Fort Ord Regional Trail & Greenway
SR 218 Undercrossing Bridge
Del Rey Oaks, California

(Post Mile MON 0.921)

Foundation Report for Bridge

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 Bridge
Transportation Agency of Monterey County
Fort Ord Regional Trail & Greenway – SR 218
Undercrossing Bridge
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) SR218 Undercrossing Bridge of 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

Su Pyae Sone Soe, PE
Project Engineer

Dru Nielson, PG CEG
Principal Geologist

Norman Joyal, PE GE
Principal Engineer

PROFESSIONAL GEOLOGIST
DRU R. NIELSON
No. 5651
4-6-2023
STATE OF CALIFORNIA

CERTIFIED ENGINEERING GEOLOGIST
DRU R. NIELSON
No. 1854
4-6-2023
STATE OF CALIFORNIA

LICENSED PROFESSIONAL ENGINEER
NORMAN A. JOYAL
C 31821
4-6-2023
CIVIL
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
NORMAN A. JOYAL
GE 449
4-6-2023
GEOTECHNICAL
STATE OF CALIFORNIA

Distribution: GHD (PDF) – Email Delivery
Job No.: 6231.0

Table of Contents

List of Tables

List of Figures

Appendices

1.0 Introduction

McMillen Jacobs Associates (MJA) has prepared this foundation report for the Transportation Agency of Monterey County's (TAMC) planned State Route (SR) 218 Undercrossing Bridge (bridge) portion 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 analyses for the project and provides geotechnical design recommendations for the planned SR 218 undercrossing bridge at SR 218 Post Mile Mon 0.921 (the project area). The recommendations presented herein are based on the Bridge General Plan drawing dated 6/24/22 (General Plan), Foundation Plan drawing dated 1/23/23 from Cornerstone Structural Engineering Group (Cornerstone), and our interpretation of the geotechnical findings for the project area that are summarized herein Sections 3.0 and 5.0 herein. In addition to this report, we have provided two separate and independent reports: a Final Geotechnical Design Report (GDR) for the project and a Geotechnical Foundation Recommendations Report for Retaining Wall No. 1 (see MJA 2023a and 2023b, respectively).

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 (MJA, 2021a). Initially, the trail was planned to cross under SR 218 within a structure to be installed by trenchless or tunneling methods. The geotechnical investigation that was initially completed for the project was scoped based on that initial plan. Subsequent to the completion of project borings B-4 and B-5b, it was determined by the design team that conflicts with existing utilities and elevation requirements for the planned undercrossing would prevent the installation of an undercrossing structure by trenchless or tunneling methods, and that therefore, an undercrossing bridge would be necessary (see Section 5.3.1).

The scope of the initial geotechnical investigation for the project included 40-foot-deep exploration borings (B-4 and B-5b, as described in Sections 3.0 and 5.3) at the planned SR 218 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 below that of the peat layer encountered in boring B-4, for the purpose of designing driven-pile support for the planned bridge.

Four CPTs were planned for the project, one at each planned abutment corner of the bridge, but only two CPTs could be advanced, both on the south side of the bridge at the locations illustrated in Figure 1. The two planned CPTs on the north side of the bridge encountered sand consistent with utility backfill in the hand-auger holes excavated to clear the upper 5 to 6 feet prior to advancing the CPT. For safety, the CPT contractor's policy is to not advance CPTs in material that is consistent with trench backfill materials; consequently, the two planned CPTs on the north side of the bridge were not completed. Project CPT data is provided in Appendix D.

The current project plans call for the SR 218 undercrossing bridge to span approximately 42.5 feet along SR 218 (i.e., northwest to southeast) and 57.5 feet across SR 218 (i.e., northeast to southwest; see Figure 1). Elevations and coordinates referred to in this report are based on the 1988 North American Vertical Datum (NAVD 88) and the 1984 World Geodetic System (WGS 84), respectively.

Table 1. Summary of the Project Components Addressed in this Report

Component	SR 218 Stations [1]	Walking Trail Stations		Length (feet)	Maximum Height (feet)	Notes
		Begin	End			
SR 218 Undercrossing Bridge	104+85 to 105+27	200+82	201+40	42.5	-	Minimum vertical clearance of 10.5 feet for the walking trail

[1] SR 218 and walking trail stationing based on project drawings by Cornerstone (2022).

3.0 Geotechnical Investigation

The locations of completed exploration borings and CPTs for the project are shown in Figure 1. Boring log legends and boring logs for the project are provided in Appendices A and B, respectively, and the CPT results for the project are provided in Appendix D. Table 2 summarizes information from project borings and CPTs. The detailed descriptions of the field explorations are provided in the GDR prepared by MJA (2023a).

Table 2. Partial Summary of Borehole/CPT Data

Boring/ CPT [(1)]	Nearby Planned 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[(4)]	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[(5)]	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). Borings B-1, B-2, and B-3 were performed in areas away from the planned SR 218 Bridge and Retaining Wall No. 1. Their logs are provided in MJA (2021b).

(2) From Google Earth.

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

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

(5) CPT rig shifted when refusal was encountered on suspected concrete and was abandoned and relocated to CPT-1B.

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; a province is characterized by northwest-trending mountain ranges and valleys that run subparallel to the trend of the region's fault zones. The region's fault zones are summarized in Section 5.7.3. 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 planned 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.

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 northeast 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 planned 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 planned bridge. This slope has a gradient of less than 1.5H:1V, and heights between 5 and 12 feet.

[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. Data from CPT-1B and CPT-2 suggest the presence of this peat layer at a depth between 35 and 50 feet (i.e., where the CPT cone and sleeve resistances are nearly zero).

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

There may be other utilities in the area that are not shown in these figures. Except as noted in this report, 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 along the north shoulder side of SR 218.

5.3.2 Project Borings and CPTs

The location of borings and CPTs completed for the project are mapped in Figure 1, and schematic subsurface profiles of the project are provided in Figures 5.1 through 5.3. Boring log legends and the logs of the borings are provided in Appendix A and Appendix B, respectively, and CPT results are provided in Appendix D. A partial summary of information from the project boring logs and CPTs is provided in Table 3 and Table 4, respectively.

Table 3. Partial Summary of Information from Project Borings

Boring[1]	Top 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, γ_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, γ_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 Projects CPTs

CPT[1]	Top 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 about 2 to 6 feet thick, prominent layer 35 to 50 feet bgs.
CPT-2	88.5	66.5	14.6	sand mixtures, sands, silt mixtures, clays	N60 value ranging from 1 to 66 bpf. Sensitive fine grained layers about 2 to 6 feet thick, prominent layer 35 to 50 feet bgs.

(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 measured and logged in project borings during and immediately after their drilling (see logs of borings in Appendix B) 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, groundwater table elevation of +74 feet should be considered for design purposes.

Table 5. Measured Groundwater Level Depth or Piezometric Elevation in Project Borings and CPTs

Boring/ CPT [1]	Ground Surface Elevation (ft) [2]	Groundwater Level Depth or Piezometric Elevation [3]		Date Measured (mo/yr)
		Depth BGS (feet)	Elevation (feet)	
B-4	84.5	31.0	53.5	8/2021
B-5a	95.5	NE	NE	8/2021
B-5b	95.5	26.0	69.5	8/2021
CPT-1A	90.0	2.4	NE	2/2023
CPT-1B	91.5	17.6	74.0	2/2023
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. BGS = below ground surface.

5.5 Scour Data

The bridge does not span a watercourse, therefore there is no scour potential.

5.6 Corrosion Evaluation

As is indicated in Figure 3, native soils in the project area have low to high corrosivity potential. We had tests for corrosivity performed on one soil sample obtained during the subsurface exploration for the project. The results of the corrosivity tests are provided on Table 6 and in Appendix C. Based on the criteria provided in Caltrans corrosion guidelines (Caltrans, 2021), the results of that test indicate that the sampled soil does not meet the definition of a corrosive environment.

Table 6. Soil Corrosion Test Summary

Boring	Elevation (feet)	Minimum Resistivity (Ohm-cm)	pH	Chloride Content (ppm)	Sulfate Content (ppm)
B-4	74	1908	7.1	29	417

5.7 Seismic Information

5.7.1 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 (Vs_{30}) [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.

5.7.2 Ground Motion Parameters

Design ground motion parameters for the project are provided in Table 8 and in 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/.

5.7.3 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).

5.7.4 Liquefaction

Liquefaction develops when cyclically induced ground stresses increase pore water pressure within 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 and Boulanger, 2008). The most susceptible soils of the project area are fills, and recent 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 5.2.1 and in Figure 3. The Dupre (1990) map shows that the bridge is located in 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 evaluate liquefaction potential in the project area, CPTs were pushed to refusal (see final depths in Table 2). A numerical analysis (model) of liquefaction triggering was performed with data from the CPT using CLiq v.3.0 (GEOLOGISMIKI, 2007). Input for the model included soil parameters, soil layer thicknesses, earthquake magnitude, peak ground acceleration, and assumed groundwater depth below the surface from recent CPTs as summarized in Table 9.

Table 9. Summary of Liquefaction Input Parameters

Exploration(1)	Assumed Groundwater Depth during Earthquake(2) (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.
(2) Using a high winter time groundwater level, which is conservative relative to summer time levels reflected in the borings.

Based on liquefaction evaluation guidelines provided in Caltrans Geotechnical Manual (Caltrans, 2020c), the liquefaction analysis procedure from Youd and Idriss (2001) was used. 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 5.1 and 5.2. The results of CPT-based analysis of both CPT-1 and CPT-2 determined that potentially 5 inches of settlement could occur during a magnitude 6.81 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	

CPT	Approximated Liquefaction Elevation (feet)	Layer Thickness (feet)	Estimated Seismic-induced Settlement (inches)
	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	

5.7.5 Liquefaction-Induced Lateral Spreading

Lateral spreading is caused by the accumulation of incremental displacements towards a geologic free face (e.g., slope downgradient of the bridge) that develops 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 5.7.4, 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 groundwater elevation encountered in the recent CPTs at approximately El. +74 feet, which is below the bottom of the free face elevation (El.+80 feet) at the bridge location. However, multiple liquefiable soil layers a few feet thick were encountered within a depth of 1.2H from the bottom of the slope (where H is the height of the slope), and therefore multiple failure surfaces were considered within this depth range, per Figure 2 in Caltrans Memo to Designers (MTD) 20-15 (2017).

The side slopes across the bridge are approximately 15 feet high. As explained above, failure surfaces within liquifiable layers were considered down to El. 60 (1.2H below the bottom of the slope). According to Caltrans Memo to Designers (MTD) 20-15 (2017), pseudo-static slope stability analysis was performed using liquefied conditions 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 top of the slope, and vertically to no more than 1.2H depth from the bottom of the slope, per Figure 2 in Caltrans MTD 20-15. The methodology followed the procedure outlined in the example problem in the Caltrans Geotechnical Design Manual (Caltrans, 2020b) and using the software program Slide 2 (Rocscience, 2022).

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 or Undrained Shear Strength, S_u or S_r (psf)
93.0–74.0	Sand	100	30	c = 0
74.0–70.5	Sand (Liquefied Layer)	105	$\Phi_u = 0$	$S_r = 171$
				$S_r = 145$
				$S_r = 59$
70.5–67.5	Clay	97	$\Phi_u = 0$	$S_u = 650$
67.5–65.5	Sand (Liquefied Layer)	115	$\Phi_u = 0$	$S_r = 251$
				$S_r = 217$
				$S_r = 103$
65.5–60.5	Clay	105	$\Phi_u = 0$	$S_u = 500$
60.5–51.5	Sand (Liquefied Layer)	115	$\Phi_u = 0$	$S_r = 377$
				$S_r = 336$
				$S_r = 196$
51.5–39.5	Clay	95	$\Phi_u = 0$	$S_u = 400$
39.5–28.5	Sand (Liquefied Layer)	115	$\Phi_u = 0$	$S_r = 693$
				$S_r = 635$
				$S_r = 444$
Below 28.5	Sand	125	35	c = 0

The first step was running a pseudo-static slope stability analysis with no horizontal seismic coefficient (k_h) applied. Results showed the Factor of Safety of greater than 1.0, meaning that down to a depth of 1.2H below 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 k_h of 0.5g, the design peak horizontal ground motion per guidelines provided in Caltrans Geotechnical Manual for Liquefaction-Induced Lateral Spreading (Caltrans, 2021e). Results showed the Factor of Safety is greater than 1.0 for liquefiable layer at El 66.5, meaning that the abutment is not susceptible to lateral spreading hazard due to the liquefaction at El 66.5 layer. However, the Factor of Safety for liquefiable layer at El 72.25 was slightly less than 1.0, meaning that there is a potential for slope failure under seismic loading from El 72.25 failure plane. Therefore, stability analysis was performed on another scenario where the slope in front of the abutment and the bike trail were backfilled using Class 2AB. Result showed that the Factor of Safety for this scenario is greater than 1.0, meaning the potential for slope failure under seismic loading can be mitigated by using Class 2AB as a backfill material for the slope in front of the abutment and the bike trail. Therefore, the backfill in front of the abutment should follow the recommendations provided in Section 6.3.2. 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	Failure Type	Liquefiable Layer Elevation (ft)	Support Resistance, R_{tot} (kips)	Factor of Safety	Required Factor of Safety
Liquefied Condition with $k_h = 0$	Block Failure	72.25	0	1.082	1.0
Liquefied Condition with $k_h = 0$	Block Failure	66.5	0	1.229	1.0
Liquefied Condition with $k_h = 0.5g$	Block Failure	72.25	0	0.975	1.0
Liquefied Condition with $k_h = 0.5g$ Class 2AB Backfill	Block Failure	72.25	0	1.203	1.0
Liquefied Condition with $k_h = 0.5g$	Block Failure	66.5	0	1.376	1.0

6.0 Geotechnical Recommendations

Recommendations provided herein are intended for design and construction of the bridge 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.

The SR 218 undercrossing bridge will be supported on 16-inch-diameter (1/2-inch wall thickness), open-ended, steel pipe piles (i.e., Caltrans Class 200 "Alt W" piles) to mitigate the liquefaction-induced settlement. Driven pile foundations designed herein are based on the subsurface conditions provided in Section 5.0, especially the CPTs performed in February 2023. CPTs (i.e., CPT-1 and CPT-2) were pushed to elevation 28 feet and 22 feet, respectively, where they hit refusal. Therefore, the minimum design values from the last 5 feet of CPT data were extrapolated down for design of the pile sections below El. 22 feet.

6.1 Driven Pile Foundations

The single-span bridge will be supported by two bridge abutments, and each abutment will be supported on 32 piles. The minimum center-to-center pile spacing is 6.5 feet (i.e., about 4.9B), which exceeds our minimum 3B center-to-center pile spacing recommendations. Per Caltrans MTD 3-1 (Caltrans, 2014), pile design was completed using LRFD methods in accordance with the California Amendments to the American Association of State Highway and Transportation Officials (AASHTO) LRFD Bridge Design Specifications (BDS) with California Amendments (Caltrans LRFD BDS) (Caltrans, 2019). Foundation design information provided by the structural designer (SD) is summarized in Table 13 and Table 14.

Table 13. Foundation Design Data Sheet

Support No.	Pile Type	Finished Grade Elevation (feet)	Cut-off Elevation (feet)	Pile Cap Size (feet)		Permissible Settlement under Service Load (inches)	Number of Piles per Support
				B	L		
Abutment 1	16" Driven Steel Pipe Piles (Caltrans Class 200 "Alt W")	-	80.5	8.33	62.33	2.0	32
Abutment 2		-	80.5	8.33	62.33	2.0	32

Table 14. Foundation Factored Design Loads

Support No.	Service-I Limit State (kips)			Strength/Construction Limit State (Controlling Group, kips)				Extreme Event Limit State (Controlling Group, kips)			
	Total Load		Permanent Loads	Compression		Tension		Compression		Tension	
	Per Support	Max Per Pile		Per Support	Max Per Pile	Per Support	Max Per Pile	Per Support	Max Per Pile	Per Support	Max Per Pile
Abut 1	1,440	80	1,250	1,940	110	-	25	1,250	120	0	45
Abut 2	1,440	80	1,250	1,940	110	-	25	1,250	120	0	45

6.1.1 Axial Pile Resistance

Axial pile resistance was calculated using CPT-based procedures (Eslami and Fellenius Method and Nottingham and Schmertmann Method) presented in the FHWA Design and Construction of Driven Piles Manual (FHWA, 2016). The CPT data were obtained during our recent investigation completed near the proposed SR 218 undercrossing bridge. The axial pile analysis results from Eslami and Fellenius Method, which give more conservative values, for both Strength Limit State and Extreme Event Limit State are presented in Appendix H. Axial pile capacity of the piles for Strength Limit State design is mainly derived from frictional interaction between the pile surface and the surrounding soil (i.e., skin friction). The end bearing at the pile tip was ignored for Strength Limit State design since the pile tip is within highly weathered Monterey Formation.

Due to potentially liquefiable soils and anticipated liquefaction-induced settlement discussed in Section 5.7.4, our analyses considered the effect of downdrag in addition to the structural demand for the Extreme Event Limit State design. 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). Downdrag load considered in the Extreme Event Limit State design is summarized in Table 15. Axial pile capacity of the piles for Extreme Event Limit State design is primarily derived from frictional interaction between the pile surface and the surrounding soil (i.e., skin friction). Approximately one-third of the nominal axial pile resistance is derived from end bearing at the pile tip. Our analysis conservatively assumed that a soil plug will not develop during driving, thereby limiting the end bearing area to the area of the steel instead.

The design recommendations for the SR 218 undercrossing bridge pile foundations are presented in Table 16.

Table 15. Extreme Event Limit State Design with Downdrag

Support No.	Pile Type	Downdrag Zone Bottom Elevation (ft)	Estimated Downdrag Load (kips/pile)	Required Compression Resistance with Downdrag (kips/pile) [(1)]
Abut 1	16” Driven Steel Pipe Piles	36	110	150
Abut 2		36	110	150

[(1)] Combination of the maximum possible liquefaction-induced downdrag load and the ever-present permanent load.

A

Table 16. Foundation Design Recommendations

Support No.	Pile Type	Cut-Off El. (ft)	Service-I Limit State Load per Support (kips)		Total Permissible Support Settlement (in)	Required Nominal Resistance (kips)				Design Tip Elevation [1] (ft)	Specified Tip Elevation (ft)	Required Nominal Driving Resistance [2] (kips)
						Strength Limit		Extreme Event				
			Total	Perm		Comp. (ϕ_{qs}=0.7)	Tension (ϕ_{qs}=0.7)	Comp. (ϕ_{qs}=1)	Tension (ϕ_{qs}=1)			
Abut 1	16" Driven Steel Pipe Piles	80.50	1440	1250	2.0	160	40	120	50	26.5 (a-I)[4] 30.5 (b-I) 22.5 (a-II) 48.5(b-II) TBD (d)	22.5	210
Abut 2		80.50	1440	1250	2.0	160	40	120	50	26.5 (a-I) 30.5 (b-I) 22.5 (a-II) 48.5(b-II) TBD (d)	22.5	210

(1) Design tip elevations are controlled by the following: (a-I) Compression (Strength Limit); (b-I) Tension (Strength Limit); (a-II) Compression (Extreme Event); (b-II) Tension (Extreme Event); and (d) Lateral Load (design tip elevation for lateral load was determined to be 29 feet for both abutment by SD).

(2) Nominal driving resistance estimated based on the pile tip elevation determined in Extreme Event Compression Loading Case (a-II).

(3) Pile tip resistance was not considered for (a-I) Compression strength limit design since the pile tip is within highly weathered Monterey Formation.

6.1.1 Foundation Settlement

The total long-term service settlement of the abutments supported by on sixteen driven piles specified in Table 13, founded at specified tip elevation of 22.5 feet, and subjected to the total design load of about 80 kips per pile is calculated to be less than the specified permissible settlement under service load (2 inches) provided in Table 13. Differential settlement between the two abutments may be taken as 0.5 inch. The estimated total foundation settlement will likely occur immediately upon loading after the completion of the foundation installation since the piles are embedded in sand with thin layers of clay.

6.1.2 Lateral Pile Analysis Parameters

The lateral pile capacity analysis will be performed by the SD using the software program LPILE by Ensoft. Table 17, Table 18, and Table 19 present our recommended LPILE parameters for static loading conditions, as well as for two seismic loading cases: (1) Using post-liquefaction residual shear strengths (Boulanger and Idriss, 2014) for the liquefiable soils, and (2) using p-multipliers to account for post-liquefaction strength loss in the liquefiable soils (Caltrans, 2012).

Table 17. Recommended LPILE Parameters – Static Conditions

Depth below Pile Head (feet)	Elevation (feet)	LPILE Soil Type/Model	Effective Unit Weight (pcf)	Friction Angle, Φ (°)	Undrained Shear Strength, S_u (psf)	K value (pci)	ε_{50}
0–6	80.0–74.0	Sand (Reese et al., 1974)	100	30	-	25	-
6–7.5	74.0–72.5	Sand (Reese et al., 1974)	43	30	-	20	-
7.5–19.5	72.5–60.5	Soft Clay (Matlock, 1970)	38	-	650	-	0.02
19.5–31.5	60.5–48.5	Sand (Reese et al., 1974)	55	30	-	60	-
31.5–44.5	48.5–35.5	Soft Clay (Matlock, 1970)	38	-	400	-	0.02
> 44.5	Below 35.5	Sand (Reese et al., 1974)	63	35	-	60	-

Table 18. Recommended LPILE Parameters – Seismic Conditions With Residual Strengths

Depth below Pile Head (feet)	Elevation (feet)	LPILE Soil Type/Model	Effective Unit Weight (pcf)	Friction Angle, Φ (°)	Undrained Shear Strength, S_u (psf)	K value (pci)	ε_{50}	Residual Strength, S_r (psf)
0–6	80.0–74.0	Sand (Reese et al., 1974)	100	30	-	25	-	-
6–9.5	74.0–70.5	Soft Clay (Matlock, 1970) Liquefiable Layer	43	-	-	-	0.02	100
9.5–12.5	70.5–67.5	Soft Clay (Matlock, 1970)	35	-	650	-	0.02	-
12.5–14.5	67.5–65.5	Soft Clay (Matlock, 1970) Liquefiable Layer	53	-	-	-	0.02	200
14.5–19.5	65.5–60.5	Soft Clay (Matlock, 1970)	43	-	500	-	0.02	-
19.5–28.5	60.5–51.5	Soft Clay (Matlock, 1970) Liquefiable Layer	53	-	-	-	0.02	250
28.5–40.5	51.5–39.5	Soft Clay (Matlock, 1970)	33	-	400	-	0.02	-
40.5–51.5	39.5–28.5	Soft Clay (Matlock, 1970) Liquefiable Layer	53	30	-	60	0.02	400
> 51.5	Below 28.5	Sand (Reese et al., 1974)	63	35	-	60	-	-

Table 19. Recommended LPILE Parameters – Seismic Conditions With P-Multipliers

Depth below Pile Head (feet)	Elevation (feet)	LPILE Soil Type/Model	Effective Unit Weight (pcf)	Friction Angle, Φ (°)	Undrained Shear Strength, S_u (psf)	K value (pci)	ε_{50}	p-multiplier
0–6	80.0–74.0	Sand (Reese et al., 1974)	100	30	-	25	-	-
6–9.5	74.0–70.5	Soft Clay (Matlock, 1970) Liquefiable Layer	43	30	-	20	-	0.042
9.5–12.5	70.5–67.5	Soft Clay (Matlock, 1970)	35	-	650	-	0.02	-
12.5–14.5	67.5–65.5	Soft Clay (Matlock, 1970) Liquefiable Layer	53	30	-	60	-	0.109
14.5–19.5	65.5–60.5	Soft Clay (Matlock, 1970)	43	-	500	-	0.02	-
19.5–28.5	60.5–51.5	Soft Clay (Matlock, 1970) Liquefiable Layer	53	30	-	60	-	0.099
28.5–40.5	51.5–39.5	Soft Clay (Matlock, 1970)	33	-	400	-	0.02	-
40.5–51.5	39.5–28.5	Soft Clay (Matlock, 1970) Liquefiable Layer	53	30	-	60	-	0.125
> 51.5	Below 28.5	Sand (Reese et al., 1974)	63	35	-	60	-	-

6.2 Lateral Earth Pressures for Abutments and Wingwalls

Walls that are not free to deflect should be designed for at-rest condition while the walls that are free to rotate may be assumed to be flexible for the active condition. The following design criteria apply to the walls that are a maximum of 15 feet in height with horizontal backfill and have a drainage system consisting of drain rock with perforated drainpipes or weep holes to prevent hydrostatic pressures that might be caused by water trapped behind the wall. The contractor can select appropriate geocomposite material as an alternative drainage system, and it should be placed per the manufacturer's guidelines. The walls meeting the criteria described above can be designed for the active and passive earth pressures provided in Table 20. Where the ground descends immediately below the toe of the structure, apply passive pressure on the downgradient side of the structure starting at 12 inches below the ground surface at the toe.

Table 20. Lateral Earth Pressures

Ultimate Static Lateral Earth Pressures[1] Expressed as Equivalent Fluid Density (psf/ft in a triangular distribution)	
At-rest Pressure	60
Active Pressure	36
Passive Pressure[2]	550

[1] Safety factors should be applied. Assumes structures are less than 15 feet deep. See Section 6.2 for additional applicable pressures. Pressures were calculated per AASHTO (2017).

[2] For passive pressures, a safety factor of at least 2.0 should be applied to avoid the lateral movement of the structure, which would be necessary to reach full ultimate passive soil strength mobilization. The passive pressures should not exceed 2,000 psf.

The following modifications to design lateral earth pressures should be made to both at-rest and active pressures provided in Table 20 where applicable:

- Dynamic pressures (P_e) from seismic shaking: A dynamic earth pressure of $P_e = 35 \times H$, expressed as pounds per square foot, should be applied as a triangular distribution over a depth of H (where H = depth of wall embedment below grade in feet). The resultant should be applied at a distance of 0.3H from the bottom of the structure.
- Lateral surcharge from equipment and vehicles (Figure 10), where it exceeds the dynamic earth pressure.

In addition to passive earth pressures, the sliding friction at the base of concrete structures can be used to resist lateral loads. The coefficient of friction for the base of concrete foundations on the native soil is 0.25.

6.3 Bridge Construction Considerations

6.3.1 Pile Installation

The 16-inch diameter steel pipe pile will be able to endure some level of hard driving, although the expectation is that will not be the case based on the ground conditions down to pile tip elevation . If "refusal" is encountered above the specified tip elevation, center-relief drilling can be used to achieve deeper penetrations.

Difficult pile installation is not expected based on our current investigation data. Subsurface conditions on northeast side of the bridge used for pile design were projected from the data obtained from our investigations, and there is a potential for encountering Monterey Formation at a shallower depth on the north side of the 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 the bridge were extrapolated north across the bridge footprint, the contractor should perform confirming investigations on the northeast side of the bridge to confirm the in-situ ground conditions.

6.3.2 Backfill

In order to mitigate the potential for liquefaction-induced lateral spreading at the bridge location, the slope in front of the abutments and two feet below the trail should be backfilled using Class 2 Aggregate Base (see Section 6.3.3 for Class 2AB requirements). The base of all areas to receive backfill material should be scarified to a minimum depth of 8 inches, moisture conditioned to a soil moisture content at or near optimum, and recompacted to a minimum relative compaction of 90% as determined by ASTM D1557. If surface shrinkage cracks are present, the depth of scarifying and moisture conditioning should extend to the maximum depth of cracking.

Backfill material should be placed in lifts no greater than 8 inches in loose thickness and be compacted to a minimum relative compaction of 90% of maximum dry density at a moisture content at or near optimum as determined by ASTM D1557. The upper 12 inches of ground surface under the trail below the bridge, backfill material should be compacted to a minimum of 95% relative compaction at a moisture content at or near optimum.

6.3.3 Caltrans Class 2 Aggregate Base (Class 2AB)

Caltrans Class 2AB uniformly graded to the requirements in Table 21 can also be used as engineered fill (Caltrans, 2019).

Table 21. Class 2AB

<table>
<tr><th>Sieve Size</th><th colspan="2">Percent Passing</th></tr>
<tr><td>1 in.</td><td colspan="2">100</td></tr>
<tr><td>3/4 in.</td><td colspan="2">90–100</td></tr>
<tr><td>No. 4</td><td colspan="2">35–60</td></tr>
<tr><td>No. 30</td><td colspan="2">10–30</td></tr>
<tr><td>No. 200</td><td colspan="2">2–9</td></tr>
<tr><th>Requirement</th><th colspan="2">Limit</th></tr>
<tr><td>Plasticity Index</td><td colspan="2">< 12</td></tr>
<tr><td>Liquid Limit</td><td colspan="2">< 30</td></tr>
<tr><th>Test</th><th>California Method No.</th><th>Requirement</th></tr>
<tr><td>Resistance (R-Value)</td><td>301</td><td>78 min.</td></tr>
<tr><td>Sand Equivalent</td><td>217</td><td>22 min.</td></tr>
</table>

7.0 Limitations

This report has been prepared for the exclusive use of GHD, and TAMC for the planned SR 218 Undercrossing Bridge 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 (2022) 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). 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.

8.0 References

American Society of Civil Engineers (ASCE). 1976. Subsurface Investigation for Design and Construction of Foundations of Buildings. ASCE Manuals and Reports on Engineering Practice No. 56 (MOP 56).

American Association of State Highway and Transportation Officials (AASHTO). 2017. AASHTO LRFD Bridge Design Specifications, 8th Edition. Dated September 2017.

American Society of Civil Engineers (ASCE). 2022. Minimum design loads for buildings and other structures. ASCE 7-22.

ASTM International:

ASTM D1586 – Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils.

ASTM D2166 – Standard Test Method for Unconfined Compressive Strength of Cohesive Soil.

ASTM D2487 – Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System).

ASTM D2488 – Standard Practice for Description and Identification of Soils (Visual-Manual Procedures).

ASTM D3080M – Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions.

ASTM D5778 – Standard Test Method for Electronic Friction Cone and Piezocone Penetration Testing of Soils.

Boulanger, R. W. and Idriss, I. M. 2014. CPT-Based Liquefaction Triggering Procedure. Center for Geotechnical Modeling, Report No. UCD/CGM-14/0X. Department of Civil and Environmental Engineering, University of California, Davis.

Branum, D., R. Chen, M. Petersen, and C. Wills. 2016 (rev.). Earthquake Shaking Potential for California, California Geological Survey, Map Sheet 48 (Revised 2016).

Bryant, W.A. (compiler). 2001. Fault number 145a, Chupines fault zone, Del Rey Oaks section, in Quaternary fault and fold database of the United States: U.S. Geological Survey website, https://earthquakes.usgs.gov/hazards/faults/qfaults, accessed 11/04/2021.

Bryant, W.A. 1985. Faults in the Southern Monterey Bay Area, Monterey County; California Division of Mines and Geology (now the California Geological Survey) Fault Evaluation Report FER-167.

California Department of Transportation (Caltrans). 2012. Guidelines on Foundation Loading and Deformation Due to Liquefaction Induced Lateral Spreading. January 2012, 44 pp.

California Department of Transportation (Caltrans). 2014. Memo to Designers 3-1. Section 3-1 Deep Foundations. June 2014, 23 pp with Attachment 1.

California Department of Transportation (Caltrans). 2017. Lateral Spreading Analysis for New and Existing Bridges, Memo to Designers 20-15, dated May 2017.

California Department of Transportation (Caltrans). 2019. California Amendments to the AASHTO LRFD Bridge Design Specifications, 2017 Eighth Edition.

California Department of Transportation (Caltrans). 2020a. ARS Onlilne, Version v3.0, Accessed at https://arsonline.dot.ca.gov/.

California Department of Transportation (Caltrans). 2020b. Lateral Spreading Analysis Example, Caltrans Geotechnical Manual, dated January 2020.

California Department of Transportation (Caltrans). 2020c. Liquefaction Evaluation, Caltrans Geotechnical Manual, dated January 2020.

California Department of Transportation (Caltrans). 2020d. Liquefaction-Induced Downdrag, Caltrans Geotechnical Manual, dated January 2020.

California Department of Transportation (Caltrans). 2020e. Liquefaction-Induced Lateral Spreading, Caltrans Geotechnical Manual, dated January 2020.

California Department of Transportation (Caltrans). 2021. Corrosion Guidelines, Version 3.0.2. https://dot.ca.gov/-/media/dot-media/programs/engineering/documents/mets/corrosion-guidelines-a11y.pdf.

California Geological Survey (CGS). 2018, Earthquake Fault Zones, A Guide for Government Agencies, Property Owners/Developers, and Geoscience Practitioners for Assessing Fault Rupture Hazards in California; Special Publication 42.

Clark, J.C., Brabb, E.E., and L.I. Rosenberg. 2000. Geologic map of the Spreckles 7.5-minute quadrangle, Monterey County, California: U.S. Geological Survey Miscellaneous Field Study MF-2349.

Clark, J.C., Dupre, W.R., and L.I. Rosenberg. 1997. Geologic Map of the Monterey and Seaside 7.5-minute Quadrangles, Monterey County, California: U.S. Geological Survey Open-File Report 97-30. (don’t include since shows landslide).

Cornerstone Structural Engineering Group (Cornerstone). 2022. Drawing for FORTAG Undercrossing at SR 218, Transportation Agency for Monterey County, Dated June 2022.

Cornerstone Structural Engineering Group (Cornerstone). 2023. Drawing for FORTAG Undercrossing at SR 218, Transportation Agency for Monterey County, Dated January 2023.

Dibblee, T.W. Jr., and J.A. Minch. 2007. Geologic Map of the Monterey and Seaside Quadrangles, Monterey County, California, Dibblee Geology Center Map #DF-346.

Dibblee, T.W. Jr., J.C. Clark., H.G. Greene., and O.E. Bowen, Jr. 1974. Preliminary Geologic Map of the Monterey and Seaside 7.5-minute Quadrangles, Monterey County, California, U.S. Geological Survey.

Dupre, W.R. 1990. Maps showing geology and liquefaction susceptibility of Quaternary deposits in the Monterey, Seaside, Spreckels and Carmel Valley Quadrangles, Monterey County, California: U.S. Geological Survey Miscellaneous Field Studies Map MF-2096.

Federal Highway Administration (FHWA). 2016. Design and Construction of Driven Pile Foundations – Volume I, Publication No. FHWA-NHI-16-009. September 2016, 559 pp.

GEOLOGISMIKI. 2007. CLiq – CPT soil liquefaction software, version 3.0. Serres, Greece.

GHD. 2022. 95% submittal drawings for FORTAG Trail Project, Transportation Agency for Monterey County.

Hart, E.H., and W.A. Bryant. 1997. Fault-Rupture Hazard Zones in California. California Division of Mines and Geology Special Publication 42.

Hartwell, S.R., S.Y. Johnson., C.W. Davenport., and J.T. Watt. 2016. Offshore and Onshore Geology and Geomorphology, Offshore of Monterey Map Area, California, U.S. Geological Survey, Open-File Report 2016-1110, Sheet 10 of 10.

Heuer, R. 1974. Important ground parameters in soft ground tunneling. In *Proceedings of Subsurface Explorations for Underground Excavation and Heavy Construction*, ASCE Specialty Conference, 41–55. Henniker, NH: ASCE.

Idriss, I. M. and Boulanger, R. W. 2008. Soil Liquefaction during Earthquakes. Earthquake Engineering Research Institute, Oakland, California.

Matlock, H. 1970. Correlations for design of laterally loaded piles in soft clay, Proceedings of the II Annual Offshore Technology Conference, Houston, Texas, (OTC 1204): 577-594.

McCulloch, D.S., and Green, H.G. 1989. Geologic map of the central California continental margin, in Greene, H.G., and Kennedy, M.P., eds., Geology of the central California continental margin: California Division of Mines and Geology California Continental Margin Geologic Map Series, Map 5A, scale 1:250,000.

McMillen Jacobs Associates (MJA). 2021a. Pre-Type Structure Selection Fact Sheet for SR 218 Undercrossing; Fort Ord Regional Trail & Greenway Canyon Del Rey/SR 218 Segment.

McMillen Jacobs Associates (MJA). 2021b. Draft Geotechnical Data Report, Fort Ord Regional Trail & Greenway Canyon Del Rey/SR 218 Segment.

McMillen Jacobs Associates (MJA). 2023a. Fort Ord Regional Trail & Greenway SR 218 Undercrossing Bridge and Retaining Wall No. 1 – Final Geotechnical Design Report. Prepared for Transportation Agency of Monterey County.

McMillen Jacobs Associates (MJA). 2023b. Fort Ord Regional Trail & Greenway Retaining Wall No. 1 – Final Foundation Report for Retaining Wall. Prepared for Transportation Agency of Monterey County.

Reese, L. C.,Cox, W. R., and Koop, F. D. 1974. Field testing and analysis of laterally loaded piles in sand. Proceedings of the VI Annual Offshore Technology Conference, Houston, Texas, 2(OTC 2080): 473-485

Robertson, P.K., R.G. Campanella, D. Gillespie, and J. Greig. 1986. Use of piezometer cone data. In *Proceedings of In Situ '86, ASCE Specialty Conference, Blacksburg, Virginia*.

Robertson, P.K. and C.E. Wride. 1998. Evaluating cyclic liquefaction potential using the cone penetration test. *Canadian Geotechnical Journal*, 35: 442–459.

Robertson, P.K. 1990. Soil classification using the Cone Penetration Test. *Canadian Geotechnical Journal*, 27: 151–158. DOI: 10.1139/T90-014.

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

Robertson, P.K. 2010. Soil behaviour type from the CPT: An Update. In *Proceedings of 2nd International Symposium on Cone Penetration Testing, Huntington Beach*, 2: 575–583.

Rocscience. 2022. Slide2 – 2D Limit Equilibrium Analysis for Slopes, version 9.025.

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

Tinsley, J.C. III, Egan, J.A., Kayen, R.E., Bennett, M.J., Kropp, A., and T.L. Holzer, 1998, Appendix: Maps and Descriptions of Liquefaction and Associated Effects, in T.L. Holzer, ed., The Loma Prieta, California, Earthquake of October 17, 1989-Liquefaction; U.S. Geological Survey Professional Paper 1551-B.

U.S. Geological Survey (USGS). 2016. Fact Sheet 2016-3020. https://pubs.usgs.gov/fs/2016/3020/fs20163020.pdf.

U.S. Soil/Natural Resources Conservation Service (NRCS). 2020. Web Soil Survey, U.S. Department of Agriculture. http://websoilsurvey.sc.egov.usda.gov.

Wair, B.R., DeJong, J.T. and Shantz, T. 2012. Guidelines for estimation of shear wave velocity profiles. Pacific Earthquake Engineering Research Center.

Whitson Engineers (Whitson). 2020. Topographic Survey, State Route 218, Transportation Agency of Monterey County, Dated: January 2020.

Youd, T.L., and S.N. Hoose. 1978. Historic ground failures in northern California triggered by earthquakes; U.S. Geological Survey Professional Paper 993.

Youd, T.L., and I.M. Idriss, 2001, Liquefaction resistance of soils: summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils. *Journal of Geotechnical and Geoenvironmental Engineering*, 127(4), 297–313.

Figures

REGIONAL MAP

LEGEND

- Drainageway
- Boring location (logs in Appendix B)
- CPT location (results in Appendix D)
- Planned abutment
- Planned SR 218 Undercrossing Bridge
- Planned trail
- Planned Retaining Wall No. 1
- Planned Retaining Wall No. 2 (not in scope)
- Schematic subsurface exploration profile location (see Figures 5.1 through 5.3)

0 50 100 ft

Basemap modified from Google Satellite (2023)
Planned bridge and retaining wall location based on Cornerstone (2022)

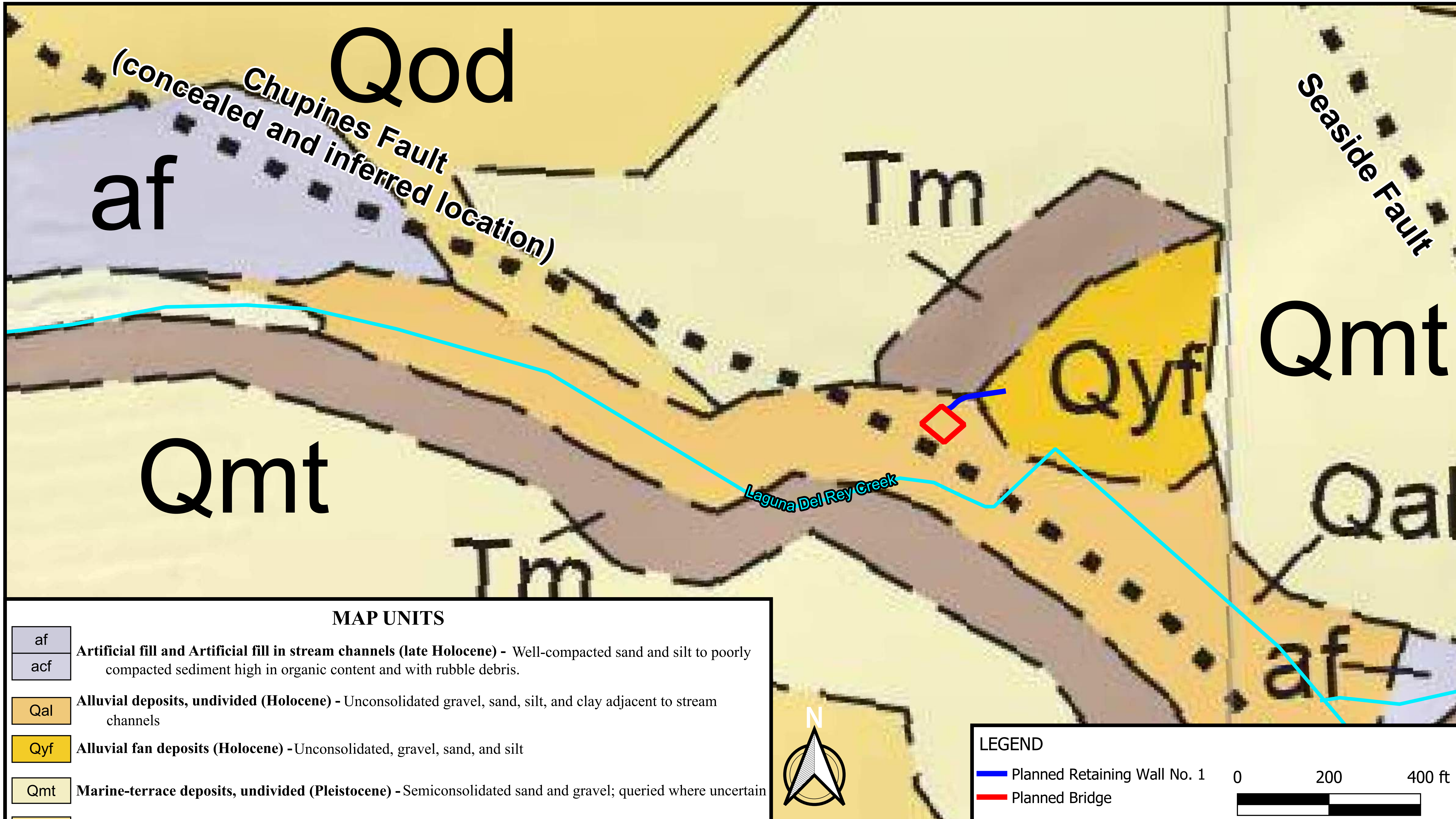
McMILLEN JACOBS ASSOCIATES

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)

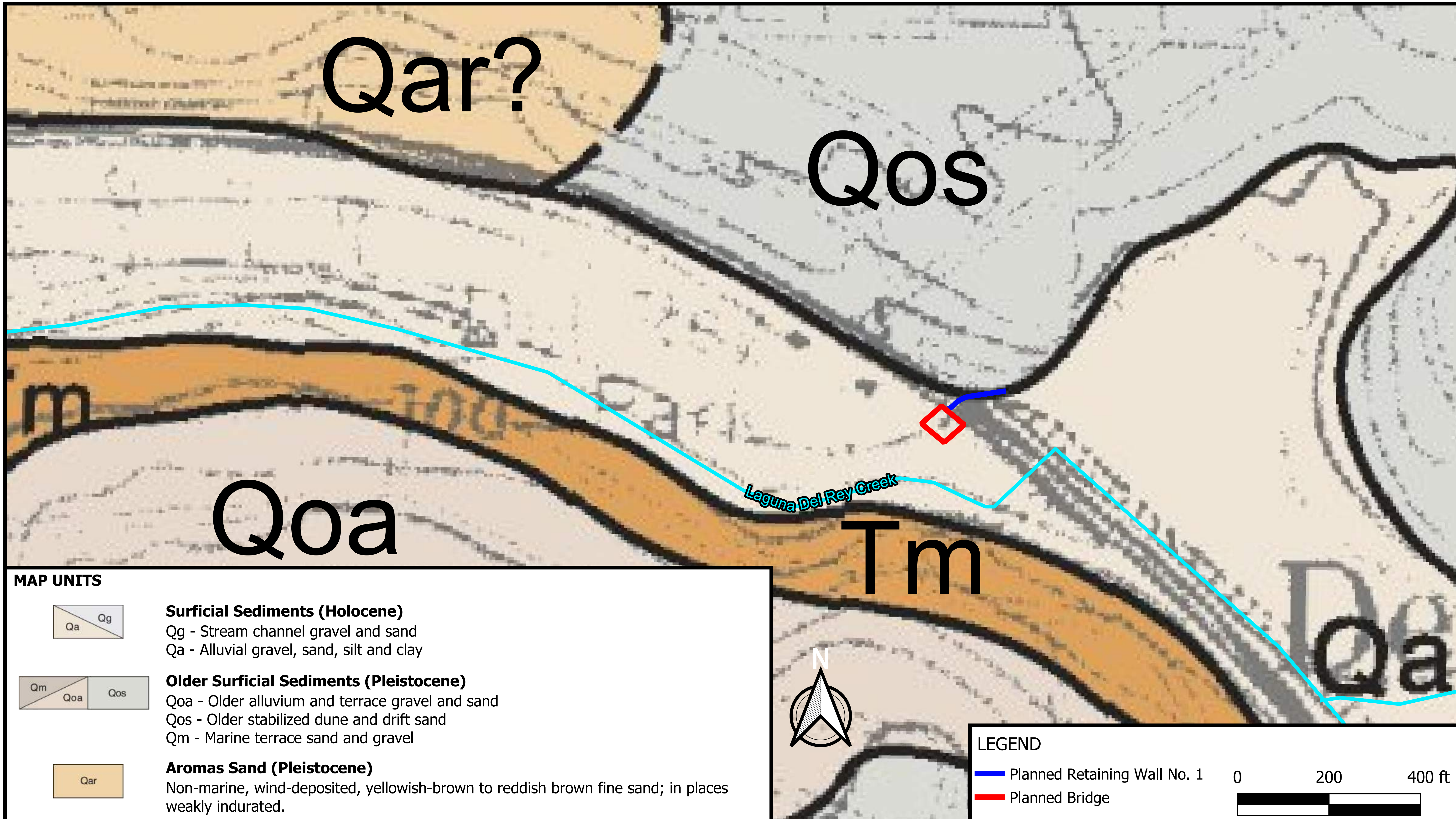
McMILLEN JACOBS ASSOCIATES

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)

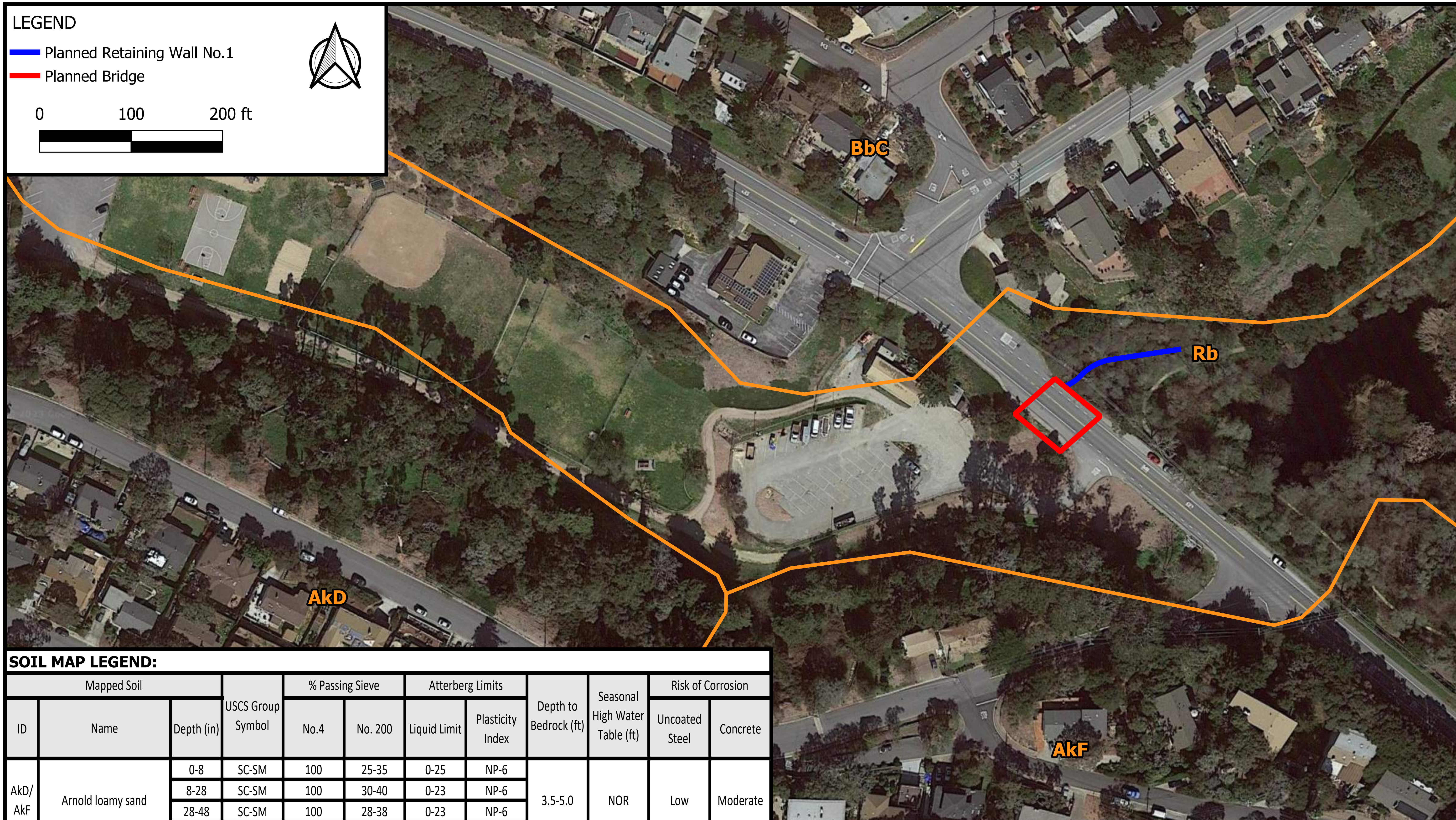
McMILLEN JACOBS ASSOCIATES

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 2.2



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)

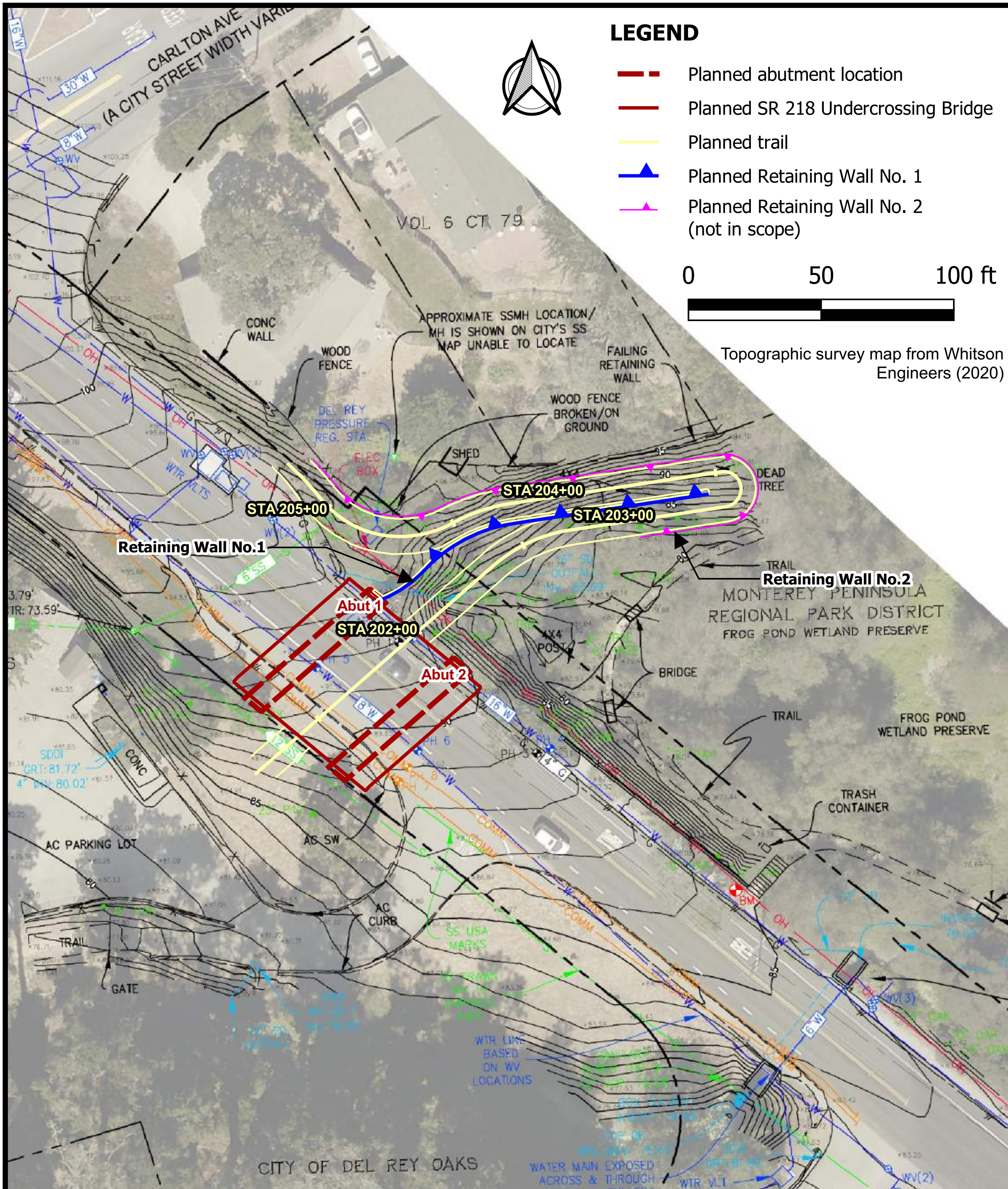
McMILLEN JACOBS ASSOCIATES

File No. 6231.0 | April 2023

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

Soil Map

Figure 3



McMILLEN JACOBS ASSOCIATES

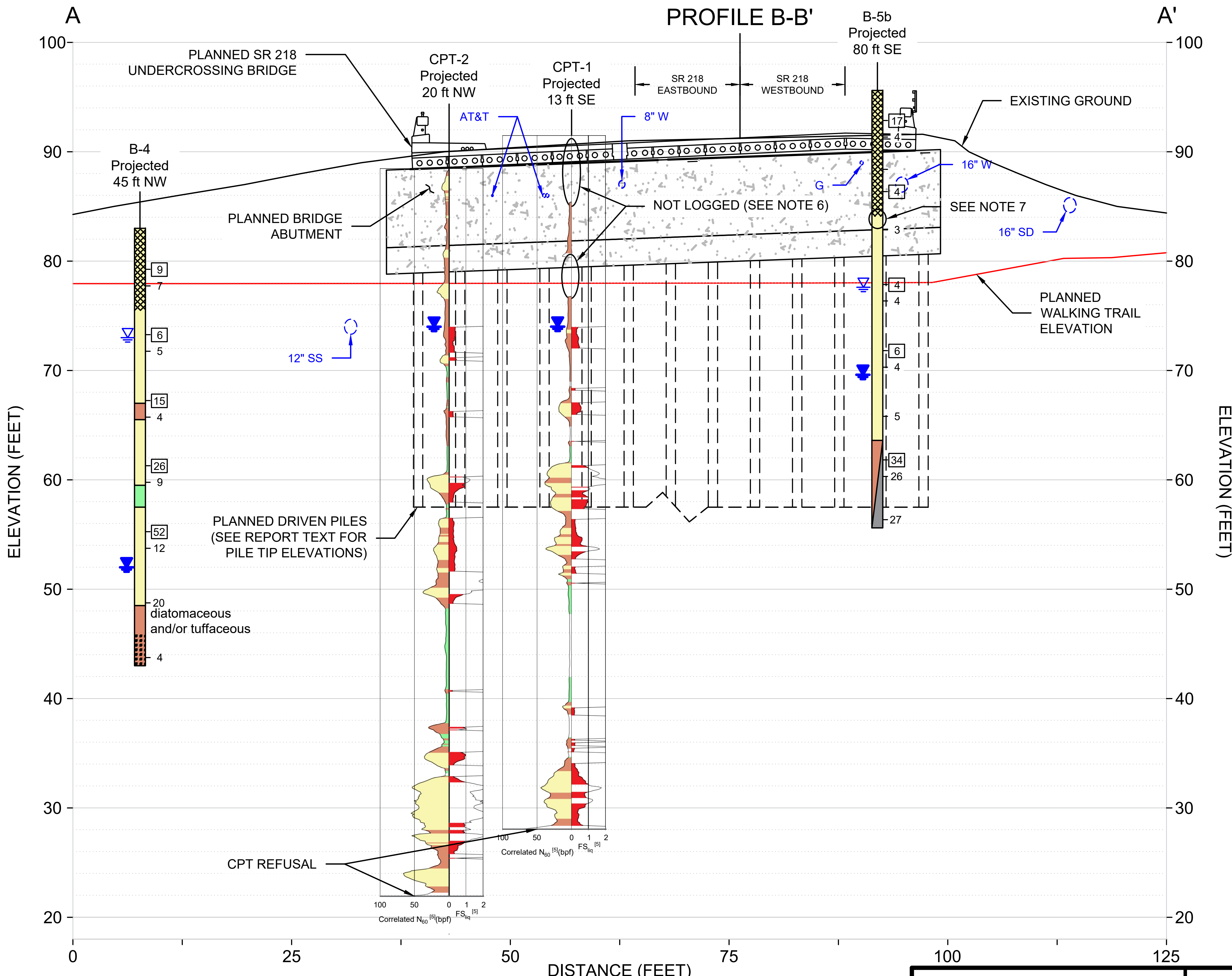
File No. 6231.0 | April 2023

GHD

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

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.

McMILLEN JACOBS ASSOCIATES

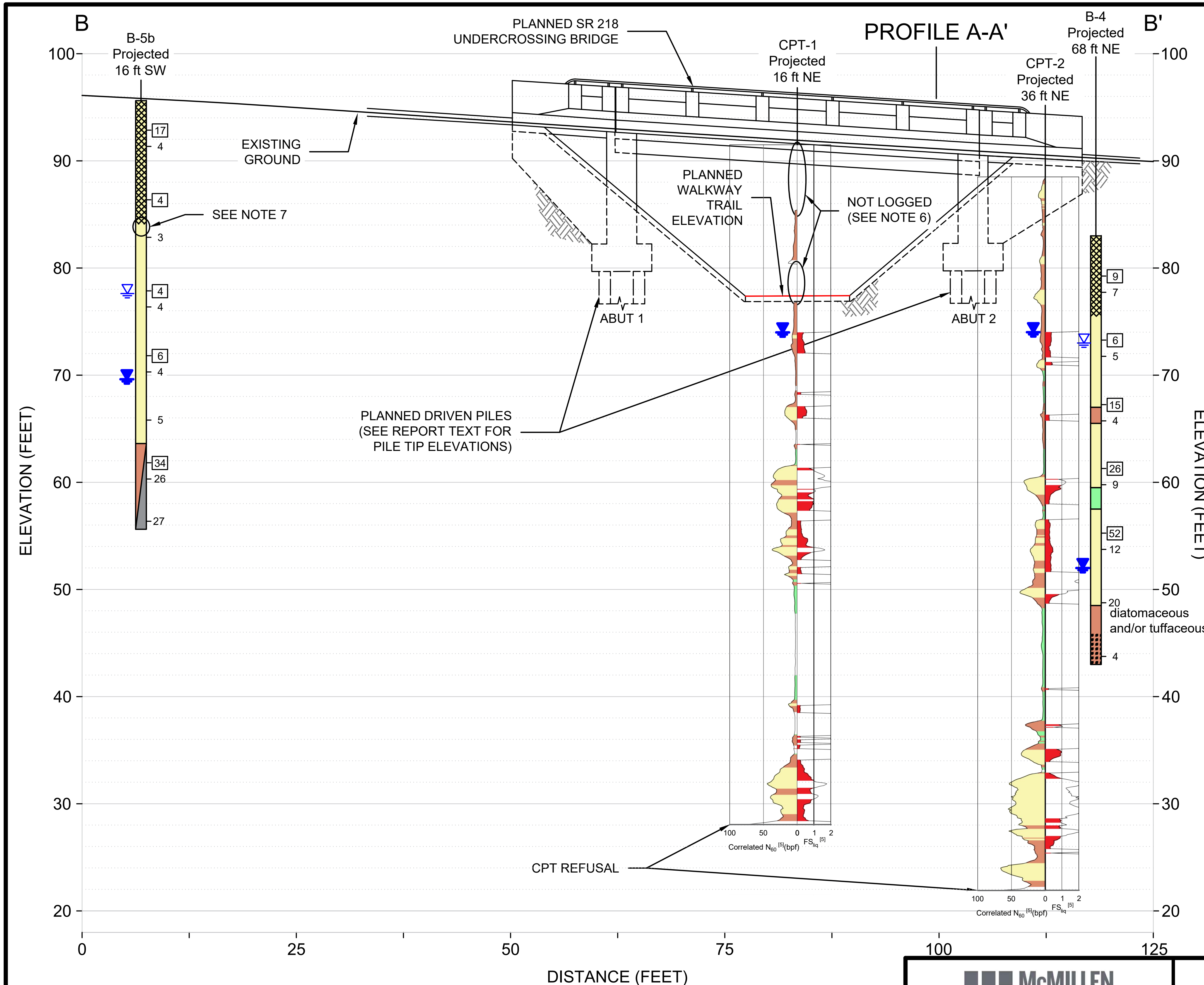
File No. 6231.0 | April 2023

GHD

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

Schematic Subsurface Exploration Profile A-A'

Figure 5.1



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.

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

Schematic Subsurface Exploration Profile B-B'

Figure 5.2

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)

McMILLEN JACOBS ASSOCIATES

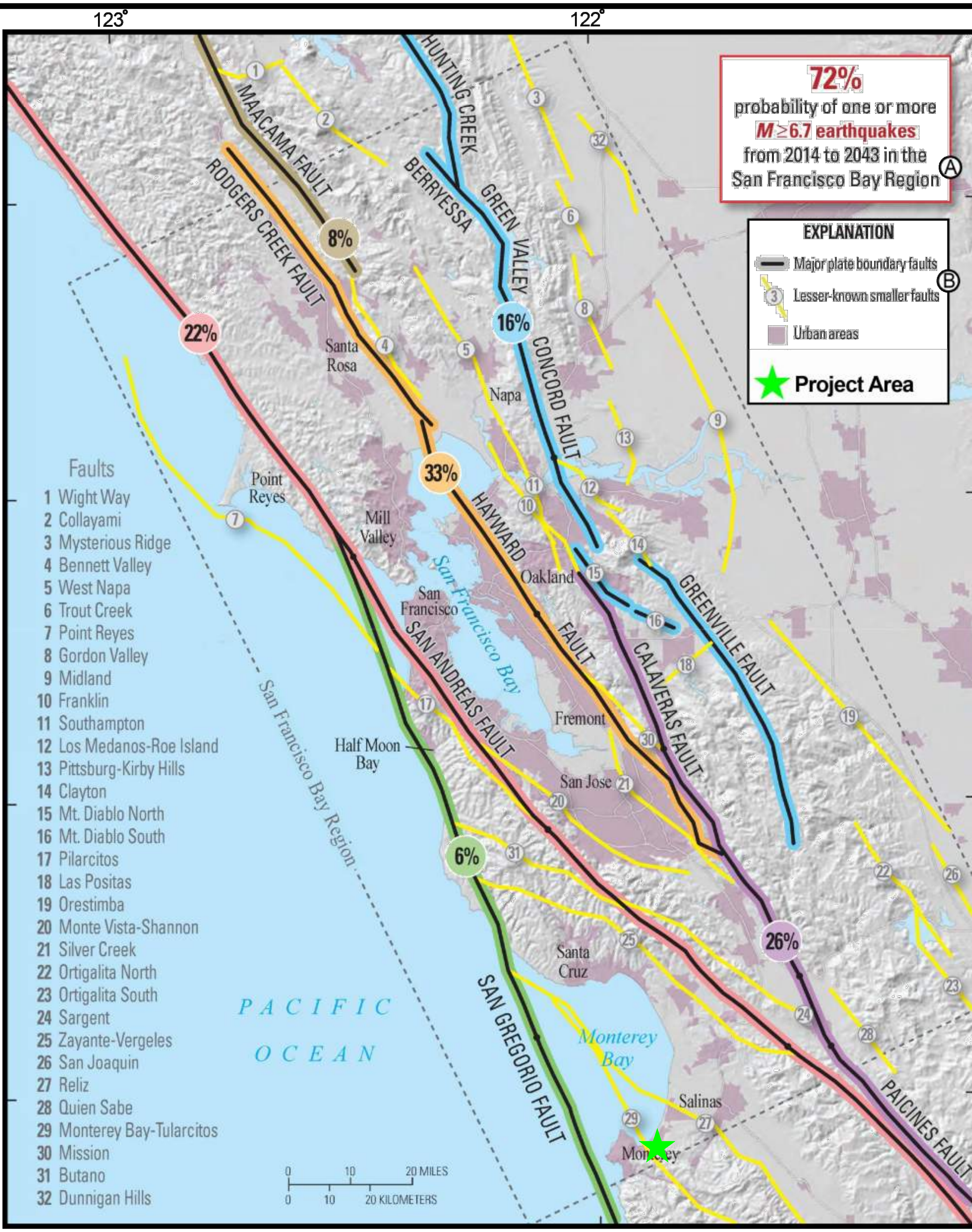
File No. 6231.0 | April 2023

GHD

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

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%.

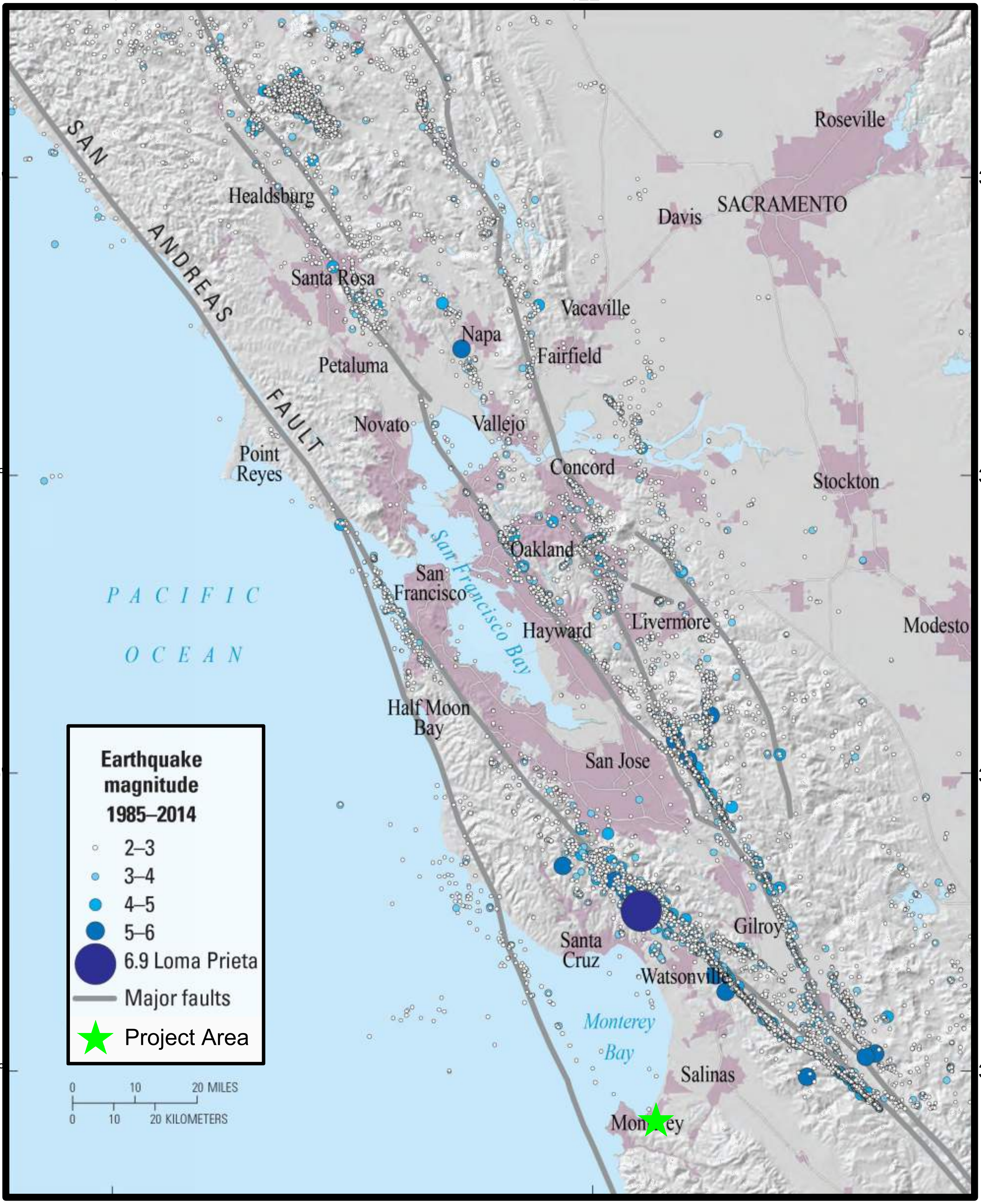
McMILLEN JACOBS ASSOCIATES

File No. 6231.0	April 2023

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

Fault Map

Figure 7



Map modified from USGS Fact Sheet 2016-3020

McMILLEN JACOBS ASSOCIATES		GHD Transportation Agency for Monterey County FORTAG - Canyon Del Rey/SR218 Segment Del Rey Oaks, California Earthquake Map	Figure 8
File No. 6231.0	April 2023		

LEGEND

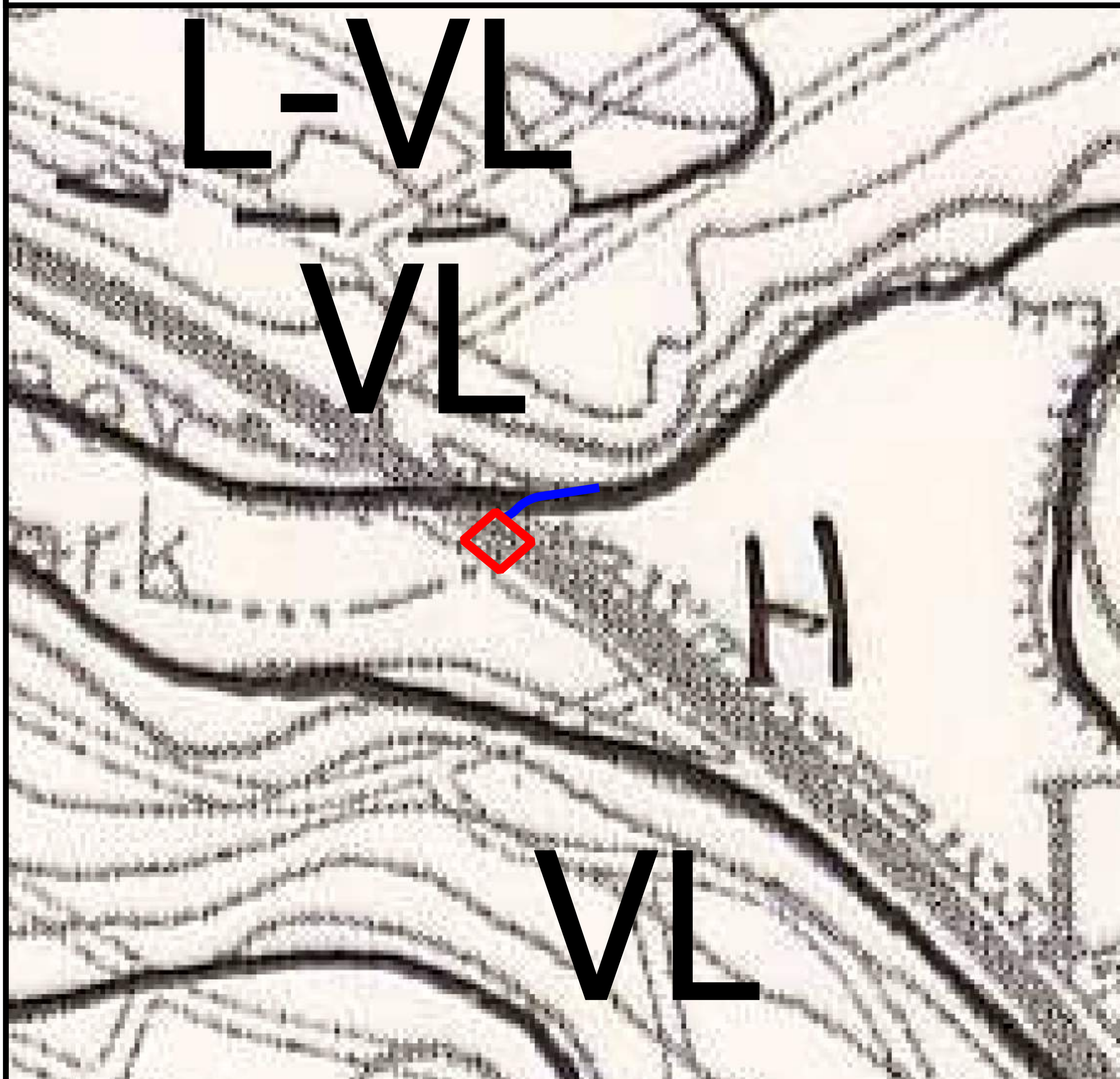
Planned Retaining Wall No.1
Planned Bridge

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)



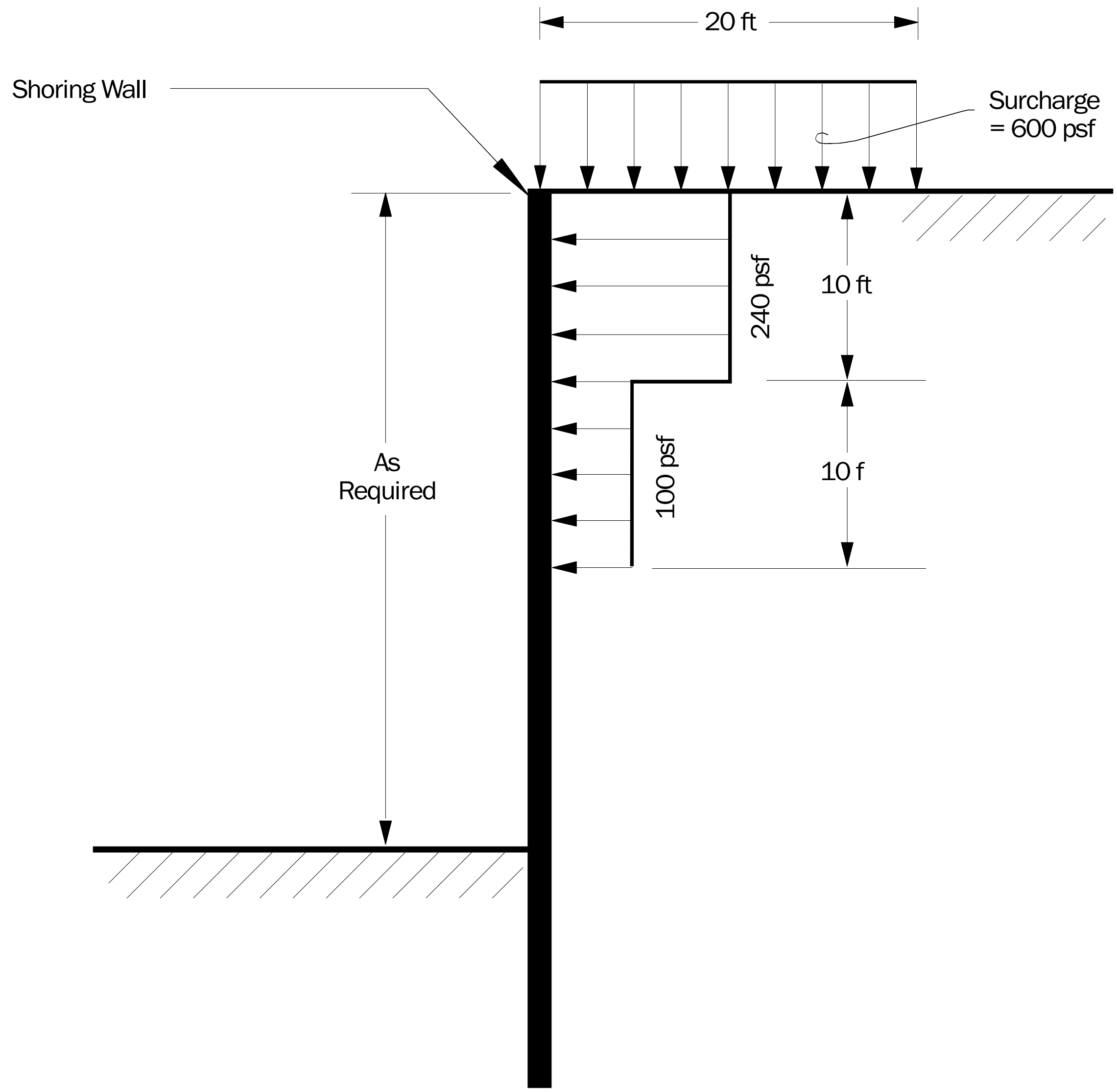
McMILLEN JACOBS ASSOCIATES

File No. 6231.0 | April 2023

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

Liquefaction Susceptibility Map

Figure 9



These are minimum shoring pressures to be used for traffic and equipment surcharges. Shoring pressures from construction activities or equipment that produce larger or different surcharge loading patterns than that shown should be determined by the shoring designer using geotechnical computational methods.

McMILLEN JACOBS ASSOCIATES

File No. 6231.0 | April 2023

GHD

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

Minimum Shoring Pressure for Traffic and Equipment Surcharge

Figure

10

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).

McMILLEN JACOBS ASSOCIATES

File No. 6231.0 | April 2023

GHD

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

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	(Liquid limit-oven dried) / (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	(Liquid limit-oven dried) / (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 $Cu = \frac{D_{60}}{D_{10}}$ $Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$

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.*

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

McMILLEN JACOBS ASSOCIATES

File No. 6231.0 | April 2023

GHD

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

Bedrock Descriptors

Figure A-2

Appendix B

LOG OF BORING B-4 ①

LOCATION: see Figure 1

DEPTH feet	SAMPLE NO.	TYPE	PENETRATION RESISTANCE blows/ft.	③ GROUNDWATER	DESCRIPTION ②	% 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 - loose - few gravel - dry to moist - nonplastic - concrete at 2' to 3'					6	80	14	FINES 7% Silt 7% Clay		
	2		9			8	103								
5	3		7												
					SILTY CLAYEY SAND (SM/SC) - dark gray to black - loose - trace gravel - moist to wet - slightly to medium plastic CORROSION TEST Sample B-4-5 See Appendix C										
10	4		6	▽									FINES 31% Silt 19% Clay		
	5		5							<1	50	50			
15	6		15	▽	**SILT WITH SAND (ML)** - dark gray - slightly plastic - trace gravel - soft to medium stiff - few clay - wet	43	74							270	21°
	7		4					46	12						
					SILTY CLAYEY SAND (SM/SC) - dark gray - loose to medium dense - slightly plastic - wet										
20	8		26			26	98						FINES 25% Silt 8% Clay		
	9		9		**LEAN TO FAT CLAY (CL/CH)** - dark gray - medium to highly plastic - trace to little sand - stiff - wet						67	33			
25					**POORLY GRADED SAND WITH SILT (SP-SM)** - dark gray - medium dense to dense - trace clay and gravel - wet - nonplastic										

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.

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 (1 of 2)

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)										
					POORLY GRADED SAND WITH SILT (SP-SM) - dark gray - trace clay and gravel - nonplastic - medium dense to dense - wet										
	10		52			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)

DEPTH feet	SAMPLE NO.	TYPE	PENETRATION RESISTANCE blows/ft.	③ GROUNDWATER	LOG OF BORING B-5a ① LOCATION: see Figure 1 DESCRIPTION ②	% 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-5b[1]

LOCATION: see Figure 1

DEPTH feet	SAMPLE NO.	TYPE	PENETRATION RESISTANCE blows/ft.	GROUNDWATER[3]	DESCRIPTION[2]	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
					SILTY SAND (SM) - FILL										
					- light brown; - nonplastic; - medium dense; - dry										
	1		17			2	101								
					POORLY GRADED SAND WITH SILT AND CLAY (SP-SM/SP-SC) - FILL										
	2		4							3	88	9 (FINES: 5% Silt, 4% Clay)			
5					- yellowish brown and brown to light brown with some reddish brown; - trace gravel; - nonplastic; - loose; - dry to moist										
					- light brown with some reddish brown										
	3		4												
						4	95								
10	4a				- sample 4a bouncing on apparent concrete at 10.5'; - drilled through concrete between 10.5' and 12'										
					SILTY CLAYEY SAND (SM/SC)										
	4b		3		- 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															
					SILTY CLAYEY SAND (SM/SC)										
					- dark brown; - trace gravel; - medium plastic fines; - loose; - moist to wet										
	7		6												
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 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-5b (1 of 2)										
				▽	**SILTY CLAYEY SAND (SM/SC)** - dark brown - trace gravel - medium plastic fines - loose - wet										
30	9		5												
					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										
	10		34			15	104						0.4		
35	11		26												
	12		27					87	25						
40					**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

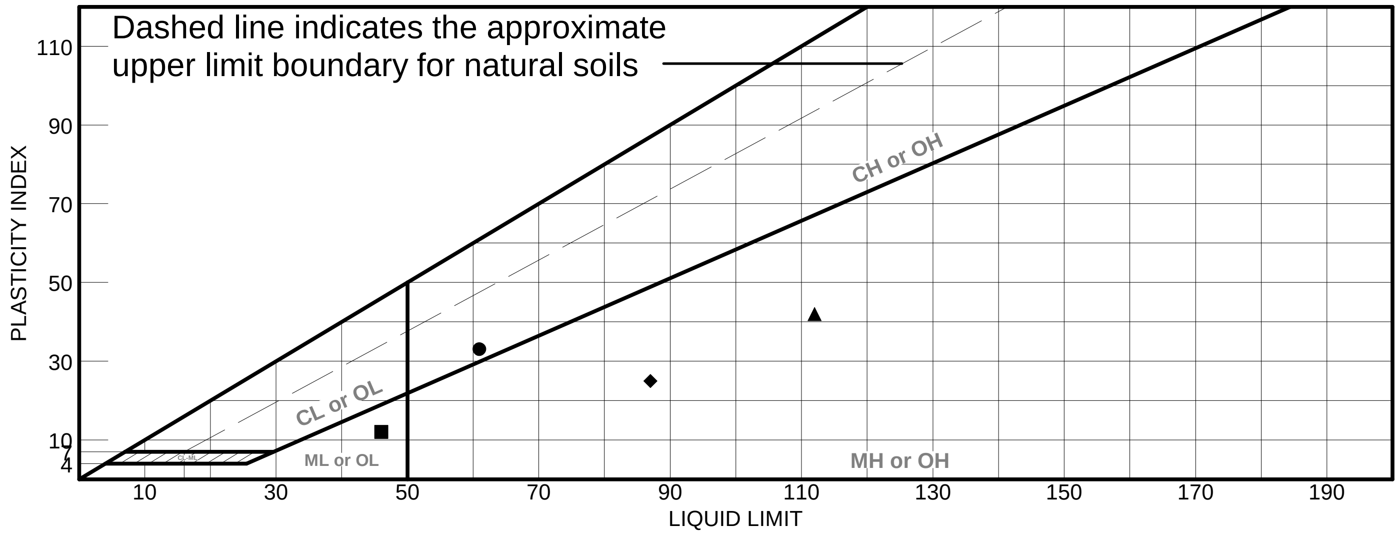
Log of Boring B-5b

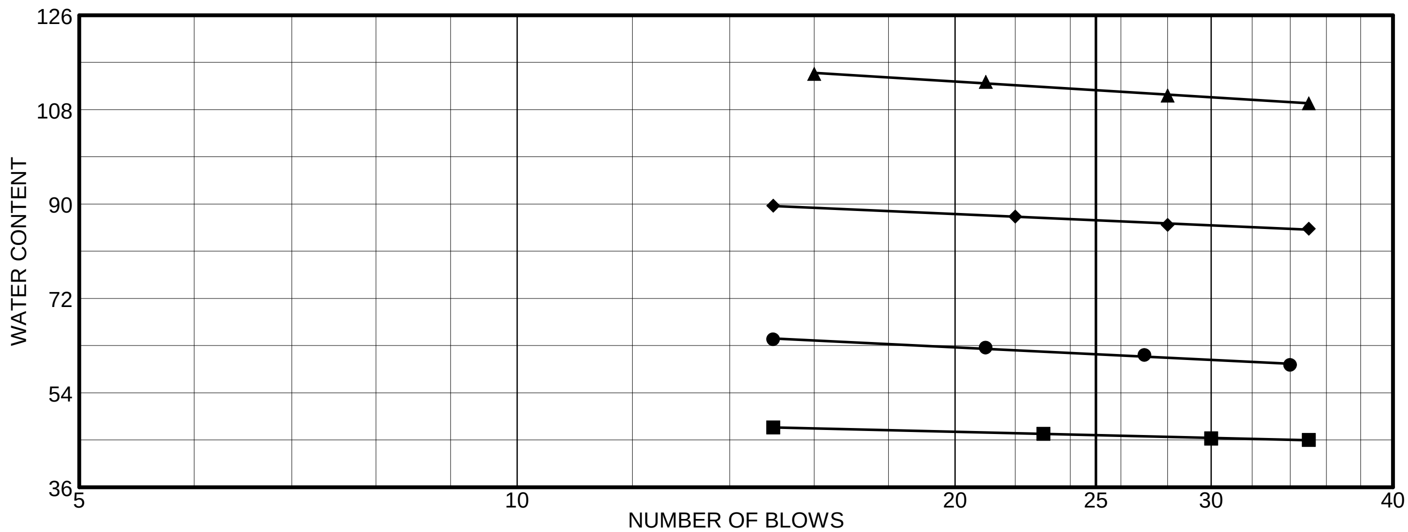
Figure B-5b

(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.

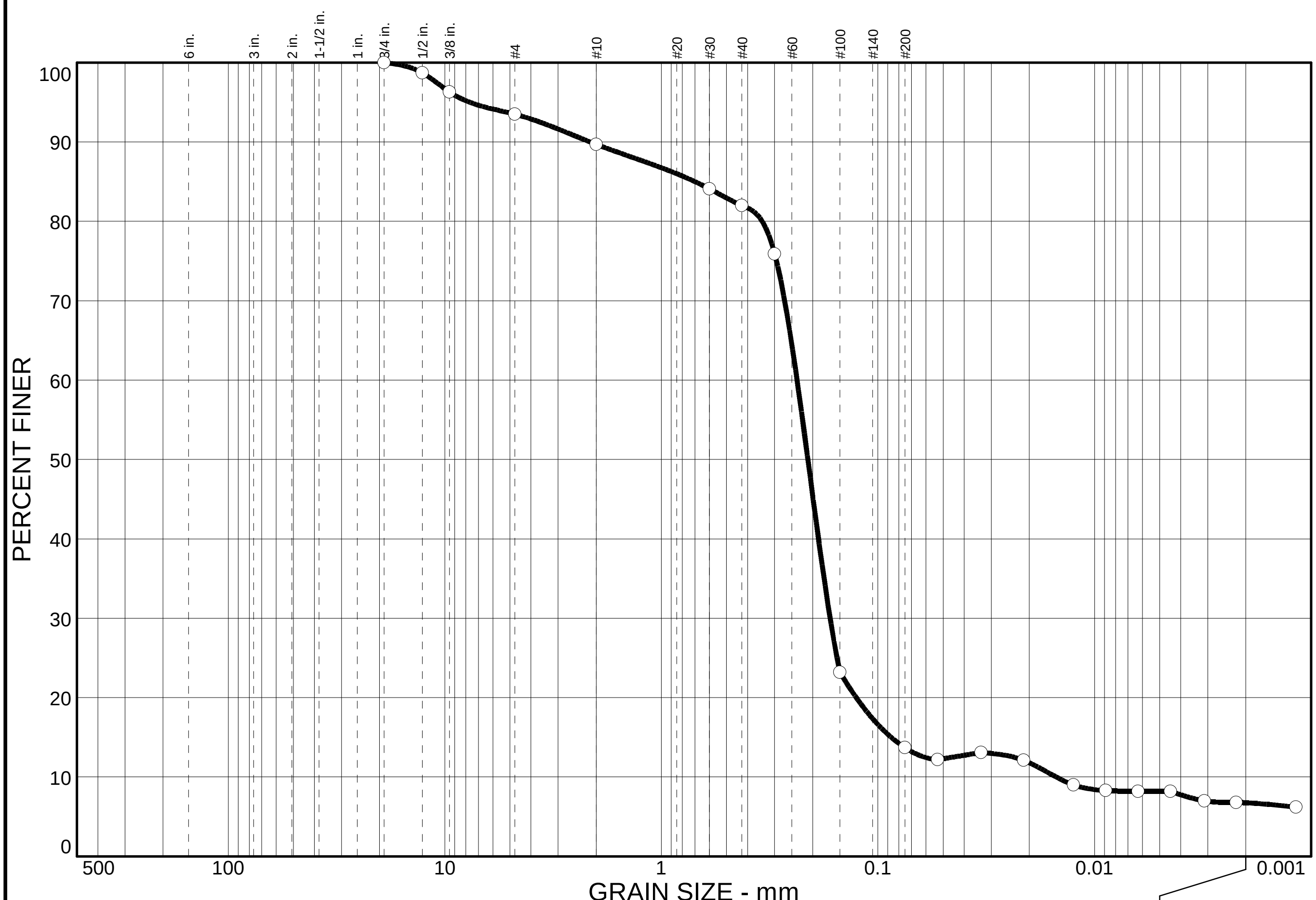
◆ Sample was prepared using the wet prep method.

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

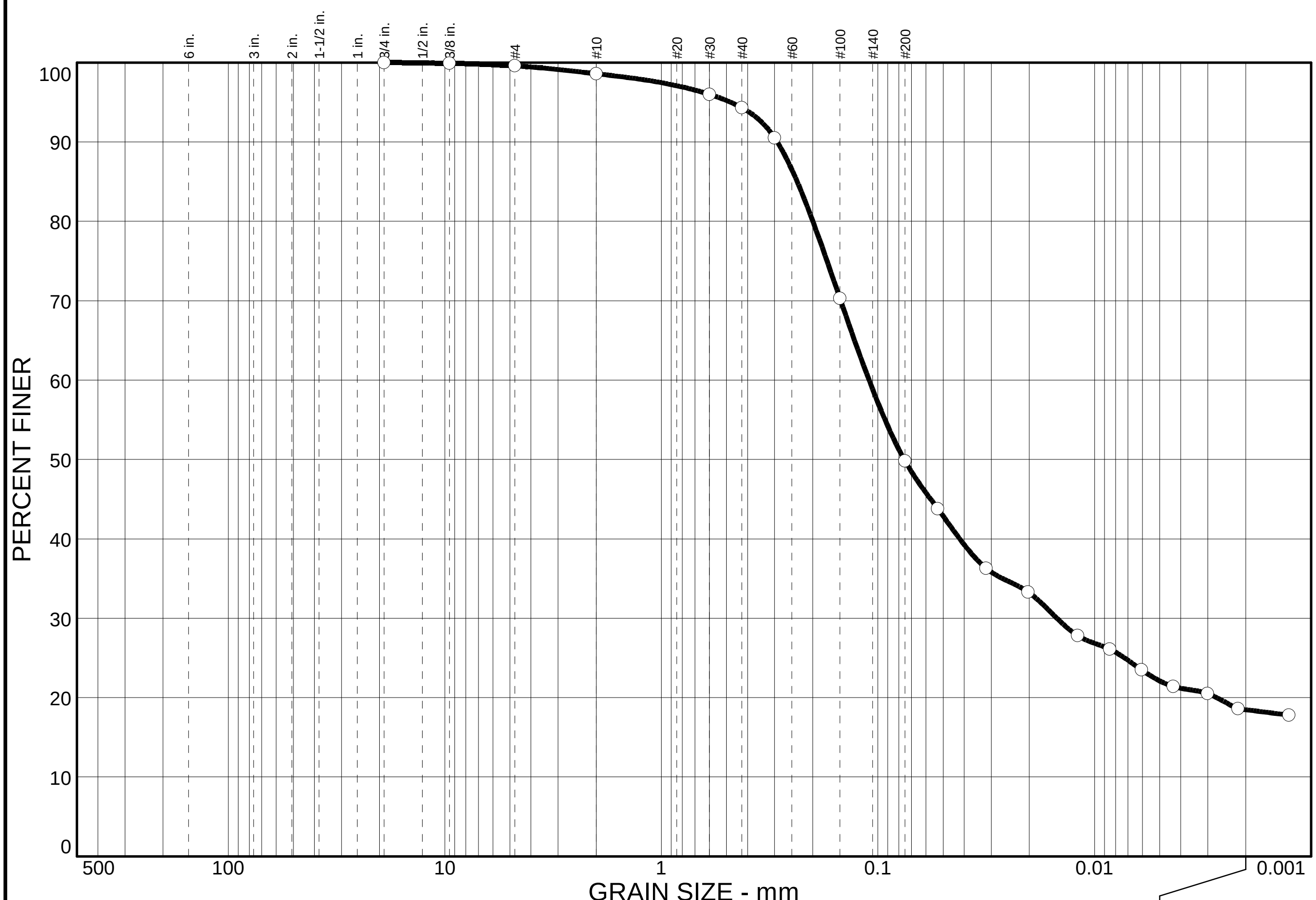
Sample No.: **Source of Sample:** B-4-1 **Date:** 9/14/21
Location: **Elev./Depth:**

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

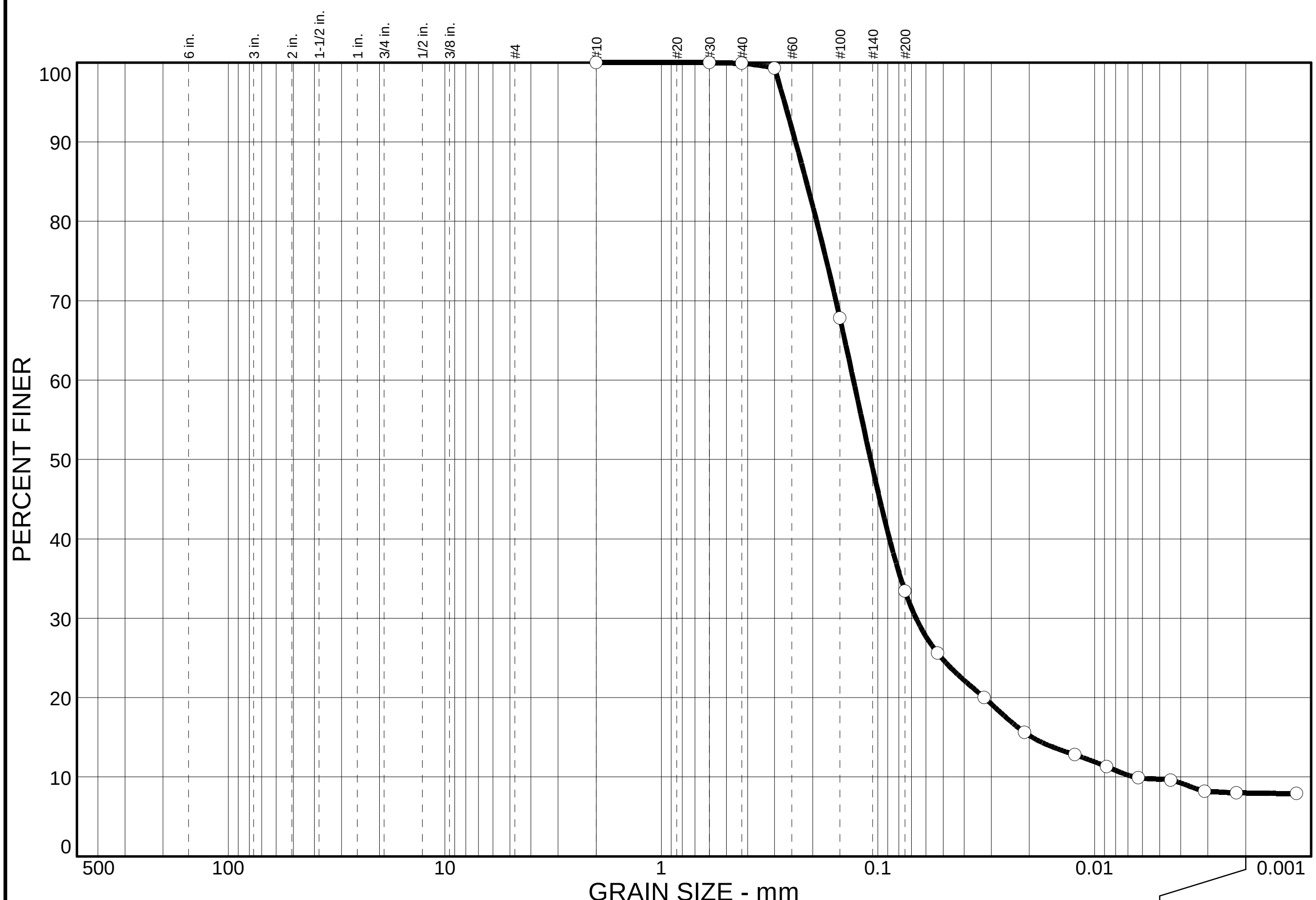
Sample No.: **Source of Sample:** B-4-5 **Date:** 9/17/21
Location: **Elev./Depth:**

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

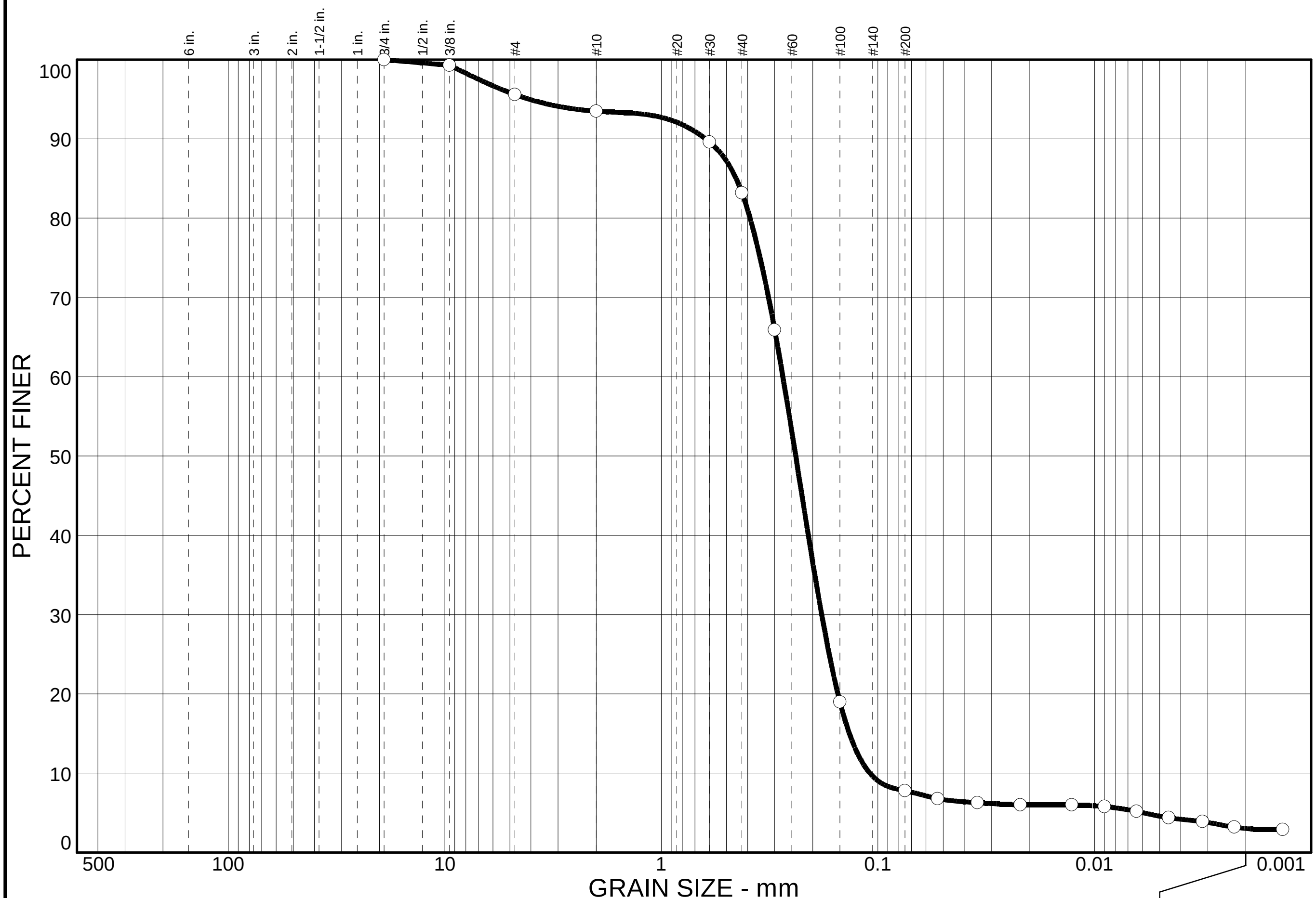
Sample No.: **Source of Sample:** B-4-9 **Date:** 9/14/21
Location: **Elev./Depth:**

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

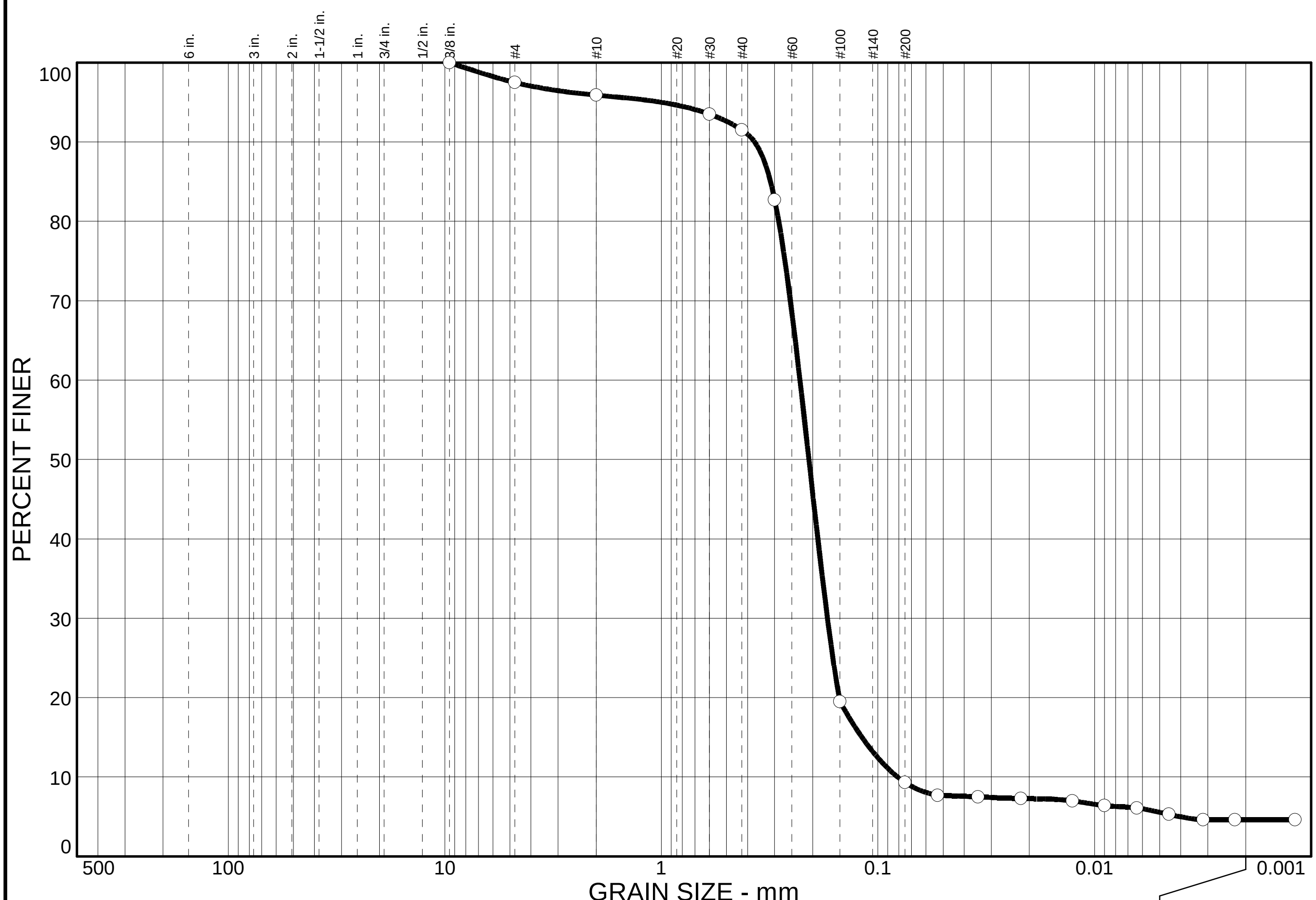
Sample No.: **Source of Sample:** B-4-10 **Date:** 9/16/21
Location: **Elev./Depth:**

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

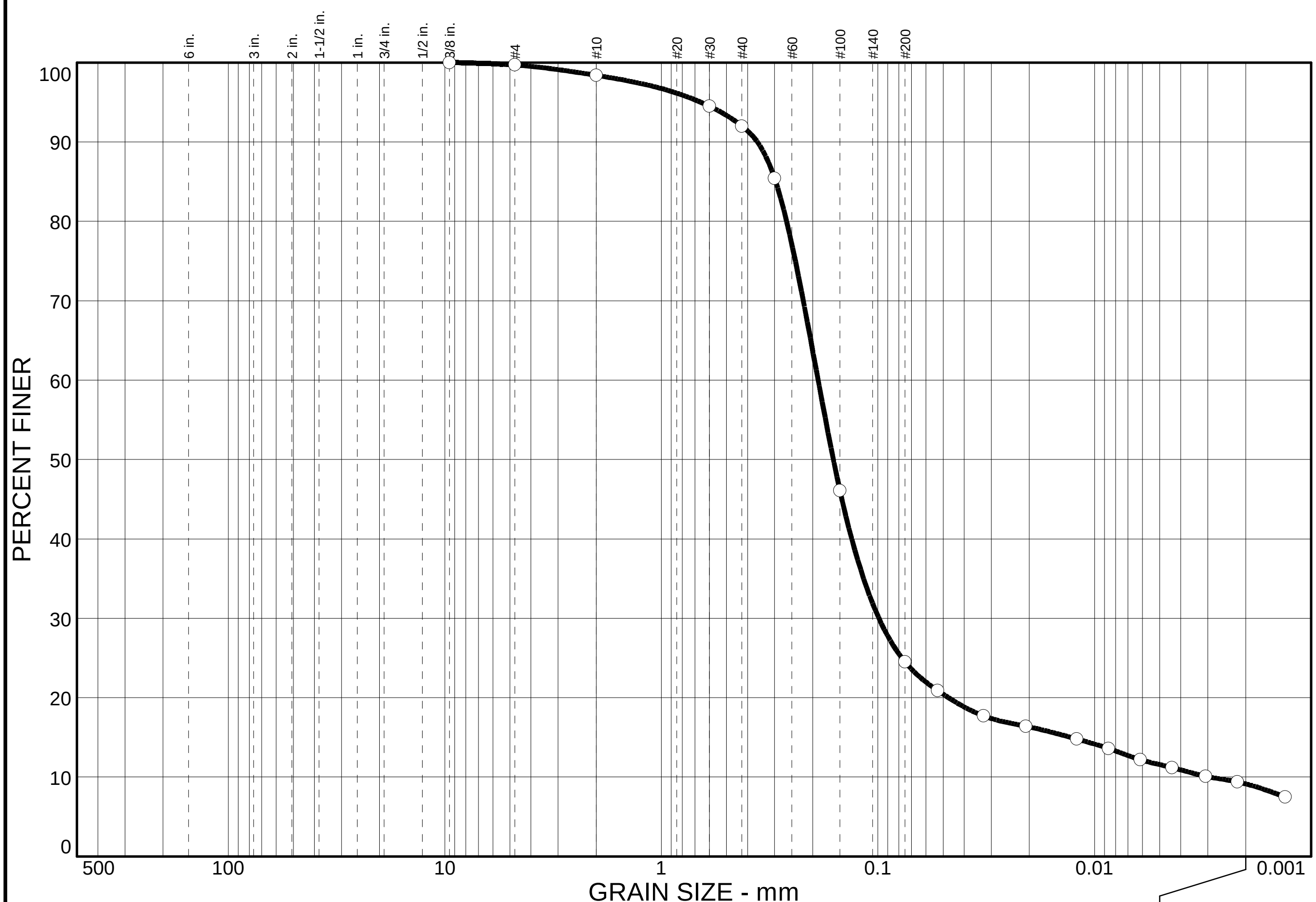
Sample No.: **Source of Sample:** B-5-2 **Date:** 9/14/21
Location: **Elev./Depth:**

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=

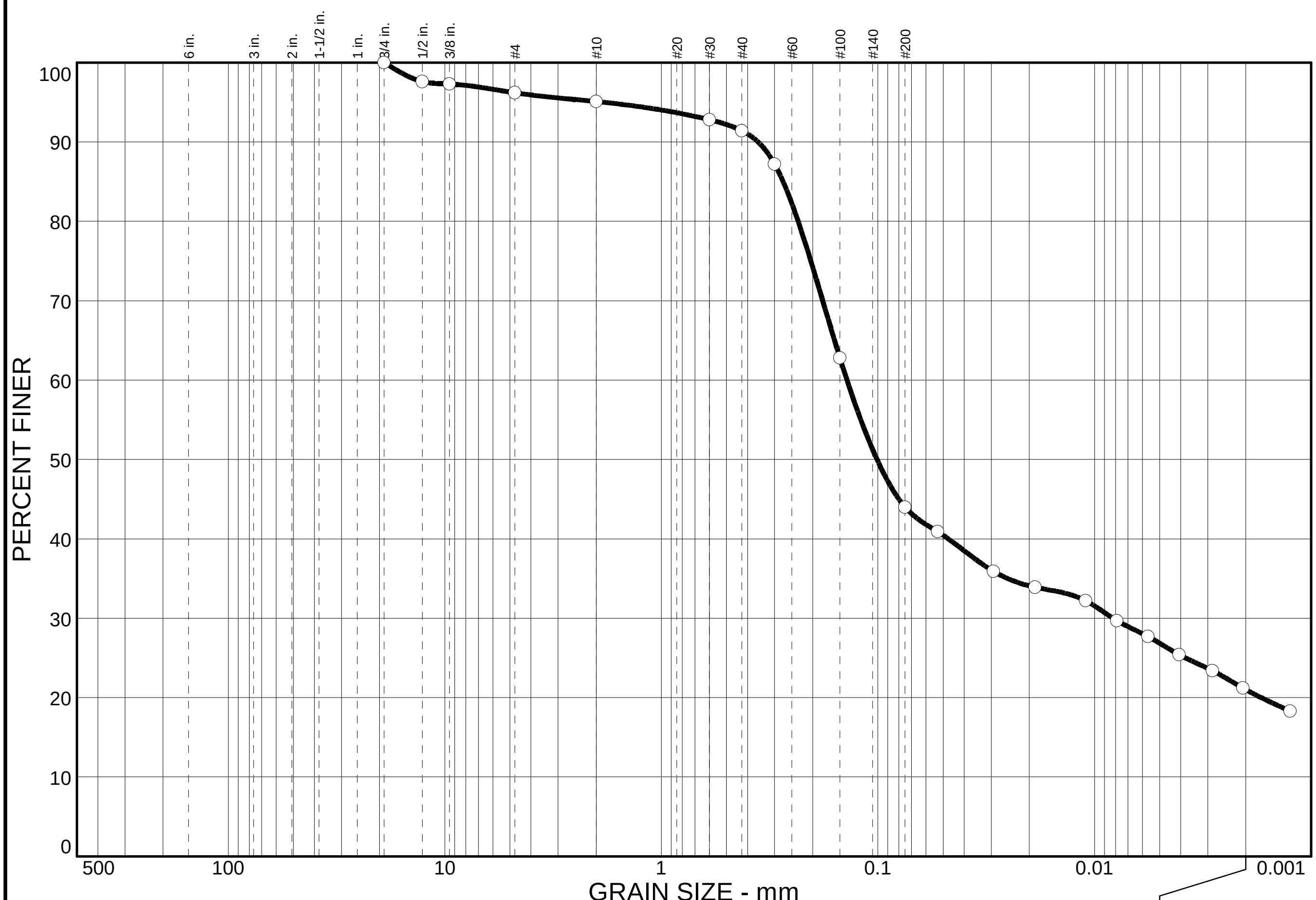
Remarks

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

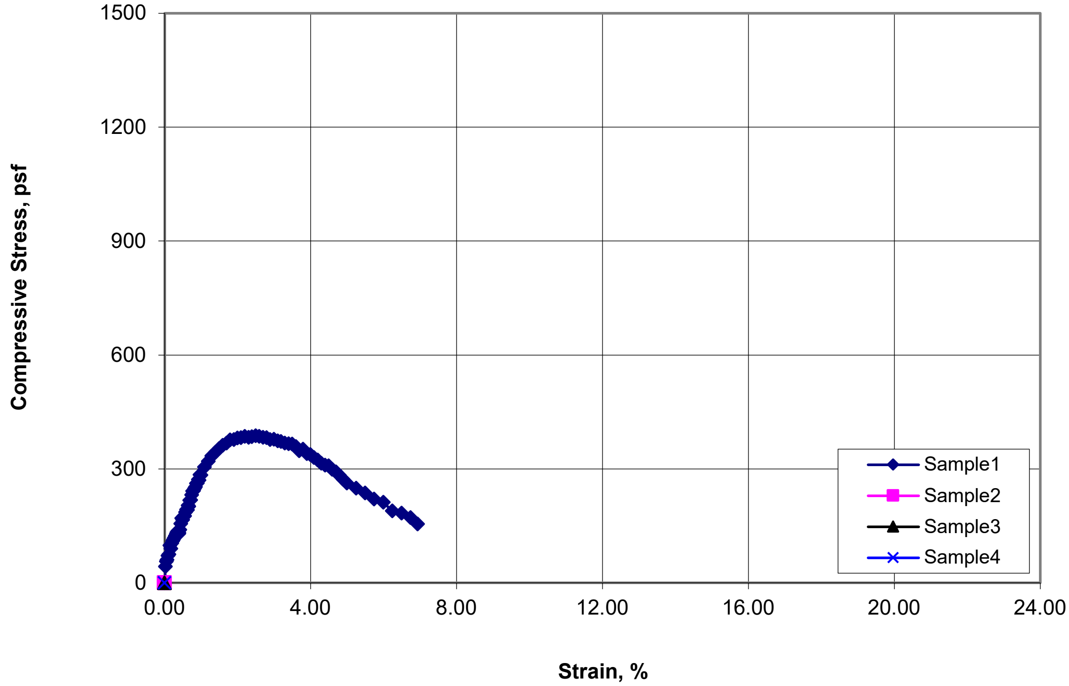
Sample No.: **Source of Sample:** B-5-8 **Date:** 9/16/21
Location: **Elev./Depth:**

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

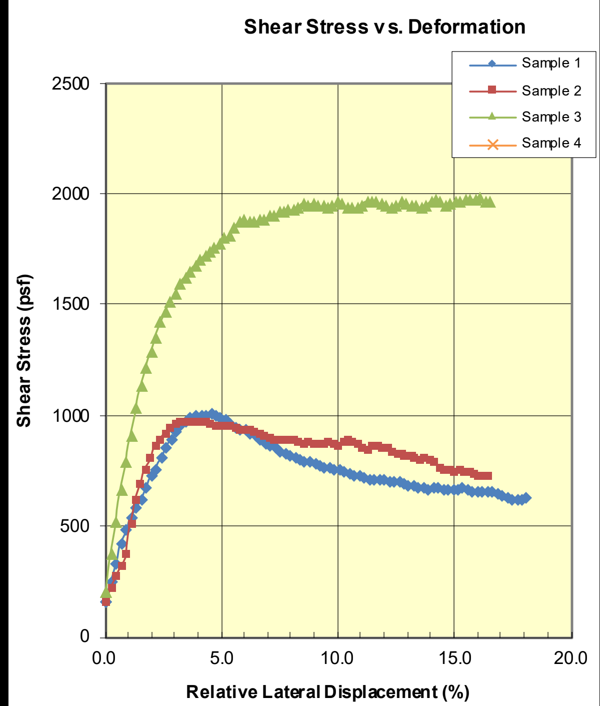
COOPER TESTING LABORATORY

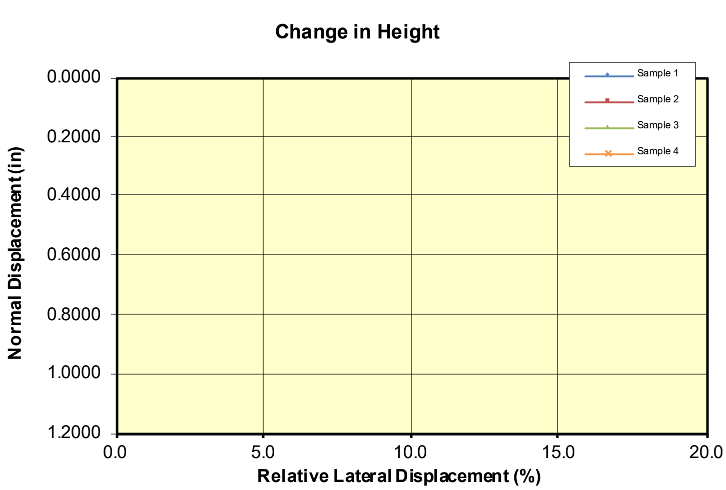
Consolidated Undrained Direct Shear (ASTM D3080M)

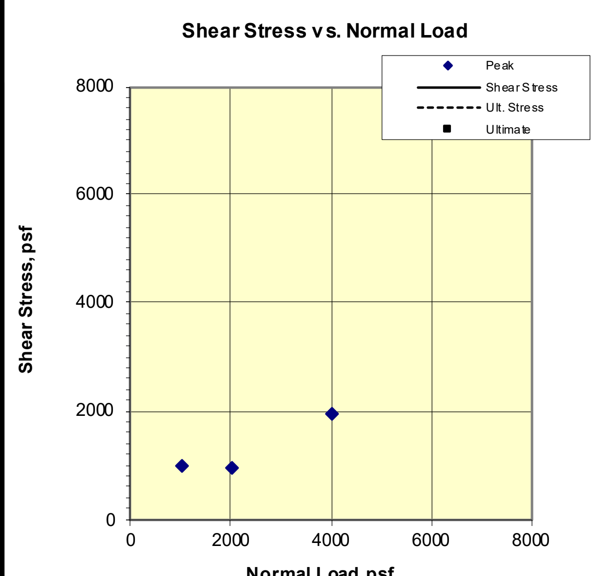
CTL Job #:	1022-034	Project #:	6231	By:	MD
Client:	McMillen Jacbos Associates	Date:	9/9/2021	Checked:	PJ
Project Name:		Remolding Info:			

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

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)				







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

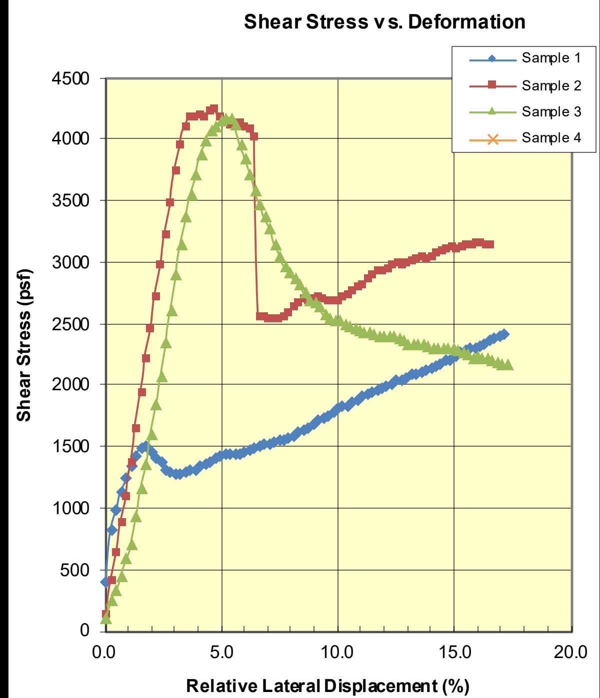
COOPER TESTING LABORATORY

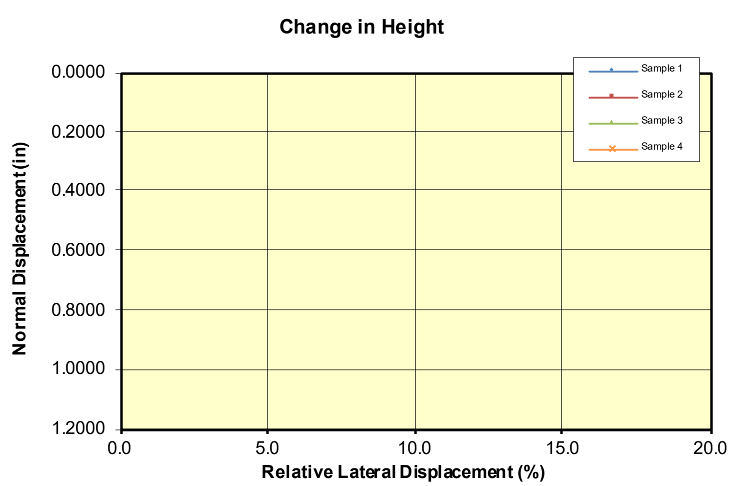
Consolidated Undrained Direct Shear (ASTM D3080M)

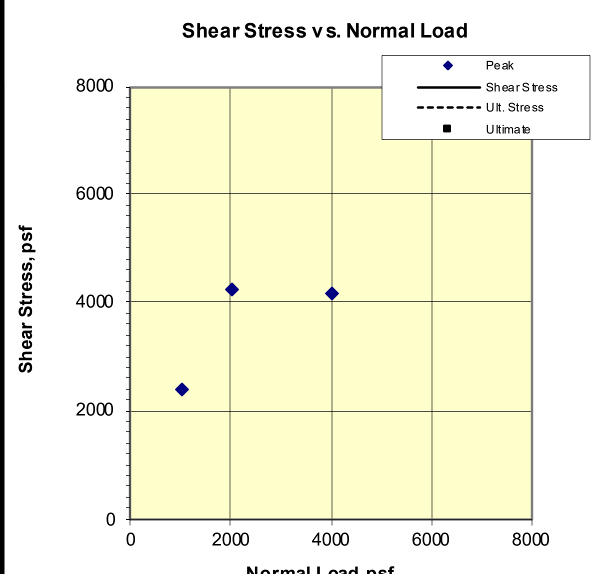
CTL Job #:	1022-034	Project #:	6231	By:	MD
Client:	McMillen Jacobs Associates	Date:	9/10/2021	Checked:	PJ
Project Name:		Remolding Info:			

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

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)				







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.

©

Corrosivity Test Summary

COOPER TESTING LABORATORY

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	Soil Visual Description
Boring	Sample, No.	Depth, ft.	As Rec.	Minimum	Saturated	mg/kg	mg/kg	%		(Redox)	At Test	
						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

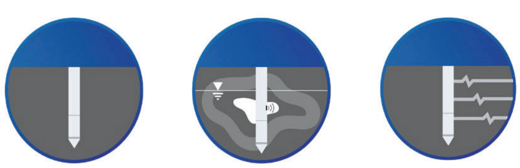
Project Start Date: 2023-Feb-21
Project End Date: 2023-Feb-21
Report Date: 2023-Feb-22

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

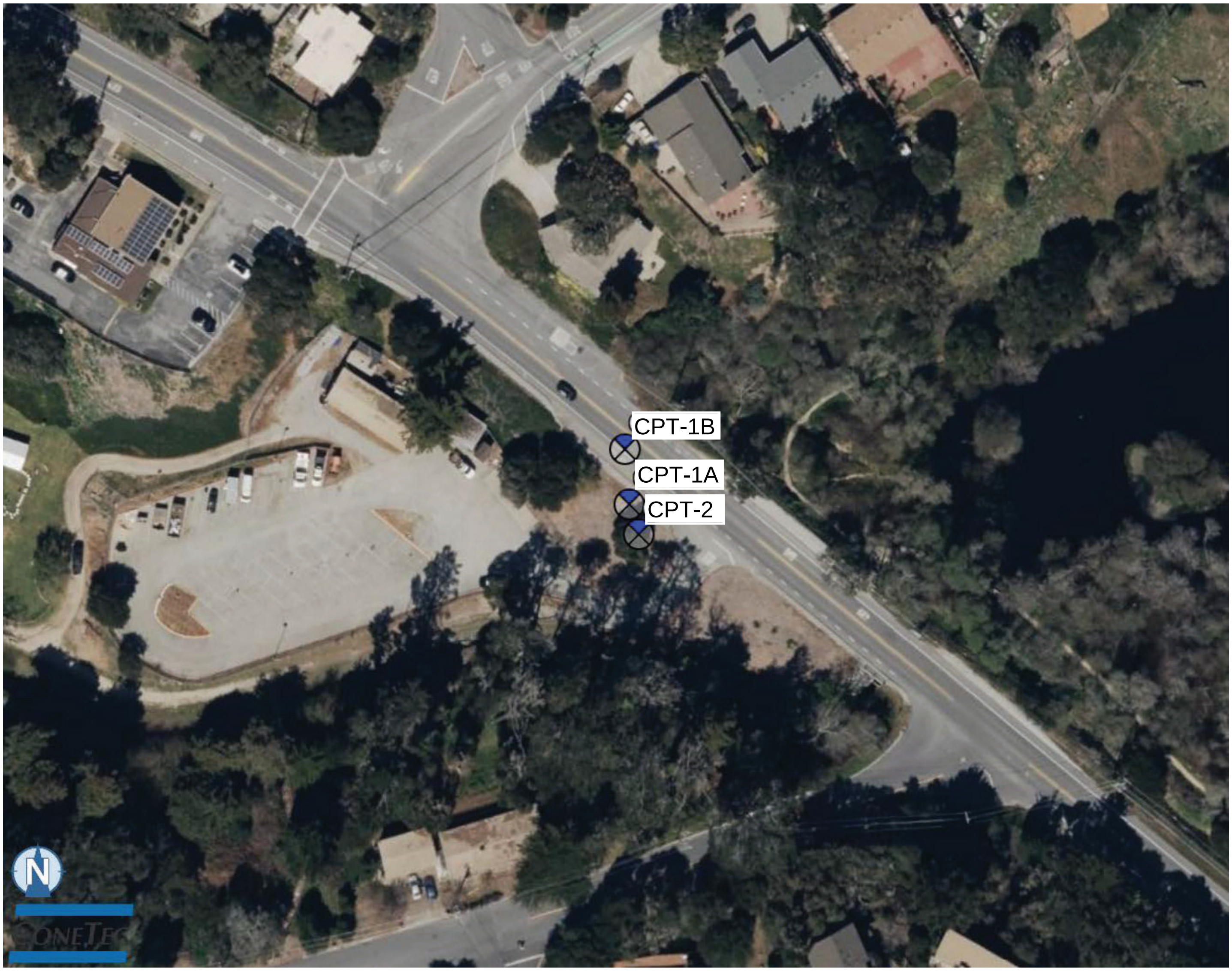
- ConeTec was retained by Delve Underground
- The "Report" refers to this report titled FORTAG Phase 1 Canyon Del Rey SR218 Segment
- ConeTec was retained to collect and provide the raw data ("Data") which is included in the Report.

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	

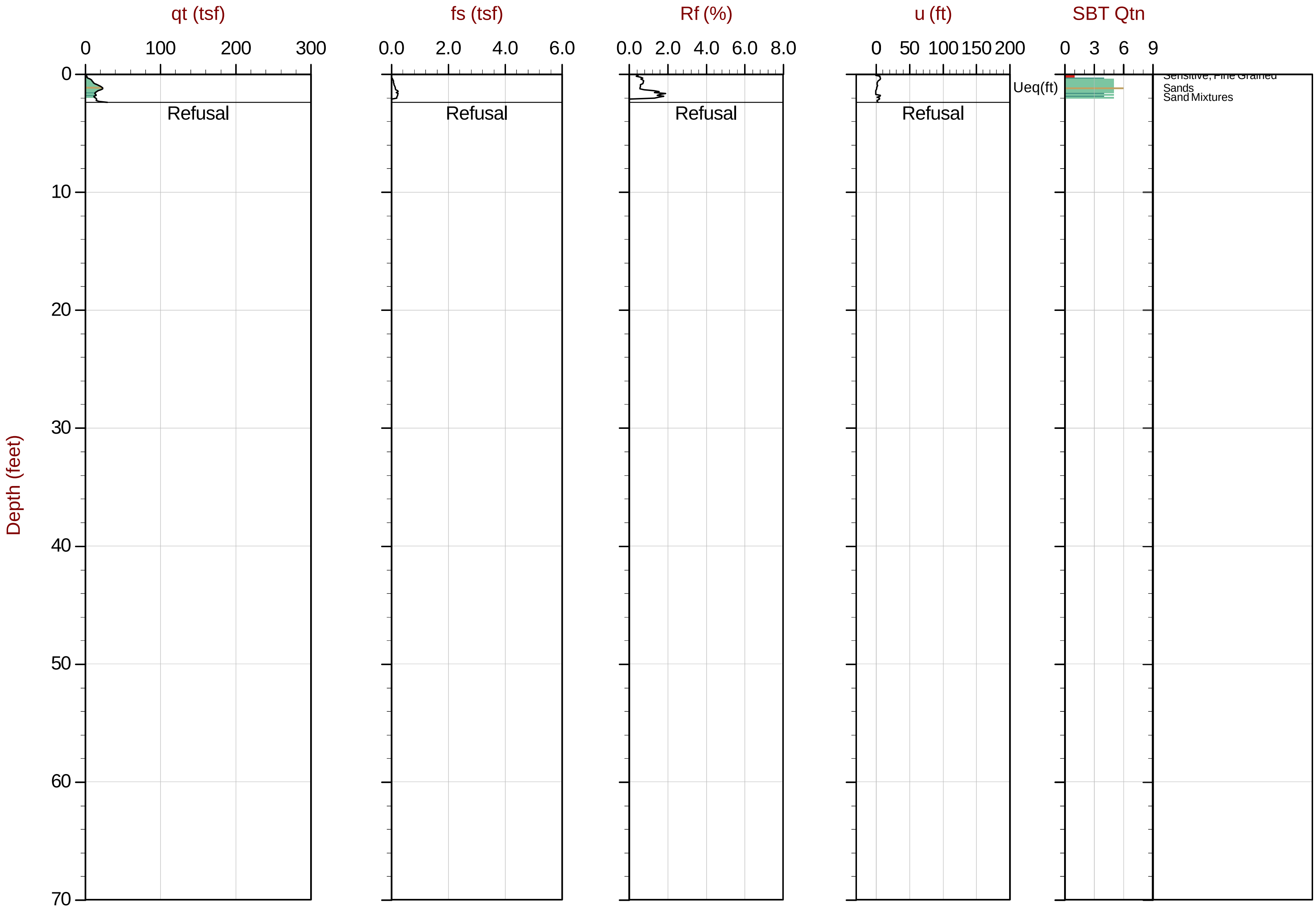
1. The assumed phreatic surface was based off the shallowest pore pressure dissipation tests performed within or nearest the sounding. Hydrostatic conditions were assumed for the calculated parameters.
2. The coordinates were collected using consumer grade GPS equipment. EPSG number: 32610 (WGS84 / UTM 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

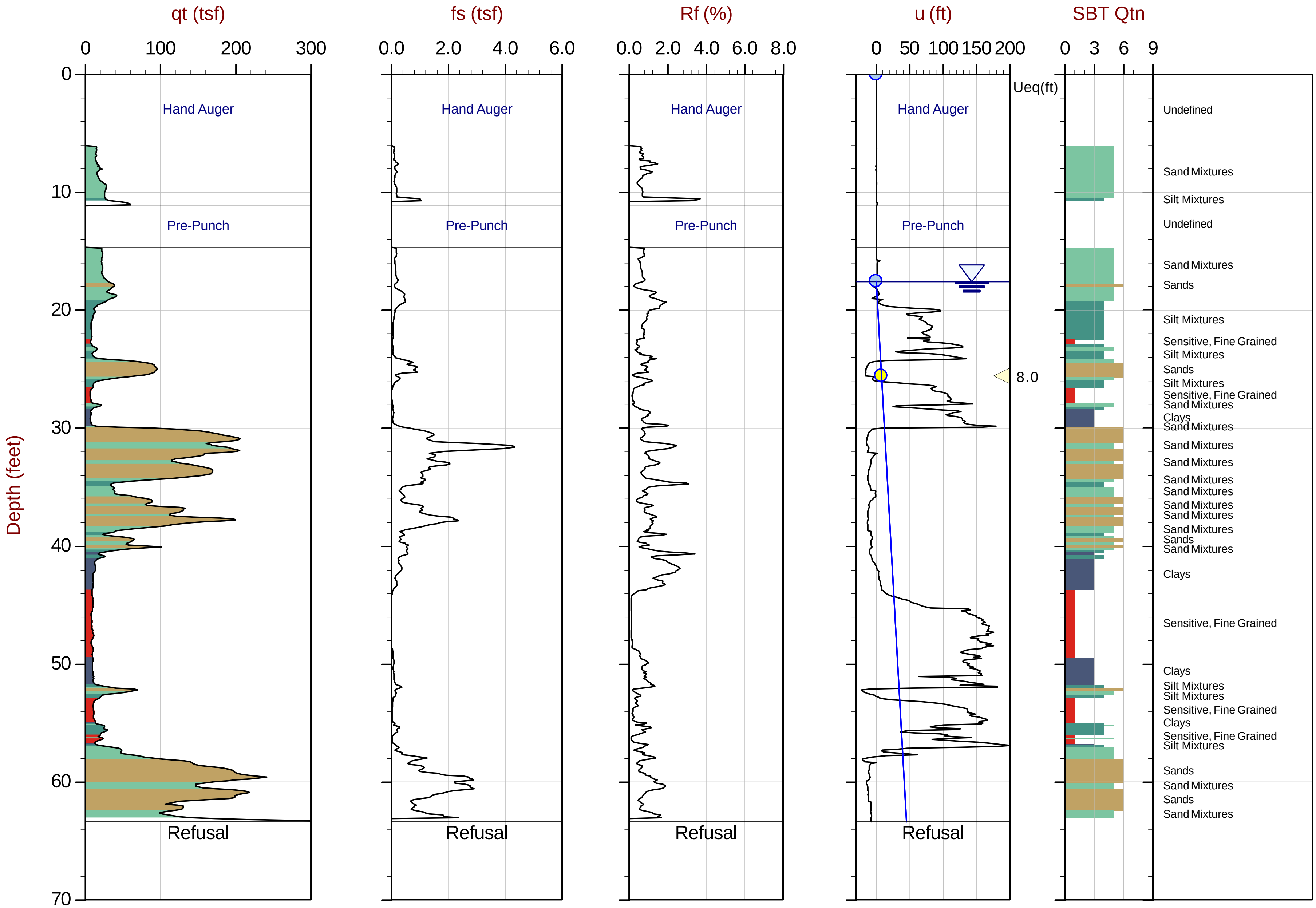
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

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

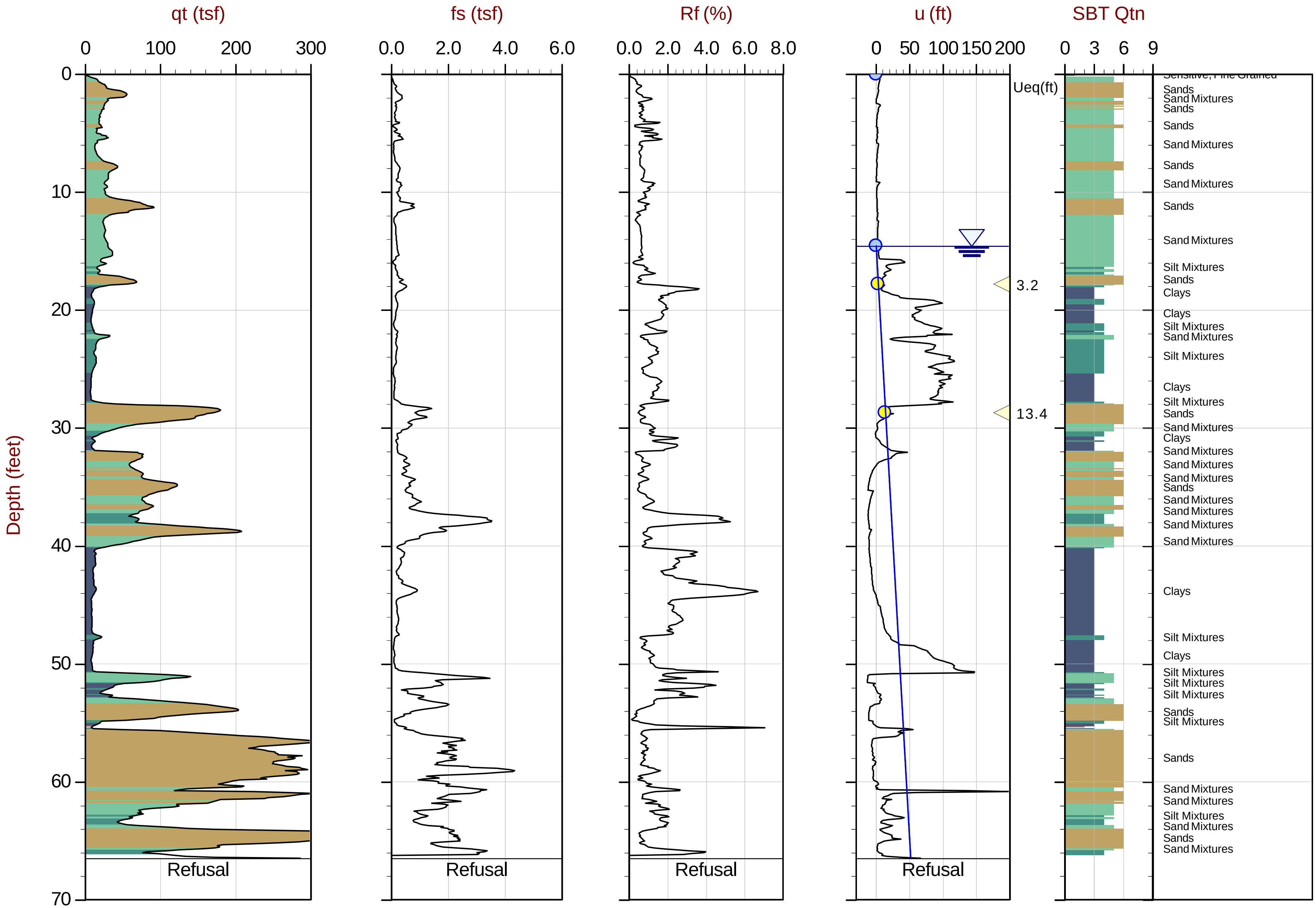
Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

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

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)

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

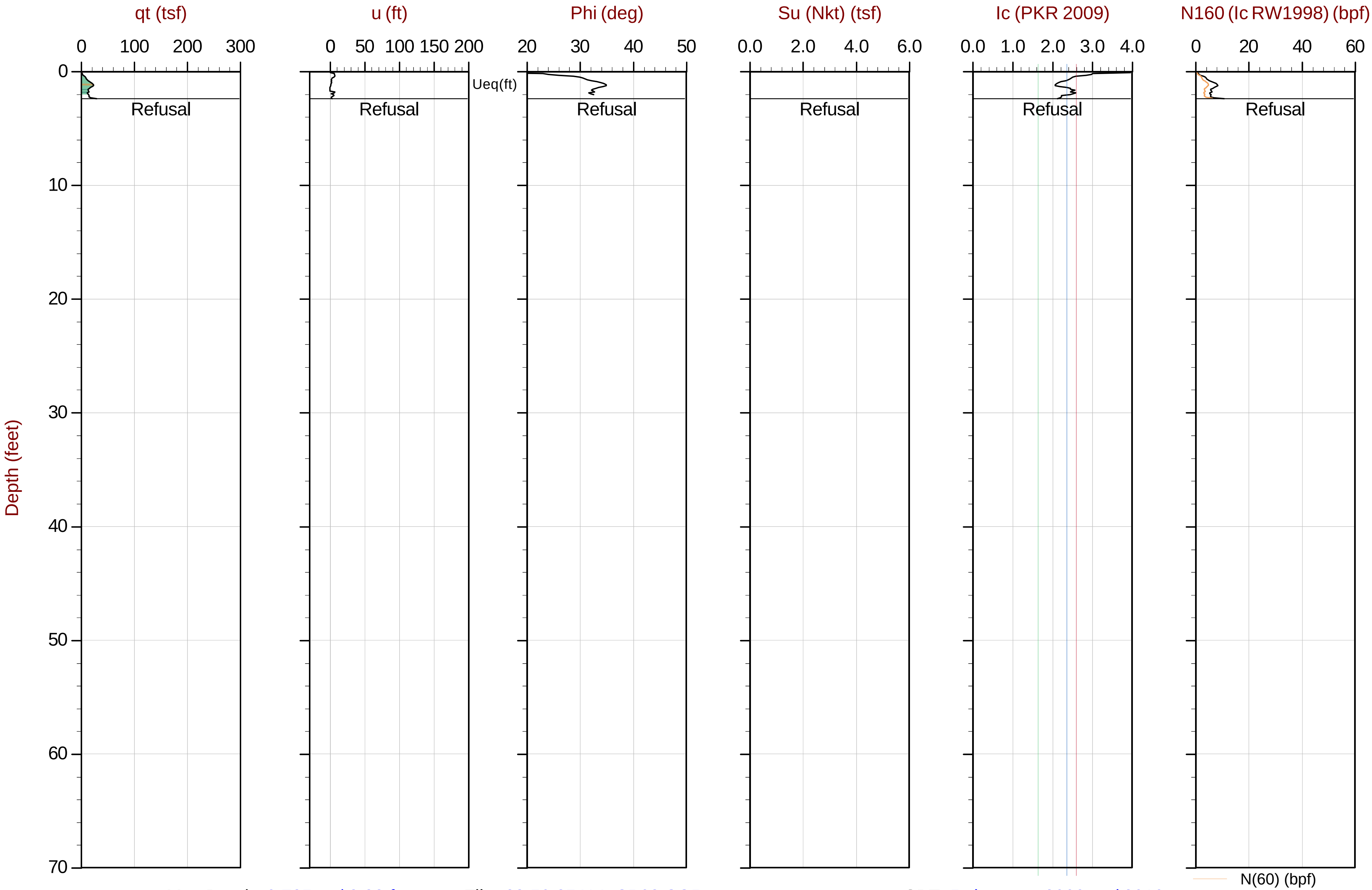
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

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

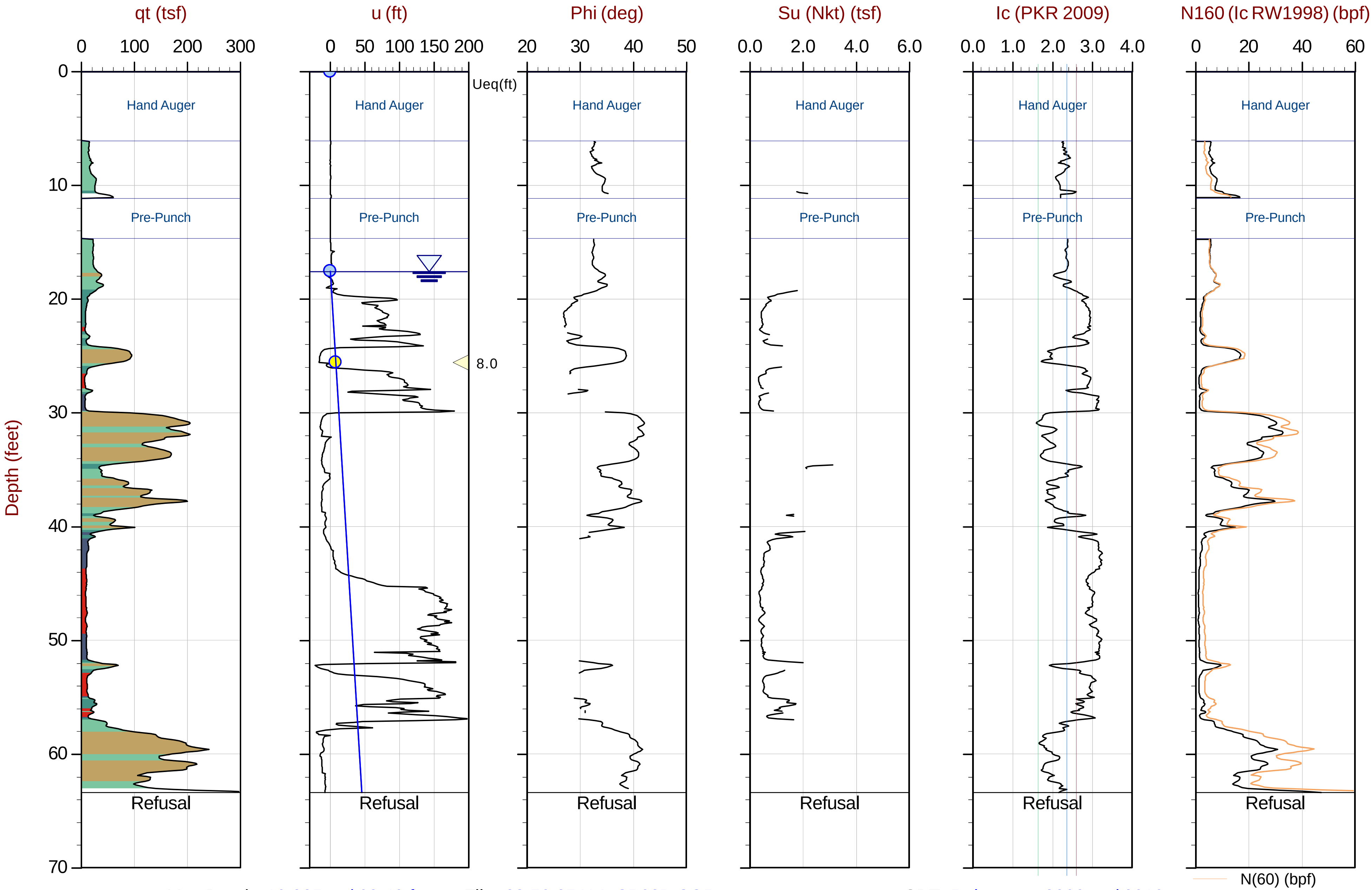
SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4050435m E: 604098m

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

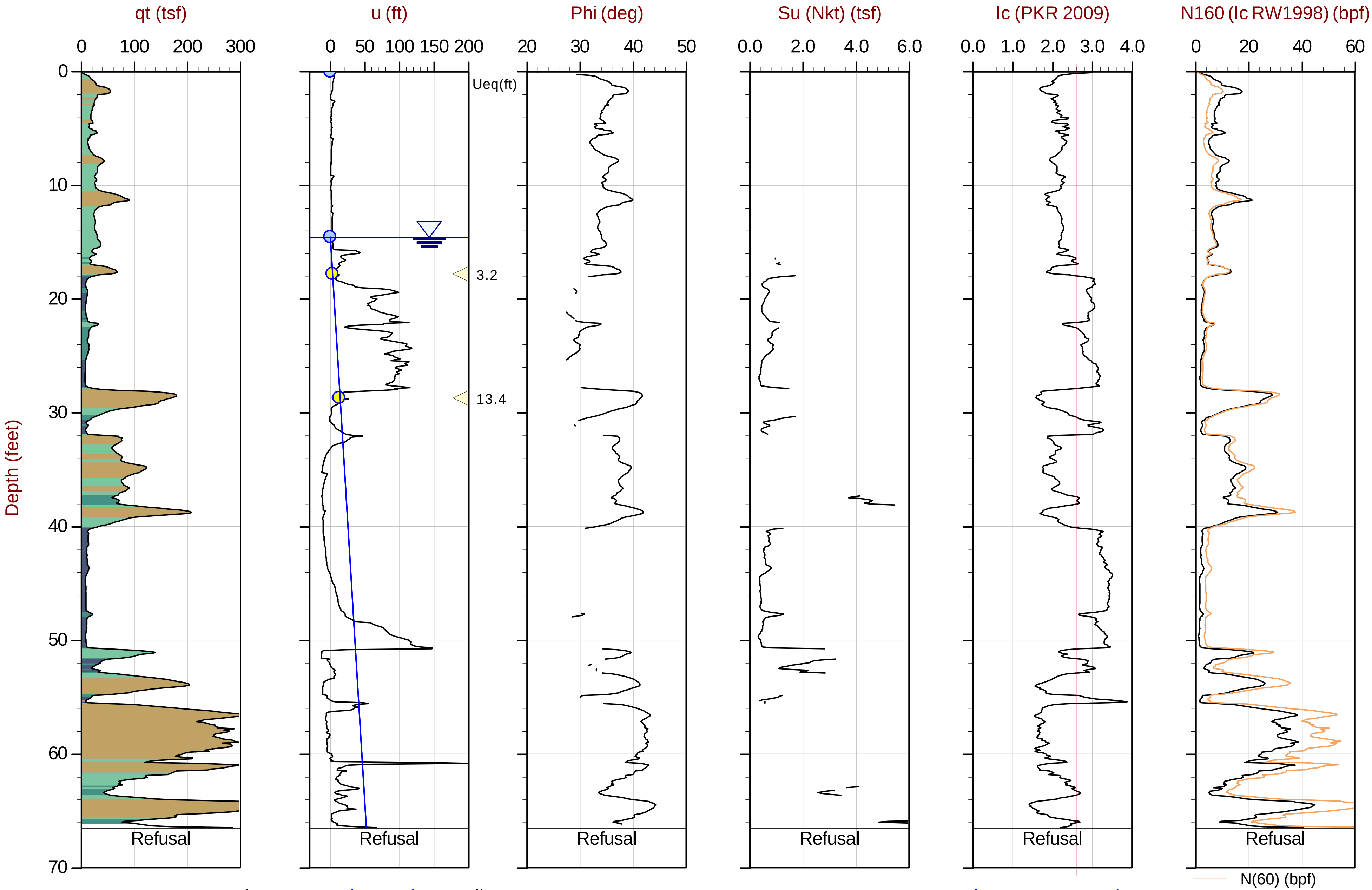
SBT: Robertson, 2009 and 2010
Coords: UTM 10S N: 4050446m E: 604097m

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.

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

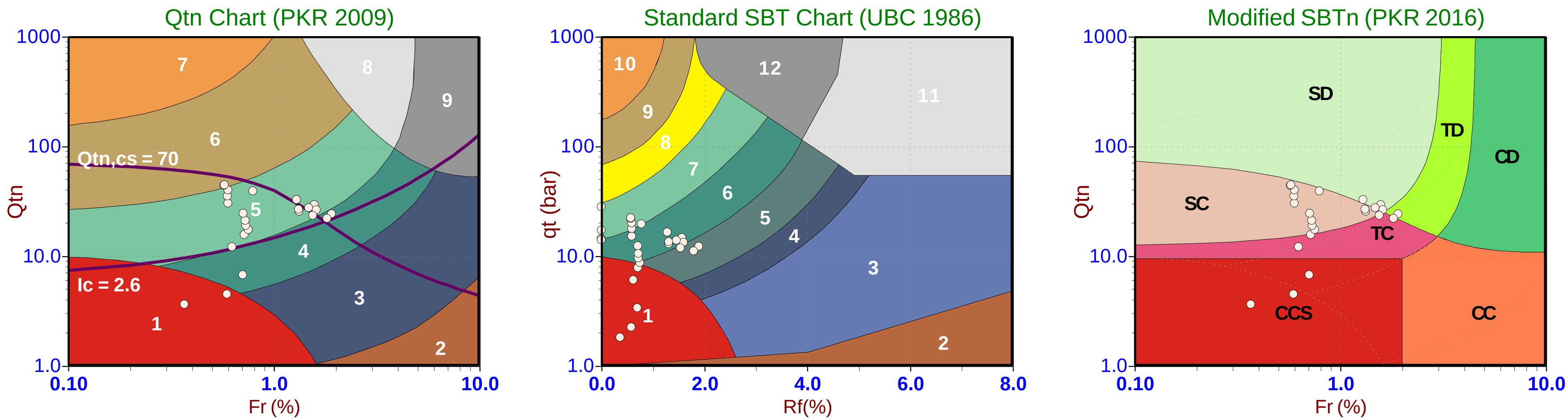
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

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



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

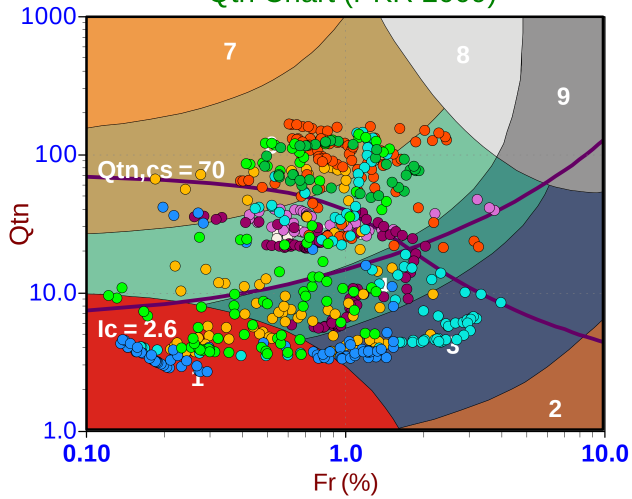
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

CONETEC

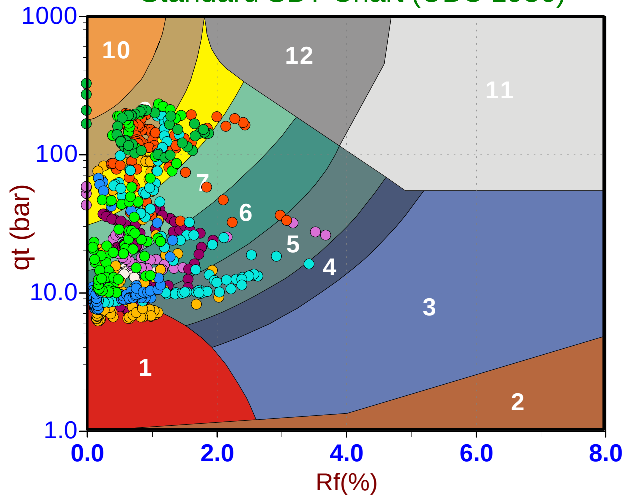
Delve Underground

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

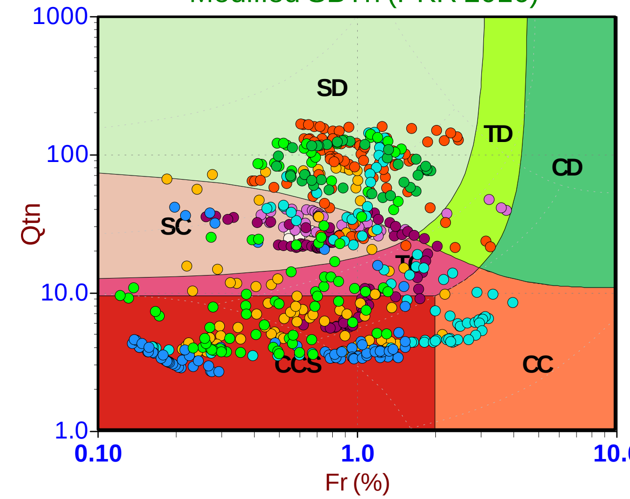
Qtn Chart (PKR 2009)



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

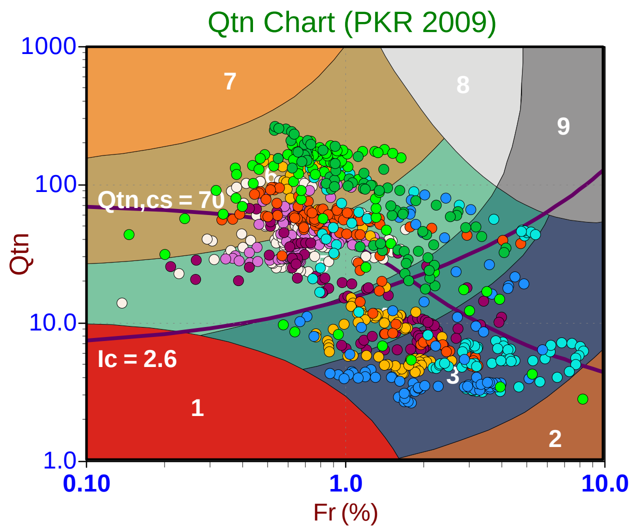
Legend

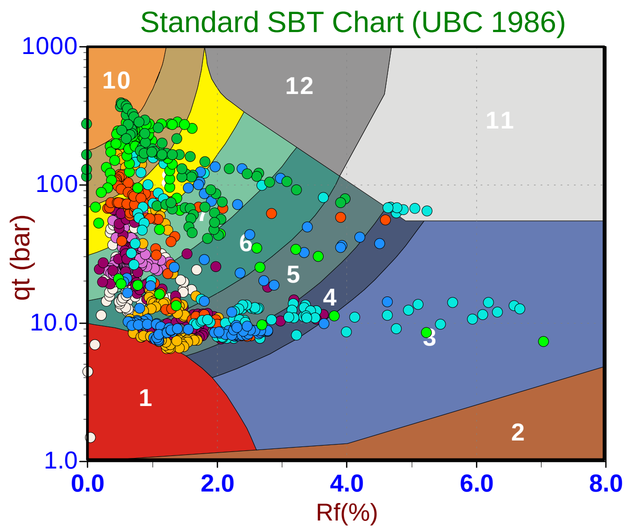
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

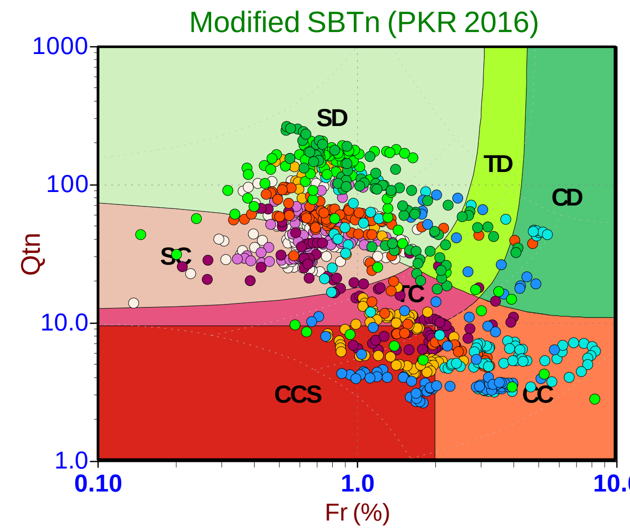
CONETEC

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







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)

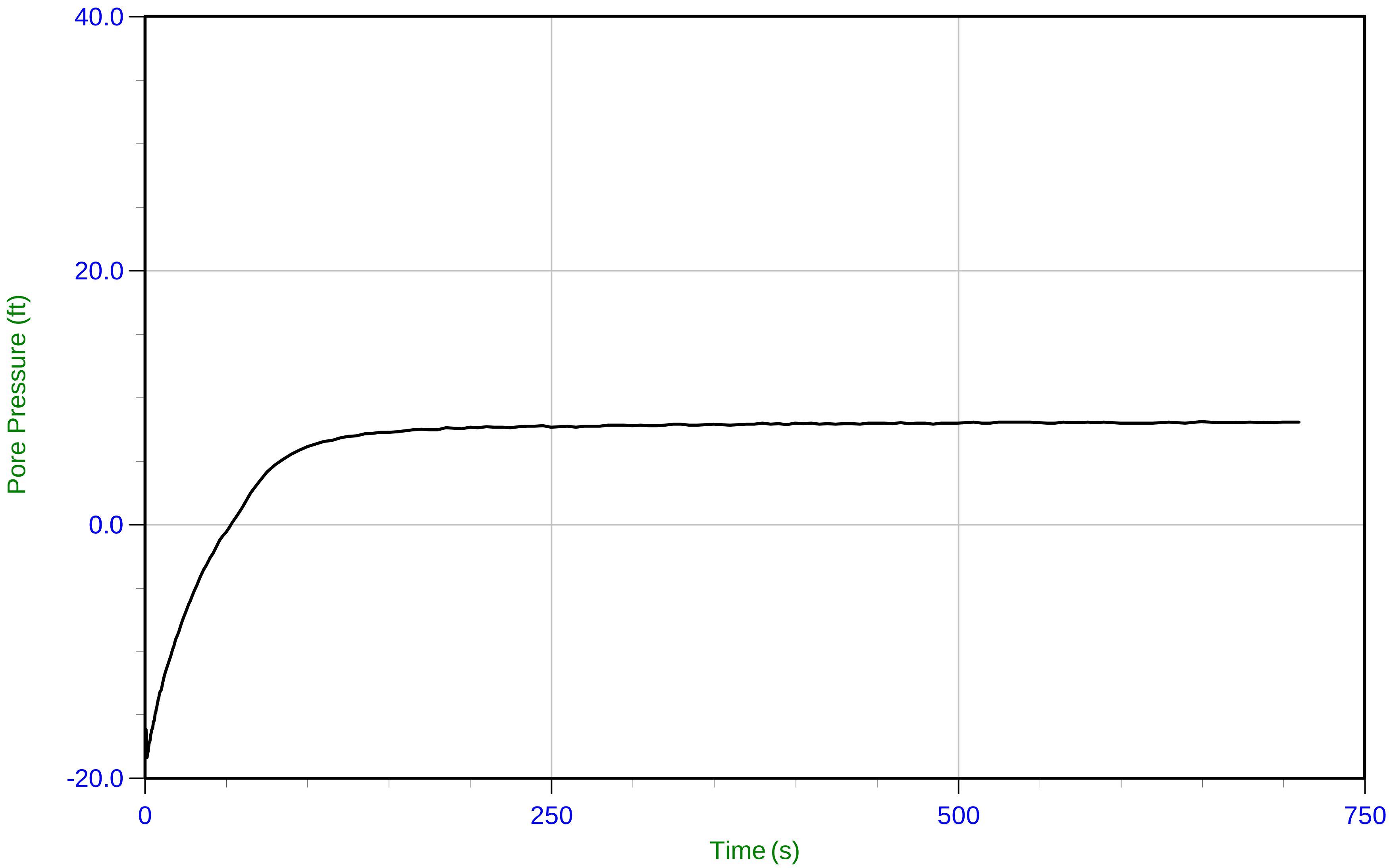
Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

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

Delve Underground	Job No: 23-56-25414 Date: 02/21/2023 11:57 Site: FORTAG Phase 1 Canyon Del Rey SR218 Segment	Sounding: CPT- 1B Cone: 795:T1500F15U35 Area=15 cm²

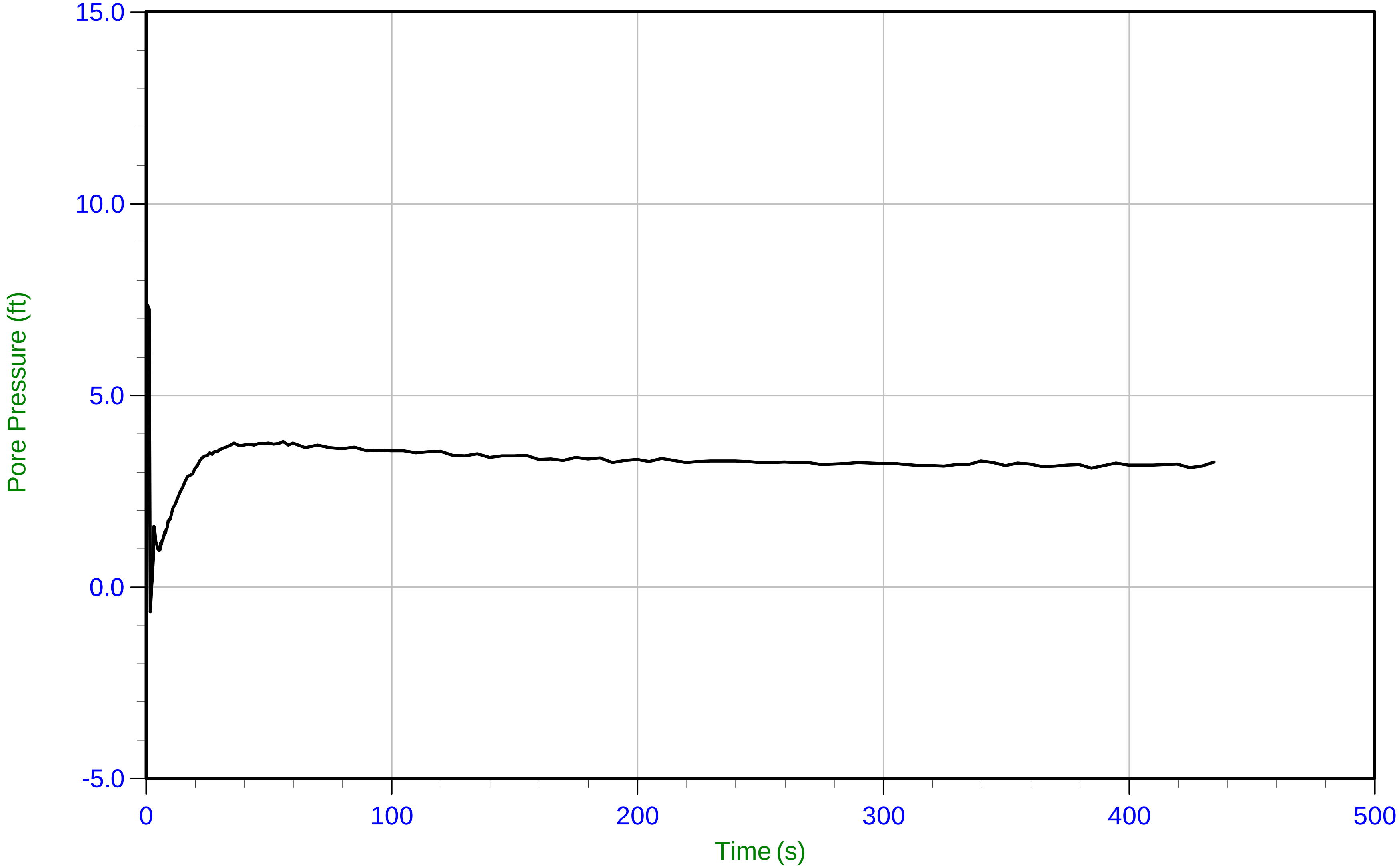


Trace Summary:

Filename: 23-56-25414_CP03B.ppf2	u Min: -18.3 ft	WT: 5.354 m / 17.567 ft
Depth: 7.800 m / 25.590 ft	u Max: 8.1 ft	Ueq: 8.0 ft
Duration: 709.9 s	u Final: 8.0 ft	

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:

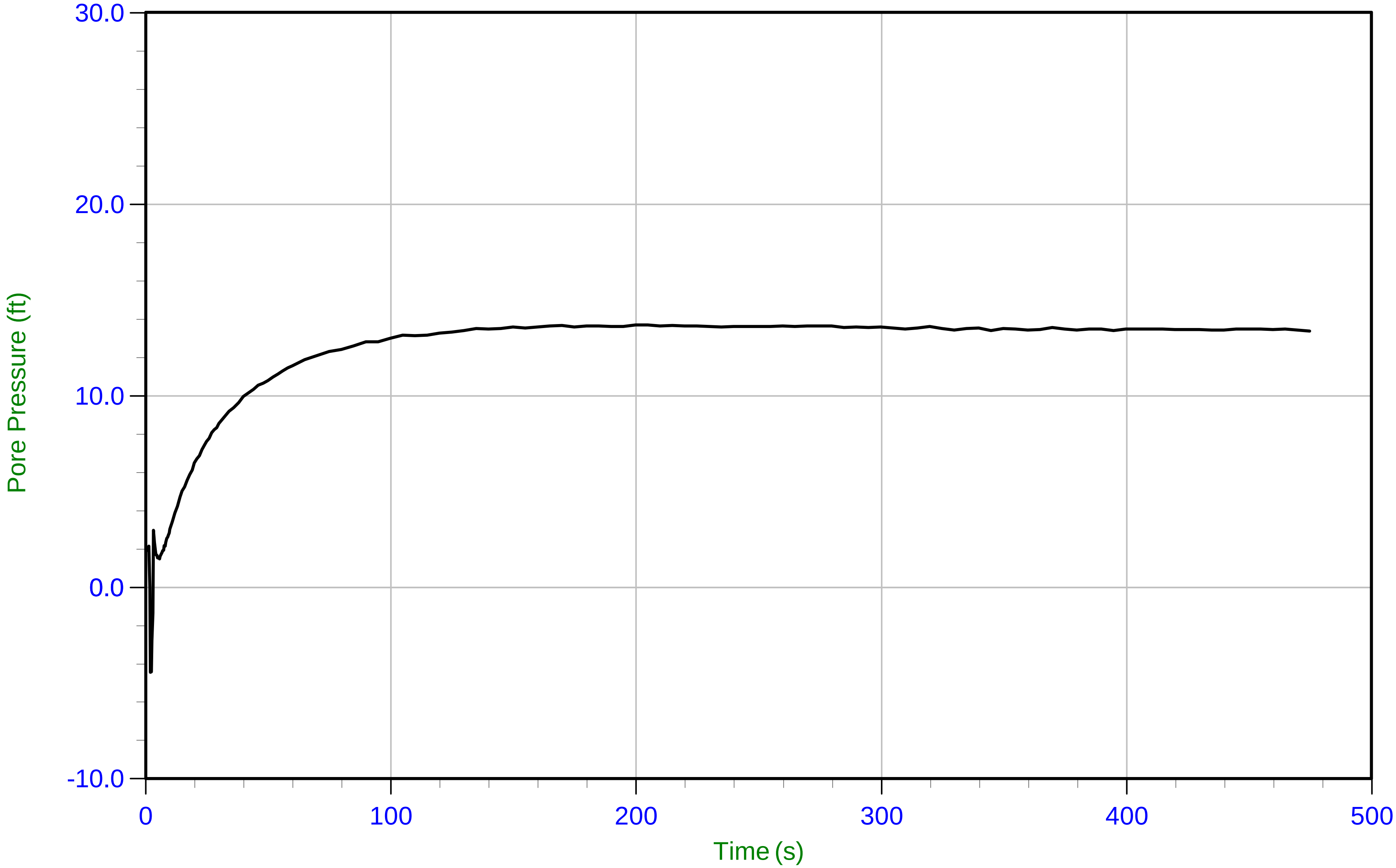
Filename: 23-56-25414_CP04.ppf2
Depth: 5.425 m / 17.798 ft
Duration: 434.9 s

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

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.

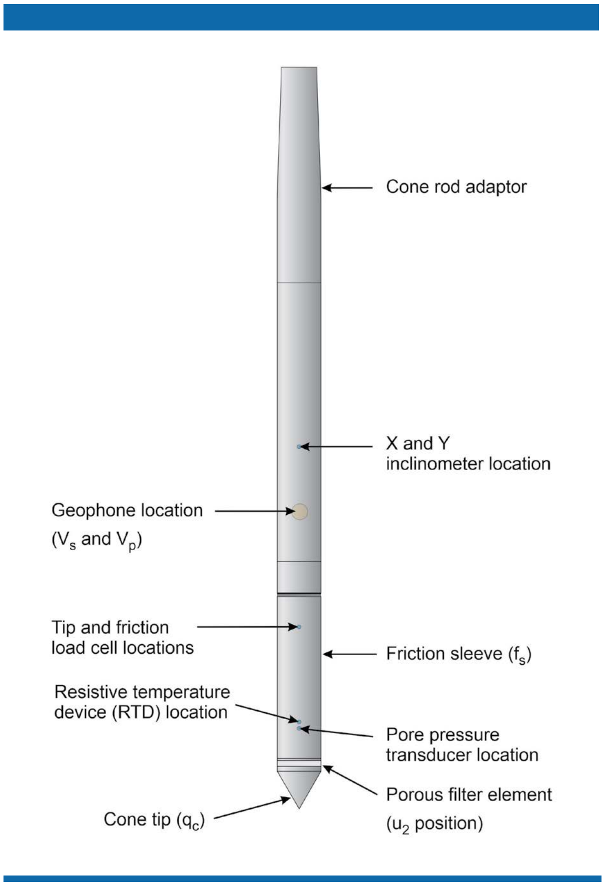


Figure CPTu. Piezocone Penetrometer (15 cm²)

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.

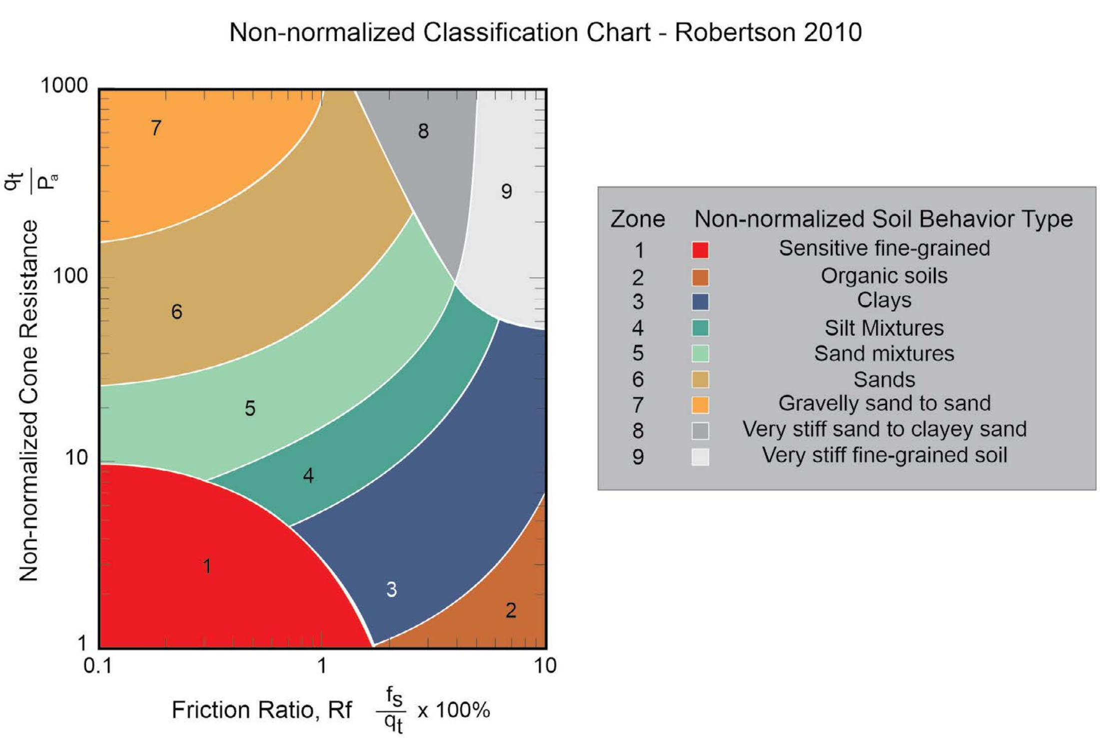


Figure SBT. Non-Normalized Soil Behavior Type Classification Chart (SBT)

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):

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

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).

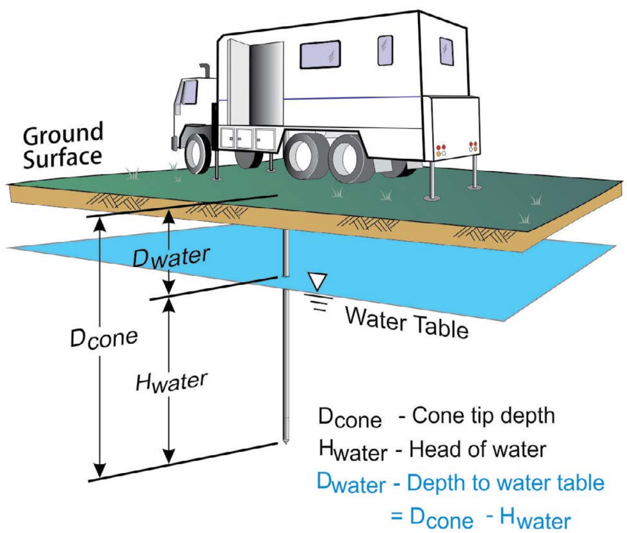


Figure PPD-1. Pore pressure dissipation test setup

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.

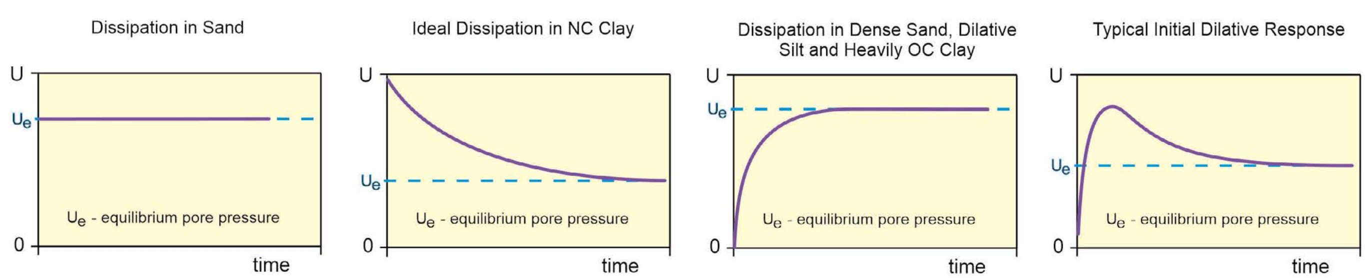


Figure PPD-2. Pore pressure dissipation curve examples

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)

CONETEC

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.

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.

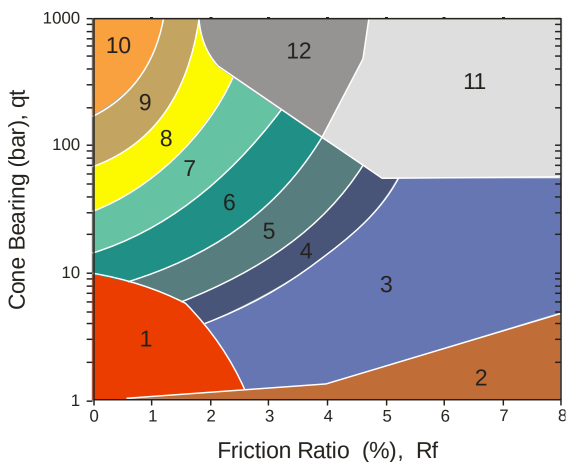
Effective vertical overburden stresses are calculated based on a hydrostatic distribution of equilibrium pore pressures below the water table or from a user defined equilibrium pore pressure profile (typically obtained from CPT dissipation tests) or a combination of the two. For over water projects the 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.

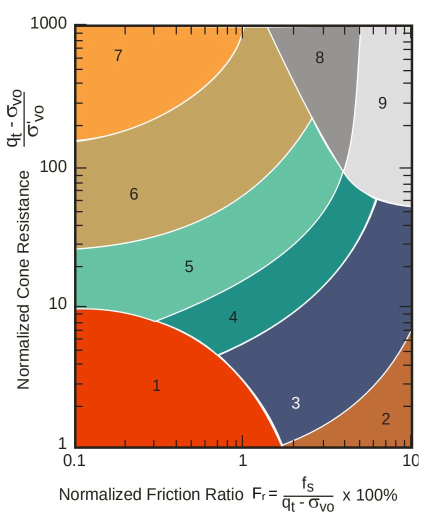


Zone	qt / N	Soil Behavior Type
1	2	sensitive fine grained
2	1	organic material
3	1	clay
4	1.5	silty clay to clay
5	2	clayey silt to silty clay
6	2.5	sandy silt to clayey silt
7	3	silty sand to sandy silt
8	4	sand to silty sand
9	5	sand
10	6	gravelly sand to sand
11	1	very stiff fine grained *
12	2	sand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

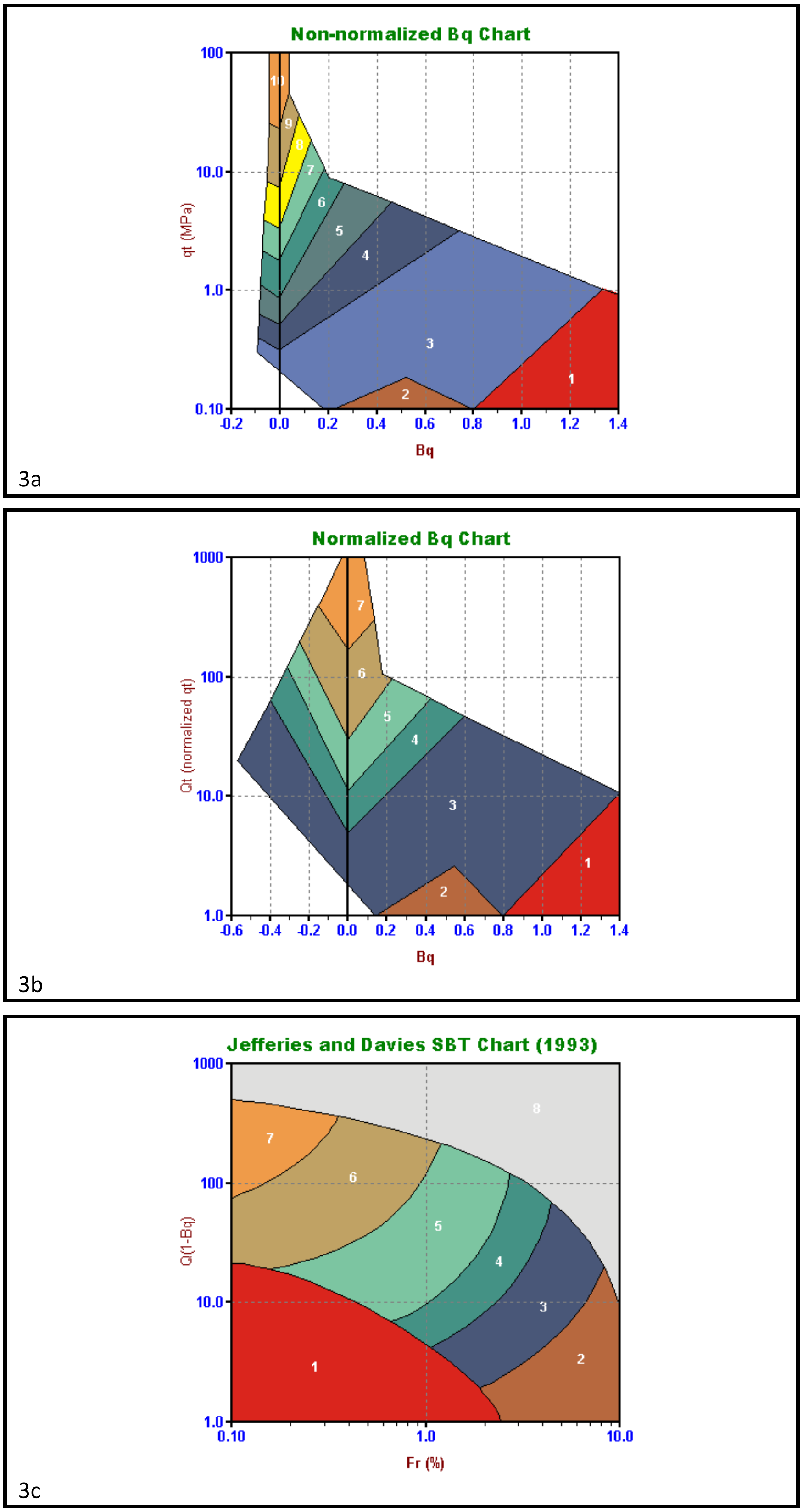


Figure 3. Alternate Soil Behavior Type Charts

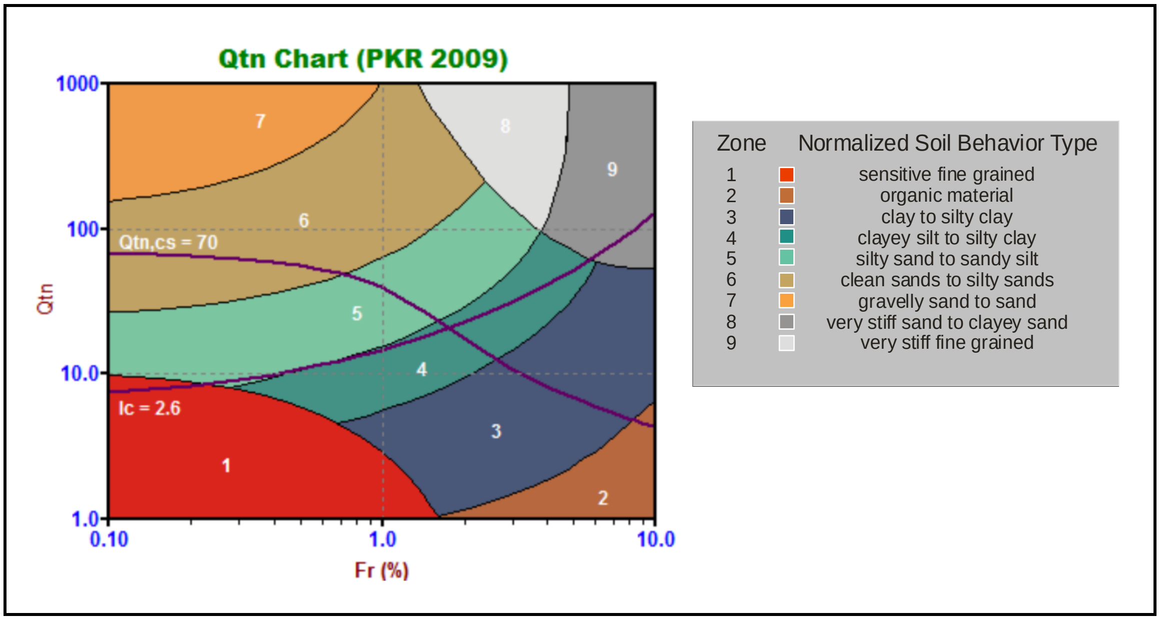


Figure 4. Normalized Soil Behavior Type Chart using Q_{tn} (SBT Qtn)

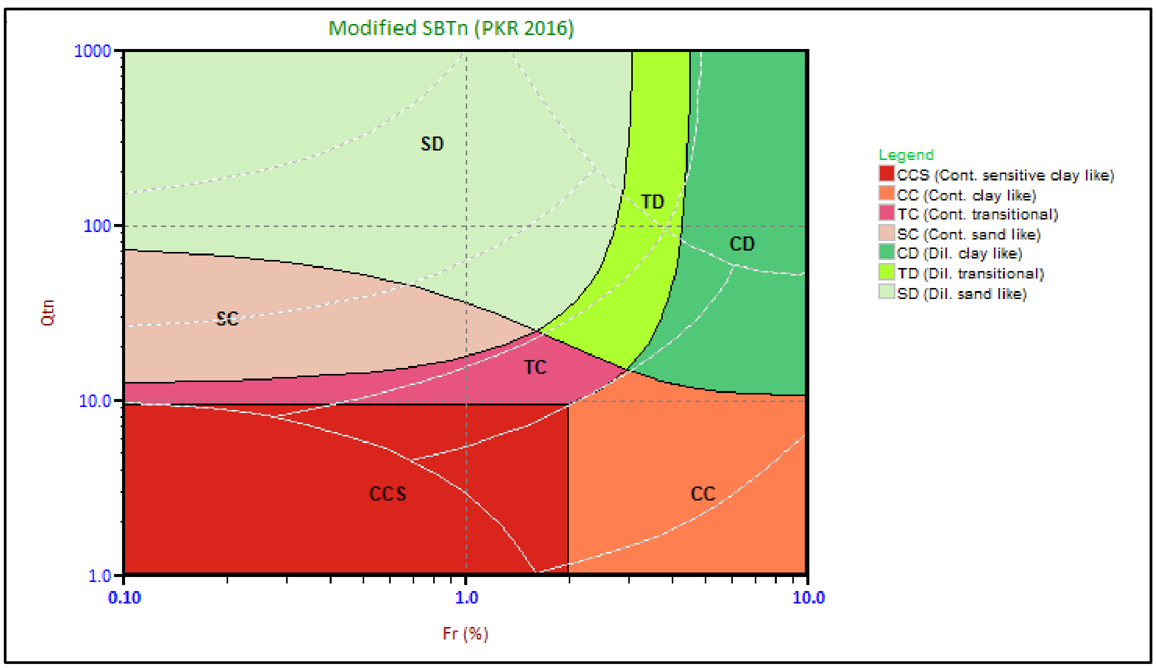


Figure 5. Modified SBTn Behavior Based Chart

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	$K_o = (1 - \sin\Phi')\ OCR^{\sin\Phi'}$	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	$(N_1)_{60} = C_n \bullet N_{60}$	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) $(N_1)_{60}Ic = C_n \bullet (N_{60}\, I_c)$ 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	$Su = \dfrac{qt - \sigma_v}{N_{kt}}$	1, 5
Su or Su (Ndu)	Undrained shear strength based on pore pressure S_u factor $N_{\Delta u}$ is user selectable	$Su = \dfrac{u_2 - u_{eq}}{N_{\Delta u}}$	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)	$= \dfrac{\Delta u}{qt}$ *where:* $\Delta u = u - u_{eq}$ *and* *u = dynamic pore pressure* u_{eq} *= equilibrium pore pressure*	CK*
Bq	Pore pressure parameter	$Bq = \dfrac{\Delta u}{qt - \sigma_v}$ 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 $m' = 1 - \dfrac{0.28}{1+(I_c/2.65)^{25}}$ 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 $(N_1)60$	$K_{SPT} = 1 + ((0.75/30) \cdot (FC - 5))$	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}	$q_{c1ncs} = q_{c1n} \cdot K_{cpt}$	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	$\dfrac{Su(Liq)}{\sigma_v'} = 0.03 + 0.0143(q_{c1})$ Note: σ_v' and s_v' are synonymous	13
Su(Liq)/ESv (PKR 2010)	Liquefied shear strength ratio as defined by Robertson (2010)	$\dfrac{Su(Liq)}{\sigma_v'}$ 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

No.	Reference
1	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.
2	Robertson, P.K., 1990, "Soil Classification Using the Cone Penetration Test", Canadian Geotechnical Journal, Volume 27. This includes the discussions and replies.
3	Robertson, P.K. and Wride (Fear), C.E., 1998, "Evaluating cyclic liquefaction potential using the cone penetration test", Canadian Geotechnical Journal, 35: 442-459.
4	Robertson, P.K. and Wride, C.E., 1997, "Cyclic Liquefaction and its Evaluation Based on SPT and CPT", NCEER Workshop Paper, January 22, 1997.
5	Lunne, T., Robertson, P.K. and Powell, J. J. M., 1997, " Cone Penetration Testing in Geotechnical Practice," Blackie Academic and Professional.
6	Plewes, H.D., Davies, M.P. and Jefferies, M.G., 1992, "CPT Based Screening Procedure for Evaluating Liquefaction Susceptibility", 45th Canadian Geotechnical Conference, Toronto, Ontario, October 1992.
7	Jefferies, M.G. and Davies, M.P., 1993, "Use of CPTu to Estimate equivalent N_{60}", Geotechnical Testing Journal, 16(4): 458-467.
8	Been, K. and Jefferies, M.P., 1985, "A state parameter for sands", Geotechnique, 35(2), 99-112.
9	Schmertmann, 1978, "Guidelines for Cone Penetration Test Performance and Design", Federal Highway Administration Report FHWA-TS-78-209, U.S. Department of Transportation.
10	Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils, Salt Lake City, 1996, chaired by Leslie Youd.
11	Kulhawy, F.H. and Mayne, P.W., 1990, "Manual on Estimating Soil Properties for Foundation Design, Report No. EL-6800", Electric Power Research Institute, Palo Alto, CA, August 1990, 306 p.
12	Olson, S.M. and Stark, T.D., 2002, "Liquefied strength ratio from liquefied flow failure case histories", Canadian Geotechnical Journal, 39: 951-966.
13	Olson, Scott M. and Stark, Timothy D., 2003, "Yield Strength Ratio and Liquefaction Analysis of Slopes and Embankments", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, August 2003.
14	Jamiolkowski, M.B., Lo Presti, D.C.F. and Manassero, M., 2003, "Evaluation of Relative Density and Shear Strength of Sands from CPT and DMT", Soil Behaviour and Soft Ground Construction, ASCE, GSP N0. 119, 201-238.
15	Robertson, P.K., 2009, "Interpretation of cone penetration tests – a unified approach", Canadian Geotechnical Journal, 46: 1337-1355.
16	Robertson, P.K., 2010, "Evaluation of Flow Liquefaction and Liquefied Strength Using the Cone Penetration Test", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, June 2010.
17	Mayne, P.W. and Kulhawy, F.H., 1982, "Ko-OCR Relationships in Soil", Journal of the Geotechnical Engineering Division, ASCE, Vol. 108, GT6, pp. 851-872.
18	Mayne, P.W., Robertson P.K. and Lunne T., 1998, "Clay stress history evaluated from seismic piezocone tests", Proceedings of the First International Conference on Site Characterization – ISC '98, Atlanta Georgia, Volume 2, 1113-1118.

No.	Reference
19	Mayne, P.W., 2014, "Generalized CPT Method for Evaluating Yield Stress in Soils", Geocharacterization for Modeling and Sustainability (GSP 235: Proc. GeoCongress 2014, Atlanta, GA), ASCE, Reston, Virginia: 1336-1346.
20	Mayne, P.W., 2015, "Geocharacterization by In-Situ Testing", Continuing Education Course, Vancouver, BC, January 6-8, 2015.
21	Robertson, P.K. and Fear, C.E., 1995, "Liquefaction of sands and its evaluation", Proceedings of the First International Conference on Earthquake Engineering, Keynote Lecture IS Tokyo '95, Tokyo Japan, 1995.
22	Mayne, P.W., Peuchen, J. and Boumeester, D., 2010, "Soil unit weight estimation from CPTs", Proceeding of the 2nd International Symposium on Cone Penetration Testing (CPT '10), Vol 2, Huntington Beach, California; Omnipress: 169-176.
23	Mayne, P.W., 2007, "NCHRP Synthesis 368 on Cone Penetration Test", Transportation Research Board, National Academies Press, Washington, D.C., 118 pages.
24	Mayne, P.W., 2014, "Interpretation of geotechnical parameters from seismic piezocone tests.", Key note address #2, proceedings, 3rd International Symposium on Cone Penetration Testing (CPT'14, Las Vegas), ISSMGE Technical Committee TC102.
25	Winckler, Christina, Davidson, Richard, Yenne, Lisa, Pilz, Jorgen, 2014, "CPTu-Based State Characterization of Tailings Liquefaction Susceptibility", Tailings and Mine Waste, 2014.
25a	Winckler, Christina, Davidson, Richard, Yenne, Lisa, Pilz, Jorgen, 2014, "CPTu-Based State Characterization of Tailings Liquefaction Susceptibility", Powerpoint presentation, Tailings and Mine Waste, 2014.
26	Schneider, J.A. and Moss, R.E.S., 2011, "Linking cyclic stress and cyclic strain based methods for assessment of cyclic liquefaction triggering in sands", Geotechnique Letters 1, 31-36.
27	Rice, A., 1984, "The Seismic Cone Penetrometer", M.A.Sc. thesis submitted to the University of British Columbia, Dept. of Civil Engineering, Vancouver, BC, Canada.
28	Gillespie, D.G., 1990, "Evaluating Shear Wave Velocity and Pore Pressure Data from the Seismic Cone Penetration Test", Ph.D. thesis submitted to the University of British Columbia, Dept. of Civil Engineering, Vancouver, BC, Canada.
29	Robertson, P.K and Cabal, K.L., 2010, "Estimating soil unit weight from CPT", Proceedings of the 2nd International Symposium on Cone Penetration Testing (CPT '10), Huntington Beach, California.
30	Robertson, P.K., 2016, "Cone penetration test (CPT)-based soil behaviour type (SBT) classification system – an update", Canadian Geotechnical Journal, July 2016.
31	Robertson, P.K., 2012, "Interpretation of in-situ tests – some insights", Mitchell Lecture, ISC'4, Recife, Brazil.
32	Robertson, P.K., Cabal, K.L. 2015, "Guide to Cone Penetration Testing for Geotechnical Engineering", 6th Edition.

Appendix E

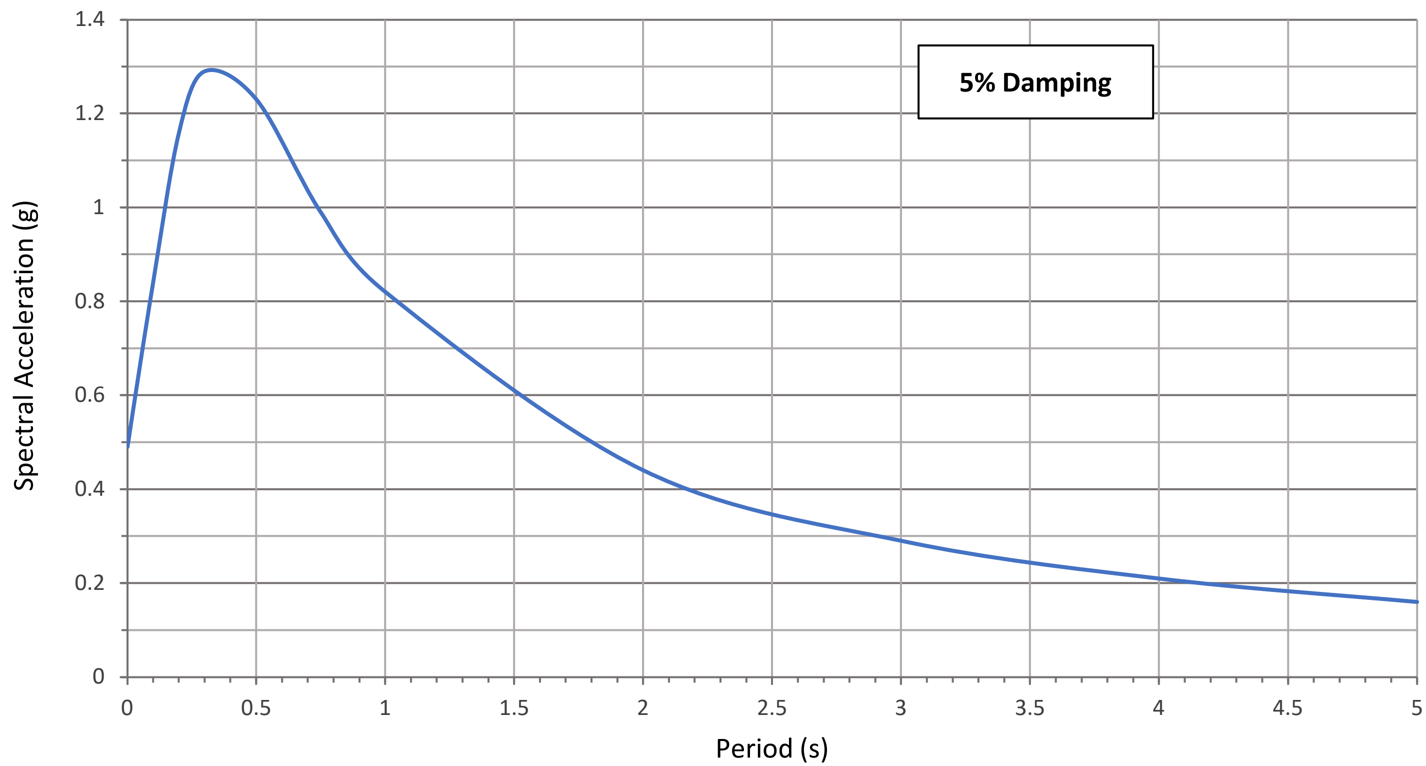
Project Name: FORTAG SR 218 Undercrossing Bridge and Retaining Wall No. 1

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.

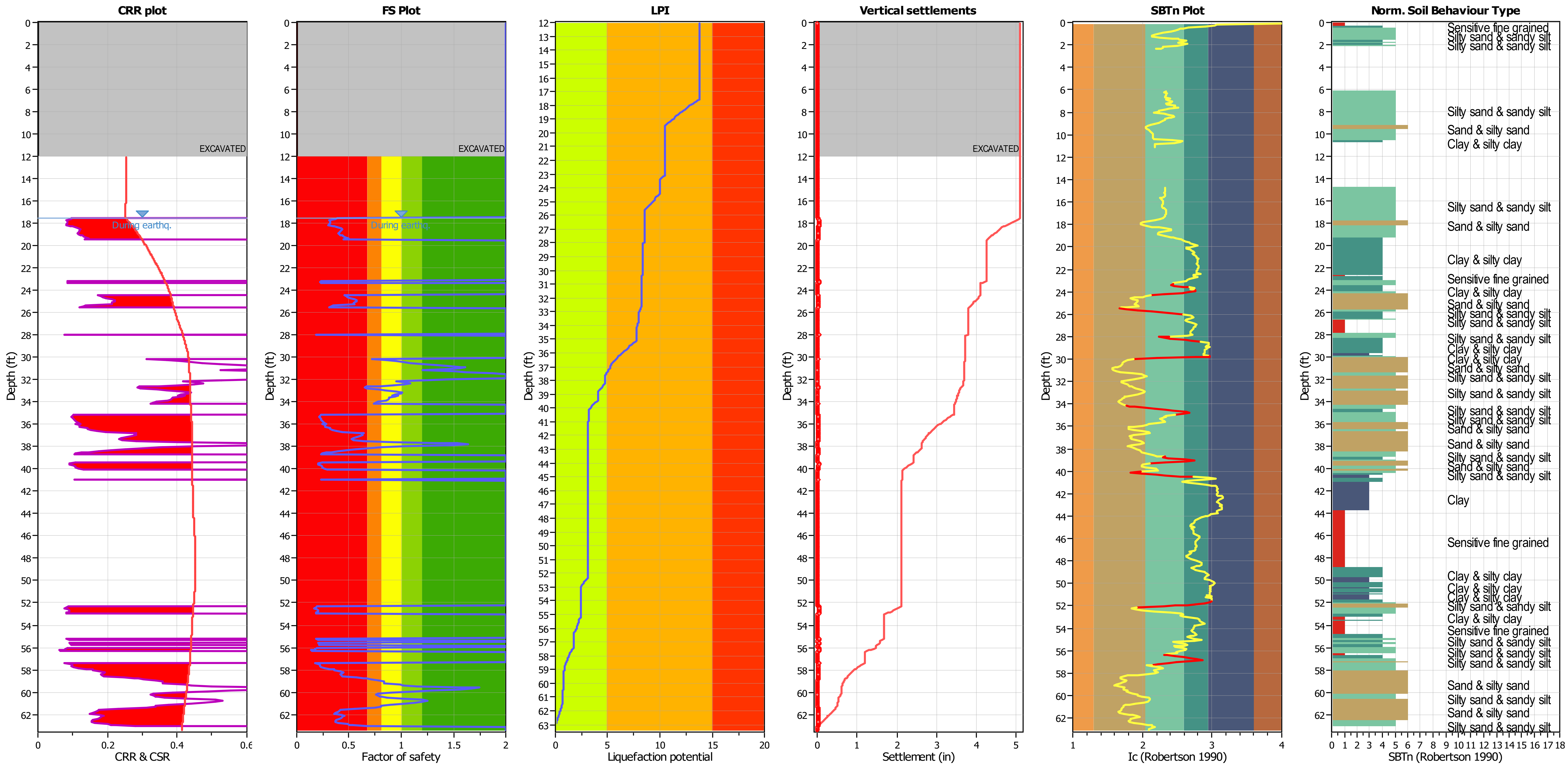
V_{s30}: 720 feet/sec (220 meters/sec)
PGA: 0.49g
Mean Moment Magnitude (for PGA): M = 6.75

Appendix F

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-01
Total depth: 63.40 ft

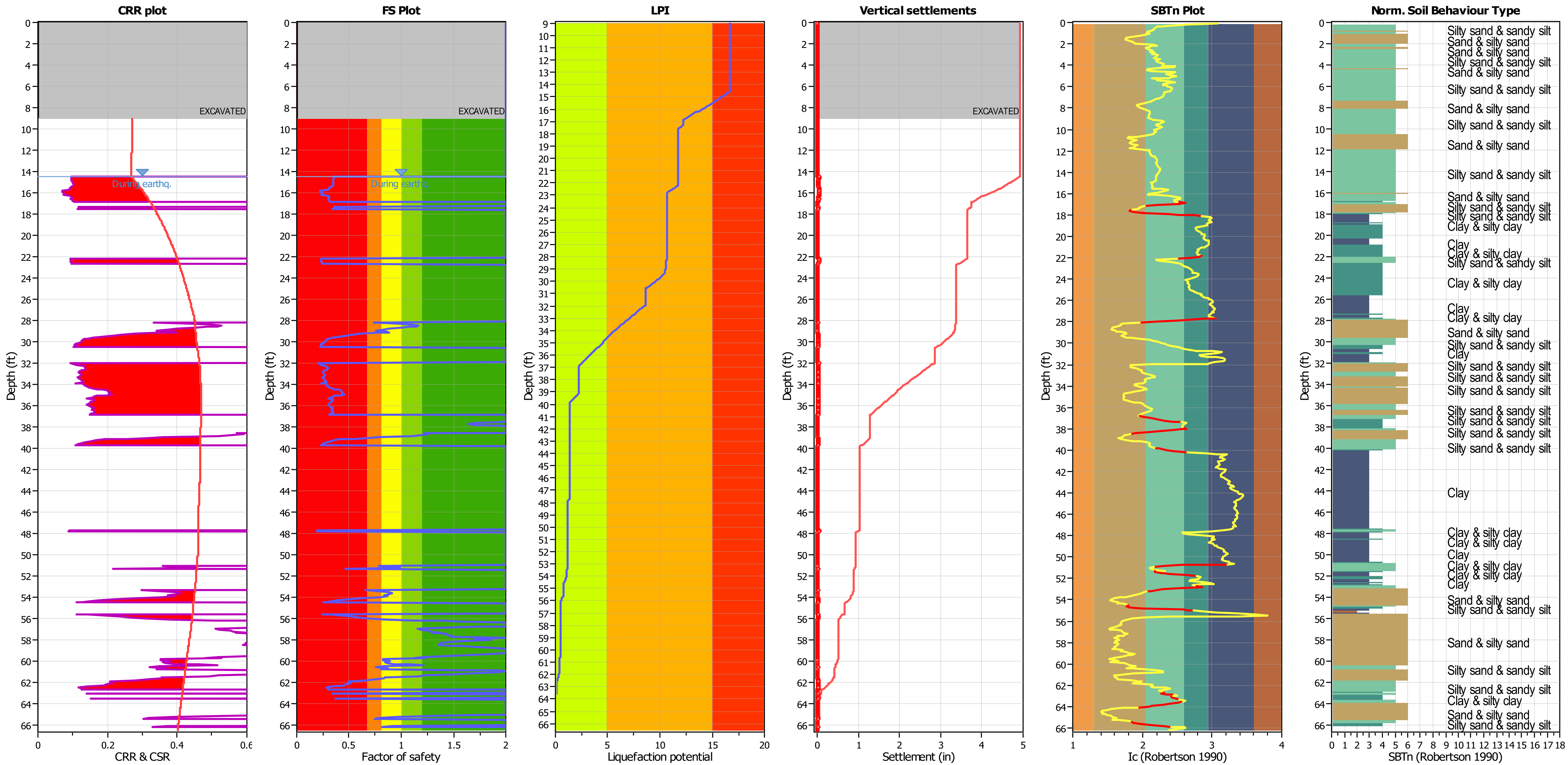


Analysis method:	NCEER (1998)	G.W.T. (in-situ):	17.60 ft	Excavation:	Yes	Clay like behavior applied:	Sands only		
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	17.50 ft	Excavation depth:	12.00 ft	Limit depth applied:	No		
Points to test:	Based on Ic value	Average results interval:	1	Footing 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

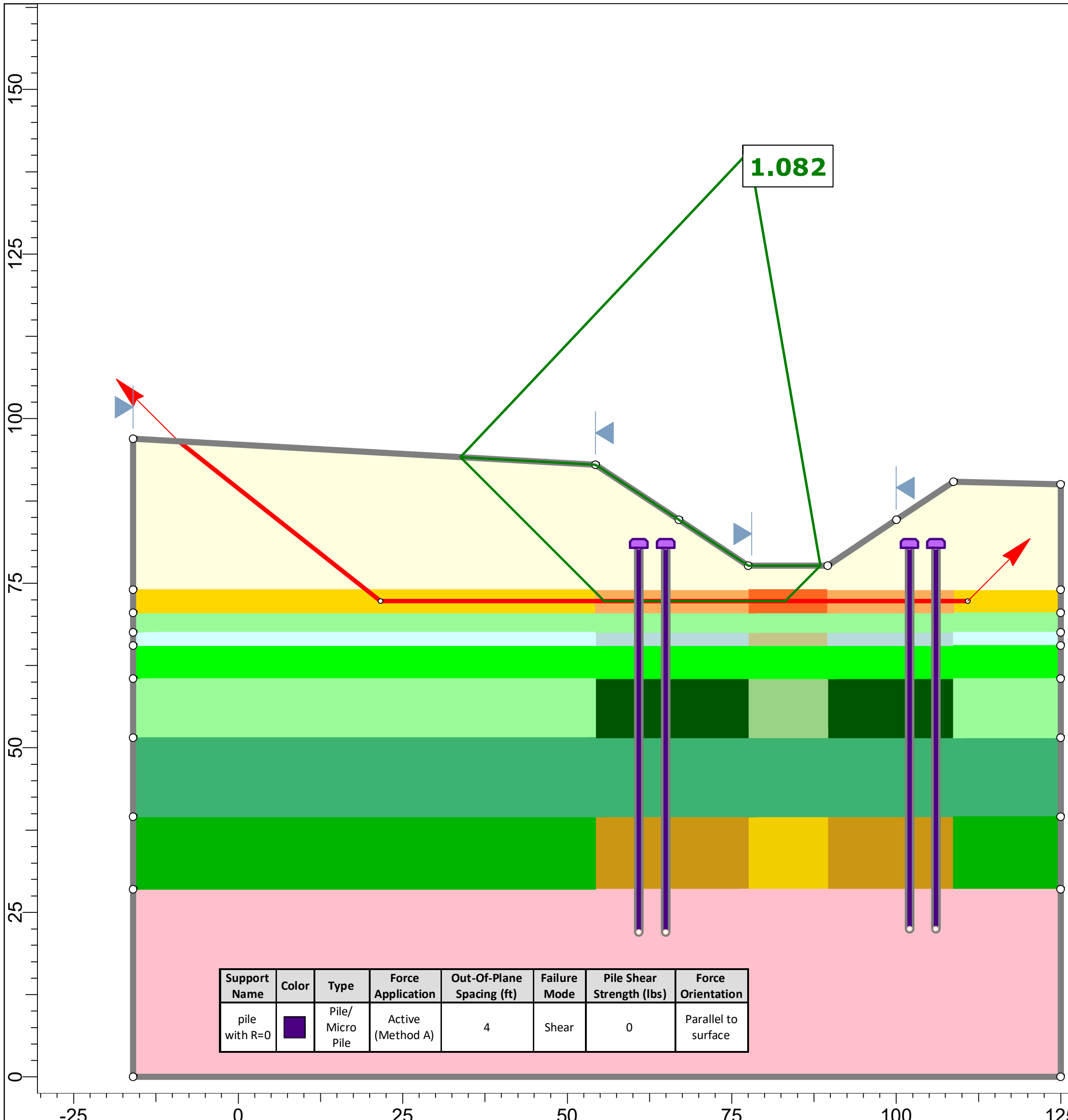
Project: FORTAG
Location: Del Rey Oaks, CA

CPT: CPT-02
Total depth: 66.52 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	14.60 ft	Excavation:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	14.50 ft	Excavation depth:	9.00 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:	I&B, 2008
Peak ground acceleration:	0.50	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Appendix G



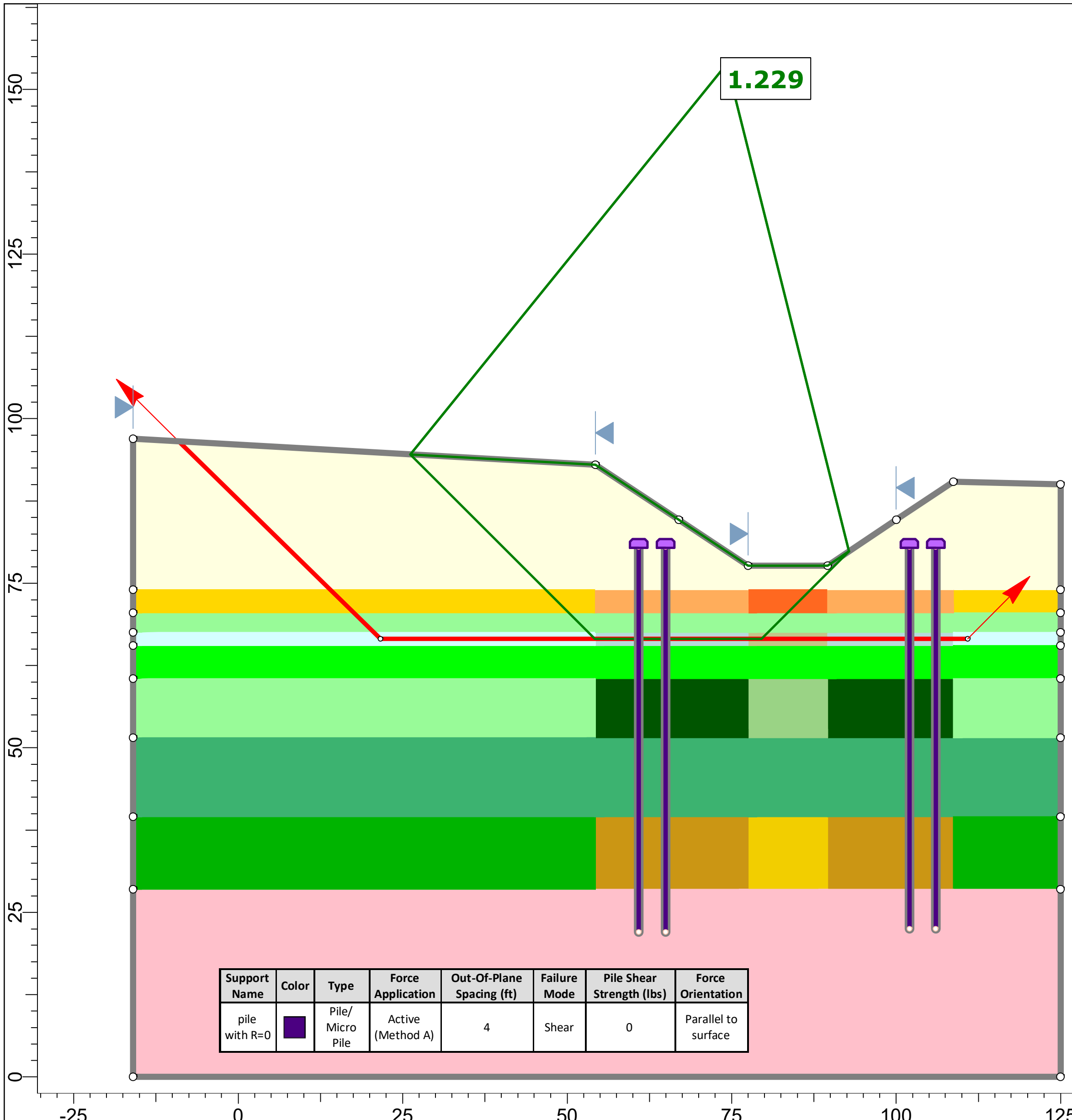
Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Failure Mode	Pile Shear Strength (lbs)	Force Orientation
pile with R=0		Pile/ Micro Pile	Active (Method A)	4	Shear	0	Parallel to surface

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	171	
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	251	
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	377	
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	693	
Layer 8 sand (L2)		115	Undrained	635	
Layer 8 sand (L3)		115	Undrained	444	
Layer 9 sand		125	Undrained	0	

McMILLEN JACOBS ASSOCIATES

SLIDEINTERPRET 9.025

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



Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Failure Mode	Pile Shear Strength (lbs)	Force Orientation
pile with R=0		Pile/ Micro Pile	Active (Method A)	4	Shear	0	Parallel to surface

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	171	
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	251	
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	377	
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	693	
Layer 8 sand (L2)		115	Undrained	635	
Layer 8 sand (L3)		115	Undrained	444	
Layer 9 sand		125	Undrained	0	

McMILLEN JACOBS ASSOCIATES

SLIDEINTERPRET 9.025

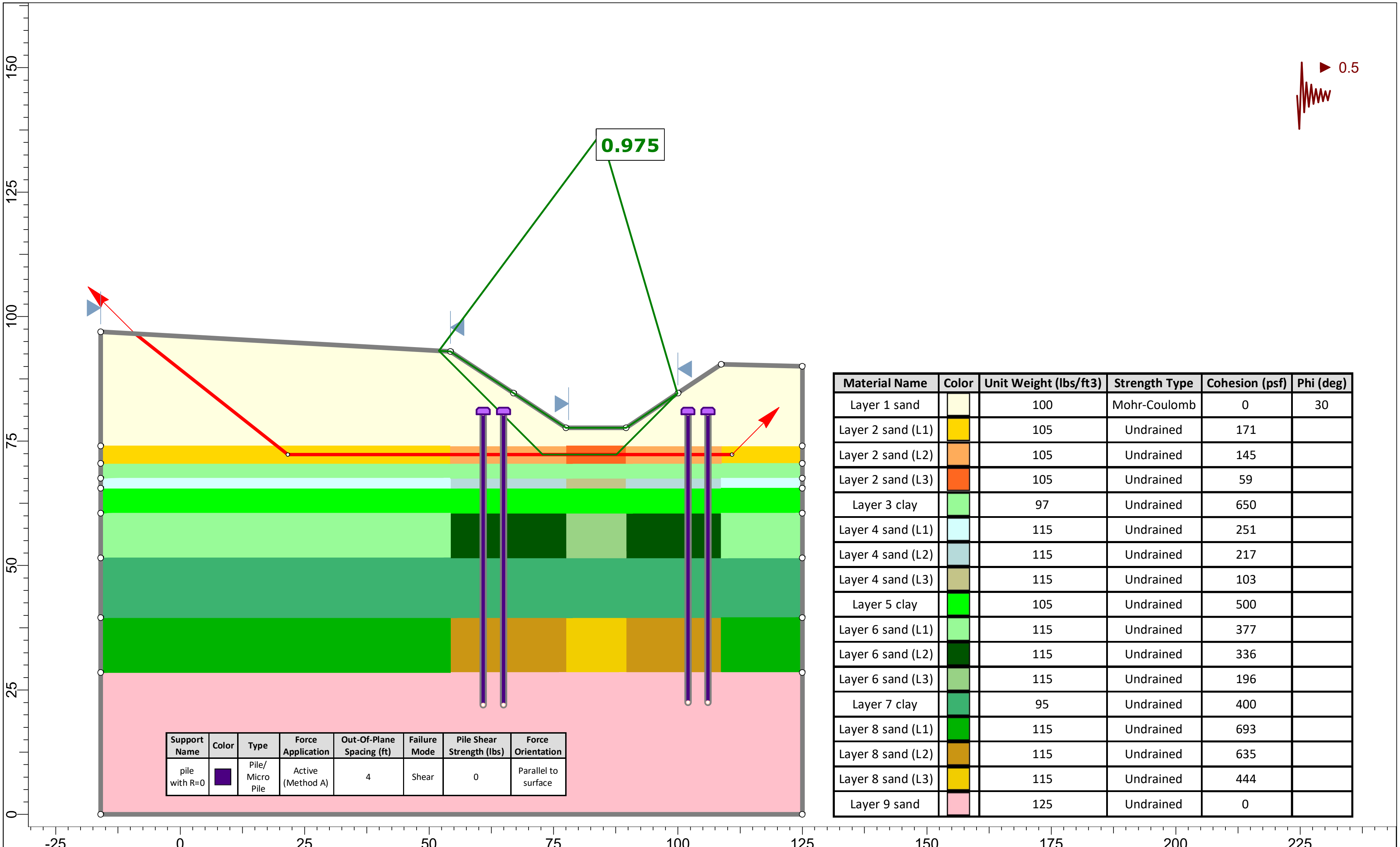
Project Name
TAMC - FORTAG Canyon Del Rey/SR218 Segement

Project No.
6231.0

Scenario
Liquefiable Layer at El 66.5, kh=0

Date
April 2023

Location
Del Rey Oaks, California



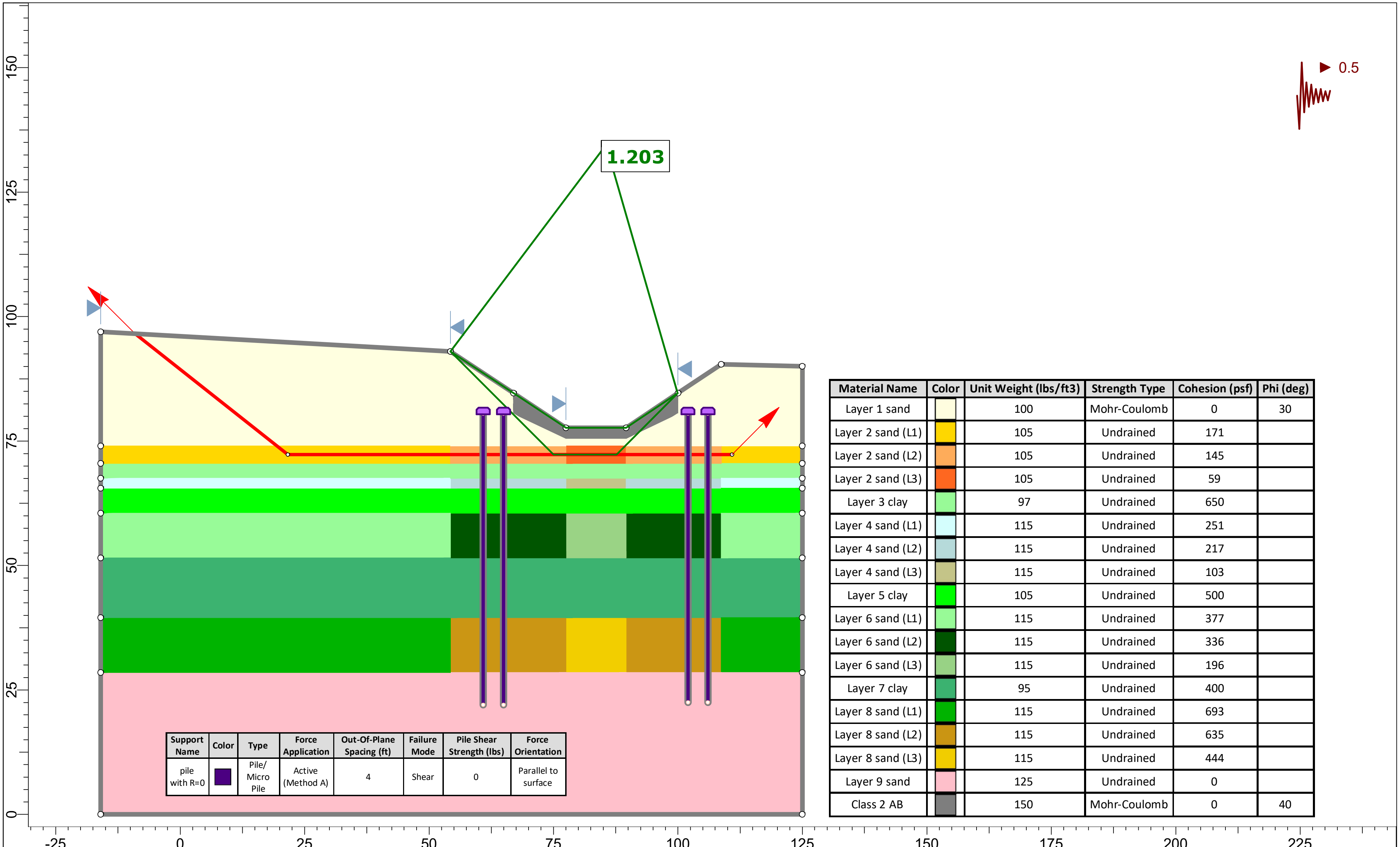
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Layer 1 sand		100	Mohr-Coulomb	0	30
Layer 2 sand (L1)		105	Undrained	171	
Layer 2 sand (L2)		105	Undrained	145	
Layer 2 sand (L3)		105	Undrained	59	
Layer 3 clay		97	Undrained	650	
Layer 4 sand (L1)		115	Undrained	251	
Layer 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	377	
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	693	
Layer 8 sand (L2)		115	Undrained	635	
Layer 8 sand (L3)		115	Undrained	444	
Layer 9 sand		125	Undrained	0	

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Failure Mode	Pile Shear Strength (lbs)	Force Orientation
pile with R=0		Pile/ Micro Pile	Active (Method A)	4	Shear	0	Parallel to surface

MCMILLEN JACOBS ASSOCIATES

SLIDEINTERPRET 9.025

Project Name TAMC - FORTAG Canyon Del Rey/SR218 Segement	
Project No. 6231.0	*Scenario* Liquefiable Layer at El 72.25, kh=0.5g
Date April 2023	*Location* Del Rey Oaks, California



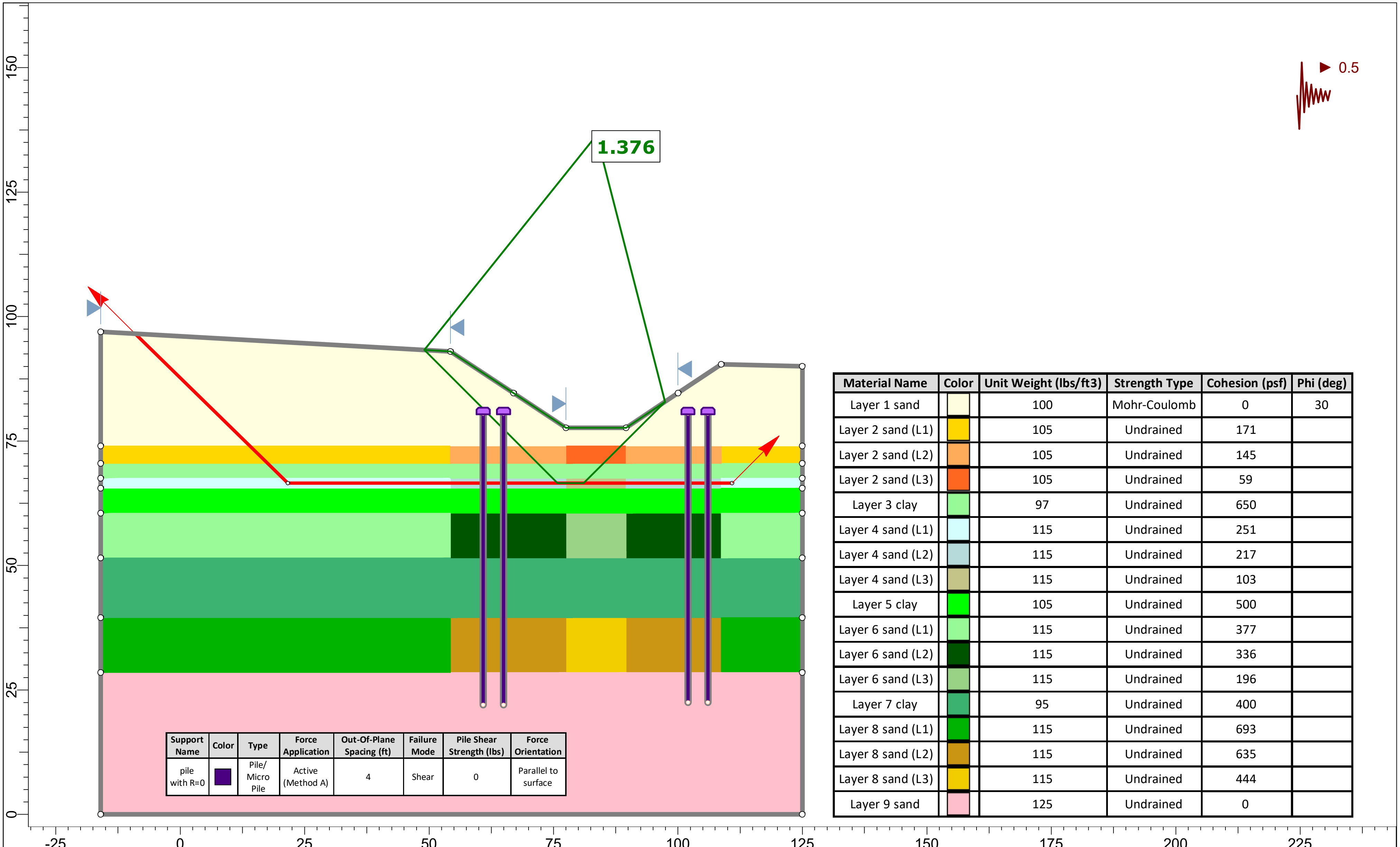
Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Failure Mode	Pile Shear Strength (lbs)	Force Orientation
pile with R=0		Pile/ Micro Pile	Active (Method A)	4	Shear	0	Parallel to surface

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	171	
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	251	
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	377	
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	693	
Layer 8 sand (L2)		115	Undrained	635	
Layer 8 sand (L3)		115	Undrained	444	
Layer 9 sand		125	Undrained	0	
Class 2 AB		150	Mohr-Coulomb	0	40

McMILLEN JACOBS ASSOCIATES

SLIDEINTERPRET 9.025

Project Name TAMC - FORTAG Canyon Del Rey/SR218 Segement	
Project No. 6231.0	*Scenario* Liquefiable Layer at El 72.25, kh=0.5g, Class 2AB backfill
Date April 2023	*Location* Del Rey Oaks, California



Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Failure Mode	Pile Shear Strength (lbs)	Force Orientation
pile with R=0		Pile/ Micro Pile	Active (Method A)	4	Shear	0	Parallel to surface

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	171	
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	251	
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	377	
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	693	
Layer 8 sand (L2)		115	Undrained	635	
Layer 8 sand (L3)		115	Undrained	444	
Layer 9 sand		125	Undrained	0	

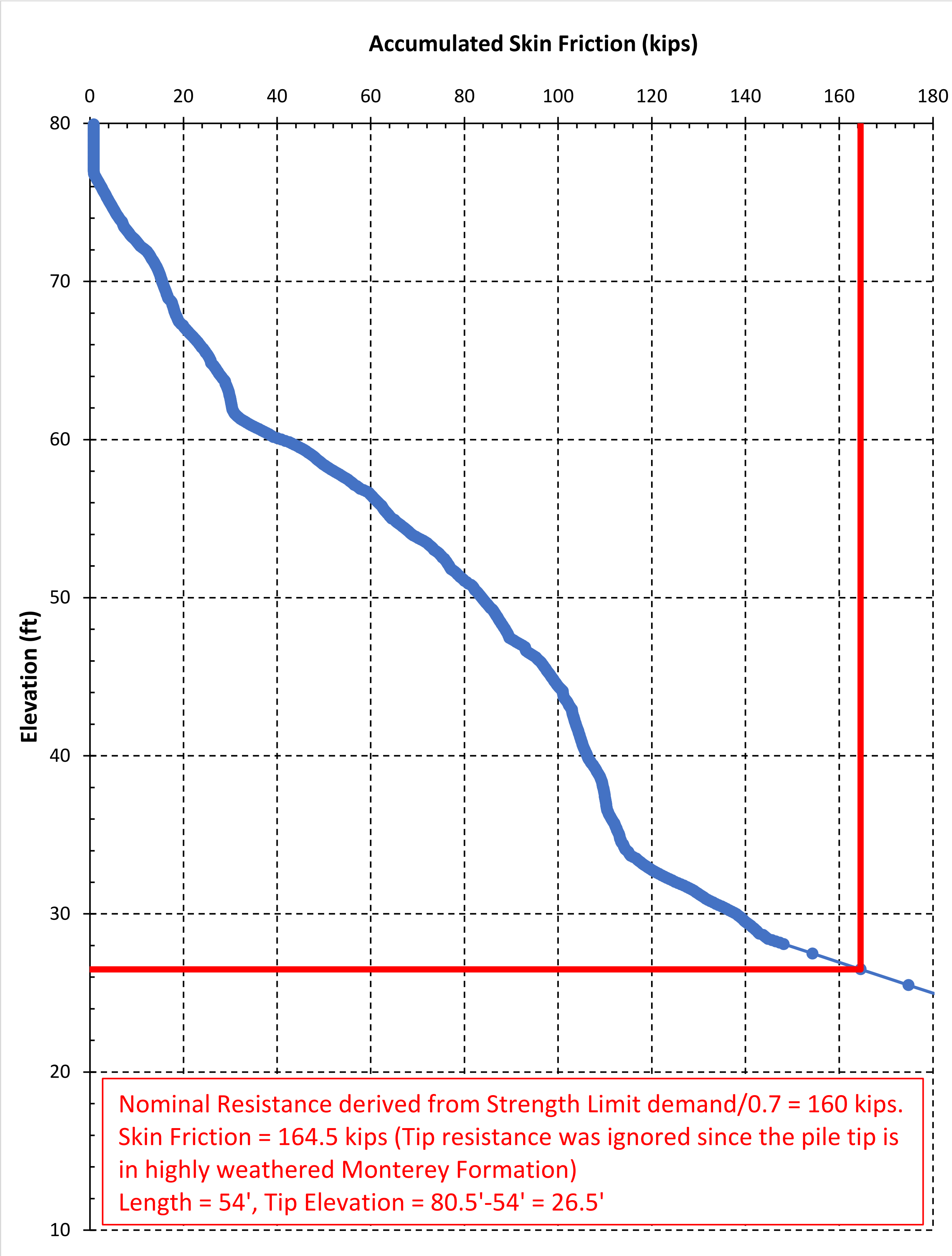
McMILLEN JACOBS ASSOCIATES

SLIDEINTERPRET 9.025

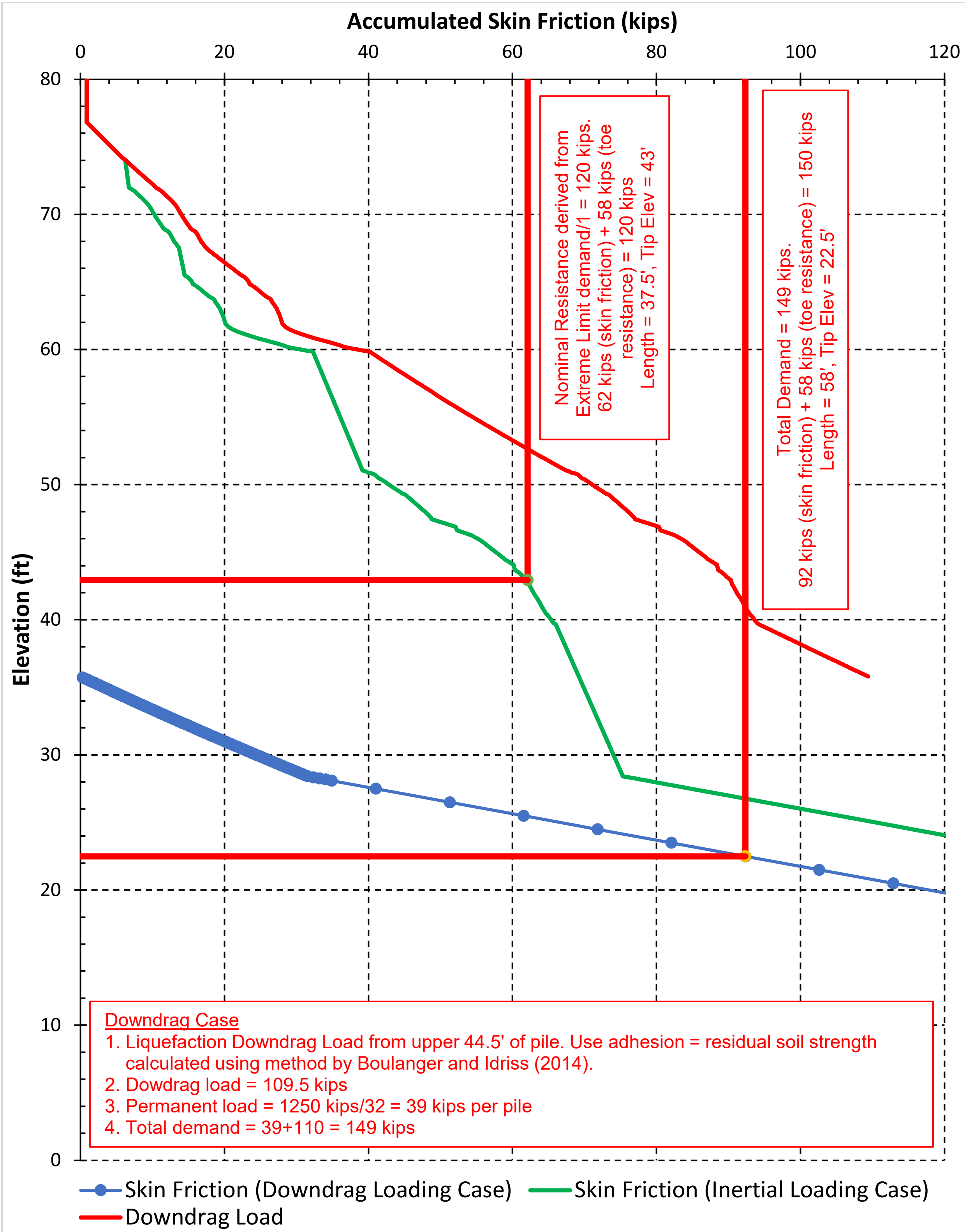
Project Name TAMC - FORTAG Canyon Del Rey/SR218 Segement	
Project No. 6231.0	*Scenario* Liquefiable Layer at El 66.5, kh=0.5g
Date April 2023	*Location* Del Rey Oaks, California

Appendix H

16-Inch Steel Pipe Piles (Caltrans Class 200 "Alt W"), Strength Limit



16-Inch Steel Pipe Piles (Caltrans Class 200 "Alt W"), Extreme Event Limit



Appendix I

Comment and Response Form

Review of Draft Foundation Report

McMillen Jacobs Associates, "Final Foundation Report for Bridge, Transportation Agency of Montery County, Fort Ord Regional Trail & Greenway - SR 218 Undercrossing Bridge, Del Rey Oaks, California" dated March 10, 2023

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

Response Date: March 28, 2023

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 this report has been stamped, but 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.	revised
Chris McMahon/59-3660	Section 3.7.5 Liquefaction-Induced Lateral Spreading: The results of the analysis should be tabulated in this section.	revised
Chris McMahon/59-3660	Section 4.0 Recommendations: Liquefaction is not discussed, despite the potential for nearly a half-foot of settlement?	added
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	added
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.

		Response Date: March 28, 2023
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 3.7.5 Liquefaction-Induced Lateral Spreading: A horizontal seismic coefficient of 0.5g is very high. Please reconsider.	Horizontal seismic coefficient of 0.5g was used based on Caltrans guideline for Liquefaction-Induced Lateral Spreading. The guideline for the earth retaining system says we can use 1/2 of the PGA (0.25g) assuming that we are allowing about 1 to 2 inches of deflection. I was not sure if we can use the same guideline on the abutment.
Justin Anderson/59-3660	Table 13 Foundation Design Recommendations: Please add an additional column for compression with downdrag. Confirm that the design tip elevation accounts for downdrag.	A table summerizing downdrag load is added. Refer to the second paragraph of Section 6.1.1 for the write up confirming that the design tip elevation accounts for downdrag load.
Justin Anderson/59-3660	Table 13 Foundation Design Recommendations: Please round required nominal resistance up to the nearest multiple of 10 i.e. 157.1 -> 160	revised
Justin Anderson/59-3660	Table 13 Foundation Design Recommendations: Specified Tip Elevation appears to be based on Compression (Extreme Event) but required nominal driving resistance notes it is based on the strength limit state. Please check.	revised
Justin Anderson/59-3660	Scour Data: Please add a scour section and state there is no scour if there isn't any for clarity	added