planned Retaining Wall No. 1
🟥	Planned Bridge

0 200 400 ft

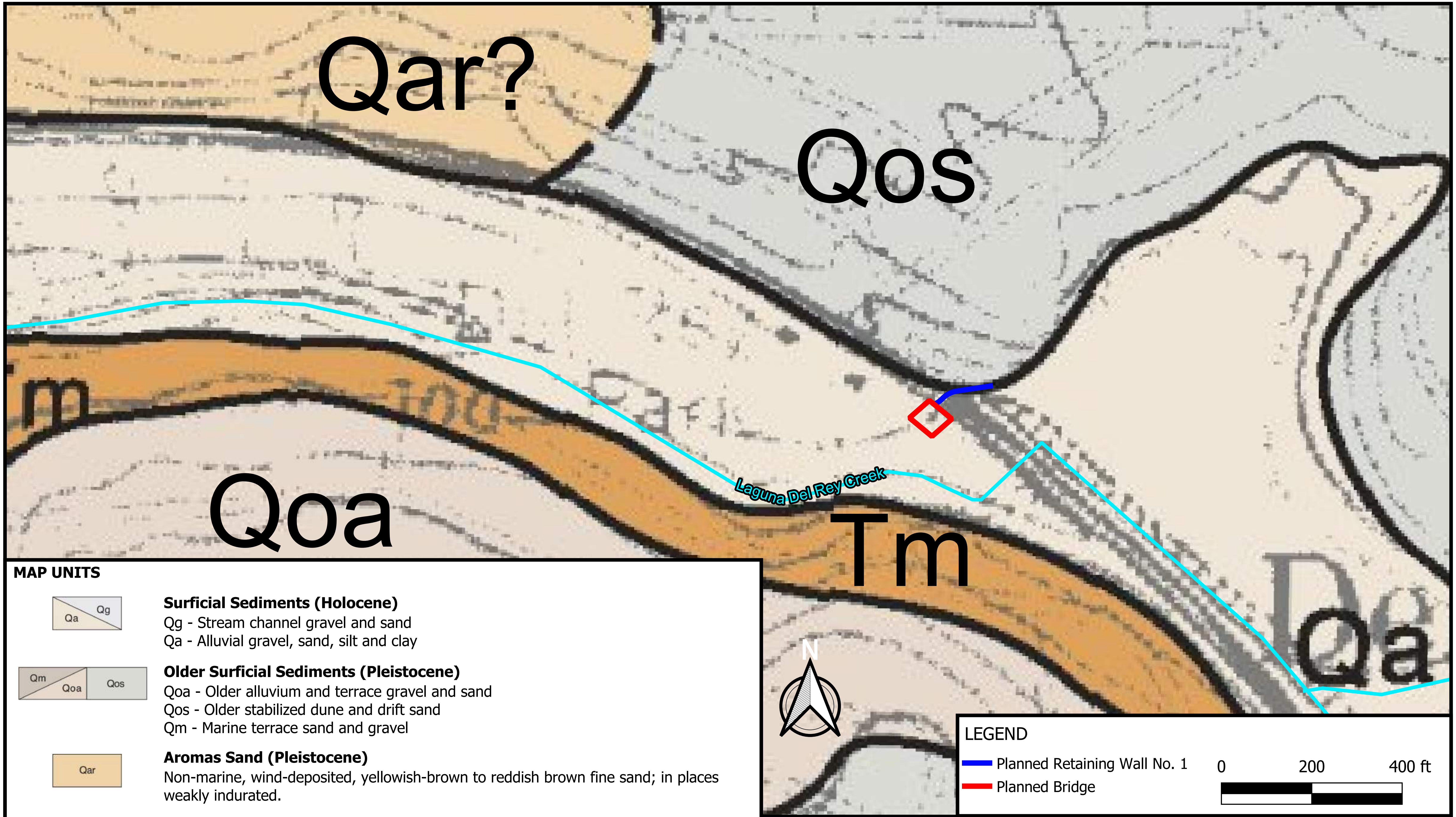
McMILLEN JACOBS ASSOCIATES

File No. 6231.0 April 2023

GHD
Transportation Agency for Monterey County
FORTAG - Canyon Del Rey/SR218 Segment
Del Rey Oaks, California

Geologic Map

Figure
2.1



MAP UNITS

Qa / Qg	**Surficial Sediments (Holocene)** Qg - Stream channel gravel and sand Qa - Alluvial gravel, sand, silt and clay
Qm / Qoa / Qos	**Older Surficial Sediments (Pleistocene)** Qoa - Older alluvium and terrace gravel and sand Qos - Older stabilized dune and drift sand Qm - Marine terrace sand and gravel
Qar	**Aromas Sand (Pleistocene)** Non-marine, wind-deposited, yellowish-brown to reddish brown fine sand; in places weakly indurated.
Tm	**Monterey Formation (middle to late Miocene)** Marine biogenic and clastic deposits; including white-weathering siliceous shale, and white, soft and commonly silty diatomite and bentonite (volcanic ash fall tuff).

Descriptions modified from Dibblee et al., (1974) and Dibblee and Minch (2007)
Map modified from Dibblee and Minch (2007)

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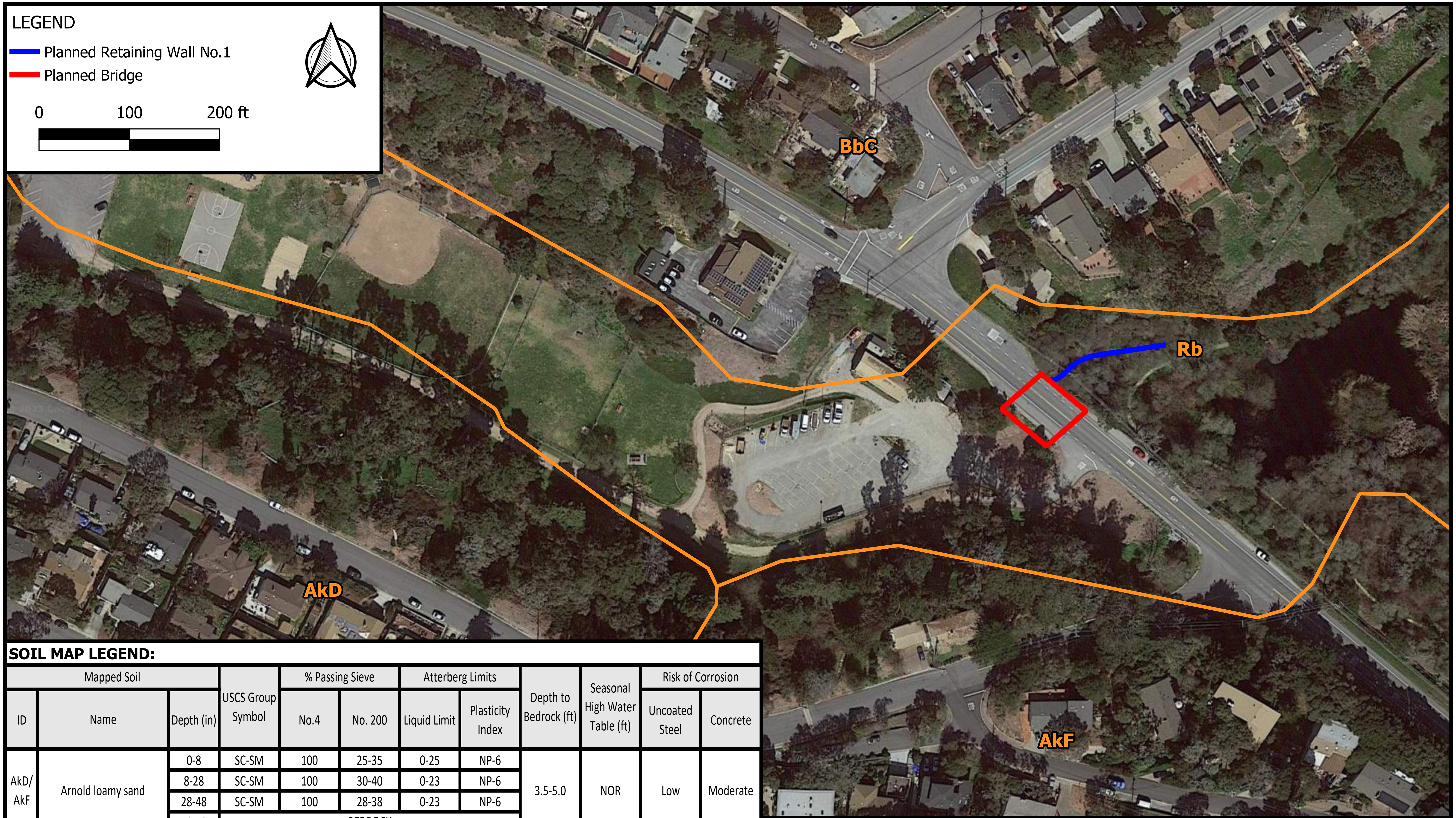
Geologic Map

Figure
2.2



SOIL MAP LEGEND:

Mapped Soil			USCS Group Symbol	% Passing Sieve		Atterberg Limits		Depth to Bedrock (ft)	Seasonal High Water Table (ft)	Risk of Corrosion	
ID	Name	Depth (in)		No.4	No. 200	Liquid Limit	Plasticity Index			Uncoated Steel	Concrete
AkD/ AkF	Arnold loamy sand	0-8	SC-SM	100	25-35	0-25	NP-6	3.5-5.0	NOR	Low	Moderate
		8-28	SC-SM	100	30-40	0-23	NP-6				
		28-48	SC-SM	100	28-38	0-23	NP-6				
		48-79	BEDROCK								
BbC	Baywood sand	0-60	SM, SP-SM	100	5-30	0-27	NP	NOR	NOR	Moderate	Moderate
Rb	Rindge muck	0-60	PT	-	-	-	-	NOR	0-6.0	High	Moderate

NP - Nonplastic
NOR - None Reported

Descriptions modified from U.S. Soil/Natural Resources Conservation Service (2020)

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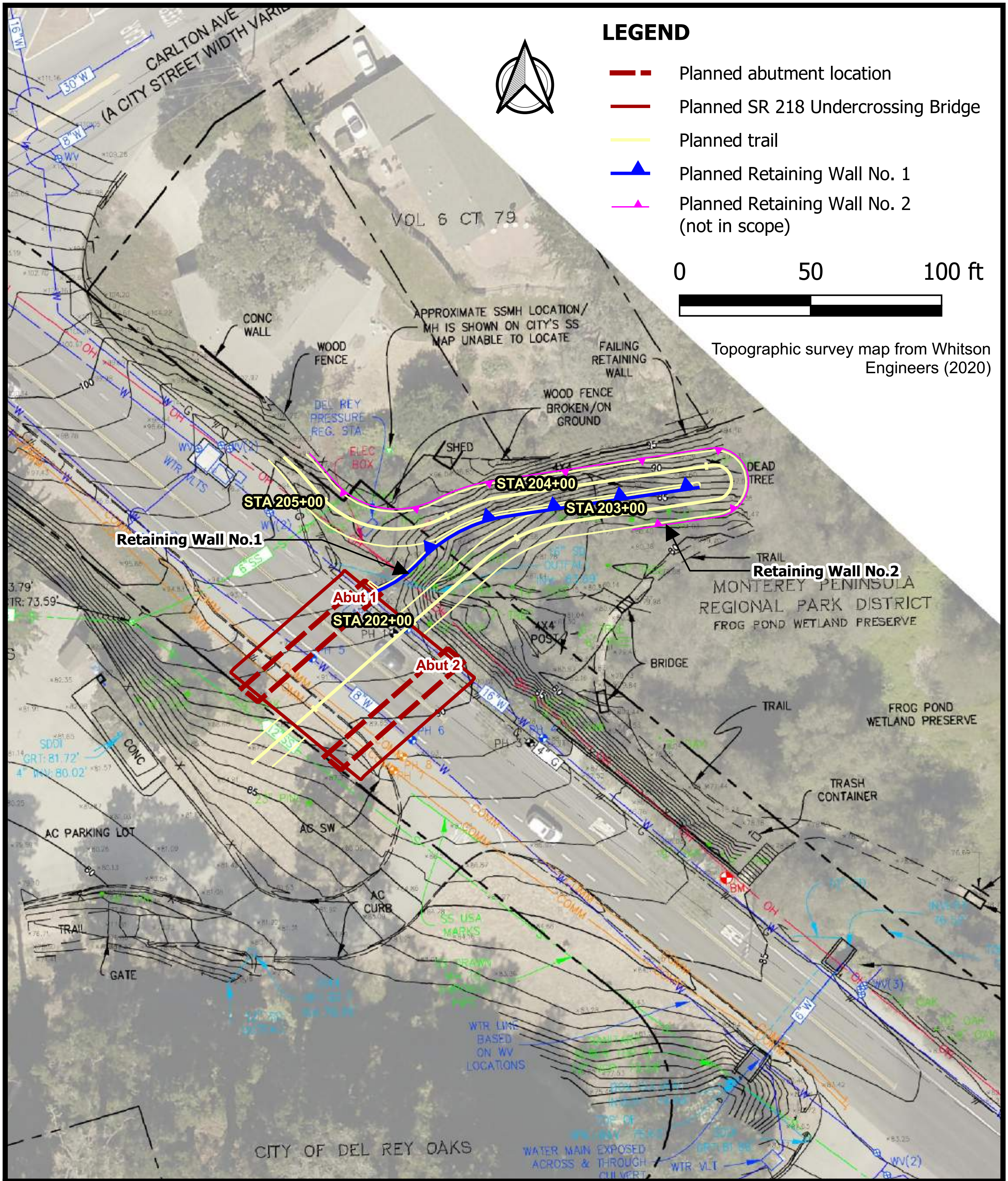
Soil Map

Figure
3



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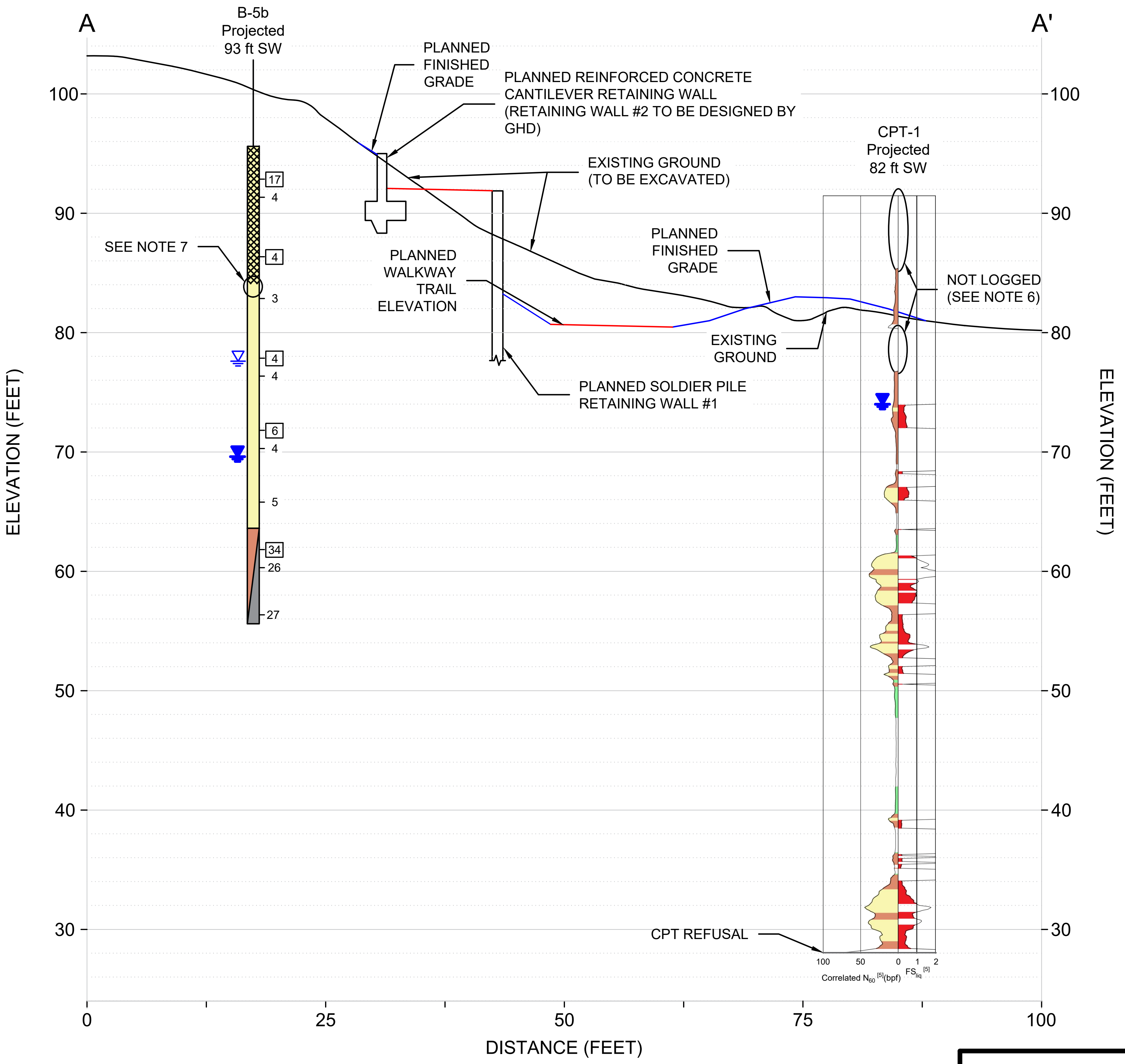
Topographic Survey Map

Figure

4



LEGEND:

B-1/ CPT-1 - Projected graphic boring and CPT logs (See plan locations in Figure 1 and logs in Appendices B and D)

▨ - Artificial fill

🟩 - Predominantly clay soil behavior type

🟧 - Predominantly silt soil behavior type

🟨 - Predominantly sand soil behavior type

⬛ - Weathered bedrock (MONTEREY FORMATION?)

▦ - Peat

☐ - Sensitive, fine grained soil behavior type

13– - Standard Penetration Test (SPT) sampler resistance "N-Value" blows per foot (bpf) count (ASTM D1586), see Appendices A and B for legend and logs

29– - Modified California sampler penetration resistance blows per foot (bpf) count, see Appendices A and B for legend and logs

▼ - Depth of free groundwater measured in boring after drilling or inferred by pore pressure dissipation test in CPT

▽ - Depth of free groundwater first noted seeping into boring during drilling

NOTES:

1. Location of borings, CPTs, and schematic profile are shown in Figure 1.
2. Profile details (e.g. distances and elevations) are based on FORTAG Undercrossing at SR 218 Bridge General Plan from Cornerstone (2022), and Improvement Plan and Profile from GHD (2022).
3. Ground types and groundwater conditions shown are projected from boring and CPT locations. These types and conditions may differ away from boring and CPT locations, including from stratal undulations, lensing, lateral facies changes, and effects of paleotopography.
4. Width and placement of graphic boring and CPT logs is exaggerated and approximated for clarity.
5. N_{60} is the SPT N-value, corrected for field procedures and apparatus, and correlated from the CPT results based on Robertson (2009). FS_{liq} is Factor of Safety against liquefaction; the red shaded zone represents $FS_{liq}<1$.
6. Hand augered to 6'; obstruction encountered from 11' to 15'; punched through the obstruction and advanced to refusal.
7. Concrete between 10.5' - 12' bgs.

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Schematic Subsurface Exploration Profile A-A'

Figure

5

LEGEND:

- ━ Planned SR 218 Undercrossing Bridge
- ━ Planned Retaining Wall No.1
- ━ Drainageway

Mean Vs30

- 🟧 1150 ft/sec to 1200 ft/sec
- 🟥 1250 ft/sec to 1300 ft/sec

Map modified from Branum et al., (2016)

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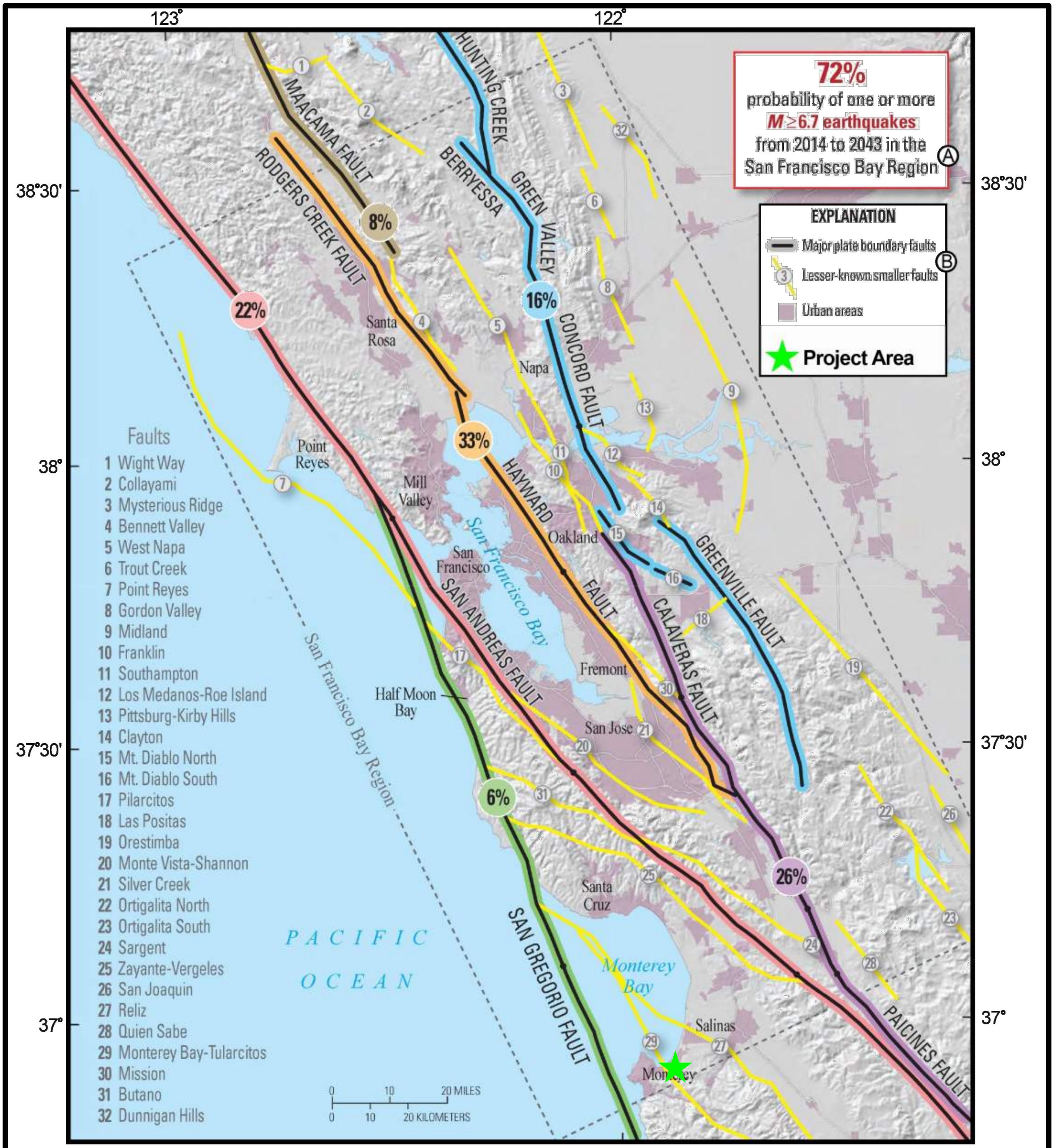
Vs30 Map

Figure

6



Map modified from USGS Fact Sheet 2016-3020

Ⓐ On major plate boundary faults, lesser-known faults, and unknown faults.
Ⓑ The probability that a M > 6.7 earthquake will involve one of the lesser known faults is 13%.

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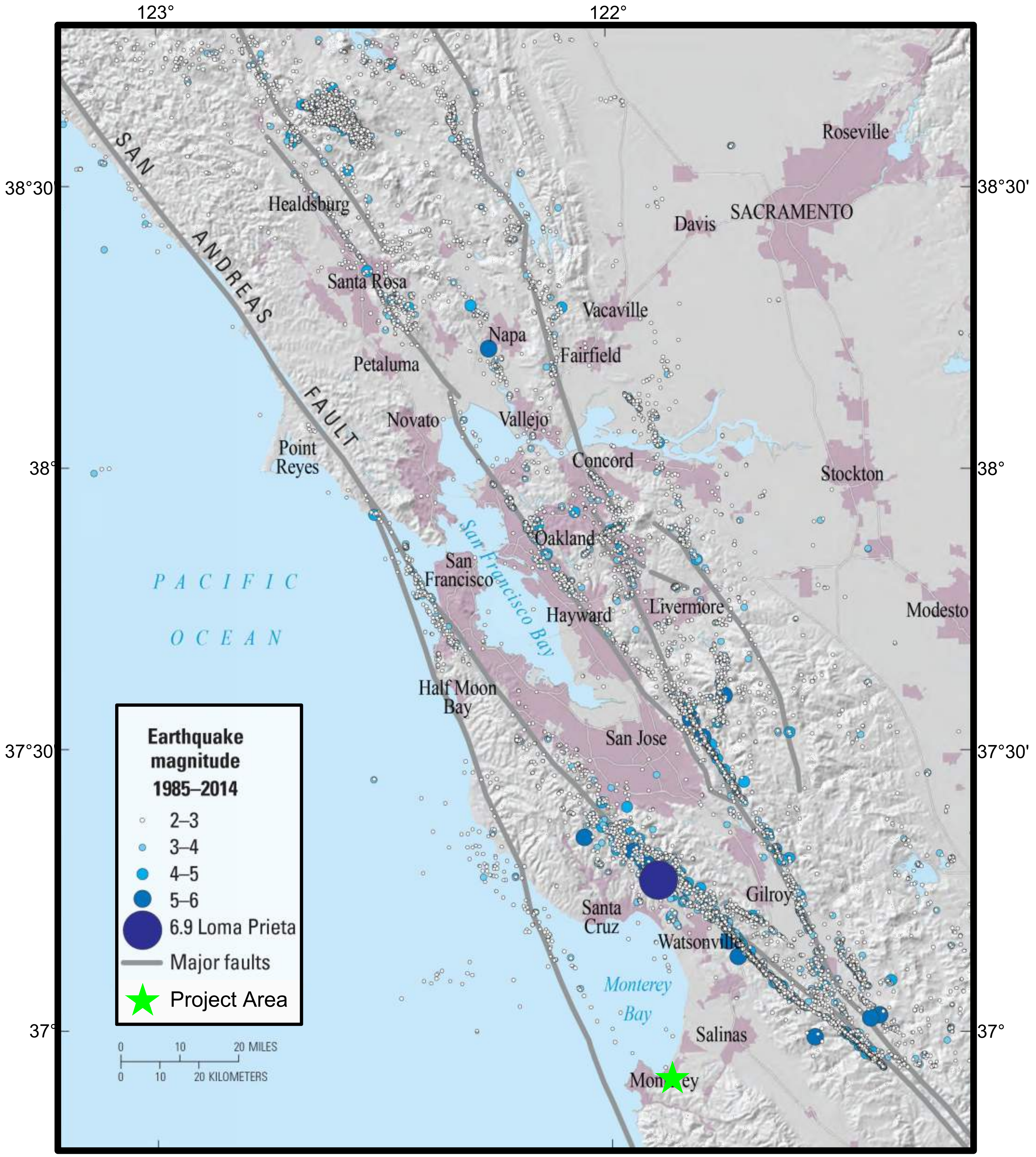
Fault Map

Figure

7



Map modified from USGS Fact Sheet 2016-3020

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Earthquake Map

Figure

8

LEGEND

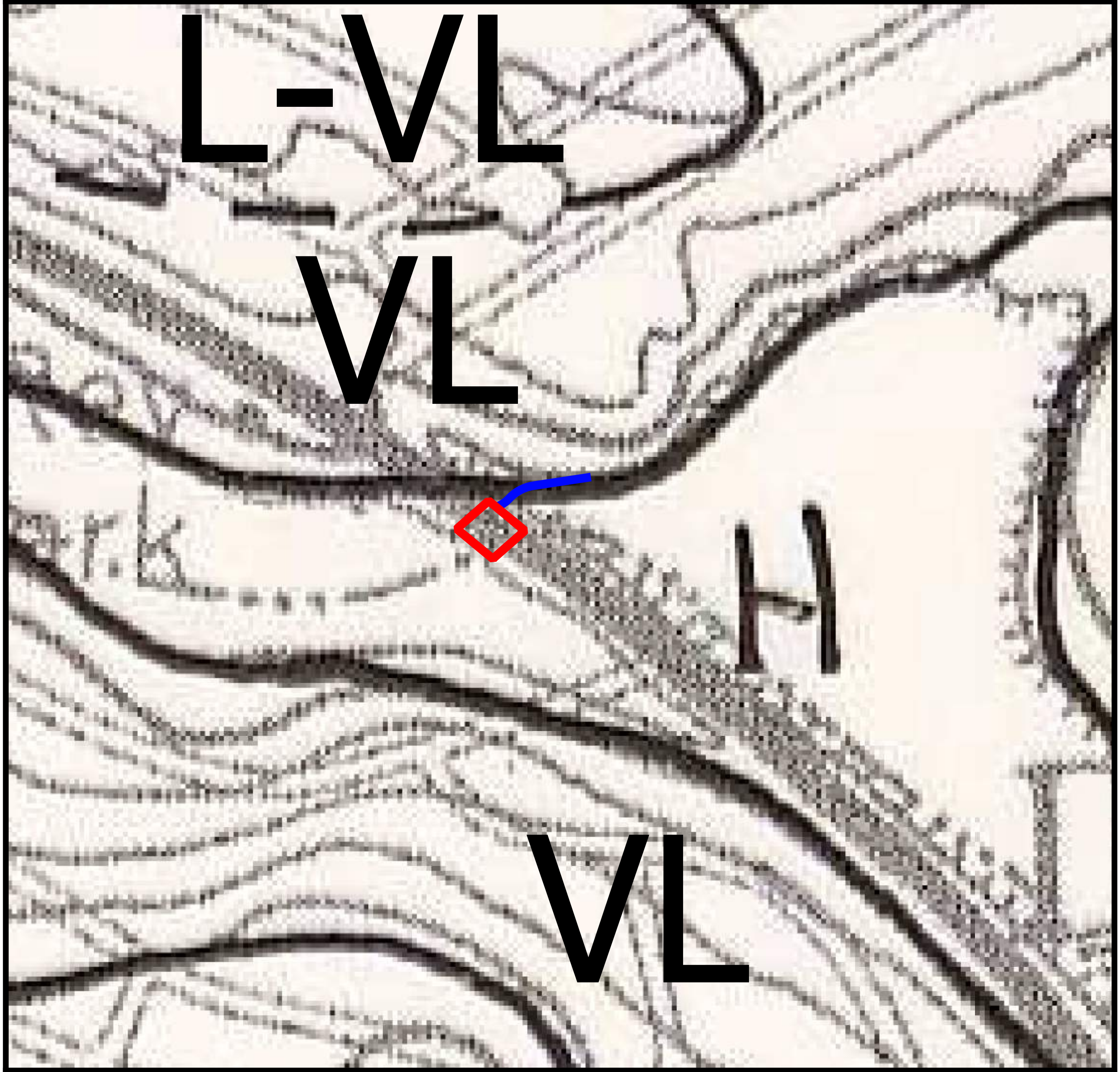
▬ Planned Retaining Wall No.1
▬ Planned Bridge

LIQUEFACTION SUSCEPTIBILITY

VL - Very Low
L - Low
M - Medium
H - High
VH - Very High
Vb - Variable

0 200 400 ft

Map modified from Dupre (1990)



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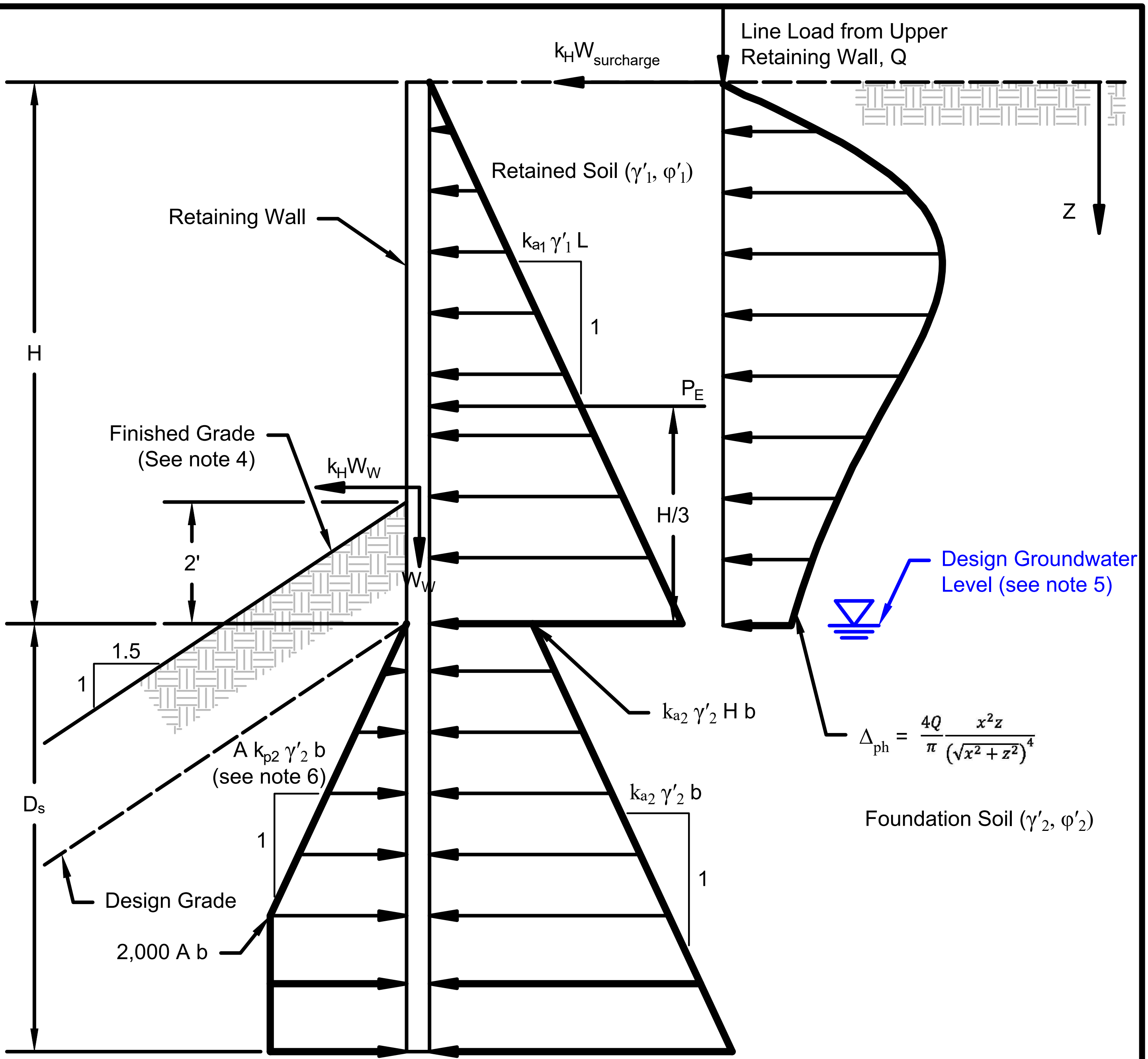
Liquefaction Susceptibility Map

Figure 9



NOTES:

1. See section 4.1 of the report for material properties (γ, φ', k_a, k_p). $\gamma' = \gamma$ above design groundwater level and $\gamma' = \gamma - 62.4$ below design groundwater level.
2. L is the center to center span between soldier piles, b is the diameter of the concrete backfilled soldier pile embedded in the foundation material, D_s is the thickness of foundation sand, D_r is the foundation rock embedment.
3. Earth pressure and surcharge distributions are per AASHTO (2017).
4. Finished grade is sloped from the retaining wall from the bridge abutment to 90 feet upstation. The remainder of finished grade is approximately horizontal at the retaining wall.
5. Design groundwater level is at +74 feet elevation based on CPTs data.
6. Arching in passive resistance will occur over 3 times the soldier pile diameter. "A" is taken as the ratio of the soldier pile center to center spacing over the soldier pile diameter, with a maximum value of 3.

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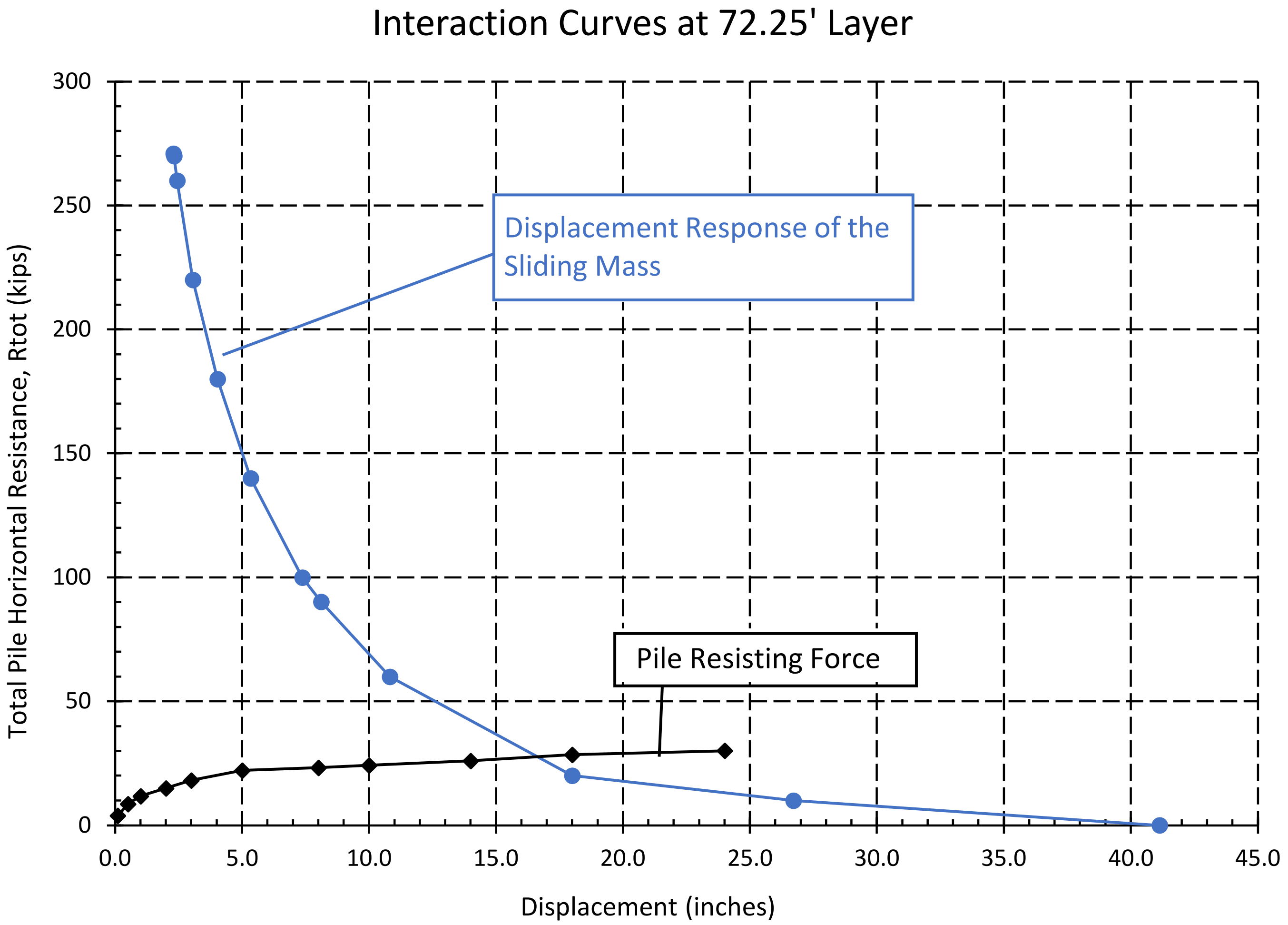
Lateral Earth Pressures for Retaining Wall No. 1

Figure

10



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Interaction Curves

Figure

11

Appendix A

LEGEND FOR BORING LOGS IN APPENDIX B

☒	Grab sample
■	2.5" I.D./3" O.D. Modified California Sampler (ASTM D3550) with steel liners (MCS)
◧	2" I.D./2.5" O.D. Split spoon sampler (SSS) (ASTM D1586)
◤	1.4" I.D./2" O.D. Standard Penetration Test (ASTM D1586) sampler (SPT)
▽	Depth of free groundwater seepage first noted into boring during drilling
▼	Depth of free groundwater level measured in boring after drilling

RELATIVE DENSITY		CONSISTENCY		
SANDS AND GRAVELS	SPT, N	SILTS AND CLAYS	SPT, N	UNCONFINED COMPRESSIVE STRENGTH, tsf
VERY LOOSE	0-4	VERY SOFT	0-2	0-0.25
LOOSE	4-10	SOFT	2-4	0.25-0.50
MEDIUM DENSE	10-30	MEDIUM STIFF	4-8	0.50-1.00
DENSE	30-50	STIFF	8-15	1.00-2.00
VERY DENSE	50+	VERY STIFF	15-30	2.00-4.00
		HARD	30+	>4.00

Reference: Terzaghi, K. and Peck, R., SOIL MECHANICS IN ENGINEERING PRACTICE, 2nd ed., John Wiley and Sons, New York, 1967. Page 341 Table 45.1 and pp. 347 Table 45.2.

CONSTITUENT DESCRIPTIONS

DESCRIPTION	CRITERIA
TRACE	less than 5%
FEW	5% to 10%
LITTLE	15% to 25%
SOME	30% to 45%
MOSTLY	50% to 100%

Reference: ASTM D2488, Note 15

MOISTURE CONDITION

DESCRIPTION	CRITERIA
DRY	Absence of moisture, dusty, dry to the touch
MOIST	Damp but no visible water
WET	Visible free water, usually soil is below water table

Reference: ASTM D2488, Table 3 - Criteria for Describing Moisture Condition

GROUND BEHAVIOR	CLASSIFICATION
Ground that can be excavated without initial support to shallow depths (typically less than 10 feet) and where shoring can be installed before the ground starts to move. For example, unfissured hard clay when not highly overstressed.	Firm
Ground of which chunks or flakes begin to fall off excavation walls. If raveling starts within a few minutes of excavation then it is "fast" raveling; otherwise, it is "slow" raveling. Silts and sands with clay binder may be fast raveling. Stiff fissured clays may be slow or fast raveling depending upon the degree of overstress.	Raveling
Ground that squeezes or plastically extrudes into excavations without visible fracturing. Can occur at shallow to medium depth in very soft to medium stiff clay, and can occur in stiff to hard clay under high overstress.	Squeezing
Ground consisting of clean dry granular material (e.g., sand and gravel) that moves by gravity to its angle of repose.	Running
Ground in a fluid-like condition (e.g., a disturbed mixture of predominantly silt, sand and/or gravel with water), that flows across pressure gradients.	Flowing
Ground that expands in volume due to the absorption of water (e.g., clays).	Swelling

Reference: Modified from Heuer, R.E., 1974, Important ground parameters in soft ground tunneling, Subsurface exploration for underground excavation and heavy construction, New England College, Henniker, New Hampshire, American Society of Civil Engineers, New York, P. 41-55.

NOTES:

1. Project borings were made with a SIMCO 2400 SK-1 Longstroke drill rig uisng 7-inch diameter continuous hollow stem augers as indicated on the respective log. Lines separating strata in the logs represent approximate boundaries and are dashed where strata change depth is less certain. Strata change may be gradual across the boundary lines logged. Logged groundwater depths are subject to limitations described in the text of the report.

2. Penetration Resistance (blows/ft.) are the last 12 inches of an 18-inch drive using a 140-pound cathead sampling hammer falling 30 inches per blow unless noted otherwise. The Penetration Resistance values noted on the logs are actual blows per foot of penetration for the respective sampler type (e.g., MCS sampler penetration resistance blow counts have not been reduced to SPT sampler "N" values).

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Boring Log Legend

Figure

A-1

(1 of 2)

LEGEND FOR BORING LOGS IN APPENDIX B (Cont'd)

CRITERIA FOR ASSIGNING GROUP SYMBOLS AND GROUP NAMES [A]				GROUP SYMBOL	GROUP NAME [B]
COARSE-GRAINED SOILS More than 50% retained on No. 200 sieve	**GRAVELS** More than 50% of coarse fraction retained on No. 4 sieve	**Clean Gravels** < 5% fines [C]	Cu ≥ 4 and 1 ≤ Cc ≤ 3 [E]	**GW**	Well-graded gravel [F]
			Cu < 4 and/or 1 > Cc > 3 [E]	**GP**	Poorly graded gravel [F]
		Gravels with Fines > 12% fines [C]	Fines classify as ML or MH	**GM**	Silty gravel [F,G,H]
			Fines classify as CL or CH	**GC**	Clayey gravel [F,G,H]
	SANDS 50% or more of coarse fraction passes No. 4 sieve	**Clean Sands** < 5% fines [D]	Cu ≥ 6 and 1 ≤ Cc ≤ 3 [E]	**SW**	Well-graded sand [I]
			Cu < 6 and/or 1 > Cc > 3 [E]	**SP**	Poorly graded sand [I]
		Sands with Fines > 12% fines [D]	Fines classify as ML or MH	**SM**	Silty sand [G,H,I]
			Fines classify as CL or CH	**SC**	Clayey sand [G,H,I]
FINE-GRAINED SOILS 50% or more passes the No. 200 sieve	**SILTS AND CLAYS** Liquid limit ≤ 50	**Inorganic**	PI > 7 plots on or above "A" line [J]	**CL**	Lean clay [K,L,M]
			PI < 4 plots below "A" line [J]	**ML**	Silt [K,L,M]
		Organic	$\frac{\text{Liquid limit-oven dried}}{\text{Liquid limit-not dried}} < 0.75$	**OL**	Organic Clay [K,L,M,N]
					Organic Silt [K,L,M,O]
	SILTS AND CLAYS Liquid limit > 50	**Inorganic**	PI plots on or above "A" line	**CH**	Fat clay [K,L,M]
			PI plots below "A" line	**MH**	Elastic silt [K,L,M]
		Organic	$\frac{\text{Liquid limit-oven dried}}{\text{Liquid limit-not dried}} < 0.75$	**OH**	Organic Clay [K,L,M,P]
					Organic Silt [K,L,M,Q]
HIGHLY ORGANIC SOILS		**Primarily organic matter, dark color and organic odor**		**PT**	Peat

NOTES:

A Based on the material passing the 3-inch (75mm) sieve.

B If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.*

C Gravels with 5% to 12% fines require dual symbols:
- GW-GM well-graded gravel with silt
- GW-GC well-graded gravel with clay
- GP-GM poorly graded gravel with silt
- GP-GC poorly graded gravel with clay

D Sands with 5% to 12% fines require dual symbols:
- SW-SM well-graded sand with silt
- SW-SC well-graded sand with clay
- SP-SM poorly graded sand with silt
- SP-SC poorly graded sand with clay

E $Cu = \frac{D_{60}}{D_{10}} \qquad Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$

F If soil contains ≥15% sand, add "with sand" to group name.

G If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.

H If fines are organic, add "with organic fines" to group name.

I If soil contains ≥ 15% gravel, add "with gravel" to group name.

J If Atterberg limits plot in hatched area, soil is a CL-ML (silty clay).

K If soil contains 15% to 29% plus No. 200,add "with sand" or "with gravel", whichever is predominant.

L If soil contains ≥ 30% plus No.200, predominantly sand, add "sandy" to group name.

M If soil contains ≥ 30% plus No.200, predominantly gravel, add "gravelly" to group name.

N PI ≥ 4 and plots on or above "A" line.

O PI < 4 or plots below "A" line.

P PI plots on or above "A" line.

Q PI plots below "A" line.

PLASTICITY

Term	PI	Dry Strength	Field Test
Nonplastic	0-3	Very low	Falls apart easily
Slightly plastic	3-15	Slight	Easily crushed with fingers
Medium plastic	15-30	Medium	Difficult to crush
Highly plastic	30 or more	High	Impossible to crush with fingers

Reference: Sowers, George F., Introductory Soil Mechanics and Foundations: Geotechnical Engineering, 4th ed., Macmillan Publishing Co., Inc., New York. 1979, Page 83 Table 2:10.

GRAIN SIZE

Group	Texture	Sieve	Dimension, mm
Boulder	-	> 12"	> 305
Cobble	-	3"	75
Gravel	Coarse	3/4"	19
	Fine	No. 4	4.75
Sand	Coarse	No. 10	2.00
	Medium	No. 40	0.425
	Fine	No. 200	0.075
Fines	Silt	< No. 200	0.002
	Clay		< 0.002

Reference: modified from ASTM D2487

**The largest particle that could have been retrieved from a boring is a function of the diameter of the boring, drill bit, and sampler. Intact cobble- and boulder-size particles, if any, are too large to retrieve from small diameter borings performed for the project. Therefore, there may have been larger particles (e.g., cobble- and boulder-size) in the borings than were retrieved in samples, observed in drill cuttings and consequently logged in borings.*

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Boring Log Legend

Figure

A-1

(2 of 2)

WEATHERING CRITERIA

FRESH - Rock fresh, crystals bright, few joints show slight staining. Rock rings under hammer if crystalline.

VERY SLIGHT - Rock generally fresh, joints stained, some joints may show thin clay coatings, crystals in broken face show bright. Rock rings under hammer if crystalline.

SLIGHT - Rock generally fresh, joints stained, and discoloration extends into rock up to 1 inch. Joints may contain clay. In granitoid rocks some occasional feldspar crystals are dull and discolored. Crystalline rocks ring under hammer.

MODERATE - Significant portions of rock show discoloration and weathering effects. In granitoid rocks, most feldspars are dull and discolored; some show clayey. Rock has dull sound under hammer and shows significant loss of strength as compared with fresh rock.

MODERATELY SEVERE - All rocks except quartz discolored or stained. In granitoid rocks, all feldspars dull and discolored and majority show kaolinization. Rock shows severe loss of strength and can be excavated with geologist's pick. Rock goes "clunk" when struck.

SEVERE - All rocks except quartz discolored or stained. Rock "fabric" clear and evident, but reduced in strength to strong soil. In granitoid rocks, all feldspars kaolinized to some extent. Some fragments of strong rock usually left.

VERY SEVERE - All rock except quartz discolored or stained. Rock "fabric" discernible, but mass effectively reduced to "soil" with only fragments of strong rock remaining.

COMPLETE - Rock reduced to "soil". Rock "fabric" not discernible or discernable only in small scattered locations. Quartz may be present as dikes or stringers.

HARDNESS

VERY HARD - Cannot be scratched with knife or sharp pick. Breaking of hand specimens requires several hard blows of geologist's pick.

HARD - Can be scratched with knife or pick only with difficulty. Hard blow of hammer required to detach hand specimen.

MODERATELY HARD - Can be scratched with knife or pick. Gouges or grooves to ¼ inch deep can be excavated by hard blow of point of a geologist's pick. Hand specimens can be detached by moderate blow.

MEDIUM - Can be grooved or gouged 1/16 inch deep by firm pressure on knife or pick point. Can be excavated in small chips to pieces about 1-inch maximum size by hard blows of the point of a geologist's pick.

SOFT - Can be gouged or grooved readily with knife or pick point. Can be excavated in chips to pieces several inches in size by moderate blows of a pick point. Small thin pieces can be broken by finger pressure.

VERY SOFT - Can be carved with a knife. Can be excavated readily with point of pick. Pieces 1-inch or more in thickness can be broken with finger pressure. Can be scratched readily by fingernail.

Reference: Subsurface Investigation for Design and Construction of Foundations of Buildings, ASCE-Manuals and Reports on Engineering Practice-No. 56, 1976, by American Society of Civil Engineers.

STRENGTH

PLASTIC - moldable

FRIABLE - crumbles easily by rubbing with fingers

WEAK - an unfractured specimen of such material will crumble under light hammer blows

MODERATELY STRONG - specimen will withstand a few heavy hammer blows before breaking

STRONG - specimen will withstand a few heavy ringing hammer blows but will yield larger fragments with difficulty

VERY STRONG - specimen will resist heavy ringing hammer blows and will yield only dust and small flying fragments with difficulty

ANGLE FROM HORIZONTAL	DESCRIPTION
0-5°	horizontal
5-35°	shallow
35-55°	moderate
55-85°	steep
85-90°	vertical

DISCONTINUITIES

SPACING	FRACTURING	BEDDING
Less then ½ inch	crushed	laminated
½ inch to 2 inches	very close	very thin
2 inches to 1 foot	close	thin
1 foot to 3 feet	moderately close	medium
3 feet to 10 feet	wide	thick
More than 10 feet	very wide	very thick

APERTURE

STRUCTURE	DESCRIPTION
tight	no visible separation
open	amount of separation, staining or coatings on fracture surfaces, and fracture surface moisture conditions may be noted
healed	degree of healing, (i.e., partial or complete), thickness and mineralogy/hardness may be noted
filled	degree of filling, (i.e. partial or complete), thickness and type of filling may be noted

ROUGHNESS

SURFACE	DESCRIPTION
stepped	near normal steps and ridges occur on fracture surface
rough	large, angular asperities can be seen
moderately rough	asperities are clearly visible and fracture surface feels abrasive
slightly rough	small asperities on the fracture surface visible and can be felt
smooth	no asperities, smooth to touch
polished	extreamly smooth and shiny

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Bedrock Descriptors

Figure

A-2

Appendix B

LOG OF BORING B-4①

LOCATION: see Figure 1

DEPTH feet	SAMPLE NO.	TYPE	PENETRATION RESISTANCE blows/ft.	③GROUNDWATER	DESCRIPTION②	MOISTURE %	DRY DENSITY lbs./ft.³	LIQUID LIMIT	PLASTICITY INDEX	GRAIN SIZE Gravel (>#4 sieve) %	GRAIN SIZE Sand (#4 to #200 sieve) %	GRAIN SIZE Fines (<#200 sieve) %	UNCONFINED COMPRESSIVE STRENGTH kips/ft.²	DIRECT SHEAR Cohesion p.s.f.	DIRECT SHEAR Internal Friction Angle
	1	☒			**SILTY CLAYEY SAND (SM/SC) - FILL** - dark to very dark brown - few gravel - nonplastic - concrete at 2' to 3' - loose - dry to moist					6	80	14	**FINES** 7% Silt 7% Clay		
	2	■	9			8	103								
5	3	◣	7												
10	4	■	6	▽	**SILTY CLAYEY SAND (SM/SC)** - dark gray to black - trace gravel - slightly to medium plastic - loose - moist to wet **CORROSION TEST** Sample B-4-5 See Appendix C										
	5	◣	5							<1	50	50	**FINES** 31% Silt 19% Clay		
15	6	■	15	▽	**SILT WITH SAND (ML)** - dark gray - trace gravel - few clay - slightly plastic - soft to medium stiff - wet	43	74							270	21°
	7	◣	4					46	12						
20					**SILTY CLAYEY SAND (SM/SC)** - dark gray - slightly plastic - loose to medium dense - wet										
	8	■	26			26	98								
	9	◣	9		**LEAN TO FAT CLAY (CL/CH)** - dark gray - trace to little sand - medium to highly plastic - stiff - wet						67	33	**FINES** 25% Silt 8% Clay		
25					**POORLY GRADED SAND WITH SILT (SP-SM)** - dark gray - trace clay and gravel - nonplastic - medium dense to dense - wet										

LOG CONTINUED AT 27 FEET ON FIGURE B-4 (2 of 2)

NOTES

① Drilled 08/26/2021 using a SIMCO 2400 SK-1 Longstroke, 7" hollow stem augers, and a 30" drop by 140 lb. cathead sampling hammer. See notes in Figure A-1, Appendix A.

② See report text and figures in Appendices A and C for additional definitions, boring information, lab test results, and ground descriptions.

③ Groundwater seepage was encountered in samples or during drilling at a depth of 10' and 16.5' and a groundwater level was measured at 31' prior to boring backfilling on 08/26/2021.

LOG OF BORING B-4 (continued)①

DEPTH feet	SAMPLE NO.	TYPE	PENETRATION RESISTANCE blows/ft.	GROUNDWATER	DESCRIPTION	% MOISTURE %	DRY DENSITY lbs./ft.³	LIQUID LIMIT	PLASTICITY INDEX	GRAIN SIZE Gravel (>#4 sieve) %	GRAIN SIZE Sand (#4 to #200 sieve) %	GRAIN SIZE Fines (<#200 sieve) %	UNCONFINED COMPRESSIVE STRENGTH kips/ft.²	DIRECT SHEAR Cohesion p.s.f.	DIRECT SHEAR Internal Friction Angle
					LOG CONTINUED FROM 27 FEET ON FIGURE B-4 (1 of 2)										
	10	■	52		**POORLY GRADED SAND WITH SILT (SP-SM)** - dark gray - trace clay and gravel - nonplastic - medium dense to dense - wet	22	100			4	88	8	FINES 5% Silt 3% Clay		
	11	◤	12												
30				▼											
	12	◤	20												
35					**ELASTIC SILT (MH)** - white - diatomaceous and/or tuffaceous (bentonitic volcanic ash ?) - highly plastic - medium stiff - wet										
					ELASTIC SILT (MH) and PEAT (PT) - black - highly plastic - medium stiff - wet										
	13	◤	4					112	42						
40					**BOTTOM OF BORING AT 40 FEET**										
45															
50															

NOTES

① See notes on Figure B-4 (1 of 2).

McMILLEN JACOBS ASSOCIATES

File No. 6231.0

April 2023

GHD

Transportation Agency for Monterey County
FORTAG - Canyon Del Rey/SR218 Segment
Del Rey Oaks, California

Log of Boring B-4

Figure

B-4

(2 of 2)

LOG OF BORING B-5a①

LOCATION: see Figure 1

DEPTH feet	SAMPLE NO.	TYPE	PENETRATION RESISTANCE blows/ft.	③GROUNDWATER	DESCRIPTION②	% MOISTURE %	DRY DENSITY lbs./ft.³	LIQUID LIMIT	PLASTICITY INDEX	GRAIN SIZE Gravel (>#4 sieve) %	GRAIN SIZE Sand (#4 to #200 sieve) %	GRAIN SIZE Fines (<#200 sieve) %	UNCONFINED COMPRESSIVE STRENGTH kips/ft.²	DIRECT SHEAR Cohesion p.s.f.	DIRECT SHEAR Internal Friction Angle
					POORLY GRADED SAND (SP) - FILL - light brown - dry - trace clay and gravel - nonplastic										
					BORING B-5a REFUSAL AT 2 FEET ON APPARENT CONCRETE AND METAL, MOVED 10 FEET TO THE SOUTHWEST AND DRILLED BORING B-5b										
5															
10															
15															
20															
25															

NOTES

① Drilled 08/26/2021 using a SIMCO 2400 SK-1 Longstroke, 7" hollow stem augers, and a 30" drop by 140 lb. cathead sampling hammer. See notes in Figure A-1, Appendix A.

② See report text and figures in Appendices A and C for additional definitions, boring information, lab test results, and ground descriptions.

③ Groundwater seepage was not encountered during drilling nor prior to boring backfilling on 08/26/2021.

McMILLEN JACOBS ASSOCIATES

File No. 6231.0

April 2023

GHD

Transportation Agency for Monterey County
FORTAG - Canyon Del Rey/SR218 Segment
Del Rey Oaks, California

Log of Boring B-5a

Figure

B-5a

LOG OF BORING B-5b①

LOCATION: see Figure 1

DEPTH feet	SAMPLE NO.	TYPE	PENETRATION RESISTANCE blows/ft.	③GROUNDWATER	DESCRIPTION②	MOISTURE %	DRY DENSITY lbs./ft.³	LIQUID LIMIT	PLASTICITY INDEX	GRAIN SIZE Gravel (>#4 sieve) %	GRAIN SIZE Sand (#4 to #200 sieve) %	GRAIN SIZE Fines (<#200 sieve) %	UNCONFINED COMPRESSIVE STRENGTH kips/ft.²	DIRECT SHEAR Cohesion p.s.f.	DIRECT SHEAR Internal Friction Angle
	1	■	17		**SILTY SAND (SM) - FILL** - light brown - nonplastic - medium dense - dry	2	101								
	2	◤	4		**POORLY GRADED SAND WITH SILT AND CLAY (SP-SM/SP-SC) - FILL** - yellowish brown and brown to light brown with some reddish brown - trace gravel - nonplastic - loose - dry to moist					3	88	9 FINES 5% Silt 4% Clay			
5															
	3	■	4		- light brown with some reddish brown	4	95								
10	4a	◣			- sample 4a bouncing on apparent concrete at 10.5' - drilled through concrete between 10.5' and 12'										
	4b	◤	3		**SILTY CLAYEY SAND (SM/SC)** - very dark gray - tuffaceous layers (?) - trace gravel - medium to highly plastic fines - very loose to loose - moist to wet										
15															
	5	■	4	▽		78	52							620	26°
	6	◤	4							<1	75	25 FINES 16% Silt 9% Clay			
20															
	7	■	6		**SILTY CLAYEY SAND (SM/SC)** - dark brown - trace gravel - medium plastic fines - loose - moist to wet										
25	8	◤	4	▼						4	52	44 FINES 23% Silt 21% Clay			

LOG CONTINUED AT 27 FEET ON FIGURE B-5b (2 of 2)

NOTES

① Drilled 08/26/2021 using a SIMCO 2400 SK-1 Longstroke, 7" hollow stem augers, and a 30" drop by 140 lb. cathead sampling hammer. See notes in Figures A-1 and A-2, Appendix A.

② See report text and figures in Appendices A and C for additional definitions, boring information, lab test results, and ground descriptions.

③ Groundwater seepage was encountered in samples or during drilling at a depth of 18' and 29', and groundwater level was measured at 26' prior to boring backfilling on 08/26/2021.

McMILLEN JACOBS ASSOCIATES

File No. 6231.0

April 2023

GHD

Transportation Agency for Monterey County
FORTAG - Canyon Del Rey/SR218 Segment
Del Rey Oaks, California

Log of Boring B-5b

Figure

B-5b

(1 of 2)

LOG OF BORING B-5b (continued) ①

DEPTH feet	SAMPLE NO.	TYPE	PENETRATION RESISTANCE blows/ft.	GROUNDWATER	DESCRIPTION	% MOISTURE %	DRY DENSITY lbs./ft.^{3}	LIQUID LIMIT	PLASTICITY INDEX	GRAIN SIZE			UNCONFINED COMPRESSIVE STRENGTH kips/ft.^{2}	DIRECT SHEAR	
										Gravel (>#4 sieve) %	Sand (#4 to #200 sieve) %	Fines (<#200 sieve) %		Cohesion p.s.f.	Internal Friction Angle
					LOG CONTINUED FROM 27 FEET ON FIGURE B-5b (1 of 2)										
30	9	◤	5	▽	**SILTY CLAYEY SAND (SM/SC)** - dark brown - trace gravel - medium plastic fines - loose - wet										
	10	■	34		**ELASTIC SILT WITH SAND (MH) and CLAYSTONE/SILTSTONE - MONTEREY FORMATION (?)** - gray, trace blue mottling - few clay - medium plastic and highly plastic - cemented soil, to very severely weathered bedrock - very stiff soil, and soft rock hardness - wet	15	104						0.4		
35	11	◤	26												
40	12	◤	27					87	25						
					BOTTOM OF BORING AT 40 FEET										
45															
50															

NOTES

① See notes on Figure B-5b (1 of 2).

McMILLEN JACOBS ASSOCIATES

File No. 6231.0

April 2023

GHD

Transportation Agency for Monterey County
FORTAG - Canyon Del Rey/SR218 Segment
Del Rey Oaks, California

Log of Boring B-5b

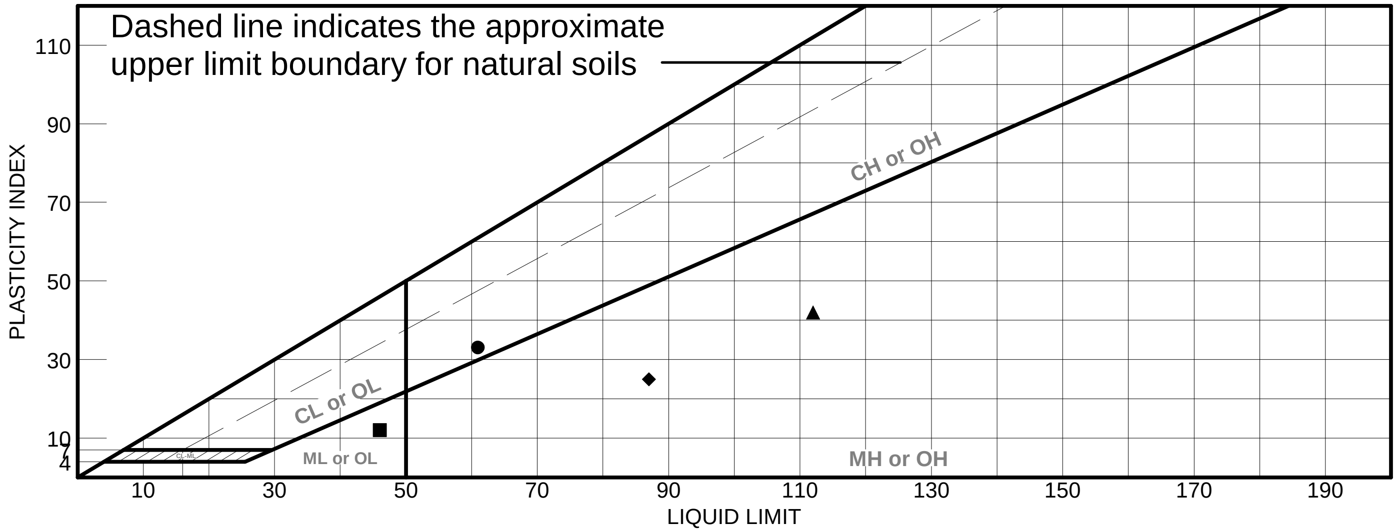
Figure

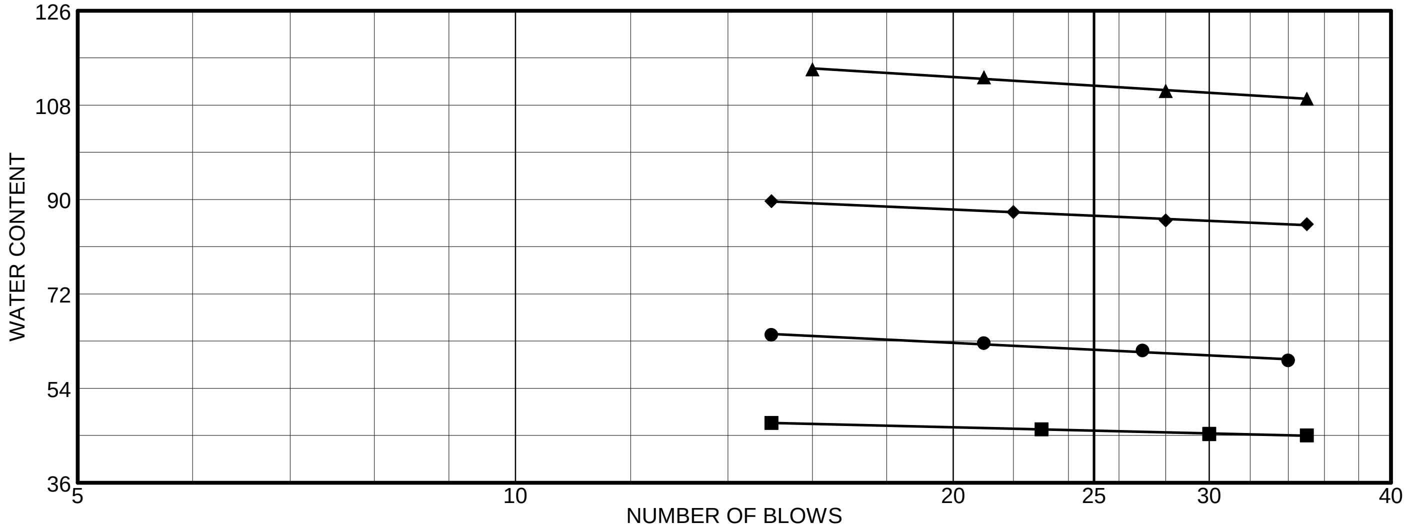
B-5b

(2 of 2)

Appendix C

LIQUID AND PLASTIC LIMITS TEST REPORT (ASTM D4318)





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Very Dark Olive Gray Fat CLAY	61	28	33			
■	Very Dark Bluish Gray SILT w/ Sand	46	34	12			
▲	Very Dark Olive Brown Elastic SILT	112	70	42			
◆	Dark Olive Brown Elastic SILT	87	62	25			

Project No. 1022-034 **Client:** McMillen Jacobs Associates

Project: 6231

● **Source:** B-3-3 **Elev./Depth:** 5'
■ **Source:** B-4-7 **Elev./Depth:** 16.5'
▲ **Source:** B-4-13 **Elev./Depth:** 38.5'
◆ **Source:** B-5-12 **Elev./Depth:** 38.5'

Remarks:

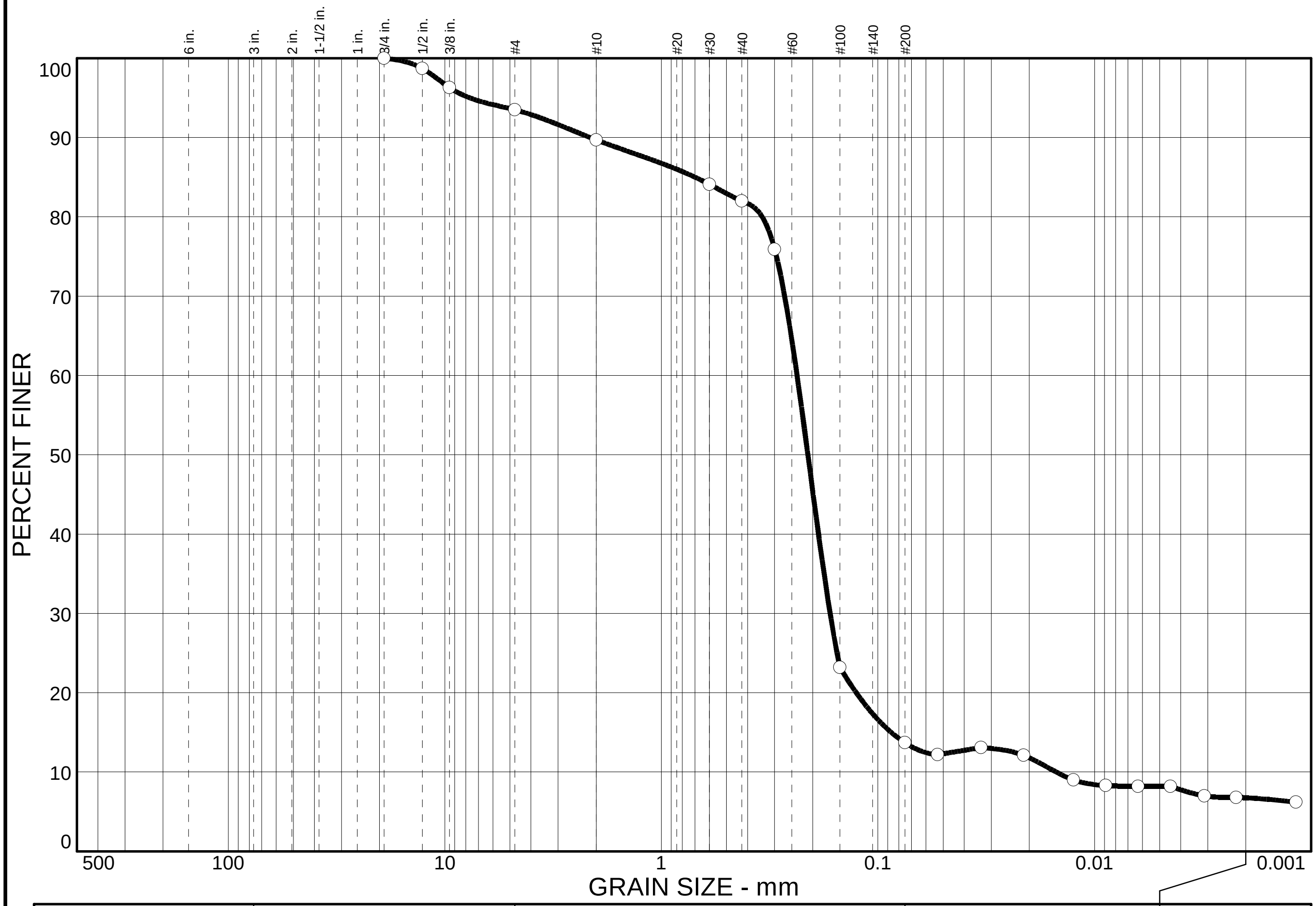
- ● Sample was prepared using the wet prep method.
- ■ Sample was prepared using the wet prep method.
- ▲ Sample was prepared using the wet prep method.
- ◆ Sample was prepared using the wet prep method.

LIQUID AND PLASTIC LIMITS TEST REPORT (ASTM D4318)

COOPER TESTING LABORATORY

Figure

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	6.5	79.8	6.9	6.8

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4 in.	100.0		
1/2 in.	98.7		
3/8 in.	96.3		
#4	93.5		
#10	89.7		
#30	84.1		
#40	82.0		
#50	75.9		
#100	23.2		
#200	13.7		
#270	12.2		
0.0334 mm.	13.1		
0.0212 mm.	12.1		
0.0125 mm.	9.0		
0.0089 mm.	8.3		
0.0063 mm.	8.2		
0.0045 mm.	8.2		
0.0031 mm.	7.0		
0.0022 mm.	6.8		
0.0012 mm.	6.2		

* (no specification provided)

Soil Description

Dark Reddish Brown Silty SAND

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.700	D_{60}= 0.237	D_{50}= 0.211
D_{30}= 0.166	D_{15}= 0.0870	D_{10}= 0.0150
C_u= 15.73	C_c= 7.78	

Classification

USCS= AASHTO=

Remarks

Sample No.: **Source of Sample:** B-4-1 **Date:** 9/14/21

Location: **Elev./Depth:**

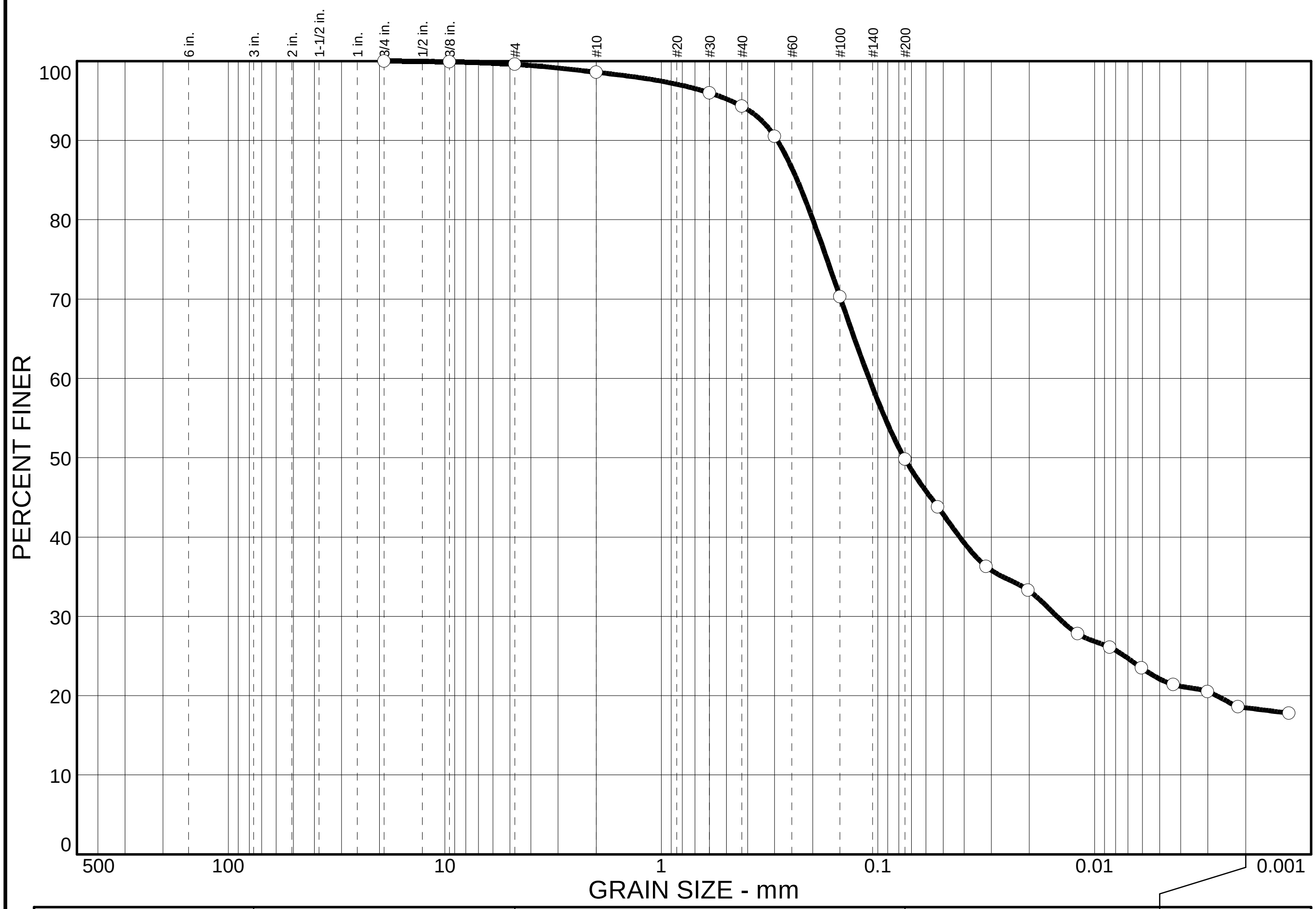
COOPER TESTING LABORATORY

Client: McMillen Jacobs Associates

Project: 6231

Project No: 1022-034 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.4	49.8	31.3	18.5

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4 in.	100.0		
3/8 in.	99.9		
#4	99.6		
#10	98.6		
#30	96.0		
#40	94.3		
#50	90.5		
#100	70.3		
#200	49.8		
#270	43.8		
0.0317 mm.	36.3		
0.0203 mm.	33.3		
0.0120 mm.	27.8		
0.0085 mm.	26.1		
0.0061 mm.	23.5		
0.0043 mm.	21.4		
0.0030 mm.	20.5		
0.0022 mm.	18.6		
0.0013 mm.	17.8		

* (no specification provided)

Soil Description

Black Silty SAND

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.236 D_{60}= 0.110 D_{50}= 0.0757

D_{30}= 0.0149 D_{15}= D_{10}=

C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Sample No.: **Source of Sample:** B-4-5 **Date:** 9/17/21

Location: **Elev./Depth:**

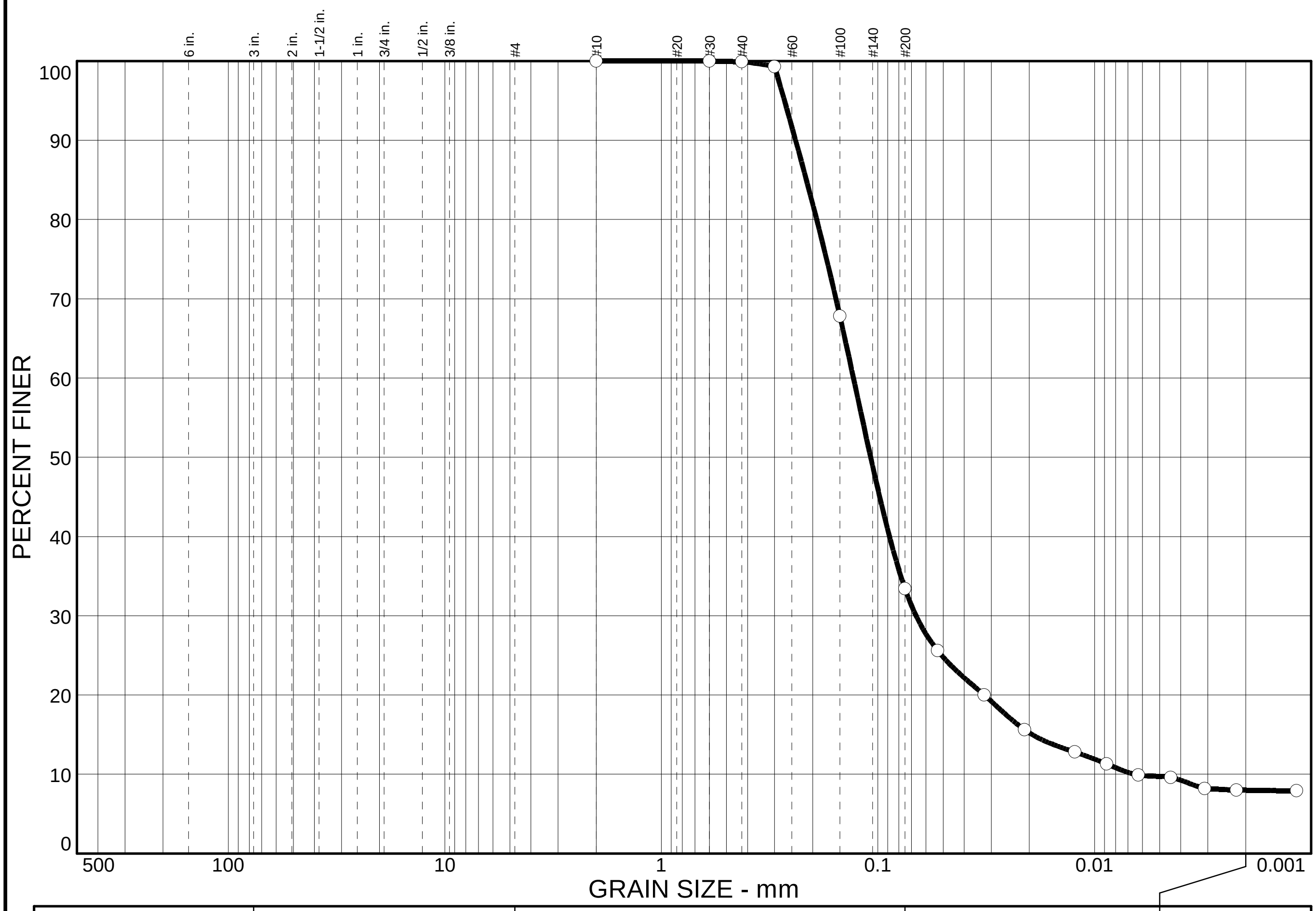
COOPER TESTING LABORATORY

Client: McMillen Jacobs Associates

Project: 6231

Project No: 1022-034 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	66.6	25.4	8.0

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#30	100.0		
#40	99.9		
#50	99.3		
#100	67.8		
#200	33.4		
#270	25.6		
0.0323 mm.	20.0		
0.0210 mm.	15.6		
0.0123 mm.	12.8		
0.0088 mm.	11.3		
0.0063 mm.	9.9		
0.0044 mm.	9.6		
0.0031 mm.	8.2		
0.0022 mm.	8.0		
0.0012 mm.	7.9		

* (no specification provided)

Soil Description

Olive Gray Clayey SAND

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.214 D_{60}= 0.130 D_{50}= 0.108
D_{30}= 0.0667 D_{15}= 0.0194 D_{10}= 0.0065
C_u= 19.87 C_c= 5.25

Classification

USCS= AASHTO=

Remarks

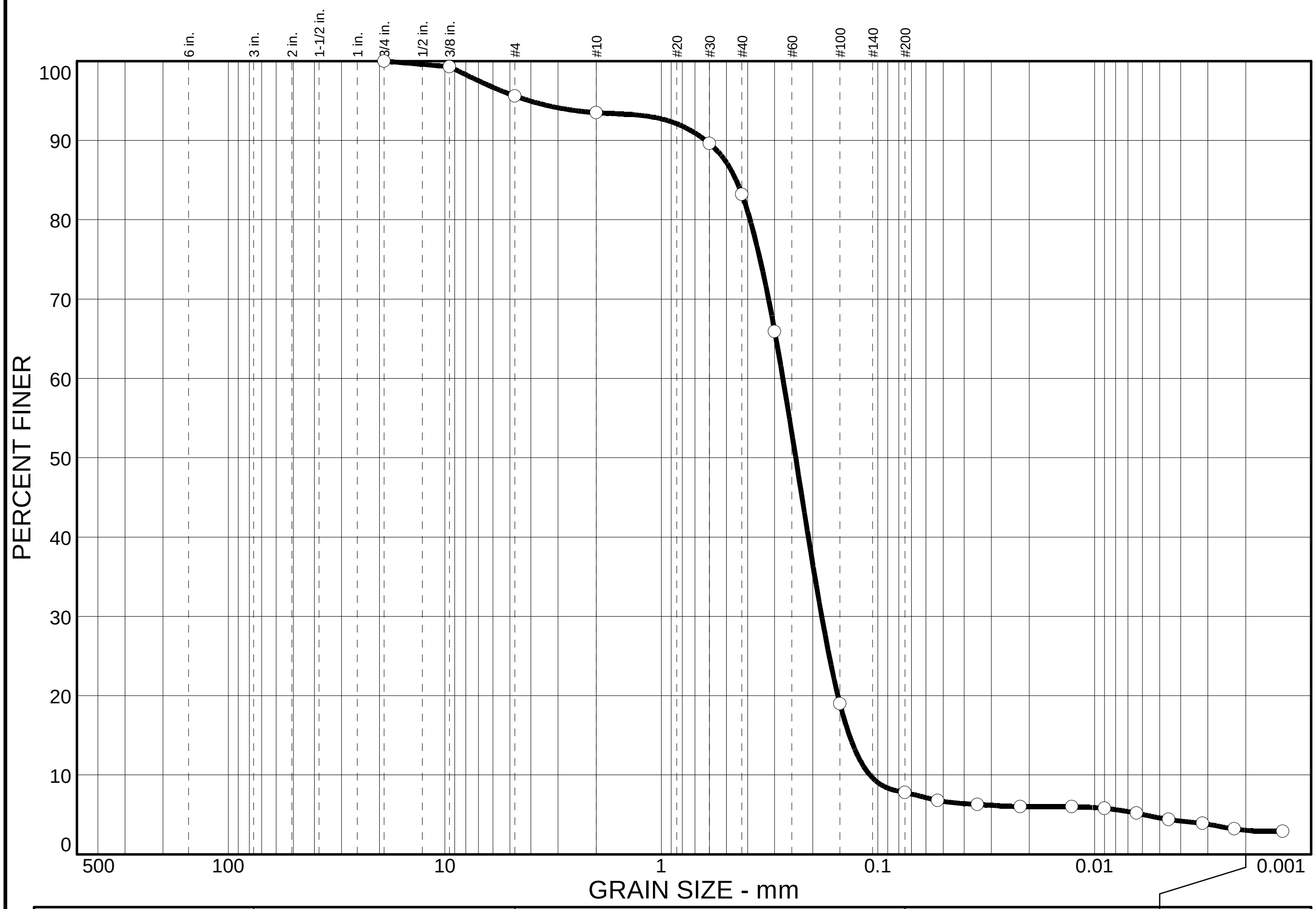
Sample No.: **Source of Sample:** B-4-9 **Date:** 9/14/21
Location: **Elev./Depth:**

COOPER TESTING LABORATORY

Client: McMillen Jacobs Associates
Project: 6231

Project No: 1022-034 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	4.4	87.8	4.8	3.0

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4 in.	100.0		
3/8 in.	99.3		
#4	95.6		
#10	93.5		
#30	89.6		
#40	83.2		
#50	65.9		
#100	19.0		
#200	7.8		
#270	6.8		
0.0347 mm.	6.3		
0.0220 mm.	6.0		
0.0127 mm.	6.0		
0.0090 mm.	5.8		
0.0064 mm.	5.2		
0.0046 mm.	4.4		
0.0032 mm.	3.9		
0.0023 mm.	3.2		
0.0014 mm.	2.9		

* (no specification provided)

Soil Description

Gray Poorly Graded SAND w/ Silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.452	D_{60}= 0.275	D_{50}= 0.240
D_{30}= 0.182	D_{15}= 0.136	D_{10}= 0.109
C_u= 2.52	C_c= 1.10	

Classification

USCS= AASHTO=

Remarks

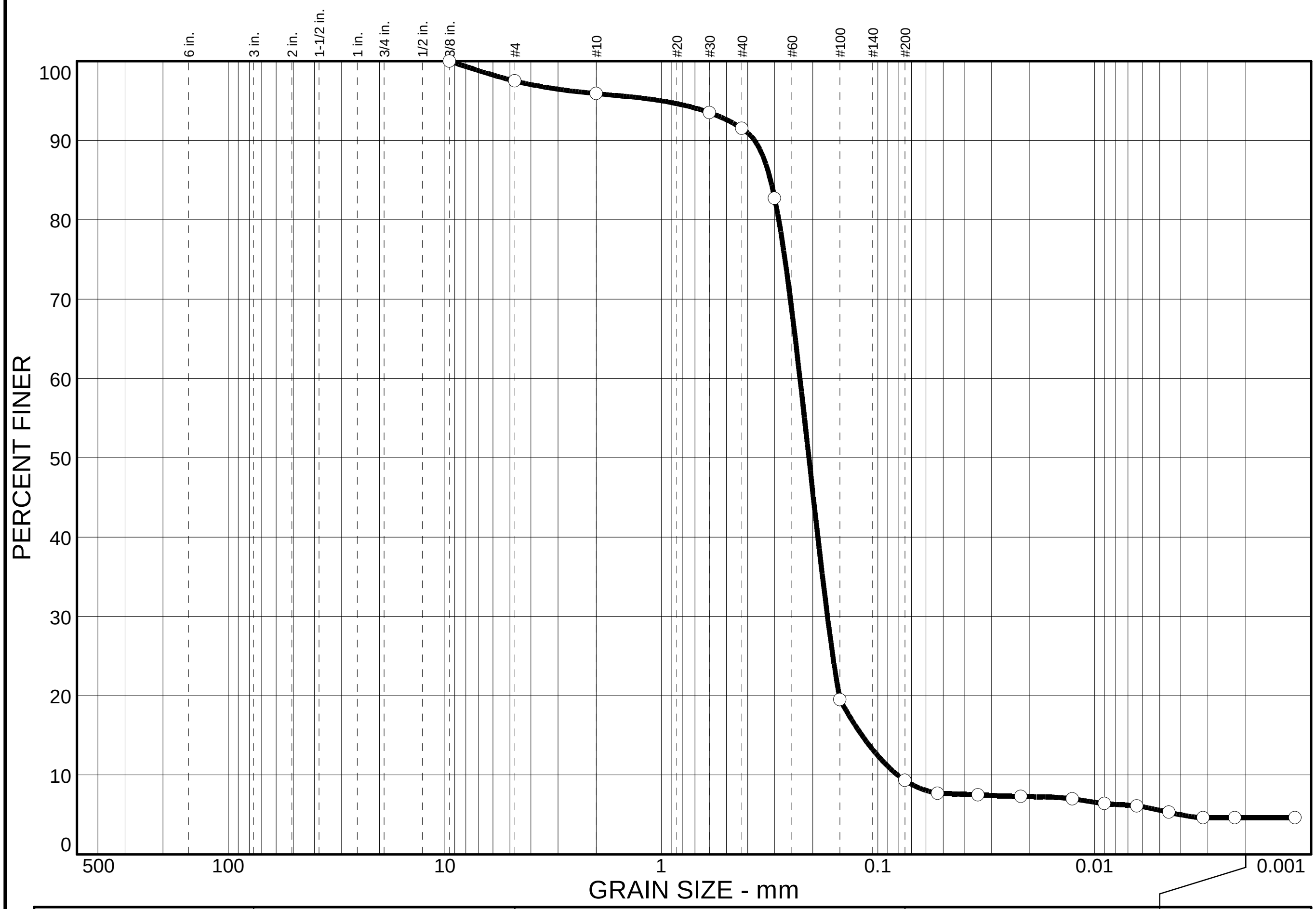
Sample No.: **Source of Sample:** B-4-10 **Date:** 9/16/21
Location: **Elev./Depth:**

COOPER TESTING LABORATORY

Client: McMillen Jacobs Associates
Project: 6231

Project No: 1022-034 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	2.5	88.2	4.7	4.6

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8 in.	100.0		
#4	97.5		
#10	95.9		
#30	93.5		
#40	91.5		
#50	82.7		
#100	19.5		
#200	9.3		
#270	7.7		
0.0345 mm.	7.5		
0.0219 mm.	7.3		
0.0127 mm.	7.0		
0.0090 mm.	6.4		
0.0064 mm.	6.1		
0.0045 mm.	5.3		
0.0032 mm.	4.6		
0.0022 mm.	4.6		
0.0012 mm.	4.6		

* (no specification provided)

Soil Description

Reddish Brown Poorly Graded SAND w/ Silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.314 D_{60}= 0.229 D_{50}= 0.209

D_{30}= 0.171 D_{15}= 0.119 D_{10}= 0.0812

C_u= 2.82 C_c= 1.57

Classification

USCS= AASHTO=

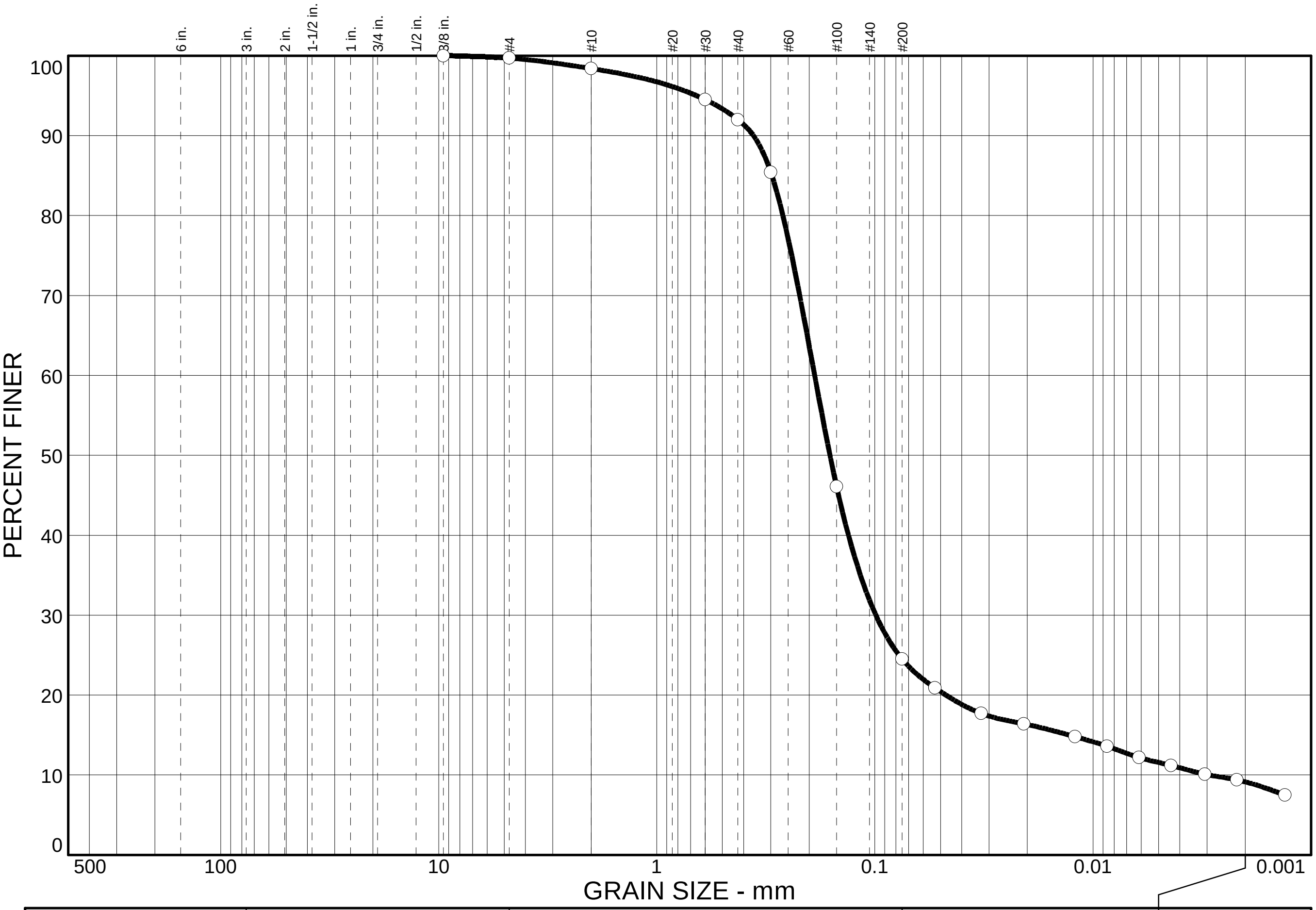
Remarks

Sample No.:
Source of Sample: B-5-2
Date: 9/14/21
Location:
Elev./Depth:

COOPER TESTING LABORATORY

Client: McMillen Jacobs Associates
Project: 6231
Project No: 1022-034
Figure

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.3	75.2	15.4	9.1

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8 in.	100.0		
#4	99.7		
#10	98.4		
#30	94.5		
#40	92.0		
#50	85.4		
#100	46.1		
#200	24.5		
#270	20.9		
0.0325 mm.	17.7		
0.0207 mm.	16.4		
0.0121 mm.	14.8		
0.0086 mm.	13.6		
0.0062 mm.	12.2		
0.0044 mm.	11.2		
0.0031 mm.	10.1		
0.0022 mm.	9.4		
0.0013 mm.	7.5		

* (no specification provided)

Soil Description

Dark Yellowish Brown Silty SAND

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.297 D_{60}= 0.189 D_{50}= 0.161
D_{30}= 0.0991 D_{15}= 0.0129 D_{10}= 0.0029
C_u= 64.44 C_c= 17.68

Classification

USCS= AASHTO=

Remarks

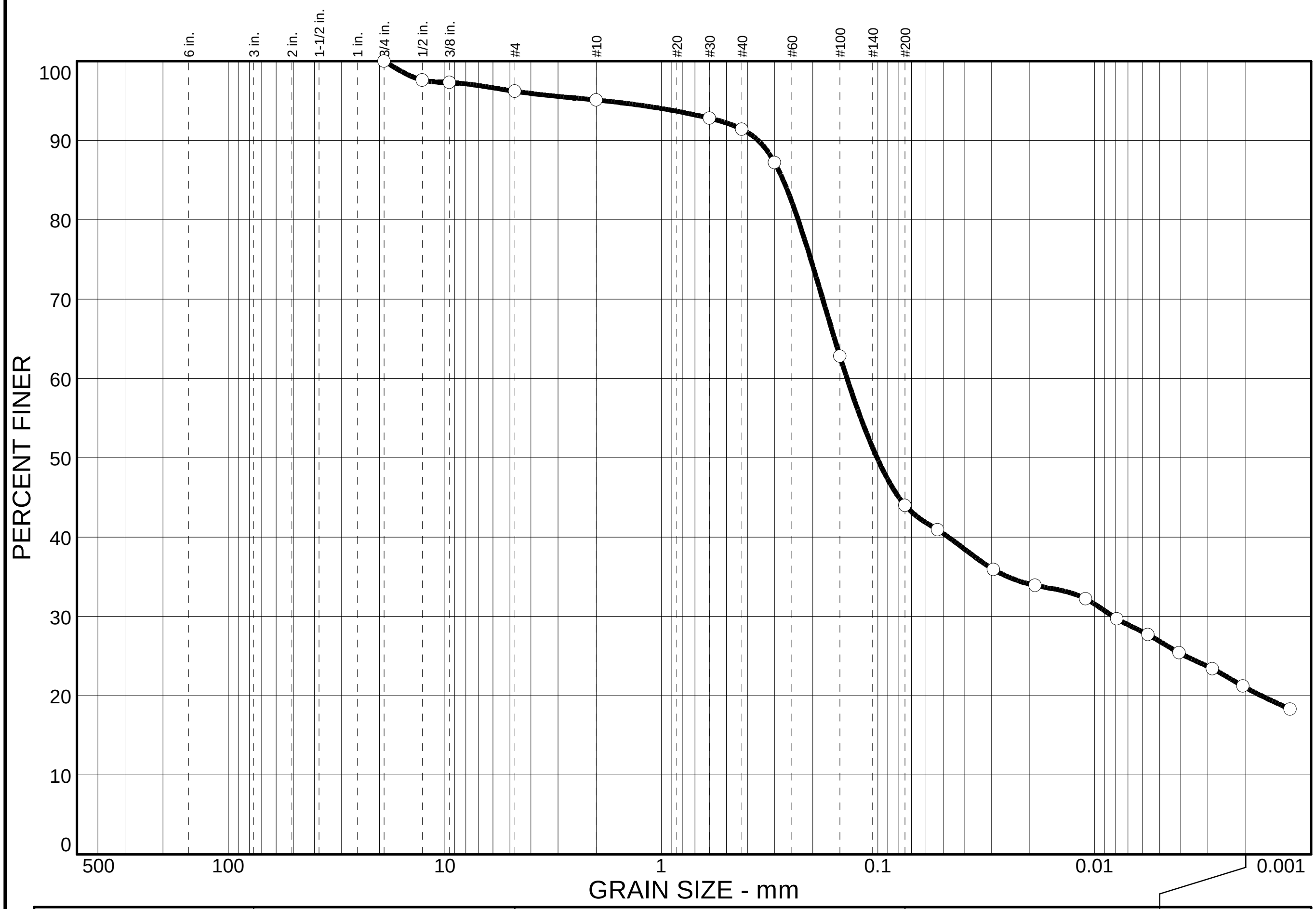
Sample No.: **Source of Sample:** B-5-6 **Date:** 9/16/21
Location: **Elev./Depth:**

COOPER TESTING LABORATORY

Client: McMillen Jacobs Associates
Project: 6231

Project No: 1022-034 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	3.8	52.2	23.0	21.0

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4 in.	100.0		
1/2 in.	97.6		
3/8 in.	97.3		
#4	96.2		
#10	95.1		
#30	92.8		
#40	91.4		
#50	87.2		
#100	62.8		
#200	44.0		
#270	40.9		
0.0293 mm.	35.9		
0.0188 mm.	33.9		
0.0110 mm.	32.2		
0.0079 mm.	29.7		
0.0057 mm.	27.7		
0.0041 mm.	25.4		
0.0029 mm.	23.4		
0.0021 mm.	21.2		
0.0012 mm.	18.3		

* (no specification provided)

Soil Description

Dark Grayish Brown Clayey SAND

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.274 D_{60}= 0.139 D_{50}= 0.101
D_{30}= 0.0082 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Sample No.: **Source of Sample:** B-5-8 **Date:** 9/16/21
Location: **Elev./Depth:**

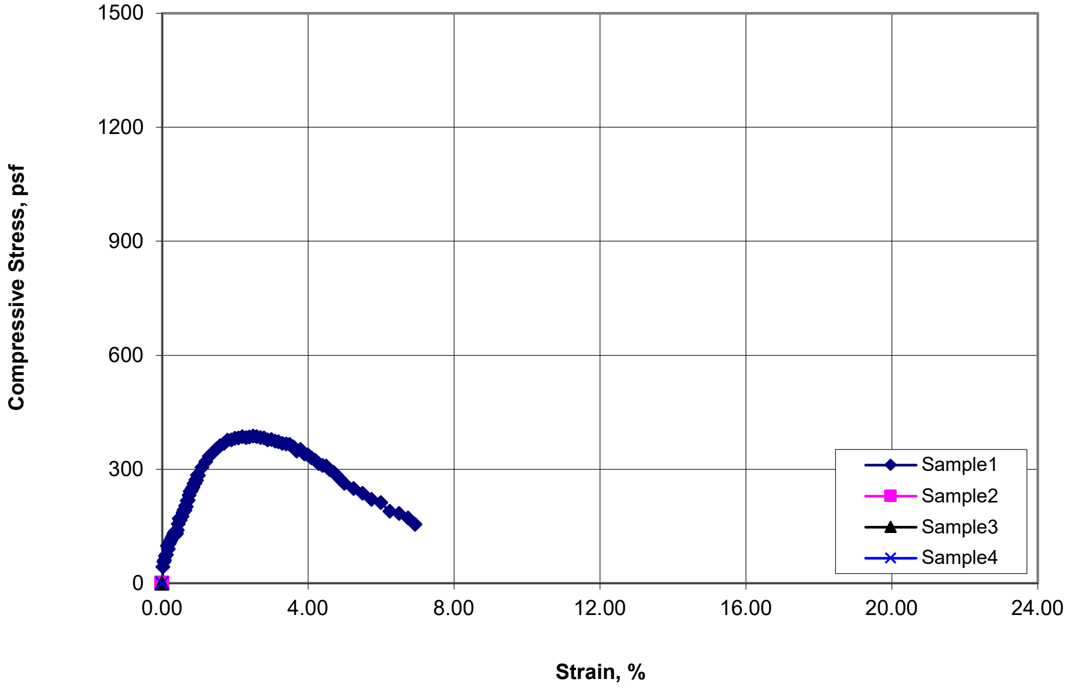
COOPER TESTING LABORATORY

Client: McMillen Jacobs Associates
Project: 6231

Project No: 1022-034 **Figure**

Unconfined Compressive Strength

ASTM D2166



Sample No.:	1	2	3	4	
Unconfined Compressive Strength, psf	388				
Unconfined Compressive Strength, psi	2.7				
Undrained Shear Strength, psf	194				
Failure Strain, %	2.5				
Strain Rate, % per minute	1.0				
Strain Rate, inches/minute	0.05				
Moisture Content, %	14.9				
Dry Density, pcf	103.6				
Saturation, %	64.3				
Void Ratio	0.627				
Specimen Diameter, inches	2.390				
Specimen Height, inches	5.00				
Height to Diameter Ratio	2.1				
Assumed Specific Gravity	2.70				

Sample Location				
	Boring	**Sample**	**Depth, ft.**	**Soil Description**
1	B-5-10		34-34.5	Dark Yellowish Brown Silty SAND
2				
3				
4				

Job No.:	1022-034	**Type of Sample**	Undisturbed
Client:	McMillen Jacobs Associates		
Project:	6231	Remarks:	
Date:	9/9/2021 **By:** MD/RU		

COOPER
TESTING LABORATORY

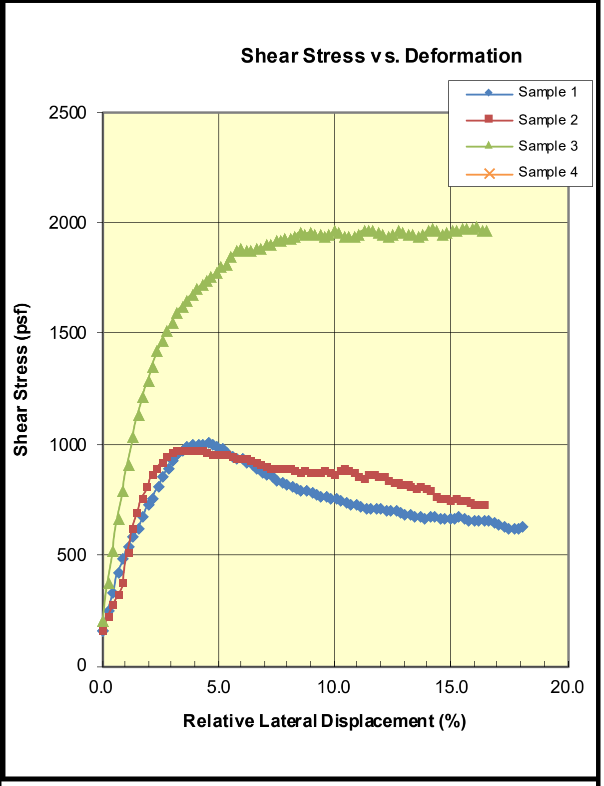
COOPER TESTING LABORATORY

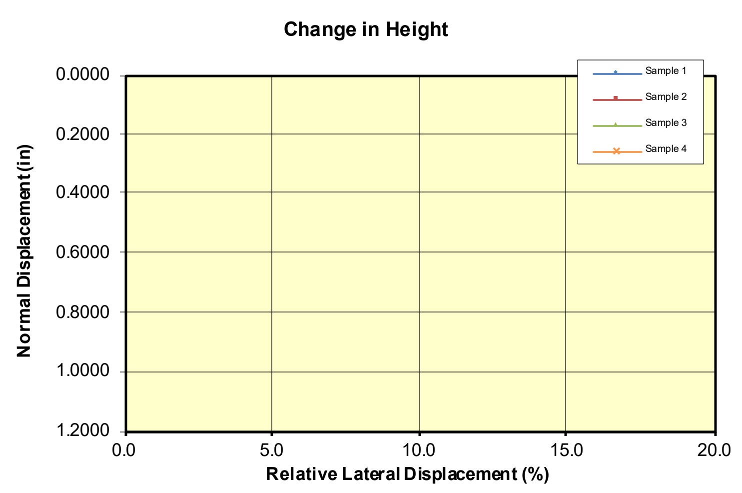
Consolidated Undrained Direct Shear
(ASTM D3080M)

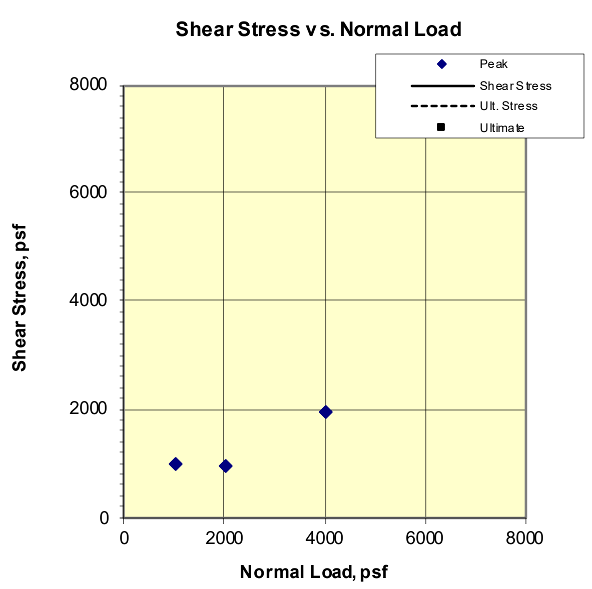
CTL Job #:	1022-034	Project #:	6231	By:	MD
Client:	McMillen Jacbos Associates	Date:	9/9/2021	Checked:	PJ
Project Name:		Remolding Info:			

Specimen Data				
	1	2	3	4
Boring:	B-4-6a	B-4-6a	B-4-6a	
Sample:				
Depth (ft):				
Visual Description:	Gray Sandy CLAY	Gray Sandy CLAY	Gray Sandy CLAY	
Normal Load (psf)	1000	2000	4000	
Dry Mass of Specimen (g)	85.9	88.5	94.6	
Initial Height (in)	1.01	1.01	1.00	
Initial Diameter (in)	2.42	2.42	2.42	
Initial Void Ratio	1.383	1.326	1.149	
Initial Moisture (%)	46.3	44.6	36.9	
Initial Wet Density (pcf)	103.5	104.8	107.4	
Initial Dry Density (pcf)	70.7	72.5	78.4	
Initial Saturation (%)	90.4	90.8	86.7	
ΔHeight Consol (in)	0.0198	0.0411	0.0786	
At Test Void Ratio	1.336	1.231	0.980	
At Test Moisture (%)	46.7	44.1	34.8	
At Test Wet Density (pcf)	105.9	108.9	114.7	
At Test Dry Density (pcf)	72.2	75.5	85.1	
At Test Saturation (%)	94.4	96.8	95.8	
Strain Rate (%/min)	1.2	1.0	1.1	
Strengths Picked at	Peak	Peak	Peak	
Shear Stress (psf)	1008	974	1979	
ΔHeight (in) at Peak				
Ultimate Stress (psf)				

Phi (deg)		Ult. Phi (deg)	
Cohesion (psf)		Ult. Cohesion (psf)	







Remarks: *DS-CU* A fully undrained condition may not be attained in this test. ΔH is not measured during undrained direct shear tests. Engineering judgement is required to determine phi and cohesion, no phi or cohesion is reported. To add phi and cohesion to the report go to the "phi" tab and in cells G30, G31, H30, and H31 enter end points for a line through the 3 data points. The points plotted can be changed on the "Eng Values" tab using cells L6, A2, C2, and E2.

©

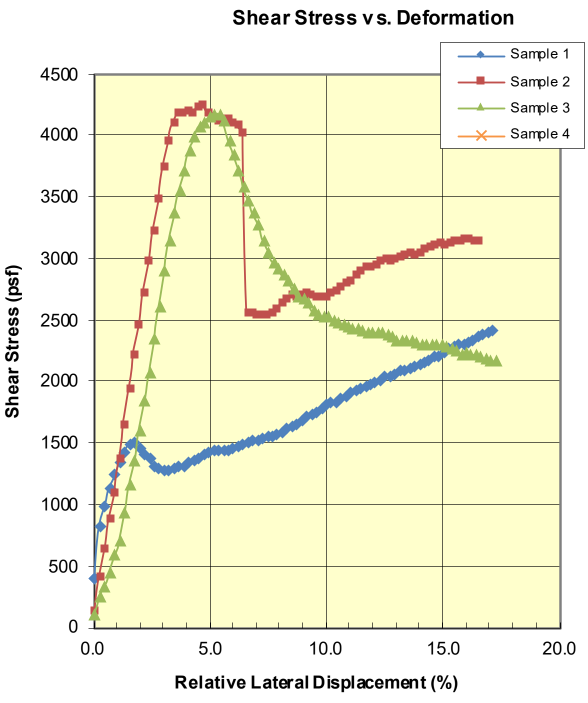
COOPER
TESTING LABORATORY

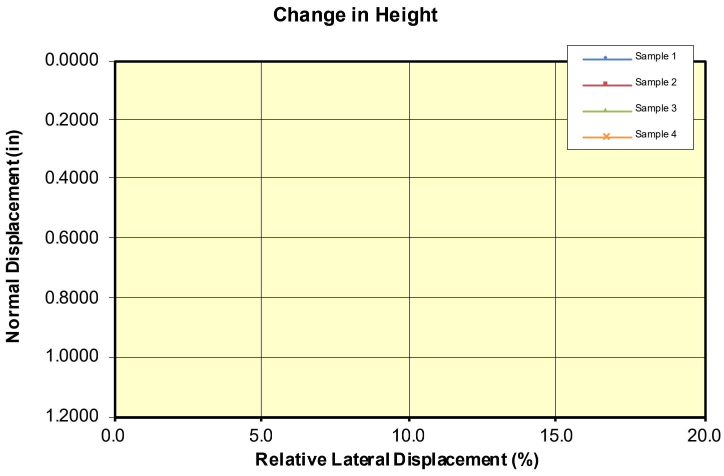
Consolidated Undrained Direct Shear
(ASTM D3080M)

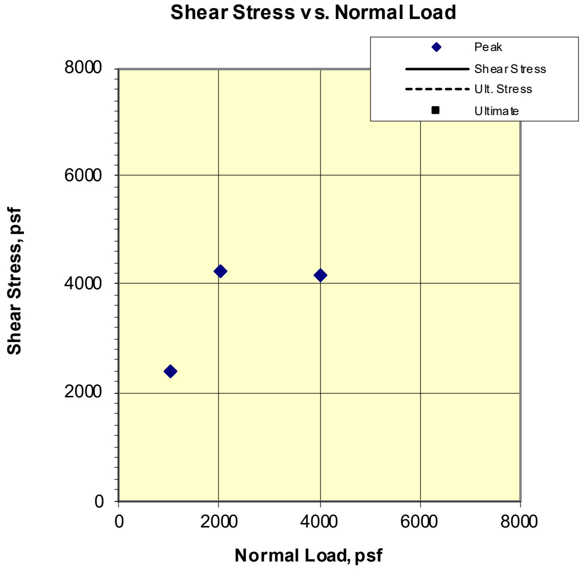
CTL Job #: 1022-034
Project #: 6231
By: MD
Client: McMillen Jacobs Associates
Date: 9/10/2021
Checked: PJ
Project Name:
Remolding Info:

Specimen Data				
	1	2	3	4
Boring:	B-5-5b	B-5-5b	B-5-5b	
Sample:				
Depth (ft):				
Visual Description:	Olive Gray Sandy SILT	Olive Gray Sandy SILT	Olive Gray Sandy SILT	
Normal Load (psf)	1000	2000	4000	
Dry Mass of Specimen (g)	61.1	62.7	65.2	
Initial Height (in)	1.00	1.00	1.02	
Initial Diameter (in)	2.42	2.42	2.42	
Initial Void Ratio	2.338	2.251	2.185	
Initial Moisture (%)	79.3	78.2	75.6	
Initial Wet Density (pcf)	90.6	92.4	92.9	
Initial Dry Density (pcf)	50.5	51.9	52.9	
Initial Saturation (%)	91.6	93.8	93.4	
ΔHeight Consol (in)	0.0069	0.0097	0.0233	
At Test Void Ratio	2.315	2.219	2.112	
At Test Moisture (%)	81.7	79.3	77.3	
At Test Wet Density (pcf)	92.4	93.9	96.0	
At Test Dry Density (pcf)	50.9	52.4	54.2	
At Test Saturation (%)	95.3	96.5	98.8	
Strain Rate (%/min)	1.0	1.0	1.1	
Strengths Picked at	Peak	Peak	Peak	
Shear Stress (psf)	2414	4255	4176	
ΔHeight (in) at Peak				
Ultimate Stress (psf)				

Phi (deg)		**Ult. Phi (deg)**	
Cohesion (psf)		**Ult. Cohesion (psf)**	







Remarks: *DS-CU* A fully undrained condition may not be attained in this test. ΔH is not measured during undrained direct shear tests. Engineering judgement is required to determine phi and cohesion, no phi or cohesion is reported. To add phi and cohesion to the report go to the “phi” tab and in cells G30, G31, H30, and H31 enter end points for a line through the 3 data points. The points plotted can be changed on the "Eng Values" tab using cells L6, A2, C2, and E2.

COOPER
TESTING LABORATORY

Corrosivity Test Summary

CTL # 1022-034 **Date:** 9/16/2021 **Tested By:** PJ **Checked:** PJ

Client: McMillen Jacobs Associates **Project:** **Proj. No:** 6231

Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate		pH	ORP	Moisture	
Boring	Sample, No.	Depth, ft.	As Rec.	Minimum	Saturated	mg/kg	mg/kg	%		(Redox)	At Test	Soil Visual Description
						Dry Wt.	Dry Wt.	Dry Wt.		mv	%	
			ASTM G57	Cal 643	ASTM G57	Cal 422-mod.	Cal 417-mod.	Cal 417-mod.	Cal 643	SM 2580B	ASTM D2216	
B-4-5	-	-	-	1908	-	29	417	0.0417	7.1	-	2.8	Black Silty SAND

Appendix D

PRESENTATION OF SITE INVESTIGATION RESULTS

FORTAG Phase 1 Canyon Del Rey SR218 Segment

Prepared for:

Delve Underground

ConeTec Job No: 23-56-25414

Project Start Date: 2023-Feb-21
Project End Date: 2023-Feb-21
Report Date: 2023-Feb-22

Prepared by:

ConeTec Inc.

820 Aladdin Avenue, San Leandro, CA 95477
Tel: (510) 357-3677

ConeTecCA@conetec.com
www.conetec.com
www.conetecdataservices.com

ABOUT THIS REPORT

The enclosed report presents the results of the site investigation program conducted by ConeTec, Inc. The program consisted of Piezocone Penetration Testing and Pore Pressure Dissipation Testing. Please note that this report, which also includes all accompanying data, are subject to the 3rd Party Disclaimer and Client Disclaimer that follow in the 'Limitations' section of this report.

Project Information	
Client	Delve Underground
Project	FORTAG Phase 1 Canyon Del Rey SR218 Segment
ConeTec Project Number	23-56-25414
Rig Description	30-ton Truck CPT Rig (C-15)

Coordinates	
Collection Method	Consumer Grade GPS
EPSG Number	32610 (WGS 84 / UTM 10S)

Cone Penetration Test (CPTu)	
Depth Reference	Existing ground surface at the time of the investigation
Sleeve data offset	0.1 Meters

Calculated Geotechnical Parameters Tables	
Additional Information	The Normalized Soil Behaviour Type Chart based on Q_{tn} (SBT Q_{tn}) (Robertson, 2009) was used to classify the soil for this project. A detailed set of calculated CPTu parameters have been generated and are provided in Excel format files in the release folder. The CPTu parameter calculations are based on values of corrected tip resistance (q_t) sleeve friction (f_s) and pore pressure (u_2). Effective stresses are calculated based on unit weights that have been assigned to the individual soil behaviour type zones and the assumed equilibrium pore pressure profile. Soils were classified as either drained or undrained based on the Q_{tn} Normalized Soil Behaviour Type Chart (Robertson, 2009). Calculations for both drained and undrained parameters were included for materials that classified as silt mixtures (zone 4).

Please refer to the list of attached documents following the text of this report. A test summary, location map, and plots are included. Thank you for the opportunity to work on this project.

LIMITATIONS

3rd Party Disclaimer

- The “Report” refers to this report titled FORTAG Phase 1 Canyon Del Rey SR218 Segment
- The Report was prepared by ConeTec for Delve Underground

The Report is confidential and may not be distributed to or relied upon by any third parties without the express written consent of ConeTec. Any third parties gaining access to the Report do not acquire any rights as a result of such access. Any use which a third party makes of the Report, or any reliance on or decisions made based on it, are the responsibility of such third parties. ConeTec accepts no responsibility for loss, damage and/or expense, if any, suffered by any third parties as a result of decisions made, or actions taken or not taken, which are in any way based on, or related to, the Report or any portion(s) thereof.

Client Disclaimer

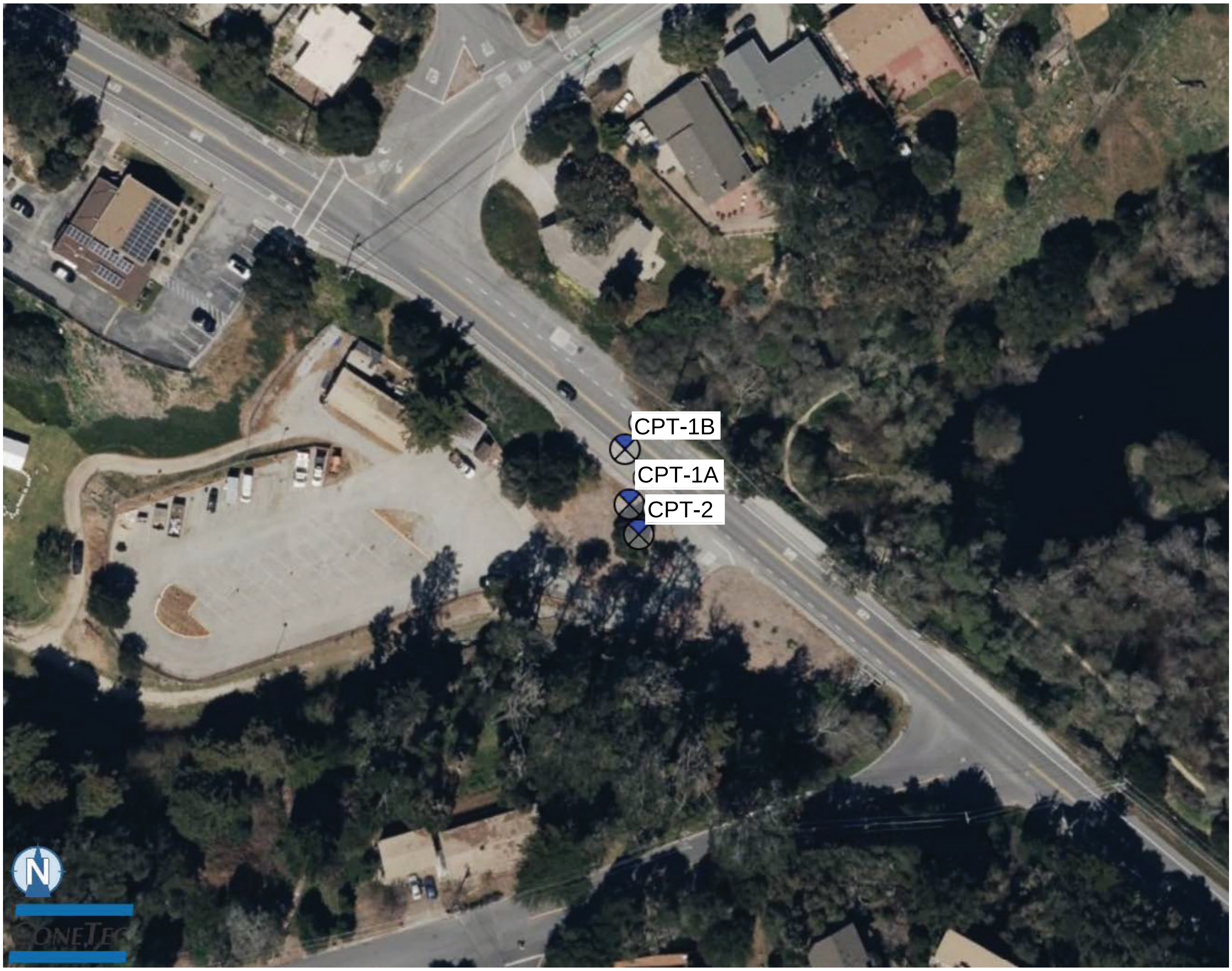
- ConeTec was retained by Delve Underground
- The “Report” refers to this report titled FORTAG Phase 1 Canyon Del Rey SR218 Segment
- ConeTec was retained to collect and provide the raw data (“Data”) which is included in the Report.

ConeTec has collected and reported the Data in accordance with current industry standards. No other warranty, express or implied, with respect to the Data is made by ConeTec. In order to properly understand the Data included in the Report, reference must be made to the documents accompanying and other sources referenced in the Report in their entirety. Other than the Data, the contents of the Report (including any Interpretations) should not be relied upon in any fashion without independent verification and ConeTec is in no way responsible for any loss, damage or expense resulting from the use of, and/or reliance on, such material by any party.

CONTENTS

The following listed below are included in the report:

SITE MAP



ConeTec Job Number: 23-56-25414

Client: Delve Underground

Project: FORTAG Phase 1 Canyon Del Rey SR218 Segment

Report Date: 2023-Feb-22

⊗ **Sounding Location**

All sounding locations are approximate

Cone Penetration Test Summary and Standard Cone Penetration Test Plots

Job No: 23-56-25414
Client: Delve Underground
Project: FORTAG Phase 1 Canyon Del Rey SR218 Segment
Start Date: 21-Feb-2023
End Date: 21-Feb-2023

CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Cone	Cone Area (cm^2)	Assumed Phreatic Surface[1] (ft)	Final Depth (ft)	Northing[2]	Easting[2]	Elevation[3] (ft)	Refer to Notation Number
CPT-1A	23-56-25414_CP03	21-Feb-2023	EC795:T1500F15U35	15	>2.4	2.38	4050435	604098	89	4
CPT-1B	23-56-25414_CP03B	21-Feb-2023	EC795:T1500F15U35	15	17.6	63.40	4050446	604097	93	
CPT-2	23-56-25414_CP04	21-Feb-2023	EC795:T1500F15U35	15	14.6	66.52	4050429	604100	87	

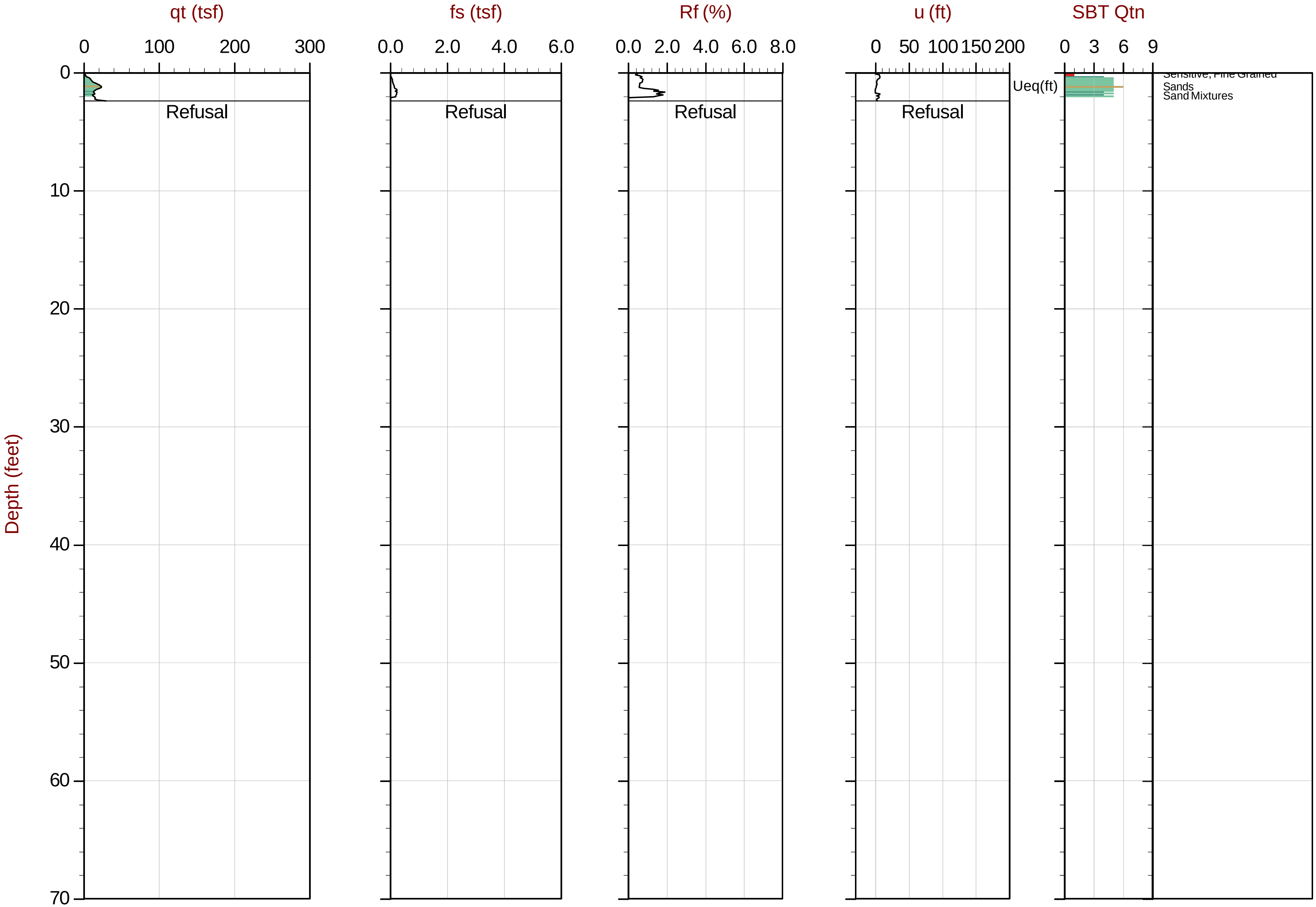
1. The assumed phreatic surface was based off the shallowest pore pressure dissipation tests performed within or nearest the sounding. Hydrostatic conditions were assumed for the calculated parameters.
2. The coordinates were collected using consumer grade GPS equipment. EPSG number: 32610 (WGS84 / UTM Zone 10S).
3. Elevations are referenced to the ground surface and were acquired from the Google Earth Elevation for the recorded coordinates.
4. The assumed phreatic surface is based on the pore pressure dissipation tests nearby soundings.

CONETEC

Delve Underground

Job No: 23-56-25414
Date: 2023-02-21 09:10
Site: FORTAG Phase 1 Canyon Del Rey SR218 Segment

Sounding: CPT-1A
Cone: 795:T1500F15U35



Max Depth: 0.725 m / 2.38 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-56-25414_CP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4050435m E: 604098m

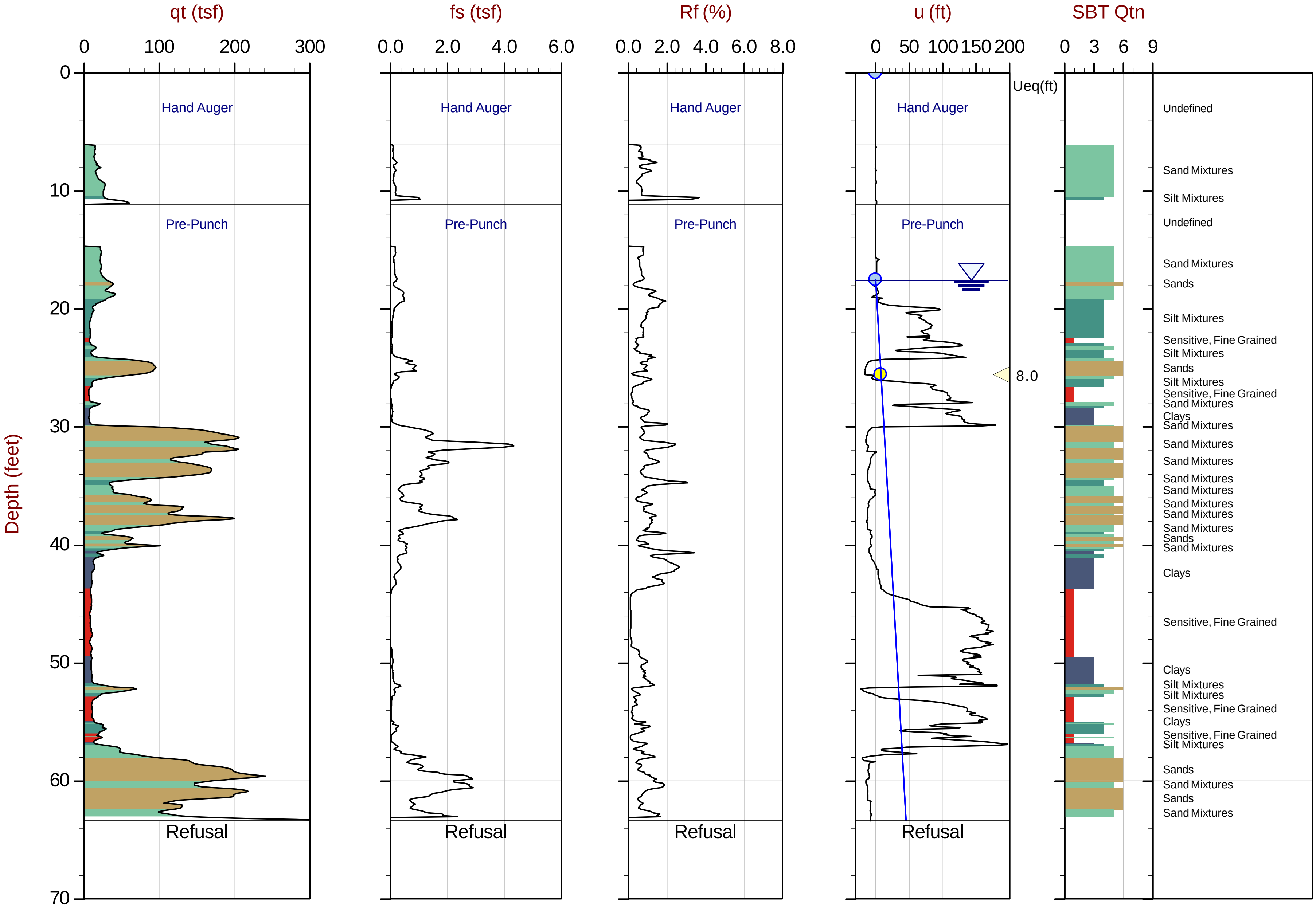
● Equilibrium Pore Pressure (Ueq) ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved — Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Delve Underground

Job No: 23-56-25414
Date: 2023-02-21 11:57
Site: FORTAG Phase 1 Canyon Del Rey SR218 Segment
Sounding: CPT-1B
Cone: 795:T1500F15U35



Max Depth: 19.325 m / 63.40 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-56-25414_CP03B.COR
Unit Wt: SBTQtn (PKR2009)

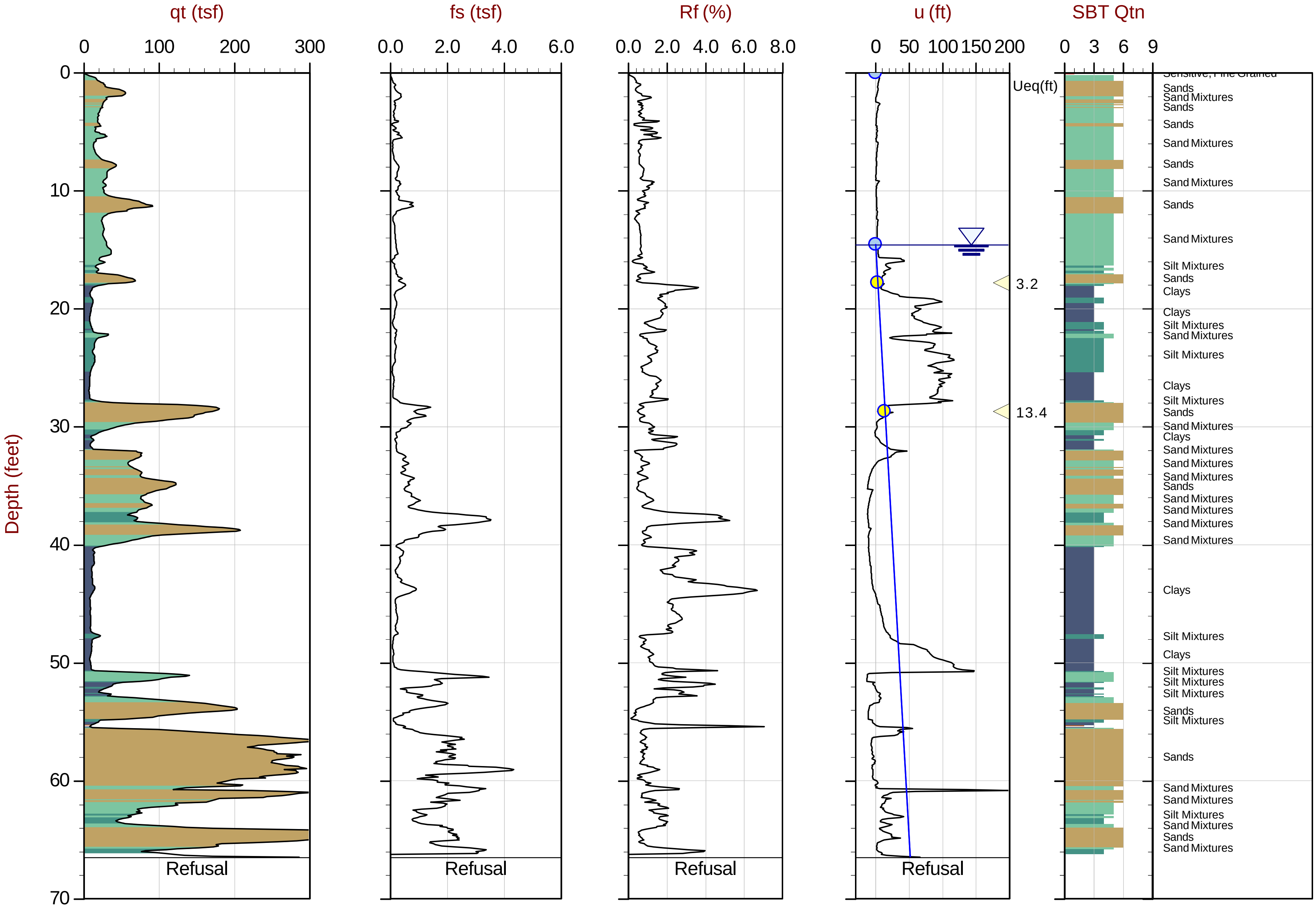
SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4050446m E: 604097m

● Equilibrium Pore Pressure (Ueq) ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved — Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Delve Underground

Job No: 23-56-25414
Date: 2023-02-21 07:18
Site: FORTAG Phase 1 Canyon Del Rey SR218 Segment
Sounding: CPT-2
Cone: 795:T1500F15U35



Max Depth: 20.275 m / 66.52 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-56-25414_CP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4050429m E: 604100m

● Equilibrium Pore Pressure (Ueq) ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved — Hydrostatic Line

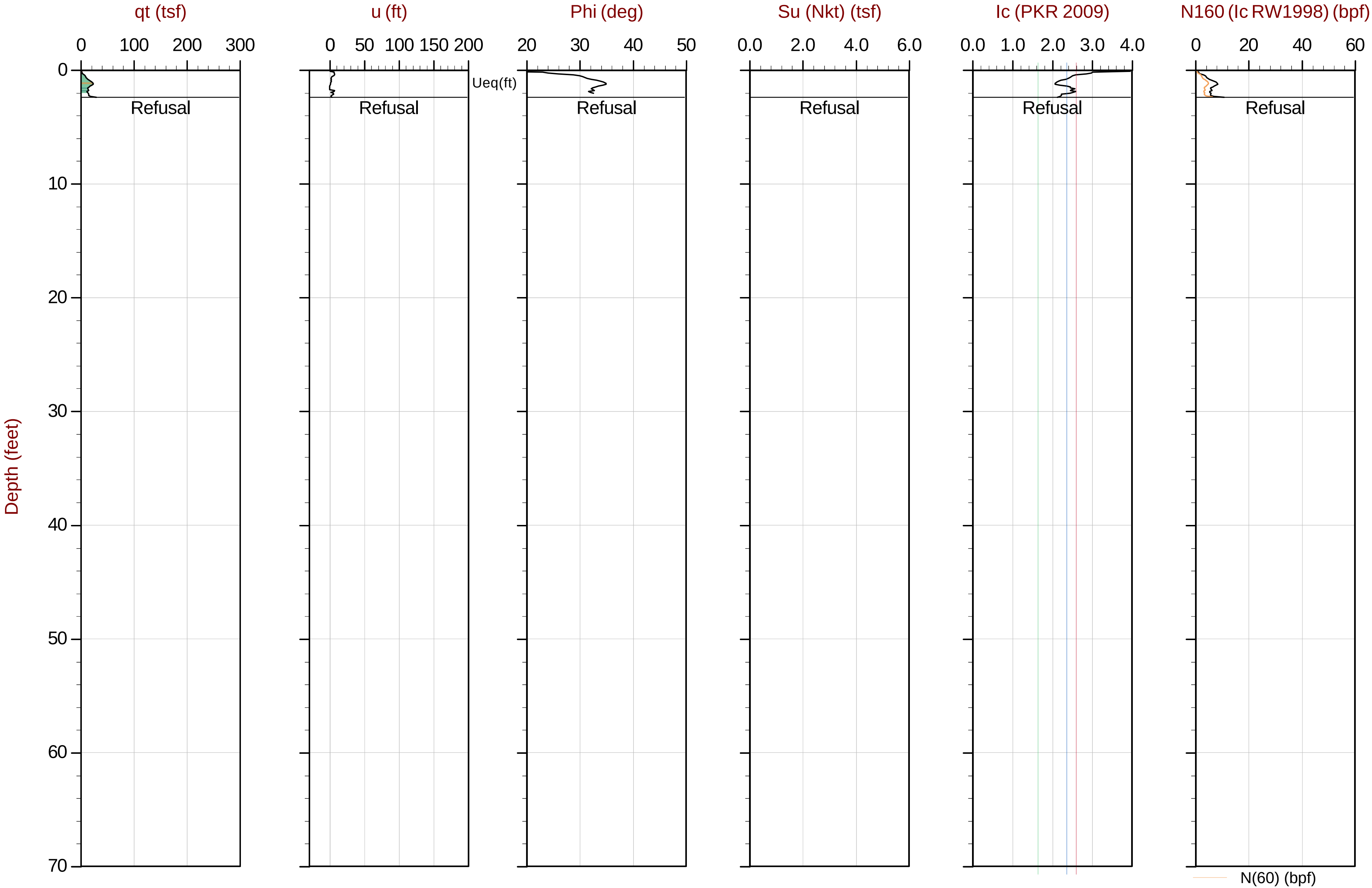
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Advanced Cone Penetration Test Plots

CONETEC

Delve Underground

Job No: 23-56-25414
Date: 2023-02-21 09:10
Site: FORTAG Phase 1 Canyon Del Rey SR218 Segment
Sounding: CPT- 1A
Cone: 795:T1500F15U35



Max Depth: 0.725 m / 2.38 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-56-25414_CP03.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4050435m E: 604098m

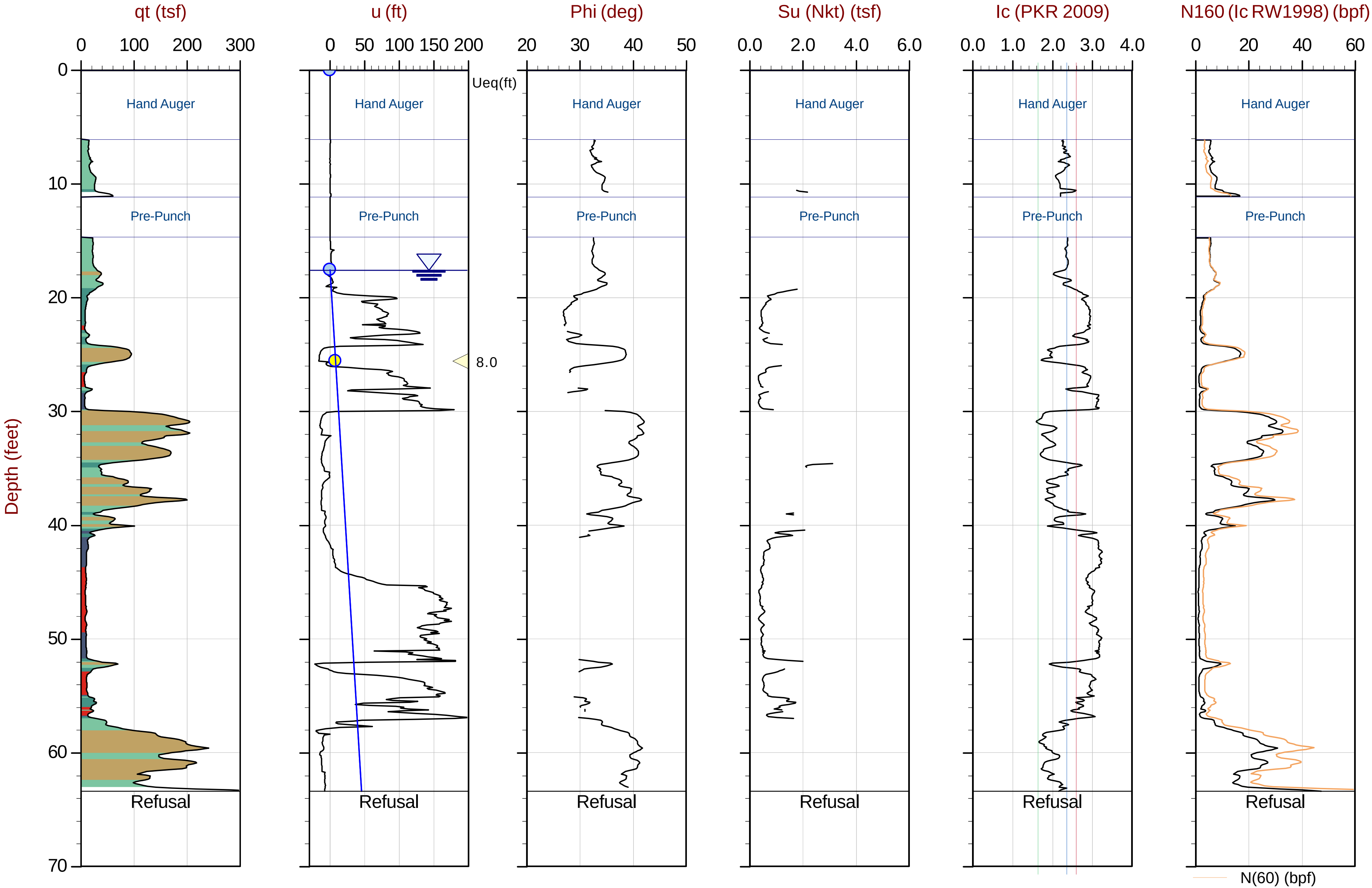
● Equilibrium Pore Pressure (Ueq) ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved — Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Delve Underground

Job No: 23-56-25414
Date: 2023-02-21 11:57
Site: FORTAG Phase 1 Canyon Del Rey SR218 Segment

Sounding: CPT-1B
Cone: 795:T1500F15U35



Max Depth: 19.325 m / 63.40 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-56-25414_CP03B.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4050446m E: 604097m

● Equilibrium Pore Pressure (Ueq) ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved — Hydrostatic Line

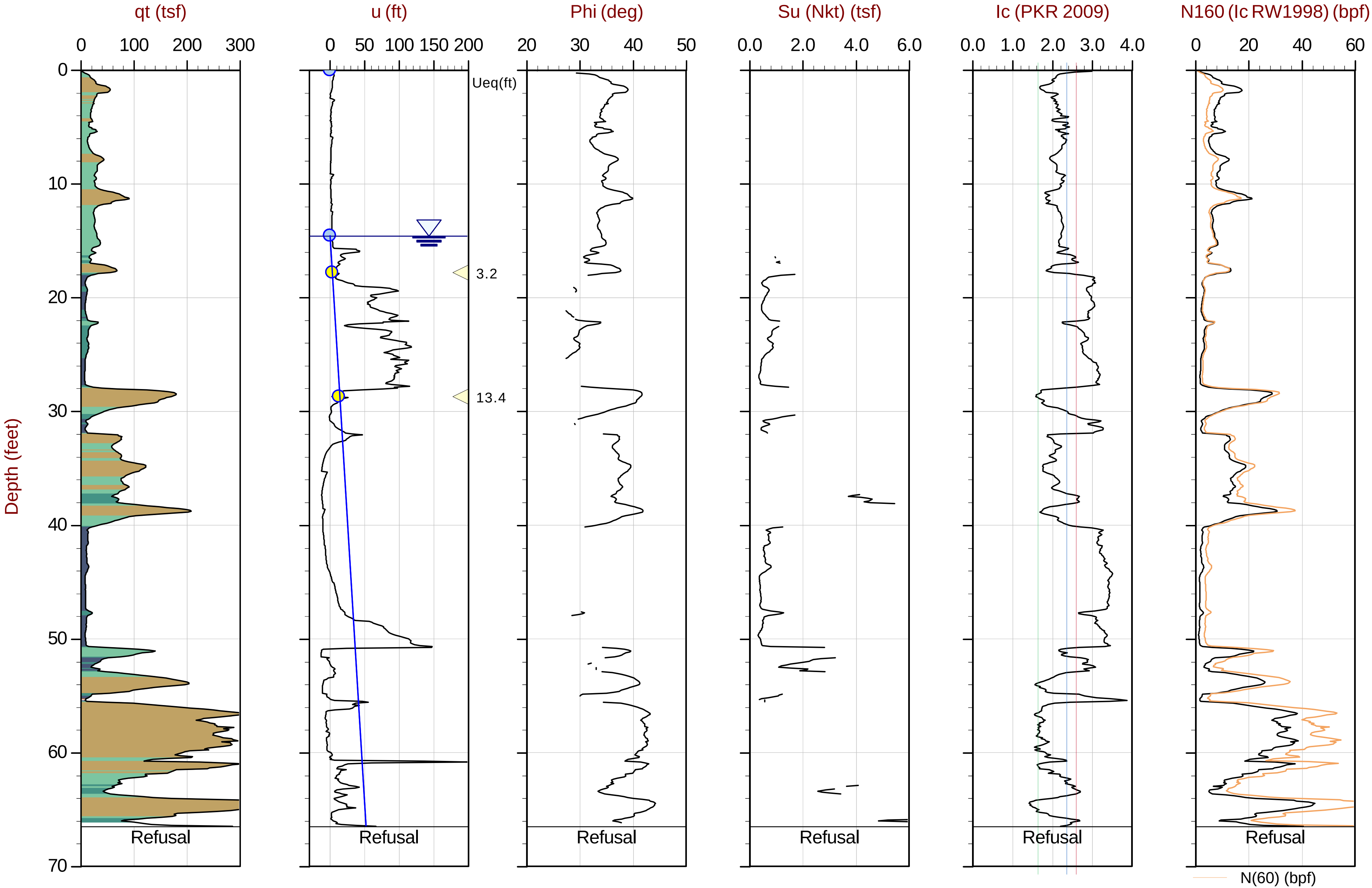
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Delve Underground

Job No: 23-56-25414
Date: 2023-02-21 07:18
Site: FORTAG Phase 1 Canyon Del Rey SR218 Segment

Sounding: CPT-2
Cone: 795:T1500F15U35



Max Depth: 20.275 m / 66.52 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-56-25414_CP04.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4050429m E: 604100m

● Equilibrium Pore Pressure (Ueq) ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved — Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Soil Behavior Type (SBT) Scatter Plots

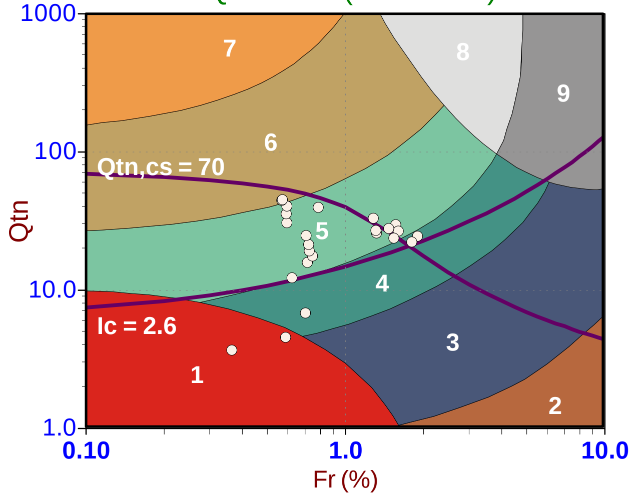
CONETEC

Delve Underground

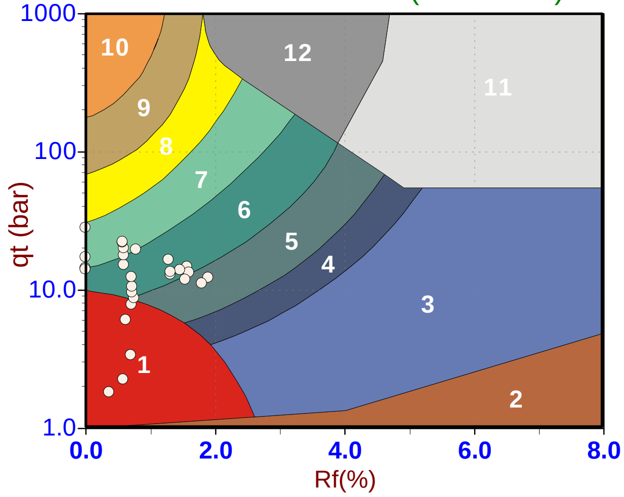
Job No: 23-56-25414
Date: 2023-02-21 09:10
Site: FORTAG Phase 1 Canyon Del Rey SR218 Segment

Sounding: CPT-1A
Cone: 795:T1500F15U35

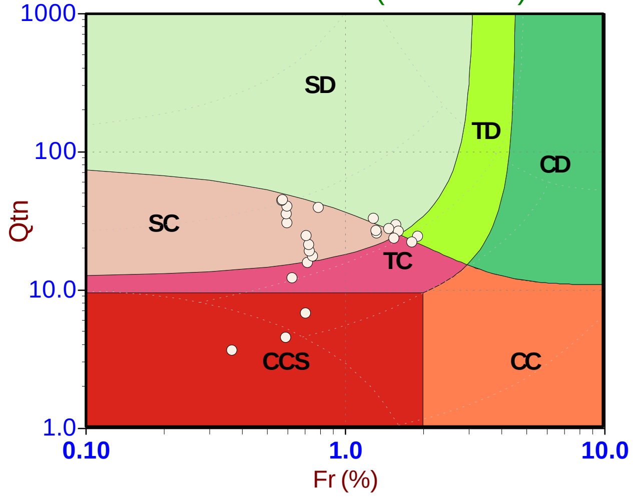
Qtn Chart (PKR 2009)



Standard SBT Chart (UBC 1986)



Modified SBTn (PKR 2016)



Depth Ranges

- ○ >0.0 to 7.5 ft
- ● >7.5 to 15.0 ft
- ● >15.0 to 22.5 ft
- ● >22.5 to 30.0 ft
- ● >30.0 to 37.5 ft
- ● >37.5 to 45.0 ft
- ● >45.0 to 52.5 ft
- ● >52.5 to 60.0 ft
- ● >60.0 to 67.5 ft
- ● >67.5 to 75.0 ft
- ● >75.0 ft

Legend

- 🟥 Sensitive, Fine Grained
- 🟫 Organic Soils
- 🟦 Clays
- 🟩 Silt Mixtures
- 🟩 Sand Mixtures
- 🟫 Sands
- 🟧 Gravelly Sand to Sand
- ⬜ Stiff Sand to Clayey Sand
- ⬛ Very Stiff Fine Grained

Legend

- 🟥 Sensitive Fines
- 🟫 Organic Soil
- 🟦 Clay
- 🟦 Silty Clay
- ⬛ Clayey Silt
- 🟩 Silt
- 🟩 Sandy Silt
- 🟨 Silty Sand/Sand
- 🟫 Sand
- 🟧 Gravelly Sand
- ⬜ Stiff Fine Grained
- ⬛ Cemented Sand

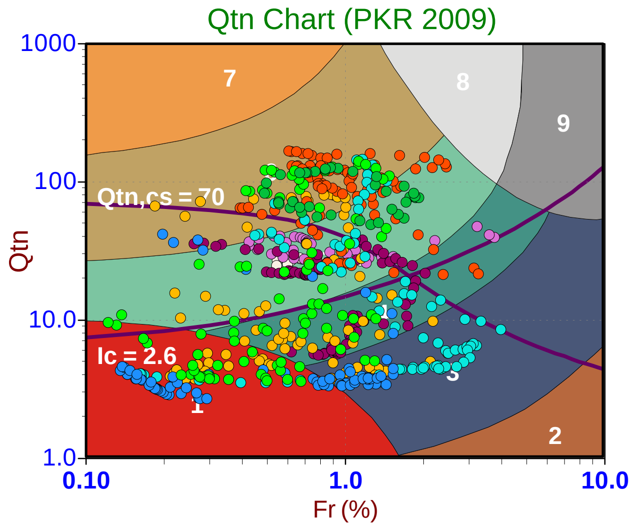
Legend

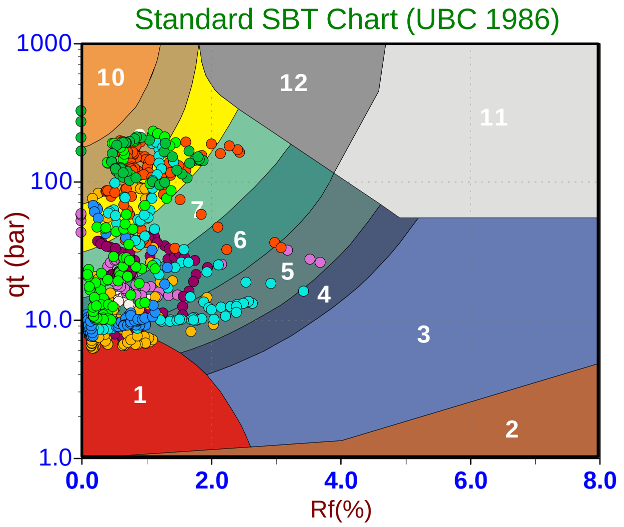
- 🟥 CCS (Cont. sensitive clay like)
- 🟧 CC (Cont. clay like)
- 🟥 TC (Cont. transitional)
- 🟫 SC (Cont. sand like)
- 🟩 CD (Dil. clay like)
- 🟩 TD (Dil. transitional)
- 🟩 SD (Dil. sand like)

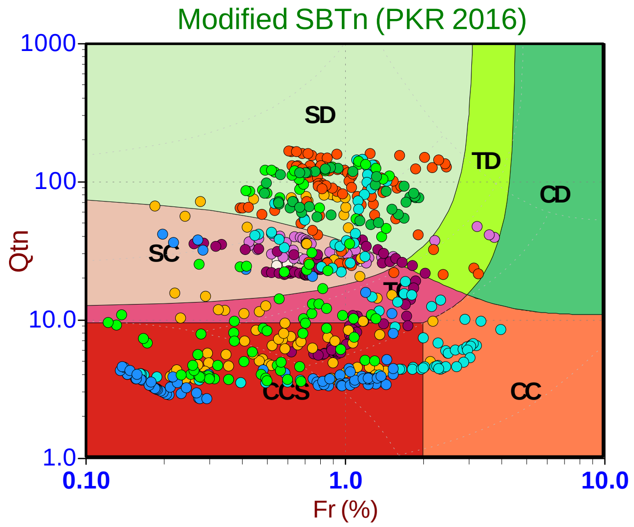
CONETEC

Delve Underground

Job No: 23-56-25414
Date: 2023-02-21 11:57
Site: FORTAG Phase 1 Canyon Del Rey SR218 Segment
Sounding: CPT-1B
Cone: 795:T1500F15U35


Qtn Chart (PKR 2009)


Standard SBT Chart (UBC 1986)


Modified SBTn (PKR 2016)

Depth Ranges
- ○ >0.0 to 7.5 ft
- ● >7.5 to 15.0 ft
- ● >15.0 to 22.5 ft
- ● >22.5 to 30.0 ft
- ● >30.0 to 37.5 ft
- ● >37.5 to 45.0 ft
- ● >45.0 to 52.5 ft
- ● >52.5 to 60.0 ft
- ● >60.0 to 67.5 ft
- ● >67.5 to 75.0 ft
- ● >75.0 ft

Legend
- 🟥 Sensitive, Fine Grained
- 🟫 Organic Soils
- ■ Clays
- ■ Silt Mixtures
- ■ Sand Mixtures
- ■ Sands
- 🟧 Gravelly Sand to Sand
- ■ Stiff Sand to Clayey Sand
- ■ Very Stiff Fine Grained

Legend
- 🟥 Sensitive Fines
- 🟫 Organic Soil
- 🟦 Clay
- ■ Silty Clay
- ■ Clayey Silt
- ■ Silt
- ■ Sandy Silt
- 🟨 Silty Sand/Sand
- ■ Sand
- 🟧 Gravelly Sand
- ■ Stiff Fine Grained
- ■ Cemented Sand

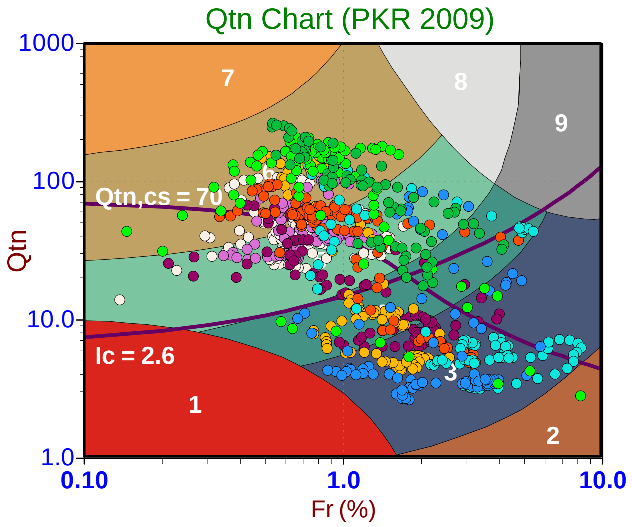
Legend
- 🟥 CCS (Cont. sensitive clay like)
- 🟧 CC (Cont. clay like)
- ■ TC (Cont. transitional)
- ■ SC (Cont. sand like)
- 🟩 CD (Dil. clay like)
- 🟩 TD (Dil. transitional)
- ■ SD (Dil. sand like)

CONETEC

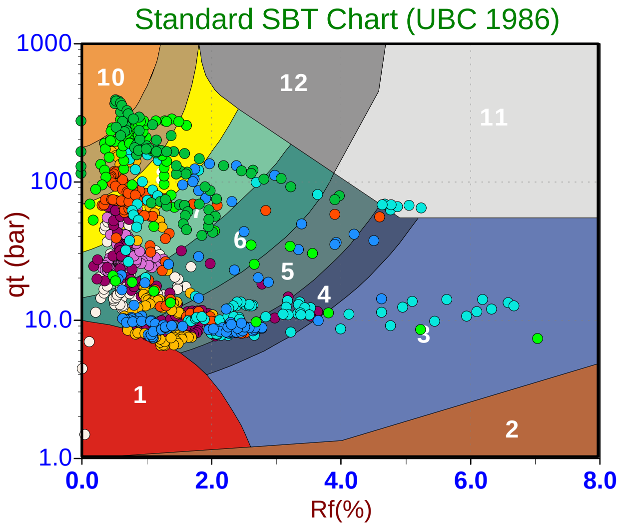
Delve Underground

Job No: 23-56-25414
Date: 2023-02-21 07:18
Site: FORTAG Phase 1 Canyon Del Rey SR218 Segment

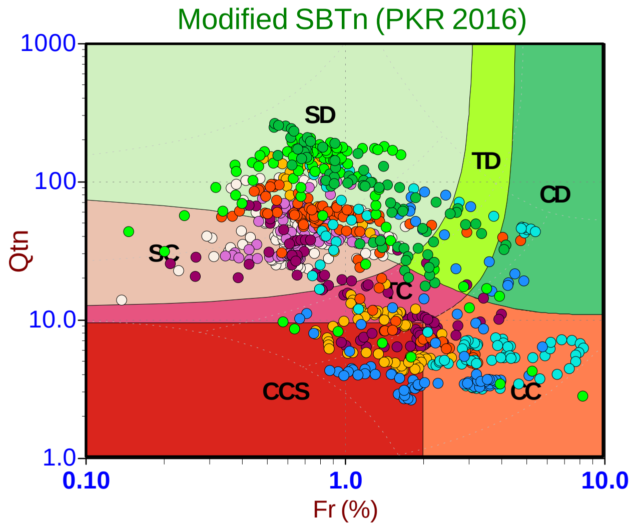
Sounding: CPT-2
Cone: 795:T1500F15U35



Qtn Chart (PKR 2009)



Standard SBT Chart (UBC 1986)



Modified SBTn (PKR 2016)

Depth Ranges

- ● >0.0 to 7.5 ft
- ● >7.5 to 15.0 ft
- ● >15.0 to 22.5 ft
- ● >22.5 to 30.0 ft
- ● >30.0 to 37.5 ft
- ● >37.5 to 45.0 ft
- ● >45.0 to 52.5 ft
- ● >52.5 to 60.0 ft
- ● >60.0 to 67.5 ft
- ● >67.5 to 75.0 ft
- ● >75.0 ft

Legend

- ■ Sensitive, Fine Grained
- ■ Organic Soils
- ■ Clays
- ■ Silt Mixtures
- ■ Sand Mixtures
- ■ Sands
- ■ Gravelly Sand to Sand
- ■ Stiff Sand to Clayey Sand
- ■ Very Stiff Fine Grained

Legend

- ■ Sensitive Fines
- ■ Organic Soil
- ■ Clay
- ■ Silty Clay
- ■ Clayey Silt
- ■ Silt
- ■ Sandy Silt
- ■ Silty Sand/Sand
- ■ Sand
- ■ Gravelly Sand
- ■ Stiff Fine Grained
- ■ Cemented Sand

Legend

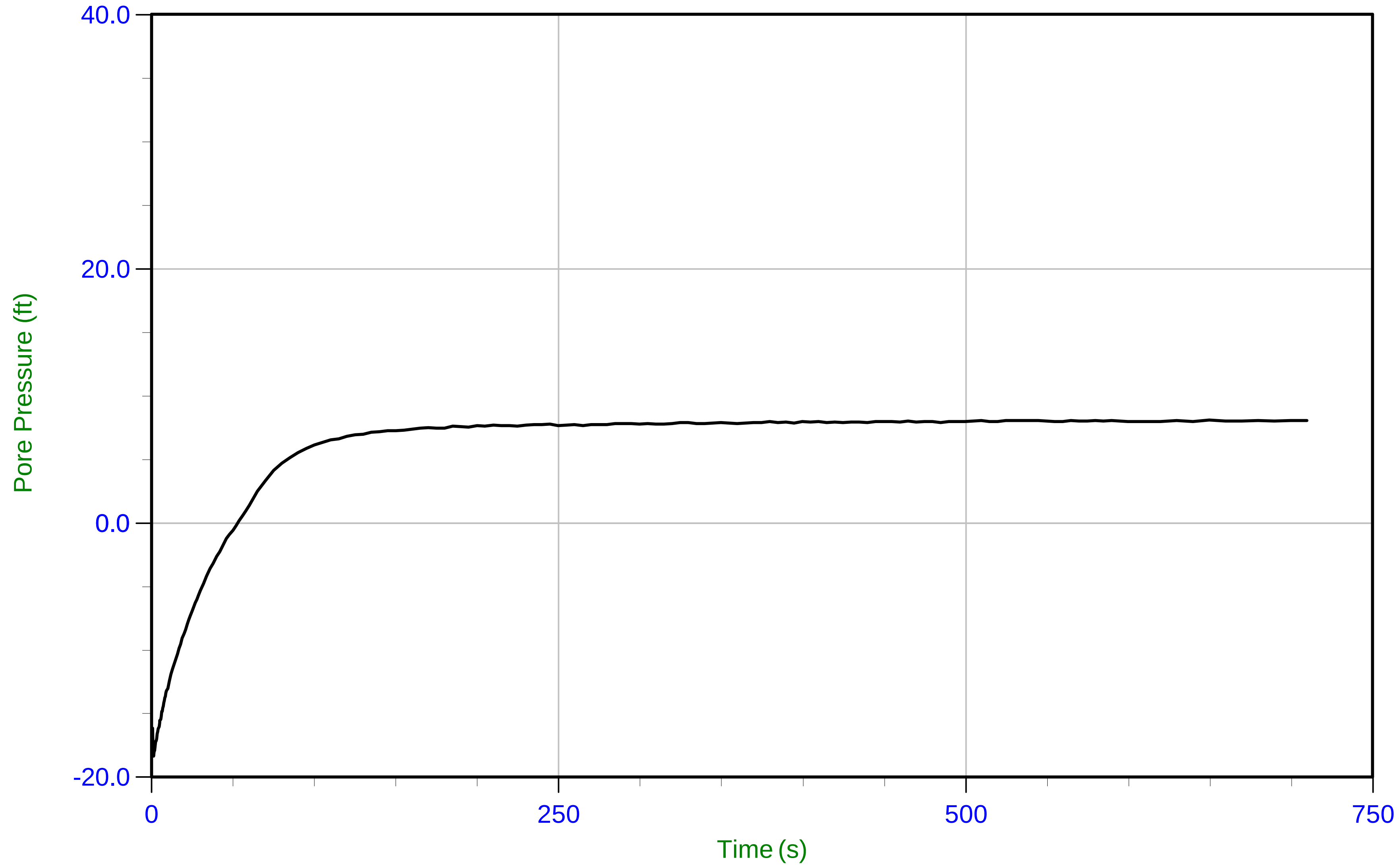
- ■ CCS (Cont. sensitive clay like)
- ■ CC (Cont. clay like)
- ■ TC (Cont. transitional)
- ■ SC (Cont. sand like)
- ■ CD (Dil. clay like)
- ■ TD (Dil. transitional)
- ■ SD (Dil. sand like)

Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

Job No: 23-56-25414
Client: Delve Underground
Project: FORTAG Phase 1 Canyon Del Rey SR218 Segment
Start Date: 21-Feb-2023
End Date: 21-Feb-2023

CPTu PORE PRESSURE DISSIPATION SUMMARY

Sounding ID	File Name	Cone Area (cm^2)	Duration (s)	Test Depth (ft)	Estimated Equilibrium Pore Pressure U_{eq} (ft)	Calculated Phreatic Surface (ft)
CPT-1B	23-56-25414_CP03B	15	710	25.59	8.0	17.6
CPT-2	23-56-25414_CP04	15	435	17.80	3.2	14.6
CPT-2	23-56-25414_CP04	15	475	28.71	13.4	15.3



Trace Summary:

Filename: 23-56-25414_CP03B.ppf2
Depth: 7.800 m / 25.590 ft
Duration: 709.9 s

u Min: -18.3 ft
u Max: 8.1 ft
u Final: 8.0 ft

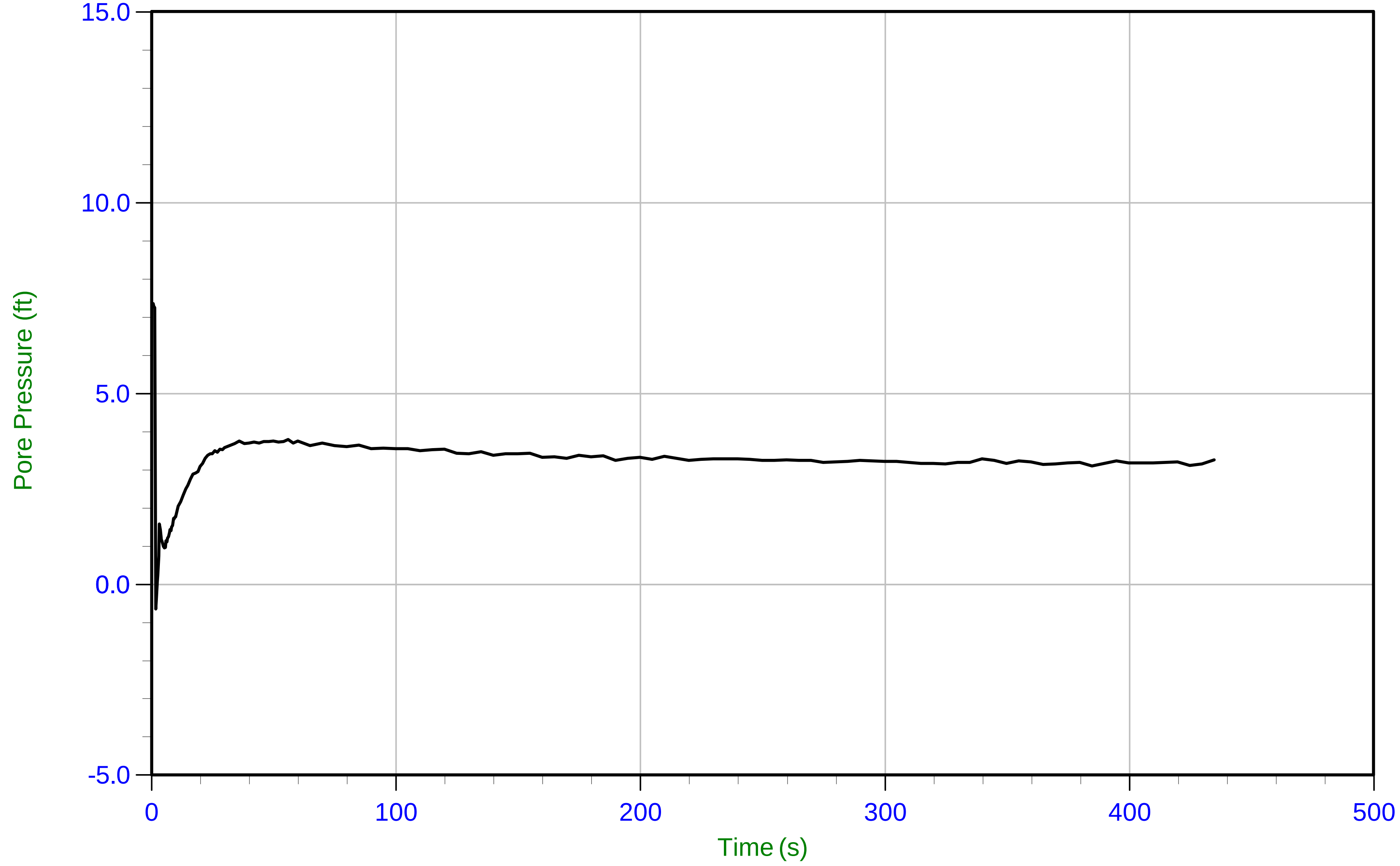
WT: 5.354 m / 17.567 ft
Ueq: 8.0 ft

CONETEC

Delve Underground

Job No: 23-56-25414
Date: 02/21/2023 07:18
Site: FORTAG Phase 1 Canyon Del Rey SR218 Segment

Sounding: CPT-2
Cone: 795:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 23-56-25414_CP04.ppf2
Depth: 5.425 m / 17.798 ft
Duration: 434.9 s

u Min: -0.6 ft
u Max: 7.4 ft
u Final: 3.3 ft

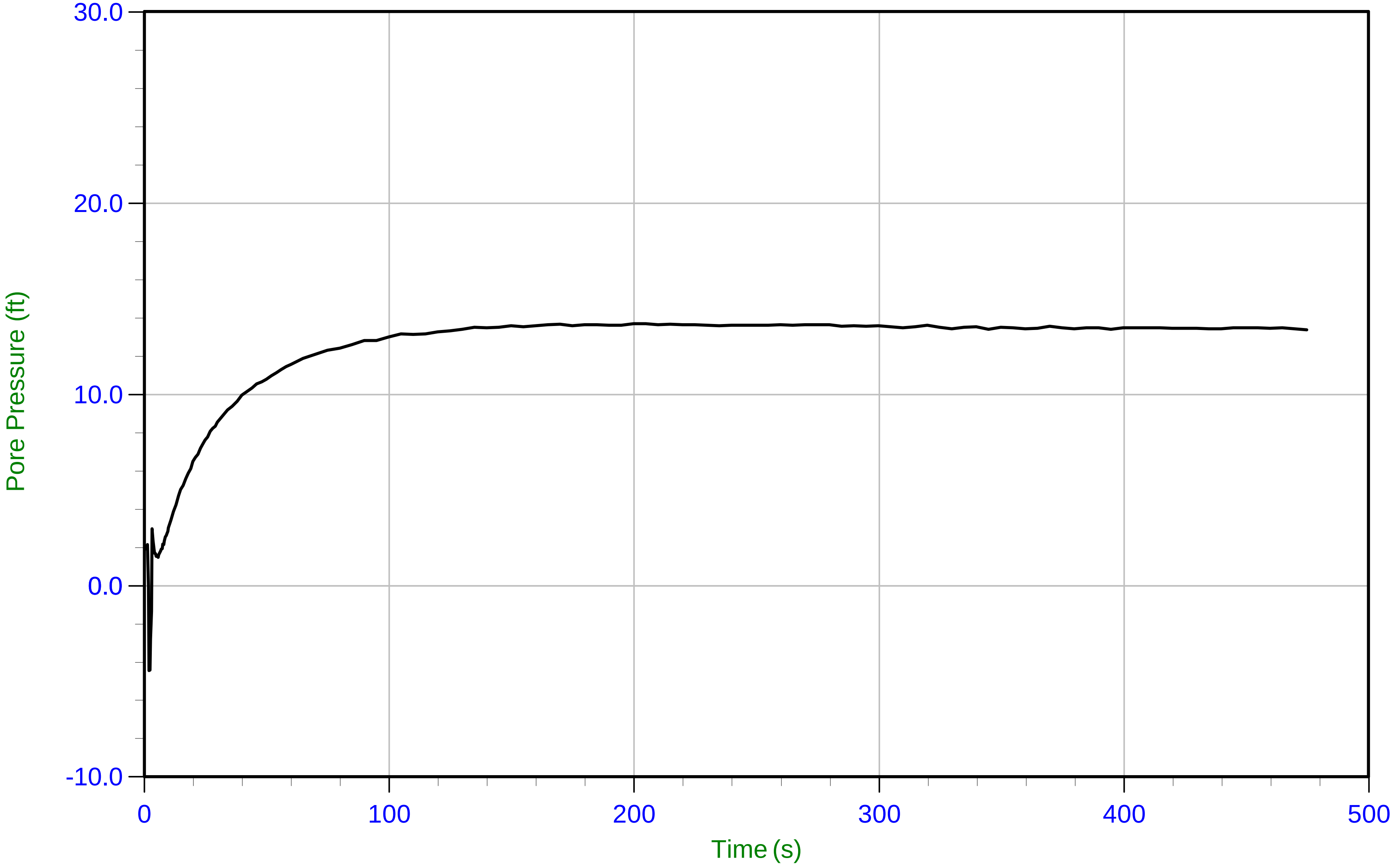
WT: 4.441 m / 14.570 ft
Ueq: 3.2 ft

CONETEC

Delve Underground

Job No: 23-56-25414
Date: 02/21/2023 07:18
Site: FORTAG Phase 1 Canyon Del Rey SR218 Segment

Sounding: CPT-2
Cone: 795:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 23-56-25414_CP04.ppf2
Depth: 8.750 m / 28.707 ft
Duration: 474.9 s

u Min: -4.4 ft
u Max: 13.7 ft
u Final: 13.4 ft

WT: 4.662 m / 15.296 ft
Ueq: 13.4 ft

Methodology Statements and Data File Formats

METHODOLOGY STATEMENTS

CONE PENETRATION TEST (CPTu) - eSeries

Cone penetration tests (CPTu) are conducted using an integrated electronic piezocone penetrometer and data acquisition system manufactured by Adara Systems Ltd., a subsidiary of ConeTec.

ConeTec's piezocone penetrometers are compression type designs in which the tip and friction sleeve load cells are independent and have separate load capacities. The piezocones use strain gauged load cells for tip and sleeve friction and a strain gauged diaphragm type transducer for recording pore pressure. The piezocones also have a platinum resistive temperature device (RTD) for monitoring the temperature of the sensors, an accelerometer type dual axis inclinometer and two geophone sensors for recording seismic signals. All signals are amplified and measured with minimum sixteen-bit resolution down hole within the cone body, and the signals are sent to the surface using a high bandwidth, error corrected digital interface through a shielded cable.

ConeTec penetrometers are manufactured with various tip, friction and pore pressure capacities in both 10 cm^2 and 15 cm^2 tip base area configurations in order to maximize signal resolution for various soil conditions. The specific piezocone used for each test is described in the CPT summary table. The 15 cm^2 penetrometers do not require friction reducers as they have a diameter larger than the deployment rods. The 10 cm^2 piezocones use a friction reducer consisting of a rod adapter extension behind the main cone body with an enlarged cross sectional area (typically 44 millimeters diameter over a length of 32 millimeters with tapered leading and trailing edges) located at a distance of 585 millimeters above the cone tip.

The penetrometers are designed with equal end area friction sleeves, a net end area ratio of 0.8 and cone tips with a 60 degree apex angle.

All ConeTec piezocones can record pore pressure at various locations. Unless otherwise noted, the pore pressure filter is located directly behind the cone tip in the "u_2" position (ASTM Type 2). The filter is six millimeters thick, made of porous plastic (polyethylene) having an average pore size of 125 microns (90-160 microns). The function of the filter is to allow rapid movements of extremely small volumes of water needed to activate the pressure transducer while preventing soil ingress or blockage.

The piezocone penetrometers are manufactured with dimensions, tolerances and sensor characteristics that are in general accordance with the current ASTM D5778 standard. ConeTec's calibration criteria also meets or exceeds those of the current ASTM D5778 standard. An illustration of the piezocone penetrometer is presented in Figure CPTu.

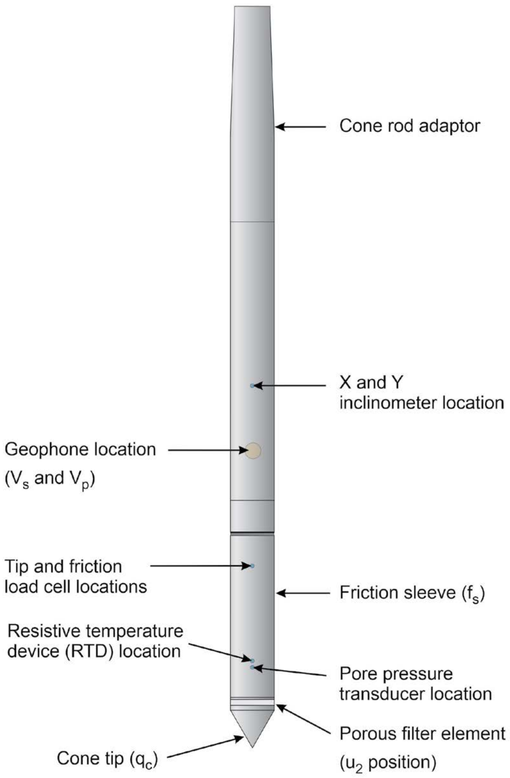


Figure CPTu. Piezocone Penetrometer (15 cm^2)

The ConeTec data acquisition system consists of a Windows based computer, signal interface box, and power supply. The signal interface combines depth increment signals, seismic trigger signals and the downhole digital data. This combined data is then sent to the Windows based computer for collection and presentation. The data is recorded at fixed depth increments using a depth encoder that is either portable or integrated into the rig. The typical recording interval is 2.5 centimeters; custom recording intervals are possible.

The system displays the CPTu data in real time and records the following parameters to a storage media during penetration:

- Depth
- Uncorrected tip resistance (q_c)
- Sleeve friction (f_s)
- Dynamic pore pressure (u)
- Additional sensors such as resistivity, passive gamma, ultra violet induced fluorescence, if applicable

All testing is performed in accordance to ConeTec's CPTu operating procedures which are in general accordance with the current ASTM D5778 standard.

Prior to the start of a CPTu sounding a suitable cone is selected, the cone and data acquisition system are powered on, the pore pressure system is saturated with silicone oil and the baseline readings are recorded with the cone hanging freely in a vertical position.

The CPTu is conducted at a steady rate of two centimeters per second, within acceptable tolerances. Typically one meter length rods with an outer diameter of 1.5 inches are added to advance the cone to the sounding termination depth. After cone retraction final baselines are recorded.

Additional information pertaining to ConeTec's cone penetration testing procedures:

- Each filter is saturated in silicone oil under vacuum pressure prior to use
- Baseline readings are compared to previous readings
- Soundings are terminated at the client's target depth or at a depth where an obstruction is encountered, excessive rod flex occurs, excessive inclination occurs, equipment damage is likely to take place, or a dangerous working environment arises
- Differences between initial and final baselines are calculated to ensure zero load offsets have not occurred and to ensure compliance with ASTM standards

The interpretation of piezocone data for this report is based on the corrected tip resistance (q_t), sleeve friction (f_s) and pore water pressure (u). The interpretation of soil type is based on the correlations developed by Robertson, P.K., 2010. The Soil Behavior Type (SBT) classification chart developed by Robertson, P.K., 2010 is presented in Figure SBT. It should be noted that it is not always possible to accurately identify a soil behavior type based on these parameters. In these situations, experience, judgment and an assessment of other parameters may be used to infer soil behavior type.

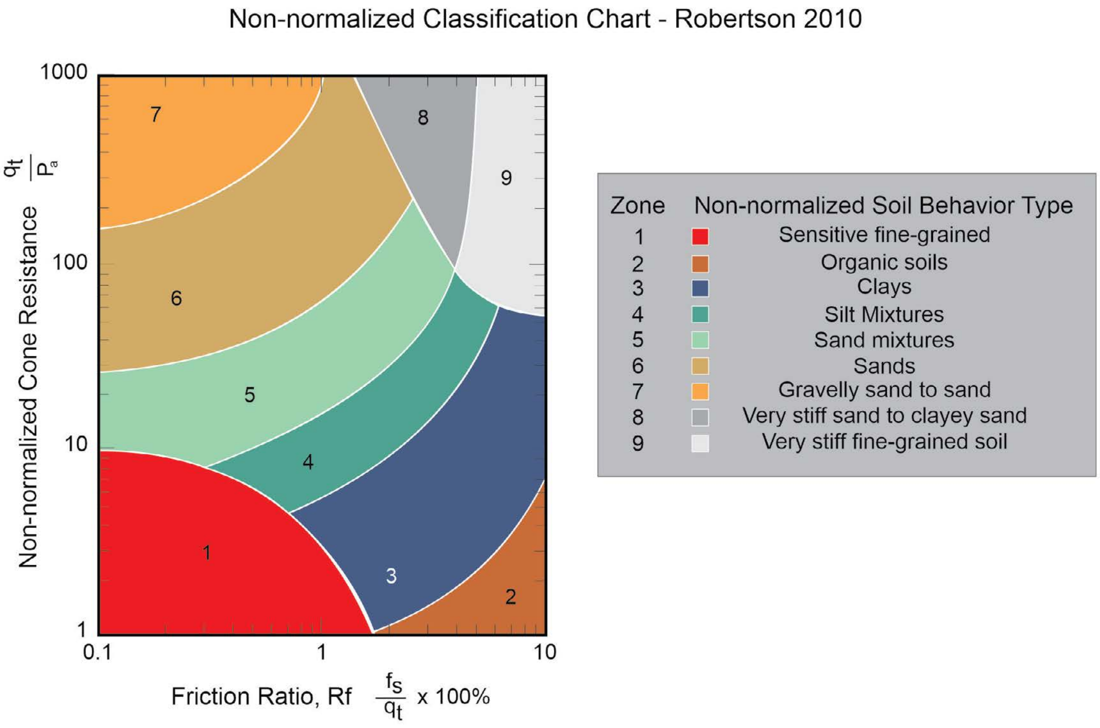


Figure SBT. Non-Normalized Soil Behavior Type Classification Chart (SBT)

The recorded tip resistance (q_c) is the total force acting on the piezocone tip divided by its base area. The tip resistance is corrected for pore pressure effects and termed corrected tip resistance (q_t) according to the following expression presented in Robertson et al. (1986):

$$q_t = q_c + (1\text{-}a) \bullet u_2$$

where: q_t is the corrected tip resistance

q_c is the recorded tip resistance

u_2 is the recorded dynamic pore pressure behind the tip (u_2 position)

a is the Net Area Ratio for the piezocone (0.8 for ConeTec probes)

The sleeve friction (f_s) is the frictional force on the sleeve divided by its surface area. As all ConeTec piezocones have equal end area friction sleeves, pore pressure corrections to the sleeve data are not required.

The dynamic pore pressure (u) is a measure of the pore pressures generated during cone penetration. To record equilibrium pore pressure, the penetration must be stopped to allow the dynamic pore pressures to stabilize. The rate at which this occurs is predominantly a function of the permeability of the soil and the diameter of the cone.

The friction ratio (Rf) is a calculated parameter. It is defined as the ratio of sleeve friction to the tip resistance expressed as a percentage. Generally, saturated cohesive soils have low tip resistance, high friction ratios and generate large excess pore water pressures. Cohesionless soils have higher tip resistances, lower friction ratios and do not generate significant excess pore water pressure.

For additional information on CPTu interpretations and calculated geotechnical parameters, refer to Robertson et al. (1986), Lunne et al. (1997), Robertson (2009), Mayne (2013, 2014) and Mayne and Peuchen (2012).

REFERENCES

ASTM D5778-20, 2020, “Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils”, ASTM International, West Conshohocken, PA. DOI: 10.1520/D5778-20.

Lunne, T., Robertson, P.K. and Powell, J. J. M., 1997, “Cone Penetration Testing in Geotechnical Practice”, Blackie Academic and Professional.

Mayne, P.W., 2013, “Evaluating yield stress of soils from laboratory consolidation and in-situ cone penetration tests”, Sound Geotechnical Research to Practice (Holtz Volume) GSP 230, ASCE, Reston/VA: 406-420. DOI: 10.1061/9780784412770.027.

Mayne, P.W. and Peuchen, J., 2012, “Unit weight trends with cone resistance in soft to firm clays”, Geotechnical and Geophysical Site Characterization 4, Vol. 1 (Proc. ISC-4, Pernambuco), CRC Press, London: 903-910.

Mayne, P.W., 2014, “Interpretation of geotechnical parameters from seismic piezocone tests”, CPT’14 Keynote Address, Las Vegas, NV, May 2014.

Robertson, P.K., Campanella, R.G., Gillespie, D. and Greig, J., 1986, “Use of Piezometer Cone Data”, Proceedings of InSitu 86, ASCE Specialty Conference, Blacksburg, Virginia.

Robertson, P.K., 2009, “Interpretation of cone penetration tests – a unified approach”, Canadian Geotechnical Journal, Volume 46: 1337-1355. DOI: 10.1139/T09-065.

Robertson, P.K., 2010. Soil behavior type from the CPT: an update. 2nd International Symposium on Cone Penetration Testing, CPT’10, Huntington Beach, CA, USA

PORE PRESSURE DISSIPATION TEST

The cone penetration test is halted at specific depths to carry out pore pressure dissipation (PPD) tests, shown in Figure PPD-1. For each dissipation test the cone and rods are decoupled from the rig and the data acquisition system measures and records the variation of the pore pressure (u) with time (t).

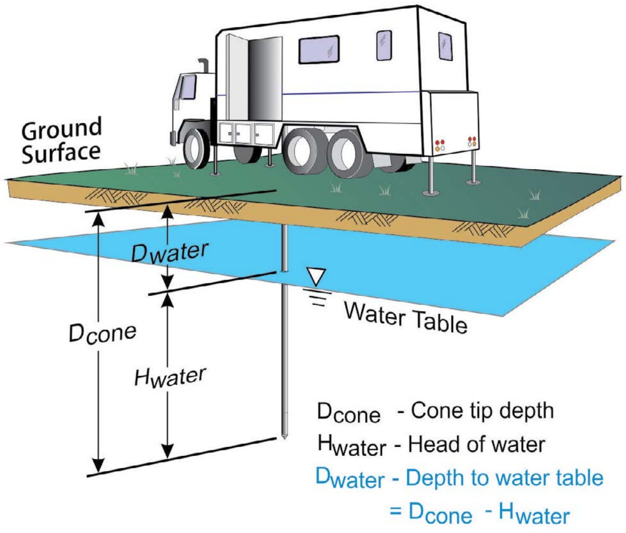


Figure PPD-1. Pore pressure dissipation test setup

Pore pressure dissipation data can be interpreted to provide estimates of ground water conditions, permeability, consolidation characteristics and soil behavior.

The typical shapes of dissipation curves shown in Figure PPD-2 are very useful in assessing soil type, drainage, in situ pore pressure and soil properties. A flat curve that stabilizes quickly is typical of a freely draining sand. Undrained soils such as clays will typically show positive excess pore pressure and have long dissipation times. Dilative soils will often exhibit dynamic pore pressures below equilibrium that then rise over time. Overconsolidated fine-grained soils will often exhibit an initial dilatory response where there is an initial rise in pore pressure before reaching a peak and dissipating.

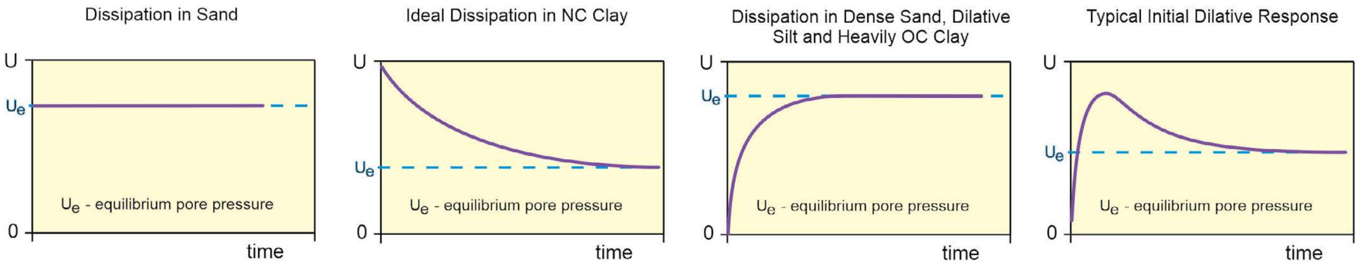


Figure PPD-2. Pore pressure dissipation curve examples

In order to interpret the equilibrium pore pressure (u_{eq}) and the apparent phreatic surface, the pore pressure should be monitored until such time as there is no variation in pore pressure with time as shown for each curve in Figure PPD-2.

CONE PENETRATION DIGITAL FILE FORMATS - eSeries

CPT Data Files (COR Extension)

ConeTec CPT data files are stored in ASCII text files that are readable by almost any text editor. ConeTec file names start with the job number (which includes the two digit year number) an underscore as a separating character, followed by two letters based on the type of test and the sounding ID. The last character position is reserved for an identifier letter (such as b, c, d etc) used to uniquely distinguish multiple soundings at the same location. The CPT sounding file has the extension COR. As an example, for job number 21-02-00001 the first CPT sounding will have file name 21-02-00001_CP01.COR

The sounding (COR) file consists of the following components:

1. Two lines of header information
2. Data records
3. End of data marker
4. Units information

Header Lines

Line 1: Columns 1-6 may be blank or may indicate the version number of the recording software
Columns 7-21 contain the sounding Date and Time (Date is MM:DD:YY)
Columns 23-38 contain the sounding Operator
Columns 51-100 contain extended Job Location information

Line 2: Columns 1-16 contain the Job Location
Columns 17-32 contain the Cone ID
Columns 33-47 contain the sounding number
Columns 51-100 may contain extended sounding ID information

Data Records

The data records contain 4 or more columns of data in floating point format. A comma and spaces separate each data item:

Column 1: Sounding Depth (meters)
Column 2: Tip (q_c), recorded in units selected by the operator
Column 3: Sleeve (f_s), recorded in units selected by the operator
Column 4: Dynamic pore pressure (u), recorded in units selected by the operator
Column 5: Empty or may contain other requested data such as Gamma, Resistivity or UVIF data

End of Data Marker

After the last line of data there is a line containing an ASCII 26 (CTL-Z) character (small rectangular shaped character) followed by a newline (carriage return / line feed). This is used to mark the end of data.

Units Information

The last section of the file contains information about the units that were selected for the sounding. A separator bar makes up the first line. The second line contains the type of units used for depth, q_c, f_s and u. The third line contains the conversion values required for ConeTec's software to convert the recorded data to an internal set of base units (bar for q_c, bar for f_s and meters for u). Additional lines intended for internal ConeTec use may appear following the conversion values.

CPT Data Files (XLS Extension)

Excel format files of ConeTec CPT data are also generated from corresponding COR files. The XLS files have the same base file name as the COR file with a -BSC suffix. The information in the file is presented in table format and contains additional information about the sounding such as coordinate information, and tip net area ratio.

The BSCI suffix is given to XLS files which are enhanced versions of the BSC files and include the same data records in addition to inclination data collected for each sounding.

CPT Dissipation Files (XLS Extension)

Pore pressure dissipation files are provided in Excel format and contain each dissipation trace that exceeds a minimum duration (selected during post-processing) formatted column wise within the spreadsheet. The first column (Column A) contains the time in seconds and the second column (Column B) contains the time in minutes. Subsequent columns contain the dissipation trace data. The columns extend to the longest trace of the data set.

Detailed header information is provided at the top of the worksheet. The test depth in meters and feet, the number of points in the trace and the particular units are all presented at the top of each trace column.

CPT Dissipation files have the same naming convention as the CPT sounding files with a "–PPD" suffix.

Data Records

Each file will contain dissipation traces that exceed a minimum duration (selected during post-processing) in a particular column. The dissipation pore pressure values are typically recorded at varying time intervals throughout the trace; rapidly to start and increasing as the duration of the test lengthens. The test depth in meters and feet, the number of points in the trace and the trace number are identified at the top of each trace column.

Cone Type Designations

Cone ID	Cone Description	Tip Cross Sect. Area (cm^2)	Tip Capacity (bar)	Sleeve Area (cm^2)**	Sleeve Capacity (bar)	Pore Pressure Capacity (bar)
EC###	A15T1500F15U35	15	1500	225	15	35
EC###	A15T375F10U35	15	375	225	10	35
EC###	A10T1000F10U35	10	1000	150	10	35

refers to the Cone ID number

**Outer Cylindrical Area

Description of Methods for Calculated CPT Geotechnical Parameters

CALCULATED CPT GEOTECHNICAL PARAMETERS

A Detailed Description of the Methods Used in ConeTec's CPT Geotechnical Parameter Calculation and Plotting Software

Revision SZW-Rev 14

Revised November 26, 2019
Prepared by Jim Greig, M.A.Sc, P.Eng (BC)

Limitations

The geotechnical parameter output was prepared specifically for the site and project named in the accompanying report subject to objectives, site conditions and criteria provided to ConeTec by the client. The output may not be relied upon by any other party or for any other site without the express written permission of ConeTec Group (ConeTec) or any of its affiliates. For this project, ConeTec has provided site investigation services, prepared factual data reporting and produced geotechnical parameter calculations consistent with current best practices. No other warranty, expressed or implied, is made.

To understand the calculations that have been performed and to be able to reproduce the calculated parameters the user is directed to the basic descriptions for the methods in this document and the detailed descriptions and their associated limitations and appropriateness in the technical references cited for each parameter.

BETTER INFORMATION, **BETTER DECISIONS**

ConeTec's Calculated CPT Geotechnical Parameters as of November 26, 2019

ConeTec's CPT parameter calculation and plotting routine provides a tabular output of geotechnical parameters based on current published CPT correlations and is subject to change to reflect the current state of practice. Due to drainage conditions and the basic assumptions and limitations of the correlations, not all geotechnical parameters provided are considered applicable for all soil types. The results are presented only as a guide for geotechnical use and should be carefully examined for consideration in any geotechnical design. Reference to current literature is strongly recommended. ConeTec does not warranty the correctness or the applicability of any of the geotechnical parameters calculated by the program and does not assume liability for any use of the results in any design or review. For verification purposes we recommend that representative hand calculations be done for any parameter that is critical for design purposes. The end user of the parameter output should also be fully aware of the techniques and the limitations of any method used by the program. The purpose of this document is to inform the user as to which methods were used and to direct the end user to the appropriate technical papers and/or publications for further reference.

The geotechnical parameter output was prepared specifically for the site and project named in the accompanying report subject to objectives, site conditions and criteria provided to ConeTec by the client. The output may not be relied upon by any other party or for any other site without the express written permission of ConeTec Group (ConeTec) or any of its affiliates.

The CPT calculations are based on values of tip resistance, sleeve friction and pore pressures considered at each data point or averaged over a user specified layer thickness (e.g. 0.20 m). Note that q_t is the tip resistance corrected for pore pressure effects and q_c is the recorded tip resistance. The corrected tip resistance (corrected using u_2 pore pressure values) is used for all of the calculations. Since all ConeTec cones have equal end area friction sleeves pore pressure corrections to sleeve friction, f_s, are not required.

The tip correction is: $q_t = q_c + (1-a) \bullet u_2$ (consistent units are implied)

where: q_t is the corrected tip resistance
q_c is the recorded tip resistance
u_2 is the recorded dynamic pore pressure behind the tip (u_2 position)
a is the Net Area Ratio for the cone (typically 0.80 for ConeTec cones)

The total stress calculations are based on soil unit weight values that have been assigned to the Soil Behavior Type (SBT) zones, from a user defined unit weight profile, by using a single uniform value throughout the profile, through unit weight estimation techniques described in various technical papers or from a combination of these methods. The parameter output files indicate the method(s) used.

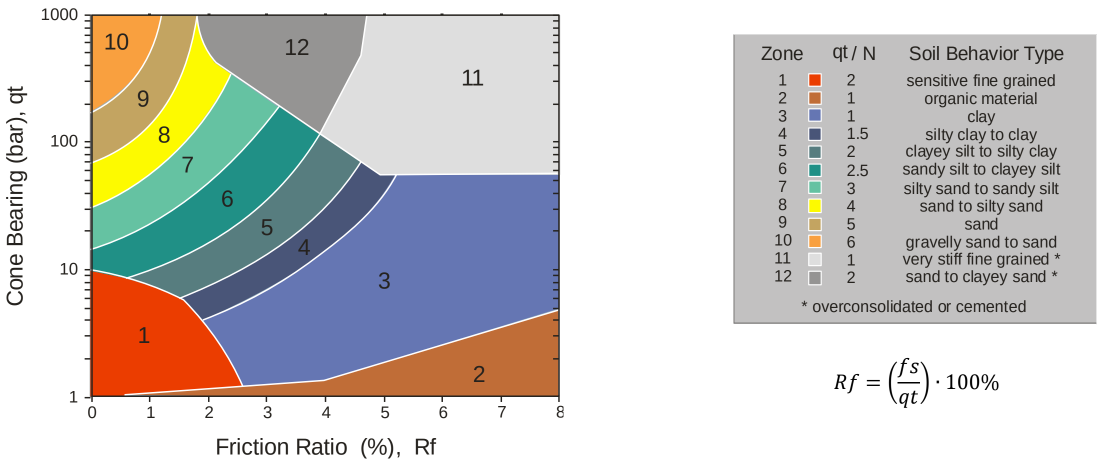
Effective vertical overburden stresses are calculated based on a hydrostatic distribution of equilibrium pore pressures below the water table or from a user defined equilibrium pore pressure profile (typically obtained from CPT dissipation tests) or a combination of the two. For over water projects the stress effects of the column of water above the mudline have been taken into account as has the appropriate unit weight of water. How this is done depends on where the instruments were zeroed (i.e. on deck or at the mudline). The parameter output files indicate the method(s) used.

A majority of parameter calculations are derived or driven by results based on material types as determined by the various soil behavior type charts depicted in Figures 1 through 5. The parameter output files indicate the method(s) used.

The Soil Behavior Type classification chart shown in Figure 1 is the classic non-normalized SBT Chart developed at the University of British Columbia and reported in Robertson, Campanella, Gillespie and Greig (1986). Figure 2 shows the original normalized (linear method) SBT chart developed by Robertson (1990). The Bq classification charts shown in Figures 3a and 3b incorporate pore pressures into the SBT classification and are based on the methods described in Robertson (1990). Many of these charts have been summarized in Lunne, Robertson and Powell (1997). The

Jefferies and Davies SBT chart shown in Figure 3c is based on the techniques discussed in Jefferies and Davies (1993) which introduced the concept of the Soil Behavior Type Index parameter, I_c. Please note that the I_c parameter developed by Robertson and Fear (1995) and Robertson and Wride (1998) is similar in concept but uses a slightly different calculation method than that used by Jefferies and Davies (1993) as the latter incorporates pore pressure in their technique through the use of the Bq parameter. The normalized Qtn SBT chart shown in Figure 4 is based on the work by Robertson (2009) utilizing a variable stress ratio exponent, n, for normalization based on a slightly modified redefinition and iterative approach for I_c. The boundary curves drawn on the chart are based on the work described in Robertson (2010).

Figure 5 shows a revised behavior based chart by Robertson (2016) depicting contractive-dilative zones. As the zones represent material behavior rather than soil gradation ConeTec has chosen a set of zone colors that are less likely to be confused with material type colors from previous SBT charts. These colors differ from those used by Dr. Robertson.

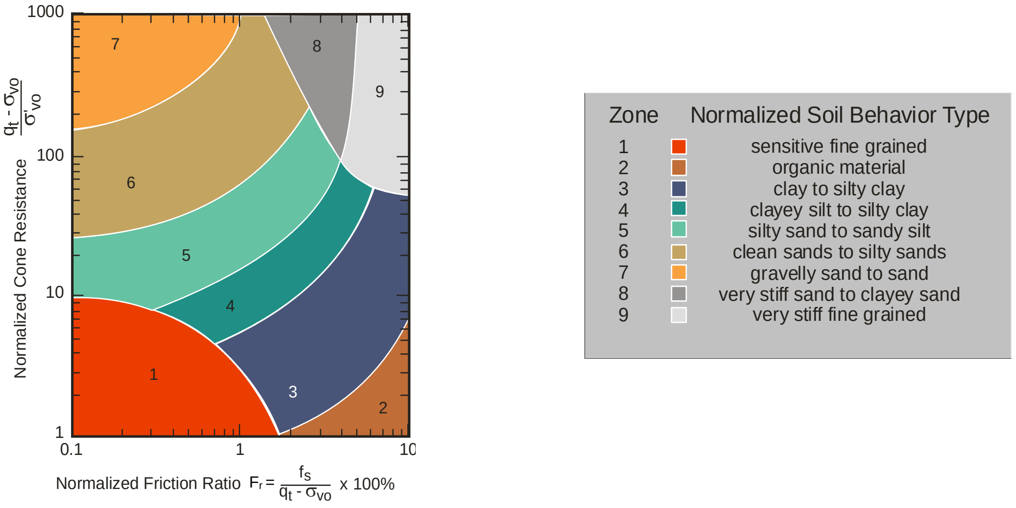


Zone		qt / N	Soil Behavior Type
1	■	2	sensitive fine grained
2	■	1	organic material
3	■	1	clay
4	■	1.5	silty clay to clay
5	■	2	clayey silt to silty clay
6	■	2.5	sandy silt to clayey silt
7	■	3	silty sand to sandy silt
8	■	4	sand to silty sand
9	■	5	sand
10	■	6	gravelly sand to sand
11	■	1	very stiff fine grained *
12	■	2	sand to clayey sand *

* overconsolidated or cemented

$$Rf = \left(\frac{fs}{qt}\right) \cdot 100\%$$

Figure 1. Non-Normalized Soil Behavior Type Classification Chart (SBT)



Zone		Normalized Soil Behavior Type
1	■	sensitive fine grained
2	■	organic material
3	■	clay to silty clay
4	■	clayey silt to silty clay
5	■	silty sand to sandy silt
6	■	clean sands to silty sands
7	■	gravelly sand to sand
8	■	very stiff sand to clayey sand
9	■	very stiff fine grained

Figure 2. Normalized Soil Behavior Type Classification Chart (SBTn)

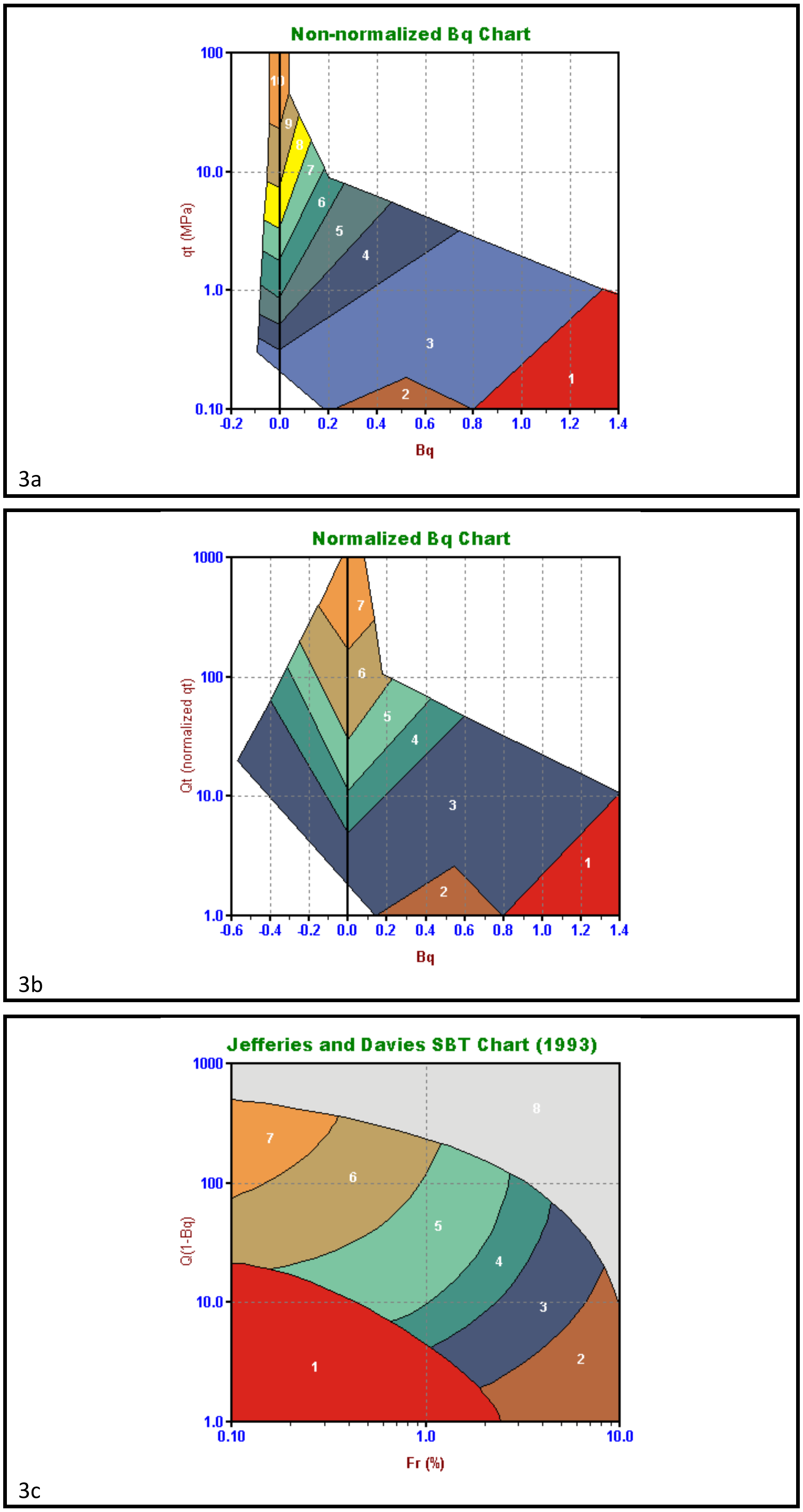


Figure 3. Alternate Soil Behavior Type Charts

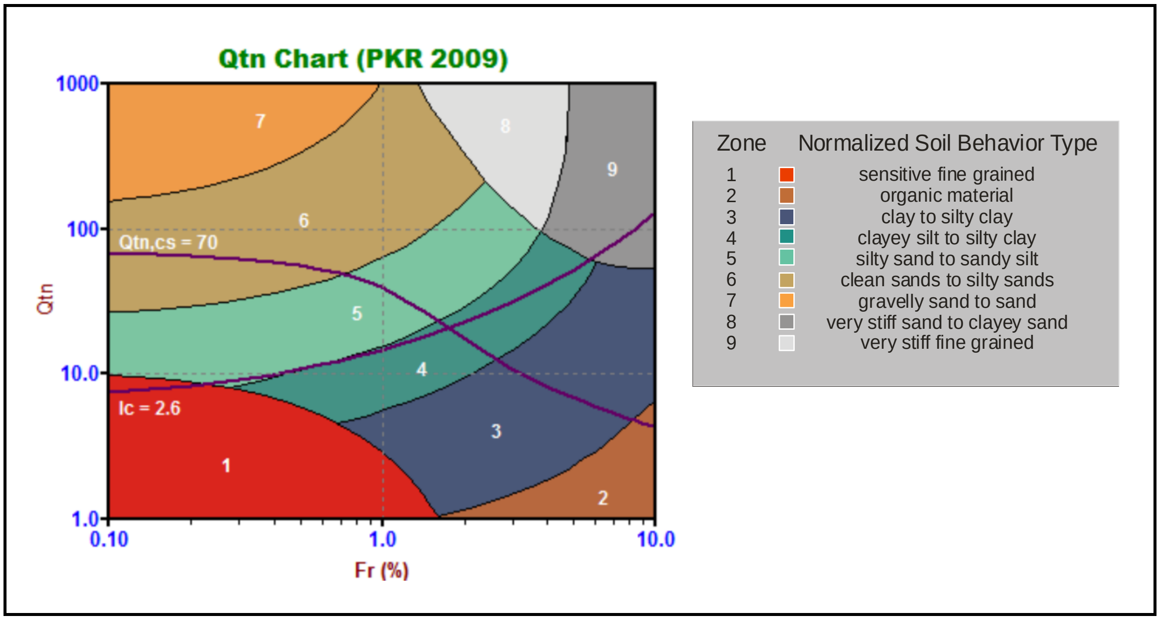


Figure 4. Normalized Soil Behavior Type Chart using Q_{tn} (SBT Qtn)

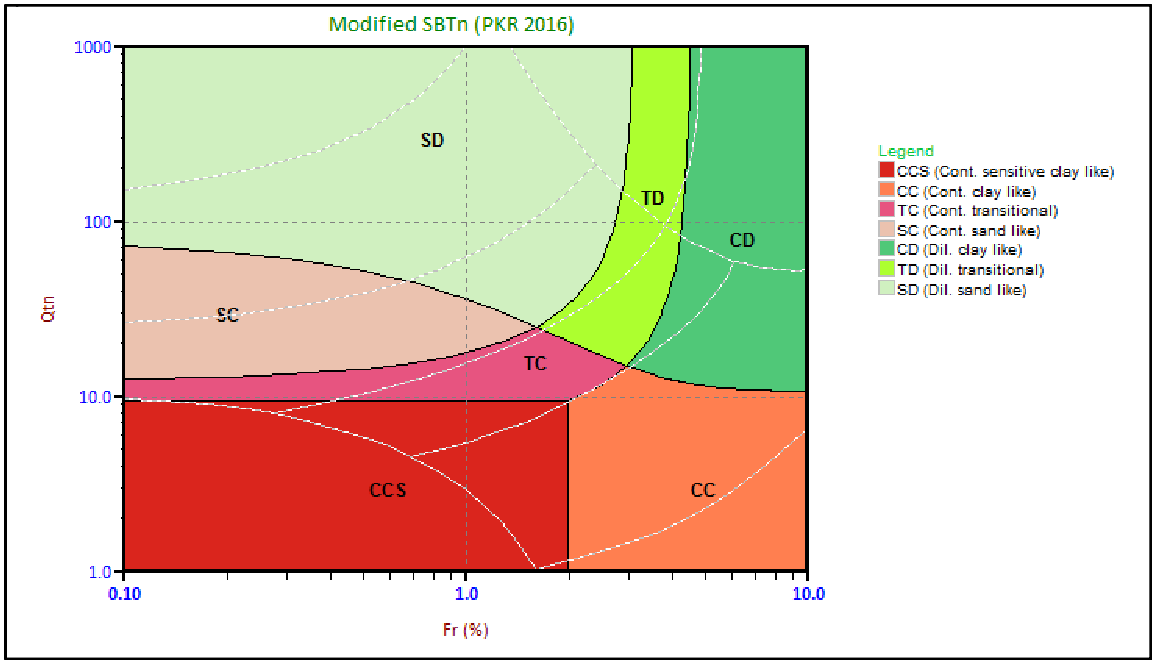


Figure 5. Modified SBTn Behavior Based Chart

Details regarding the geotechnical parameter calculations are provided in Tables 1a and 1b. The appropriate references cited are listed in Table 2. Non-liquefaction specific parameters are detailed in Table 1a and liquefaction specific parameters are detailed in Table 1b.

Where methods are based on charts or techniques that are too complex to describe in this summary the user should refer to the cited material. Specific limitations for each method are described in the cited material.

Where the results of a calculation/correlation are deemed *'invalid'* the value will be represented by the text strings "-9999", "-9999.0", the value 0.0 (Zero) or an empty cell. Invalid results will occur because of (and not limited to) one or a combination of:

1. Invalid or undefined CPT data (e.g. drilled out section or data gap).
2. Where the calculation method is inappropriate, for example, drained parameters in a material behaving as an undrained material (and vice versa).
3. Where input values are beyond the range of the referenced charts or specified limitations of the correlation method.
4. Where pre-requisite or intermediate parameter calculations are invalid.

The parameters selected for output from the program are often specific to a particular project. As such, not all of the calculated parameters listed in Table 1 may be included in the output files delivered with this report.

The output files are typically provided in Microsoft Excel XLS or XLSX format. The ConeTec software has several options for output depending on the number or types of calculated parameters desired or requested by the client. Each output file is named using the original COR file base name followed by a three or four letter indicator of the output set selected (e.g. BSC, TBL, NLI, NL2, IFI, IFI2) and possibly followed by an operator selected suffix identifying the characteristics of the particular calculation run.

Table 1a. CPT Parameter Calculation Methods – Non liquefaction Parameters

Calculated Parameter	Description	Equation	Ref
Depth	Mid Layer Depth *(where calculations are done at each point then Mid Layer Depth = Recorded Depth)*	*[Depth (Layer Top) + Depth (Layer Bottom)]/ 2.0*	CK*
Elevation	Elevation of Mid Layer based on sounding collar elevation supplied by client or through site survey	Elevation = Collar Elevation - Depth	CK*
Avg qc	Averaged recorded tip value (q_c)	$Avgqc = \frac{1}{n}\sum_{i=1}^{n} q_c$ *n=1 when calculations are done at each point*	CK*
Avg qt	Averaged corrected tip (q_t) where: $q_t = q_c + (1-a) \bullet u_2$	$Avgqt = \frac{1}{n}\sum_{i=1}^{n} q_t$ *n=1 when calculations are done at each point*	1
Avg fs	Averaged sleeve friction (f_s)	$Avgfs = \frac{1}{n}\sum_{i=1}^{n} fs$ *n=1 when calculations are done at each point*	CK*
Avg Rf	Averaged friction ratio (R_f) where friction ratio is defined as: $Rf = 100\% \bullet \frac{fs}{q_t}$	$AvgRf = 100\% \cdot \frac{Avgfs}{Avgqt}$ *n=1 when calculations are done at each point*	CK*
Avg u	Averaged dynamic pore pressure *(u)*	$Avgu = \frac{1}{n}\sum_{i=1}^{n} u_i$ *n=1 when calculations are done at each point*	CK*

Calculated Parameter	Description	Equation	Ref
Avg Res	Averaged Resistivity (this data is not always available since it is a specialized test requiring an additional module)	$AvgRes = \frac{1}{n}\sum_{i=1}^{n} Resistivity_i$ *n=1 when calculations are done at each point*	CK*
Avg UVIF	Averaged UVIF ultra-violet induced fluorescence (this data is not always available since it is a specialized test requiring an additional module)	$AvgUVIF = \frac{1}{n}\sum_{i=1}^{n} UVIF_i$ *n=1 when calculations are done at each point*	CK*
Avg Temp	Averaged Temperature (this data is not always available since it requires specialized calibrations)	$AvgTemp = \frac{1}{n}\sum_{i=1}^{n} Temperature_i$ *n=1 when calculations are done at each point*	CK*
Avg Gamma	Averaged Gamma Counts (this data is not always available since it is a specialized test requiring an additional module)	$AvgGamma = \frac{1}{n}\sum_{i=1}^{n} Gamma_i$ *n=1 when calculations are done at each point*	CK*
SBT	Soil Behavior Type as defined by Robertson et al 1986 (often referred to as Robertson and Campanella, 1986)	See Figure 1	1, 5
SBTn	Normalized Soil Behavior Type as defined by Robertson 1990 (linear normalization)	See Figure 2	2, 5
SBT-Bq	Non-normalized Soil Behavior type based on the Bq parameter	See Figure 3	1, 2, 5
SBT-Bqn	Normalized Soil Behavior based on the Bq parameter	See Figure 3	2, 5
SBT-JandD	Soil Behavior Type as defined by Jeffries and Davies	See Figure 3	7
SBT Qtn	Soil Behavior Type as defined by Robertson (2009) using a variable stress ratio exponent for normalization based on I_c	See Figure 4	15
Modified SBTn (contractive /dilative)	Modified SBTn chart as defined by Robertson (2016) indicating zones of contractive/dilative behavior.	See Figure 5	30
Unit Wt.	Unit Weight of soil determined from one of the following user selectable options: 1) uniform value 2) value assigned to each SBT zone 3) value assigned to each SBTn zone 4) value assigned to SBTn zone as determined from Robertson and Wride (1998) based on q_{c1n} 5) values assigned to SBT Qtn zones 6) Mayne fs (sleeve friction) method 7) Robertson 2010 method 8) user supplied unit weight profile The last option may co-exist with any of the other options	See references	3, 5, 15, 21, 24, 29

Calculated Parameter	Description	Equation	Ref
TStress σ_v	Total vertical overburden stress at Mid Layer Depth *A layer is defined as the averaging interval specified by the user where depths are reported at their respective mid-layer depth.* *For data calculated at each point layers are defined using the recorded depth as the mid-point of the layer. Thus, a layer starts half-way between the previous depth and the current depth unless this is the first point in which case the layer start is at zero depth. The layer bottom is half-way from the current depth to the next depth unless it is the last data point.* *Defining layers affects how stresses are calculated since the unit weight attributed to a data point is used throughout the entire layer. This means that to calculate the stresses the total stress at the top and bottom of a layer are required. The stress at mid layer is determined by adding the incremental stress from the layer top to the mid-layer depth. The stress at the layer bottom becomes the stress at the top of the subsequent layer. Stresses are NOT calculated from mid-point to mid-point.* *For over-water work the total stress due to the column of water above the mud line is taken into account where appropriate.*	$TStress = \sum_{i=1}^{n} \gamma_i h_i$ *where* γ_i is layer unit weight h_i is layer thickness	CK*
EStress σ_v'	Effective vertical overburden stress at mid-layer depth	$\sigma_v' = \sigma_v - u_{eq}$	CK*
Equil u u_{eq} or u_0	Equilibrium pore pressure determined from one of the following user selectable options: 1) hydrostatic below water table 2) user supplied profile 3) combination of those above When a user supplied profile is used/provided a linear interpolation is performed between equilibrium pore pressures defined at specific depths. If the profile values start below the water table then a linear transition from zero pressure at the water table to the first defined pointed is used. Equilibrium pore pressures may come from dissipation tests, adjacent piezometers or other sources. Occasionally, an extra equilibrium point ("assumed value") will be provided in the profile that does not come from a recorded value to smooth out any abrupt changes or to deal with material interfaces. These "assumed" values will be indicated on our plots and in tabular summaries.	For hydrostatic option: $u_{eq} = \gamma_w \cdot (D - D_{wt})$ where u_{eq} is equilibrium pore pressure γ_w is unit weight of water D is the current depth D_{wt} is the depth to the water table	CK*
K_0	Coefficient of earth pressure at rest, K_0	$K_0 = (1 - \sin\Phi')\ OCR^{\sin\Phi'}$	17
C_n	Overburden stress correction factor used for $(N_1)_{60}$ and older CPT parameters	$C_n = (P_a/\sigma_v')^{0.5}$ where $0.0 < C_n < 2.0$ *(user adjustable, typically 1.7)* P_a *is atmospheric pressure (100 kPa)*	12
C_q	Overburden stress normalizing factor	$C_q = 1.8 / (0.8 + (\sigma_v'/P_a))$ where $0.0 < C_q < 2.0$ *(user adjustable)* P_a *is atmospheric pressure (100 kPa)*	3, 12

Calculated Parameter	Description	Equation	Ref
N_{60}	SPT N value at 60% energy calculated from q_t/N ratios assigned to each SBT zone. This method has abrupt N value changes at zone boundaries.	*See Figure 1*	5
(N1)60	SPT N_{60} value corrected for overburden pressure	$(N_1)_{60} = C_n \cdot N_{60}$	4
N60Ic	SPT N_{60} values based on the I_c parameter [as defined by Roberston and Wride 1998 (5), or by Robertson 2009 (15)].	$(q_t/P_a)/ N_{60} = 8.5 (1 – I_c/4.6)$ $(q_t/P_a)/ N_{60} = 10^{(1.1268 – 0.2817Ic)}$ Pa being atmospheric pressure	5 15, 31
(N1)60Ic	SPT N_{60} value corrected for overburden pressure (using N_{60} I_c). User has 3 options.	1) $(N_1)_{60}Ic = C_n \cdot (N_{60} I_c)$ 2) $q_{c1n}/ (N_1)_{60}I_c = 8.5 (1 – I_c/4.6)$ 3) $(Q_{tn})/ (N_1)_{60}I_c = 10^{(1.1268 – 0.2817Ic)}$	4 5 15, 31
Su or Su (Nkt)	Undrained shear strength based on q_t S_u factor N_{kt} is user selectable	$Su = \frac{qt - \sigma_v}{N_{kt}}$	1, 5
Su or Su (Ndu)	Undrained shear strength based on pore pressure S_u factor $N_{\Delta u}$ is user selectable	$Su = \frac{u_2 - u_{eq}}{N_{\Delta u}}$	1, 5
Dr	Relative Density determined from one of the following user selectable options: a) Ticino Sand b) Hokksund Sand c) Schmertmann (1978) d) Jamiolkowski (1985) - All Sands e) Jamiolkowski et al (2003) (various compressibilities, K_o)	See reference (methods a through d) Jamiolkowski et al (2003) reference	5 14
PHI ϕ	Friction Angle determined from one of the following user selectable options (methods a through d are for sands and method e is for silts and clays): a) Campanella and Robertson b) Durgunoglu and Mitchel c) Janbu d) Kulhawy and Mayne e) NTH method (clays and silts)	See appropriate reference	5 5 5 11 23
Delta U/qt	Differential pore pressure ratio (older parameter used before B_q was established)	$= \frac{\Delta u}{qt}$ *where:* $\Delta u = u - u_{eq}$ *and* *u = dynamic pore pressure* u_{eq} *= equilibrium pore pressure*	CK*
Bq	Pore pressure parameter	$Bq = \frac{\Delta u}{qt - \sigma_v}$ where : $\Delta u = u - u_{eq}$ *and* *u = dynamic pore pressure* u_{eq} *= equilibrium pore pressure*	1, 2, 5
Net qt or qtNet	Net tip resistance (used in many subsequent correlations)	$qt - \sigma_v$	CK*
qe	Effective tip resistance (using the dynamic pore pressure u_2 and not equilibrium pore pressure)	$qt - u_2$	CK*

Calculated Parameter	Description	Equation	Ref
qeNorm	Normalized effective tip resistance	$\frac{qt - u_2}{\sigma_v'}$	CK*
Q_t or Norm: Qt	Normalized q_t for Soil Behavior Type classification as defined by Robertson (1990) using a linear stress normalization. Note this is different from Q_{tn}.	$Qt = \frac{qt - \sigma_v}{\sigma_v'}$	2, 5
F_r or Norm: Fr	Normalized Friction Ratio for Soil Behavior Type classification as defined by Robertson (1990)	$Fr = 100\% \cdot \frac{fs}{qt - \sigma_v}$	2, 5
Q(1-Bq)	Q(1-Bq) grouping as suggested by Jefferies and Davies for their classification chart and the establishment of their I_c parameter	$Q \cdot (1 - Bq)$ *where Bq is defined as above and Q is the same as the normalized tip resistance, Q_t, defined above*	6, 7
qc1	Normalized tip resistance, q_{c1}, using a fixed stress ratio exponent, n (this method has stress units)	$q_{c1} = q_t \cdot (Pa/\sigma_v')^{0.5}$ where: Pa = atmospheric pressure	21
qc1 (0.5)	Normalized tip resistance, q_{c1}, using a fixed stress ratio exponent, n (this method is unit-less)	$q_{c1}\ (0.5) = (q_t/P_a) \cdot (Pa/\sigma_v')^{0.5}$ where: Pa = atmospheric pressure	5
qc1 (Cn)	Normalized tip resistance, q_{c1}, based on C_n (this method has stress units)	$q_{c1}(Cn) = C_n * q_t$	5, 12
qc1 (Cq)	Normalized tip resistance, q_{c1}, based on C_q (this method has stress units)	$q_{c1}\ (Cq) = C_q * q_t$ (some papers use q_c)	5, 12
qc1n	normalized tip resistance, q_{c1n}, using a variable stress ratio exponent, n (where n=0.0, 0.70, 1.0) (this method is unit-less)	$q_{c1n} = (q_t / P_a)(P_a/\sigma_v')^n$ where: P_a = atm. Pressure and n varies as described below	3, 5
I_c or Ic (RW1998)	Soil Behavior Type Index as defined by Robertson and Fear (1995) and Robertson and Wride (1998) for estimating grain size characteristics and providing smooth gradational changes across the SBTn chart	$I_c = [(3.47 - \log_{10} Q)^2 + (\log_{10} Fr + 1.22)^2]^{0.5}$ *Where:* $Q = \left(\frac{qt - \sigma_v}{P_a}\right)\left(\frac{P_a}{\sigma_v'}\right)^n$ *Or* $Q = q_{c1n} = \left(\frac{qt}{P_a}\right)\left(\frac{P_a}{\sigma_v'}\right)^n$ *depending on the iteration in determining I_c* *And* *Fr is in percent* *P_a = atmospheric pressure* *n varies between 0.5, 0.70 and 1.0 and is selected in an iterative manner based on the resulting I_c*	3, 5, 21
Ic (PKR 2009)	Soil Behavior Type Index, I_c (PKR 2009) based on a variable stress ratio exponent n, which itself is based on I_c (PKR 2009). An iterative calculation is required to determine Ic (PKR 2009) and its corresponding n (PKR 2009).	$I_c\ (PKR\ 2009) = [(3.47 - \log_{10} Q_{tn})^2 + (1.22 + \log_{10} F_r)^2]^{0.5}$	15

Calculated Parameter	Description	Equation	Ref
n (PKR 2009)	Stress ratio exponent n, based on I_c (PKR 2009). An iterative calculation is required to determine n (PKR 2009) and its corresponding Ic (PKR 2009).	$n\ (PKR\ 2009) = 0.381\ (I_c) + 0.05\ (\sigma_v'/P_a) - 0.15$	15
Qtn (PKR 2009)	Normalized tip resistance using a variable stress ratio exponent based on I_c (PKR 2009) and n (PKR 2009). An iterative calculation is required to determine Qtn (PKR 2009).	$Q_{tn} = [(qt - \sigma_v)/P_a](P_a/\sigma_v')^n$ where P_a = atmospheric pressure (100 kPa) n = stress ratio exponent described above	15
FC	Apparent fines content (%)	$FC=1.75(Ic^{3.25}) - 3.7$ $FC=100$ for $I_c > 3.5$ $FC=0$ for $I_c < 1.26$ $FC = 5\%$ if $1.64 < I_c < 2.6$ AND $F_r<0.5$	3
I_c Zone	This parameter is the Soil Behavior Type zone based on the I_c parameter (valid for zones 2 through 7 on SBTn or SBT Qtn charts)	$I_c < 1.31$ Zone = 7 $1.31 < I_c < 2.05$ Zone = 6 $2.05 < I_c < 2.60$ Zone = 5 $2.60 < I_c < 2.95$ Zone = 4 $2.95 < I_c < 3.60$ Zone = 3 $I_c > 3.60$ Zone = 2	3
State Param or State Parameter or ψ	The state parameter index, ψ, is defined as the difference between the current void ratio, e, and the critical void ratio, e_c. Positive ψ - contractive soil Negative ψ - dilative soil This is based on the work by Been and Jefferies (1985) and Plewes, Davies and Jefferies (1992) - vertical effective stress is used rather than a mean normal stress	See reference	6, 8
Yield Stress σ_p'	Yield stress is calculated using the following methods a) General method b) 1st order approximation using q_tNet (clays) c) 1st order approximation using Δu_2 (clays) d) 1st order approximation using q_e (clays)	All stresses in kPa a) $\sigma_p'= 0.33\cdot(q_t - \sigma_v)^{m'}\ (\sigma_{atm}/100)^{1-m'}$ where $m'=1-\frac{0.28}{1+(I_c/2.65)^{25}}$ b) $\sigma_p' = 0.33\cdot(q_t - \sigma_v)$ c) $\sigma_p' = 0.54\cdot\ (\Delta u_2)$ $\Delta u_2 = u_2 - u_0$ d) $\sigma_p' = 0.60 \cdot (q_t - u_2)$	19 20 20 20
OCR OCR(JS1978) OCR(Mayne2014) OCR (qtNet) OCR (deltaU) OCR (qe) OCR (Vs) OCR (PKR2015)	Over Consolidation Ratio based on a) Schmertmann (1978) method involving a plot plot of S_u/σ_v' /($S_u/\sigma_v')_{NC}$ and OCR b) based on Yield stresses described above c) approximate version based on qtNet d) approximate version based on Δu e) approximate version based on effective tip, q_e f) approximate version based on shear wave velocity, V_s g) based on Qt	a) requires a user defined value for NC S_u/P_c' ratio b through f) *based on yield stresses* g) OCR = $0.25\cdot(Qt)^{1.25}$	9 19 20 20 20 18 32

Calculated Parameter	Description	Equation	Ref
Es/qt	Intermediate parameter for calculating Young's Modulus, E, in sands. It is the Y axis of the reference chart.	Based on Figure 5.59 in the reference	5
Es Young's Modulus E	Young's Modulus based on the work done in Italy. There are three types of sands considered in this technique. The user selects the appropriate type for the site from: a) OC Sands b) Aged NC Sands c) Recent NC Sands Each sand type has a family of curves that depend on mean normal stress. The program calculates mean normal stress and linearly interpolates between the two extremes provided in the E_s/q_t chart. Es is evaluated for an axial strain of 0.1%.	Mean normal stress is evaluated from: $\sigma'_m = \frac{1}{3}\left(\sigma'_v + \sigma'_h + \sigma'_h\right)^3$ where σ_v'= vertical effective stress σ_h'= horizontal effective stress *and* $\sigma_h = K_o \bullet \sigma_v'$ with K_o assumed to be 0.5	5
Delta U/TStress	Differential pore pressure ratio with respect to total stress	$= \frac{\Delta u}{\sigma_v}$ *where:* $\Delta u = u - u_{eq}$	CK*
Delta U/Estress, P Value, Excess Pore Pressure Ratio	Differential pore pressure ratio with respect to effective stress. Key parameter (P, Normalized Pore Pressure Parameter, Excess Pore Pressure Ratio) in the Winckler et. al. static liquefaction method.	$= \frac{\Delta u}{\sigma'_v}$ *where:* $\Delta u = u - u_{eq}$	25, 25a, CK*
Su/EStress	Undrained shear strength ratio with respect to vertical effective overburden stress using the S_u (N_{kt}) method	$= Su\ (N_{kt})\ /\ \sigma_v'$	CK*
Gmax	G_{max} determined from SCPT shear wave velocities (not estimated values)	$G_{max} = \rho V_s^2$ where ρ is the mass density of the soil determined from the estimated unit weights at each test depth	27
qtNet/Gmax	Net tip resistance ratio with respect to the small strain modulus G_{max} determined from SCPT shear wave velocities (not estimated values)	$= (qt - \sigma_v) / G_{max}$ where $G_{max} = \rho V_s^2$ and ρ is the mass density of the soil determined from the estimated unit weights at each test depth	15, 28, 30

*CK – common knowledge

Table 1b. CPT Parameter Calculation Methods – Liquefaction Parameters

Calculated Parameter	Description	Equation	Ref
K_{SPT}	Equivalent clean sand factor for $(N_1)60$	$K_{SPT} = 1 + ((0.75/30) \bullet (FC - 5))$	10
K_{CPT} or K_C (RW1998)	Equivalent clean sand correction for q_{c1N}	$K_{cpt} = 1.0$ for $I_c \leq 1.64$ $K_{cpt} = f(I_c)$ for $I_c > 1.64$ (see reference) $K_c = -0.403\ I_c^4 + 5.581\ I_c^3 - 21.63 I_c^2 + 33.75\ I_c - 17.88$	3, 10
Kc (PKR 2010)	Clean sand equivalent factor to be applied to Q_{tn}	$K_c = 1.0$ for $I_c \leq 1.64$ $K_c = -0.403\ I_c^4 + 5.581\ I_c^3 - 21.63 I_c^2 + 33.75\ I_c - 17.88$ for $I_c > 1.64$	16
$(N_1)_{60cs}Ic$	Clean sand equivalent SPT $(N_1)_{60}I_c$. User has 3 options.	1) $(N_1)_{60cs}Ic = \alpha + \beta((N_1)_{60}I_c)$ 2) $(N_1)_{60cs}Ic = K_{SPT} * ((N_1)_{60}I_c)$ 3) $(q_{c1ncs})/\ (N_1)_{60cs}I_c = 8.5\ (1 - I_c/4.6)$ FC ≤ 5%: $\alpha = 0$, $\beta = 1.0$ FC ≥ 35% $\alpha = 5.0$, $\beta = 1.2$ 5% < FC < 35% $\alpha = \exp[1.76 - (190/FC^2)]$ $\beta = [0.99 + (FC^{1.5}/1000)]$	10 10 5
q_{c1ncs}	Clean sand equivalent q_{c1n}	$q_{c1ncs} = q_{c1n} \bullet K_{cpt}$	3
Qtn,cs (PKR 2010)	Clean sand equivalent for Q_{tn} described above - Q_{tn} being the normalized tip resistance based on a variable stress exponent as defined by Robertson (2009)	$Q_{tn,cs} = Q_{tn} \cdot K_c$ (PKR 2016)	16
Su(Liq)/ESv	Liquefied shear strength ratio as defined by Olson and Stark	$\frac{Su(Liq)}{\sigma_v'} = 0.03 + 0.0143(q_{c1})$ Note: σ_v' and s_v' are synonymous	13
Su(Liq)/ESv (PKR 2010)	Liquefied shear strength ratio as defined by Robertson (2010)	$\frac{Su(Liq)}{\sigma_v'}$ Based on a function involving $Q_{tn,cs}$	16
Su (Liq) (PKR 2010)	Liquefied shear strength derived from the liquefied shear strength ratio and effective overburden stress		16
Cont/Dilat Tip	Contractive / Dilative qc1 Boundary based on $(N_1)_{60}$	$(\sigma_v')_{boundary} = 9.58 \times 10^{-4}\ [(N_1)_{60}]^{4.79}$ $qc1$ is calculated from specified qt(MPa)/N ratio	13
CRR	Cyclic Resistance Ratio (for Magnitude 7.5)	$q_{c1ncs} < 50$: $CRR_{7.5} = 0.833\ [q_{c1ncs}/1000] + 0.05$ $50 \leq q_{c1ncs} < 160$: $CRR_{7.5} = 93\ [q_{c1ncs}/1000]^3 + 0.08$	10
Kg	Small strain Stiffness Ratio Factor, Kg	$[Gmax/qt]/[qc1n^{-m}]$ m = empirical exponent, typically 0.75	26

Calculated Parameter	Description	Equation	Ref
SP Distance	State Parameter Distance, Winckler static liquefaction method	Perpendicular distance on Qtn chart from plotted point to state parameter Ψ = -0.05 curve	25
URS NP Fr	Normalized friction ratio point on Ψ = -0.05 curve used in SP Distance calculation		25
URS NP Qtn	Normalized tip resistance (Qtn) point on Ψ = -0.05 curve used in SP Distance calculation		25

Table 2. References

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Appendix E

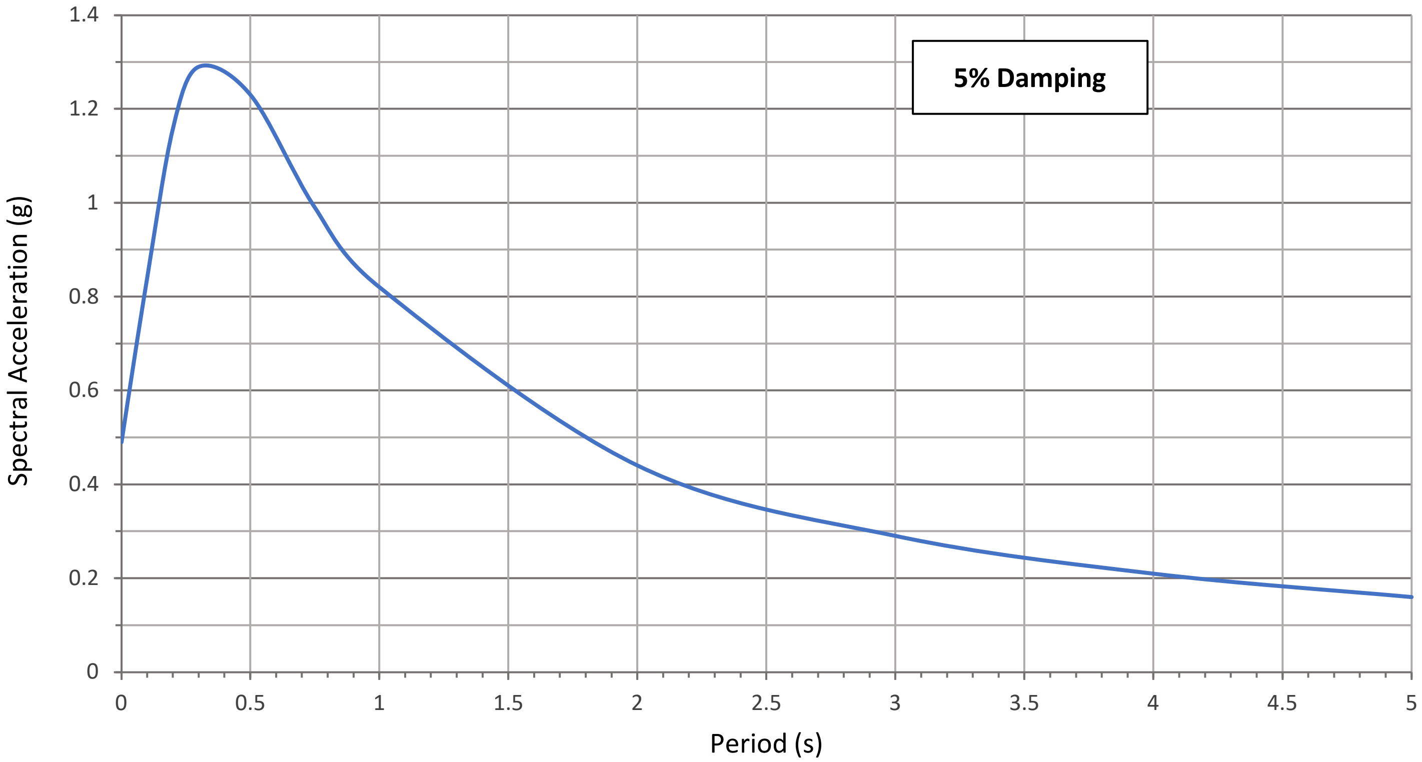
Project Name: FORTAG SR 218 Undercrossing Bridge and Retaining Wall No. 1

ARS Data

Period, T (sec)	Spectral Acceleration, S_a (g)
0	0.49
0.1	0.84
0.2	1.16
0.3	1.29
0.5	1.23
0.75	0.99
1	0.82
2	0.44
3	0.29
4	0.21
5	0.16

Acceleration Response Spectrum

Latitude:	36.5937°
Longitude:	-121.8362°



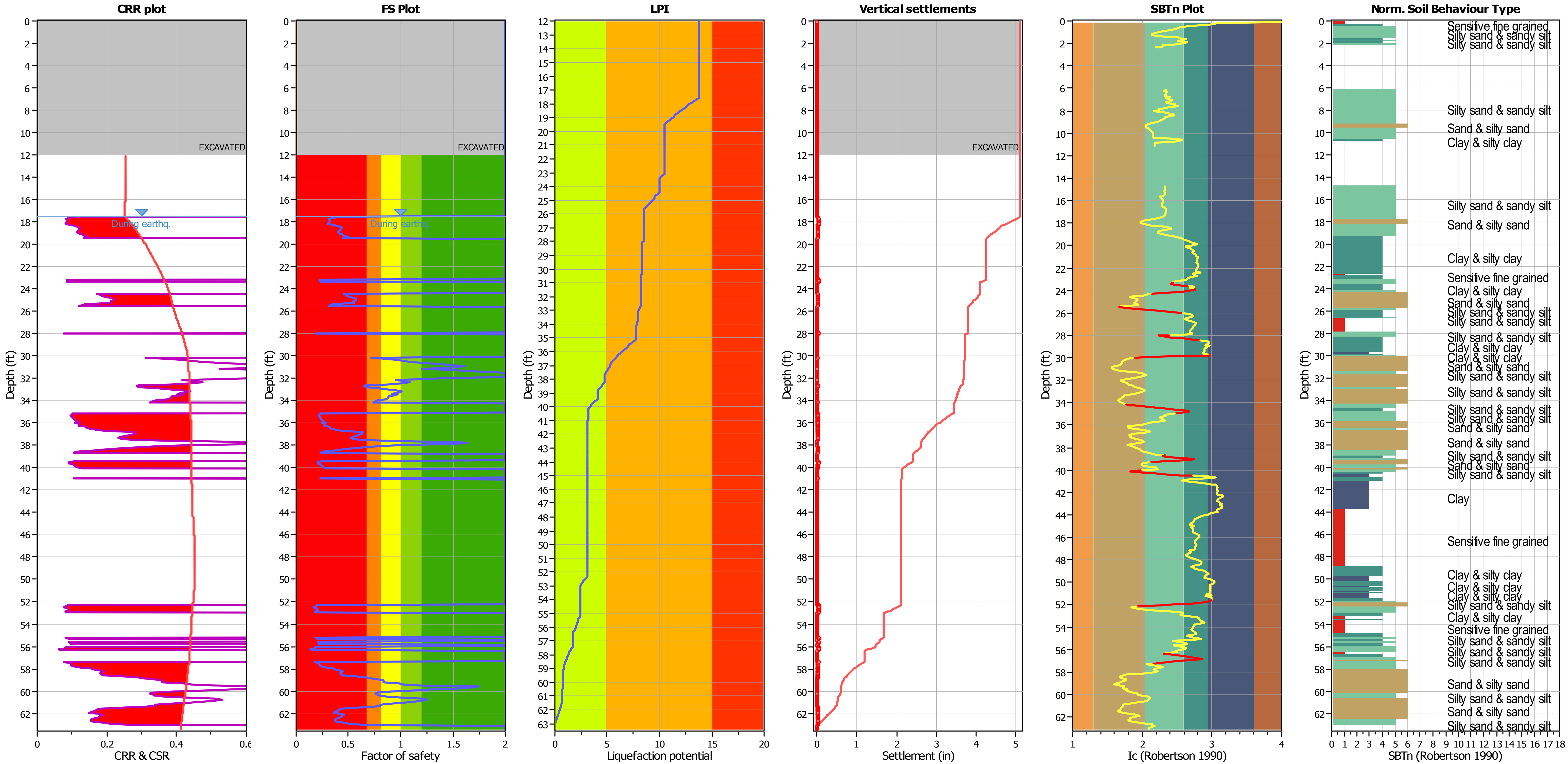
The ARS was based on the USGS' 2014 National Seismic Hazard Map for 975-years return period. (Hazard Model/Edition "Dynamic Conterminous U.S. 2014 (Update)(V4.2.0)") hazard data obtained by using ARS online v3.0.2. Modifications for basin-effects and/or near-fault effects were applied, where applicable.

V_{s30}: 720 feet/sec (220 meters/sec)
PGA: 0.49g
Mean Moment Magnitude (for PGA): M = 6.75

Appendix F

Project: FORTAG
Location: Del Rey Oaks, CA

CPT: CPT-01
Total depth: 63.40 ft

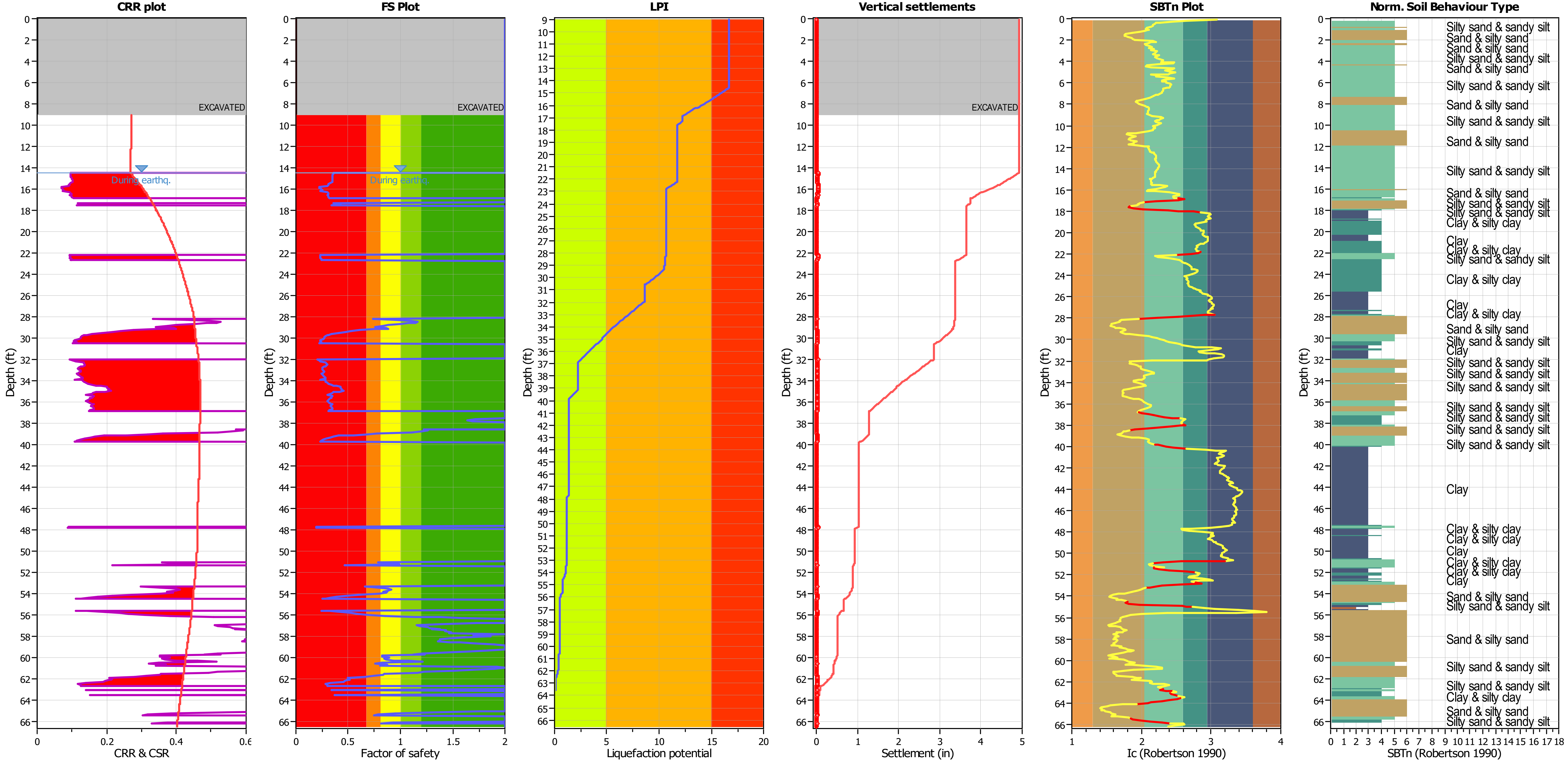


Analysis method:	NCEER (1998)	G.W.T. (in-situ):	17.60 ft	Excavation:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	17.50 ft	Excavation depth:	12.00 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	1	Footing load:	0.00 tsf	Limit depth:	N/A
Earthquake magnitude M_w:	6.81	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.50	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

GHD
Transportation Agency for Monterey County
FORTAG - Canyon Del Rey/SR 218 Segment
Del Rey Oaks, California

Project: **FORTAG**
Location: **Del Rey Oaks, CA**

CPT: CPT-02
Total depth: 66.52 ft



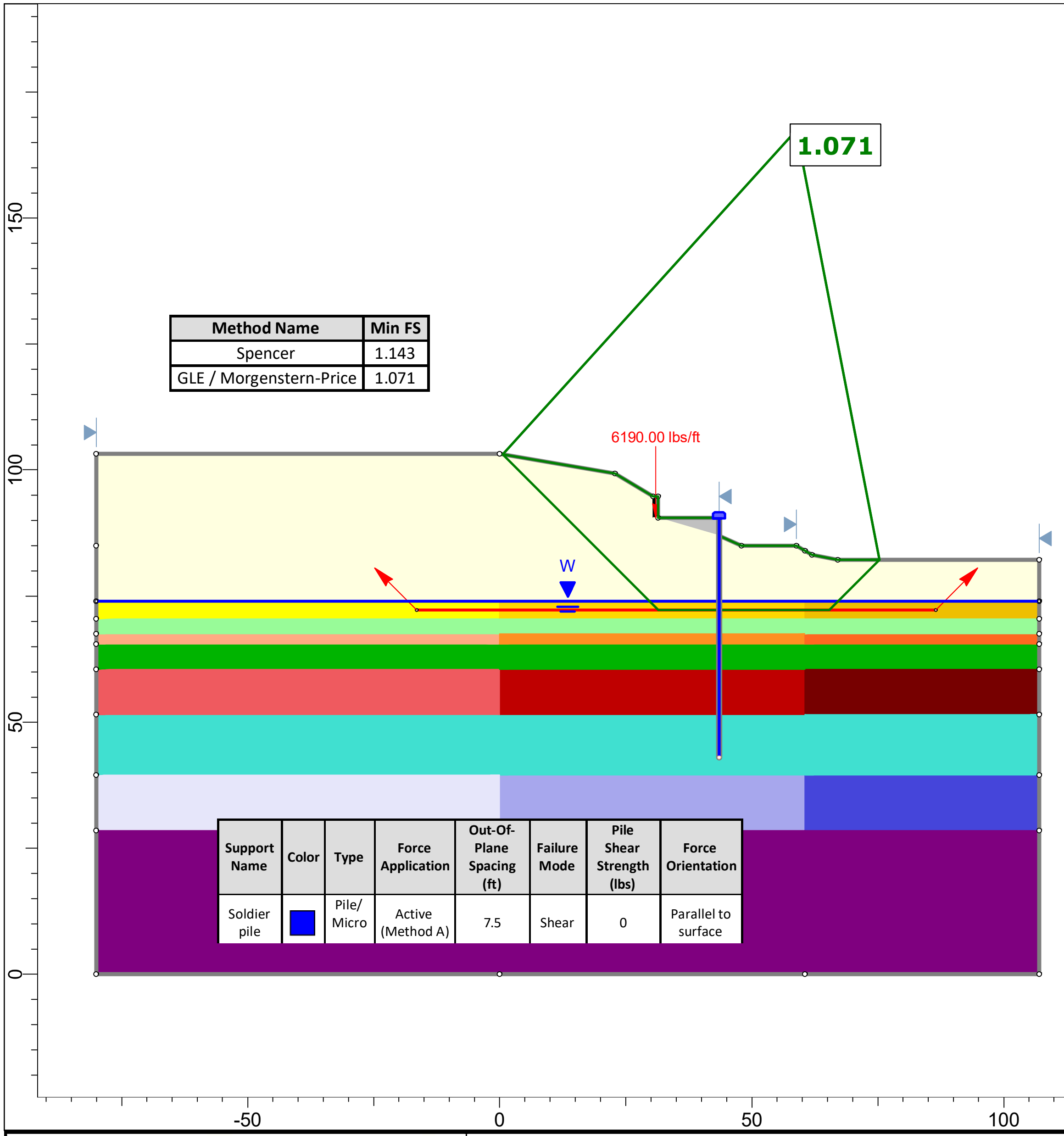
Analysis method: NCEER (1998)
Fines correction method: NCEER (1998)
Points to test: Based on Ic value
Earthquake magnitude M_w: 6.81
Peak ground acceleration: 0.50

G.W.T. (in-situ): 14.60 ft
G.W.T. (earthq.): 14.50 ft
Average results interval: 1
Ic cut-off value: 2.60
Unit weight calculation: Based on SBT

Excavation: Yes
Excavation depth: 9.00 ft
Footing load: 0.00 tsf
Trans. detect. applied: Yes
K_σ applied: Yes

Clay like behavior applied: Sands only
Limit depth applied: No
Limit depth: N/A
MSF method: I&B, 2008

Appendix G



Method Name	Min FS
Spencer	1.143
GLE / Morgenstern-Price	1.071

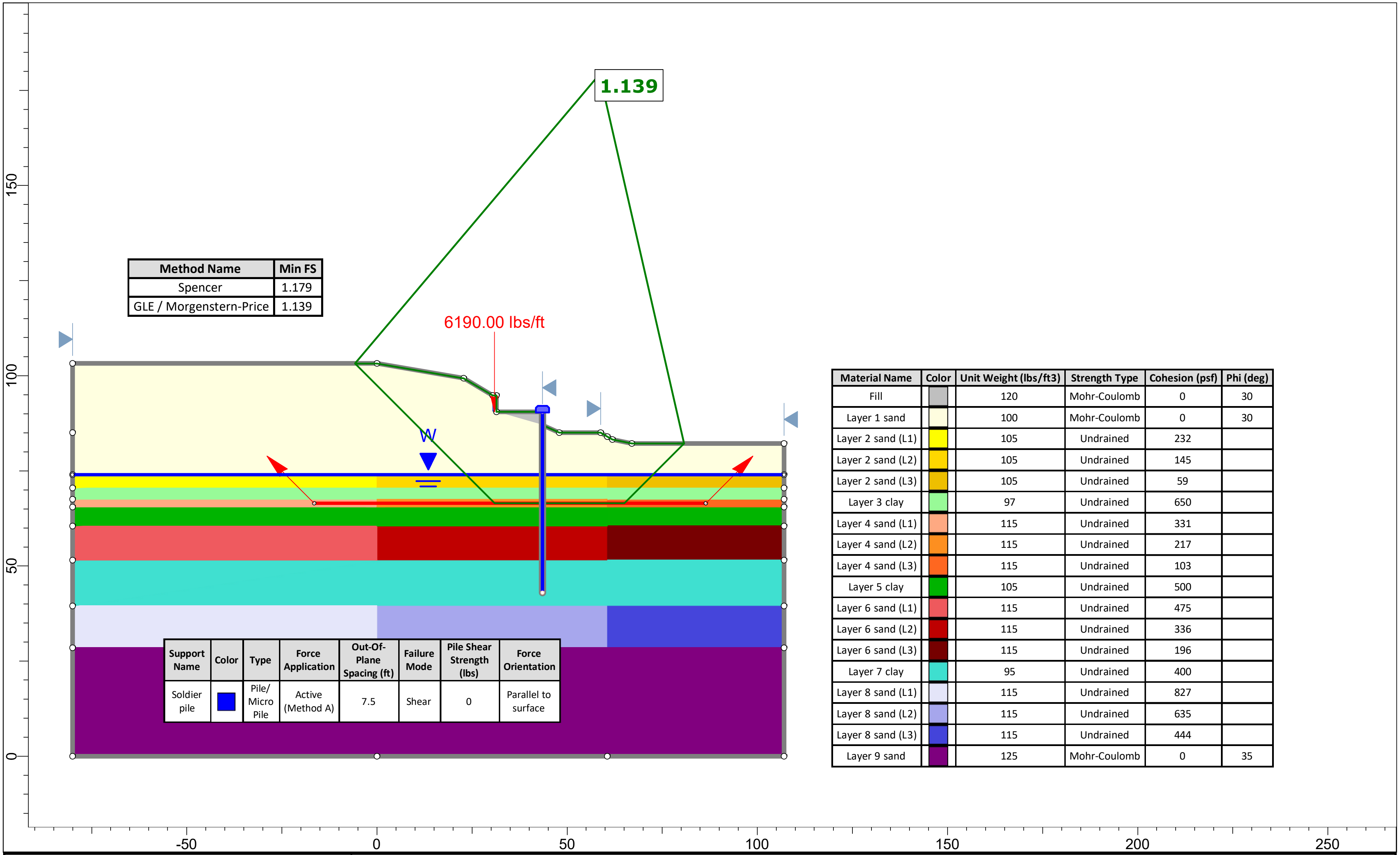
Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Failure Mode	Pile Shear Strength (lbs)	Force Orientation
Soldier pile	🟦	Pile/ Micro	Active (Method A)	7.5	Shear	0	Parallel to surface

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Ru
Fill	⬜	120	Mohr-Coulomb	0	30	
Layer 1 sand	⬜	100	Mohr-Coulomb	0	30	
Layer 2 sand (L1)	🟨	105	Undrained	232		
Layer 2 sand (L2)	🟨	105	Undrained	145		
Layer 2 sand (L3)	🟨	105	Undrained	59		
Layer 3 clay	🟩	97	Undrained	650		
Layer 4 sand (L1)	🟧	115	Undrained	331		
Layer 4 sand (L2)	🟧	115	Undrained	217		
Layer 4 sand (L3)	🟧	115	Undrained	103		
Layer 5 clay	🟩	105	Undrained	500		
Layer 6 sand (L1)	🟥	115	Undrained	475		
Layer 6 sand (L2)	🟥	115	Undrained	336		
Layer 6 sand (L3)	🟥	115	Undrained	196		
Layer 7 clay	🟦	95	Undrained	400		
Layer 8 sand (L1)	⬜	115	Undrained	827		
Layer 8 sand (L2)	🟪	115	Undrained	635		
Layer 8 sand (L3)	🟦	115	Undrained	444		
Layer 9 sand	🟪	125	Mohr-Coulomb	0	35	0
Retaining Wall No. 2	⬛	150	Infinite strength			0

McMILLEN JACOBS ASSOCIATES

SLIDEINTERPRET 9.027

Project Name	TAMC - FORTAG Canyon Del Rey/SR218 Segement
Project No.	6231.0
Scenario	Liquefiable Layer at El 72.25, kh=0
Date	April 2023
Location	Del Rey Oaks, California



Method Name	Min FS
Spencer	1.179
GLE / Morgenstern-Price	1.139

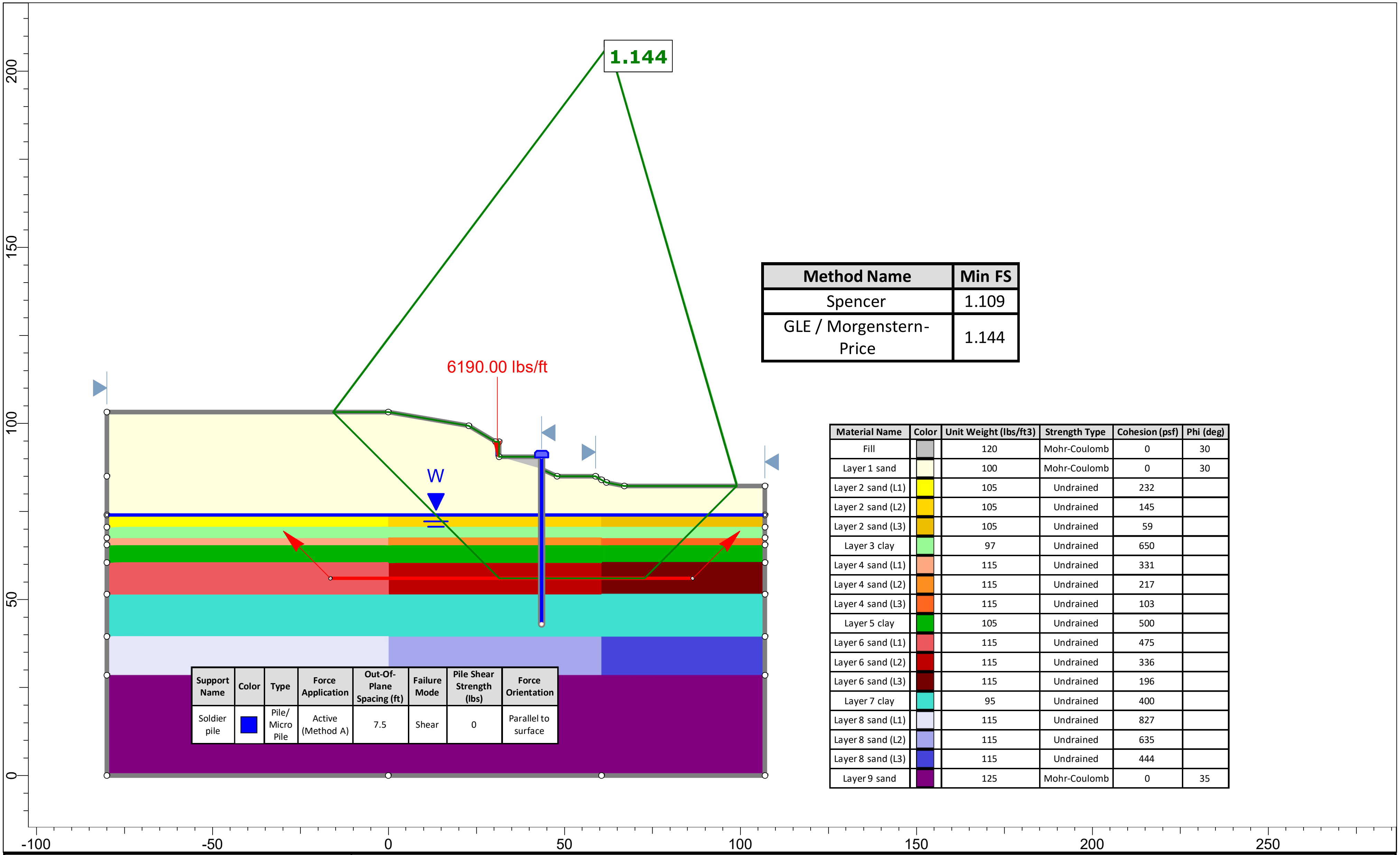
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Fill	⬜	120	Mohr-Coulomb	0	30
Layer 1 sand	⬜	100	Mohr-Coulomb	0	30
Layer 2 sand (L1)	🟨	105	Undrained	232	
Layer 2 sand (L2)	🟨	105	Undrained	145	
Layer 2 sand (L3)	🟨	105	Undrained	59	
Layer 3 clay	🟩	97	Undrained	650	
Layer 4 sand (L1)	🟧	115	Undrained	331	
Layer 4 sand (L2)	🟧	115	Undrained	217	
Layer 4 sand (L3)	🟧	115	Undrained	103	
Layer 5 clay	🟩	105	Undrained	500	
Layer 6 sand (L1)	🟥	115	Undrained	475	
Layer 6 sand (L2)	🟥	115	Undrained	336	
Layer 6 sand (L3)	🟥	115	Undrained	196	
Layer 7 clay	🟦	95	Undrained	400	
Layer 8 sand (L1)	⬜	115	Undrained	827	
Layer 8 sand (L2)	🟪	115	Undrained	635	
Layer 8 sand (L3)	🟦	115	Undrained	444	
Layer 9 sand	🟪	125	Mohr-Coulomb	0	35

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Failure Mode	Pile Shear Strength (lbs)	Force Orientation
Soldier pile	🟦	Pile/ Micro Pile	Active (Method A)	7.5	Shear	0	Parallel to surface

McMILLEN JACOBS ASSOCIATES

SLIDEINTERPRET 9.027

Project Name TAMC - FORTAG Canyon Del Rey/SR218 Segement	
Project No. 6231.0	*Scenario* Liquefiable Layer at El 66.5, kh=0
Date April 2023	*Location* Del Rey Oaks, California



Method Name	Min FS
Spencer	1.109
GLE / Morgenstern-Price	1.144

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Fill	⬜	120	Mohr-Coulomb	0	30
Layer 1 sand	⬜	100	Mohr-Coulomb	0	30
Layer 2 sand (L1)	🟨	105	Undrained	232	
Layer 2 sand (L2)	🟨	105	Undrained	145	
Layer 2 sand (L3)	🟨	105	Undrained	59	
Layer 3 clay	🟩	97	Undrained	650	
Layer 4 sand (L1)	🟧	115	Undrained	331	
Layer 4 sand (L2)	🟧	115	Undrained	217	
Layer 4 sand (L3)	🟧	115	Undrained	103	
Layer 5 clay	🟩	105	Undrained	500	
Layer 6 sand (L1)	🟥	115	Undrained	475	
Layer 6 sand (L2)	🟥	115	Undrained	336	
Layer 6 sand (L3)	🟫	115	Undrained	196	
Layer 7 clay	🟦	95	Undrained	400	
Layer 8 sand (L1)	⬜	115	Undrained	827	
Layer 8 sand (L2)	🟪	115	Undrained	635	
Layer 8 sand (L3)	🟦	115	Undrained	444	
Layer 9 sand	🟪	125	Mohr-Coulomb	0	35

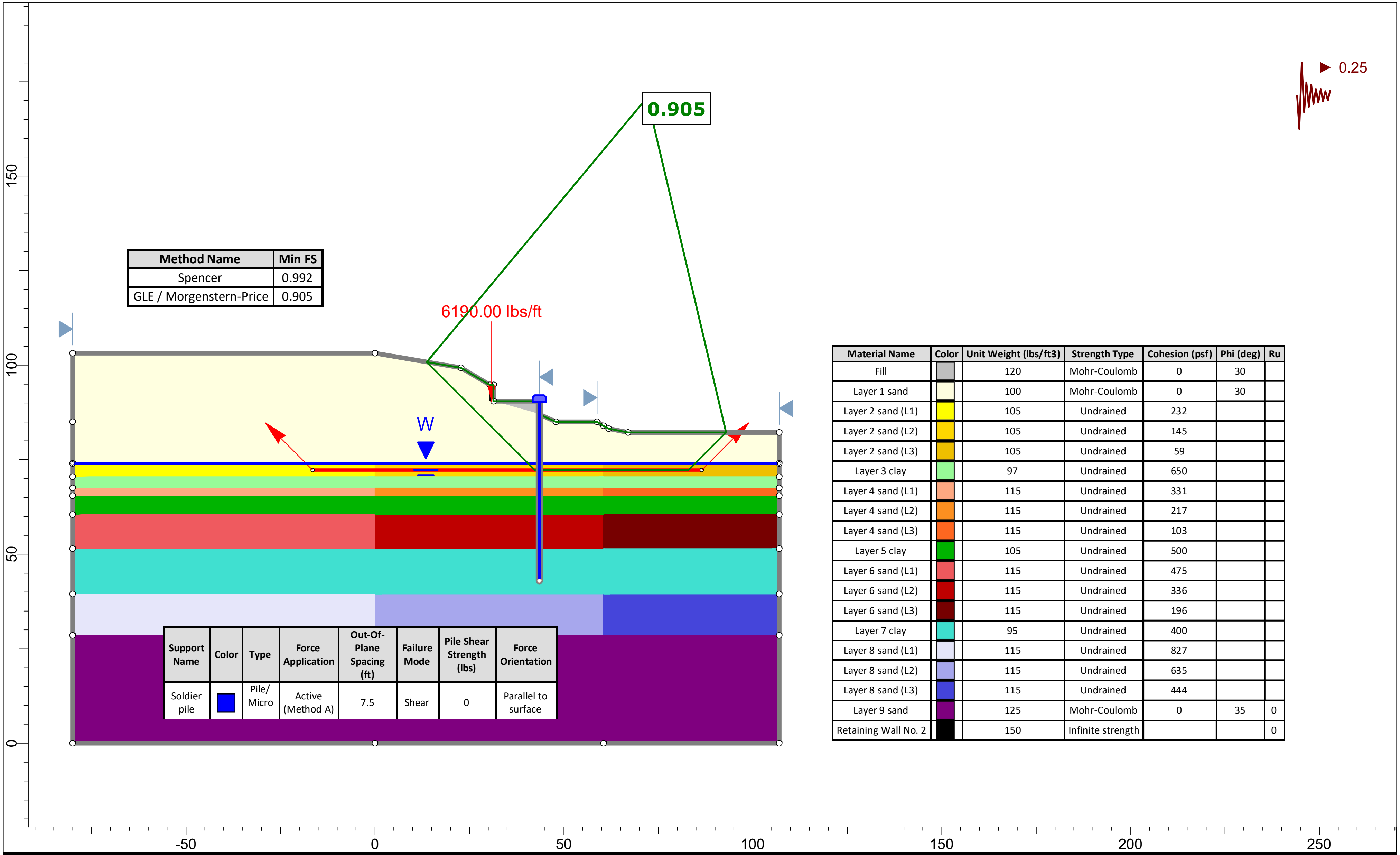
Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Failure Mode	Pile Shear Strength (lbs)	Force Orientation
Soldier pile	🟦	Pile/ Micro Pile	Active (Method A)	7.5	Shear	0	Parallel to surface

McMILLEN JACOBS ASSOCIATES

SLIDEINTERPRET 9.027

Project Name	TAMC - FORTAG Canyon Del Rey/SR218 Segement

Project No.	6231.0	*Scenario*	Liquefiable Layer at El 56, kh=0
Date	April 2023	*Location*	Del Rey Oaks, California



Method Name	Min FS
Spencer	0.992
GLE / Morgenstern-Price	0.905

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Failure Mode	Pile Shear Strength (lbs)	Force Orientation
Soldier pile	🟦	Pile/ Micro	Active (Method A)	7.5	Shear	0	Parallel to surface

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Ru
Fill	⬜	120	Mohr-Coulomb	0	30	
Layer 1 sand	⬜	100	Mohr-Coulomb	0	30	
Layer 2 sand (L1)	🟨	105	Undrained	232		
Layer 2 sand (L2)	🟨	105	Undrained	145		
Layer 2 sand (L3)	🟨	105	Undrained	59		
Layer 3 clay	🟩	97	Undrained	650		
Layer 4 sand (L1)	🟧	115	Undrained	331		
Layer 4 sand (L2)	🟧	115	Undrained	217		
Layer 4 sand (L3)	🟧	115	Undrained	103		
Layer 5 clay	🟩	105	Undrained	500		
Layer 6 sand (L1)	🟥	115	Undrained	475		
Layer 6 sand (L2)	🟥	115	Undrained	336		
Layer 6 sand (L3)	🟫	115	Undrained	196		
Layer 7 clay	🟦	95	Undrained	400		
Layer 8 sand (L1)	⬜	115	Undrained	827		
Layer 8 sand (L2)	🟪	115	Undrained	635		
Layer 8 sand (L3)	🟦	115	Undrained	444		
Layer 9 sand	🟪	125	Mohr-Coulomb	0	35	0
Retaining Wall No. 2	⬛	150	Infinite strength			0

McMILLEN JACOBS ASSOCIATES

SLIDEINTERPRET 9.027

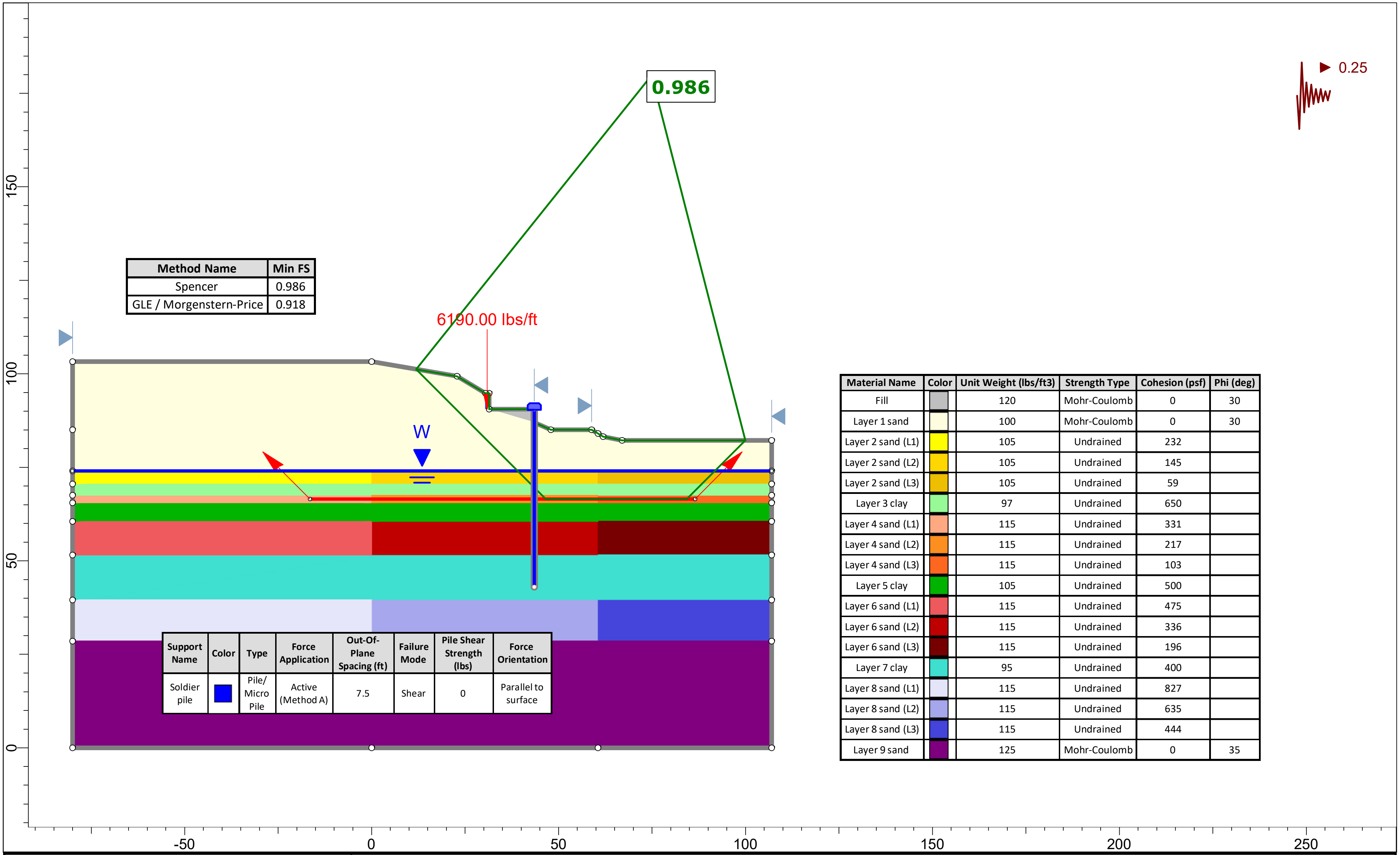
Project Name
TAMC - FORTAG Canyon Del Rey/SR218 Segement

Project No.
6231.0

Scenario
Liquefiable Layer at El 72.25, kh=0.25

Date
April 2023

Location
Del Rey Oaks, California



Method Name	Min FS
Spencer	0.986
GLE / Morgenstern-Price	0.918

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Fill	⬜	120	Mohr-Coulomb	0	30
Layer 1 sand	⬜	100	Mohr-Coulomb	0	30
Layer 2 sand (L1)	🟨	105	Undrained	232	
Layer 2 sand (L2)	🟨	105	Undrained	145	
Layer 2 sand (L3)	🟨	105	Undrained	59	
Layer 3 clay	🟩	97	Undrained	650	
Layer 4 sand (L1)	🟧	115	Undrained	331	
Layer 4 sand (L2)	🟧	115	Undrained	217	
Layer 4 sand (L3)	🟧	115	Undrained	103	
Layer 5 clay	🟩	105	Undrained	500	
Layer 6 sand (L1)	🟥	115	Undrained	475	
Layer 6 sand (L2)	🟥	115	Undrained	336	
Layer 6 sand (L3)	🟫	115	Undrained	196	
Layer 7 clay	🟦	95	Undrained	400	
Layer 8 sand (L1)	⬜	115	Undrained	827	
Layer 8 sand (L2)	🟪	115	Undrained	635	
Layer 8 sand (L3)	🟦	115	Undrained	444	
Layer 9 sand	🟪	125	Mohr-Coulomb	0	35

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Failure Mode	Pile Shear Strength (lbs)	Force Orientation
Soldier pile	🟦	Pile/ Micro Pile	Active (Method A)	7.5	Shear	0	Parallel to surface

Project Name
TAMC - FORTAG Canyon Del Rey/SR218 Segement

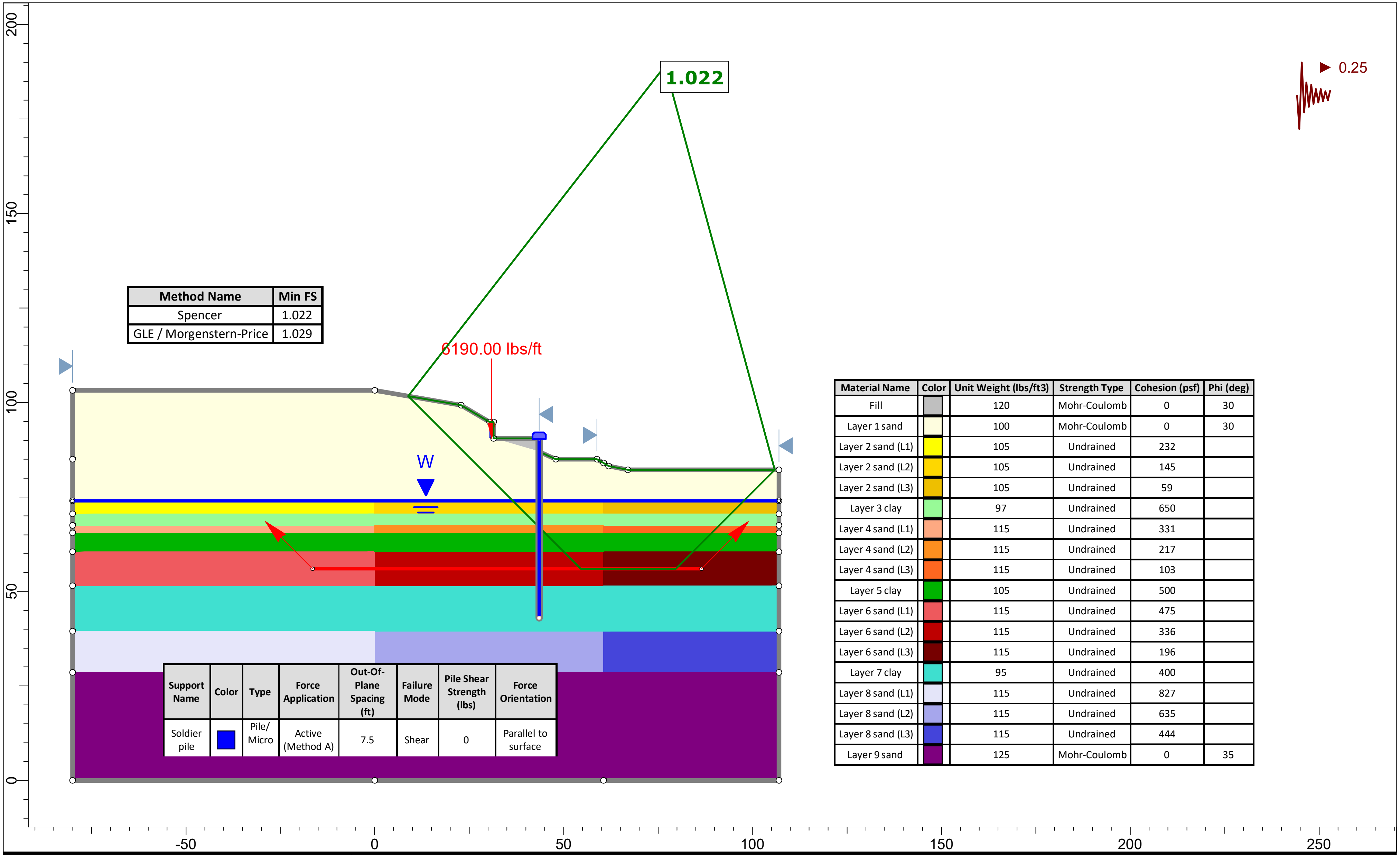
Project No.
6231.0

Scenario
Liquefiable Layer at El 66.5, kh=0.25

Date
April 2023

Location
Del Rey Oaks, California

SLIDEINTERPRET 9.027



Method Name	Min FS
Spencer	1.022
GLE / Morgenstern-Price	1.029

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Failure Mode	Pile Shear Strength (lbs)	Force Orientation
Soldier pile	🟦	Pile/ Micro	Active (Method A)	7.5	Shear	0	Parallel to surface

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Fill	⬜	120	Mohr-Coulomb	0	30
Layer 1 sand	⬜	100	Mohr-Coulomb	0	30
Layer 2 sand (L1)	🟨	105	Undrained	232	
Layer 2 sand (L2)	🟨	105	Undrained	145	
Layer 2 sand (L3)	🟨	105	Undrained	59	
Layer 3 clay	🟩	97	Undrained	650	
Layer 4 sand (L1)	🟧	115	Undrained	331	
Layer 4 sand (L2)	🟧	115	Undrained	217	
Layer 4 sand (L3)	🟧	115	Undrained	103	
Layer 5 clay	🟩	105	Undrained	500	
Layer 6 sand (L1)	🟥	115	Undrained	475	
Layer 6 sand (L2)	🟥	115	Undrained	336	
Layer 6 sand (L3)	🟫	115	Undrained	196	
Layer 7 clay	🟦	95	Undrained	400	
Layer 8 sand (L1)	⬜	115	Undrained	827	
Layer 8 sand (L2)	🟪	115	Undrained	635	
Layer 8 sand (L3)	🟦	115	Undrained	444	
Layer 9 sand	🟪	125	Mohr-Coulomb	0	35

McMILLEN JACOBS ASSOCIATES

Project Name
TAMC - FORTAG Canyon Del Rey/SR218 Segement

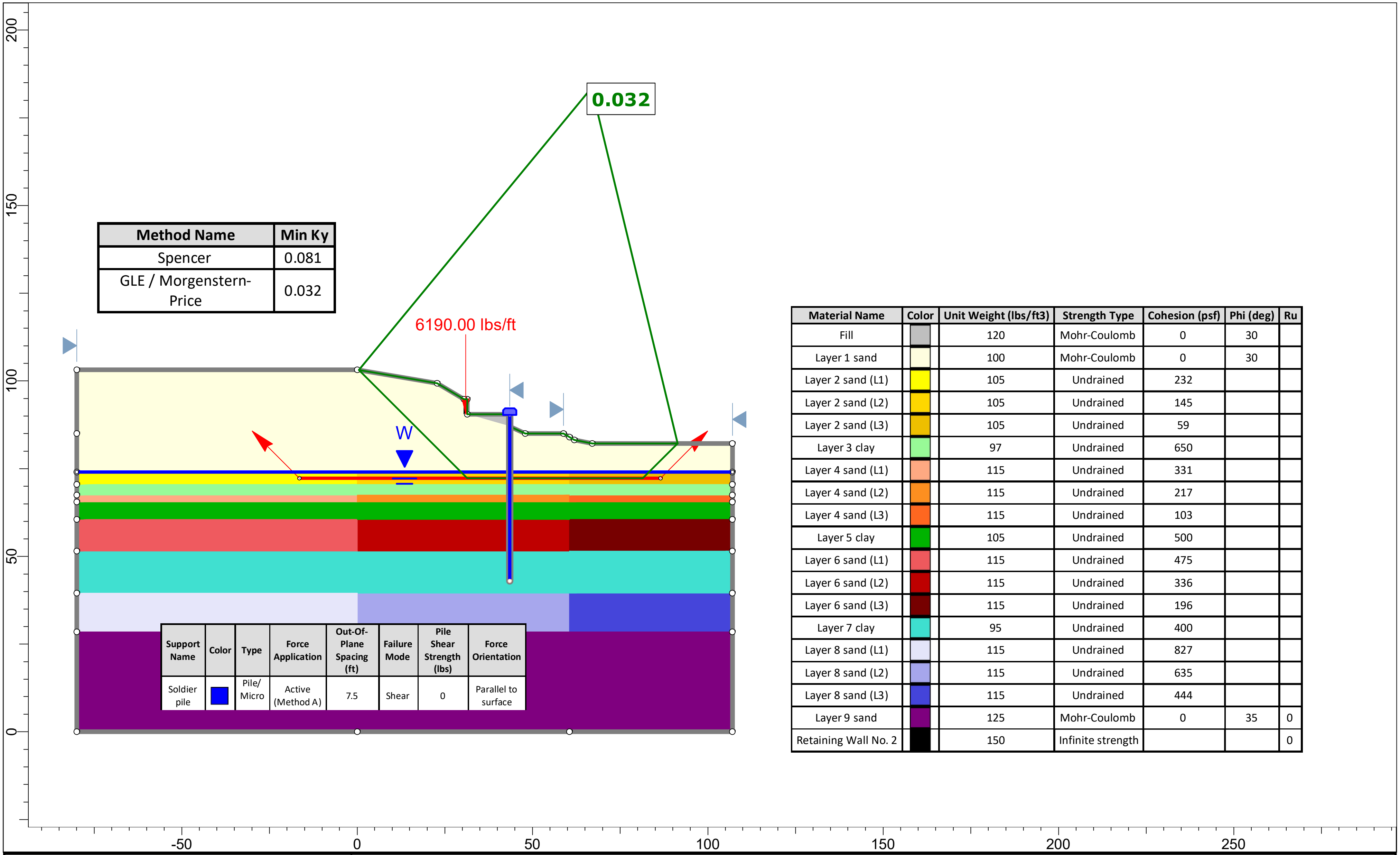
Project No.
6231.0

Scenario
Liquefiable Layer at El 56, kh=0.25

Date
April 2023

Location
Del Rey Oaks, California

SLIDEINTERPRET 9.027



Method Name	Min Ky
Spencer	0.081
GLE / Morgenstern-Price	0.032

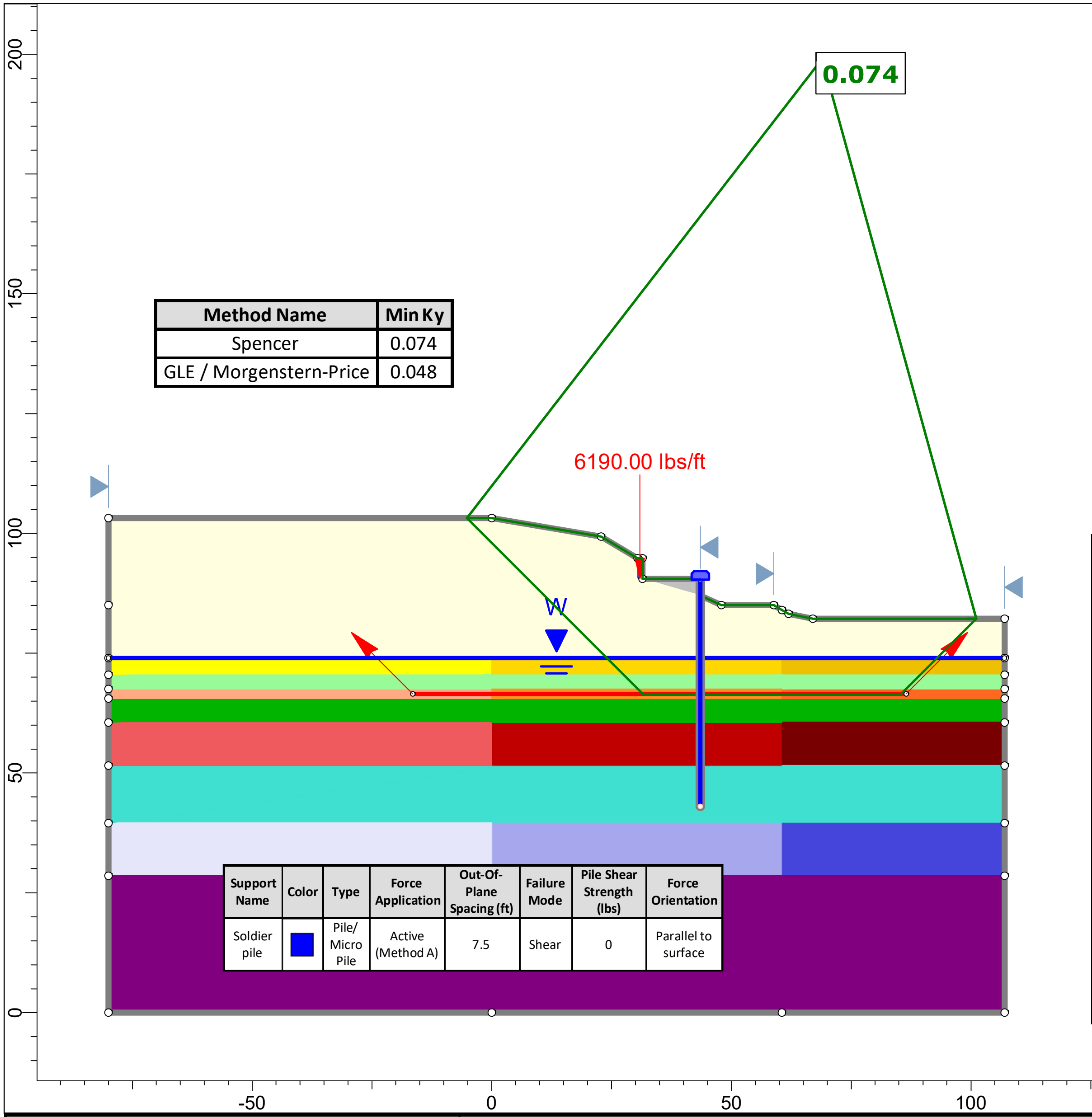
Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Failure Mode	Pile Shear Strength (lbs)	Force Orientation
Soldier pile	🟦	Pile/ Micro	Active (Method A)	7.5	Shear	0	Parallel to surface

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Ru
Fill	⬜	120	Mohr-Coulomb	0	30	
Layer 1 sand	⬜	100	Mohr-Coulomb	0	30	
Layer 2 sand (L1)	🟨	105	Undrained	232		
Layer 2 sand (L2)	🟨	105	Undrained	145		
Layer 2 sand (L3)	🟨	105	Undrained	59		
Layer 3 clay	🟩	97	Undrained	650		
Layer 4 sand (L1)	🟧	115	Undrained	331		
Layer 4 sand (L2)	🟧	115	Undrained	217		
Layer 4 sand (L3)	🟧	115	Undrained	103		
Layer 5 clay	🟩	105	Undrained	500		
Layer 6 sand (L1)	🟥	115	Undrained	475		
Layer 6 sand (L2)	🟥	115	Undrained	336		
Layer 6 sand (L3)	🟥	115	Undrained	196		
Layer 7 clay	🟦	95	Undrained	400		
Layer 8 sand (L1)	⬜	115	Undrained	827		
Layer 8 sand (L2)	🟪	115	Undrained	635		
Layer 8 sand (L3)	🟦	115	Undrained	444		
Layer 9 sand	🟪	125	Mohr-Coulomb	0	35	0
Retaining Wall No. 2	⬛	150	Infinite strength			0

McMILLEN JACOBS ASSOCIATES

Project Name	TAMC - FORTAG Canyon Del Rey/SR218 Segement		
Project No.	6231.0	*Scenario*	Liquefiable Layer at El 72.25, ky
Date	April 2023	*Location*	Del Rey Oaks, California

SLIDEINTERPRET 9.027



Method Name	Min Ky
Spencer	0.074
GLE / Morgenstern-Price	0.048

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Failure Mode	Pile Shear Strength (lbs)	Force Orientation
Soldier pile	🟦	Pile/ Micro Pile	Active (Method A)	7.5	Shear	0	Parallel to surface

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Fill	⬜	120	Mohr-Coulomb	0	30
Layer 1 sand	⬜	100	Mohr-Coulomb	0	30
Layer 2 sand (L1)	🟨	105	Undrained	232	
Layer 2 sand (L2)	🟨	105	Undrained	145	
Layer 2 sand (L3)	🟨	105	Undrained	59	
Layer 3 clay	🟩	97	Undrained	650	
Layer 4 sand (L1)	🟧	115	Undrained	331	
Layer 4 sand (L2)	🟧	115	Undrained	217	
Layer 4 sand (L3)	🟧	115	Undrained	103	
Layer 5 clay	🟩	105	Undrained	500	
Layer 6 sand (L1)	🟥	115	Undrained	475	
Layer 6 sand (L2)	🟥	115	Undrained	336	
Layer 6 sand (L3)	🟥	115	Undrained	196	
Layer 7 clay	🟦	95	Undrained	400	
Layer 8 sand (L1)	⬜	115	Undrained	827	
Layer 8 sand (L2)	🟦	115	Undrained	635	
Layer 8 sand (L3)	🟦	115	Undrained	444	
Layer 9 sand	🟪	125	Mohr-Coulomb	0	35

McMILLEN JACOBS ASSOCIATES

SLIDEINTERPRET 9.027

Project Name
TAMC - FORTAG Canyon Del Rey/SR218 Segement

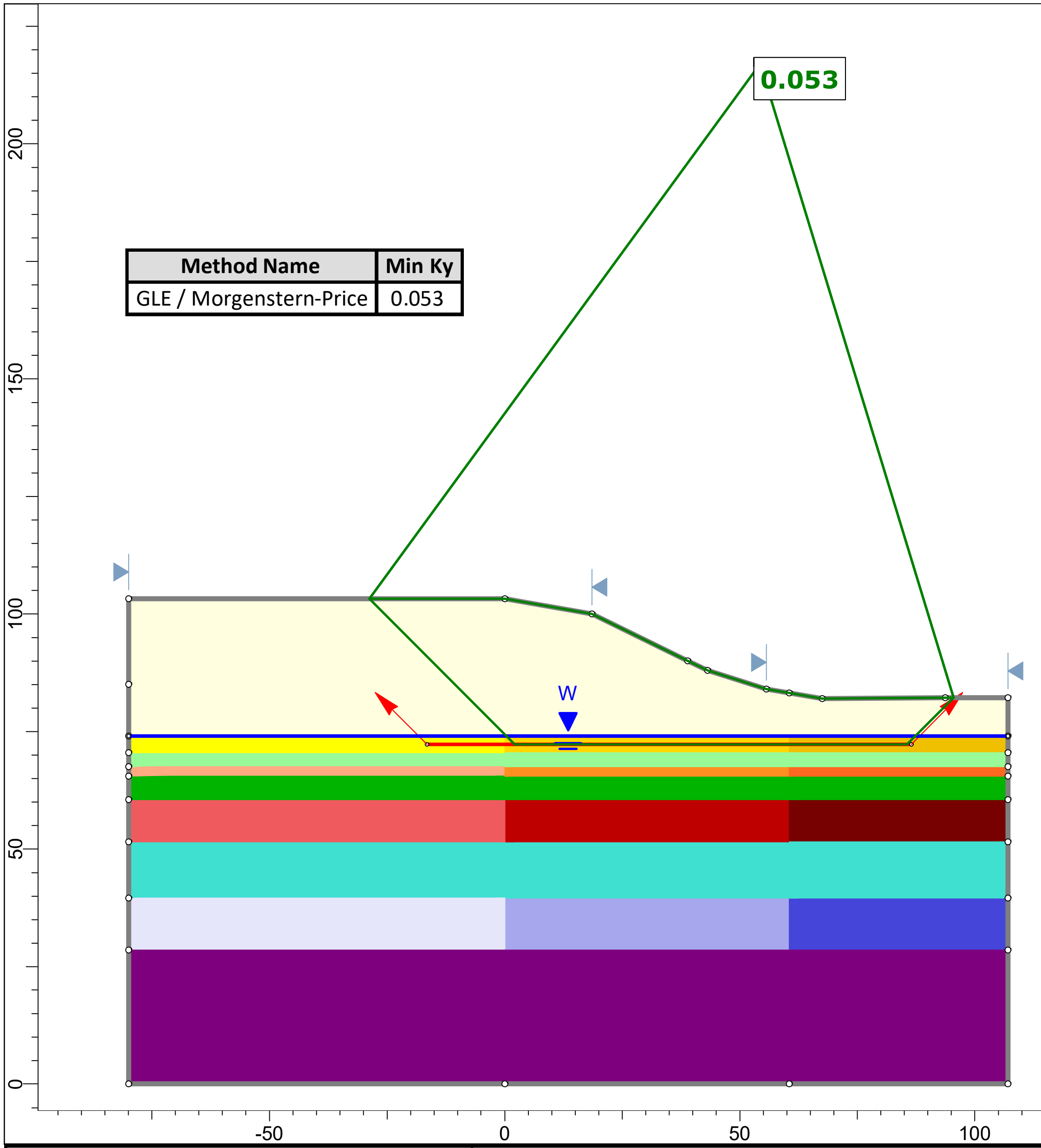
Project No.
6231.0

Scenario
Liquefiable Layer at El 65.5, ky

Date
April 2023

Location
Del Rey Oaks, California

Appendix H



Method Name	Min Ky
GLE / Morgenstern-Price	0.053

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Layer 1 sand	⬜	100	Mohr-Coulomb	0	30
Layer 2 sand (L1)	🟨	105	Undrained	232	
Layer 2 sand (L2)	🟨	105	Undrained	145	
Layer 2 sand (L3)	🟨	105	Undrained	59	
Layer 3 clay	🟩	97	Undrained	650	
Layer 4 sand (L1)	🟧	115	Undrained	331	
Layer 4 sand (L2)	🟧	115	Undrained	217	
Layer 4 sand (L3)	🟧	115	Undrained	103	
Layer 5 clay	🟩	105	Undrained	500	
Layer 6 sand (L1)	🟥	115	Undrained	475	
Layer 6 sand (L2)	🟥	115	Undrained	336	
Layer 6 sand (L3)	🟫	115	Undrained	196	
Layer 7 clay	🟦	95	Undrained	400	
Layer 8 sand (L1)	⬜	115	Undrained	827	
Layer 8 sand (L2)	🟪	115	Undrained	635	
Layer 8 sand (L3)	🟦	115	Undrained	444	
Layer 9 sand	🟪	125	Mohr-Coulomb	0	35

McMILLEN JACOBS ASSOCIATES

SLIDEINTERPRET 9.027

Project Name
TAMC - FORTAG Canyon Del Rey/SR218 Segement

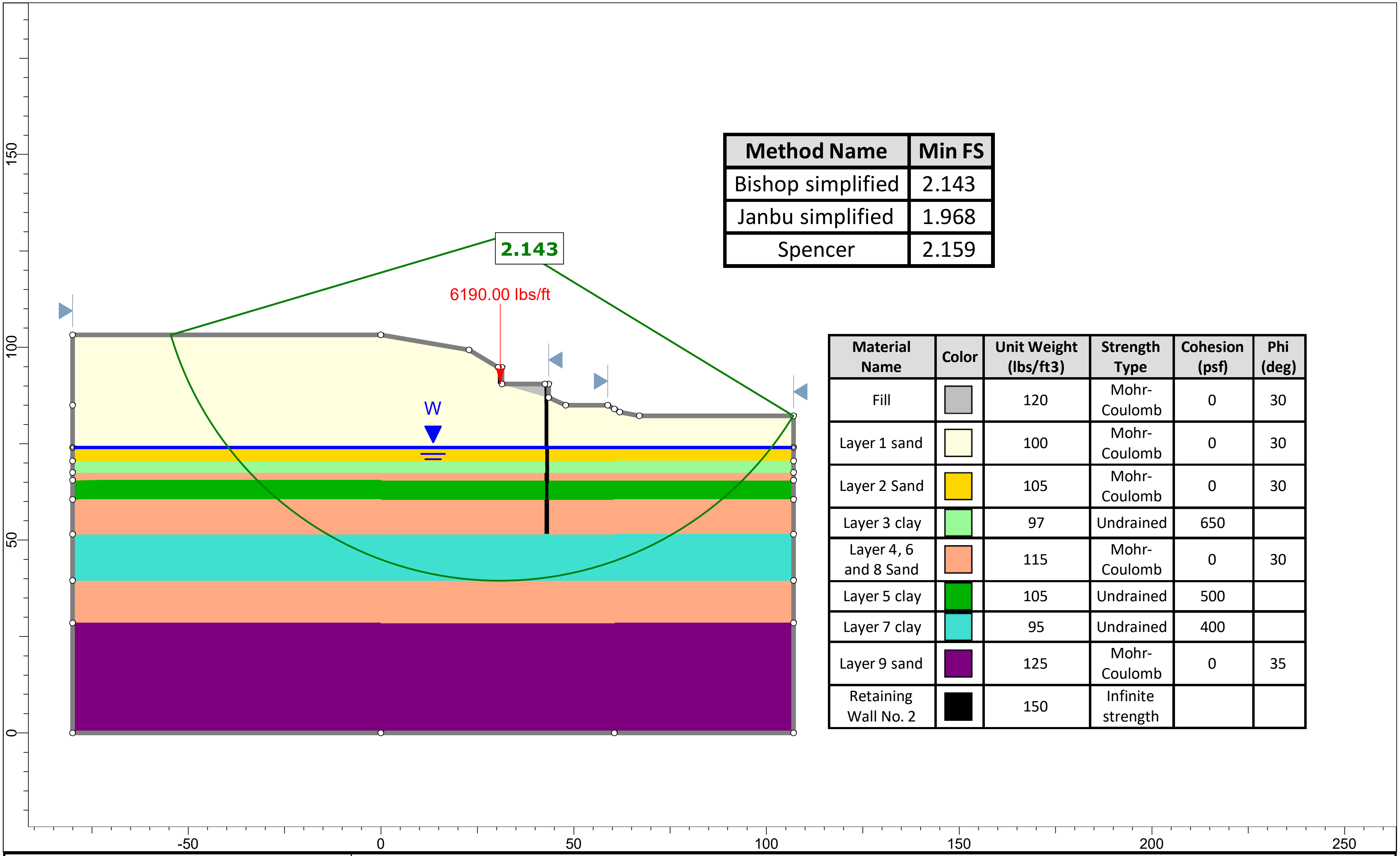
Project No.
6231.0

Scenario
Existing Slope - Khy

Date
April 2023

Location
Del Rey Oaks, California

Appendix I



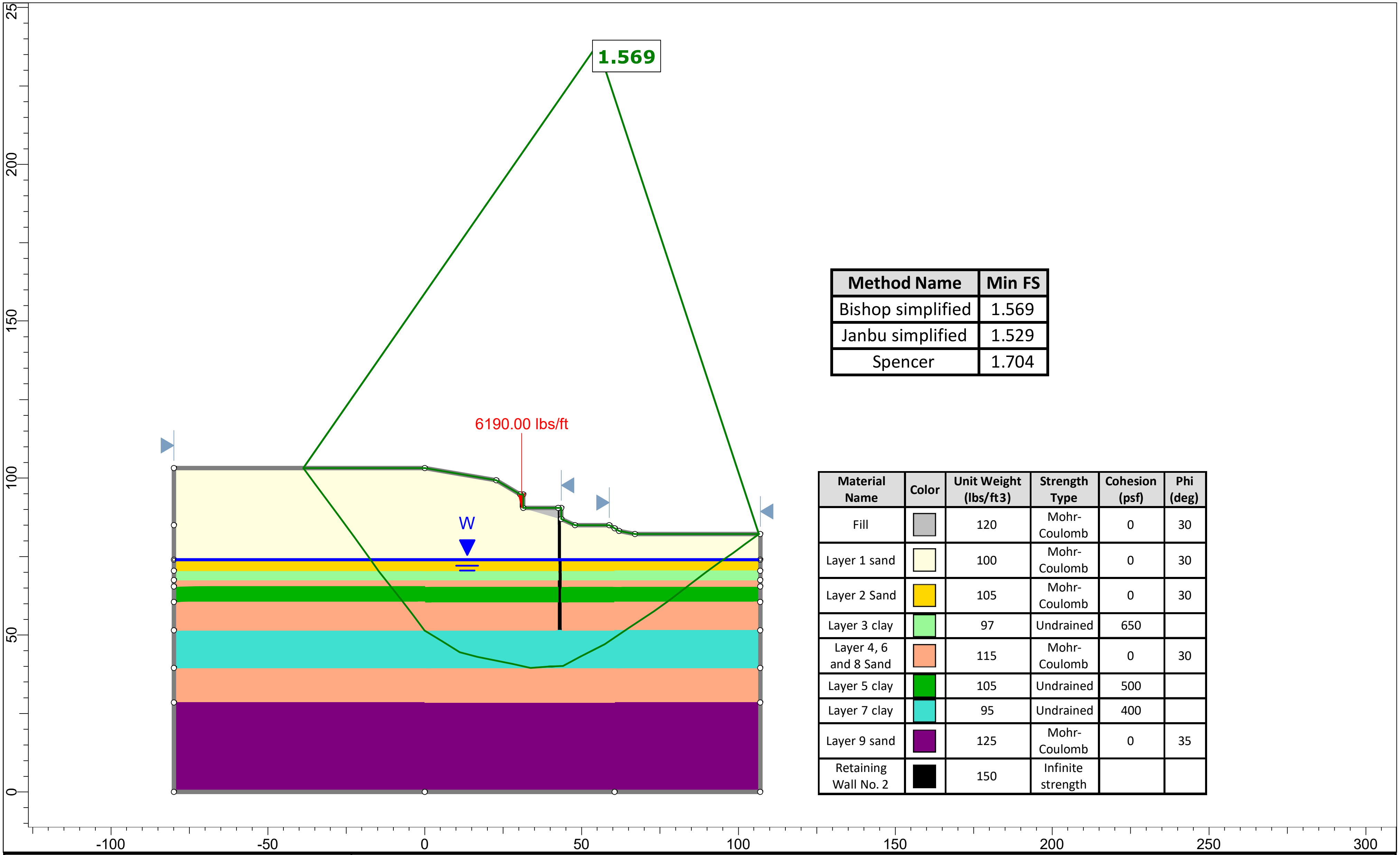
Method Name	Min FS
Bishop simplified	2.143
Janbu simplified	1.968
Spencer	2.159

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Fill	▨	120	Mohr-Coulomb	0	30
Layer 1 sand	⬜	100	Mohr-Coulomb	0	30
Layer 2 Sand	🟨	105	Mohr-Coulomb	0	30
Layer 3 clay	🟩	97	Undrained	650	
Layer 4, 6 and 8 Sand	🟧	115	Mohr-Coulomb	0	30
Layer 5 clay	🟩	105	Undrained	500	
Layer 7 clay	🟦	95	Undrained	400	
Layer 9 sand	🟪	125	Mohr-Coulomb	0	35
Retaining Wall No. 2	■	150	Infinite strength		

McMILLEN JACOBS ASSOCIATES

SLIDEINTERPRET 9.027

Project Name	TAMC - FORTAG Canyon Del Rey/SR218 Segement
Project No.	6231.0
Scenario	Global Stability - Circular Failure
Date	April 2023
Location	Del Rey Oaks, California



Method Name	Min FS
Bishop simplified	1.569
Janbu simplified	1.529
Spencer	1.704

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Fill	▨	120	Mohr-Coulomb	0	30
Layer 1 sand	⬜	100	Mohr-Coulomb	0	30
Layer 2 Sand	🟨	105	Mohr-Coulomb	0	30
Layer 3 clay	🟩	97	Undrained	650	
Layer 4, 6 and 8 Sand	🟧	115	Mohr-Coulomb	0	30
Layer 5 clay	🟩	105	Undrained	500	
Layer 7 clay	🟦	95	Undrained	400	
Layer 9 sand	🟪	125	Mohr-Coulomb	0	35
Retaining Wall No. 2	⬛	150	Infinite strength		

McMILLEN JACOBS ASSOCIATES

SLIDEINTERPRET 9.027

Project Name
TAMC - FORTAG Canyon Del Rey/SR218 Segement

Project No.
6231.0

Scenario
Global Stability - Block Failure

Date
April 2023

Location
Del Rey Oaks, California

Appendix J

Comment and Response Form

Review of Draft Foundation Report

McMillen Jacobs Associates, "Foundation Report for Bridge, Transportation Agency of Montery County, Fort Ord Regional Trail & Greenway - Retaining Wall No. 1, Del Rey Oaks, California" dated March 7, 2023

05-MON-218 EA 05-1M570 EFIS 0520000029 Phase/Sub-Object 1/100

Reviewer's Name/Unit	Comments/Questions Please reference document section (e.g., paragraph, page #, etc.)	Circulator's Response to Comments/Questions
Chris McMahon/59-3660	Signature Block: The signature block on the draft report does not include a California-licensed Professional Geologist (PG) or Certified Engineering Geologist (CEG). A PG (or CEG) must sign and stamp the final version of this report.	added
Chris McMahon/59-3660	Section 3.7.3 Fault Rupture: This section discusses several faults (Reliz, Monterey Bay-Tularcitos, Chupines, and Seaside, but does not clearly state where any of them are located relative to the planned structure. Please revise to indicate whether or not the project site is located within a mapped Alquist-Priolo Earthquake Fault Zone (or other fault hazard zone), or within 1,000 feet of an unzoned fault that is Holocene or younger in age, and note if the planned structure is susceptible to fault rupture hazards per Caltrans Memo To Designers 20-10 (MTD 20-10).	revised
Chris McMahon/59-3660	Section 3.7.4 Liquefaction: The results of the analysis should be tabulated in this section. Additionally, there are multiple instances of "Error! Reference source not found." in the text of the reviewed PDF.	revised
Chris McMahon/59-3660	Section 4.2.1 Global Stability Analysis: The results of the analysis should be tabulated in this section.	revised
Chris McMahon/59-3660	Section 4.2.2 Liquefaction-Induced Lateral Spreading Mitigation: The results of the analysis should be tabulated in this section. Additionally, this section is incomplete, and does not include the mitigation recommendations.	Result in Table 14. Section 6.2.2 was revised per our discussion with Caltrans
Chris McMahon/59-3660	Section 4.2 Design Considerations: Liquefaction (outside of lateral spreading) is not discussed, despite the potential for nearly a half-foot of settlement?	See Section 6.2.1
Justin Anderson/59-3660	General Note: The report sections should match the sections outlined in the Geotechnical Manual - Foundation Reports for ERS. This includes organizing the report material as recommended	revised
Justin Anderson/59-3660	Section 3.1 Geologic Unit Mapping: References a Section 0, which doesn't exist	I do not see Section 0 referenced in Section 3.1. This may be an error. We've fixed all the referencing errors
Justin Anderson/59-3660	Section 3.5 Groundwater Level: Please provide a recommended groundwater table elevation for design	revised
Justin Anderson/59-3660	Section 3.7.1 Site Seismic Parameters: Based on the provided boring logs, the suggested VS30 seems unlikely. Please calculate the shear wave velocity.	revised
Justin Anderson/59-3660	Section 3.7.2 Ground Motion Parameters: Mean Site-to-Fault Source Distance is based on all nearby faults, but the note suggests it's based on the San Gregorio Fault. Please clarify	It was based on ARS online tool, so it was based on all nearby faults.

Reviewer's Name/Unit	**Comments/Questions** **Please reference document section (e.g., paragraph, page #, etc.)**	**Circulator's Response to Comments/Questions**
Justin Anderson/59-3660	Section 4.1 Ground Material Properties, Models, and Loading: kh is presented here as 0.3g, whereas in the lateral spreading section, kh is presented as 0.25g. Which is correct?	kh calculated per AASHTO Section 11.6.5.2 was 0.28g and I rounded up to 0.3g. I'll revise this to 0.28g. Per AASHTO Sections 11.6.5.2, kh is equal to 0.5kh0, where kh0 = Fpga PGA. Fpga in our case is 1.1 per ASCE 7-16 (Site Class D with PGA of 0.5). 0.25g used in lateral spreading section is for the stability analysis per Caltrans guideline. Based on my understanding, 0.28g considered factor of safety of 1.1 while 0.25g is for factor of safety of 1.
Justin Anderson/59-3660	Section 4.1 Ground Material Properties, Models, and Loading: Please provide arching factors	Provided in the last paragraph for Section 6.1.
Justin Anderson/59-3660	Section 4.2 Retaining Wall No. 1 Design Considerations: is there a minimum lagging embedment that you recommend?	There is no minimum lagging embedment into the ground. Laggings are only to support the active earth pressures from retained soil.
Justin Anderson/59-3660	Section 4.2.2 Liquefaction-Induced Lateral Spreading Mitigation: Suggest checking multiplen sliding block methods as outlined in AASHTO LRFD Bridge Design Specifications, 8th Edition	Analyses performed using two methods.
Justin Anderson/59-3660	Section 4: Are there any notes for specifications? Perhaps expected difficult pile installation?	Added