

Geotechnical Evaluation Via Princessa Park City of Santa Clarita Santa Clarita, California

Psomas

555 S. Flower Street, Suite 4300 | Los Angeles, California 90071

June 16, 2023 | Project No. 212109001



Geotechnical | Environmental | Construction Inspection & Testing | Forensic Engineering & Expert Witness

Geophysics | Engineering Geology | Laboratory Testing | Industrial Hygiene | Occupational Safety | Air Quality | GIS

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

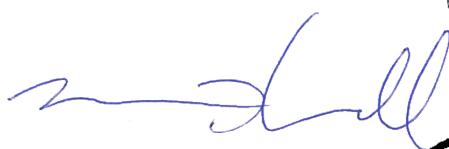
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Mr. Jeremy Johnson

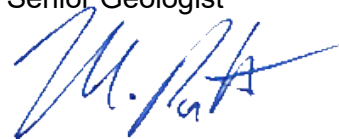
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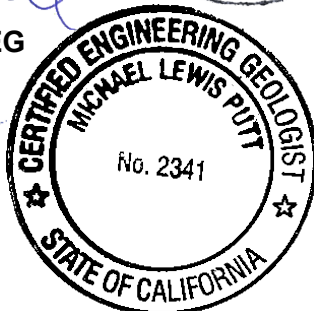
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1 INTRODUCTION

In accordance with your request, we have performed a geotechnical evaluation for the City of Santa Clarita Via Princessa Park Project located at 19201 Via Princessa in Santa Clarita, California (Figure 1). We understand that the project will develop an approximately 25-acre primarily undeveloped site into a public park. The project also includes a new stormwater infiltration facility that was evaluated by another consultant. The geotechnical report was provided for our review as a part of this evaluation (R.T. Frankian & Associates, 2022). The purpose of our geotechnical services was to evaluate the soil, geologic and groundwater conditions at the project site and to provide conclusions and recommendations regarding the geotechnical aspects of the planned park improvements. This report presents our findings, conclusions, and recommendations based on our background review, site reconnaissance, subsurface evaluation, laboratory testing, and geotechnical analyses.

2 SCOPE OF SERVICES

Our scope of services included the following:

- Project planning and coordination, including review of readily available geologic maps, aerial images, seismic data, groundwater data, project planning documents provided by Psomas and the City of Santa Clarita, and geotechnical literature.
- Attendance of Southern California Regional Rail Authority/Metrolink railway safety training by Ninyo & Moore representatives and our subcontractors.
- Acquisition of boring permits from the County of Los Angeles Department of Public Health.
- Acquisition of a Right-of-Entry permit from the Southern California Regional Rail Authority for working near an active railroad and crossing the railroad tracks to access the boring and cone penetrometer test (CPT) locations.
- Geotechnical reconnaissance to map and document the existing surficial conditions at the project site and to mark proposed boring and CPT locations for utility clearance with Underground Service Alert.
- Subsurface exploration consisting of the drilling, logging and sampling of 18 small-diameter borings to depths between approximately 21½ and 101½ feet below the ground surface to evaluate the subsurface soil and groundwater conditions within the planned project limits. Two borings were drilled to a depth of approximately 101½ feet to evaluate subsurface conditions at the pedestrian and vehicle maintenance tunnel location at the rail road right-of-way. Four borings were drilled to a depth of approximately 31½ feet to evaluate the areas where creek bank stabilization will be performed. One boring was drilled to a depth of approximately 61½ feet to evaluate the subsurface conditions for the Honby Creek vehicular access crossing. The remaining borings were drilled to depths between 21½ and 31½ feet to evaluate the subsurface conditions near the proposed amenity building foundations, lighting foundations, shade canopies, vehicular access roads, and other hardscape areas. The borings were logged by a representative from our firm, and bulk and relatively undisturbed soil samples were collected at selected intervals for laboratory testing.

- CPTs at 4 locations. The CPT operations were observed by a representative of our firm. Continuous soil profiles, including cone tip resistance, sleeve friction, and pore pressure, were recorded from the soundings. Shear wave velocity tests were performed at one location and the results were recorded at approximately 2 to 3-foot intervals.
- The borings and CPTs were backfilled with cement-bentonite grout in accordance with County Health Department requirements.
- Geotechnical laboratory testing of representative soil samples to evaluate in-situ moisture and dry density, gradation, percent of particles finer than No. 200 sieve, direct shear strength, expansion index, Proctor density, unconfined compressive strength of soil-cement material, R-value, sand equivalent, and soil corrosivity.
- Compilation and geotechnical analysis of field and laboratory data.
- Preparation of this geotechnical report presenting our findings, conclusions, and recommendations regarding the project.

3 SITE DESCRIPTION

The proposed Via Princessa Park site is located on an approximately 25-acre parcel of vacant land along the south bank of the Santa Clara River, east of Whites Canyon Road and north of the existing railroad tracks (Figure 1). The site is bordered to the east by residential properties associated with the Cordova Estates Mobile Home Park. The site is primarily undeveloped with the embankment and south bridge abutment for Whites Canyon Road occupying approximately 1 acre on the west end of the site. Honby Creek, which has an existing concrete culvert located to the north of the existing railroad tracks, crosses the middle of the site towards the Santa Clara River. The Metrolink Via Princessa train station is situated along the southern portion of the site with the railroad tracks bisecting the site. The Metrolink station includes station facilities, restrooms and a 395-space parking lot. Prior to the current site improvements for the train station and facilities, a review of historical aerial photographs from 1930 and 1940, show that the site was used predominantly for agricultural purposes.

4 PROJECT DESCRIPTION

Via Princessa Park will include four sports fields, lighting for the sports fields, sports fields seating, shade canopies, playgrounds, amenity building for restrooms and security, structural stormwater infiltration facilities, bank protection lining using soil-cement for the eastern and western side of Honby Creek and southern side of Santa Clara River, hardscape rated for vehicular use, pedestrian and maintenance access undercrossing beneath the railroad right-of-way, an at-grade service vehicle access across the railroad right-of-way, and a new pedestrian/maintenance vehicle crossing over Honby Creek that will include extending the existing culvert. The proposed undercrossing beneath the railroad right-of-way will consist of a vehicular access road

approximately 22 feet wide and a pedestrian sidewalk. The sidewalk will include a stairway and an Americans with Disabilities Act access ramp at the south side of the tracks. Relatively deep excavations on the order of 20 to 25 feet deep are anticipated for the railroad right-of-way undercrossing, creek bank stabilization, and structural infiltration facilities west of Honby Creek, while relatively shallow grading is anticipated to prepare the ground surface for the sports fields and ancillary improvements.

5 SUBSURFACE EVALUATION AND LABORATORY TESTING

Our subsurface evaluation was conducted on February 21 through February 23, March 1, and March 15, 2023, and consisted of the drilling, logging, and sampling of 18 small-diameter borings (B-1 through B-18) to depths ranging from approximately 21½ feet to 101½ feet, and four CPTs advanced to refusal depths ranging from approximately 11.4 to 35.4 feet below the ground surface. The exploratory borings were drilled using truck and track-mounted drilling equipment utilizing hollow stem augers. The purpose of the borings and CPTs were to evaluate the subsurface conditions at the project site and to collect bulk and relatively undisturbed soil samples for laboratory testing. Logs of the exploratory borings are presented in Appendix A and the CPT soundings are presented in Appendix B. The approximate locations of the borings and CPTs are presented on Figures 2 through 13.

As a part of our evaluation, a geotechnical investigation and infiltration study for the proposed structural infiltration facilities west of Honby Creek by R.T. Frankian and Associates dated March 30, 2022 was provided for our review. The prior study included four exploratory borings to depths ranging from approximately 21 feet to 51½ feet and 12 CPTs to depths ranging from approximately 12 to 44 feet. The locations of the previous borings are also shown on Figures 2 through 13 and copies of the boring logs are included in Appendix C.

Laboratory testing was performed to evaluate moisture content, in-situ moisture and dry density, gradation, percent finer than the No. 200 sieve, expansion index, direct shear strength, sand equivalent, unconfined compressive strength of soil-cement material, R-value and soil corrosivity (soil pH, electrical resistivity, water-soluble sulfate content, and chloride content). The results of the in-situ moisture content and dry density tests are presented on the boring logs in Appendix A. The remaining laboratory testing results are presented in Appendix D.

6 GEOLOGY

The project site is situated in the northern portion of the Ventura Basin, within the Transverse Ranges Geomorphic Province, which encompasses an approximately 40- to 60-mile-wide area

(north to south) that extends approximately 320 miles (west to east) from Point Arguello and San Miguel Island to the Eagle and Pinto Mountains of the Mojave Desert (Norris and Webb, 1990). The province generally consists of a region of east to west-trending mountain ranges considered atypical to the predominant northwest to southeast structural fabric of California. The atypical trend of the Province is the result of a restraining bend (“the Big Bend”) on the San Andreas fault that has rotated and compressed the region to its current configuration. The compression has resulted in regional uplift, and folding and reverse/thrust faulting with similar east to west trends as the topography. This portion of the Transverse Ranges province consists of thick sequences marine and non-marine sedimentary rock deposited in trans tensional basins associated with movement along the San Andreas and San Gabriel faults (Norris and Webb, 1990). The site lies along the southern side of the Santa Clara River Valley.

According to the geologic map by Dibblee (1996), the park area and surrounding vicinity are mapped as being underlain by Holocene-age young alluvium. A map of the regional geology at the site is presented in Figure 14.

6.1 Subsurface Conditions

Earth materials encountered during our subsurface exploration at the site include fill soils and Holocene to late Pleistocene-age alluvial soils that were encountered in the borings to the depths explored of up to approximately 101½ feet. Generalized descriptions of the materials encountered during our subsurface exploration are presented below. Detailed descriptions of the subsurface conditions are presented on the boring logs in Appendix A.

6.1.1 Pavement

Pavement was encountered in borings B-1 and B-18, and generally consisted of asphalt concrete (AC) over aggregate base (AB). The thickness of the AC over AB ranged from approximately 6 inches over approximately 6 to 7 inches. The aggregate base material generally consisted of a medium dense, silty gravel with sand.

6.1.2 Fill

Fill soils were encountered in borings B-1, B-6, B-12, B-14, and B-16 through B-18 to depths ranging from approximately 3 to 7 feet below the ground surface. The fill encountered at the boring locations generally consisted of gray to dark gray, grayish brown and olive brown, moist, medium dense, silty sand and clayey sand with gravel and gravel-sized asphalt concrete and Portland cement concrete fragments, and stiff, sandy lean clay.

6.1.3 Alluvial Deposits

Alluvial deposits were generally encountered beneath the fill or from the surface to the full depths explored of up to approximately 101½ feet and generally consisted of moist to wet, loose to very dense, silty sand, clayey sand, poorly graded sand with silt, poorly graded sand, and sandy silt with varying amounts of gravel, and stiff to hard, sandy lean clay. Trace amounts of scattered cobbles were encountered in the alluvium. While boulders were not encountered in our borings, considering the depositional environment and proximity to the Santa Clara River, boulders should be anticipated within the alluvial deposits. The alluvial soils were typically loose near the surface and became medium dense to very dense with depth. Difficult drilling conditions were observed where cobbles were encountered in the borings.

6.2 Groundwater

Groundwater was encountered in our exploratory borings during drilling in boring B-12 at a depth of approximately 60 feet below the ground surface, and in borings B-16 and B-18 at a depth of approximately 70 feet below the ground surface. The groundwater level was measured in boring B-12 at approximately 55 feet below the ground surface approximately 5 minutes after drilling, in boring B-16 the groundwater level was measured at approximately 65½ feet below the ground surface approximately 30 minutes after drilling, and in boring B-18 the groundwater level was measured at approximately 68 feet below the ground surface approximately 30 minutes after drilling. Historical high groundwater levels in the site vicinity is reported at approximately 15 feet below the ground surface (California Geological Survey [CGS], 1998). In addition, groundwater monitoring well data from the County of Los Angeles Department of Public Works indicates that the depth to groundwater has been recorded as being as shallow as 10 to 13 feet below the ground surface at three wells located at the site (LADPW, 2023), including recent groundwater depth measurements following recent heavy rains. It should be noted that fluctuations in the level of groundwater will occur due to variations in ground surface topography, subsurface stratification, rainfall, irrigation practices, groundwater pumping, and other factors which may not have been evident at the time of our field evaluation.

6.3 Flood Hazards

The project site is located adjacent to the south side of the Santa Clara River, which is considered by the Federal Emergency Management Agency (FEMA) as an active regulatory floodway. In addition, the active Honby Creek channel transects the western portion of the site and flows north to the Santa Clara river. Based on this information and review of the FEMA flood insurance rate map (2021), the majority of the site is mapped within the 0.2% annual chance flood hazard with

areas of 1% annual flood with an average depth of one foot or with drainage areas of less than one square mile, and areas of the site in close proximity to the Santa Clara River and Honby Creek are mapped as being within Zone AE, which is a 1% chance annual flood zone. Based on this review, the potential for flooding of the site is possible and should be evaluated by the project civil engineer.

7 FAULTING AND SEISMICITY

Based on our review of pertinent readily available geologic literature, geologic maps, stereoscopic aerial photographs, and our geologic field reconnaissance, no active faults are known to cross the subject site and the site is not located within a State of California Earthquake Fault Zone (EFZ) formerly known as an Alquist-Priolo Special Studies Zone (Hart and Bryant, 2018). However, the site is located in a seismically active area, as is the majority of southern California, and the potential for strong ground motion in the project area is considered significant during the design life of the proposed project. Figure 15 shows the approximate site location relative to the major faults in the region.

In general, seismic hazards that could impact the project include ground surface rupture, strong ground motion, liquefaction, liquefaction-induced dynamic settlement and seismically induced landsliding. These potential hazards are discussed in the following sections.

7.1 Surface Fault Rupture

Based on our review of the referenced literature and our site reconnaissance, no active faults are known to cross the project site. Therefore, the probability of damage from surface ground rupture is considered to be low. However, lurching or cracking of the ground surface as a result of nearby seismic events is possible.

7.2 Strong Ground Motion

7.2.1 Site-specific Ground Motion

Considering the proximity of the site to active faults capable of producing a $M_{\max}$ of 6.0 or more, the project area has a high potential for experiencing strong ground motion. The 2022 CBC specifies that the risk-targeted maximum considered earthquake (MCE_R) ground motion response accelerations be used to evaluate seismic loads for design of buildings and other structures.

In accordance with Chapter 20 of the American Society of Civil Engineers (ASCE) Publication 7-16 (2016) for the Minimum Design Loads and Associated Criteria for Building and Other

Structures, the Site Class should be classified based on the average shear wave velocity (V_s) in the upper 30 meters (100 feet) (V_{s30}), the average field standard penetration resistance in the upper 30 meters (100 feet) (blow counts, N), or the average undrained shear strength in the upper 30 meters (100 feet) (S_u). After reviewing the blow counts obtained from our borings B-16 and B-18, the average field penetration blow count in the upper 30 meters (100 feet) is between 29 to 35 blows per foot. We evaluated the average shear wave velocity in the upper 30 meters (100 feet) of the subsurface profile (V_{s30}) by analyzing the measured standard penetration test (SPT) blow counts from borings B-16 and B-18 based on empirical correlations (Brandenberg et al., 2010) and reviewing the CGS's shear wave velocity map (CGS, 2015). Based on our evaluation, The V_{s30} is estimated to be 280 m/s (918 ft/s). ASCE 7-16 defines a site as Site Class D (stiff soil) if the V_{s30} ranges from 180 to 370 m/s (600 to 1,200 ft/s) and if the blow counts in the upper 30 meters (100 feet) ranges from 15 to 50 blows per foot. Accordingly, the site is classified as Site Class D.

Per the 2022 CBC, a site-specific ground motion hazard analysis shall be performed for structures on Site Class D with a mapped MCE_R , 5 percent damped, spectral response acceleration parameter at a period of 1 second (S_1) greater than or equal to 0.2g in accordance with Sections 21.2 and 21.3 of ASCE 7-16. We calculated that the S_1 for the site is equal to 0.822g using the 2023 Applied Technology Council (ATC) seismic design tool (web-based); therefore, a site-specific ground motion hazard analysis was performed for the project area.

The site-specific ground motion hazard analysis consisted of the review of available seismologic information for nearby faults and performance of probabilistic seismic hazard analysis (PSHA) and deterministic seismic hazard analysis (DSHA) to develop acceleration response spectrum (ARS) curves corresponding to the MCE_R for 5 percent damping. Prior to the site-specific ground motion hazard analysis, we obtained the mapped seismic ground motion values and developed the mapped MCE_R response spectrum for 5 percent damping in accordance with Section 11.4 of ASCE 7-16 using the 2023 ATC seismic design tool. The depths to $V_s = 1,000$ m/s and $V_s = 2,500$ m/s are assumed to be 350 meters and 1,950 meters, respectively. These values were evaluated using the Open Seismic Hazard Analysis (OpenSHA) software developed by United States Geological Survey (USGS, 2021).

The 2014 new generation attenuation (NGA) West-2 relationships were used to evaluate the site-specific ground motions. The NGA relationships that we used for developing the probabilistic and deterministic response spectra are by Chiou and Youngs (2014), Campbell and Bozorgnia (2014), Boore, Stewart, Seyhan, and Atkinson (2014), and Abrahamson, Silva,

and Kamai (2014). The OpenSHA software (USGS, 2021) was used for performing the PSHA. The Calculation of Weighted Average 2014 NGA Models spreadsheet by the Pacific Earthquake Engineering Research Center was used for performing the DSHA (Seyhan, 2014).

PSHA was performed for earthquake hazards having a 2 percent chance of being exceeded in 50 years multiplied by the risk coefficients per Section 21.2.1.1 of ASCE 7-16. The maximum rotated components of ground motions were considered in PSHA with 5 percent damping. For the DSHA, we analyzed accelerations from characteristic earthquakes on active faults within the region using the hazard curves and deaggregation plots at the site obtained from the USGS Unified Hazard Tool application (USGS, 2023). A magnitude 7.5 event on the San Gabriel fault with a rupture distance of 2.3 kilometers (1.4 miles) and a magnitude 7.0 event on the Santa Susana fault with a rupture distance of 7.2 kilometers (4.5 miles) from the site were evaluated to be the controlling earthquakes. The Santa Susana fault was evaluated to be the controlling earthquake at periods from 0 to 0.5 seconds, and the San Gabriel fault was evaluated to be the controlling earthquake at periods from 0.75 to 10 seconds. Hence, the DSHA was performed for the site using these events and corrections were made to the spectral accelerations for the 84th percentile of the maximum rotated component of ground motion with 5 percent damping.

The site-specific MCE_R response spectrum was taken as the lesser of the spectral response acceleration at any period from the PSHA and DSHA, and the site-specific general response spectrum was determined by taking two-thirds of the MCE_R response spectrum with some conditions in accordance with Section 21.3 of ASCE 7-16. Figure 16 presents the site-specific MCE_R response spectrum and the site-specific design response spectrum. The general mapped design response spectrum calculated in accordance with Section 11.4 of ASCE 7-16 is also presented on Figure 16 for comparison. The site-specific spectral response acceleration parameters, consistent with the 2022 CBC, are provided in Section 9.2 of this report for the evaluation of seismic loads on buildings and other structures.

ASCE 7-16 specifies that the potential for liquefaction and soil strength loss be evaluated, where applicable, for the maximum considered earthquake geometric mean (MCE_G) peak ground acceleration adjusted for site effects (PGA_M). The PGA_M is based on the geometric mean peak ground acceleration with a 2 percent probability of exceedance in 50 years. The site-specific PGA_M was calculated as 0.904g.

7.2.2 Caltrans Ground Motion Study

Based on our discussions with the design teams, it is our understanding that the proposed railroad undercrossing would be designed in accordance Caltrans Seismic Design Criteria (SDC) (Caltrans, 2019). Based on the Caltrans Acceleration Response Spectra (ARS) online web tool (Caltrans, 2023) and using V_{s30} of 280 m/s for the site, a mean magnitude of 6.8 and a PGA of 0.77g is appropriate for the proposed undercrossing site. The design Caltrans ARS curve evaluated for the site is presented on Figure 17.

7.2.3 AREMA Ground Motion Study

American Railway Engineering and Maintenance-of-Way Association (AREMA, 2022) recommends a three-level ground motion and performance criteria for structures associated with operation of railway traffic with the objective of reducing costs of damage and loss of use caused by potential earthquakes. Ground motion levels correspond to the desired performance criteria of structures as follows: a Level 1 earthquake with an occasional frequency of occurrence corresponds to a “serviceability” criterion, a Level 2 earthquake with a rare frequency of occurrence corresponds to an “ultimate” criterion, and a Level 3 earthquake with a very rare frequency of occurrence corresponds to a “survivability” criterion of performance. The earthquake frequency is dictated by the anticipated return period of a seismic event. AREMA (2022) defines an occasional frequency as an earthquake having an average return period between 50 and 100 years (Level 1). Similarly, the rare and very rare frequencies are defined as earthquakes having average return periods ranging from 200 to 475 years (Level 2) and 1,000 to 2,475 years (Level 3), respectively. Based on our correspondence with the client and RailPros structural engineer, the design return periods for Level 1, Level 2, and Level 3 earthquake events were considered to be 100, 475 and 2,475 years, respectively, for the proposed bridge.

As discussed in the previous sections, site class D with an average shear wave velocity of 280 m/s (918 ft/s) was considered in our analysis. In evaluating the peak ground acceleration (PGA) values and the ARS curves using OpenSHA, we used the NGA relations proposed by Chiou and Youngs (2014), Campbell and Bozorgnia (2014), Boore, Stewart, Seyhan, and Atkinson (2014), and Abrahamson, Silva, and Kamai (2014). The peak ground acceleration (PGA) values calculated for the site are presented in the following Table 1 and the acceleration response spectra are presented in Figure 18.

Table 1 – AREMA Site Class and Peak Ground Acceleration

Site Class	Level 1	Level 2	Level 3
D	0.33g	0.62g	0.99g

7.3 Liquefaction

Liquefaction is the phenomenon in which loosely deposited granular soils and non-plastic fine-grained soils located below the water table undergo rapid loss of shear strength due to excess pore pressure generation when subjected to strong earthquake-induced ground shaking. Sufficient ground shaking duration results in the loss of grain-to-grain contact due to a rapid rise in pore water pressure. This causes the soil to behave as a fluid for a short period of time. Liquefaction is known generally to occur in saturated or near-saturated cohesionless soils at depths shallower than 50 feet below the ground surface. Factors known to influence liquefaction potential include composition and thickness of soil layers, grain size, relative density, groundwater level, degree of saturation, and both intensity and duration of ground shaking.

The historic high depth to groundwater is mapped in the vicinity of the site as approximately 15 feet below the existing ground surface (CGS, 1998). Our review of readily available groundwater data in the vicinity of the site indicates that for a site located adjacent to the southeast side of the subject site, the depth to groundwater has been recorded as being as shallow as 10 feet (LADWP, 2023). The State of California Seismic Hazard Zones Map (Figure 19) indicates the project area is located within an area mapped as subject to seismically induced liquefaction hazards. Based on the unconsolidated nature of the underlying soil materials and the historically shallow groundwater levels in the site vicinity, the site has a potential for dynamic settlement due to liquefaction.

The liquefaction analysis was based on the National Center for Earthquake Engineering Research procedure (Youd, et al., 2001) using the computer program LiquefyPro (CivilTech, 2008). Accordingly, a design earthquake moment magnitude of 6.8 and an associated ground acceleration of 0.77g was used in the liquefaction analysis in accordance with Caltrans SDC. Our liquefaction analysis indicates that the interbedded, medium dense to very dense, granular soil layers below the historic high groundwater level at depth of approximately 22 to 65 feet below the ground surface are susceptible to liquefaction during the design seismic event.

7.4 Dynamic Settlement of Dry Soils

Relatively dry soils (e.g., soils above the groundwater table) with low density or softer consistency tend to undergo dynamic settlement during a seismic event. Earthquake shaking often induces significant cyclic shear strain in a soil mass, which responds to the vibration by undergoing volumetric changes. Volumetric changes in dry soils take place primarily through changes in the void ratio (usually contraction in loose or normally consolidated, soft soils and dilation in dense or over consolidated, stiff soils) and secondarily through particle reorientation. Such volumetric changes are generally non-recoverable. Our analysis, which was based on the method proposed

by Tokimatsu and Seed (1987), indicated that up to approximately ¼ inch dynamic settlement of dry sand may occur during the design seismic event.

7.5 Liquefaction-Induced Dynamic Settlement of Saturated Soils

In order to estimate the amount of dynamic settlement of saturated soils, the method proposed by Tokimatsu and Seed (1987) was used. Our analysis indicated that approximately ½ inch of dynamic settlement may occur for the saturated sand at a depth of approximately 22 to 50 feet bgs during the design seismic event. Based on the guidelines presented in CGS Special Publication 117 (2008) and assuming relatively consistent subsurface stratigraphy across the site, we estimate differential settlement on the order ¼ inch over a lateral distance of 40 feet. Results of our analysis are presented in Appendix E.

7.6 Lateral Spread

Lateral spread of the ground surface during an earthquake usually takes place along weak shear zones that have formed within a liquefiable soil layer. Lateral spread has generally been observed to take place in the direction of a free-face (i.e., retaining wall, slope, creek) but has also been observed to a lesser extent on ground surfaces with very gentle slopes. An empirical model developed by Bartlett and Youd (1995), is typically used to predict the amount of horizontal ground displacement within the site. For sites located in proximity to a free face, the amount of lateral ground displacement is strongly correlated with the distance of the site from the free-face. Other factors such as earthquake magnitude, distance from the earthquake epicenter, thickness of the liquefiable layers, and the fines content and particle sizes of the liquefiable layers also affect the amount of lateral ground displacement. Due to the lack of soil layers with a corrected blowcount, $(N1)_{60}$, less than 15, seismically induced lateral spread is not a design consideration for this project.

7.7 Liquefaction-Induced Loss of Bearing Capacity

For the potential loss of bearing capacity during the design earthquake, we evaluated the residual shear strength of the liquefiable layers at a depth of 22 to 50 feet below the existing ground surface using the method presented in Earthquake Engineering Research Institute Monograph - Soil Liquefaction During Earthquakes (Idriss and Boulanger, 2008). Based on our review of the boring logs B-16 and B-18, the subsurface soils underlying the proposed improvement are relatively uniform and consist of interbedded brown, and gray, moist to wet, medium dense to dense, silt, silty sand, clayey sand and sand and stiff to hard clay. Based on a corrected Standard Penetration Test blow count $(N1)_{60}$ ranging from 25 to 28 and a fines content of 10 for the liquefiable layer at a depth of 22 to 50 feet, our analysis indicated that the residual shear strength of this liquefiable

soil layer underlying the railroad undercrossing site is approximately 500 pounds per square foot (psf). The residual shear strength was used in the pile analysis but not for the shallow foundation analysis because the liquefiable soil layer is below the influence of shallow foundation loading.

7.8 Seismic-Induced Landsliding

Landslides may be induced by strong vibratory motion produced by earthquakes. Research and historical data indicate that seismically induced landslides tend to occur in weak soil and rock on sloping terrain. The process for zoning earthquake-induced landslides incorporates expected future earthquake shaking, existing landslide features, slope gradient and strength of earth materials on the slope. The project area is not mapped in an area considered susceptible to seismically induced landslides (CGS, 1998).

Based on our review of background geologic information, the park area and surrounding vicinity are mapped as being underlain by Holocene-age young alluvium. Based on our review of historical aerial images (Google Earth, 2023; Historical Aerials, 2023), and on our site reconnaissance, we did not observe evidence of significant slope failures or debris flows on the slopes adjacent to the park area and the slopes are well-vegetated. The project civil engineer should design appropriate drainage improvements to accommodate surface runoff from the hillsides entering and exiting the site and routine maintenance should be anticipated to clear drainage improvements of eroded debris.

8 CONCLUSIONS

Based on our subsurface evaluation, review of background information, and our experience in the area, it is our opinion that the proposed project is feasible from a geotechnical standpoint, provided that the following preliminary recommendations are incorporated into the design and construction of the proposed project.

The primary geotechnical considerations for the project include developing appropriate geotechnical foundation design recommendations for the vehicular and pedestrian undercrossing underneath the existing railroad tracks, new restroom building, shade canopies, and field lighting, and providing appropriate structural pavement recommendations for new parking areas and other vehicular roads.

An additional geotechnical consideration for the project is the depth to groundwater that could impact earthwork during construction. Due to the project location being adjacent to the Santa Clara River, there are wide seasonal fluctuations in the depths to groundwater that are highly dependent on seasonal rainfall. Although groundwater was more than 50 feet deep when

measured in our borings shortly after drilling, historical and relatively recent groundwater measurements in LADPW wells have been as shallow 10 feet deep. Therefore, depending on when site grading and construction of the railroad undercrossing are performed, groundwater may be encountered and dewatering may be needed. Scheduling of the deeper grading for the soil-cement bank protection as well as the railroad undercrossing in the later summer months may reduce or eliminate the need for construction dewatering. Consideration should be given to installing groundwater monitoring wells in the areas of deeper excavation to monitor the depths to groundwater prior to these construction activities.

In general, the following additional conclusions were made:

- The site is generally underlain by pavement, fill soils and alluvium generally consisting of medium dense to very dense, silty sand, clayey sand, poorly graded sand with silt, poorly graded sand, and sandy silt with varying amounts of gravel and stiff to hard sandy clay. Some of the alluvial soils contained cobbles that resulted in difficult drilling conditions.
- Based on our review of the State of California Seismic Hazard Zones map (CGS, 1998), the site is located in a State of California Liquefaction Hazard Zone. Our liquefaction analysis indicated that liquefaction-induced dynamic settlement of approximately one inch may occur during the design seismic event.
- Groundwater was measured in our exploratory borings after drilling at depths ranging from approximately 55 to 68 feet below the ground surface in borings B-12, B-16 and B-18. Historical high groundwater levels in the site vicinity is reported at approximately 15 feet below the ground surface. Groundwater monitoring well data from the County of Los Angeles Department of Public Works indicates that the depth to groundwater has been recorded as being as shallow as 10 to 13 feet below the ground surface at three wells located at the site. Groundwater levels are subject to variation depending on rainfall, irrigation, groundwater pumping and other factors.
- Excavations during site grading should be feasible with earthmoving equipment in good working order. Difficult excavating conditions may be encountered where cobbles are present. We anticipate that the existing fill and alluvial materials may be reused as compacted fill, however, some oversize cobbles may need to be screened from fill and trench backfill.
- The subject site is not located within a State of California EFZ. The potential for surface fault rupture as defined by the Alquist-Priolo Earthquake Fault Zoning Act is considered to be relatively low.
- On-site soils should be considered as Type C soils in accordance with Occupational Safety and Health Administration (OSHA) soil classifications. Sandy soil may be prone to caving during earthwork operations.
- The PGAM was estimated to be 0.904 g for the site.
- Based on laboratory test results, the near-surface site soils can be classified as non-corrosive based on California Department of Transportation (Caltrans, 2021) corrosion guidelines.
- A setback of 15 feet or more should be maintained between the invert of proposed infiltration systems and a 1:1 (horizontal to vertical) plane drawn down from the bottom outside edges of adjacent structural foundations and/or hardscape areas.

- The expansion index of the near-surface soil tested for this project ranged from low to very low.

9 RECOMMENDATIONS

The recommendations presented in the following sections provide general geotechnical criteria regarding the design and construction of the proposed improvements. The recommendations are based on the results of our subsurface evaluation, laboratory testing, review of referenced geologic materials, experience in the general vicinity of the project area, and geotechnical analyses. The proposed work should be performed in conformance with the recommendations presented in this report, project specifications, and appropriate agency standards.

9.1 Earthwork

9.1.1 Construction Plan Review and Pre-Construction Conference

We recommend that the grading and foundation plans be submitted to Ninyo & Moore for review to evaluate conformance to the geotechnical recommendations provided in this report. We further recommend that a pre-construction conference be held in order to discuss the recommendations presented in this report. The owner and/or their representative, the governing agencies' representatives, the civil engineer, Ninyo & Moore, and the contractor should be in attendance to discuss the work plan, project schedule, and earthwork requirements.

9.1.2 Site Clearing and Preparation

Prior to performing excavations or other earthwork, the site should be cleared of debris resulting from the demolition process, including existing site improvements, gravel, debris, vegetation, and loose or otherwise unsuitable materials. Existing utilities to remain in place (if any) should be located and protected from damage by construction activities. Materials generated from the clearing/demolition operations should be removed from the project site and disposed of at a legal dump site.

9.1.3 Excavation Characteristics

Based on our field exploration, we anticipate that excavations at the site may be accomplished with conventional earthmoving equipment in good working condition. The majority of excavations at the site will encounter alluvial soils, and in some areas undocumented fill. We anticipate that existing fill and alluvium encountered during construction will be generally comprised of silty sand, clayey sand, poorly graded sand, poorly graded sand with silt, sandy silt, and sandy clay with varying amounts gravel and occasional

cobbles. In the event that oversize material, including cobbles and boulders, are encountered during excavation operations, the oversized material (larger than 4 inches) is not suitable for use as fill in shallow fill areas or within trench backfill. However, some oversize material may be placed in selected areas where deeper excavations are planned in the creek stabilization area. Recommendations for placement of oversize rock are provided in Section 9.1.7.1. Contractors should make their own independent evaluation of the excavatability of the on-site materials prior to submitting their bids.

9.1.4 Treatment of Near Surface Soils

In order to provide suitable support and reduce the potential for settlement, we recommend that remedial grading is performed in the areas of new construction. As described above, there is existing fill at the site. The remedial grading should remove any existing undocumented fill and the bottom of the excavation should expose relatively dense native deposits. Remedial grading should be performed in all areas where future settlement is a concern, including building areas, areas to receive new fill, the parking lot and other paved areas, and the sports fields. Our borings encountered fill up to 7 feet in thickness, however, deeper fill may be present in areas that were not explored.

For new building structures, such as the proposed restroom building, retaining walls, and the new extended culvert, remedial grading should be performed to a depth of 5 feet below the existing ground surface, or two feet below the bottoms of the foundations, whichever is deeper. The lateral limits of the excavation should extend 5 feet or more beyond the outside edge of the new building/structure, or to a distance equal to the depth of the excavation, whichever is more. The lateral limits of the excavation for the retaining walls for the pedestrian ramp adjacent to the rail road alignment, should extend 2 feet or more beyond the outside edge of the new retaining wall structure.

For minor structures, such as benches, picnic tables, playground equipment and monument signs, remedial grading should be performed to a depth of 3 feet below the existing ground surface, or one foot below the bottoms of the foundations, whichever is deeper. The lateral limits of the excavation should extend 3 feet or more beyond the outside edge of the new foundation, or to a distance equal to the depth of the excavation, whichever is more.

Based on the anticipated depth of excavation for the proposed undercrossing, relatively dense alluvial soils are anticipated to be exposed and additional remedial grading should not be needed. However, construction details regarding the undercrossing are being developed and should be forwarded to Ninyo & Moore for review. For preliminary planning purposes,

the exposed foundation and pavement subgrades should be scarified, moisture-conditioned, and recompact to a depth of approximately 8 inches.

In paved areas, playfields, and sports fields, the existing fill and soft and/or loose alluvium should also be removed to competent native alluvial soils. In areas where there is no undocumented fill, or where the planned cuts remove existing fill and soft and/or loose alluvium, the subgrade should be scarified, moisture-conditioned to near optimum moisture content, and recompact. The limits of the scarification should extend laterally 12 inches beyond the outside edge of the pavement.

The excavation bottoms should be evaluated by our representative during the excavation work. Additional overexcavation of loose, soft, and/or wet areas may be appropriate, depending on our observations during construction. Prior to placing new compacted fill, the exposed bottom should be scarified, moisture-conditioned, and recompact to a depth of approximately 8 inches.

Care should be taken by the contractor to avoid undermining existing improvements located adjacent to the project site. In the event that excavations will extend within this zone, our office should be notified and appropriate recommendations provided, such as temporary shoring.

9.1.5 Temporary Excavations and Shoring

We anticipate that the majority of excavations during remedial grading and utility construction will not involve temporary vertical cuts and excavations can be sloped in accordance with OSHA regulations so that shoring will not be needed. These regulations provide trench sloping and shoring design parameters for trenches up to 20 feet deep based on the soil types encountered. While not anticipated, trenches/excavations over 20 feet deep should be designed by the contractor's engineer based on site-specific geotechnical analyses. For planning purposes, we recommend that the materials on site be considered as OSHA soil Type C.

Temporary excavations should be constructed in accordance with OSHA recommendations. For trench or other excavations, OSHA requirements regarding personnel safety should be met by using appropriate shoring (including trench boxes) or by laying back the slopes no steeper than 1.5:1 (horizontal to vertical). On-site safety of personnel is the responsibility of the contractor.

Where temporary slopes are not possible, shoring will be appropriate. The design of the shoring system should consider the excavation characteristics of the onsite soil, temporary excavation stability, and the impact of construction on existing structures.

Shoring systems will be constructed through fill and alluvium. The types of shoring systems for the project are unknown at this time. We anticipate that cantilevered shoring systems will be appropriate for the project to depths up to approximately 20 feet. Cantilevered and braced excavation shoring systems should be designed using the lateral earth pressure values presented on Figures 20 and 21, respectively. The recommended design pressures are based on the assumptions that the shoring system is constructed without raising the ground surface elevation behind the sheet piles, that there are no surcharge loads, such as soil stockpiles and construction materials, and that no loads act above a 1:1 (horizontal to vertical) plane extending up and back from the base of the sheet pile system. For sheet piles subjected to the above-mentioned surcharge loads, the contractor should include the effect of these loads on the lateral pressures against the sheet wall system. The shoring systems planned for the project should be reviewed by our office to evaluate the design considerations and geotechnical parameters used.

Ground settlement may occur behind the shoring system wall during excavation. The amount of settlement depends on the type of shoring system, the contractor's workmanship, and soil conditions. We recommend that structures/improvements in the vicinity of the planned shoring installation be reviewed with regard to foundation support and tolerance to settlement. To reduce the potential for distress to adjacent structures, we recommend that the shoring system be designed to limit the ground settlement behind the shoring system to $\frac{1}{2}$ inch or less, which would equal approximately $\frac{1}{2}$ inch of deflection. Possible causes of settlement that should be addressed include settlement during installation of the shoring, excavation for structure construction, construction vibrations, and removal of the support system. The vibrations from the driving of sheet piles may result in some dynamic settlement of granular soils that may affect the adjacent structures. We recommend that shoring installation be evaluated carefully by the contractor prior to construction and that ground vibration and settlement monitoring be performed during construction. Vibration and settlement monitoring should be performed during sheet pile driving. If settlement is detected or peak particle velocities of approximately $\frac{1}{2}$ inch per second or more are measured adjacent to existing improvements, the sheet pile driving should be stopped and evaluated. The evaluation may include changing the hammer vibration frequency and monitoring for settlement and vibrations. To reduce the potential for settlement associated with sheet pile removal, sheet piles may be left in place. In the event excessive settlement or other damage

occurs associated with the pile driving operations, it may be appropriate to perform grouting beneath nearby structure(s) to mitigate the pile driving effects. We recommend that shoring installation be evaluated carefully by the contractor prior to construction.

The contractor should retain a licensed, qualified and experienced engineer to design the shoring system. The shoring parameters presented in this report are minimum requirements, and the contractor should evaluate the adequacy of these parameters and make the required modifications for their design. We recommend that the contractor take appropriate measures to protect workers. OSHA requirements pertaining to worker safety should be observed.

9.1.6 Groundwater and Construction Dewatering

As described in Section 8, there are wide seasonal fluctuations in the depth to groundwater. Fluctuations in the depth to groundwater will occur and shallow groundwater depths should be anticipated. Therefore, if construction scheduling does not allow for the deeper excavations to be completed during the seasonal low point of groundwater during the summer season, seepage and/or groundwater should be anticipated during construction. Where groundwater is encountered during excavation, dewatering will be involved in order to perform work in a dry condition. The dewatering system design should be performed by a specialty dewatering contractor. The dewatering scheme may include pumping of groundwater from well points within or outside of the shoring. Drawing down of the water level within the excavation will affect the water level outside of the excavation. This will result in an increase in effective stresses and may induce settlement of the soils underlying adjacent improvements. In order to monitor the drawdown of the groundwater level outside the excavation, we recommend that monitoring wells be installed and monitored during construction where there are nearby existing settlement-sensitive improvements or structures, such as the railroad tracks.

Depending on the permeability of soil between the bottom of the excavation and the bottom of shoring system, as well as the effectiveness of water tightening between shored panels, drawing down of the water level within the excavation defined by the “cofferdam” may affect the water level outside of the excavation. We recommend that the dewatering be performed from inside the shoring system and the groundwater level be lowered no more than 3 feet below the excavation. Monitoring wells should be installed outside the excavation to monitor groundwater levels. Depending on the type of shoring and dewatering systems, the contractor may consider pump testing or an independent evaluation of the potential for groundwater inflow and/or resulting settlement. Additional measures, such as grouting and groundwater recharging may be implemented in the design to reduce the potential for groundwater inflow

and resulting settlement. Disposal of groundwater should be performed in accordance with guidelines of the Regional Water Quality Control Board.

9.1.7 Fill Material

In general, the on-site soils should be suitable for use as compacted fill provided they are free of construction/demolition debris, trash, roots, vegetation, deleterious materials, and contamination. Fill should generally be free of rocks or lumps of material in excess of 4 inches in diameter. Rocks or hard lumps larger than approximately 4 inches in diameter should be broken into smaller pieces or should be removed from the site. However, smaller particle sizes may be needed for the sports fields if recommended by the landscape architect. On-site soils used as fill will involve moisture conditioning to achieve appropriate moisture content for compaction. The clayey on-site soils are not suitable for use as retaining wall backfill and should not be placed beneath the proposed restroom building.

Fill used as backfill behind retaining walls should consist of free-draining, granular, non-expansive soil that conforms with the latest edition of “Greenbook” Standard Specifications for Public Works Construction for structure backfill. “Non-expansive” can be defined as soil having an expansion index (EI) of 20 or less in accordance with ASTM International (ASTM) Test Method D 4829 (CBC, 2022).

Import material should consist of clean, non-expansive, granular material, which conforms to the “Greenbook” for structure backfill. Soil should also be tested for corrosive properties prior to importing. We recommend that the imported materials satisfy the Caltrans (2021) criteria for non-corrosive soils (i.e., soils having a chloride concentration of 500 parts per million [ppm] or less, a soluble sulfate content of approximately 0.15 percent [1,500 ppm] or less, a pH value of 5.5 or more and an electrical resistivity of 1,500 ohm-centimeter [ohm-cm] or more). Import material should be submitted to Ninyo & Moore for review prior to importing to the site. The contractor should be responsible for the uniformity of import material brought to the site.

9.1.7.1 Oversize Materials

As indicated above, coarse gravel, cobbles, and boulders, can be placed within the compacted fill in selected zones in the creek bank stabilization areas. However, in slope areas, material larger than 4 inches should not be placed within a horizontal distance of 10 feet from the finish slope face. Blended mixtures of on-site sand and over-size material up to 2 feet in diameter may be placed in deeper fill areas.

Materials that are larger than 2 feet should be placed separately in windrows and should not be nested. These rock areas should be filled with clean sand flooded into the voids. After the initial sand flooding, the oversize rock zone should be track walked and then flooded again so that sand is filling the voids. General details for placement of oversize materials are presented on Figure 22. Details regarding the procedure should be evaluated at the time of construction based on the materials exposed and percentage of rock. Placement of rock-fill materials is a specialty procedure and should be evaluated and observed on a full-time basis during construction by our representative.

9.1.8 Fill Placement and Compaction

Fill soils placed should be compacted in horizontal lifts to a relative compaction of 90 percent or more as evaluated by ASTM D 1557. The lift thickness for fill soils will vary depending on the type of compaction equipment used but should generally be placed in horizontal lifts not exceeding 8 inches in loose thickness. Fill soils should be placed at slightly above the optimum moisture content as evaluated by ASTM D 1557. Special care should be taken to avoid damage to utility lines and foundations when compacting fill and subgrade materials.

9.1.9 Bank Protection with Soil-Cement

Based on our review of the preliminary design set drawings for City of Santa Clarita Via Princessa Park prepared by Psomas dated February 2023, soil-cement bank protection consisting of a buried soil-cement slope and compacted fill are proposed along the eastern side of Honby Creek and the southern side of the Santa Clara River for the proposed park site development. The proposed soil-cement bank protection will consist of an 8-foot-wide soil-cement reinforced slope that will be buried by compacted fill. To construct the bank protection slope, temporary excavations up to 20 feet in height will be performed. The temporary cuts will consist of a 1.5:1 (horizontal to vertical) slope from the park side and a 2:1 (horizontal to vertical) slope from the creek/river side. The planned excavation will effectively create a keyway that will have a 20-foot-wide bottom between the temporary backcuts. The excavation will be backfilled with an 8-foot-wide soil-cement zone upwards at 1.5:1 (horizontal to vertical) toward the park site and the remainder of the bank protection excavation will be backfilled with compacted fill. The compacted fill will bury the soil cement zone. The finish grade of the slopes that will ascend from the creek/river elevation will be constructed at 3:1 (horizontal to vertical) and will be less than 10 feet in height.

The upper 20 feet on-site soils at the proposed soil-cement slope areas generally include interbedded layers of sandy silt, sandy clay, sand with silt, silty sand, and poorly graded sand with gravel. The on-site fine-grained soils including silt and clay are not suitable for

constructing the 1.5:1 (horizontal to vertical) soil-cement zone. Accordingly, we selected two types of on-site soils from our borings B-2, B-3, B-4, B-5 and B-7 for our soil-cement laboratory testing. One of the samples combined the surficial silty sands (SM) from boring B-4 and B-5 and the other sample combined the more poorly graded sands (SP-SM) that have less fines content from the interbedded poorly graded sand layers in the upper 20 feet from borings B-2, B-3, B-4, B-5 and B-7. The gradation test results for these two soil samples are provided in Table 2.

Table 2 – Gradations for On-Site Soils Used in Soil-Cement Testing		
Soil Type	SM	SP-SM
Particle Size	Percent Passing	Percent Passing
3 inches	100	100
¾ inch	95	93
No. 4	80	73
No. 30	55	33
No. 50	43	18
No. 200	21	7

According to the guidelines from Portland Cement Association (PCA, 2006), the soil cement for water resources applications including bank protection should have a 7-day compressive strength greater than 750 pounds per square inch (psi). Our soil-cement compressive strength tests of SM type soils were performed in accordance with ASTM D558 and ASTM D1633 using 3, 5, 7 and 9 percent of cement by dry weight. The compressive strength tests were conducted with soil-cement samples cured for 3, 7 and 28 days for each sample. However, the soil-cement sample with 9% cement mix compressive test was performed only at 7 days curing due to the lack of sufficient sample volume. The results of the compressive strength tests for the SM soils are provided in Table 3.

Table 3 – Compressive Strength Test Results		
Soil Type	SM	SP-SM
Cement % by weight/ Days of curing	Compressive Strength (psi) SM Soil Type	Compressive Strength (psi) SP-SM Soil Type
3/3	350	
3/7	370	
3/28	380	
5/3	430	
5/7	540	
5/28	610	
7/3	540	
7/7	570	

Table 3 – Compressive Strength Test Results		
Soil Type	SM	SP-SM
Cement % by weight/ Days of curing	Compressive Strength (psi) SM Soil Type	Compressive Strength (psi) SP-SM Soil Type
7/28	610	
9/7	640	
9/7		970
9/7		1000

Since the soil-cement 7-day compressive strength for the SM soil type did not meet the required 750 psi, select SP-SM soil samples were combined to conduct additional compressive strength tests. In order to increase the strength, we selected to compact the soil-cement mix to 95 percent of the Proctor density as evaluated by ASTM D1557 method. Our test results indicated the 7-day compressive strength of the soil-cement using select SP-SM soils ranging from 970 to 1,000 psi as shown in Table 3

Based on our test results and experience, we estimate approximately 9 to 11 percent cement by dry weight of soil will need to be added to the selective granular onsite soils to achieve the desired strength for the creek/river bank stabilization. The selective granular on-site soils should conform with the gradations presented in Table 4 below.

Table 4 – Recommended Gradations for On-Site Soils Used in Soil-Cement	
Particle Size	Percent Passing
1 to 3 inches	98 - 100
¾ inch	80 - 100
No. 4	55 - 80
No. 40	30 - 70
No. 200	5 - 15
Note: Sand Equivalent (SE) should be 15 or more in accordance with California Test (CT) 217.	

Due to the variation of on-site soils available for soil-cement application, a laboratory testing program should be implemented to evaluate soil-cement materials proposed for use by the contractor and establish a suitable mix design prior to construction. Selective grading should be anticipated by the contractor so that the more granular onsite soils are used in the soil-cement zone.

Soil-cement may either be mixed in a central mixing plant or mixed in-place using traveling mixing machines. The soil and cement should be mixed well, moisture conditioned to near optimum moisture content (between one percent below to two percent above the optimum

moisture content), placed, and compacted to 95 percent or more of the maximum dry density of the soil content mixture as evaluated by ASTM D 1557. We recommend that the mixed soil-cement be placed in successive horizontal layers in lifts 8 inches or less in thickness measured prior to compaction. In addition, we recommend a soil-cement test section be conducted prior to the construction of soil-cement bank so that the plant, equipment, material and mix designs proposed for the work meet the intent of the specifications. Construction of soil-cement should be performed in accordance with the project specifications, the suggested guidelines by the Portland Cement Association (PCA, 2006) as well as the relevant ordinances from government agencies.

9.2 Seismic Design Considerations

Design of the proposed improvements should be performed in accordance with the requirements of governing jurisdictions and applicable building codes. Table 5 presents the seismic design parameters for the site in accordance with 2022 CBC guidelines.

Table 5 – 2022 California Building Code Seismic Design Criteria	
Site Coefficients and Spectral Response Acceleration Parameters	Values
Site Class	D
Mapped Spectral Response Acceleration at 0.2-second Period, S_s	2.276g
Mapped Spectral Response Acceleration at 1.0-second Period, S_1	0.822g
Site-Specific Spectral Response Acceleration at 0.2-second Period, S_{MS}	2.301g
Site-Specific Spectral Response Acceleration at 1.0-second Period, S_{M1}	1.891g
Site-Specific Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	1.534g
Site-Specific Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	1.261g
Site-Specific Maximum Considered Earthquake Geometric Mean (MCE_G) Peak Ground Acceleration, PGA_M Ground Acceleration, PGA_M	0.904g

9.3 Foundations

The proposed restroom building and other minor structures such as picnic tables, concrete benches, playground equipment, and signage may be supported on shallow spread footing foundations or mat foundations bearing on engineered fill material compacted in accordance with the recommendations presented in the Earthwork section of this report. Foundations should be designed in accordance with structural considerations and the following recommendations. In addition, requirements of the appropriate governing jurisdictions and applicable building codes should be considered in the design of the structure.

9.3.1 Spread Footings

Footings should extend 24 inches or more below the adjacent finished grade. Continuous footings should have a width of 18 inches or more and should be reinforced with two No. 5

steel reinforcing bars, one placed near the top and one placed near the bottom of the footings. Further details should be in accordance with the recommendations of the structural engineer.

Footings, as described above, may be designed using an allowable bearing capacity of 3,000 psf. The allowable bearing capacity may be increased by one-third when considering loads of short duration such as wind or seismic forces. Total and differential settlements for footings designed and constructed in accordance with the above recommendations are estimated to be less than approximately one and ½ inch over a horizontal span of 40 feet, respectively under the static loading condition. Under the seismic event, we anticipate that the total and differential settlements are 3 inches and 1½ inch over a horizontal span of 40 feet, respectively.

Footings bearing on compacted fill may be designed using a coefficient of friction of 0.35, where the total frictional resistance equals the coefficient of friction times the dead load. Footings may be designed using a passive resistance of 350 psf per foot of depth for a level ground condition up to a value of 3,500 psf. The allowable lateral resistance can be taken as the sum of the frictional resistance and passive resistance, provided the passive resistance does not exceed one-half of the total allowable resistance. The passive resistance may be increased by one-third when considering loads of short duration such as wind or seismic forces.

9.3.2 Mat Foundation

Based on our analysis and in accordance with Table 12.13-3 of ASCE 7-16, it is our opinion that the proposed restroom and other minor structures may be supported by a mat foundation. The mat foundation should be founded approximately 2 feet below the adjacent finish grade and designed using a net allowable bearing capacity of 3,500 psf and a coefficient of friction of 0.35 for sliding resistance. Under the static loading condition, the total and differential settlement corresponding to this allowable bearing load are estimated to be less than approximately 1 inch and ½ inch over a horizontal span of 40 feet, respectively. Under the seismic event, we anticipate that the total and differential settlements are 3 inches and 1½ inch over a horizontal span of 40 feet, respectively.

Mat foundations typically experience some deflection due to the loads placed on the mat and the reaction of the soils directly underlying the mat. A design modulus of subgrade reaction (K) of 100 tons per cubic foot may be used for the compacted subgrade soils in evaluating such deflection.

9.3.3 Slabs-On-Grade

Slabs-on-grade, such as for the proposed pickle ball courts, should be designed by the project structural engineer based on the anticipated loading conditions. As a minimum we recommend that slabs have a thickness of 5 inches or more, and be reinforced with No. 4 steel reinforcing bars placed 18 inches on-center (each way) in the middle one-third of the slab height. Exterior slabs-on-grade may be 4 inches thick. The proper placement of the reinforcement in the slab is vital for satisfactory performance. The floor slab and foundations should be tied together by extending the slab reinforcement into the footings. The slab should be underlain by a polyethylene vapor retarder, 10-mil or thicker. The vapor retarder should further be underlain by a 4-inch-thick layer of sand or gravel with a particle size of approximately $\frac{3}{4}$ inch or smaller. The vapor retarder is recommended in areas where moisture sensitive floor coverings are anticipated. Soils underlying the slabs should be moisture conditioned and compacted in accordance with the recommendations contained in this report prior to concrete placement. Joints should be constructed at intervals designed by the structural engineer to help reduce random cracking of the slab.

9.3.4 Deep Foundations for Railroad Undercrossing

Due to the heavy demand in lateral load for the proposed railroad undercrossing foundation, we understand the driven Cast-In-Steel-Shell (CISS) pile was specified by the design structural engineer. In addition, we understand the proposed undercrossing is to be designed per AREMA LFD Bridge Design Specifications along with Caltrans Amendments to AASHTO LRFD (Caltrans, 2019) using the factored design loads presented in Table 6.

Table 6 – Foundation Factored Design Loads for Railroad Undercrossing												
Location	Service I Limit State (kips)				Strength Limit State (kips)				Extreme Limit State (kips)			
	Comp. (kips)	Tension (kips)	Shear (kips)	Moment (kip-ft)	Comp. (kips)	Tension (kips)	Shear (kips)	Moment (kip-ft)	Comp. (kips)	Tension (kips)	Shear (kips)	Moment (kip-ft)
Abutment 1/2	600	0	20	660	1,000	0	43	923	890	250	555	7,222

Axial pile capacities and the recommended lengths for the piles were analyzed using the computer program Apile (Ensoft, 2019). Lateral pile capacities were analyzed using the computer program LPile (Ensoft, 2022). Soil strength parameters used in the analysis as presented in Table 7 were estimated from our sampler blow counts and direct shear test data.

Table 7 – Soil Parameters for Pile Analysis

					Apile Parameters		Lpile Parameters	
Soil Layer	Depth below Pile Head (ft)	Approximate Elevation (ft)	Effective Unit Weight, γ_t (pcf)	Friction Angle, Φ (degrees)	Soil Type	Undrained Shear Strength (psf)	Soil Type	Soil Modulus, K (pci)
1	0 to 12	1390 to 1378	0*	0	Sand	0	Sand	p-y curves
2	12 to 15	1378 to 1375	115.0	32	Sand	0	Sand	160
3	15 to 34	1375 to 1356	52.6	32	Sand	0	Sand	125
4	34 to 62	1356 to 1328	52.6	0	Clay**	500	Liquefied Sand	-
5	62 to 93	1328 to 1297	62.6	36	Sand	0	Sand	125

Notes:

*1 pcf used in Lpile due to program limits

**Residual shear strength used in Apile for liquefied layer

ft – feet

psf – pounds per square foot

pci – pounds per cubic inch

pcf – pounds per cubic foot

Pile capacities were evaluated for 5-foot and 6-foot diameter CISS piles. Axial capacities were based on side friction resistance assuming soil plugging during pile driving plus only the end bearing from the tip of the steel shell (not with the concrete inside the CISS) in our analysis. Our analysis indicated that the recommended 5-ft-diameter and 6-ft-diameter CISS piles provided in Table 8 are suitable to support the design loads. Results of our LFD analysis are provided in Appendix F.

Table 8 – Foundation Design Recommendations (LFD) for Railroad Undercrossing

Location	Pile Type	Cut-Off Elevation (feet)	Total Permissible Pile Settlement (inch)	Required Factored Nominal Resistance Per Pile (kips)						Design Tip Elevation (feet)	Specified Tip Elevation (feet)
				Service I Limit State ($\phi = 0.7$)	Strength Limit State ($\phi = 0.7$)	Extreme Limit State ($\phi = 1.0$)					
				Comp. (kips)	Comp. (kips)	Comp. (kips)	Tension (kips)	Shear (kips)	Moment (kip-ft)		
Abutment 1/2	60" CISS	1,390	1.0	600	1,000	890	250	555	7,222 ⁴	1,313 (a-I)	1,313
										1,324 (a-II)	
										1,325 (c)	
										1,320 (d)	
Abutment 1/2	72" CISS	1,390	1.0	600	1,000	890	250	555	7,222 ⁵	1,316 (a-I)	1,316
										1,327 (a-II)	
										1,325 (c)	
										1,316 (d)	

Notes:

- Design tip elevations are controlled by: (a-I) Compression (Strength Limit), (a-II) Compression (Extreme Event), (c) Settlement, and (d) Lateral Load.
- The specified tip elevation shall not be raised above the design tip elevations for tension, lateral, and tolerable settlement.
- The Loading information at the top of the CISS pile for the extreme event was provided by the project structure consultant.
- Fixed head condition, estimated deflection at pile head 1.9 inches.
- Fixed head condition, estimated deflection at pile head 1.2 inches.

9.3.5 Construction Considerations for CISS Piles

We recommended that Section 49-2 “Driven Piling” of the Caltrans Standard Specifications (2018) be followed for construction of steel piles. Section 49-3.03 provides guidelines for driven steel shells filled with concrete and reinforcement. All piles installation should be observed by a geotechnical engineer or regulatory agency. The contractor should furnish specific data of pile driving equipment, operating hammer and energy information. The contractor should carefully examine the subsurface soil conditions and make their own interpretation and perform independent study on the constructability of the piles. Prior to implementation of pile driving activities, the contractor should establish a construction monitoring program as described in Section 10. If unanticipated pile driving conditions are encountered during production driving, further consultation may be required.

Moderate to hard driving conditions should be anticipated below 20 feet deep of driving. Central relief drilling may be required during pile installation to attain specified tip elevation. Pile capacity is expected to develop after driving as a result of soil “freeze” and dissipation of excess pore water pressure. The gain of pile capacity after initial driving may be evaluated based on “re-driving” after a minimum of 48-hour set-up. We recommended that a Pile Driving Analyzer (PDA) be used to evaluate the pile capacity. Typical applications of the PDA include capacity evaluation during driving and re-striking.

The inside of CISS piles will be drilled out and replaced with reinforcement and concrete to meet structural design requirements. The depth of interior cleaning and concrete replacement will depend on the structural demands. Per Caltrans Memo to Designers 3-1 (2017), a minimum of 2 times the pile diameter of the soil plug should be left in the bottom portion of shells for pile construction. In addition, a concrete seal course at least 4 feet thick should be placed on top of the soil plug. Thicker seal course may be required to prevent quick soil condition. Due to presence of sandy soils, the procedures for constructing the seal course should be such that the tremied concrete does not get contaminated with native sand and fines (silt and clay) inside the steel piles. It would be necessary to take appropriate steps to allow sand particles and fines (silt and clay) to settle down in the water to reduce such contamination. The use of polymer slurry and rock/gravel bag may be considered to help construction of the seal course. During drilling and cleaning of the inside of the piles, the water head inside the piles should be maintained to counterbalance water pressure from the bottom of the shell.

9.3.6 Cast in Drilled Hole (CIDH) Piles for Pole Foundations

Light poles and awnings typically impose relatively light axial loads on cast in drilled hole (CIDH) pile foundations and the pile dimensions will generally be controlled by the lateral load demand. Based on our geotechnical engineering analyses, we recommend that CIDH piles be designed for an allowable skin friction of 200 pounds per square foot (psf) for downward axial loads and 150 psf for uplift loads. In addition, an allowable soil lateral bearing pressure of 150 psf per foot of depth can be used for evaluating the lateral pile capacity. Due to the soil arching effect, the soil lateral bearing may be applied to two times of the pile diameter. The allowable bearing values may be increased by one-third when considering loading of short duration such as wind or seismic forces. The pile dimensions (i.e., diameter and embedment) should be evaluated by the project structural engineer.

9.3.7 CIDH Piles for Pole Foundations Construction Considerations

The pile drilling contractor should mobilize equipment of sufficient size and operating capability to achieve the recommended embedment length. The excavation technique chosen by the contractor should not adversely affect the quality or strength of the shaft side or end bearing materials. If refusal is encountered in these materials during actual installation, Ninyo & Moore should be retained to evaluate the subsurface condition to establish that refusal has been met with adequate drilling equipment.

It is anticipated that the soils at the site may be subject to caving and heaving. Depending on the depth of the pile and the time of drilling, groundwater may be encountered during drilling and the contractor should be prepared to advance the CIDH piles under such conditions. Casing may be needed in the shaft hole to keep the drilled shaft from caving and to reduce water infiltration. While placing concrete, the casing should be withdrawn gradually.

The concrete should be placed in the annular space between the steel reinforcing cage and the drill-hole surface using the tremie method. Concrete utilized in the drilled shaft should be a fluid mix with sufficient slump so that it will fill the void between the steel reinforcing cage and the drill-hole wall. The contractor should take care to reduce enlargement of the excavation at the tops of the CIDH pile, which could result in mushrooming of the pile top.

The drilled hole should be cleaned prior to placement of concrete. Care should be taken to check that the soils at the drilled shaft bottom have not been disturbed. The successful advancement of the drill hole for the construction of CIDH pile will depend largely on the suitability of the drilling equipment and the skill of the operator. The drilled foundation contractor should make an effort to reduce the time during which the excavation remains

open. The contractor should schedule the sequence of operations so that the excavation can be finished, the steel reinforcing cage placed, and the concrete placed within the same work day. The drilled hole should not be left open overnight. In case of delay in placing concrete within the drilled hole due to equipment breakdown or other unforeseen circumstances, casing may be used to maintain the integrity of the hole. While placing concrete, the casing should be withdrawn gradually.

The CIDH pile installation should be observed by Ninyo & Moore to check that, among other things: 1) subsurface conditions are as anticipated, 2) the CIDH pile is constructed to the specified size and penetration, 3) the CIDH pile is within allowable tolerances for plumbness, and 4) reinforcements are placed per project specifications. These items are fundamental to the installation and behavior of the CIDH pile. Furthermore, we recommend the following for the installation of the CIDH pile:

- The clear spacing between the steel reinforcing cage and the drill-hole surface should be three times the maximum size of the coarse aggregate used in the concrete.
- Centralizers should be installed to keep the steel reinforcing cage positioned per project specifications.
- If casing is used, a sufficient head of concrete that fills the casing should be used before pulling the casing.
- The drill-hole excavation and cleaning should be observed prior to placement of reinforcing steel and concrete.
- Materials testing and inspection services should be performed, including testing of concrete compressive strength and steel tensile strength tests.

9.4 Hardscape

We recommend that new exterior concrete sidewalks and flatwork (hardscape) have a minimum thickness of 4 inches and be reinforced with No. 4 steel reinforcing bars placed 24 inches on-center (each way) near the mid-height of the slab. The hardscape should be underlain by 4 inches of granular material such as Crushed Aggregate Base or Crushed Miscellaneous Base and installed with crack-control joints at an appropriate spacing as designed by the structural engineer to reduce the potential for shrinkage cracking. Positive drainage should be established and maintained adjacent to flatwork. To reduce the potential for differential offset, joints between the new hardscape and adjacent curbs, existing hardscape, building walls, and/or other structures, and between sections of new hardscape, should be doweled.

9.5 Cantilever Retaining Wall

Based on our understanding of the project, the proposed ramps to the undercrossing will include construction of cantilever retaining walls with heights ranging from approximately 5 to 15 feet. In addition, a cantilever retaining wall up to approximately 12 feet in height will be constructed underneath the proposed bridge that separates the pedestrian and vehicular access. After removal of the undocumented fill materials, the proposed walls may be founded on shallow spread (continuous) footings bearing on competent alluvial materials or engineered fill. For retaining wall design, we recommend using a unit weight of 120 pcf, friction angle of 32 degrees, cohesion of 0 psf for the retaining and foundation soil, where applicable.

The spread footings should be designed in accordance with structural considerations and the recommendations presented herein. In addition, requirements of the appropriate governing jurisdictions and applicable codes should be considered in the design of the retaining walls.

9.5.1 Soil Bearing Capacity and Settlement

For design purposes, it is assumed that retaining wall heights of approximately 5 feet, 10 feet, and 15 feet will correspond to the similar excavation depths from the existing ground surface. The net allowable bearing capacities at different excavation depths are presented in the following Table 9. For other excavation depths, the bearing capacities may be interpolated. The net allowable bearing capacities may be increased by one-third when considering loads of short duration such as wind or seismic forces. Footings should be embedded at a depth of 24 inches or more below the lowest adjacent finished grade and supported on competent alluvial materials or engineered fill. The retaining wall footings should have a width of 18 inches or more.

The wall footings are anticipated to undergo total static settlement of about 1 inch when subjected to the recommended net allowable bearing pressure. The differential settlement is estimated to be approximately $\frac{1}{2}$ inch over a horizontal distance of approximately 40 feet. The proposed excavations and compaction of fill for retaining wall construction is anticipated to mitigate the dynamic settlement of dry soils discussed in this report. However, the saturated soil dynamic total and differential settlement of $\frac{1}{2}$ inch and $\frac{1}{4}$ inch over a horizontal distance of 40 feet, respectively, should be considered in design.

Table 9 – Bearing Capacity for Wall Footings

Retaining Wall Height/ Approximate Excavation Depth from Existing Grade (ft)	Minimum Footing Width (ft)	Minimum Footing Embedment Depth (ft)	Total Permissible Settlement (inches) ¹	Service Limit State	Extreme Event Limit State $\phi = 1.0$
				Permissible Net Contact Stress or Net Allowable Bearing Capacity (ksf)	Gross Nominal Bearing Resistance or Ultimate Bearing Capacity (ksf)
5.0	1.5	2.0	1.5	2.0	6.0
10.0	2.0	2.0	1.5	3.0	9.0
15.0	2.5	2.0	1.5	3.5	10.5

Notes:

¹ Total settlement of 1.5 inches includes 1.0 inch of static settlement and 0.5 inch of dynamic settlement.

9.5.2 Lateral Resistance

Wall footings embedded in engineered fill or competent alluvial materials may be designed using a passive resistance value of 350 psf per foot of depth, up to a value of 3,500 psf. A coefficient of friction of 0.39 (where, the total frictional resistance equals the coefficient of friction times the dead load) may also be used under similar conditions. The allowable lateral resistance can be taken as the sum of the frictional resistance and passive resistance provided the passive resistance does not exceed one-half of the total allowable resistance. The passive resistance may be increased by one-third when considering loads of short duration such as wind or seismic forces.

9.5.3 Lateral Earth Pressures

We recommend that the active lateral earth pressure for yielding retaining walls presented in Figure 23 and at-rest lateral earth pressures for restrained retaining walls presented in Figure 24 be used in design. In addition, any surcharge loading should be considered by the designer. Measures should be taken to reduce the potential for build-up of hydrostatic pressure behind the retaining walls. Drainage design should include free-draining backfill materials and perforated drains as depicted on Figure 25. Solid outlet pipes should be connected to the perforated drains and then routed to a suitable area for discharge of accumulated water. The portions of retaining walls supporting backfill should be coated with an appropriate waterproofing compound or covered with a similar material to inhibit infiltration of moisture through the walls. It is the responsibility of the project structural engineer and/or the retaining wall contractor to provide specifications for waterproofing materials and proper methods of application.

9.6 Tieback Retaining Walls

Due to the high shear demand in the extreme loading event for the CISS piles, we recommend that a tieback supported retaining wall is proposed be used on both sides and in front of the

abutments of the railroad undercrossing. The wall should be designed in accordance with the design recommendation illustrated in Figure 26, using the proposed CISS piles and tie-backs. A drainage panel should be placed against the soil between the piles for the height of the wall and be connected to suitable outlets.

9.6.1 Tiebacks

Tieback design should include review of the soil and geologic conditions and potential conflicts at each planned tie-back location. Tie-backs may consist of either multi-strand steel tendons or high tensile strength steel bars placed in inclined drilled holes and backfilled with low-slump concrete grout. The tie-backs should be designed for an allowable tensile strength of 50 kips, should be inclined at 15 degrees below horizontal, and should be 6 inches or more in diameter. If caving occurs, casing should be provided. Tie-backs anchored into alluvial deposits may be designed using a factored ultimate bond stress of 15 psi. Please note that the preliminary bond stress values provided here are for non-pressurized grouted anchors.

An unbonded length of 15 feet or longer should be maintained for each tie-back. Our analysis indicated that a total of 10 tiebacks installed between CISS piles at a 4-feet by 4-feet grid pattern with a bonded length of 30 feet into the dense alluvium will be required for the extreme limit state loading condition. Each tieback should be proof tested. After the completion of proof test, each tieback should be locked off at the design load against the reinforced concrete retaining wall in front of the CISS piles.

Tieback anchors should be installed in drilled holes using centering devices to improve anchor uniformity. The anchor holes should be filled with concrete, placed using tremie techniques, to the limit of the bonded length. The unbonded length should remain ungrouted until after testing and lock-off of the anchor. Anchors should be backfilled with lean-mix concrete or sands after testing to provide additional protection to potentially corrosive soils. If caving becomes a problem, the unbonded length should be backfilled with well-compacted sand or be cased prior to and during testing.

9.6.2 Tieback Testing

The tieback anchors should be tested during construction to evaluate the design assumptions and allowable pullout capacities. The contractor should provide equipment and instrumentation to check the adequacy of the tiebacks. A dial gauge capable of measuring displacements to 0.01-inch precision should be used to measure the anchor movement. A hydraulic jack and pump should be used to apply the test load, and the jack and a calibrated

pressure gauge should be used to measure the load. The standard testing procedures typically consist of the following methods.

- **Performance Tests** – These tests are performed on a limited number of production anchors to check that 1) the design load may be safely carried, 2) effective bonded length corresponds to the design requirements, and 3) the residual movement is within tolerable range. The performance test consists of incrementally applying cycles of anchor loading and unloading until the reference test load is attained. In order to evaluate the long-term creep potential, each load increment is maintained until the measured deflection is negligible (i.e., displacement rate is smaller than a specified displacement increment per log cycle of load-hold time) and a one-hour creep test is conducted under the reference test load. The reference test load should be 150 percent of the design allowable pullout capacity for permanent anchors. The performance test should be conducted on one or 5 percent of the total tie-back anchors, whichever is more.
- **Proof Tests** – These tests are performed on each tie-back anchor to check that the load-deflection behavior of the production anchor is consistent with the specified acceptance criteria. The proof test consists of a single cycle of incremental loading to the reference test load (i.e., 150 percent of the design allowable pullout capacity) followed by unloading. Each load increment is maintained until the measured deflection is negligible.

The performance and proof test schedules for the anchors, including the load increments, load hold periods, acceptance criteria, and repair mechanism of failed test anchors, should be developed by the contractor utilizing his experience on similar projects and anchor design/testing recommendations and guidelines presented in this report. In general, the acceptance criteria for the tested anchors should be based on the following aspects.

- In order to allow the load transfer to reach the anchor bond length, the deflection of the anchor head should exceed 80 percent of the calculated elastic elongation of the unbonded tendon length.
- Total anchor deflection measured at the reference test load should not exceed the calculated elastic elongation of the tendon length measured from the anchor head to the center of the bond length.
- Creep displacement should not exceed 0.10 inch during the final log cycle of the load-hold period.

The test schedules of the tie-back anchors and the acceptance criteria should be included in the project plans. The project plans should be signed and stamped by a professional engineer registered in the state of California. Ninyo & Moore should be given the opportunity to review the project plans to check its compliance with design and construction recommendations presented herein

9.7 Underground Utilities

We anticipate that utility pipelines will be supported on future compacted fill or deeper native alluvial soils. The depths of the pipelines are not known; however, we anticipate that the pipe invert depths will not exceed 10 feet.

9.7.1 Pipe Bedding

We recommend that bedding material be placed around pipe zones 1 foot or more above the top of the pipe. The bedding material should be classified as sand, should be free of organic material, and have a sand equivalent of 30 or more. We do not recommend gravel be used for bedding material because of the silty nature of the subsurface material. It has been our experience that the voids within gravel material are sufficiently large to allow fines to migrate into the voids, thereby creating the potential for sinkholes and depressions to develop at the ground surface. Where soft, wet soil conditions are encountered the trench excavation should be excavated approximately 1 foot or more below the pipe invert and should be backfilled with gravel wrapped in filter fabric.

Special care should be taken not to allow voids beneath and around the pipe. Compaction of the bedding material and backfill should proceed uniformly up both sides of the pipe. Trench backfill, including bedding material, should be placed in accordance with the recommendations presented in the preceding section.

9.7.2 Trench Backfill

Based on our subsurface evaluation, the on-site soils should generally be suitable for re-use as trench backfill provided they are free of organic material, clay lumps, debris, and rocks more than approximately 4 inches in diameter. We recommend that trench backfilling be in general conformance with the Standard Specifications for Public Works Construction ("Greenbook") for structure backfill. Fill should be moisture-conditioned to at or slightly above the laboratory optimum. Wet soils should be allowed to dry to a moisture content near the optimum prior to their placement as trench backfill. Trench backfill should be compacted to a relative compaction of 90 percent as evaluated by ASTM D 1557. Lift thickness for backfill will depend on the type of compaction equipment utilized, but fill should generally be placed in horizontal lifts not exceeding 8 inches in loose thickness. Special care should be exercised to avoid damaging the pipe during compaction of the backfill.

9.7.3 Modulus of Soil Reaction

The modulus of soil reaction is used to characterize the stiffness of soil backfill placed on the sides of buried flexible pipelines for the purpose of evaluating lateral deflection caused by the weight of the backfill above the pipe. We recommend that a modulus of soil reaction of 1,000 pounds per square inch be used for design, provided that granular bedding material is placed adjacent to the pipe, as recommended in this report.

9.8 Preliminary Pavement Design

We anticipate that future new parking lots will include new flexible pavement (AC) and rigid pavement (PCC). Traffic loading information was not available for our design at the time of preparation of this report. For planning purposes, we have assumed traffic index (TI) values of 5.0, 6.0 and 7.0 for the preliminary design of pavement sections. The TIs of 5.0 and 6.0 are generally used for designing pavements that will generally be used as parking stalls and/or driveways subjected to relatively light passenger vehicles and periodic heavy equipment/truck traffic. The TI of 7.0 is generally used for designing pavements for driveway areas that are subjected to more routine heavy equipment/truck traffic. Laboratory testing was performed on a representative soil samples and indicated an R-value ranging from 47 to 72. Accordingly, we selected using an R-value of 40 for design.

Based on the traffic indices described above, a design R-value of 40, and guidelines of the Caltrans Highway Design Manual (Caltrans, 2020), we have developed the following preliminary pavement sections for the project (Table 10). For the design of PCC pavements, we used the methodology presented in the Navy Pavement Design Manual (1979) assuming a 28-day compressive strength of concrete of 2,500 psi. We recommend that these pavement sections be re-evaluated once project-specific traffic indices are developed and the as-graded near-surface earth materials are further tested.

Table 10 – Preliminary Flexible Pavement Structural Section

Traffic Index	AC/CAB or CMB (inches)	PCC (inches)
5.0	3.0/4.0	5.0
6.0	3.0/6.0	5.5
7.0	4.5/6.5	7.0

Notes:

AC – Asphalt Concrete

PCC – Portland Cement Concrete, based on 28-day compressive strength of 2,500 psi.

CAB – Crushed Aggregate Base

CMB – Crushed Miscellaneous Base

Subgrade soils in areas to be paved should be prepared as recommended in the Earthwork section of this report. Expansive clayey soils, if encountered during grading, should not be placed within the upper 12 inches of the subgrade to reduce the potential for pavement damage. Crushed aggregate base (CAB) or crushed miscellaneous base (CMB) material should conform to the latest edition of the Standard Specifications for Public Works Construction “Greenbook,” Section 200. The CAB/CMB should be compacted to a relative compaction of 95 percent as evaluated by ASTM D 1557. CAB/CMB should be placed at slightly above the optimum moisture content as evaluated by ASTM D 1557. AC should conform to Section 203 of the Greenbook and should be compacted to a relative compaction of 95 percent. Final pavement sections should be selected

based on actual anticipated traffic loading conditions and evaluation of the subgrade materials at the time of construction.

9.9 Corrosivity

Laboratory testing was performed on representative samples of near-surface soil collected from borings B-2 and B-14 to evaluate soil pH, electrical resistivity, water-soluble chloride content, and water-soluble sulfate content. The soil pH and electrical resistivity tests were performed in general accordance with California Test Method (CT) 643. The chloride content test was performed in general accordance with CT 422. Sulfate testing was performed in general accordance with CT 417. The laboratory test results are presented in Appendix D.

The soil pH ranged between approximately 6.7 and 7.1 and the electrical resistivity ranged between approximately 7,450 and 8,380 ohm-cm. The chloride content of the samples ranged between approximately 25 and 130 ppm. The sulfate content of the tested samples was approximately 0.001 percent (i.e., 10 ppm). Based on the laboratory test results and Caltrans (2021) corrosion criteria, the project site can be classified as a non-corrosive site, which is defined as having earth materials with less than 500 ppm chlorides, less than 0.15 percent sulfates (i.e., 1,500 ppm), a pH of 5.5 or more, and an electrical resistivity of 1,500 ohm-cm or more.

9.10 Concrete Placement

Concrete in contact with soil or water that contains high concentrations of water-soluble sulfates can be subject to premature chemical and/or physical deterioration. The samples tested during this evaluation indicated a water-soluble sulfate content of approximately 0.001 percent by weight (i.e., 10 ppm). Based on the American Concrete Institute (ACI) 318-14 criteria 318-14 (ACI, 2019), the potential for sulfate attack is considered negligible for water-soluble sulfate contents in soils of less than 0.1 percent by weight (1,000 ppm), moderate for water-soluble sulfate contents in soils between 0.1 and 0.2 percent by weight (1,000 to 2,000 ppm), and severe for water-soluble sulfate contents in soils between 0.2 and 2 percent by weight (2,000 to 20,000 ppm). Accordingly, the on-site soils are considered to have a negligible potential for sulfate attack. Per the ACI 318-14 criteria (ACI, 2019), Type II cement is considered to be appropriate for the project. Due to the potential variability in soil conditions across the site, Type V cement with a water/cement ratio of 0.45 or less may be considered for the project.

In order to reduce the potential for shrinkage cracks in the concrete during curing, we recommend that the concrete for the proposed structures be placed with a slump of 4 inches based on ASTM C 143. The slump should be checked periodically at the site prior to concrete placement. We also recommend that crack control joints be provided in sidewalks and exterior hardscape in

accordance with the recommendations of the structural engineer to reduce the potential for distress due to minor soil movement and concrete shrinkage. The structural engineer should be consulted for additional concrete specifications.

9.11 Drainage

Good surface drainage is imperative for satisfactory site performance. Positive drainage should be provided and maintained to channel surface water off the pavement, away from foundations and off-site. Positive drainage is defined as a slope of 2 percent or more for a distance of 5 feet or more away from foundations and tops of slopes. Runoff should then be transported by the use of swales or pipes into a collective drainage system and discharged to suitable facilities. Surface waters should not be allowed to pond adjacent to footings or on pavements. Concentrated runoff should not be allowed to flow over asphalt pavement as this can result in early deterioration of the pavement. Area drains for landscaped and paved areas are recommended.

As described in Section 8.7, the park is adjacent to natural slopes that descend to and from the site. The natural slopes may experience erosion during significant rainfall events. Appropriate drainage improvements should be incorporated into the project plans to accommodate offsite water that flows into the site. Routine maintenance to remove eroded debris should be anticipated to maintain the functionality of the drainage system.

10 CONSTRUCTION MONITORING PROGRAM

We recommend that pre-construction condition surveys be performed on structures and improvements within approximately 50 feet of the proposed deeper excavations prior to construction, particularly for the railroad track undercrossing. This survey should include locating existing cracks and measuring widths of cracks, in combination with videotape documentation of existing conditions. In addition, interviews should be conducted with railroad representatives and utility owners so that existing knowledge about the age, type, and maintenance history of affected improvements/utilities is available prior to construction.

10.1 Construction Vibrations

People can perceive vibrations from construction activities at significantly lower levels than might cause cosmetic damage to structures. Jones & Stokes (2004) indicate that transient vibrations, such as from pile driving or construction activities, may be noticeable, and therefore may result in complaints, at peak particle velocities as low as 0.02 to 0.06 inch per second (ips). The vibrations may be disturbing and result in complaints and/or damage claims at peak particle velocities as

low as 0.2 to 0.4 ips. However, these vibration levels are well below the level considered to cause cosmetic damage to residential construction.

There is also the possibility of settlement of the soil during construction activities due to vibrations. This settlement may result in damage to structures. If the construction vibrations can be maintained below a peak particle velocity of 0.2 ips, it is anticipated the settlement can be limited to acceptable levels based on past projects in similar conditions.

For the above stated reasons, we recommend that seismographs be used in the early stages of construction to monitor the vibrations. Seismographs should be located near structures and improvements next to the construction activities. Additional seismographs should be located at various structures and improvements farther from construction activities to monitor vibrations as a function of distance from the sites. Periodic vibration monitoring is recommended during other construction activities. After review of the data obtained, the number of seismographs may be reduced at the discretion of the client and the geotechnical consultant.

10.2 Ground Surface Settlement

We recommend that arrays of ground surface settlement points be installed around the proposed deeper excavations. The contractor should submit a monitoring plan showing the proposed locations of settlement points for review and approval by the project engineer. We recommend that the contractor be responsible for maintaining total settlement at any survey point to less than ½ inch. If the settlements reach this limit, we recommend that a further review of construction methodologies be performed and appropriate changes be made.

10.3 Lateral Movement for Shoring Support System

We recommend that survey points be established behind excavations located in areas where existing structures are located above a 1:1 (horizontal to vertical) plane projected from the bottom of the proposed excavations to the ground surface. The survey points should be monitored and evaluated daily during excavation activities to provide an advanced warning system of potential problems. As discussed previously, we recommend that the shoring system be designed to limit the ground settlement behind the shoring system to ½ inch or less to reduce the potential for distress to adjacent structures and improvements. If settlement values reach ¼ inch, we recommend that a review of the contractor's methods be performed and appropriate changes be made, if needed.

11 CONSTRUCTION OBSERVATION

The recommendations provided in this report are based on our understanding of the proposed project and on our evaluation of the data collected based on subsurface conditions disclosed by widely spaced exploratory borings. It is imperative that the interpolated subsurface conditions be checked by our representative during construction. Observation and testing of compacted fill and backfill should also be performed by our representative during construction. We further recommend that the project plans and specifications be reviewed by this office prior to construction. In addition, we should review the plans and specifications prior to construction. It should be noted that, upon review of these documents, some recommendations presented in this report might be revised or modified.

During construction, we recommend that the duties of the geotechnical consultant include, but not be limited to:

- Observing clearing, grubbing, and removals.
- Observe remedial grading bottoms prior to fill placement and remedial grading bottoms for new structures.
- Observe subgrade conditions for stabilization for below-grade structures.
- Observe keyway excavations.
- Observing excavation, placement, and compaction of fill and soil-cement fill.
- Evaluating imported materials prior to their use as fill (if used).
- Performing field tests to evaluate fill compaction.
- Observe driven pile and CIDH pile construction.
- Observing foundation excavations for bearing materials and cleaning prior to placement of reinforcing steel or concrete.
- Performing material testing services including concrete compressive strength and steel tensile strength tests and inspections.

The recommendations provided in this report assume that Ninyo & Moore will be retained as the geotechnical consultant during the construction phase of this project. If another geotechnical consultant is selected, we request that the selected consultant indicate to the City and to our firm in writing that our recommendations are understood and that they are in full agreement with our recommendations.

12 LIMITATIONS

The field evaluation, laboratory testing, and geotechnical analyses presented in this geotechnical report have been conducted in general accordance with current practice and the standard of care

exercised by geotechnical consultants performing similar tasks in the project area. No warranty, expressed or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be encountered during construction. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation will be performed upon request. Please also note that our evaluation was limited to assessment of the geotechnical aspects of the project, and did not include evaluation of structural issues, environmental concerns, or the presence of hazardous materials.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Ninyo & Moore should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

This report is intended for design purposes only. It does not provide sufficient data to prepare an accurate bid by contractors. It is suggested that the bidders and their geotechnical consultant perform an independent evaluation of the subsurface conditions in the project areas. The independent evaluations may include, but not be limited to, review of other geotechnical reports prepared for the adjacent areas, site reconnaissance, and additional exploration and laboratory testing.

Our conclusions, recommendations, and opinions are based on an analysis of the observed site conditions. If geotechnical conditions different from those described in this report are encountered, our office should be notified, and additional recommendations, if warranted, will be provided upon request. It should be understood that the conditions of a site could change with time as a result of natural processes or the activities of man at the subject site or nearby sites. In addition, changes to the applicable laws, regulations, codes, and standards of practice may occur due to government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Ninyo & Moore has no control.

This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

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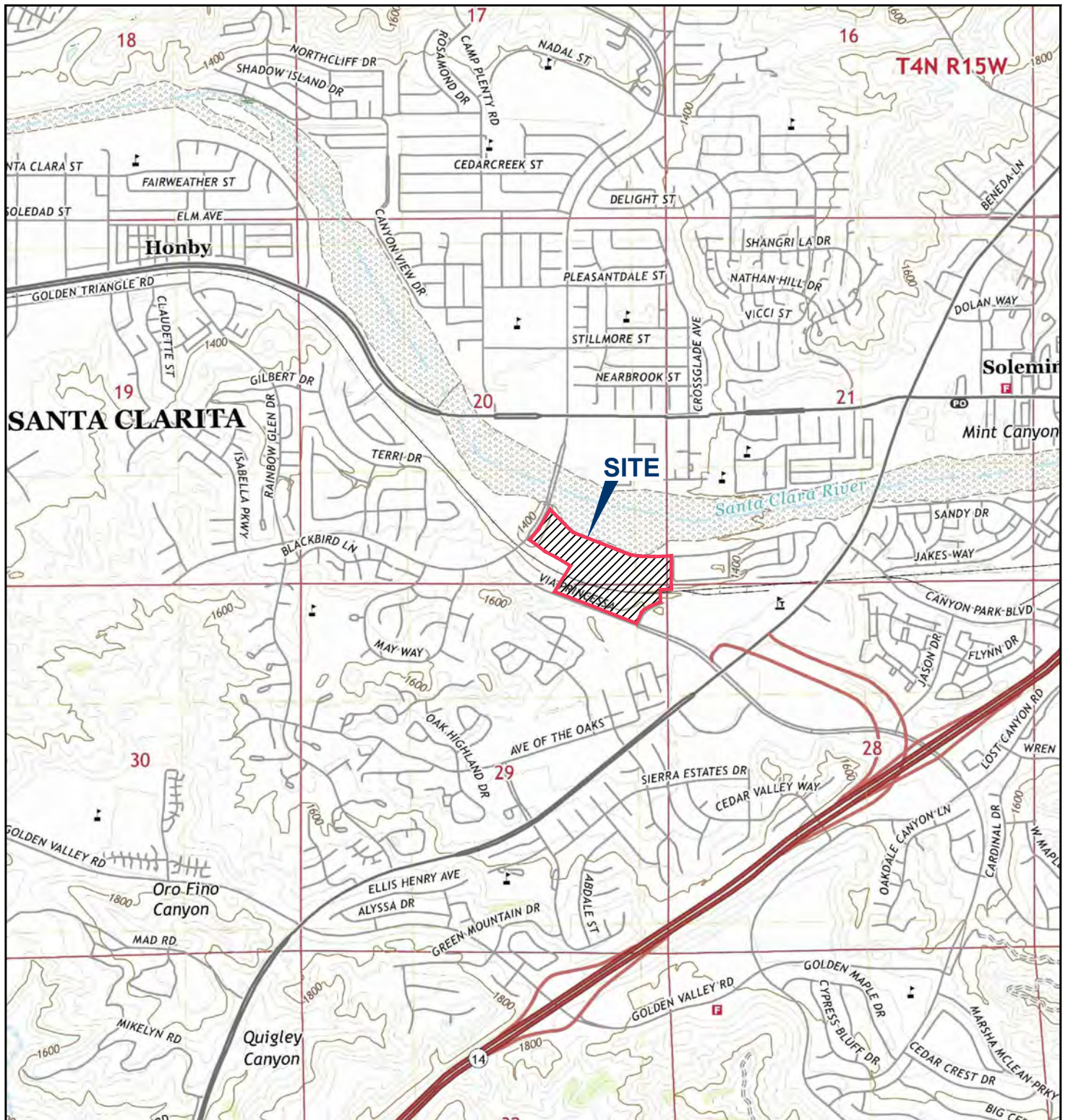
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FIGURES

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NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. | REFERENCE: USGS, 2022.

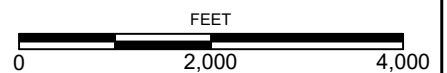


FIGURE 1

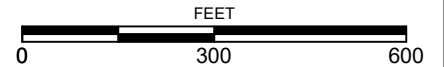
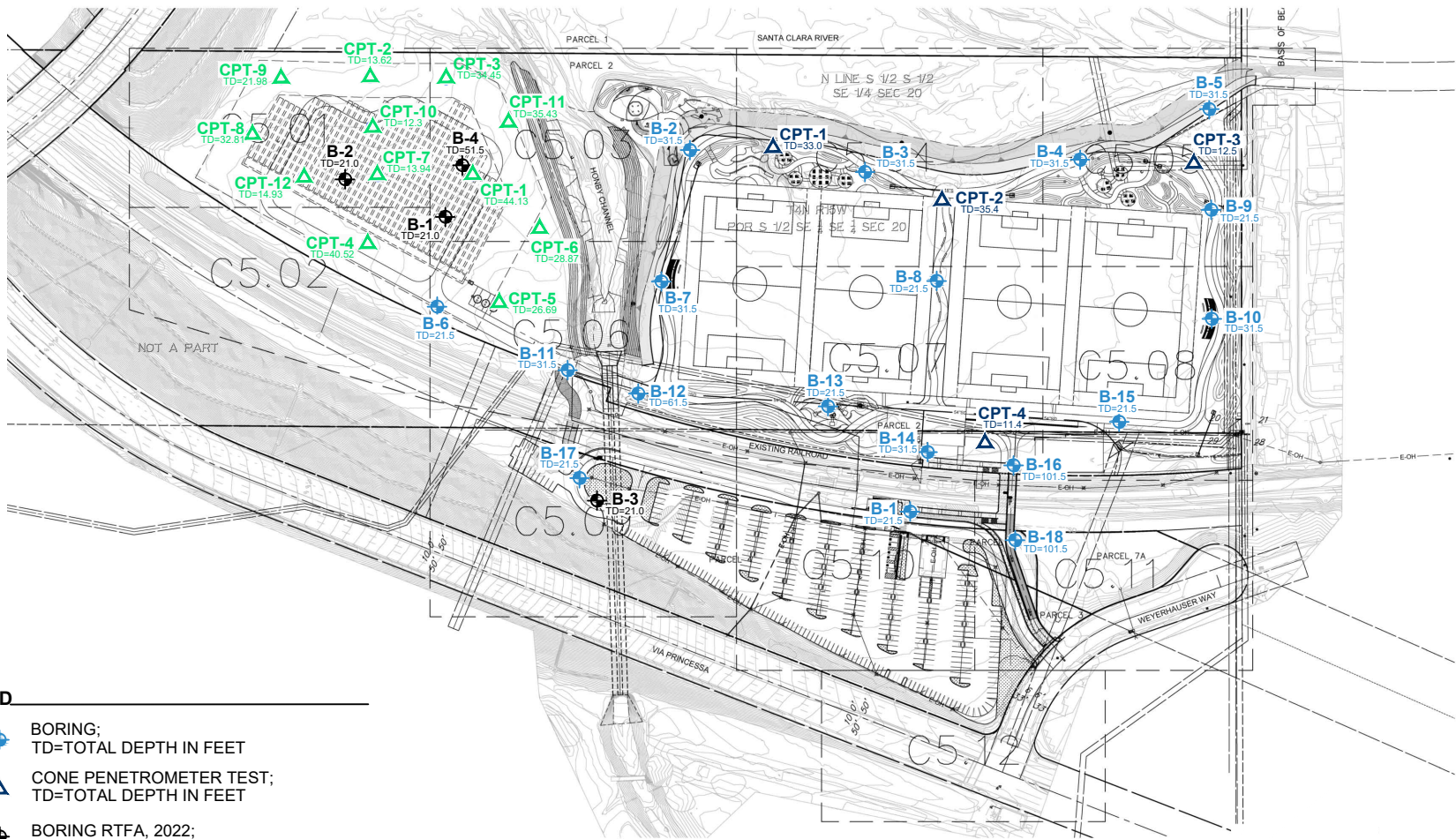


FIGURE 2

SITE PLAN AND EXPLORATION LOCATIONS

VIA PRINCESSA PARK
SANTA CLARITA, CALIFORNIA

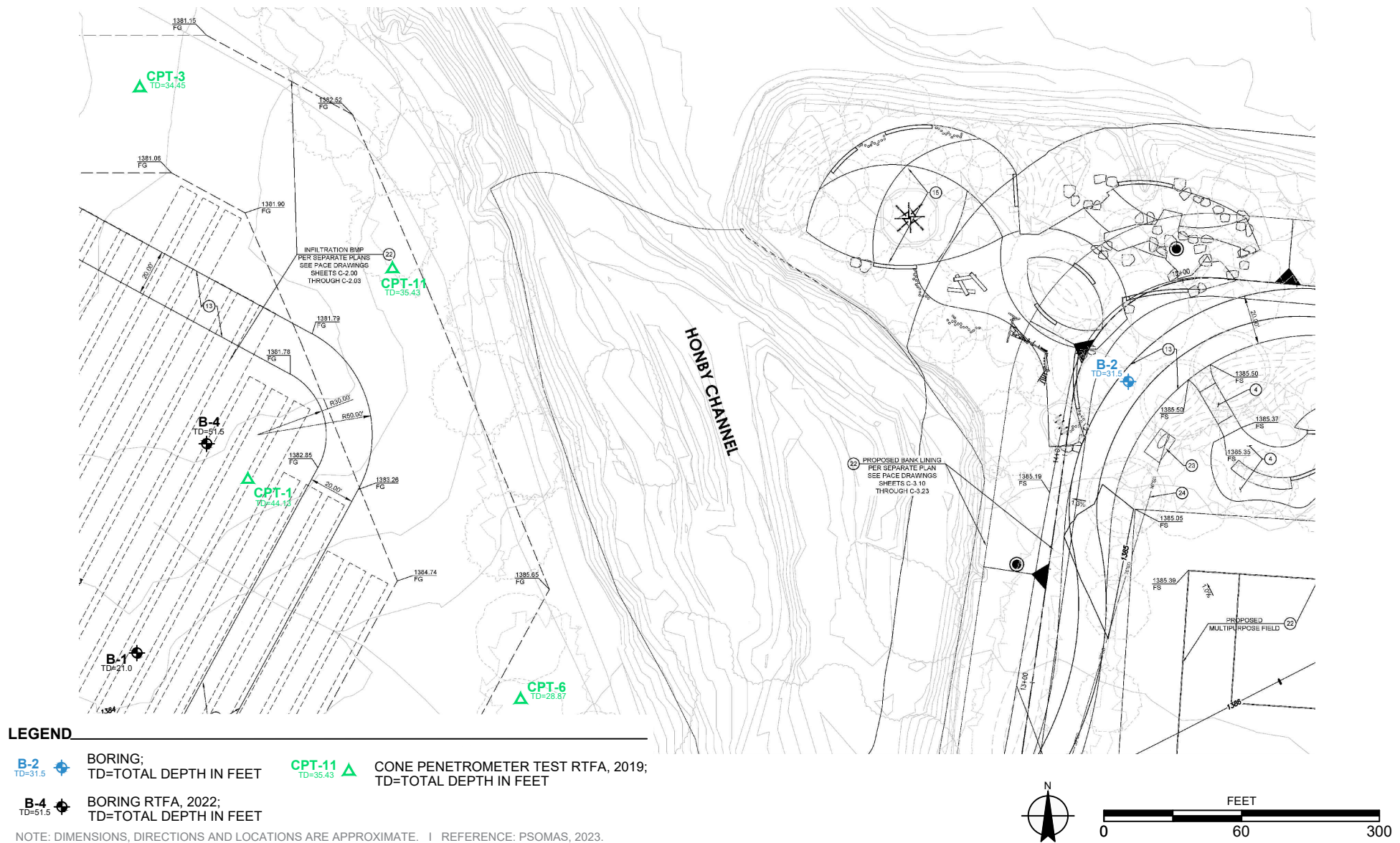


FIGURE 3

SITE PLAN AND EXPLORATION LOCATIONS

VIA PRINCESSA PARK
SANTA CLARITA, CALIFORNIA

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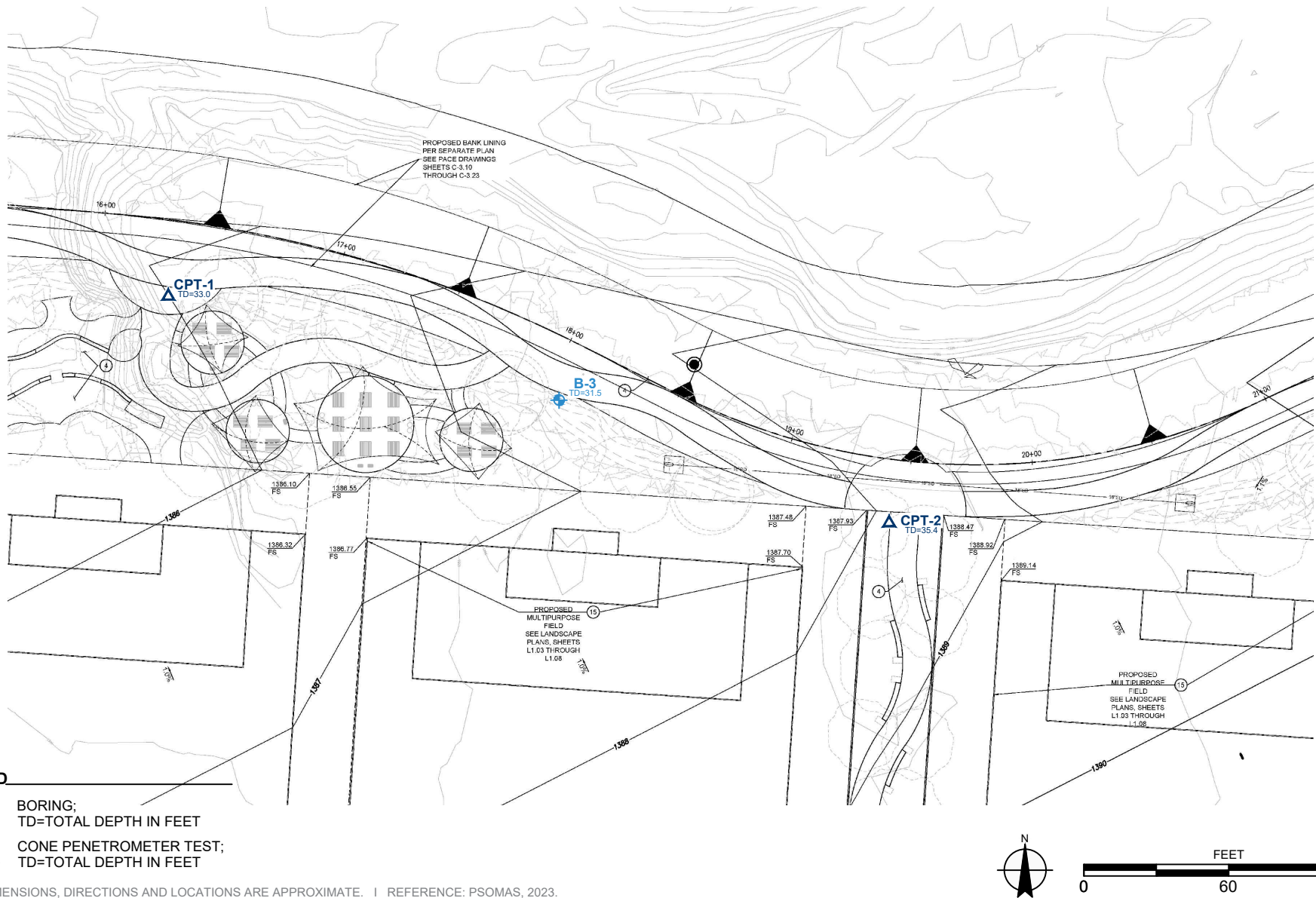


FIGURE 4

SITE PLAN AND EXPLORATION LOCATIONS

VIA PRINCESSA PARK
SANTA CLARITA, CALIFORNIA

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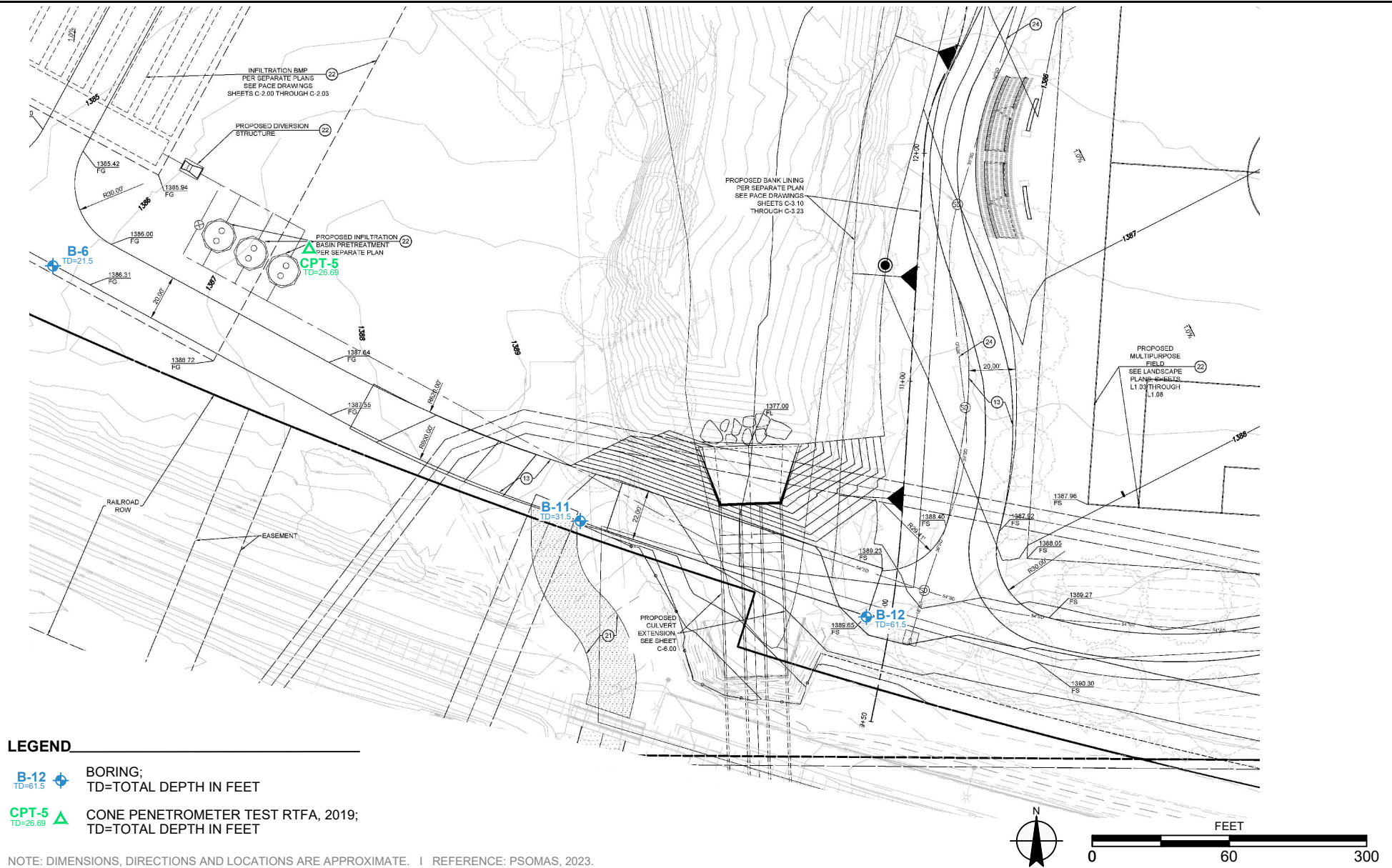
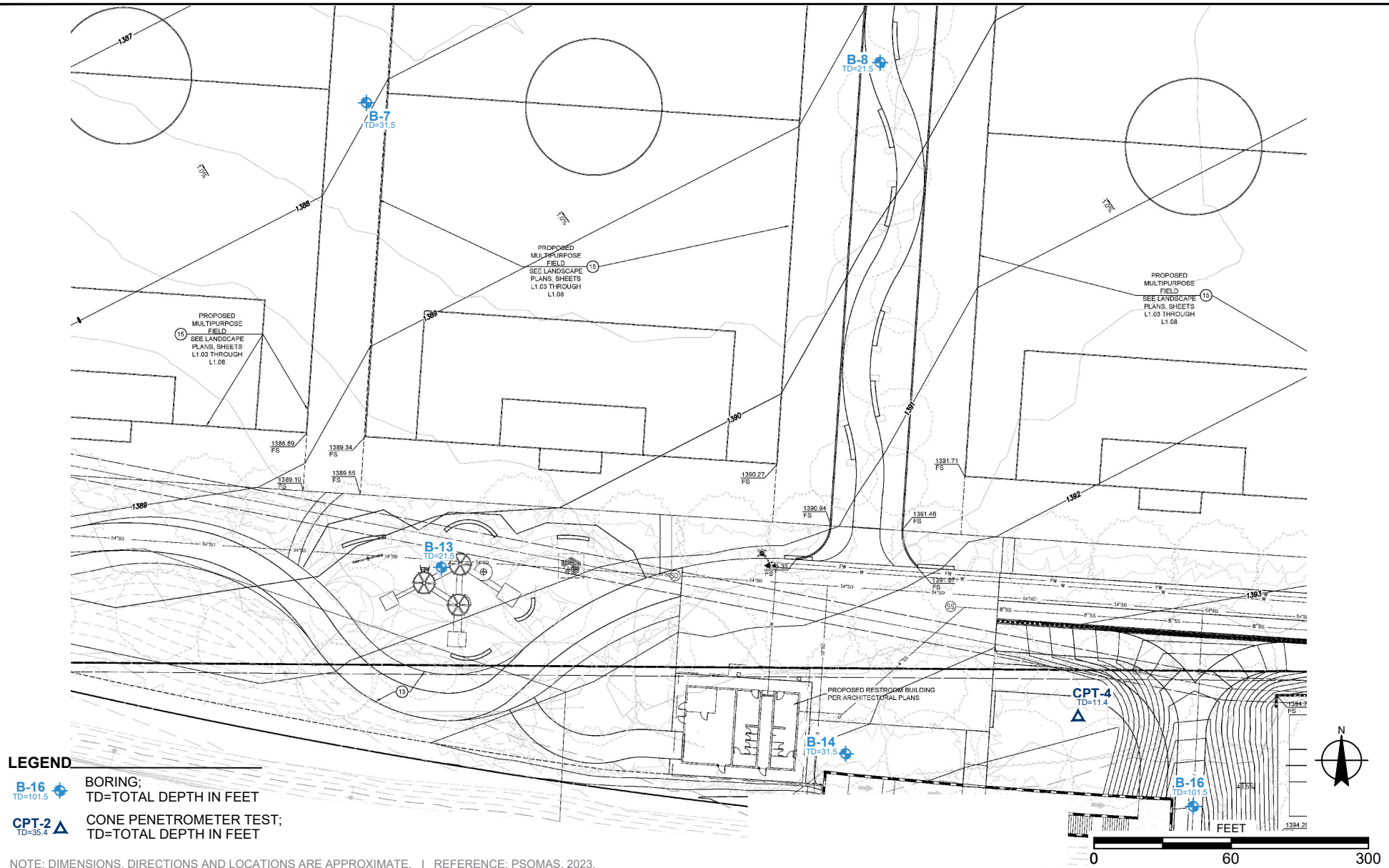


FIGURE 6

**FIGURE 7****SITE PLAN AND EXPLORATION LOCATIONS**

VIA PRINCESSA PARK
SANTA CLARITA, CALIFORNIA

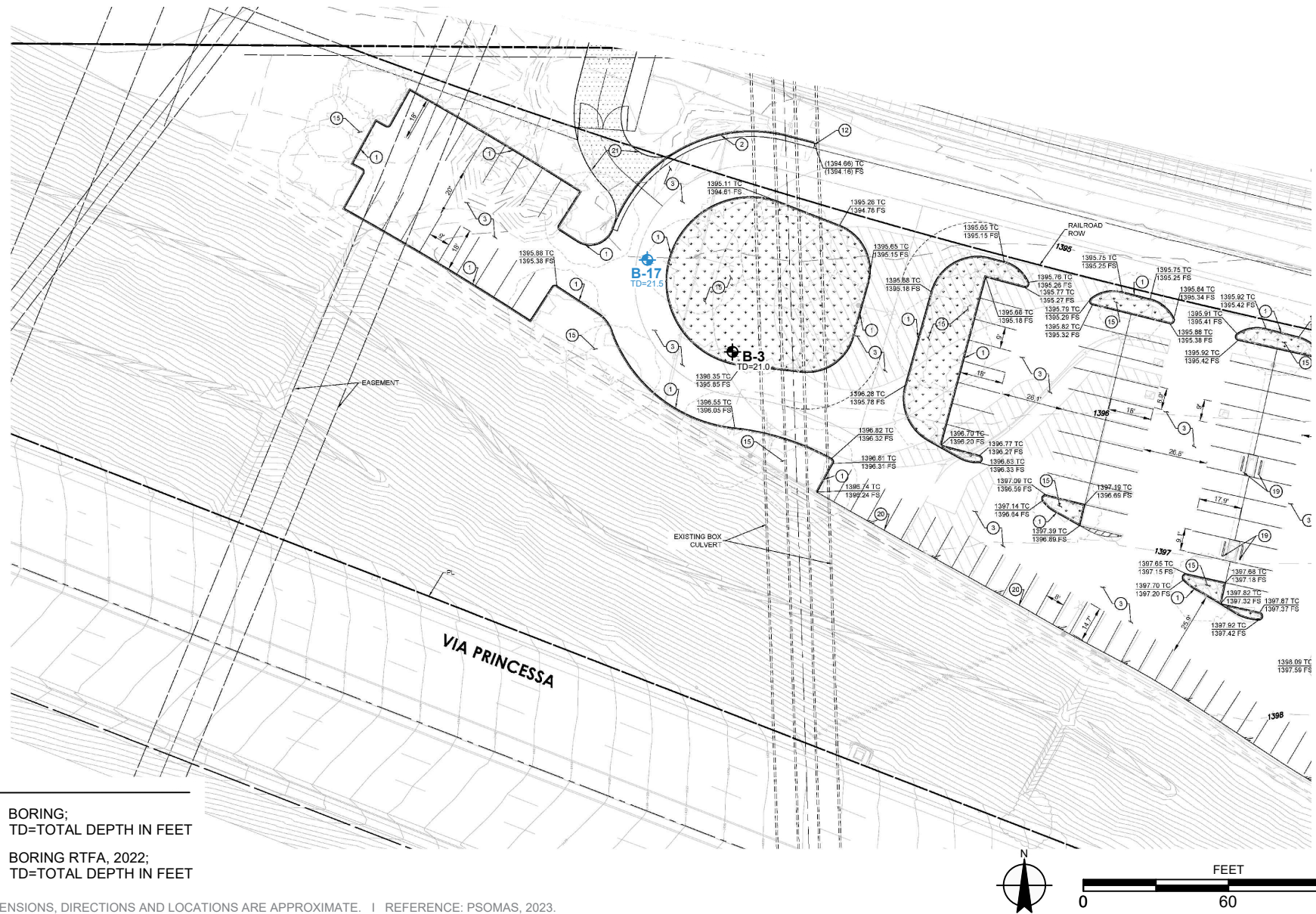
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LEGEND

B-17  BORING;
TD=101.5 TD=TOTAL DEPTH IN FEET

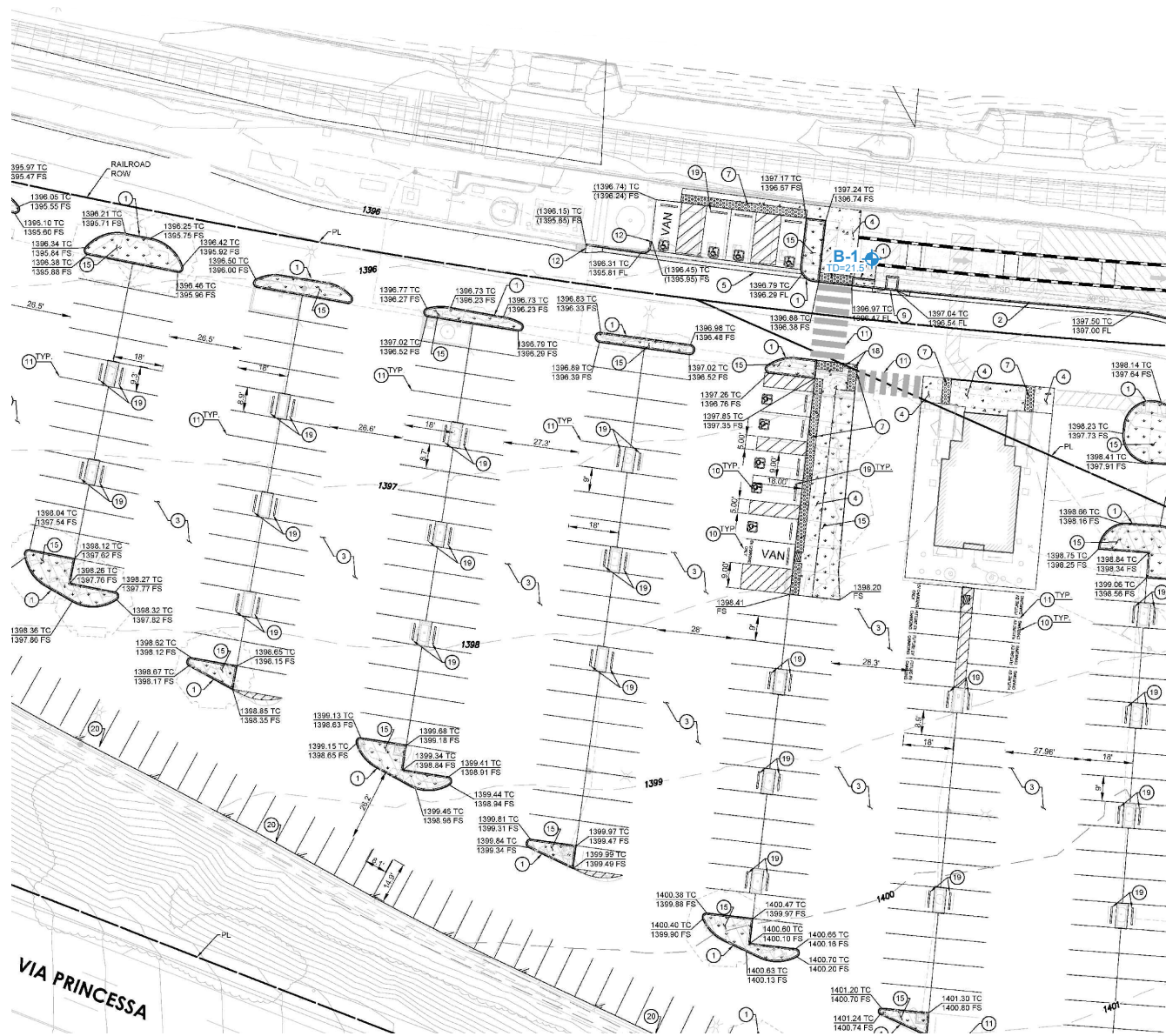
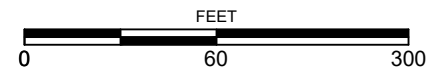
B-3  BORING RTFA, 2022;
TD=21.0 TD=TOTAL DEPTH IN FEET

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. | REFERENCE: PSOMAS, 2023.

**FIGURE 9****SITE PLAN AND EXPLORATION LOCATIONS**

VIA PRINCESSA PARK
SANTA CLARITA, CALIFORNIA

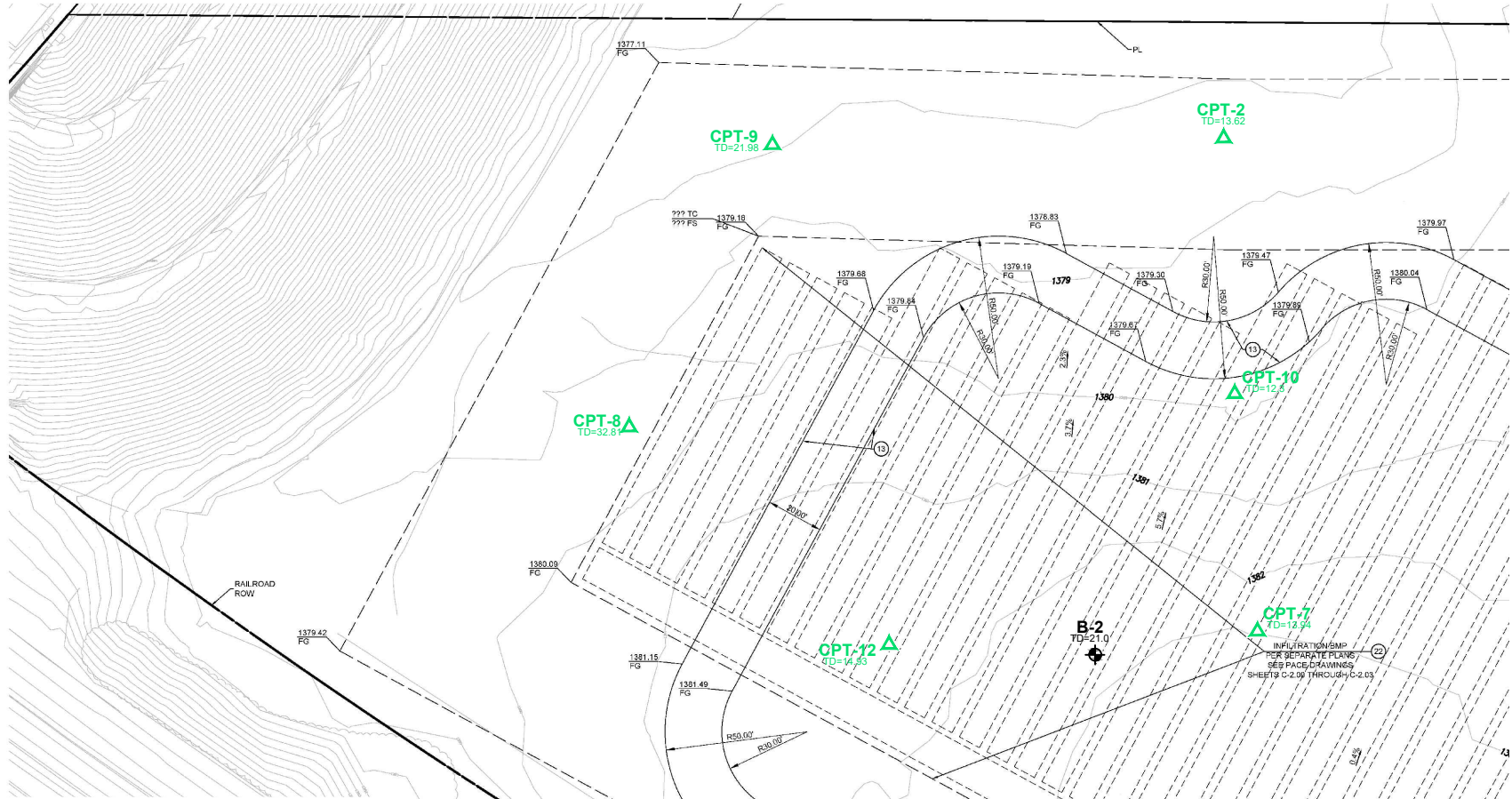
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**LEGEND**B-1
TD=21.5'BORING;
TD=TOTAL DEPTH IN FEET**FIGURE 10****SITE PLAN AND EXPLORATION LOCATIONS**

VIA PRINCESSA PARK
SANTA CLARITA, CALIFORNIA

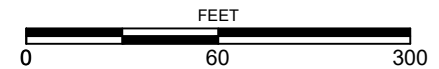
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**LEGEND**

- B-2**  BORING RTFA, 2022;
TD=TOTAL DEPTH IN FEET
- CPT-12**  CONE PENETROMETER TEST RTFA, 2019;
TD=TOTAL DEPTH IN FEET

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. | REFERENCE: PSOMAS, 2023.

**FIGURE 12****SITE PLAN AND EXPLORATION LOCATIONS**

VIA PRINCESSA PARK
SANTA CLARITA, CALIFORNIA

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LEGEND

CPT-4  CONE PENETROMETER TEST RTFA, 2019;
TD=40.52 TD=TOTAL DEPTH IN FEET

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. | REFERENCE: PSOMAS, 2023.

FIGURE 13

SITE PLAN AND EXPLORATION LOCATIONS

VIA PRINCESSA PARK
SANTA CLARITA, CALIFORNIA

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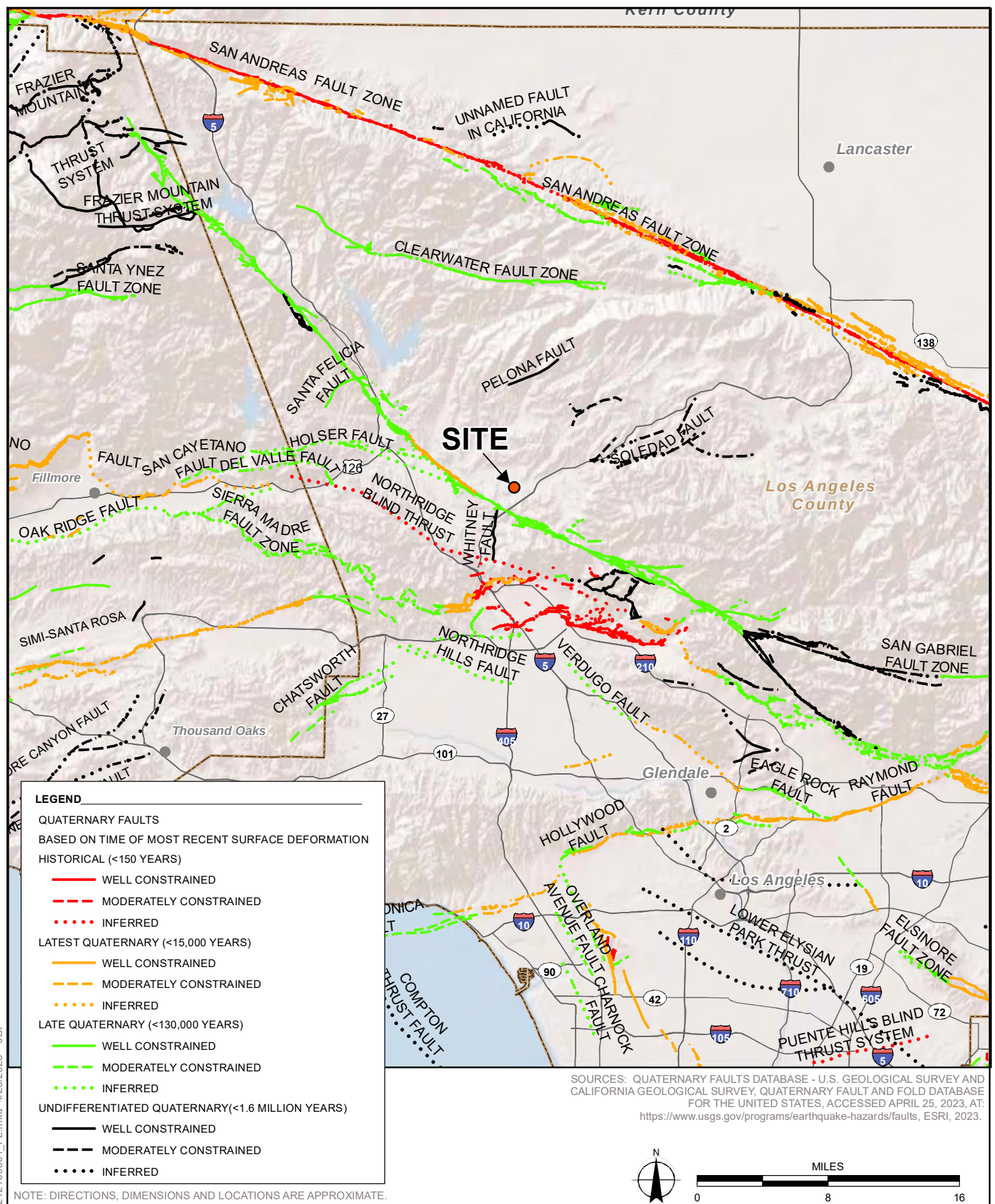


FIGURE 15

FAULT LOCATIONS

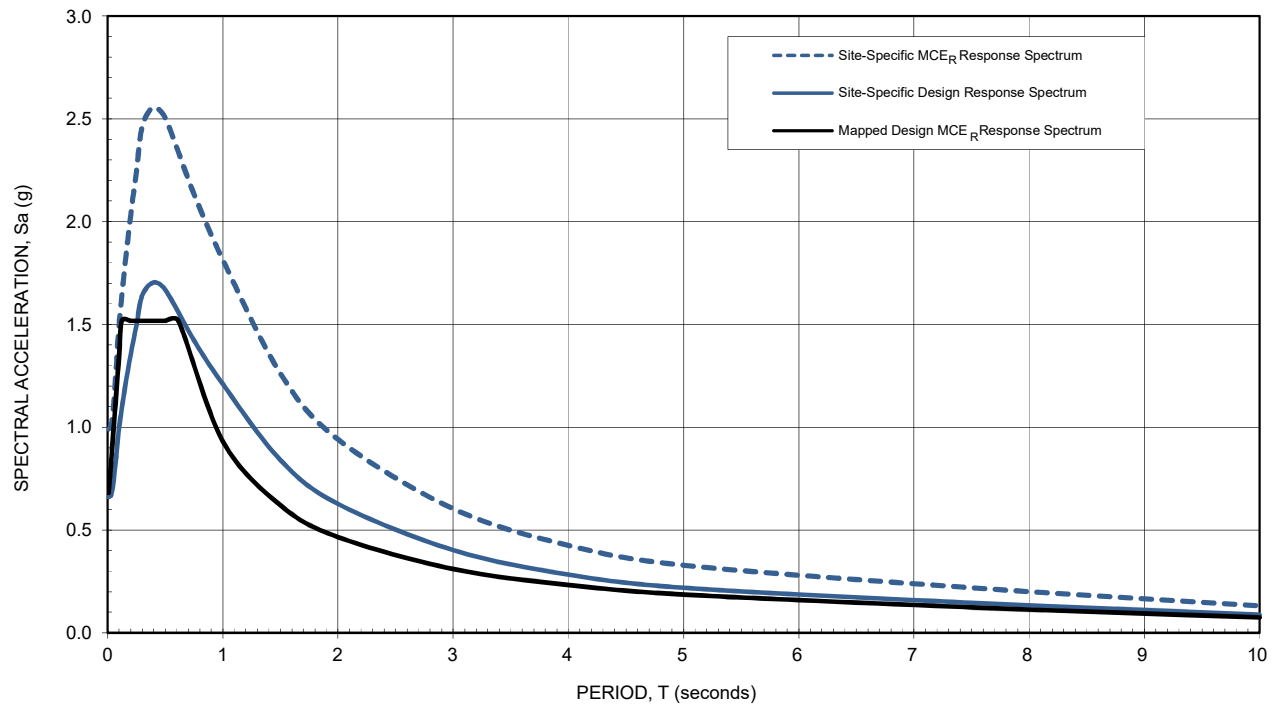
VIA PRINCESSA PARK
SANTA CLARITA, CALIFORNIA

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PERIOD (seconds)	SITE-SPECIFIC MCE _R RESPONSE SPECTRUM Sa (g)	SITE-SPECIFIC DESIGN RESPONSE SPECTRUM Sa (g)
0.010	0.992	0.661
0.020	0.997	0.664
0.030	1.003	0.669
0.050	1.098	0.732
0.075	1.295	0.864
0.100	1.501	1.001
0.150	1.786	1.191
0.200	2.031	1.354
0.250	2.245	1.497
0.300	2.462	1.641
0.400	2.557	1.704

PERIOD (seconds)	SITE-SPECIFIC MCE _R RESPONSE SPECTRUM Sa (g)	SITE-SPECIFIC DESIGN RESPONSE SPECTRUM Sa (g)
0.500	2.504	1.669
0.750	2.133	1.422
1.000	1.814	1.210
1.500	1.261	0.841
2.000	0.941	0.627
3.000	0.603	0.402
4.000	0.425	0.283
5.000	0.329	0.219
7.500	0.219	0.146
10.000	0.132	0.088

$S_{MS} = 2.301 \text{ g}$
 $S_{M1} = 1.891 \text{ g}$
 $S_{DS} = 1.534 \text{ g}$
 $S_{D1} = 1.261 \text{ g}$
 $PGA_M = 0.904 \text{ g}$



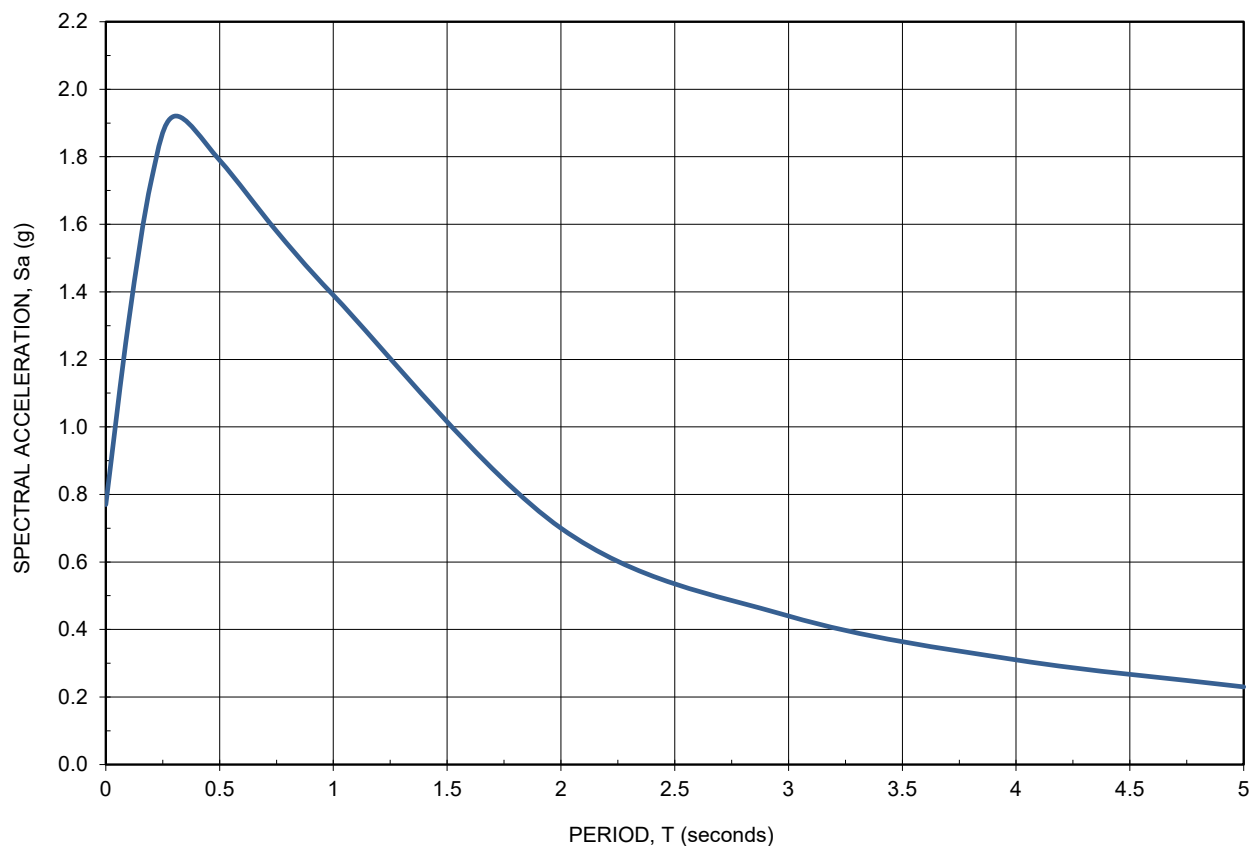
NOTES:

- 1 The probabilistic ground motion spectral response accelerations are based on the risk-targeted Maximum Considered Earthquake (MCE_R) having a 2% probability of exceedance in 50 years in the maximum direction using the Chiou & Youngs (2014), Campbell & Bozorgnia (2014), Boore et al. (2014), and Abrahamson et al. (2014) attenuation relationships and the risk coefficients per ASCE 7-16 Section 21.2.1.1.
- 2 The deterministic ground motion spectral response accelerations are the 84th percentile geometric mean values in the maximum direction using the Chiou & Youngs (2014), Campbell & Bozorgnia (2014), Boore et al. (2014), and Abrahamson et al. (2014) attenuation relationships for deep soil sites considering a Mw 7.0 event on the Santa Susana fault zone located 7.2 kilometers from the site and a Mw 7.5 event on the San Gabriel fault zone located 2.3 kilometers from the site. It conforms with the lower bound limit per ASCE 7-16 Section 21.2.2.
- 3 The Site-Specific MCE_R Response Spectrum is the lesser of the spectral ordinates of the deterministic and probabilistic accelerations at each period per ASCE 7-16 Section 21.2.3. The Site-Specific Design Response Spectrum conforms with the lower bound limit per ASCE 7-16 Section 21.3.
- 4 The Mapped Design MCE_R Response Spectrum is computed from the mapped spectral ordinates modified for Site Class D (stiff soil profile) per ASCE 7-16 Section 11.4. It is presented for the sake of comparison.

FIGURE 16

ACCELERATION RESPONSE SPECTRA

PERIOD (seconds)	DESIGN SPECTRAL ACCELERATION Sa (g)
PGA	0.77
0.10	1.31
0.20	1.73
0.30	1.92
0.50	1.79
0.75	1.58
1.00	1.39
2.00	0.70
3.00	0.44
4.00	0.31
5.00	0.23



NOTES:

- 1 SITE LATITUDE = 34.40879 DEGREES
SITE LONGITUDE = -118.46877 DEGREES
- 2 AVERAGE SHEAR WAVE VELOCITY IN THE UPPER 30 METERS (ASSUMED) = 280 METERS/SECOND
- 3 THE ACCELERATION RESPONSE SPECTRUM REPRESENTS THE 2014 USGS 5% IN 50 YEARS (975 YEAR RETURN PERIOD) HAZARD AT 5% DAMPING.
- 4 THE ACCELERATION RESPONSE SPECTRUM INCLUDES NEAR-FAULT EFFECTS AND BASIN AMPLIFICATION EFFECTS.
- 5 PEAK GROUND ACCELERATION (PGA) = 0.77g
- 6 MEAN MOMENT MAGNITUDE (M) = 6.78
- 7 SITE CLASS D

FIGURE 17

CALTRANS ACCELERATION RESPONSE SPECTRUM

PERIOD (seconds)	DESIGN SPECTRAL ACCELERATION (g)		
	Level 1 (100 yrs)	Level 2 (475 yrs)	Level 3 (2,475 yrs)
PGA	0.332	0.618	0.992
0.02	0.333	0.622	0.998
0.03	0.344	0.638	1.023
0.05	0.395	0.726	1.162
0.08	0.497	0.901	1.430
0.10	0.595	1.065	1.678
0.15	0.722	1.281	1.994
0.20	0.778	1.392	2.185
0.25	0.792	1.457	2.327
0.30	0.786	1.495	2.451
0.40	0.711	1.430	2.457
0.50	0.633	1.327	2.361
0.75	0.450	1.018	1.922
1.00	0.332	0.797	1.566
1.50	0.203	0.524	1.067
2.00	0.139	0.377	0.781
3.00	0.080	0.231	0.483
4.00	0.052	0.158	0.328
5.00	0.036	0.115	0.240

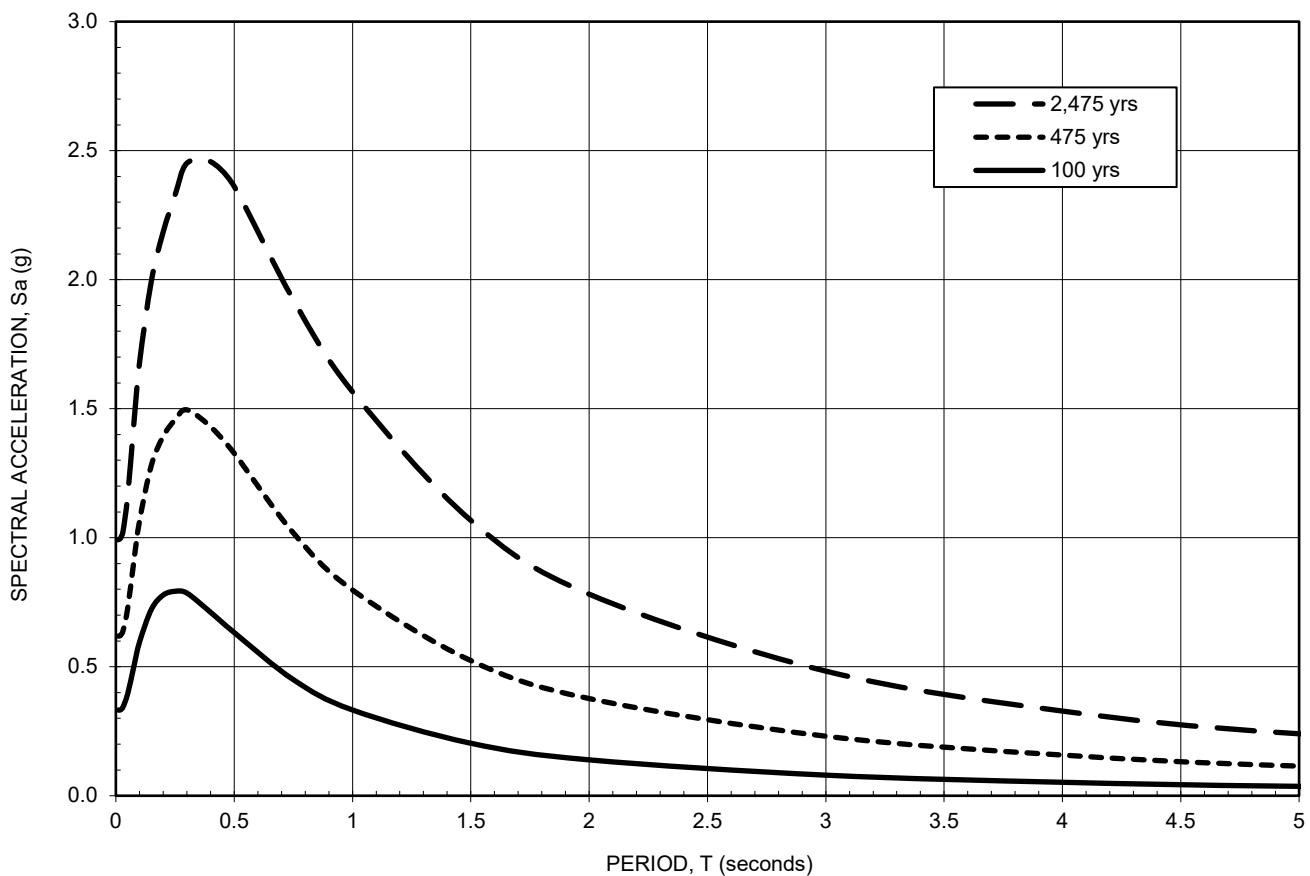
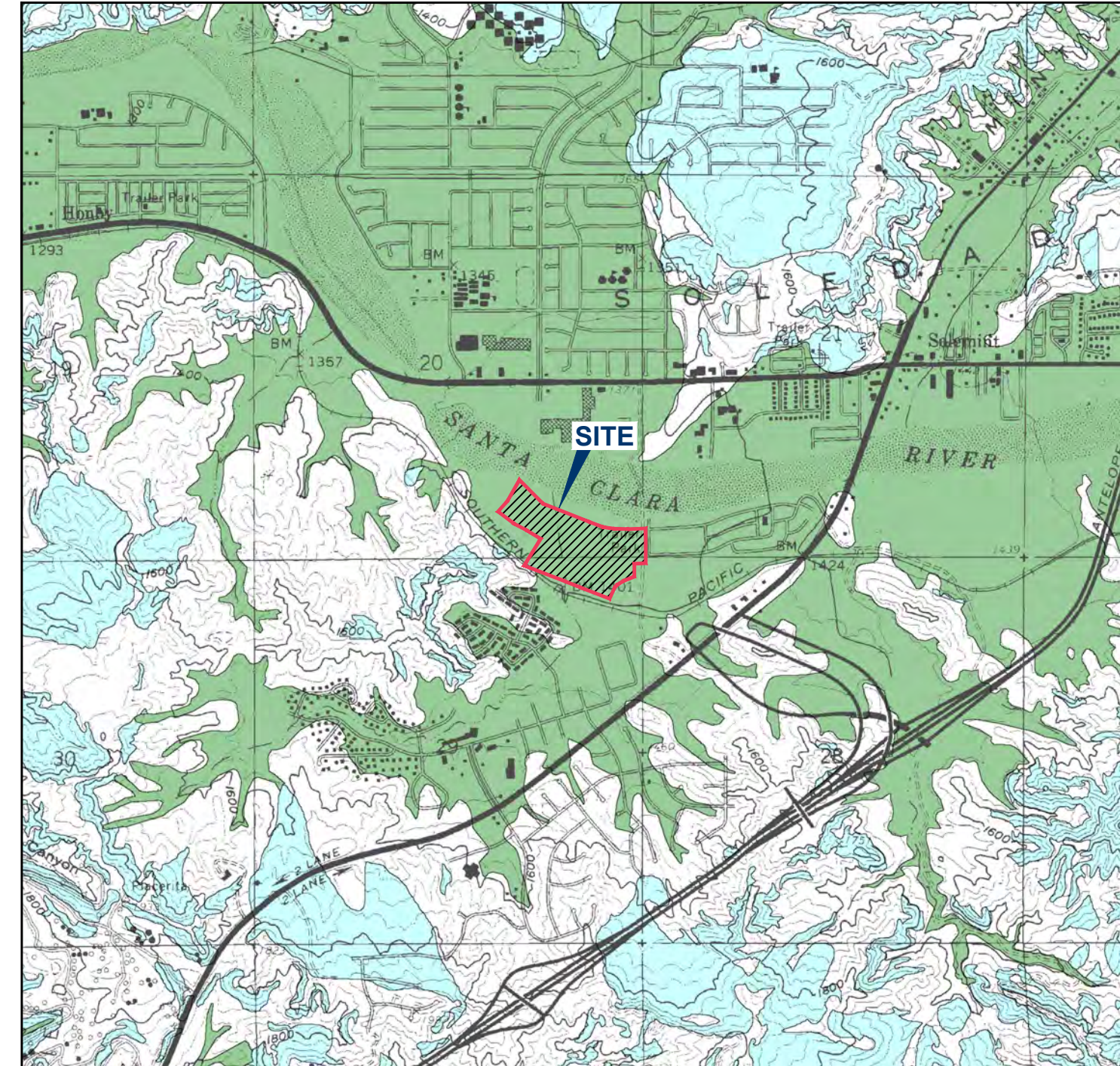


FIGURE 18

AREMA ACCELERATION RESPONSE SPECTRA

VIA PRINCESSA PARK
SANTA CLARITA, CALIFORNIA



LEGEND

EARTHQUAKE-INDUCED LANDSLIDES



Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.



LIQUEFACTION

Areas where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. | REFERENCE: CDMG, 1999.

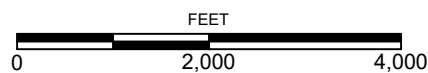
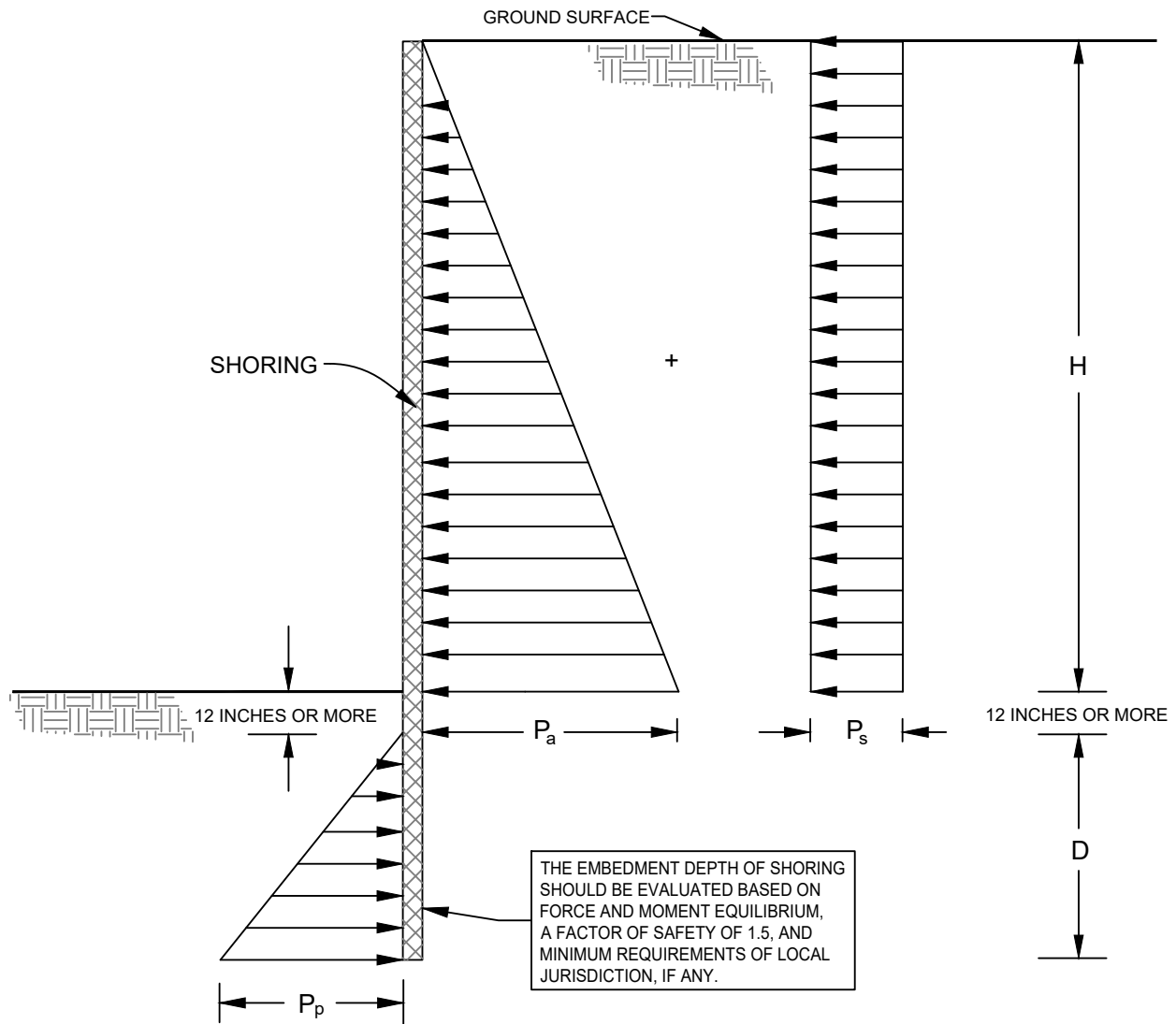


FIGURE 19

SEISMIC HAZARD ZONES

VIA PRINCESSA PARK
SANTA CLARITA, CALIFORNIA



NOTES:

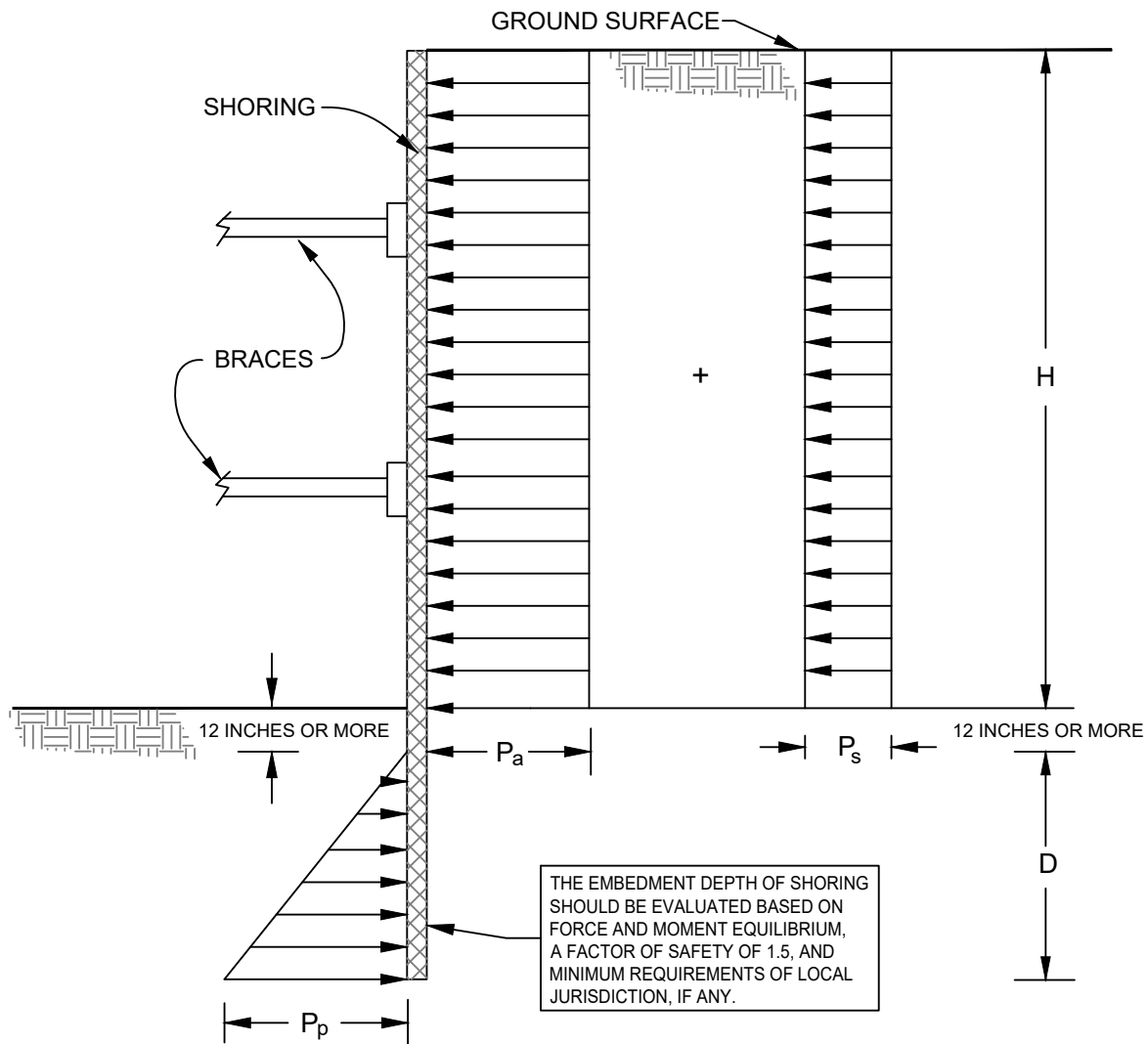
1. ACTIVE LATERAL EARTH PRESSURE, P_a
 $P_a = 40H$ psf
2. CONSTRUCTION TRAFFIC INDUCED SURCHARGE PRESSURE, P_s
 $P_s = 72$ psf
3. PASSIVE LATERAL EARTH PRESSURE, P_p
 $P_p = 350D$ psf
4. ASSUMES GROUNDWATER IS NOT PRESENT
5. SURCHARGES FROM RAILROAD, EXCAVATED SOIL, OR CONSTRUCTION MATERIALS ARE NOT INCLUDED AND SHOULD BE INCLUDED BY THE DESIGNER.
6. H AND D ARE IN FEET

NOT TO SCALE

FIGURE 20

**LATERAL EARTH PRESSURES FOR
TEMPORARY CANTILEVERED SHORING**

VIA PRINCESSA PARK
SANTA CLARITA, CALIFORNIA



NOTES:

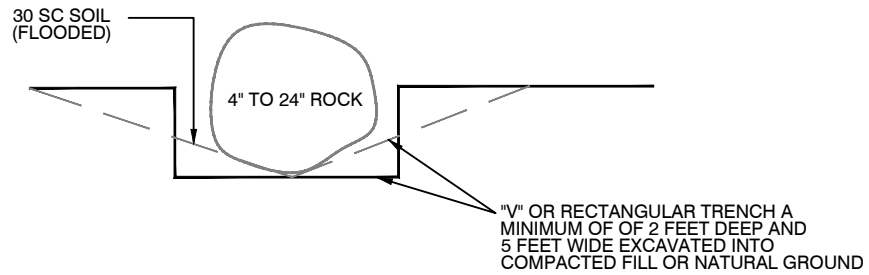
1. APPARENT LATERAL EARTH PRESSURE, P_a
 $P_a = 26H$ psf
2. CONSTRUCTION TRAFFIC INDUCED SURCHARGE PRESSURE, P_s
 $P_s = 120$ psf
3. PASSIVE LATERAL EARTH PRESSURE, P_p
 $P_p = 350D$ psf
4. ASSUMES GROUNDWATER IS NOT PRESENT.
5. SURCHARGES FROM RAILROAD, EXCAVATED SOIL, OR CONSTRUCTION MATERIALS ARE NOT INCLUDED AND SHOULD BE INCLUDED BY THE DESIGNER.
6. H AND D ARE IN FEET

NOT TO SCALE

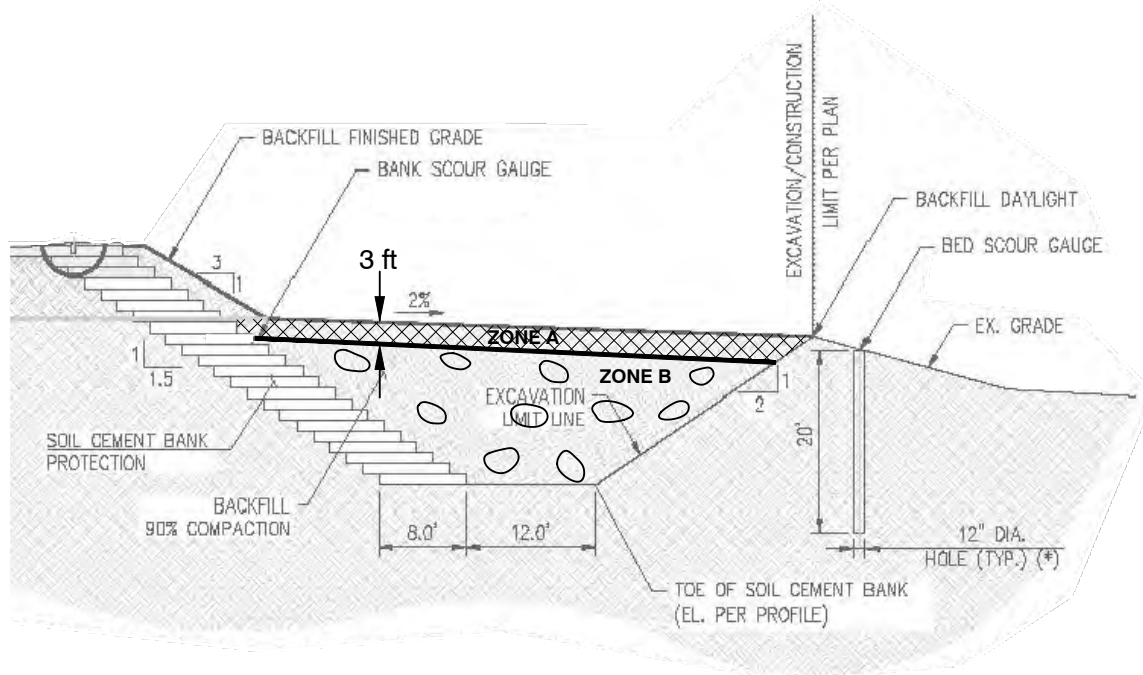
FIGURE 21

**LATERAL EARTH PRESSURES FOR
BRACED EXCAVATION (GRANULAR SOIL)**

VIA PRINCESSA PARK
SANTA CLARITA, CALIFORNIA



WINDROW SECTION



EMBANKMENT EXCAVATION ZONE

ZONE A: COMPACTED FILL WITH ROCK FRAGMENTS NO LARGER THAN 4 INCHES IN DIAMETER.

ZONE B: COMPACTED FILL WITH ROCK FRAGMENTS BETWEEN 4 AND 24 INCHES IN DIAMETER MAY BE PLACED IN STAGGERED WINDROWS SURROUNDED BY GRANULAR SOIL (30 SAND EQUIVALENT) DENSIFIED BY FLOODING AND TRACK-WALKING WITH A BULL DOZER. ROCK FRAGMENTS LESS THAN 4 INCHES IN DIAMETER MAY BE PLACED IN COMPACTED FILL SOIL.

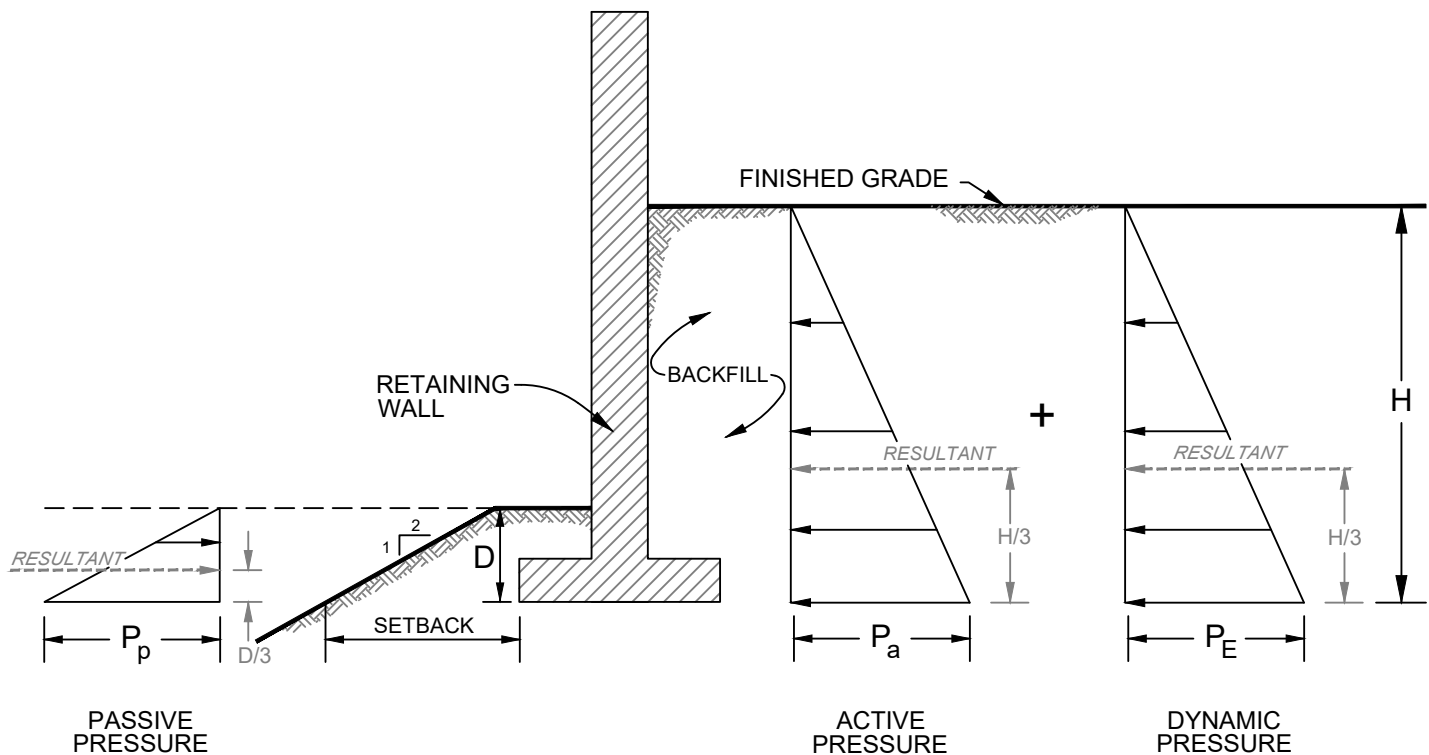
NOT TO SCALE

FIGURE 22

OVERSIZED ROCK PLACEMENT DETAIL

VIA PRINCESSA PARK
SANTA CLARITA, CALIFORNIA

212109001 | 6/23



NOTES:

1. ASSUMES NO HYDROSTATIC PRESSURE BUILD-UP BEHIND THE RETAINING WALL
2. STRUCTURAL, GRANULAR BACKFILL MATERIALS AS SPECIFIED IN GREENBOOK SHOULD BE USED FOR RETAINING WALL BACKFILL
3. DRAINS AS RECOMMENDED IN THE RETAINING WALL DRAINAGE DETAIL SHOULD BE INSTALLED BEHIND THE RETAINING WALL
4. P_E IS CALCULATED IN ACCORDANCE WITH THE RECOMMENDATIONS OF MONONOBE AND MATSUO (1929), AND ATIK AND SITAR (2010)
5. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED
6. H AND D ARE IN FEET
7. SETBACK SHOULD BE IN ACCORDANCE WITH FIGURE 1808.7.1 OF THE CBC (2022)

RECOMMENDED GEOTECHNICAL DESIGN PARAMETERS

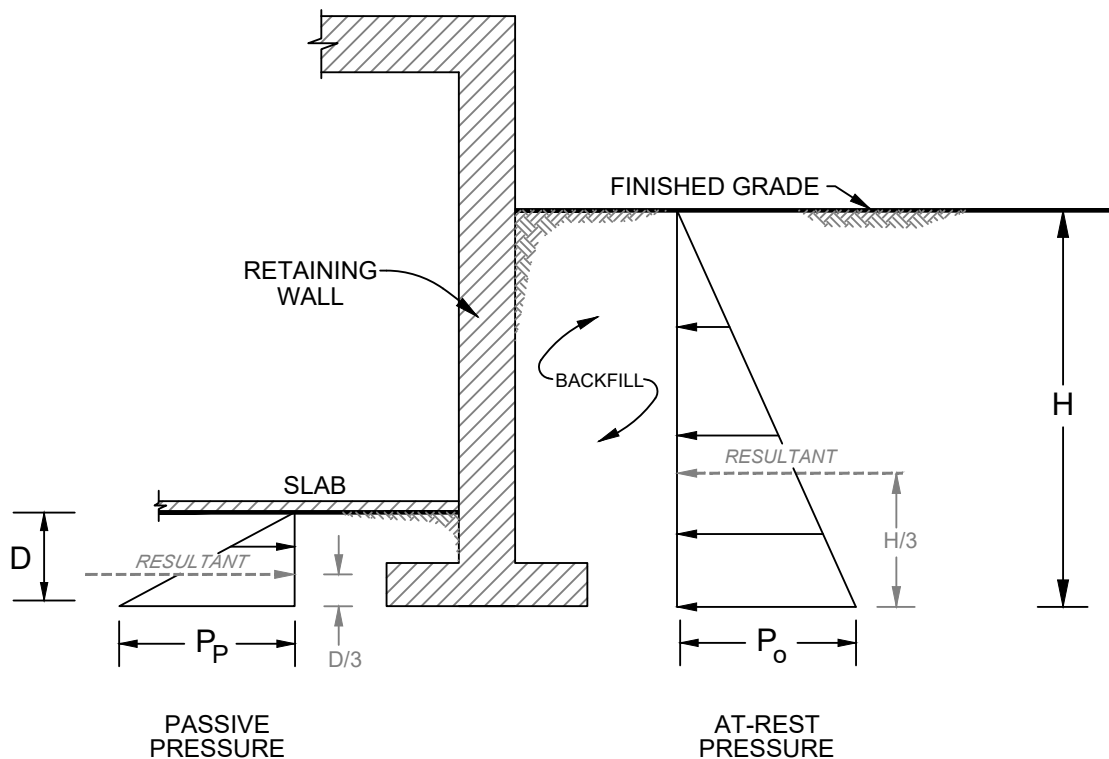
Lateral Earth Pressure	Equivalent Fluid Pressure ($\text{lb/ft}^2/\text{ft}^{(1)}$)	
P_a	Level Backfill with Granular Soils ⁽²⁾	2H:1V Sloping Backfill with Granular Soils ⁽²⁾
	37H	57H
P_E	13H (AREMA Level 1)	25H
	20H (AREMA Level 2 and Caltrans)	
P_p	Level Ground	2H:1V Descending Ground
	350D	140D

NOT TO SCALE

FIGURE 23

LATERAL EARTH PRESSURES FOR YIELDING RETAINING WALLS

VIA PRINCESSA PARK
SANTA CLARITA, CALIFORNIA



NOTES:

1. ASSUMES NO HYDROSTATIC PRESSURE BUILD-UP BEHIND THE RETAINING WALL
2. STRUCTURAL, GRANULAR BACKFILL MATERIALS AS SPECIFIED IN GREENBOOK SHOULD BE USED FOR RETAINING WALL BACKFILL
3. DRAINS AS RECOMMENDED IN THE RETAINING WALL DRAINAGE DETAIL SHOULD BE INSTALLED BEHIND THE RETAINING WALL
4. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED
5. H AND D ARE IN FEET

RECOMMENDED GEOTECHNICAL DESIGN PARAMETERS

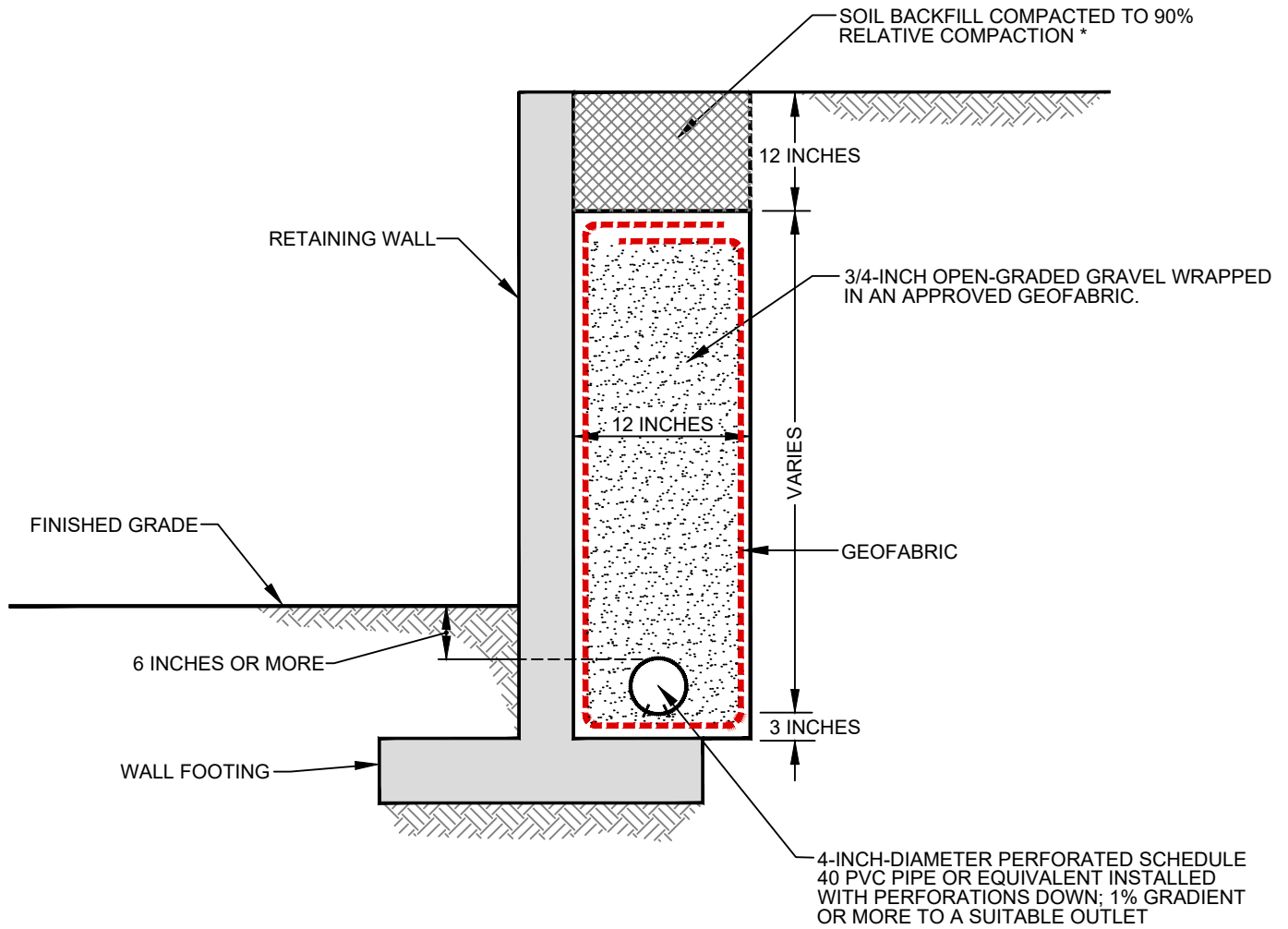
Lateral Earth Pressure	Equivalent Fluid Pressure ($\text{lb/ft}^2/\text{ft}^{(1)}$)	
P_o	Level Backfill with Granular Soils ⁽²⁾	2H:1V Sloping Backfill with Granular Soils ⁽²⁾
	57H	82H
P_p	Level Ground	2H:1V Descending Ground
	350D	140D

NOT TO SCALE

FIGURE 24

LATERAL EARTH PRESSURES FOR RESTRAINED RETAINING WALLS

VIA PRINCESSA PARK
SANTA CLARITA, CALIFORNIA



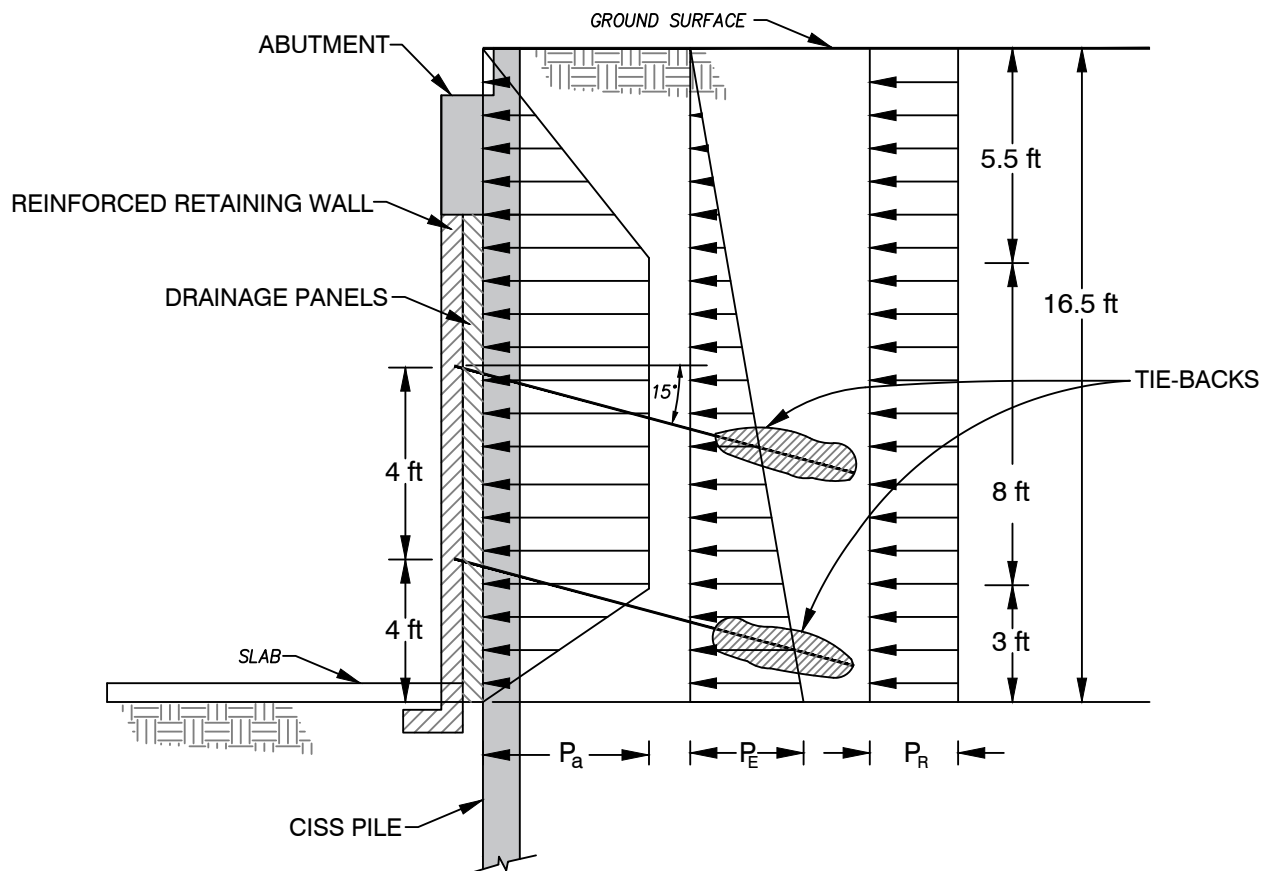
*BASED ON ASTM D1557

NOT TO SCALE

FIGURE 25

RETAINING WALL DRAINAGE DETAIL

VIA PRINCESSA PARK
SANTA CLARITA, CALIFORNIA



NOTES:

1. APPARENT LATERAL EARTH PRESSURE, P_a
 $P_a = 570 \text{ psf}$
2. TRAIN INDUCED SURCHARGE PRESSURE, P_R ,
TO BE CALCULATED BY DESIGNER
3. DYNAMIC LATERAL EARTH PRESSURE, P_E
 $P_E = 320 \text{ psf}$
4. ASSUMES GROUNDWATER IS NOT PRESENT

NOT TO SCALE

FIGURE 26

LATERAL EARTH PRESSURES FOR TIEBACK WALLS



APPENDIX A

Boring Logs

APPENDIX A

BORING LOGS

Field Procedure for the Collection of Disturbed Samples

Disturbed soil samples were obtained in the field using the following method.

Bulk Samples

Bulk samples of representative earth materials were obtained from the exploratory borings and test pit. The samples were bagged and transported to the laboratory for testing.

Field Procedure for the Collection of Relatively Undisturbed Samples

Relatively undisturbed soil samples were obtained in the field using the following method.

The Modified Split-Barrel Drive Sampler

The sampler, with an external diameter of 3 inches, was lined with 1-inch-long, thin brass rings with inside diameters of approximately 2.4 inches. The sample barrel was driven into the ground with the weight of a hammer of the drill rig in general accordance with ASTM D 3550. The driving weight was permitted to fall freely. The approximate length of the fall, the weight of the hammer, and the number of blows per foot of driving are presented on the boring logs as an index to the relative resistance of the materials sampled. The samples were removed from the sample barrel in the brass rings, sealed, and transported to the laboratory for testing.

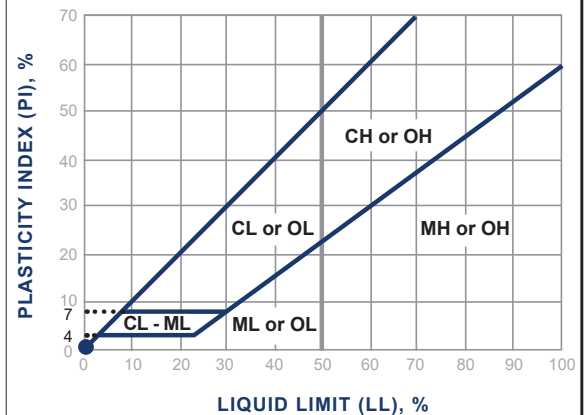
Soil Classification Chart Per ASTM D 2488

Primary Divisions			Secondary Divisions			
			Group Symbol	Group Name		
COARSE-GRAINED SOILS more than 50% retained on No. 200 sieve	GRAVEL more than 50% of coarse fraction retained on No. 4 sieve	CLEAN GRAVEL less than 5% fines		GW	well-graded GRAVEL	
				GP	poorly graded GRAVEL	
		GRAVEL with DUAL CLASSIFICATIONS 5% to 12% fines		GW-GM	well-graded GRAVEL with silt	
				GP-GM	poorly graded GRAVEL with silt	
				GW-GC	well-graded GRAVEL with clay	
				GP-GC	poorly graded GRAVEL with clay	
		GRAVEL with FINES more than 12% fines		GM	silty GRAVEL	
				GC	clayey GRAVEL	
				GC-GM	silty, clayey GRAVEL	
	SAND 50% or more of coarse fraction passes No. 4 sieve	CLEAN SAND less than 5% fines		SW	well-graded SAND	
				SP	poorly graded SAND	
		SAND with DUAL CLASSIFICATIONS 5% to 12% fines		SW-SM	well-graded SAND with silt	
				SP-SM	poorly graded SAND with silt	
				SW-SC	well-graded SAND with clay	
				SP-SC	poorly graded SAND with clay	
		SAND with FINES more than 12% fines		SM	silty SAND	
				SC	clayey SAND	
				SC-SM	silty, clayey SAND	
	FINE-GRAINED SOILS 50% or more passes No. 200 sieve	SILT and CLAY liquid limit less than 50%	INORGANIC		CL	lean CLAY
					ML	SILT
					CL-ML	silty CLAY
			ORGANIC		OL (PI > 4)	organic CLAY
					OL (PI < 4)	organic SILT
			SILT and CLAY liquid limit 50% or more	INORGANIC		CH
					MH	elastic SILT
		ORGANIC			OH (plots on or above "A"-line)	organic CLAY
					OH (plots below "A"-line)	organic SILT
		Highly Organic Soils			PT	Peat

Grain Size

Description		Sieve Size	Grain Size	Approximate Size
Boulders		> 12"	> 12"	Larger than basketball-sized
Cobbles		3 - 12"	3 - 12"	Fist-sized to basketball-sized
Gravel	Coarse	3/4 - 3"	3/4 - 3"	Thumb-sized to fist-sized
	Fine	#4 - 3/4"	0.19 - 0.75"	Pea-sized to thumb-sized
Sand	Coarse	#10 - #4	0.075 - 0.19"	Rock-salt-sized to pea-sized
	Medium	#40 - #10	0.017 - 0.075"	Sugar-sized to rock-salt-sized
	Fine	#200 - #40	0.0029 - 0.017"	Flour-sized to sugar-sized
Fines		Passing #200	< 0.0029"	Flour-sized and smaller

Plasticity Chart



Apparent Density - Coarse-Grained Soil

Apparent Density	Spooling Cable or Cathead		Automatic Trip Hammer	
	SPT (blows/foot)	Modified Split Barrel (blows/foot)	SPT (blows/foot)	Modified Split Barrel (blows/foot)
Very Loose	≤ 4	≤ 8	≤ 3	≤ 5
Loose	5 - 10	9 - 21	4 - 7	6 - 14
Medium Dense	11 - 30	22 - 63	8 - 20	15 - 42
Dense	31 - 50	64 - 105	21 - 33	43 - 70
Very Dense	> 50	> 105	> 33	> 70

Consistency - Fine-Grained Soil

Consistency	Spooling Cable or Cathead		Automatic Trip Hammer	
	SPT (blows/foot)	Modified Split Barrel (blows/foot)	SPT (blows/foot)	Modified Split Barrel (blows/foot)
Very Soft	< 2	< 3	< 1	< 2
Soft	2 - 4	3 - 5	1 - 3	2 - 3
Firm	5 - 8	6 - 10	4 - 5	4 - 6
Stiff	9 - 15	11 - 20	6 - 10	7 - 13
Very Stiff	16 - 30	21 - 39	11 - 20	14 - 26
Hard	> 30	> 39	> 20	> 26

BORING LOG EXPLANATION SHEET

DEPTH (feet)	Bulk Driven SAMPLES	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	
0							Bulk sample.
							Modified split-barrel drive sampler.
							No recovery with modified split-barrel drive sampler.
							Sample retained by others.
							Standard Penetration Test (SPT).
5							No recovery with a SPT.
	XX/XX						Shelby tube sample. Distance pushed in inches/length of sample recovered in inches.
							No recovery with Shelby tube sampler.
							Continuous Push Sample.
10							Seepage.
							Groundwater encountered during drilling.
							Groundwater measured after drilling.
						SM	MAJOR MATERIAL TYPE (SOIL):
							Solid line denotes unit change.
						CL	Dashed line denotes material change.
15							Attitudes: Strike/Dip b: Bedding c: Contact j: Joint f: Fracture F: Fault cs: Clay Seam s: Shear bss: Basal Slide Surface sf: Shear Fracture sz: Shear Zone sbs: Shear Bedding Surface
20							The total depth line is a solid line that is drawn at the bottom of the boring.

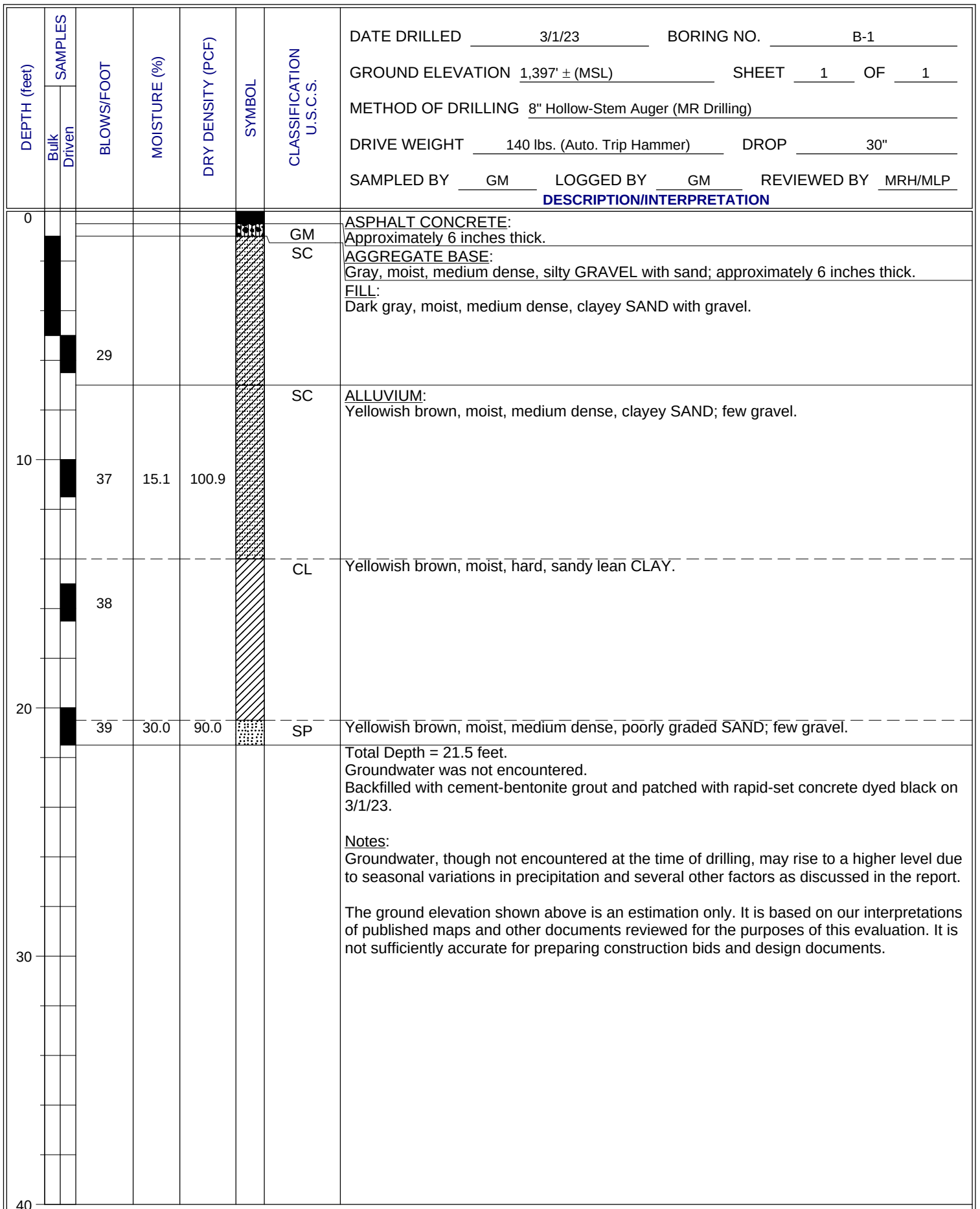


FIGURE A- 1

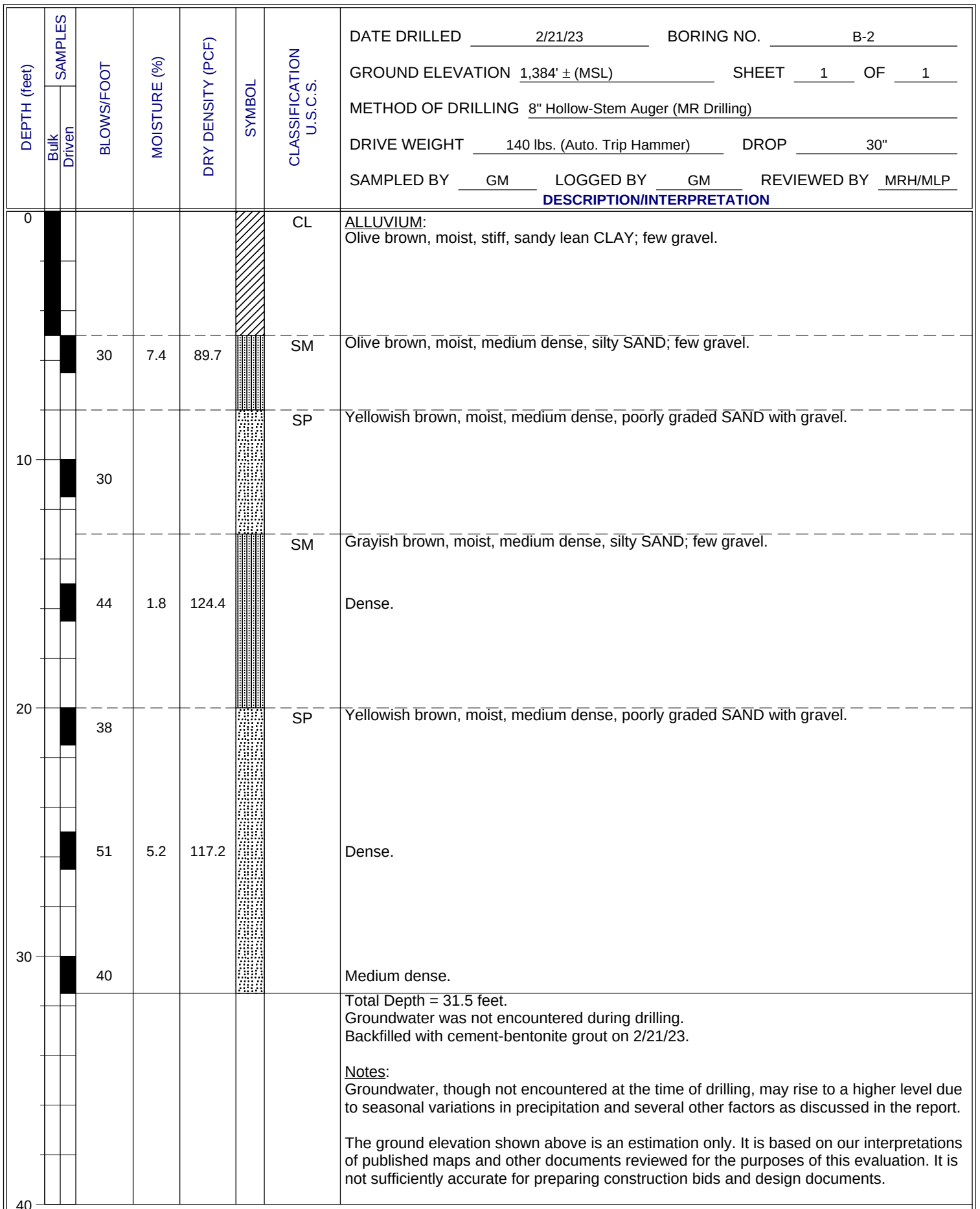


FIGURE A- 2

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 2/21/23 BORING NO. B-3	
	Bulk	Driven						GROUND ELEVATION 1,386' ± (MSL) SHEET 1 OF 1	METHOD OF DRILLING 8" Hollow-Stem Auger (MR Drilling)
								DRIVE WEIGHT 140 lbs. (Auto. Trip Hammer) DROP 30"	
								SAMPLED BY GM LOGGED BY GM REVIEWED BY MRH/MLP	
DESCRIPTION/INTERPRETATION									
0							ML	ALLUVIUM: Olive brown, moist, medium dense, sandy SILT.	
			30	9.3	99.9		SP	Yellowish brown, moist, medium dense, poorly graded SAND with gravel.	
								@ 8': Difficult drilling; possible cobble.	
10			31						
							SP-SM	Yellowish brown, moist, medium dense, SAND with silt.	
			33	3.4	115.5				
20			38						
			42	3.5	118.3				
30			36						
								Total Depth = 31.5 feet. Groundwater was not encountered during drilling. Backfilled with cement-bentonite grout on 2/21/23.	
								<u>Notes:</u> Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.	
								The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.	
40									

FIGURE A- 3

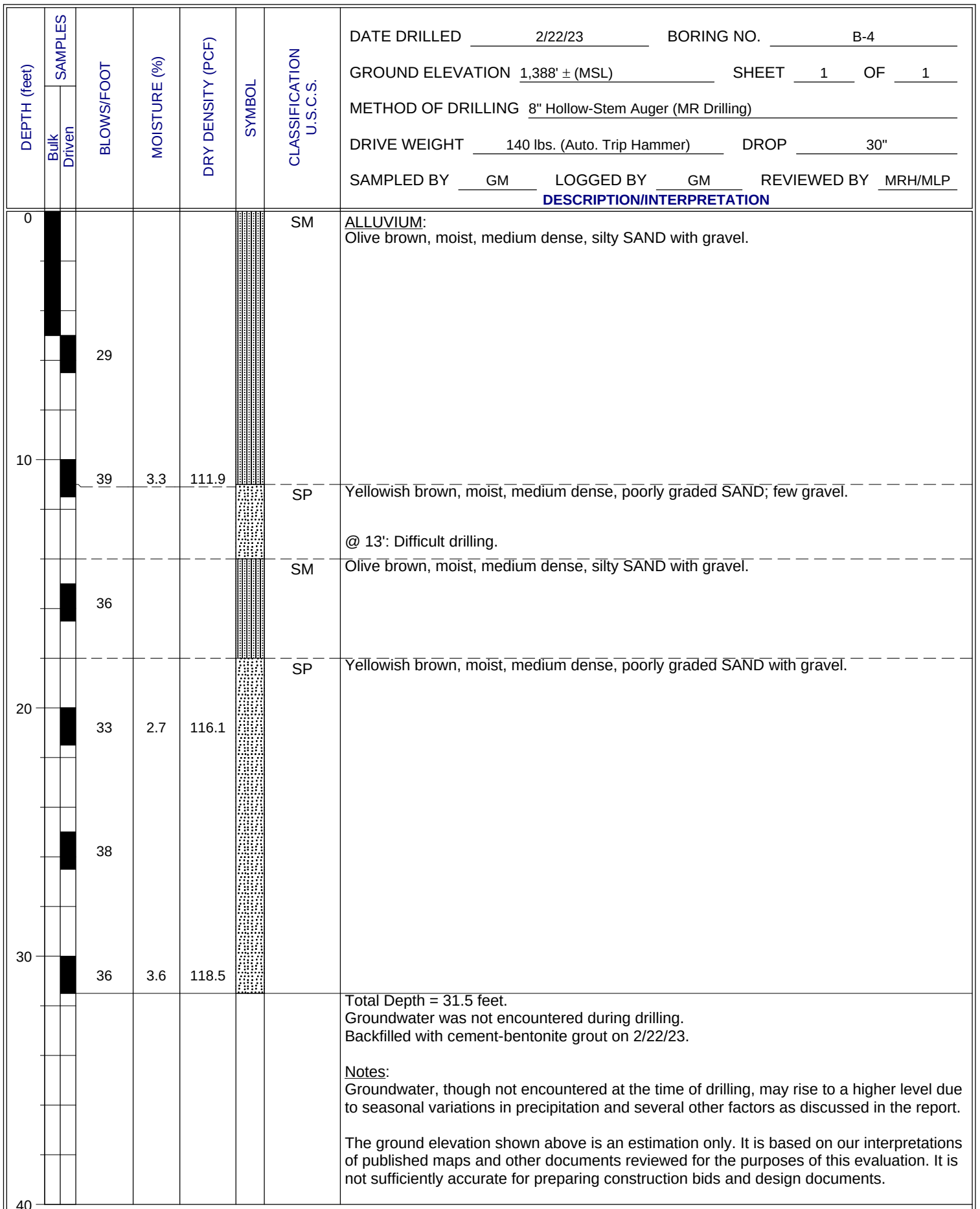


FIGURE A- 4

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 2/22/23 BORING NO. B-5	
	Bulk	Driven						GROUND ELEVATION 1,390' ± (MSL)	SHEET 1 OF 1
METHOD OF DRILLING 8" Hollow-Stem Auger (MR Drilling)									
DRIVE WEIGHT 140 lbs. (Auto. Trip Hammer) DROP 30"									
SAMPLED BY GM LOGGED BY GM REVIEWED BY MRH/MLP									
DESCRIPTION/INTERPRETATION									
0							SM	ALLUVIUM: Olive brown, moist, medium dense, silty SAND with gravel.	
			32	4.0	111.6				
10			36				SP	Olive brown, moist, medium dense, poorly graded SAND with gravel.	
								@ 12': Rig chatter; possible cobble.	
			39	6.0	119.4				
20			33				SM	Olive brown, moist, medium dense, silty SAND with gravel.	
							SP	Yellowish brown, moist, dense, poorly graded SAND with gravel.	
			43	4.3	121.7				
30			39					Medium dense.	
								Total Depth = 31.5 feet. Groundwater was not encountered during drilling. Backfilled with cement-bentonite grout on 2/22/23.	
								Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.	
								The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.	
40									

FIGURE A- 5

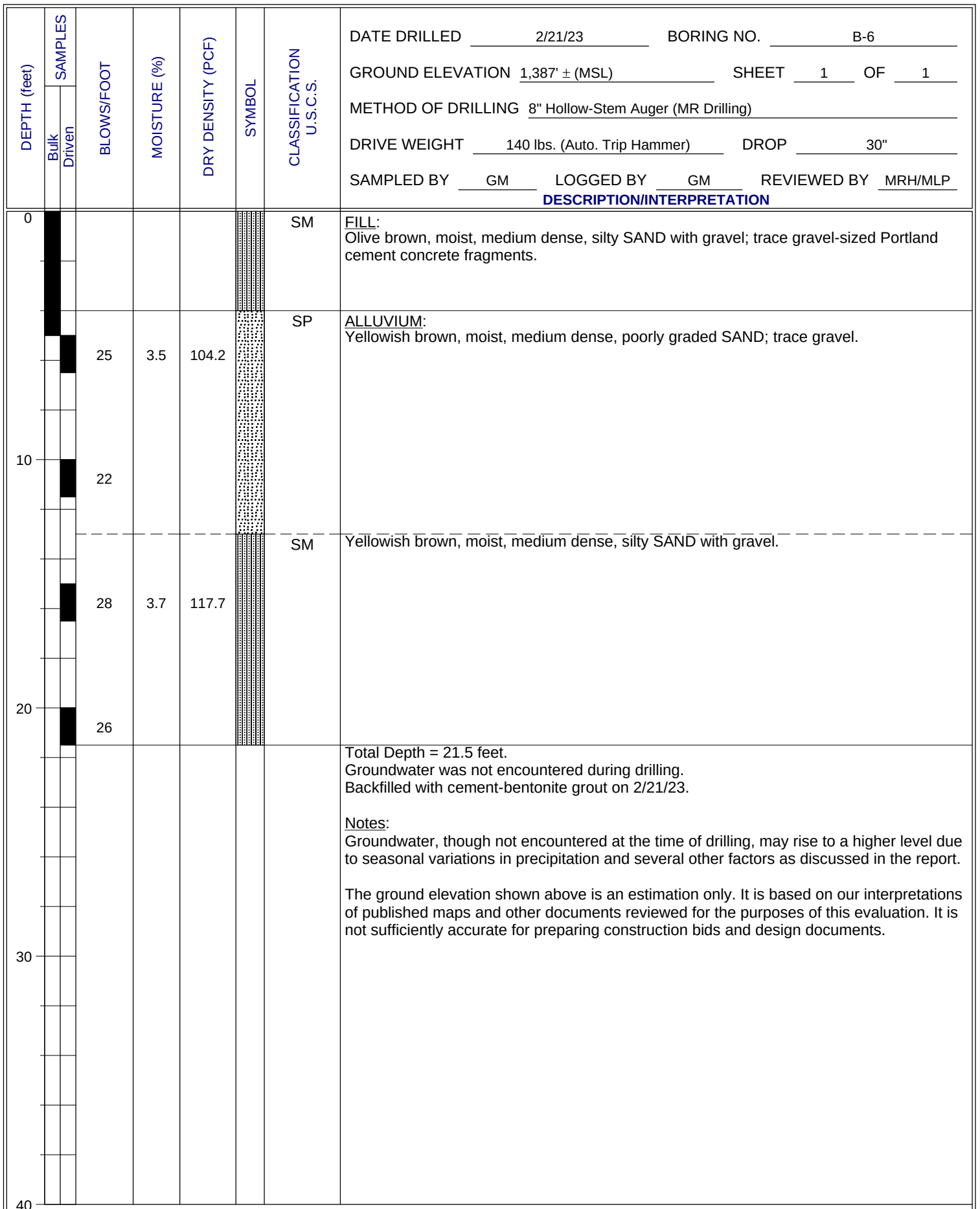


FIGURE A- 6

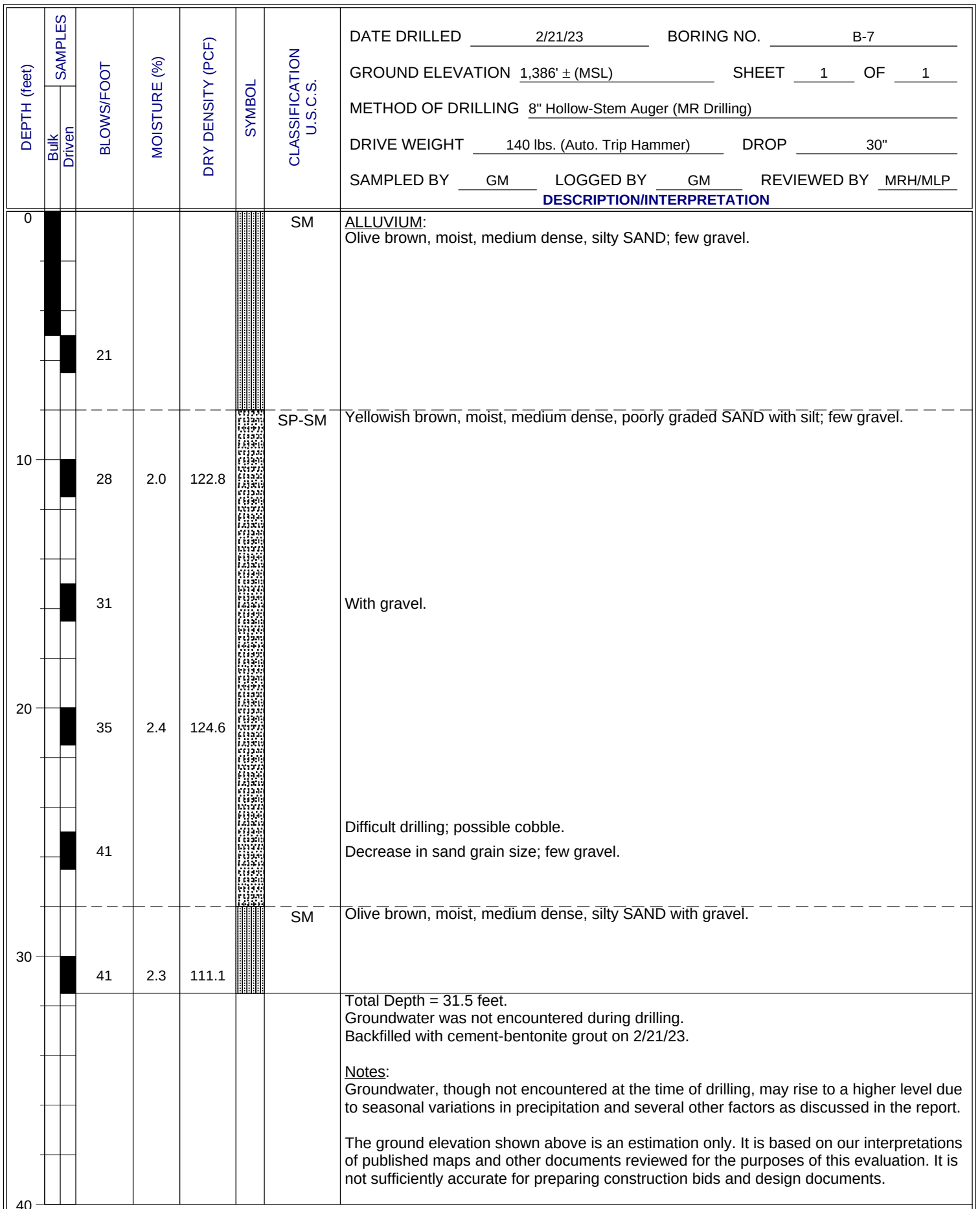


FIGURE A- 7

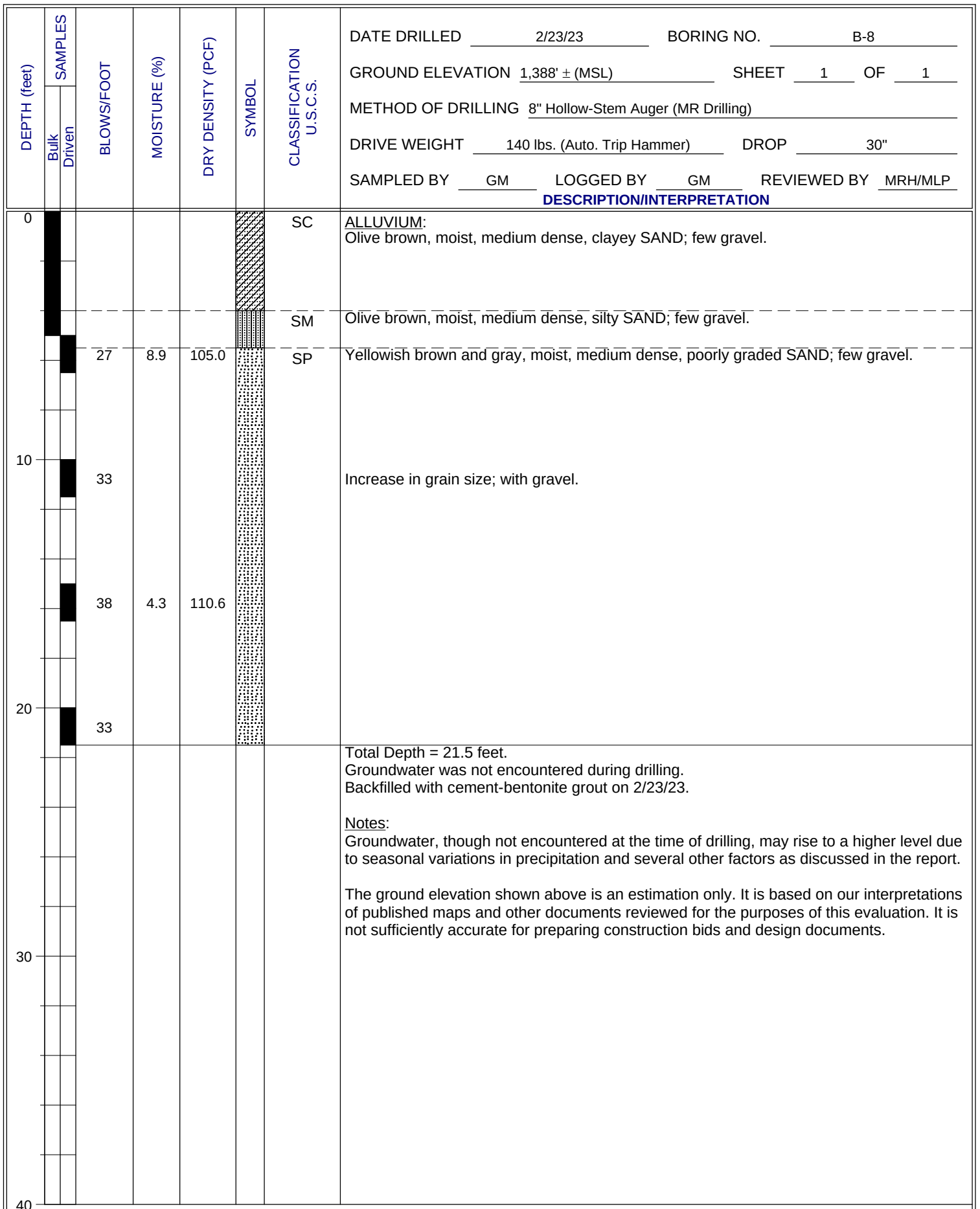


FIGURE A- 8

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 2/22/23 BORING NO. B-9	
	Bulk	Driven						GROUND ELEVATION 1,390' ± (MSL)	SHEET 1 OF 1
METHOD OF DRILLING 8" Hollow-Stem Auger (MR Drilling)									
DRIVE WEIGHT 140 lbs. (Auto. Trip Hammer) DROP 30"									
SAMPLED BY GM LOGGED BY GM REVIEWED BY MRH/MLP									
DESCRIPTION/INTERPRETATION									
0							SM	ALLUVIUM: Olive brown, moist, medium dense, silty SAND; few gravel.	
			30				SP	Yellowish brown, moist, medium dense, poorly graded SAND; few gravel.	
							SP-SM	Olive brown, moist, medium dense, poorly graded SAND with silt and gravel.	
10			32	3.4	112.2				
							SP	Grayish brown, moist, medium dense, poorly graded SAND with gravel.	
			34						
20			33	3.9	116.0				
								Total Depth = 21.5 feet. Groundwater was not encountered during drilling. Backfilled with cement-bentonite grout on 2/22/23.	
								Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.	
								The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.	
30									
40									

FIGURE A- 9

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 2/22/23 BORING NO. B-10	
	Bulk	Driven						GROUND ELEVATION 1,390' ± (MSL) SHEET 1 OF 1	METHOD OF DRILLING 8" Hollow-Stem Auger (MR Drilling)
								DRIVE WEIGHT 140 lbs. (Auto. Trip Hammer) DROP 30"	
								SAMPLED BY GM LOGGED BY GM REVIEWED BY MRH/MLP	
DESCRIPTION/INTERPRETATION									
0							SM	ALLUVIUM: Olive brown, moist, medium dense, silty SAND; few gravel.	
							SP	Yellowish brown, moist, medium dense, poorly graded SAND with gravel.	
			34	3.2	113.2				
							SM	Yellowish brown, moist, medium dense, silty SAND with gravel.	
10			34						
							SP	Yellowish brown, moist, medium dense, poorly graded SAND with gravel.	
			40	6.4	116.4		SM	Yellowish brown, moist, medium dense, silty SAND with gravel.	
20			49				SP	Yellowish brown, moist, dense, poorly graded SAND with gravel.	
			38	5.1	115.6			Medium dense.	
30			35						
								Total Depth = 31.5 feet. Groundwater was not encountered during drilling. Backfilled with cement-bentonite grout on 2/22/23.	
								Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.	
								The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.	
40									

FIGURE A- 10

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 3/1/23 BORING NO. B-11	
	Bulk	Driven						GROUND ELEVATION 1,390' ± (MSL)	SHEET 1 OF 1
METHOD OF DRILLING 8" Hollow-Stem Auger (MR Drilling)									
DRIVE WEIGHT 140 lbs. (Auto. Trip Hammer) DROP 30"									
SAMPLED BY GM LOGGED BY GM REVIEWED BY MRH/MLP									
DESCRIPTION/INTERPRETATION									
0							SM	ALLUVIUM: Yellowish brown, moist, medium dense, silty SAND with gravel.	
			34	4.5	97.9				
							SP	Yellowish brown, moist, medium dense, poorly graded SAND; few gravel.	
10			39						
							SM	Yellowish brown, moist, medium dense, silty SAND; few gravel.	
			43	3.3	103.9		SP	Yellowish brown, moist, dense, poorly graded SAND with gravel.	
20			44						
							SM	Yellowish brown, moist, dense, silty SAND with gravel. Dense.	
			51	3.3	121.6		SP	Yellowish brown, moist, dense, poorly graded SAND with gravel.	
30			50						
								Total Depth = 31.5 feet. Groundwater was not encountered during drilling. Backfilled with cement-bentonite grout on 3/1/23.	
								Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.	
								The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.	
40									

FIGURE A- 11

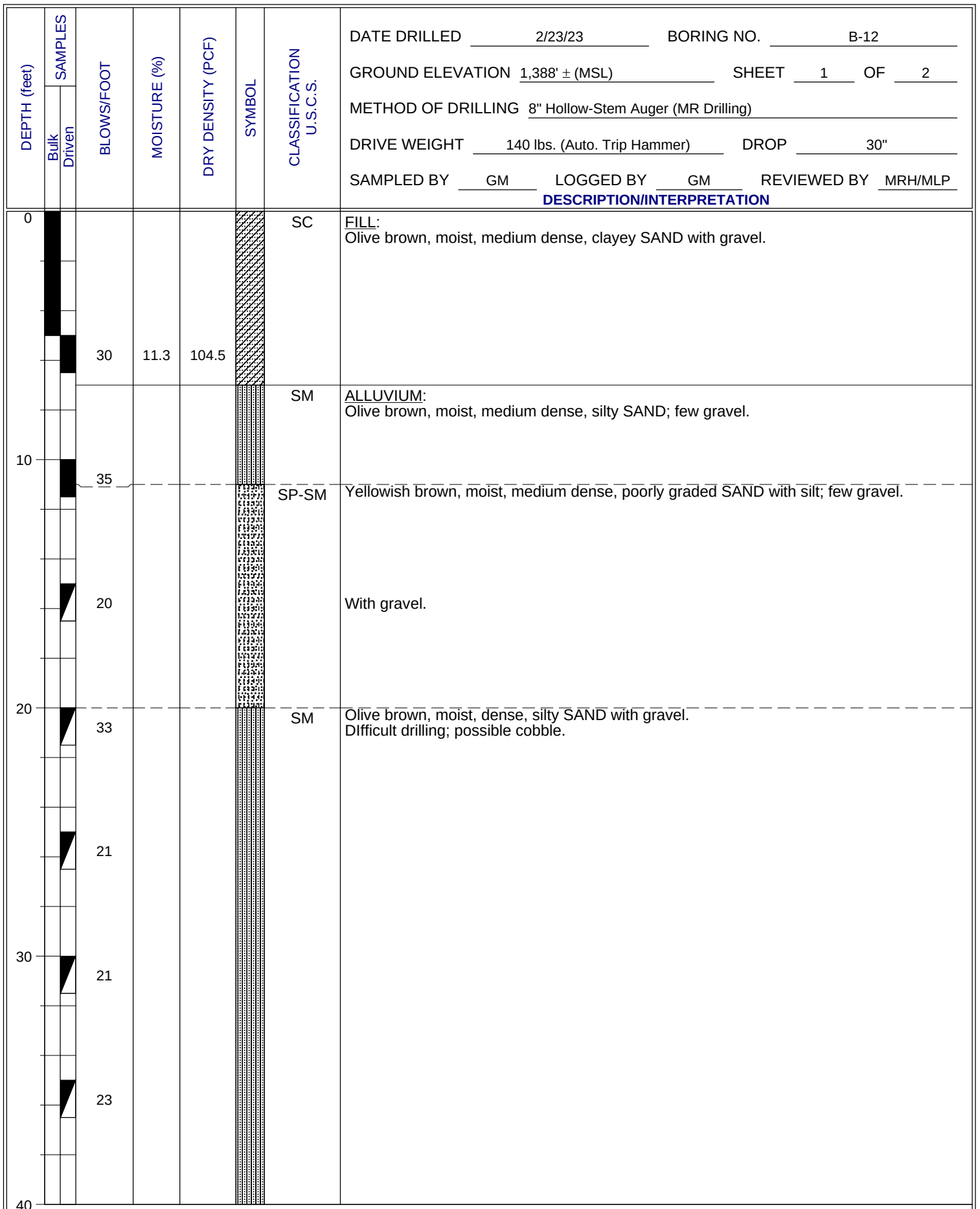


FIGURE A- 12

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 2/23/23 BORING NO. B-12	
	Bulk	Driven						GROUND ELEVATION 1,388' ± (MSL)	SHEET 2 OF 2
METHOD OF DRILLING 8" Hollow-Stem Auger (MR Drilling)									
DRIVE WEIGHT 140 lbs. (Auto. Trip Hammer) DROP 30"									
SAMPLED BY GM LOGGED BY GM REVIEWED BY MRH/MLP									
DESCRIPTION/INTERPRETATION									
40			27				SP-SM	ALLUVIUM: (Continued) Olive brown, moist, dense, poorly graded SAND with silt and gravel.	
			37				SM	Olive brown, moist, very dense, silty SAND with gravel. Difficult drilling; possible cobbles.	
50			33					Dense.	
			31					@ 55': Groundwater measured approximately 5 minutes after completion of drilling. Wet.	
60			36					@ 60': Groundwater encountered during drilling. Very dense.	
								Total Depth = 61.5 feet. Groundwater was encountered during drilling at approximately 60 feet. Groundwater was measured at approximately 55 feet after completion of drilling. Backfilled with cement-bentonite grout on 2/23/23.	
								Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.	
								The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.	
70									
80									

FIGURE A- 13



Geotechnical & Environmental Sciences Consultants

FIGURE A- 14

VIA PRINCESSA PARK
SANTA CLARITA, CALIFORNIA

212109001 | 6/23

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED <u>2/23/23</u> BORING NO. <u>B-14</u> GROUND ELEVATION <u>1,390' ± (MSL)</u> SHEET <u>1</u> OF <u>1</u> METHOD OF DRILLING <u>8" Hollow-Stem Auger (MR Drilling)</u> DRIVE WEIGHT <u>140 lbs. (Auto. Trip Hammer)</u> DROP <u>30"</u> SAMPLED BY <u>GM</u> LOGGED BY <u>GM</u> REVIEWED BY <u>MRH/MLP</u> DESCRIPTION/INTERPRETATION	
	Bulk	Driven							
0							CL	FILL: Grayish brown, moist, stiff, sandy lean CLAY.	
							SM	ALLUVIUM: Olive brown, moist, medium dense, silty SAND with gravel.	
			31						
							SP	Yellowish brown, moist, medium dense, poorly graded SAND with gravel.	
10			34	3.2	119.5				
									
			38						
									
20			42	5.3	116.6				
							SM	Olive brown, moist, medium dense, silty SAND with gravel.	
			38						
							SP	Yellowish brown, moist, medium dense, poorly graded SAND with gravel.	
30			38	3.5	120.4				
								Total Depth = 31.5 feet. Groundwater was not encountered during drilling. Backfilled with cement-bentonite grout on 2/23/23. <u>Notes:</u> Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.	
40									

FIGURE A- 15

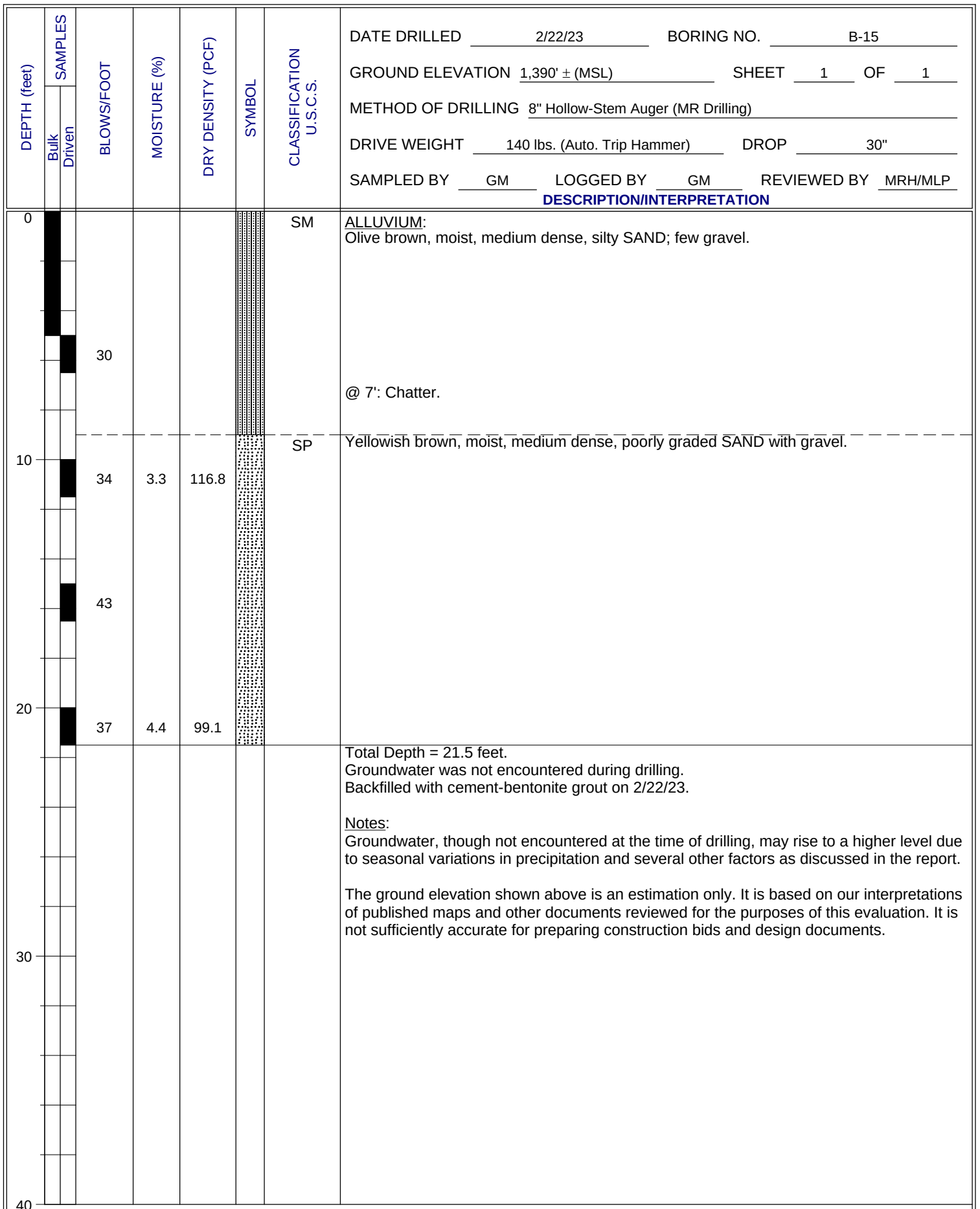


FIGURE A- 16

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED <u>3/15/23</u> BORING NO. <u>B-16</u> GROUND ELEVATION <u>1,390' ± (MSL)</u> SHEET <u>1</u> OF <u>3</u> METHOD OF DRILLING <u>8" Hollow-Stem Auger (MR Drilling)</u> DRIVE WEIGHT <u>140 lbs. (Auto. Trip Hammer)</u> DROP <u>30"</u> SAMPLED BY <u>GM</u> LOGGED BY <u>GM</u> REVIEWED BY <u>MRH/MLP</u> <u>DESCRIPTION/INTERPRETATION</u>	
	Bulk	Driven							
0							CL	FILL: Olive brown, moist, stiff, sandy lean CLAY; trace gravel.	
							SC	ALLUVIUM: Olive brown, moist, medium dense, clayey SAND; trace gravel.	
			29	16.9	111.2				
							SP	Light grayish brown, moist, medium dense, poorly graded SAND; trace gravel.	
10			29						
							SM	Yellowish brown, moist, dense, silty SAND; few gravel.	
			33						
							SP-SM	Yellowish brown, moist, dense, poorly graded SAND with silt and gravel.	
20			26	4.3					
							CL	Reddish brown, moist, hard, sandy lean CLAY.	
			40	15.1			SM	Yellowish brown, moist, very dense, silty SAND; few gravel.	
30			27	6.2				Dense.	
			32						
40									

FIGURE A- 17

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED <u>3/15/23</u> BORING NO. <u>B-16</u>
	Bulk	Driven						GROUND ELEVATION <u>1,390' ± (MSL)</u> SHEET <u>2</u> OF <u>3</u>
								METHOD OF DRILLING <u>8" Hollow-Stem Auger (MR Drilling)</u>
								DRIVE WEIGHT <u>140 lbs. (Auto. Trip Hammer)</u> DROP <u>30"</u>
								SAMPLED BY <u>GM</u> LOGGED BY <u>GM</u> REVIEWED BY <u>MRH/MLP</u>
								DESCRIPTION/INTERPRETATION
40			34				SM	ALLUVIUM: (Continued) Yellowish brown, moist, very dense, silty SAND; few gravel.
			32				SP-SM	Yellowish brown, moist, dense, poorly graded SAND with silt and gravel.
			41				SM	Yellowish brown, moist, very dense, silty SAND; few gravel.
50								
			50/5"					
60								
								@ 65.8': Groundwater measured approximately 30 minutes after completion of drilling.
							SP	Yellowish brown, wet, very dense, poorly graded SAND with gravel.
70			38					@ 70': Groundwater encountered during drilling.
							SM	Yellowish brown, wet, very dense, silty SAND with gravel.
80								

FIGURE A- 18

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED <u>3/15/23</u> BORING NO. <u>B-16</u> GROUND ELEVATION <u>1,390' ± (MSL)</u> SHEET <u>3</u> OF <u>3</u> METHOD OF DRILLING <u>8" Hollow-Stem Auger (MR Drilling)</u> DRIVE WEIGHT <u>140 lbs. (Auto. Trip Hammer)</u> DROP <u>30"</u> SAMPLED BY <u>GM</u> LOGGED BY <u>GM</u> REVIEWED BY <u>MRH/MLP</u> DESCRIPTION/INTERPRETATION
	Bulk	Driven						
80			51				SM	ALLUVIUM: (Continued) Yellowish brown, wet, very dense, silty SAND with gravel.
90			31				SP	Yellowish brown, wet, dense, poorly graded SAND; few gravel.
100			44					Very dense. Total Depth = 101.5 feet. Groundwater was encountered at approximately 70 feet during drilling. Groundwater was measured at approximately 65.8 feet approximately 30 minutes after drilling. Backfilled with cement-bentonite grout on 3/15/23.
110								Notes: Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
120								

FIGURE A- 19


FIGURE A- 20
VIA PRINCESSA PARK
SANTA CLARITA, CALIFORNIA
212109001 | 6/23

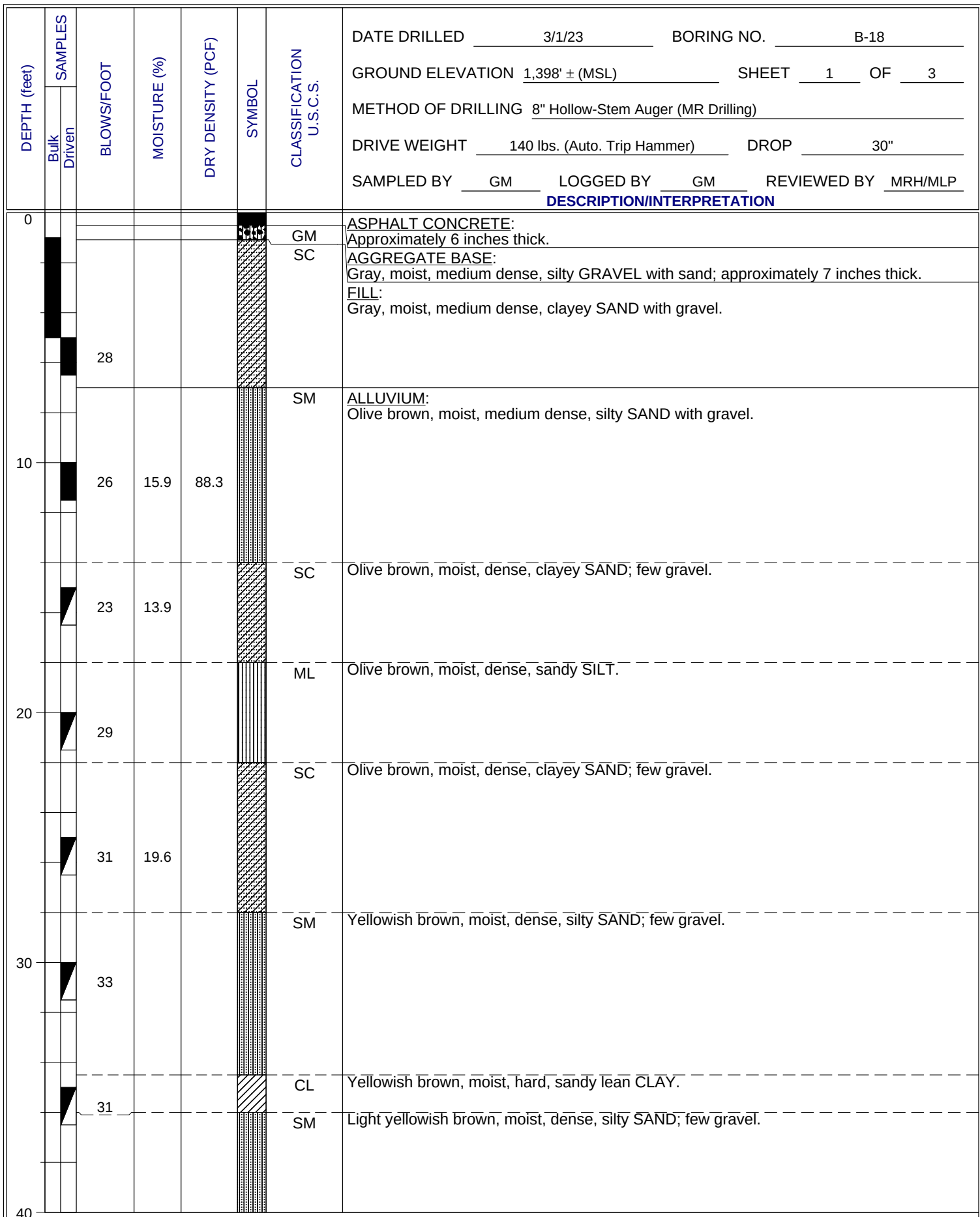


FIGURE A- 21



Geotechnical & Environmental Sciences Consultants

FIGURE A- 22

VIA PRINCESSA PARK
SANTA CLARITA, CALIFORNIA

212109001 | 6/23

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED <u>3/1/23</u> BORING NO. <u>B-18</u>	
	Bulk	Driven						GROUND ELEVATION <u>1,398' ± (MSL)</u>	SHEET <u>3</u> OF <u>3</u>
								METHOD OF DRILLING <u>8" Hollow-Stem Auger (MR Drilling)</u>	
								DRIVE WEIGHT <u>140 lbs. (Auto. Trip Hammer)</u>	DROP <u>30"</u>
								SAMPLED BY <u>GM</u>	LOGGED BY <u>GM</u> REVIEWED BY <u>MRH/MLP</u>
								DESCRIPTION/INTERPRETATION	
80			43				SP-SM	ALLUVIUM: (Continued) Light yellowish brown, wet, very dense, poorly graded SAND with silt; few gravel.	
90			41						
100			34					Total Depth = 101.5 feet. Groundwater was encountered at approximately 70 feet during drilling. Groundwater was measured at approximately 68 feet approximately 30 minutes after drilling. Backfilled with cement-bentonite grout and patched with rapid-set concrete dyed black on 3/1/23. Notes: Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.	
110									
120									

FIGURE A- 23



APPENDIX B

CPT Logs

SUMMARY OF CONE PENETRATION TEST DATA

Project:

**Psomas
19201 Via Princessa
Santa Clarita, CA
February 22, 2023**

Prepared for:

**Mr. Matthew Harrell
Ninyo & Moore
475 Goddard, Ste 200
Irvine, CA 92618-4605
Office (949) 753-7070 / Fax (949) 753-7071**

Prepared by:



KEHOE TESTING & ENGINEERING

5415 Industrial Drive
Huntington Beach, CA 92649-1518
Office (714) 901-7270 / Fax (714) 901-7289
www.kehoetesting.com

TABLE OF CONTENTS

- 1. INTRODUCTION**
- 2. SUMMARY OF FIELD WORK**
- 3. FIELD EQUIPMENT & PROCEDURES**
- 4. CONE PENETRATION TEST DATA & INTERPRETATION**

APPENDIX

- CPT Plots
- CPT Classification/Soil Behavior Chart
- Summary of Shear Wave Velocities
- CPT Data Files (sent via email)

SUMMARY OF CONE PENETRATION TEST DATA

1. INTRODUCTION

This report presents the results of a Cone Penetration Test (CPT) program carried out for the Psomas project located at 19201 Via Princessa in Santa Clarita, California. The work was performed by Kehoe Testing & Engineering (KTE) on February 22, 2023. The scope of work was performed as directed by Ninyo & Moore personnel.

2. SUMMARY OF FIELD WORK

The fieldwork consisted of performing CPT soundings at six locations to determine the soil lithology. A summary is provided in **TABLE 2.1**.

LOCATION	DEPTH OF CPT (ft)	COMMENTS/NOTES:
CPT-1	33	Refusal
CPT-2	35	Refusal
CPT-3	12	Refusal
CPT-3A	12	Refusal
CPT-4	11	Refusal
CPT-4A	11	Refusal

TABLE 2.1 - Summary of CPT Soundings

3. FIELD EQUIPMENT & PROCEDURES

The CPT soundings were carried out by **KTE** using an integrated electronic cone system manufactured by Vertek. The CPT soundings were performed in accordance with ASTM standards (D5778). The cone penetrometers were pushed using a 30-ton CPT rig. The cone used during the program was a 15 cm² cone with a cone net area ratio of 0.83. The following parameters were recorded at approximately 2.5 cm depth intervals:

- Cone Resistance (qc)
- Sleeve Friction (fs)
- Dynamic Pore Pressure (u)
- Inclination
- Penetration Speed

At locations CPT-4 & CPT4A, shear wave measurements were obtained at various depths. The shear wave is generated using an air-actuated hammer, which is located inside the front jack of the CPT rig. The cone has a triaxial geophone, which recorded the shear wave signal generated by the air hammer.

The above parameters were recorded and viewed in real time using a laptop computer. Data is stored at the KTE office for up to 2 years for future analysis and reference. A complete set of baseline readings was taken prior to each sounding to determine temperature shifts and any zero load offsets. Monitoring base line readings ensures that the cone electronics are operating properly.

4. CONE PENETRATION TEST DATA & INTERPRETATION

The Cone Penetration Test data is presented in graphical form in the attached Appendix. These plots were generated using the CPeT-IT program. Penetration depths are referenced to ground surface. The soil behavior type on the CPT plots is derived from the attached CPT SBT plot (Robertson, "Interpretation of Cone Penetration Test...", 2009) and presents major soil lithologic changes. The stratigraphic interpretation is based on relationships between cone resistance (q_c), sleeve friction (f_s), and penetration pore pressure (u). The friction ratio (R_f), which is sleeve friction divided by cone resistance, is a calculated parameter that is used along with cone resistance to infer soil behavior type. Generally, cohesive soils (clays) have high friction ratios, low cone resistance and generate excess pore water pressures. Cohesionless soils (sands) have lower friction ratios, high cone bearing and generate little (or negative) excess pore water pressures.

The CPT data files have also been provided. These files can be imported in CPeT-IT (software by GeoLogismiki) and other programs to calculate various geotechnical parameters.

It should be noted that it is not always possible to clearly identify a soil type based on q_c , f_s and u . In these situations, experience, judgement and an assessment of the pore pressure data should be used to infer the soil behavior type.

If you have any questions regarding this information, please do not hesitate to call our office at (714) 901-7270.

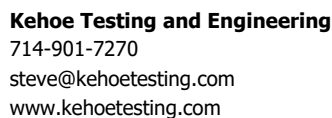
Sincerely,

KEHOE TESTING & ENGINEERING



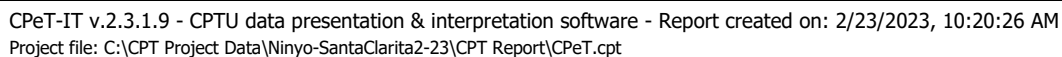
Steven P. Kehoe
President

APPENDIX



Location: 19201 Via Princessa, Santa Clarita, CA

Total depth: 33.01 ft, Date: 2/22/2023



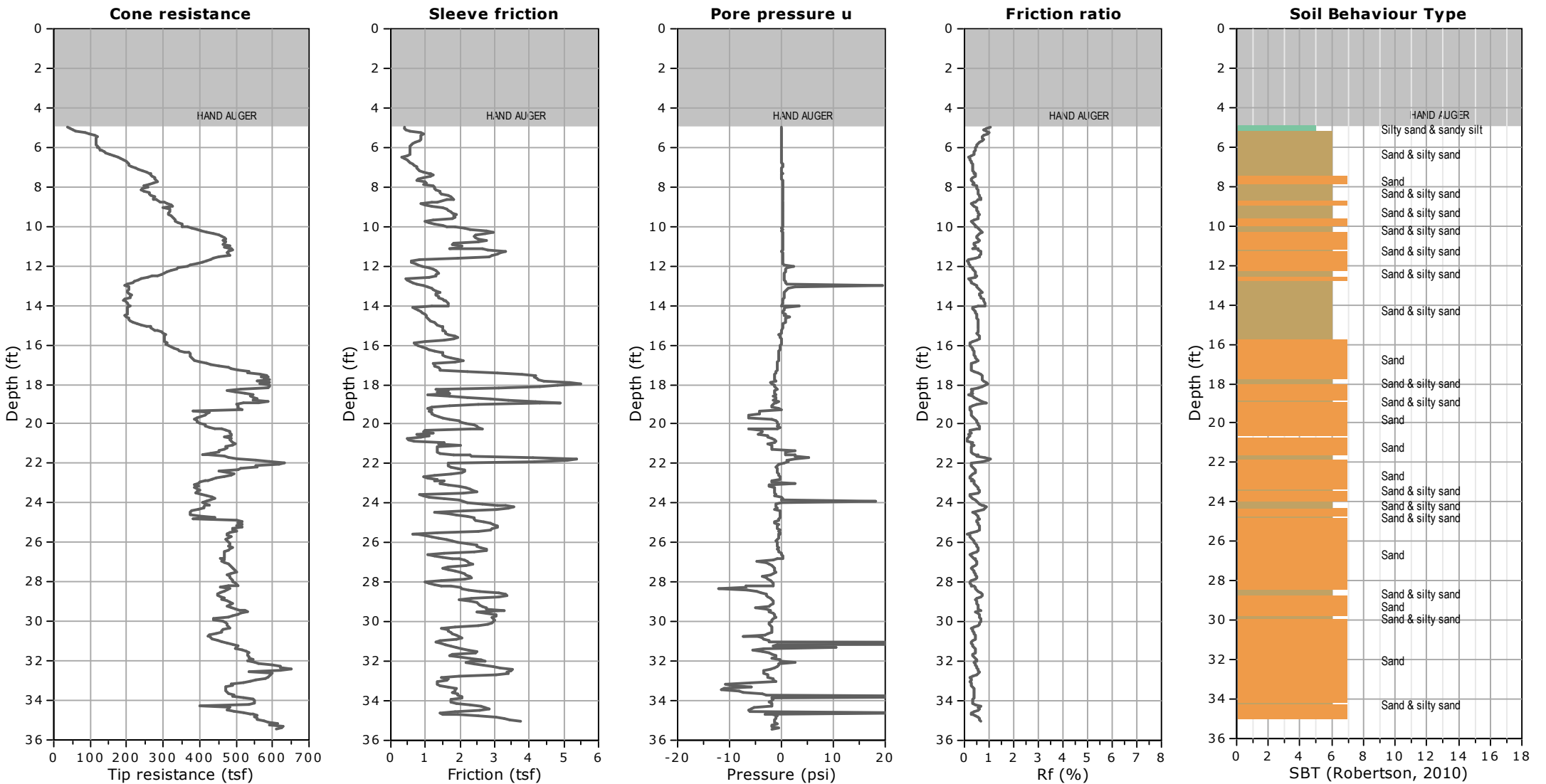


Project: Ninyo & Moore / Psomas

Location: 19201 Via Princessa, Santa Clarita, CA

CPT-2

Total depth: 35.44 ft, Date: 2/22/2023



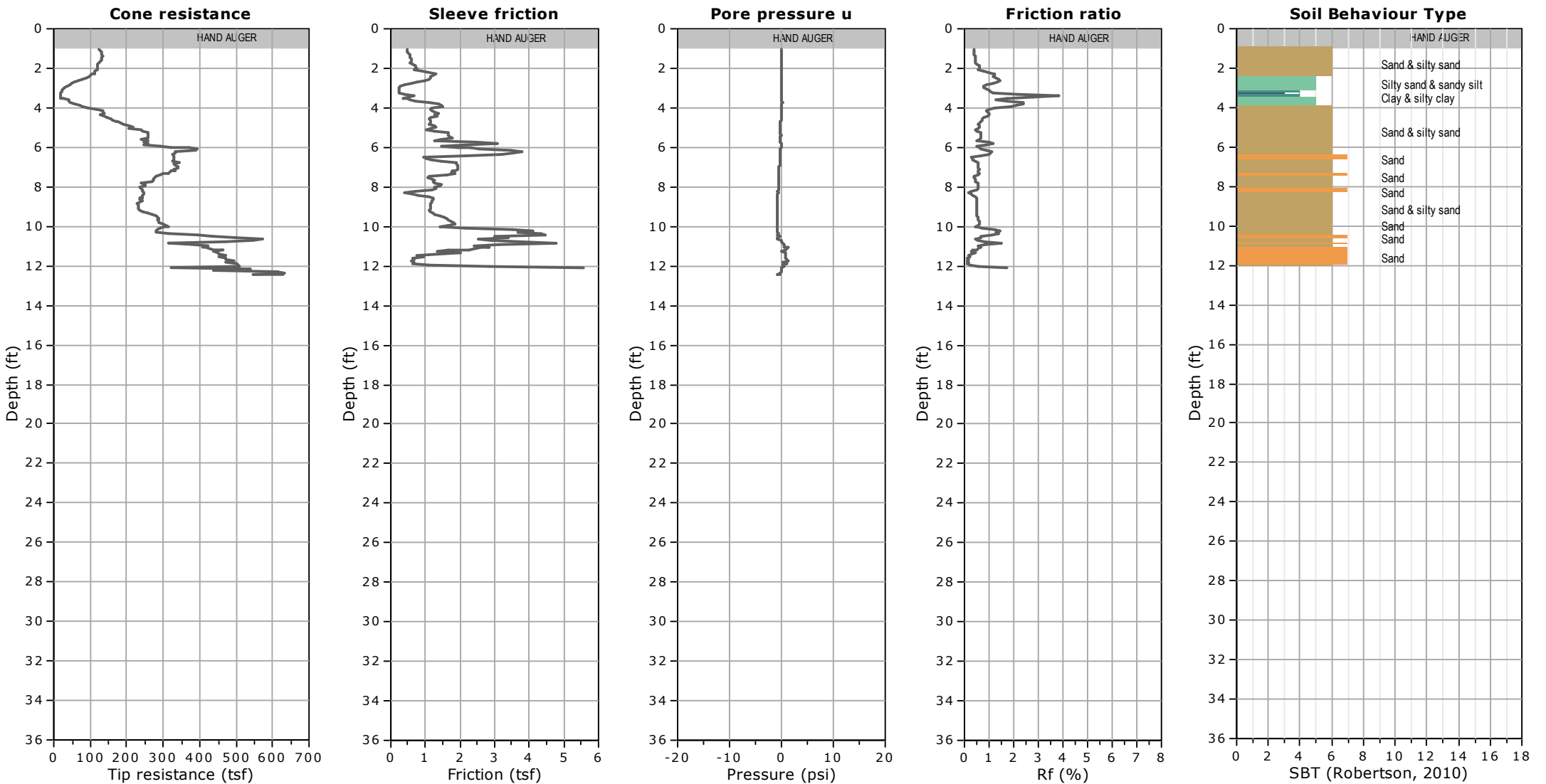


Project: Ninyo & Moore / Psomas

Location: 19201 Via Princessa, Santa Clarita, CA

CPT-3

Total depth: 12.47 ft, Date: 2/22/2023



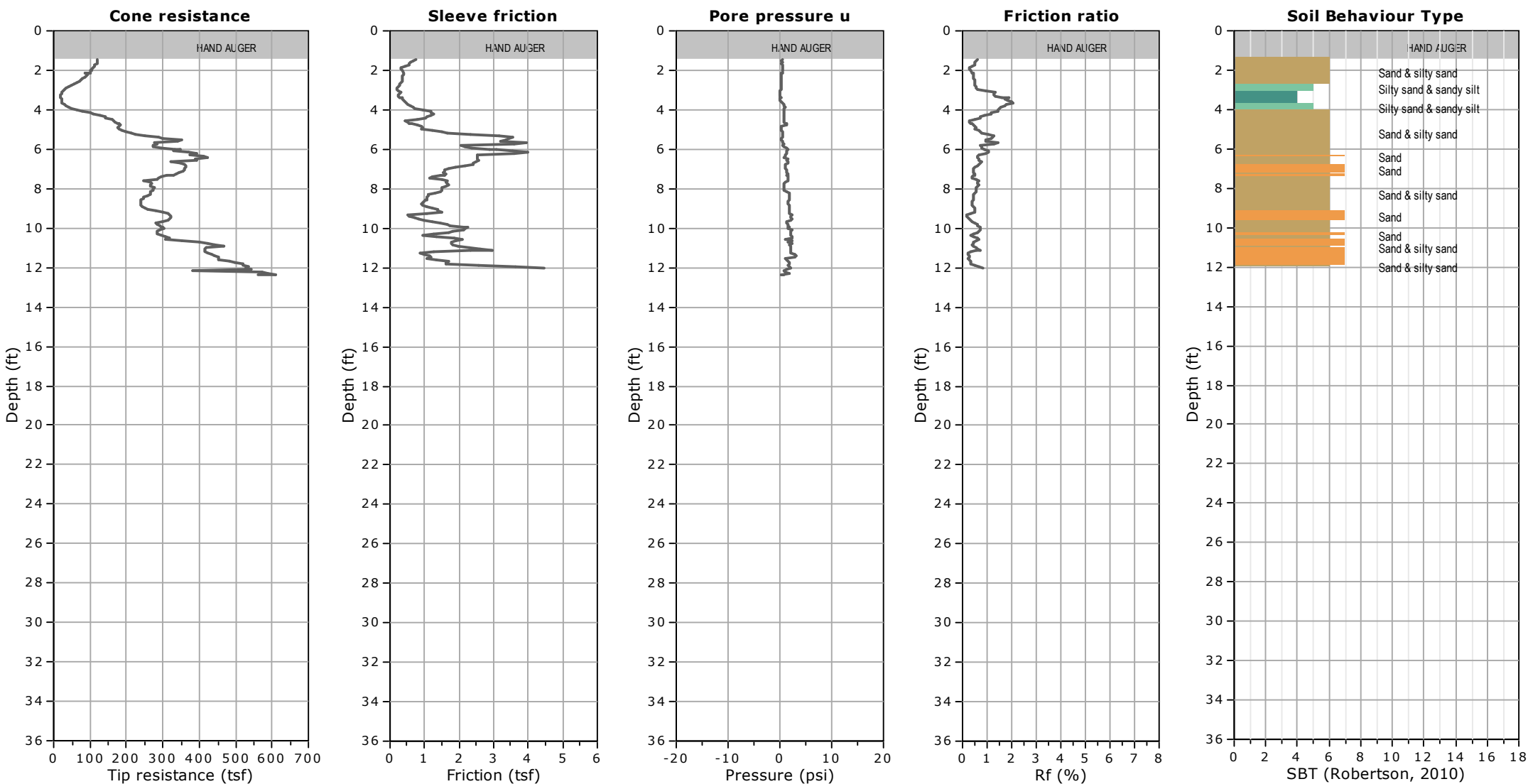


Project: Ninyo & Moore / Psomas

Location: 19201 Via Princessa, Santa Clarita, CA

CPT-3A

Total depth: 12.40 ft, Date: 2/22/2023



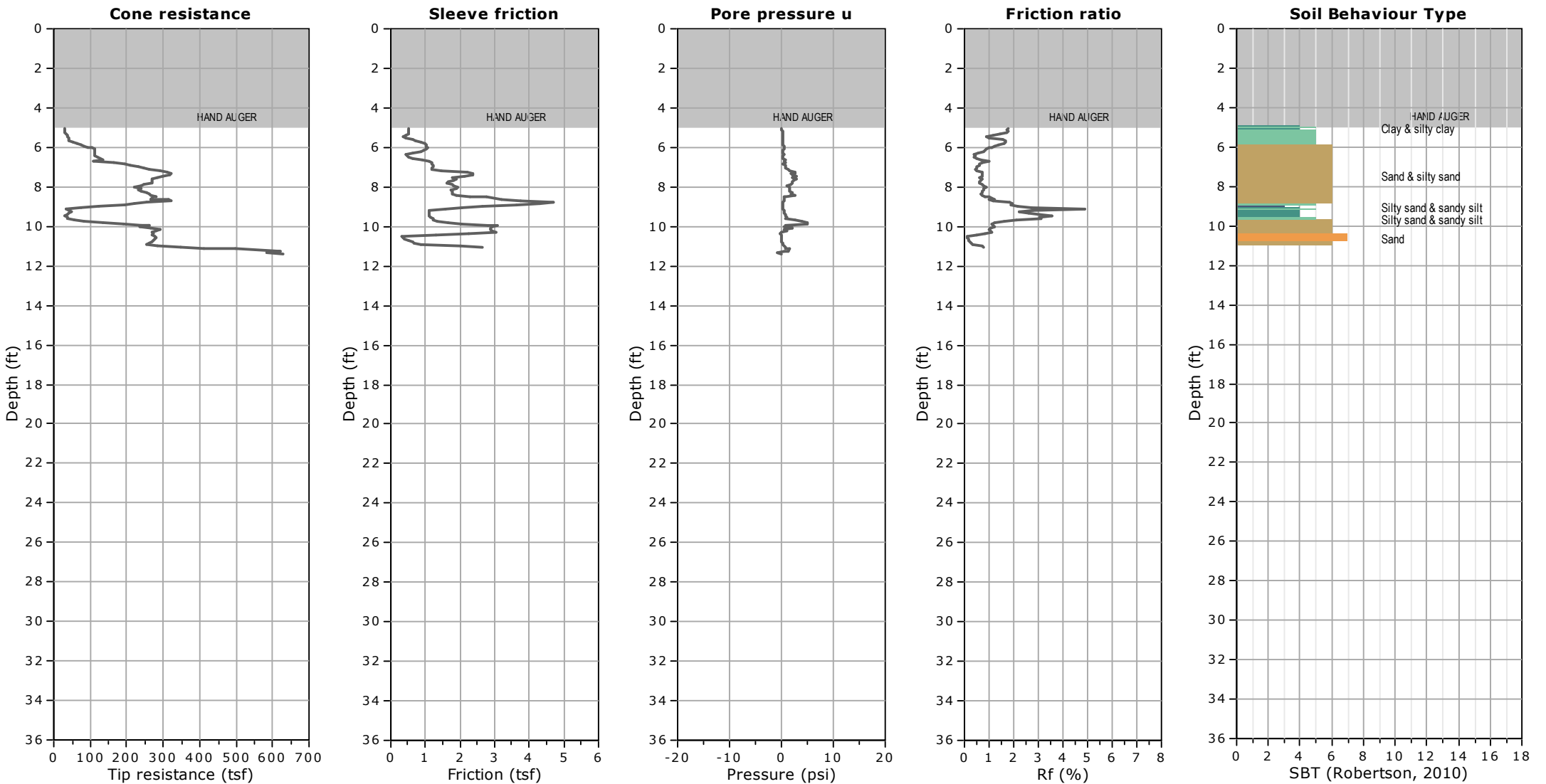


Project: Ninyo & Moore / Psomas

Location: 19201 Via Princessa, Santa Clarita, CA

CPT-4

Total depth: 11.43 ft, Date: 2/22/2023



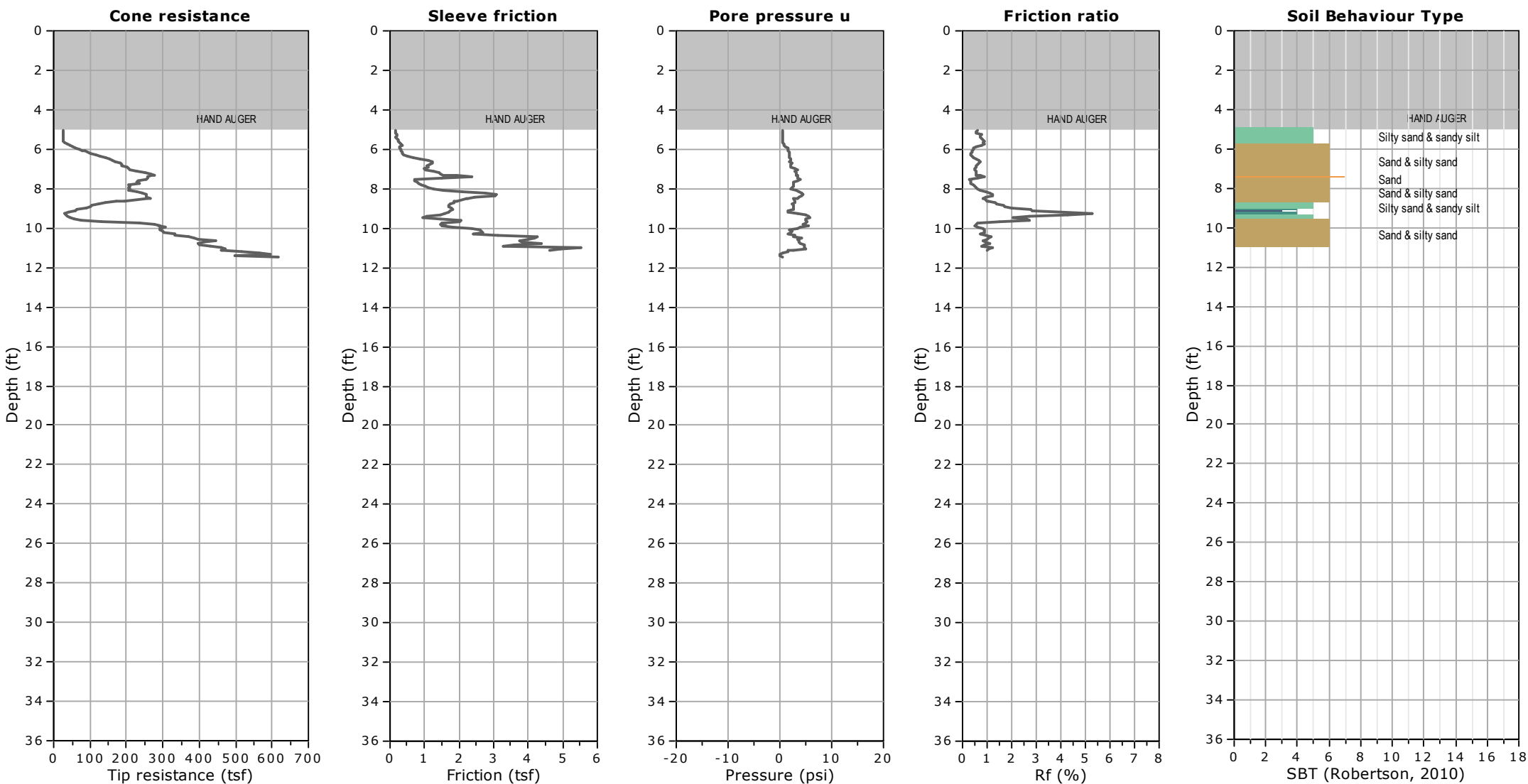


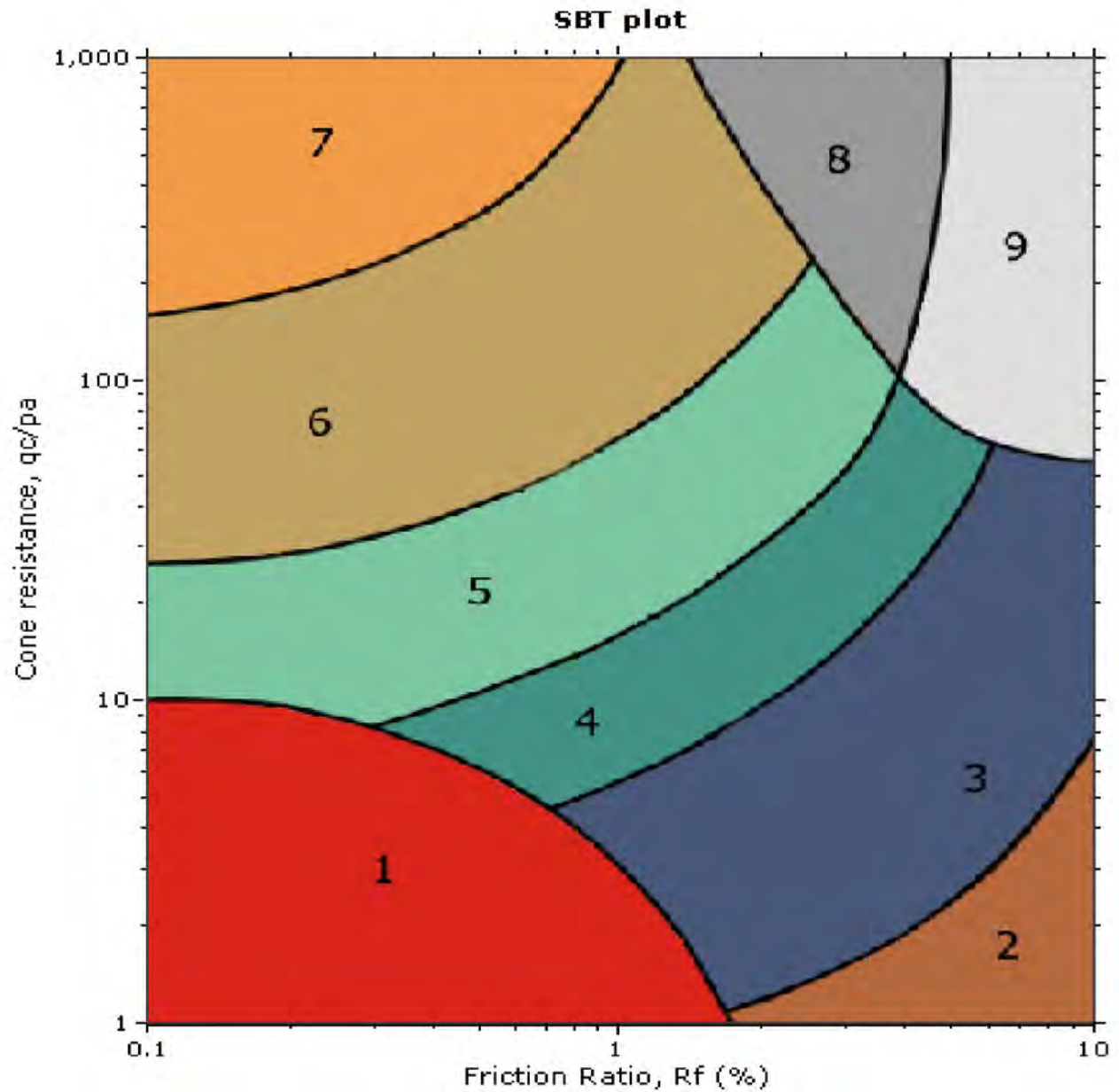
Project: Ninyo & Moore / Psomas

Location: 19201 Via Princessa, Santa Clarita, CA

CPT-4A

Total depth: 11.49 ft, Date: 2/22/2023





SBT legend

- | | | |
|---|---|---|
| ■ 1. Sensitive fine grained | ■ 4. Clayey silt to silty clay | ■ 7. Gravely sand to sand |
| ■ 2. Organic material | ■ 5. Silty sand to sandy silt | ■ 8. Very stiff sand to clayey sand |
| ■ 3. Clay to silty clay | ■ 6. Clean sand to silty sand | ■ 9. Very stiff fine grained |

Ninyo & Moore
 Psomas
 Santa Clarita, CA

CPT Shear Wave Measurements

Location	Tip Depth (ft)	Geophone Depth (ft)	Travel Distance (ft)	S-Wave Arrival (msec)	S-Wave Velocity from Surface (ft/sec)	Interval S-Wave Velocity (ft/sec)
CPT-4	5.02	4.02	4.49	8.44	532	
	10.04	9.04	9.26	14.42	642	797
CPT-4A	5.02	4.02	4.49	7.48	600	
	10.01	9.01	9.23	14.80	624	647

Shear Wave Source Offset - 2 ft

S-Wave Velocity from Surface = Travel Distance/S-Wave Arrival
 Interval S-Wave Velocity = (Travel Dist2-Travel Dist1)/(Time2-Time1)



APPENDIX C

Boring and CPT Logs RTFA (2022)

Note: The log of subsurface conditions shown hereon is approximate and applies only at the specific location and date indicated.
It is not warranted to be representative of subsurface conditions at other locations or times.

							BORING IB-1	
BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (LBS. PER CU. FT.)	N-VALUE	DEPTH (FEET)	SAMPLE LOCATION	GRAPHIC LOG	SOIL TYPE	JOB NUMBER: 2018-003-054 DATE DRILLED: 1/17/22 EQUIPMENT USED: 8" Hollow stem auger ELEVATION: 1383' LOGGED BY: SDR DEPTH: 0-20.5' HAMMER WEIGHT: 140 pounds; DROP: 30" SURFACE CONDITIONS: Level Open Field with moderate brush
12			-				SM	SILTY SAND: fine to medium with occasional coarse, medium dense, damp, light brown
								clayey, dense, moist
15			-	5			SP	SAND: fine, slightly silty, dense, moist, yellowish brown
36			-					clean, damp, tan to buff
37			-	10				lens of gravel to 3/4"
90/11"			-				SM	SILTY SAND: fine to coarse, dense, moist, medium brown
70/8"			-	15			SP	SAND: fine, clean, dense, damp, tank to buff
37/5"			-	20				fine to medium with occasional coarse
								Bottom of Boring at 20.5 feet. No water. Caving below 15'.
				25				
				30				
				35				
				40				

LOG OF BORING

Note: The log of subsurface conditions shown hereon is approximate and applies only at the specific location and date indicated.
It is not warranted to be representative of subsurface conditions at other locations or times.

							BORING IB-2	
BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (LBS. PER CU. FT.)	N-VALUE	DEPTH (FEET)	SAMPLE LOCATION	GRAPHIC LOG	SOIL TYPE	JOB NUMBER: 2018-003-054 DATE DRILLED: 1/17/22 EQUIPMENT USED: 8" Hollow stem auger ELEVATION: 1382' LOGGED BY: SDR DEPTH: 0-21' HAMMER WEIGHT: 140 pounds; DROP: 30" SURFACE CONDITIONS: Level Open Field with moderate brush
28			-				SM	SILTY SAND: fine to medium with occasional coarse, medium dense, damp, light brown
							SW	SAND: fine to coarse, slightly silty, gravel to 1/2", dense, damp, medium brown
24			-	5			SM-ML	SILTY SAND: very fine to fine, dense, moist, medium brown
25			-				SM	SILTY SAND: fine, dense, damp, light yellowish brown
28			-	10				
70/8"			-					
40/3"			-	15			SW	SAND: fine to coarse, slightly silty, dense, damp, tan to buff
70/11"			-	20			SM	SILTY SAND: fine to coarse, dense, damp, tan
								Bottom of Boring at 21 feet. No water. Caving below 15'.
				25				
				30				
				35				
				40				

LOG OF BORING

Note: The log of subsurface conditions shown hereon is approximate and applies only at the specific location and date indicated.
It is not warranted to be representative of subsurface conditions at other locations or times.

							BORING B-3	
							JOB NUMBER: 2018-003-054 DATE DRILLED: 1/17/22 EQUIPMENT USED: 8" Hollow stem auger ELEVATION: 1393' LOGGED BY: SDR DEPTH: 0-21' HAMMER WEIGHT: 140 pounds; DROP: 30" SURFACE CONDITIONS: Existing Asphalt Parking Lot	
BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (LBS. PER CU. FT.)	N-VALUE	DEPTH (FEET)	SAMPLE LOCATION	GRAPHIC LOG	SOIL TYPE	
33			-				SM	ASPHALT: 4" AC/ 4" BASE
19			-	5				ARTIFICIAL FILL SILTY SAND: fine to medium, occasional gravel to 1/2", well compacted, moist, mottled medium brown and light yellowish brown
23			-					mottled medium brown and gray
20			-	10				
30			-					
30/5"			-	15				
30			-	20			SM	NATIVE SILTY SAND: fine to coarse, gravel to 1", dense, damp, medium yellowish brown
								Bottom of Boring at 21 feet. No water. No Caving.
				25				
				30				
				35				
				40				

LOG OF BORING

Note: The log of subsurface conditions shown hereon is approximate and applies only at the specific location and date indicated.
It is not warranted to be representative of subsurface conditions at other locations or times.

							BORING B-4	
							JOB NUMBER: 2018-003-054 DATE DRILLED: 1/18/22 EQUIPMENT USED: 8" Hollow stem auger ELEVATION: 1381' LOGGED BY: SDR DEPTH: 0-51.5' HAMMER WEIGHT: 140 pounds; DROP: 30" SURFACE CONDITIONS: Level Open Field with moderate brush	
BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (LBS. PER CU. FT.)	N-VALUE	DEPTH (FEET)	SAMPLE LOCATION	GRAPHIC LOG	SOIL TYPE	
21			-				SM	SILTY SAND: fine to coarse, medium dense, damp, light brown
33			-	5			SP	SAND: fine to very coarse, clean, dense, damp to moist, light yellowish brown
24			-					fine to medium with occasional coarse, damp
46			-	10				lens of gravel to 3/4"
-			43					fine to coarse
62			-	15				fine to very coarse
-			39					
25/1"			-	20				gravel lens
-			*	25				(* 25 blows for the first 6")
-			48	30				fine to medium with occasional coarse
-			55	35				fine to very coarse, gravel to 1/2"
				40				

(CONTINUED ON THE FOLLOWING FIGURE)

LOG OF BORING

Note: The log of subsurface conditions shown hereon is approximate and applies only at the specific location and date indicated.
It is not warranted to be representative of subsurface conditions at other locations or times.

		BLOWS PER FOOT	
		MOISTURE CONTENT (%)	
		DRY UNIT WEIGHT (LBS. PER CU. FT.)	
		N-VALUE	32
		DEPTH (FEET)	53
		SAMPLE LOCATION	91/11"
		GRAPHIC LOG	
		SOIL TYPE	SP
			JOB NUMBER: 2018-003-054 DATE DRILLED: 1/18/22 EQUIPMENT USED: 8" Hollow stem auger ELEVATION: 138.1' LOGGED BY: SDR DEPTH: 0-51.5' HAMMER WEIGHT: 140 pounds; DROP: 30" SURFACE CONDITIONS: Level Open Field with moderate brush
			fine to coarse
			fine to very coarse
			Bottom of Boring at 51.5 feet. No water. No Caving.

LOG OF BORING



GREGG DRILLING & TESTING, LLC.
GEOTECHNICAL AND ENVIRONMENTAL INVESTIGATION SERVICES

11/2/18

R.T. Frankian & Associates
Attn: Jim Frankian

Subject: CPT Site Investigation
CSC LID Site X
Santa Clarita, California
GREGG Project Number: D1180644SH

Dear Mr. Frankian:

The following report presents the results of GREGG Drilling & Testing's Cone Penetration Test investigation for the above referenced site. The following testing services were performed:

1	Cone Penetration Tests	(CPTU)	☒
2	Pore Pressure Dissipation Tests	(PPD)	☒
3	Seismic Cone Penetration Tests	(SCPTU)	☐
4	UVOST Laser Induced Fluorescence	(UVOST)	☐
5	Groundwater Sampling	(GWS)	☐
6	Soil Sampling	(SS)	☐
7	Vapor Sampling	(VS)	☐
8	Pressuremeter Testing	(PMT)	☐
9	Vane Shear Testing	(VST)	☐
10	Dilatometer Testing	(DMT)	☐

A list of reference papers providing additional background on the specific tests conducted is provided in the bibliography following the text of the report. If you would like a copy of any of these publications or should you have any questions or comments regarding the contents of this report, please do not hesitate to contact our office at (562) 427-6899.

Sincerely,
GREGG Drilling & Testing, LLC.

Frank Stolfi
HRSC Division Manager, Gregg Drilling & Testing, LLC.



GREGG DRILLING & TESTING, LLC.
GEOTECHNICAL AND ENVIRONMENTAL INVESTIGATION SERVICES

Cone Penetration Test Sounding Summary

-Table 1-

CPT Sounding Identification	Date	Termination Depth (feet)	Depth of Groundwater Samples (feet)	Depth of Soil Samples (feet)	Depth of Pore Pressure Dissipation Tests (feet)
CPT-1	10/30/2018	44.13	-	-	44.4
CPT-2	10/30/2018	13.62	-	-	-
CPT-3	10/30/2018	34.45	-	10, 16	-
CPT-4	10/30/2018	40.52	-	-	15.7, 16.2
CPT-5	10/30/2018	29.69	-	15, 17, 18, 20, 25	-
CPT-6	10/31/2018	28.87	-	-	-
CPT-7	10/31/2018	13.94	-	-	-
CPT-8	10/31/2018	32.81	-	15, 18, 20, 24, 26	-
CPT-9	10/31/2018	21.98	-	-	-
CPT-10	11/1/2018	12.3	-	-	-
CPT-11	11/1/2018	35.43	-	-	-
CPT-12	11/1/2018	14.93	-	-	-



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Conference, Las Vegas, Nevada Proceedings, 1992, pp 299-312.

Copies of ASTM Standards are available through www.astm.org

Cone Penetration Testing Procedure (CPT)

Gregg Drilling carries out all Cone Penetration Tests (CPT) using an integrated electronic cone system, *Figure CPT*.

The cone takes measurements of tip resistance (q_c), sleeve resistance (f_s), and penetration pore water pressure (u_2). Measurements are taken at either 2.5 or 5 cm intervals during penetration to provide a nearly continuous profile. CPT data reduction and basic interpretation is performed in real time facilitating on-site decision making. The above mentioned parameters are stored electronically for further analysis and reference. All CPT soundings are performed in accordance with revised ASTM standards (D 5778-12).

The 5mm thick porous plastic filter element is located directly behind the cone tip in the u_2 location. A new saturated filter element is used on each sounding to measure both penetration pore pressures as well as measurements during a dissipation test (PPDT). Prior to each test, the filter element is fully saturated with oil under vacuum pressure to improve accuracy.

When the sounding is completed, the test hole is backfilled according to client specifications. If grouting is used, the procedure generally consists of pushing a hollow tremie pipe with a “knock out” plug to the termination depth of the CPT hole. Grout is then pumped under pressure as the tremie pipe is pulled from the hole. Disruption or further contamination to the site is therefore minimized.

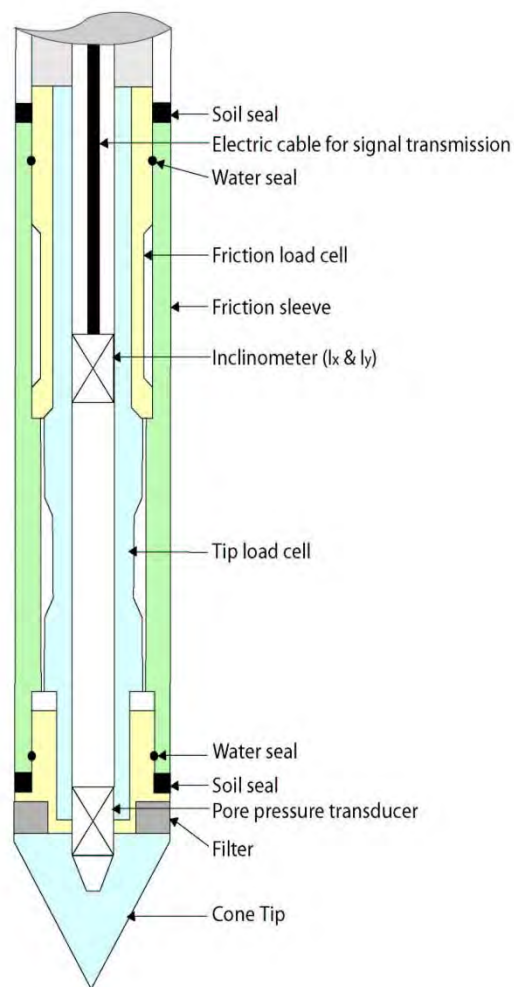


Figure CPT

Gregg 15cm² Standard Cone Specifications

Dimensions	
Cone base area	15 cm ²
Sleeve surface area	225 cm ²
Cone net area ratio	0.80
Specifications	
Cone load cell	
Full scale range	180 kN (20 tons)
Overload capacity	150%
Full scale tip stress	120 MPa (1,200 tsf)
Repeatability	120 kPa (1.2 tsf)
Sleeve load cell	
Full scale range	31 kN (3.5 tons)
Overload capacity	150%
Full scale sleeve stress	1,400 kPa (15 tsf)
Repeatability	1.4 kPa (0.015 tsf)
Pore pressure transducer	
Full scale range	7,000 kPa (1,000 psi)
Overload capacity	150%
Repeatability	7 kPa (1 psi)

Note: The repeatability during field use will depend somewhat on ground conditions, abrasion, maintenance and zero load stability.

Cone Penetration Test Data & Interpretation

The Cone Penetration Test (CPT) data collected are presented in graphical and electronic form in the report. The plots include interpreted Soil Behavior Type (SBT) based on the charts described by Robertson (1990). Typical plots display SBT based on the non-normalized charts of Robertson et al (1986). For CPT soundings deeper than 30m, we recommend the use of the normalized charts of Robertson (1990) which can be displayed as SBT_n, upon request. The report also includes spreadsheet output of computer calculations of basic interpretation in terms of SBT and SBT_n and various geotechnical parameters using current published correlations based on the comprehensive review by Lunne, Robertson and Powell (1997), as well as recent updates by Professor Robertson (Guide to Cone Penetration Testing, 2015). The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed. Gregg Drilling & Testing Inc. does not warranty the correctness or the applicability of any of the geotechnical parameters interpreted by the software and does not assume any liability for use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used in the software. Some interpretation methods require input of the groundwater level to calculate vertical effective stress. An estimate of the in-situ groundwater level has been made based on field observations and/or CPT results, but should be verified by the user.

A summary of locations and depths is available in Table 1. Note that all penetration depths referenced in the data are with respect to the existing ground surface.

Note that it is not always possible to clearly identify a soil type based solely on q_t , f_s , and u_2 . In these situations, experience, judgment, and an assessment of the pore pressure dissipation data should be used to infer the correct soil behavior type.

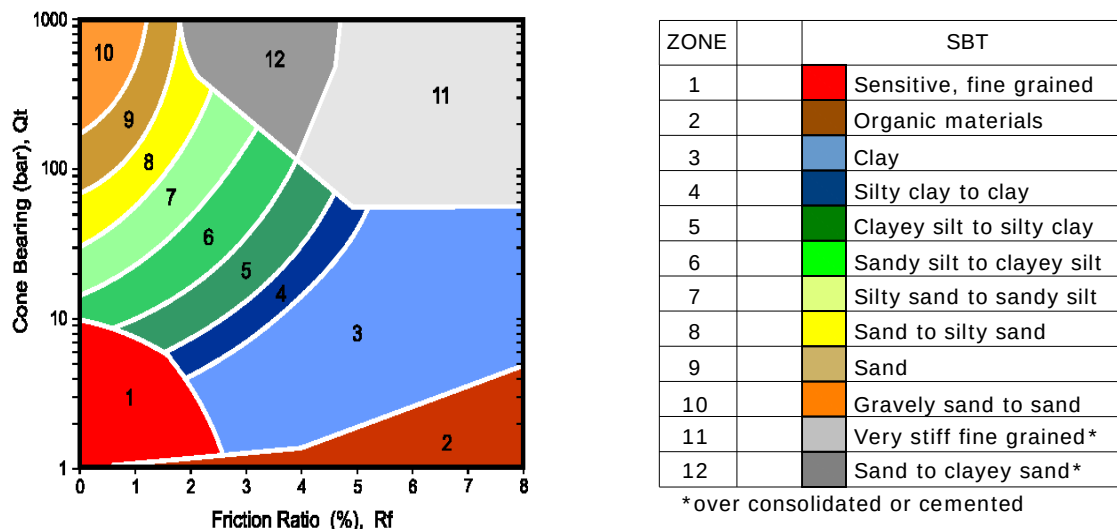


Figure SBT (After Robertson et al., 1986) – Note: Colors may vary slightly compared to plots

Cone Penetration Test (CPT) Interpretation

Gregg uses a proprietary CPT interpretation and plotting software. The software takes the CPT data and performs basic interpretation in terms of soil behavior type (SBT) and various geotechnical parameters using current published empirical correlations based on the comprehensive review by Lunne, Robertson and Powell (1997). The interpretation is presented in tabular format using MS Excel. The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed. Gregg does not warranty the correctness or the applicability of any of the geotechnical parameters interpreted by the software and does not assume any liability for any use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used in the software.

The following provides a summary of the methods used for the interpretation. Many of the empirical correlations to estimate geotechnical parameters have constants that have a range of values depending on soil type, geologic origin and other factors. The software uses 'default' values that have been selected to provide, in general, conservatively low estimates of the various geotechnical parameters.

Input:

- 1 Units for display (Imperial or metric) (atm. pressure, $p_a = 0.96$ tsf or 0.1 MPa)
- 2 Depth interval to average results (ft or m). Data are collected at either 0.02 or 0.05m and can be averaged every 1, 3 or 5 intervals.
- 3 Elevation of ground surface (ft or m)
- 4 Depth to water table, z_w (ft or m) – input required
- 5 Net area ratio for cone, a (default to 0.80)
- 6 Relative Density constant, C_{Dr} (default to 350)
- 7 Young's modulus number for sands, α (default to 5)
- 8 Small strain shear modulus number
 - a. for sands, S_G (default to 180 for SBT_n 5, 6, 7)
 - b. for clays, C_G (default to 50 for SBT_n 1, 2, 3 & 4)
- 9 Undrained shear strength cone factor for clays, N_{kt} (default to 15)
- 10 Over Consolidation ratio number, k_{ocr} (default to 0.3)
- 11 Unit weight of water, (default to $\gamma_w = 62.4$ lb/ft³ or 9.81 kN/m³)

Column

- 1 Depth, z , (m) – CPT data is collected in meters
- 2 Depth (ft)
- 3 Cone resistance, q_c (tsf or MPa)
- 4 Sleeve resistance, f_s (tsf or MPa)
- 5 Penetration pore pressure, u (psi or MPa), measured behind the cone (i.e. u_2)
- 6 Other – any additional data
- 7 Total cone resistance, q_t (tsf or MPa) $q_t = q_c + u (1-a)$

8	Friction Ratio, R_f (%)	$R_f = (f_s/q_t) \times 100\%$
9	Soil Behavior Type (non-normalized), SBT	see note
10	Unit weight, γ (pcf or kN/m ³)	based on SBT, see note
11	Total overburden stress, σ_v (tsf)	$\sigma_{vo} = \sigma_z$
12	In-situ pore pressure, u_o (tsf)	$u_o = \gamma_w (z - z_w)$
13	Effective overburden stress, σ'_{vo} (tsf)	$\sigma'_{vo} = \sigma_{vo} - u_o$
14	Normalized cone resistance, Q_{tn}	$Q_{tn} = (q_t - \sigma_{vo}) / \sigma'_{vo}$
15	Normalized friction ratio, F_r (%)	$F_r = f_s / (q_t - \sigma_{vo}) \times 100\%$
16	Normalized Pore Pressure ratio, B_q	$B_q = u - u_o / (q_t - \sigma_{vo})$
17	Soil Behavior Type (normalized), SBT_n	see note
18	SBT_n Index, I_c	see note
19	Normalized Cone resistance, Q_{tn} (n varies with I_c)	see note
20	Estimated permeability, k_{SBT} (cm/sec or ft/sec)	see note
21	Equivalent SPT N_{60} , blows/ft	see note
22	Equivalent SPT $(N_1)_{60}$ blows/ft	see note
23	Estimated Relative Density, D_r , (%)	see note
24	Estimated Friction Angle, ϕ' , (degrees)	see note
25	Estimated Young's modulus, E_s (tsf)	see note
26	Estimated small strain Shear modulus, G_o (tsf)	see note
27	Estimated Undrained shear strength, s_u (tsf)	see note
28	Estimated Undrained strength ratio	s_u/σ'_v
29	Estimated Over Consolidation ratio, OCR	see note

Notes:

- 1 Soil Behavior Type (non-normalized), SBT (Lunne et al., 1997 and table below)
- 2 Unit weight, γ either constant at 119 pcf or based on Non-normalized SBT (Lunne et al., 1997 and table below)
- 3 Soil Behavior Type (Normalized), SBT_n Lunne et al. (1997)
- 4 SBT_n Index, I_c $I_c = ((3.47 - \log Q_{tn})^2 + (\log F_r + 1.22)^2)^{0.5}$
- 5 Normalized Cone resistance, Q_{tn} (n varies with I_c)

$Q_{tn} = ((q_t - \sigma_{vo})/pa) (pa/(\sigma'_{vo}))^n$ and recalculate I_c , then iterate:

When $I_c < 1.64$, $n = 0.5$ (clean sand)
When $I_c > 3.30$, $n = 1.0$ (clays)
When $1.64 < I_c < 3.30$, $n = (I_c - 1.64)0.3 + 0.5$
Iterate until the change in n , $\Delta n < 0.01$

6 Estimated permeability, k_{SBT} based on Normalized SBT_n (Lunne et al., 1997 and table below)

7 Equivalent SPT N_{60} , blows/ft Lunne et al. (1997)

$$\frac{(q_t/p_a)}{N_{60}} = 8.5 \left(1 - \frac{I_c}{4.6} \right)$$

8 Equivalent SPT $(N_1)_{60}$ blows/ft $(N_1)_{60} = N_{60} C_N$
where $C_N = (p_a/\sigma'_{vo})^{0.5}$

9 Relative Density, D_r , (%) $D_r^2 = Q_{tn} / C_{Dr}$
Only SBT_n 5, 6, 7 & 8 Show 'N/A' in zones 1, 2, 3, 4 & 9

10 Friction Angle, ϕ' , (degrees) $\tan \phi' = \frac{1}{2.68} \left[\log \left(\frac{q_c}{\sigma'_{vo}} \right) + 0.29 \right]$
Only SBT_n 5, 6, 7 & 8 Show 'N/A' in zones 1, 2, 3, 4 & 9

11 Young's modulus, E_s $E_s = \alpha q_t$
Only SBT_n 5, 6, 7 & 8 Show 'N/A' in zones 1, 2, 3, 4 & 9

12 Small strain shear modulus, G_o
a. $G_o = S_G (q_t \sigma'_{vo} p_a)^{1/3}$ For SBT_n 5, 6, 7
b. $G_o = C_G q_t$ For SBT_n 1, 2, 3 & 4
Show 'N/A' in zones 8 & 9

13 Undrained shear strength, s_u $s_u = (q_t - \sigma_{vo}) / N_{kt}$
Only SBT_n 1, 2, 3, 4 & 9 Show 'N/A' in zones 5, 6, 7 & 8

14 Over Consolidation ratio, OCR $\text{OCR} = k_{ocr} Q_{t1}$
Only SBT_n 1, 2, 3, 4 & 9 Show 'N/A' in zones 5, 6, 7 & 8

The following updated and simplified SBT descriptions have been used in the software:

SBT Zones

- 1 sensitive fine grained
- 2 organic soil
- 3 clay
- 4 clay & silty clay
- 5 clay & silty clay
- 6 sandy silt & clayey silt

SBT_n Zones

- 1 sensitive fine grained
- 2 organic soil
- 3 clay
- 4 clay & silty clay

7	silty sand & sandy silt	5	silty sand & sandy silt
8	sand & silty sand	6	sand & silty sand
9	sand		
10	sand	7	sand
11	very dense/stiff soil*	8	very dense/stiff soil*
12	very dense/stiff soil*	9	very dense/stiff soil*

*heavily overconsolidated and/or cemented

Track when soils fall with zones of same description and print that description (i.e. if soils fall only within SBT zones 4 & 5, print 'clays & silty clays')

Estimated Permeability (see Lunne et al., 1997)

SBT _n	Permeability (ft/sec)	(m/sec)
1	3×10^{-8}	1×10^{-8}
2	3×10^{-7}	1×10^{-7}
3	1×10^{-9}	3×10^{-10}
4	3×10^{-8}	1×10^{-8}
5	3×10^{-6}	1×10^{-6}
6	3×10^{-4}	1×10^{-4}
7	3×10^{-2}	1×10^{-2}
8	3×10^{-6}	1×10^{-6}
9	1×10^{-8}	3×10^{-9}

Estimated Unit Weight (see Lunne et al., 1997)

SBT	Approximate Unit Weight (lb/ft ³)	(kN/m ³)
1	111.4	17.5
2	79.6	12.5
3	111.4	17.5
4	114.6	18.0
5	114.6	18.0
6	114.6	18.0
7	117.8	18.5
8	120.9	19.0
9	124.1	19.5
10	127.3	20.0
11	130.5	20.5
12	120.9	19.0

Pore Pressure Dissipation Tests (PPDT)

Pore Pressure Dissipation Tests (PPDT's) conducted at various intervals can be used to measure equilibrium water pressure (at the time of the CPT). If conditions are hydrostatic, the equilibrium water pressure can be used to determine the approximate depth of the ground water table. A PPDT is conducted when penetration is halted at specific intervals determined by the field representative. The variation of the penetration pore pressure (u) with time is measured behind the tip of the cone and recorded.

Pore pressure dissipation data can be interpreted to provide estimates of:

- Equilibrium piezometric pressure
- Phreatic Surface
- In situ horizontal coefficient of consolidation (c_h)
- In situ horizontal coefficient of permeability (k_h)

In order to correctly interpret the equilibrium piezometric pressure and/or the phreatic surface, the pore pressure must be monitored until it reaches equilibrium, *Figure PPDT*. This time is commonly referred to as t_{100} , the point at which 100% of the excess pore pressure has dissipated.

A complete reference on pore pressure dissipation tests is presented by Robertson et al. 1992 and Lunne et al. 1997.

A summary of the pore pressure dissipation tests are summarized in Table 1.

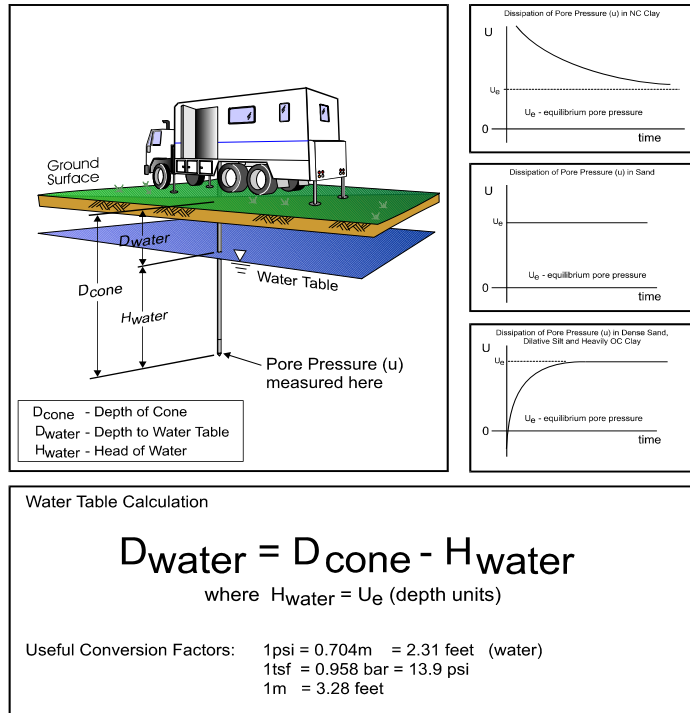


Figure PPDT

Seismic Cone Penetration Testing (SCPT)

Seismic Cone Penetration Testing (SCPT) can be conducted at various intervals during the Cone Penetration Test. Shear wave velocity (V_s) can then be calculated over a specified interval with depth. A small interval for seismic testing, such as 1-1.5m (3-5ft) allows for a detailed look at the shear wave profile with depth. Conversely, a larger interval such as 3-6m (10-20ft) allows for a more average shear wave velocity to be calculated. Gregg's cones have a horizontally active geophone located 0.2m (0.66ft) behind the tip.

To conduct the seismic shear wave test, the penetration of the cone is stopped and the rods are decoupled from the rig. An automatic hammer is triggered to send a shear wave into the soil. The distance from the source to the cone is calculated knowing the total depth of the cone and the horizontal offset distance between the source and the cone. To calculate an interval velocity, a minimum of two tests must be performed at two different depths. The arrival times between the two wave traces are compared to obtain the difference in time (Δt). The difference in depth is calculated (Δd) and velocity can be determined using the simple equation: $v = \Delta d / \Delta t$

Multiple wave traces can be recorded at the same depth to improve quality of the data.

A complete reference on seismic cone penetration tests is presented by Robertson et al. 1986 and Lunne et al. 1997.

A summary the shear wave velocities, arrival times and wave traces are provided with the report.

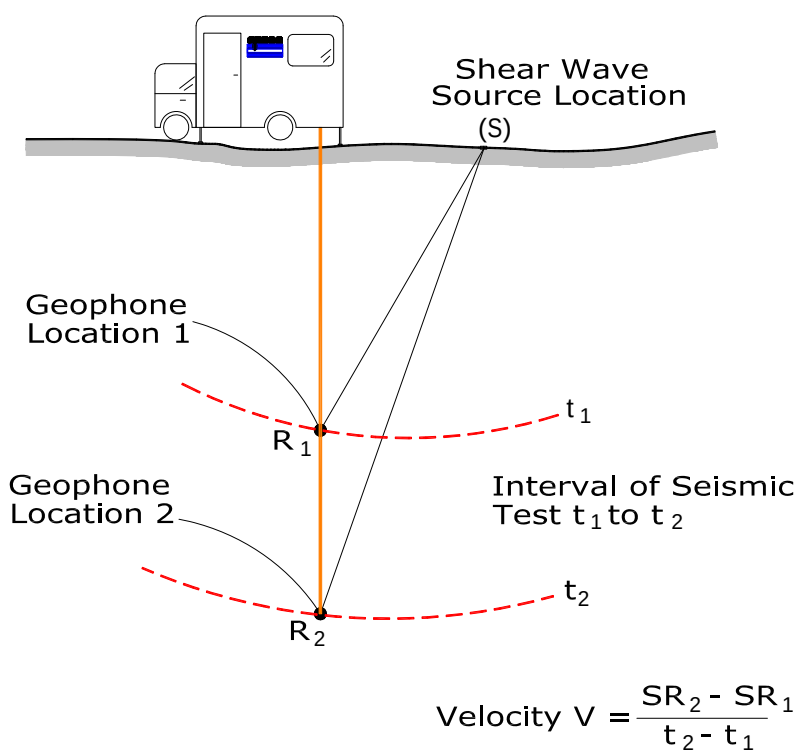


Figure SCPT

Groundwater Sampling

Gregg Drilling & Testing, Inc. conducts groundwater sampling using a sampler as shown in *Figure GWS*. The groundwater sampler has a retrievable stainless steel or disposable PVC screen with steel drop off tip. This allows for samples to be taken at multiple depth intervals within the same sounding location. In areas of slower water recharge, provisions may be made to set temporary PVC well screens during sampling to allow the pushing equipment to advance to the next sample location while the groundwater is allowed to infiltrate.

The groundwater sampler operates by advancing 44.5mm (1¾ inch) hollow push rods with the filter tip in a closed configuration to the base of the desired sampling interval. Once at the desired sample depth, the push rods are retracted; exposing the encased filter screen and allowing groundwater to infiltrate hydrostatically from the formation into the inlet screen. A small diameter bailer (approximately ½ or ¾ inch) is lowered through the push rods into the screen section for sample collection. The number of downhole trips with the bailer and time necessary to complete the sample collection at each depth interval is a function of sampling protocols, volume requirements, and the yield characteristics and storage capacity of the formation. Upon completion of sample collection, the push rods and sampler, with the exception of the PVC screen and steel drop off tip are retrieved to the ground surface, decontaminated and prepared for the next sampling event.

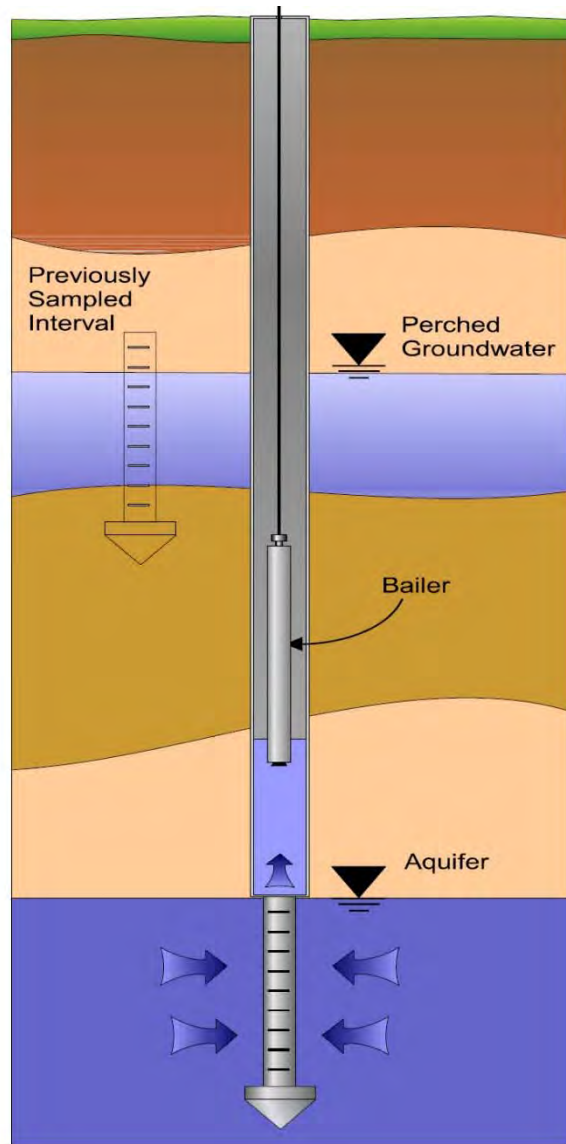


Figure GWS

For a detailed reference on direct push groundwater sampling, refer to Zemo et. al., 1992.

Soil Sampling

Gregg Drilling & Testing, Inc. uses a piston-type push-in sampler to obtain small soil samples without generating any soil cuttings, *Figure SS*. Two different types of samplers (12 and 18 inch) are used depending on the soil type and density. The soil sampler is initially pushed in a "closed" position to the desired sampling interval using the CPT pushing equipment. Keeping the sampler closed minimizes the potential of cross contamination. The inner tip of the sampler is then retracted leaving a hollow soil sampler with inner 1¼" diameter sample tubes. The hollow sampler is then pushed in a locked "open" position to collect a soil sample. The filled sampler and push rods are then retrieved to the ground surface. Because the soil enters the sampler at a constant rate, the opportunity for 100% recovery is increased. For environmental analysis, the soil sample tube ends are sealed with Teflon and plastic caps. Often, a longer "split tube" can be used for geotechnical sampling.

For a detailed reference on direct push soil sampling, refer to Robertson et al, 1998.

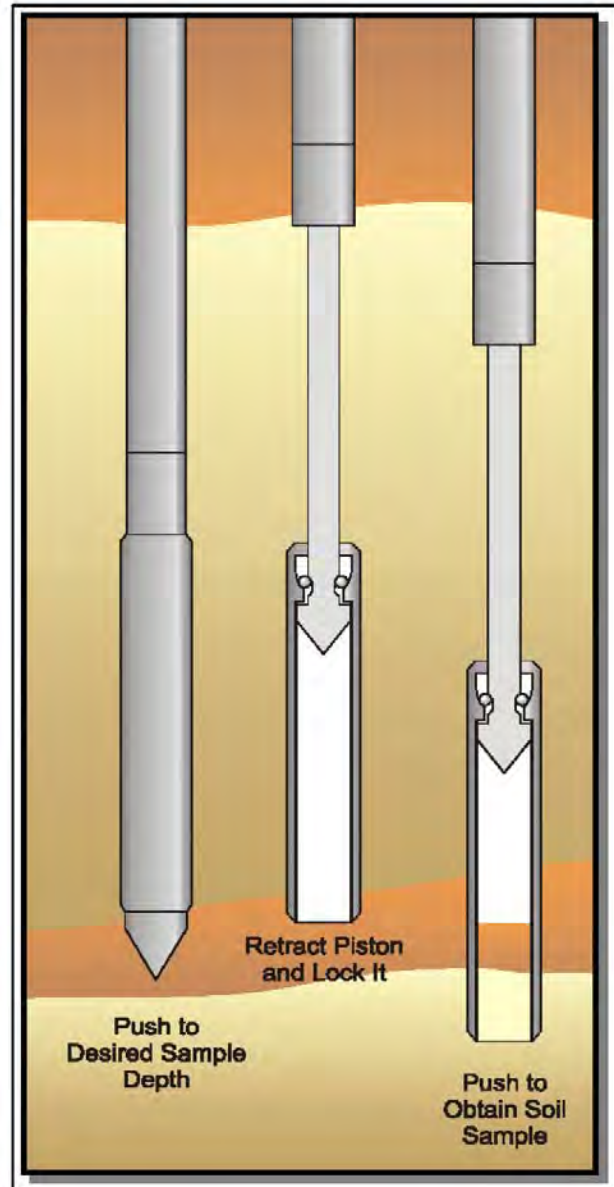


Figure SS

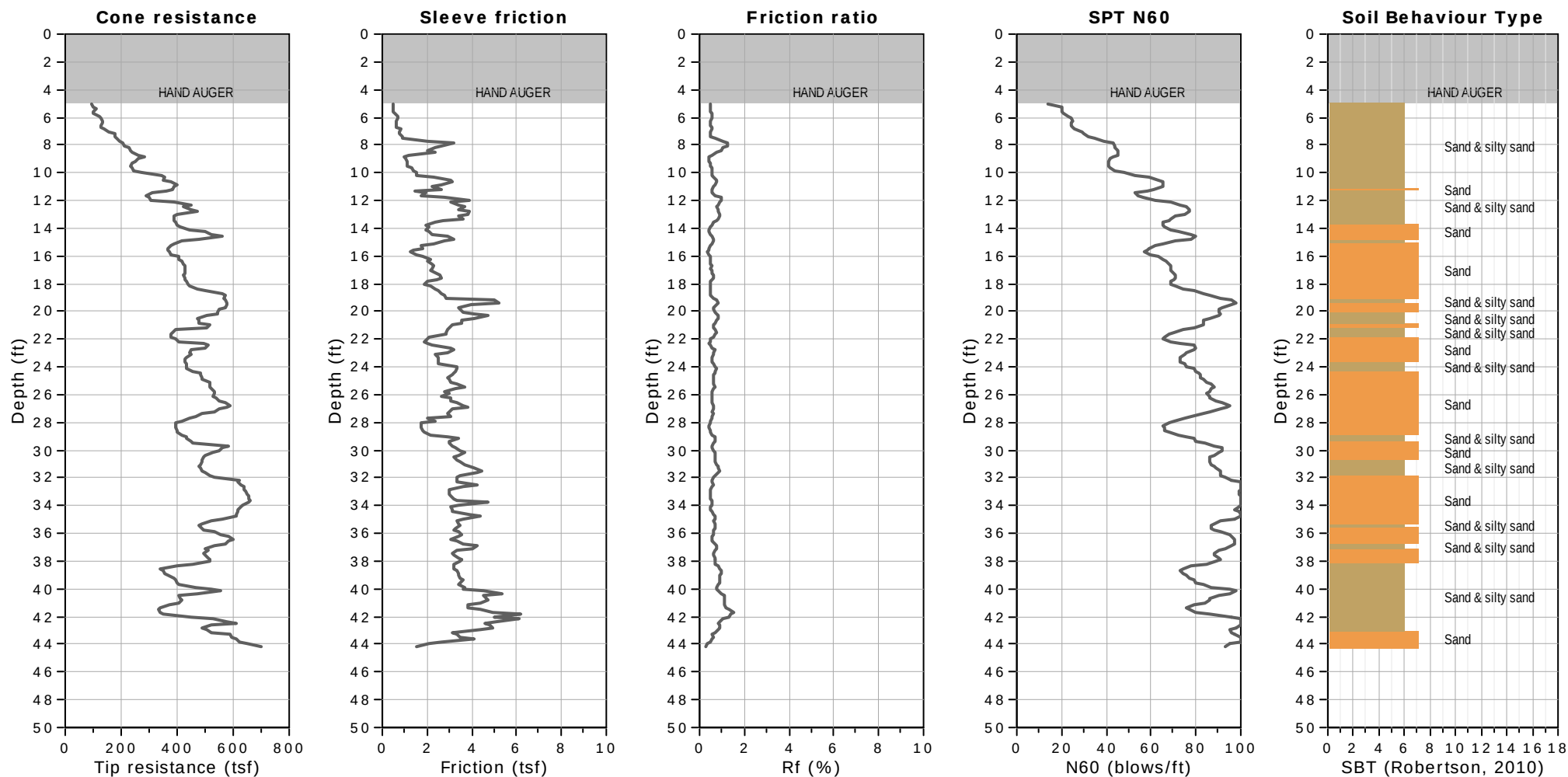


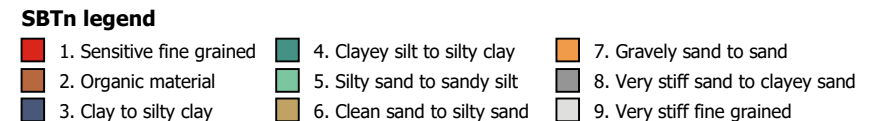
CLIENT: R.T. FRANKIAN & ASSOCIATES

SITE: CSC LID SITE X - SANTA CLARITA, CA

Field Rep: JIM

Total depth: 44.13 ft, Date: 10/30/2018



**WATER TABLE FOR ESTIMATING PURPOSES ONLY**

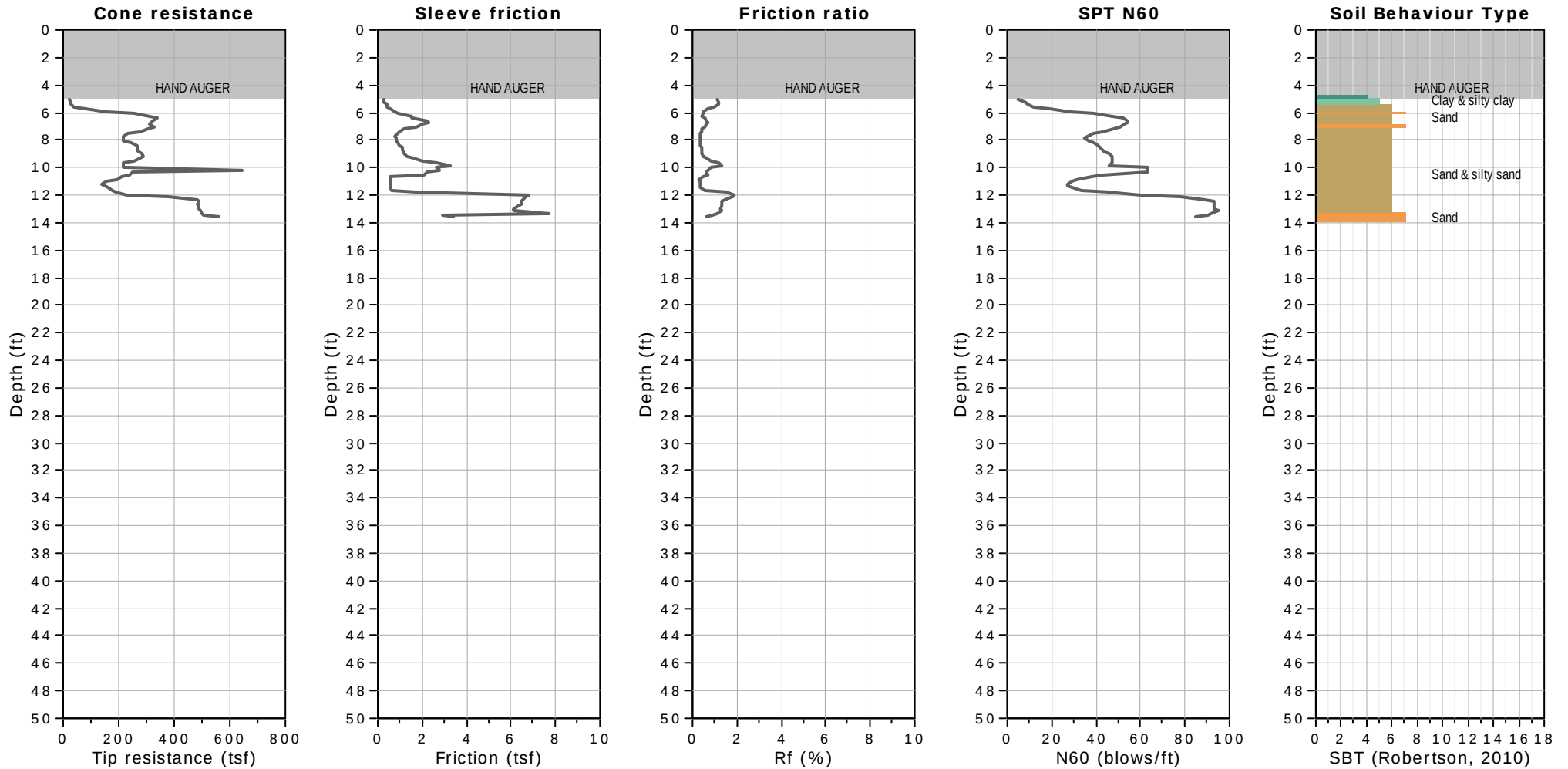


CLIENT: R.T. FRANKIAN & ASSOCIATES

SITE: CSC LID SITE X - SANTA CLARITA, CA

Field Rep: JIM

Total depth: 13.62 ft, Date: 10/30/2018



SBTn legend

- | | | |
|---------------------------|------------------------------|-----------------------------------|
| 1. Sensitive fine grained | 4. Clayey silt to silty clay | 7. Gravely sand to sand |
| 2. Organic material | 5. Silty sand to sandy silt | 8. Very stiff sand to clayey sand |
| 3. Clay to silty clay | 6. Clean sand to silty sand | 9. Very stiff fine grained |

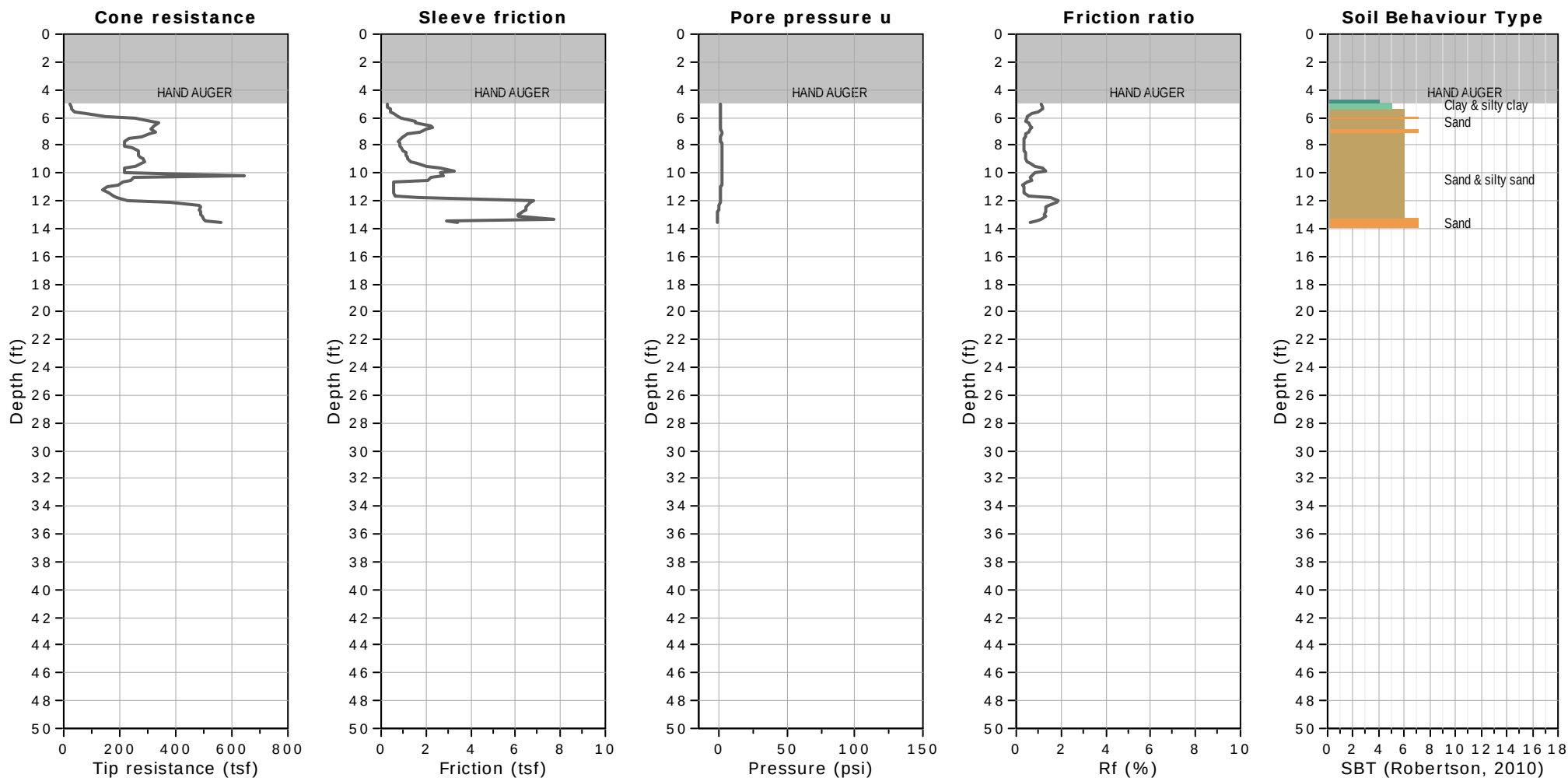


CLIENT: R.T. FRANKIAN & ASSOCIATES

SITE: CSC LID SITE X - SANTA CLARITA, CA

Field Rep: JIM

Total depth: 13.62 ft, Date: 10/30/2018



SBTn legend

- | | | |
|---------------------------|------------------------------|-----------------------------------|
| 1. Sensitive fine grained | 4. Clayey silt to silty clay | 7. Gravely sand to sand |
| 2. Organic material | 5. Silty sand to sandy silt | 8. Very stiff sand to clayey sand |
| 3. Clay to silty clay | 6. Clean sand to silty sand | 9. Very stiff fine grained |

WATER TABLE FOR ESTIMATING PURPOSES ONLY

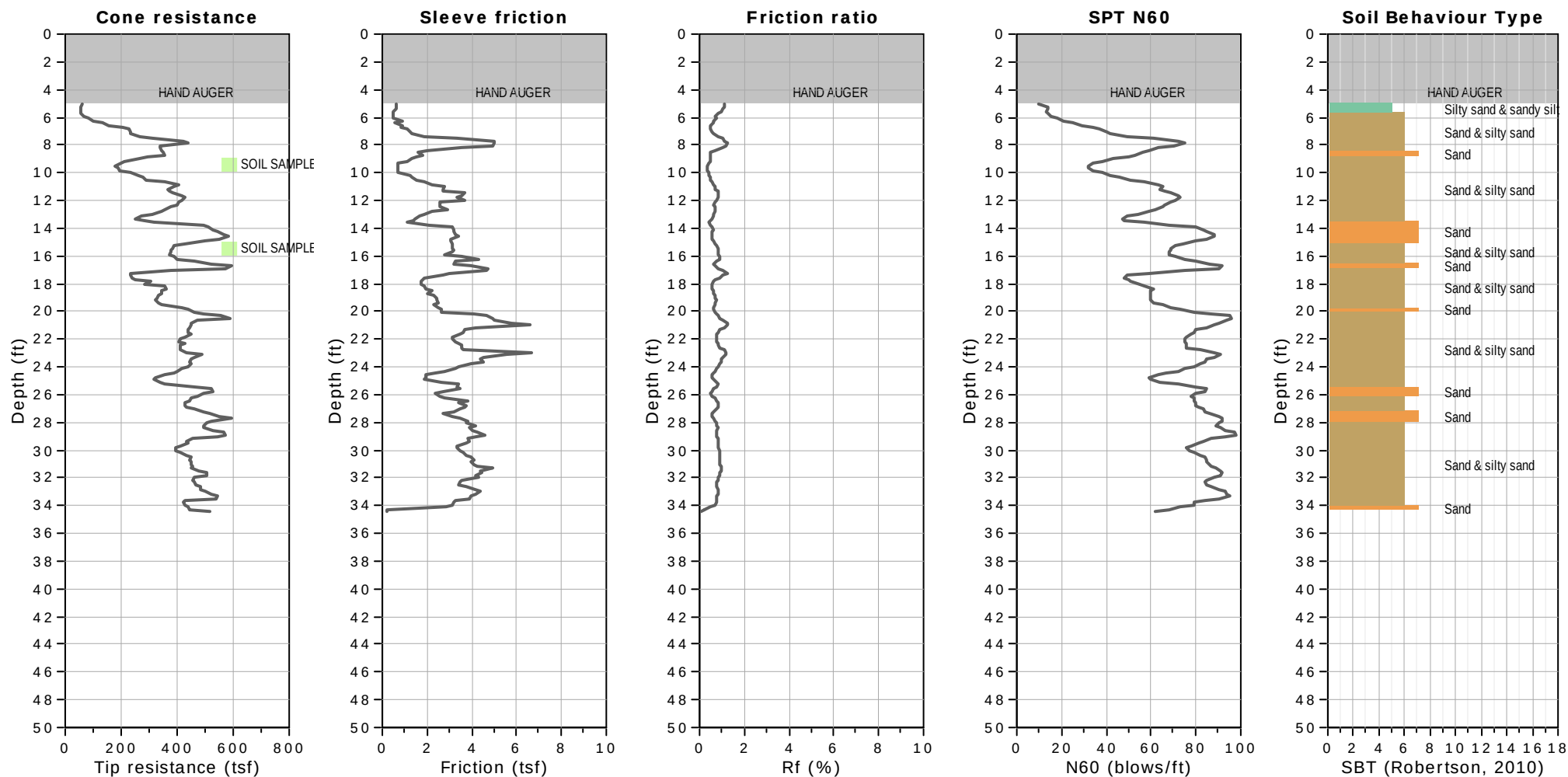


CLIENT: R.T. FRANKIAN & ASSOCIATES

SITE: CSC LID SITE X - SANTA CLARITA, CA

Field Rep: JIM

Total depth: 34.45 ft, Date: 10/30/2018



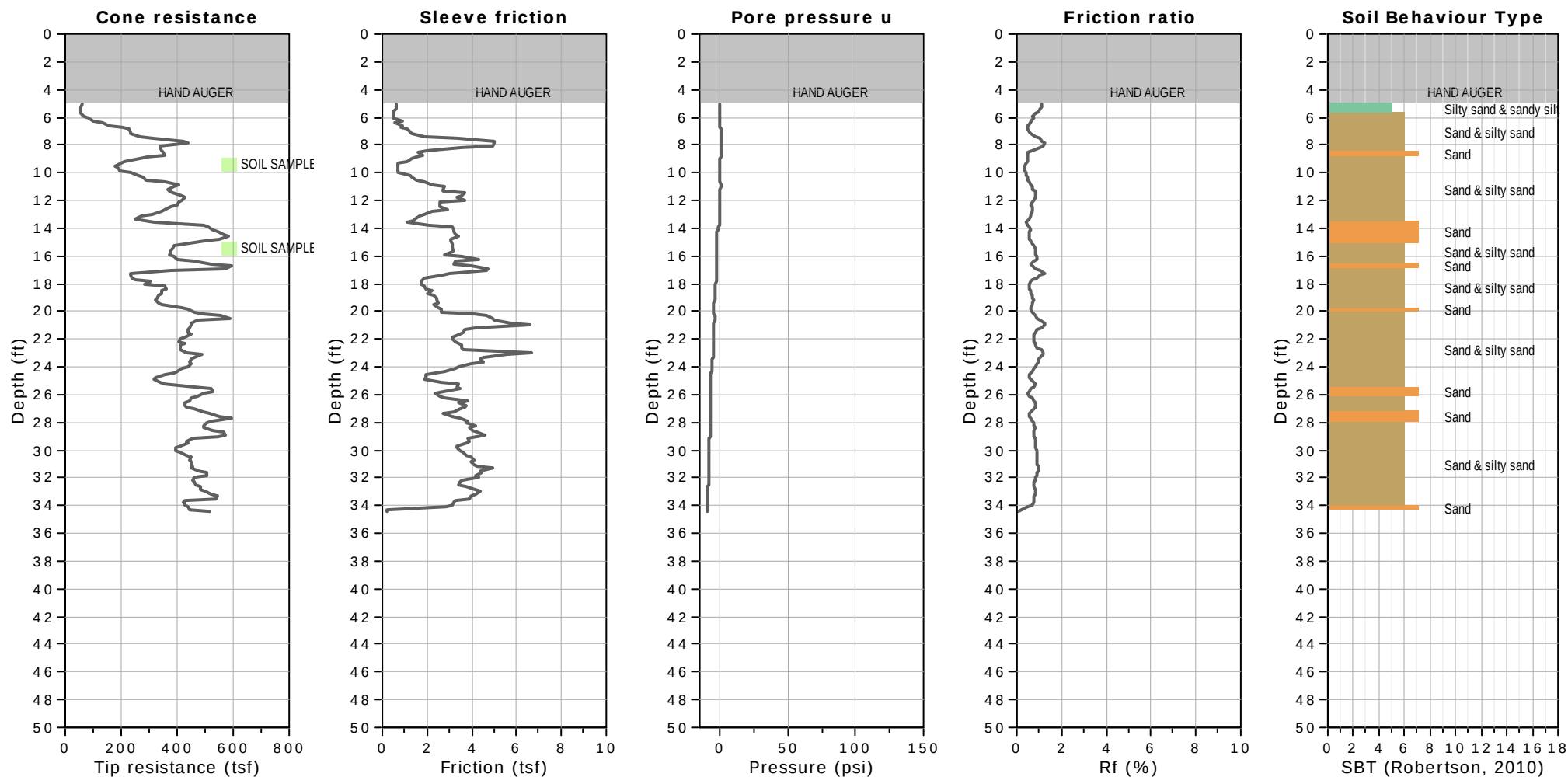


CLIENT: R.T. FRANKIAN & ASSOCIATES

SITE: CSC LID SITE X - SANTA CLARITA, CA

Field Rep: JIM

Total depth: 34.45 ft, Date: 10/30/2018



SBTn legend

- | | | |
|---------------------------|------------------------------|-----------------------------------|
| 1. Sensitive fine grained | 4. Clayey silt to silty clay | 7. Gravely sand to sand |
| 2. Organic material | 5. Silty sand to sandy silt | 8. Very stiff sand to clayey sand |
| 3. Clay to silty clay | 6. Clean sand to silty sand | 9. Very stiff fine grained |

WATER TABLE FOR ESTIMATING PURPOSES ONLY

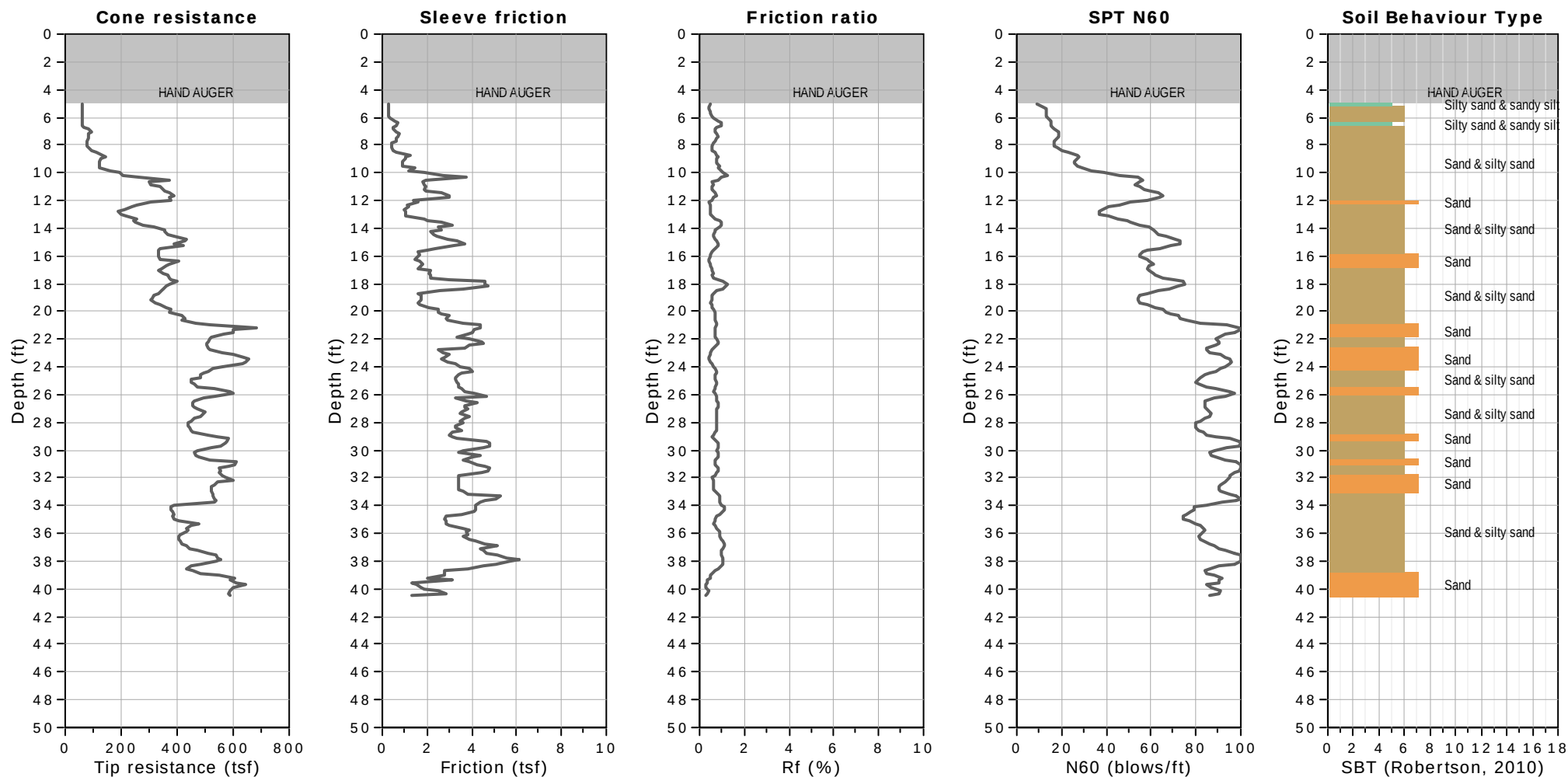


CLIENT: R.T. FRANKIAN & ASSOCIATES

SITE: CSC LID SITE X - SANTA CLARITA, CA

Field Rep: JIM

Total depth: 40.52 ft, Date: 10/30/2018



SBTn legend

- | | | |
|---------------------------|------------------------------|-----------------------------------|
| 1. Sensitive fine grained | 4. Clayey silt to silty clay | 7. Gravely sand to sand |
| 2. Organic material | 5. Silty sand to sandy silt | 8. Very stiff sand to clayey sand |
| 3. Clay to silty clay | 6. Clean sand to silty sand | 9. Very stiff fine grained |

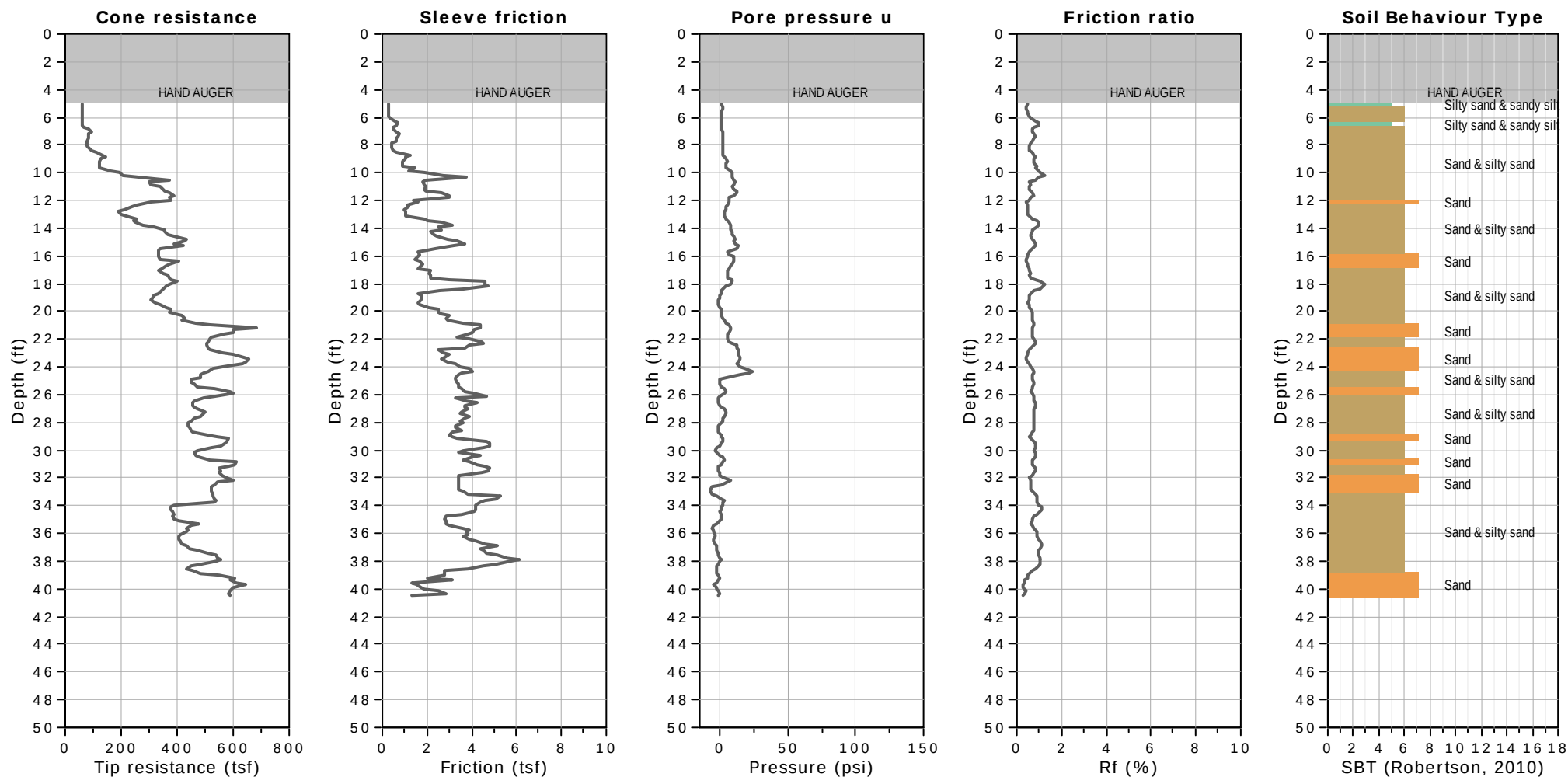


CLIENT: R.T. FRANKIAN & ASSOCIATES

SITE: CSC LID SITE X - SANTA CLARITA, CA

Field Rep: JIM

Total depth: 40.52 ft, Date: 10/30/2018



SBTn legend

- | | | |
|---------------------------|------------------------------|-----------------------------------|
| 1. Sensitive fine grained | 4. Clayey silt to silty clay | 7. Gravely sand to sand |
| 2. Organic material | 5. Silty sand to sandy silt | 8. Very stiff sand to clayey sand |
| 3. Clay to silty clay | 6. Clean sand to silty sand | 9. Very stiff fine grained |

WATER TABLE FOR ESTIMATING PURPOSES ONLY

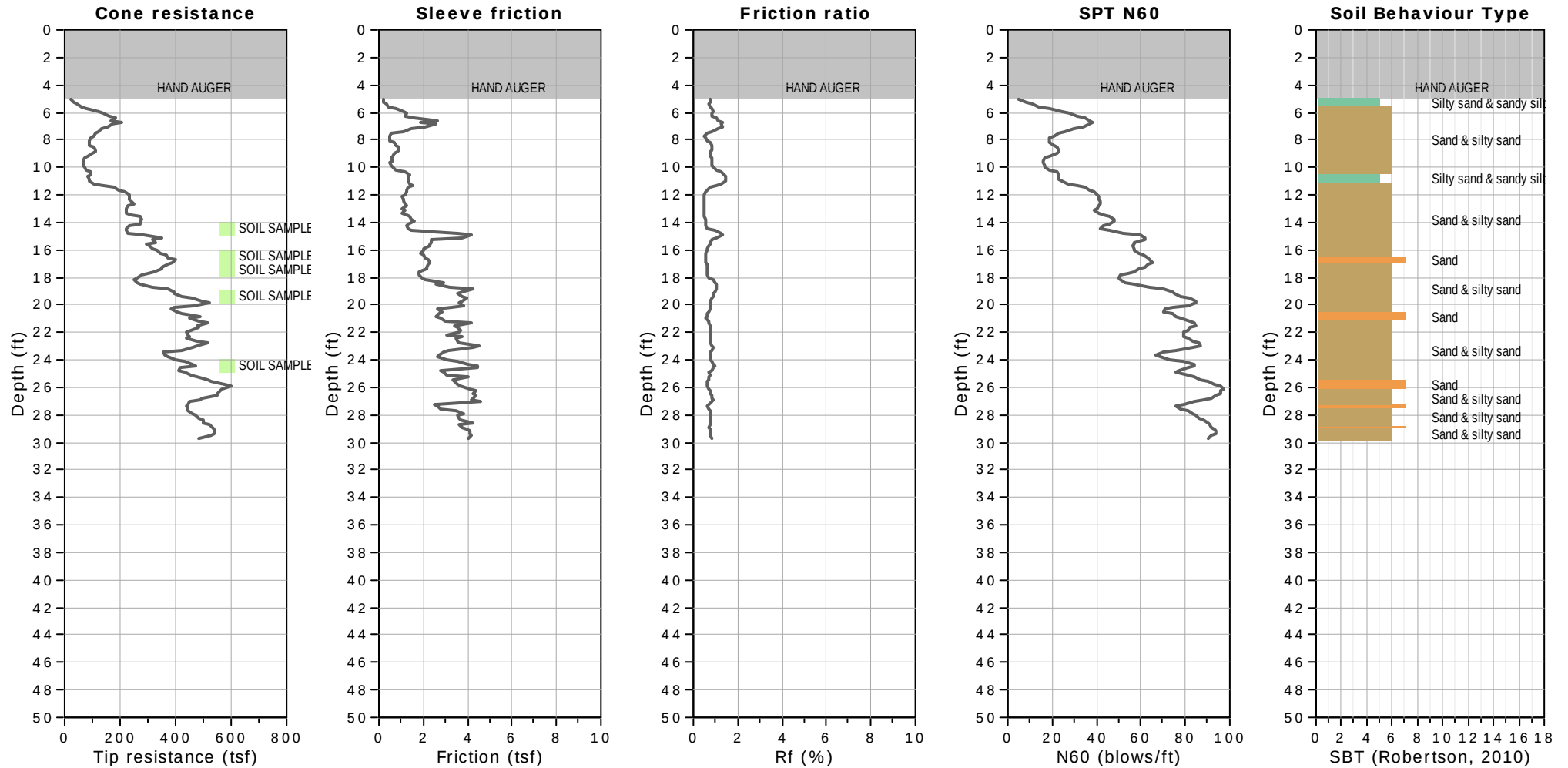


CLIENT: R.T. FRANKIAN & ASSOCIATES

SITE: CSC LID SITE X - SANTA CLARITA, CA

Field Rep: JIM

Total depth: 29.69 ft, Date: 10/30/2018



SBTn legend

- | | | |
|---------------------------|------------------------------|-----------------------------------|
| 1. Sensitive fine grained | 4. Clayey silt to silty clay | 7. Gravely sand to sand |
| 2. Organic material | 5. Silty sand to sandy silt | 8. Very stiff sand to clayey sand |
| 3. Clay to silty clay | 6. Clean sand to silty sand | 9. Very stiff fine grained |

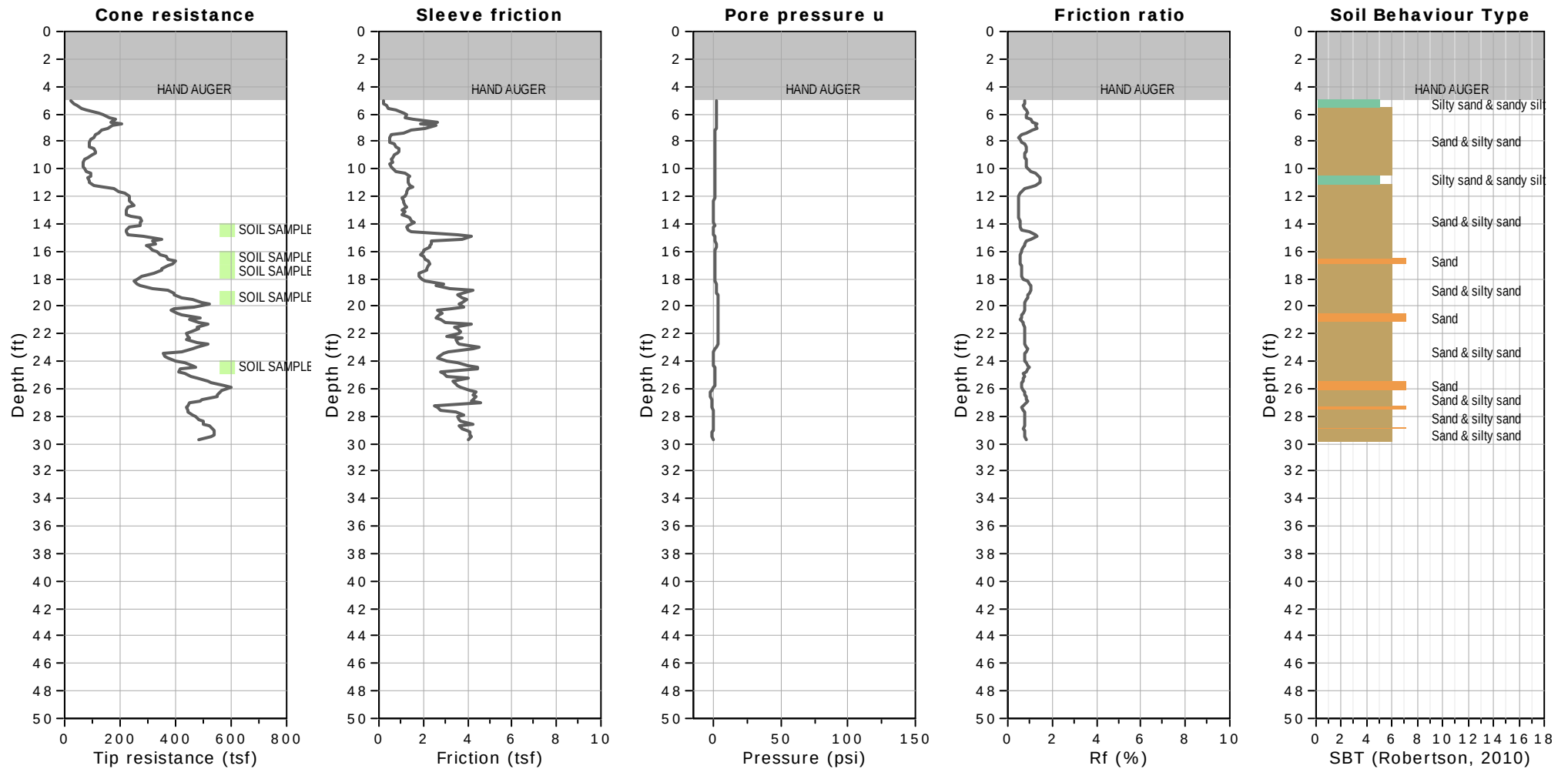


CLIENT: R.T. FRANKIAN & ASSOCIATES

SITE: CSC LID SITE X - SANTA CLARITA, CA

Field Rep: JIM

Total depth: 29.69 ft, Date: 10/30/2018



SBTn legend

- | | | |
|---------------------------|------------------------------|-----------------------------------|
| 1. Sensitive fine grained | 4. Clayey silt to silty clay | 7. Gravely sand to sand |
| 2. Organic material | 5. Silty sand to sandy silt | 8. Very stiff sand to clayey sand |
| 3. Clay to silty clay | 6. Clean sand to silty sand | 9. Very stiff fine grained |

WATER TABLE FOR ESTIMATING PURPOSES ONLY

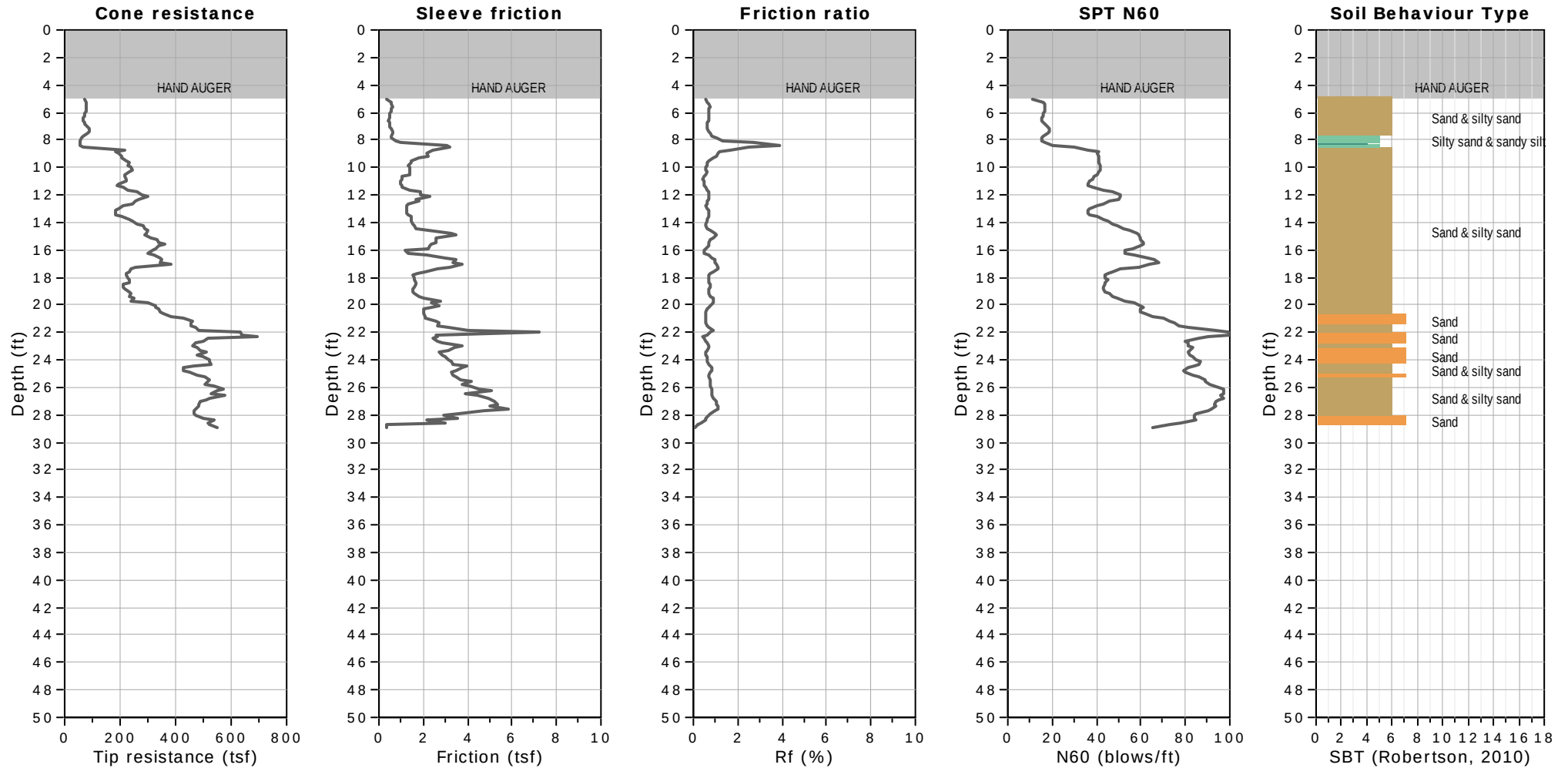


CLIENT: R.T. FRANKIAN & ASSOCIATES

SITE: CSC LID SITE X - SANTA CLARITA, CA

Field Rep: JIM

Total depth: 28.87 ft, Date: 10/31/2018



SBTn legend

- | | | |
|---------------------------|------------------------------|-----------------------------------|
| 1. Sensitive fine grained | 4. Clayey silt to silty clay | 7. Gravely sand to sand |
| 2. Organic material | 5. Silty sand to sandy silt | 8. Very stiff sand to clayey sand |
| 3. Clay to silty clay | 6. Clean sand to silty sand | 9. Very stiff fine grained |

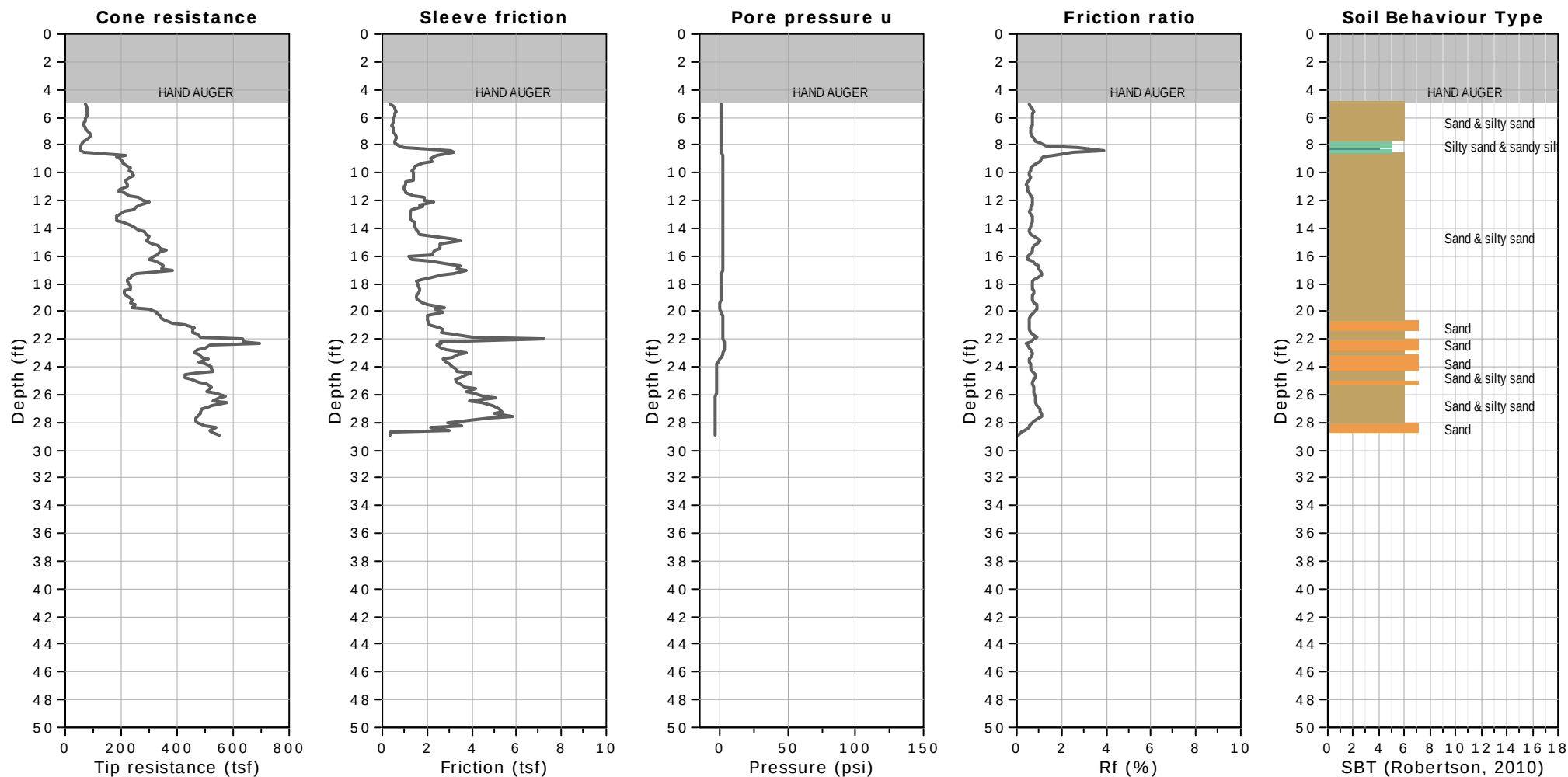


CLIENT: R.T. FRANKIAN & ASSOCIATES

SITE: CSC LID SITE X - SANTA CLARITA, CA

Field Rep: JIM

Total depth: 28.87 ft, Date: 10/31/2018



SBTn legend

- | | | |
|---------------------------|------------------------------|-----------------------------------|
| 1. Sensitive fine grained | 4. Clayey silt to silty clay | 7. Gravely sand to sand |
| 2. Organic material | 5. Silty sand to sandy silt | 8. Very stiff sand to clayey sand |
| 3. Clay to silty clay | 6. Clean sand to silty sand | 9. Very stiff fine grained |

WATER TABLE FOR ESTIMATING PURPOSES ONLY

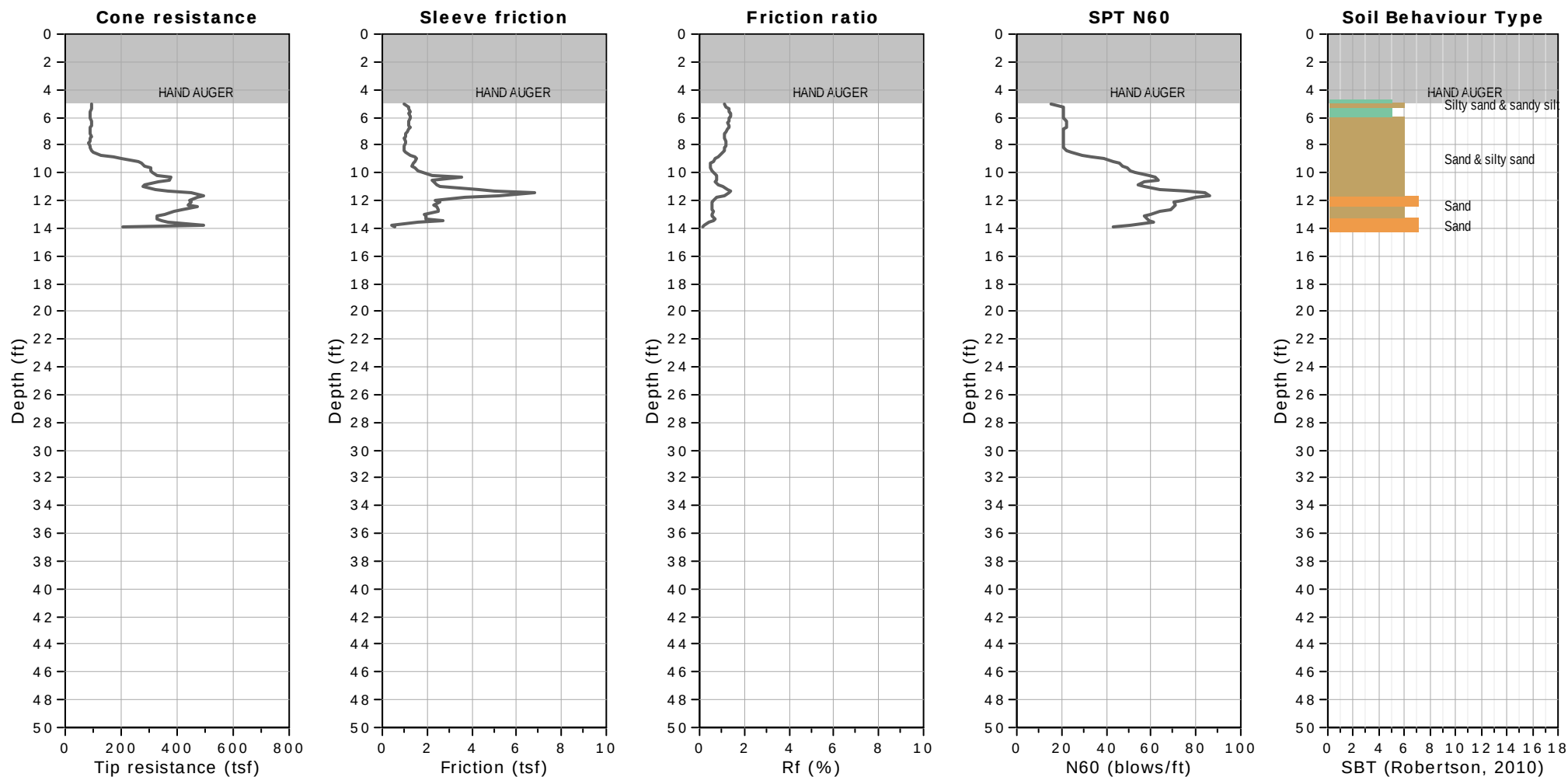


CLIENT: R.T. FRANKIAN & ASSOCIATES

SITE: CSC LID SITE X - SANTA CLARITA, CA

Field Rep: JIM

Total depth: 13.94 ft, Date: 10/31/2018



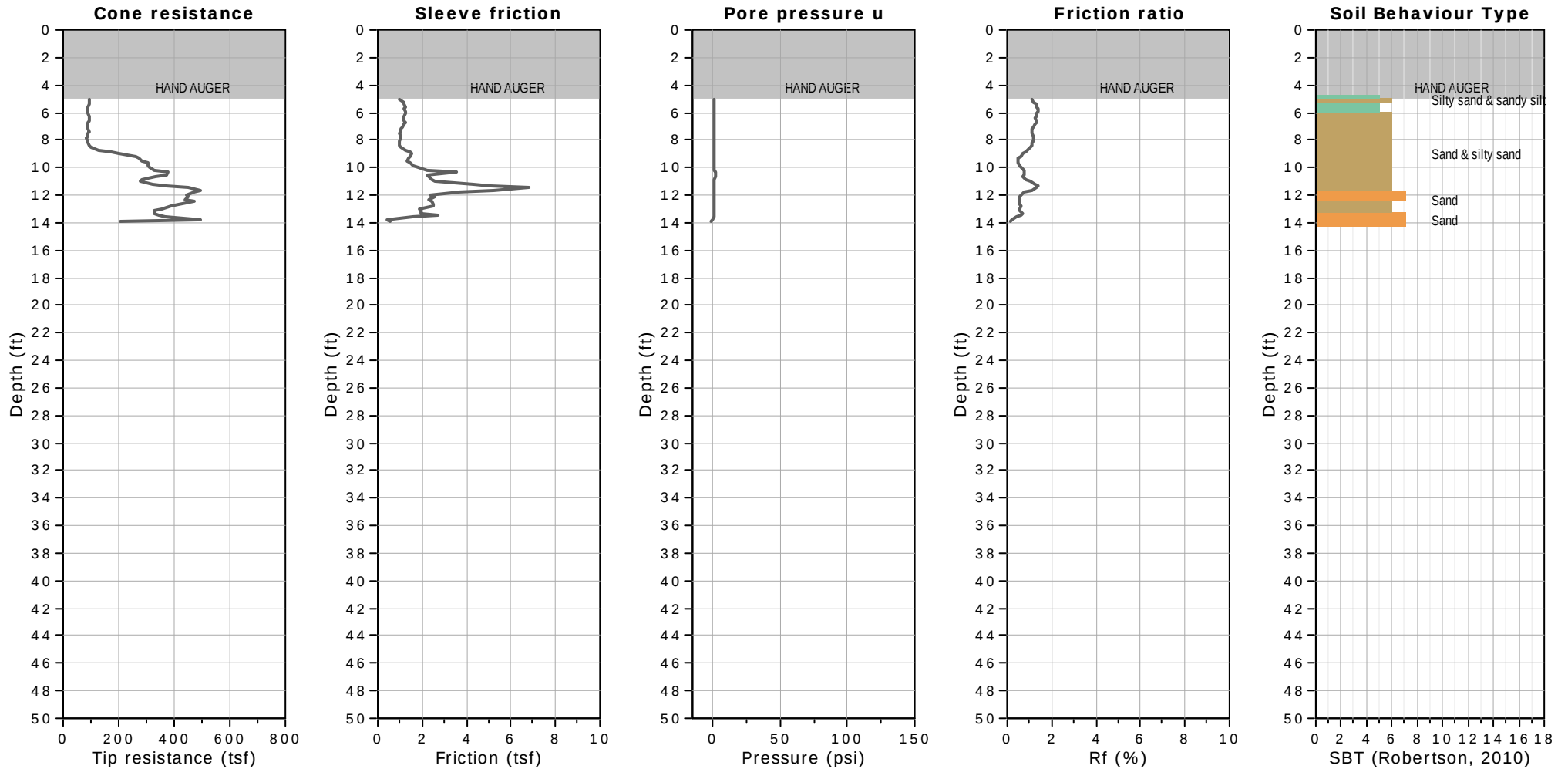


CLIENT: R.T. FRANKIAN & ASSOCIATES

SITE: CSC LID SITE X - SANTA CLARITA, CA

Field Rep: JIM

Total depth: 13.94 ft, Date: 10/31/2018



WATER TABLE FOR ESTIMATING PURPOSES ONLY

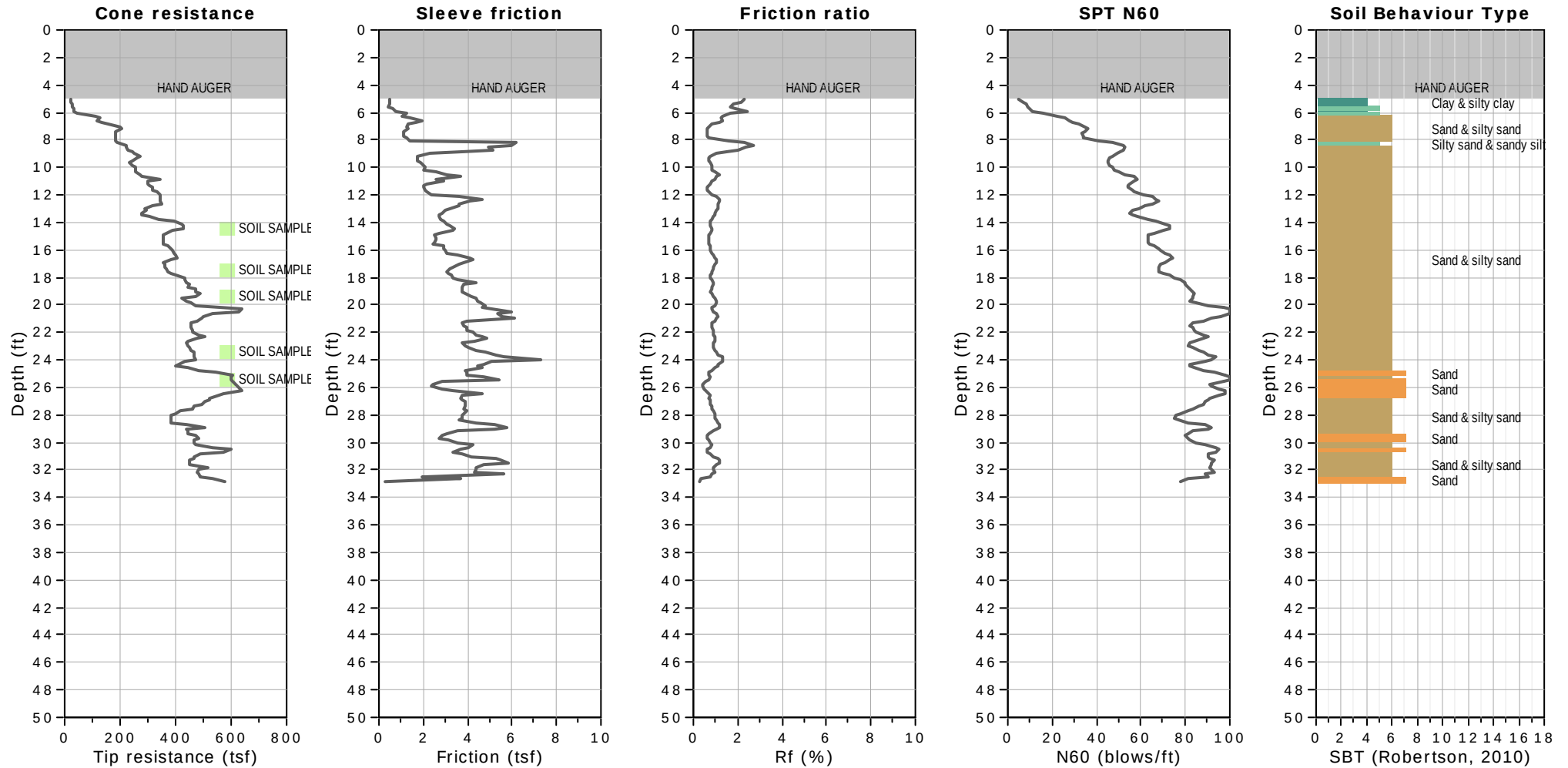


CLIENT: R.T. FRANKIAN & ASSOCIATES

SITE: CSC LID SITE X - SANTA CLARITA, CA

Field Rep: JIM

Total depth: 32.81 ft, Date: 10/31/2018



SBTn legend

- | | | |
|---------------------------|------------------------------|-----------------------------------|
| 1. Sensitive fine grained | 4. Clayey silt to silty clay | 7. Gravely sand to sand |
| 2. Organic material | 5. Silty sand to sandy silt | 8. Very stiff sand to clayey sand |
| 3. Clay to silty clay | 6. Clean sand to silty sand | 9. Very stiff fine grained |

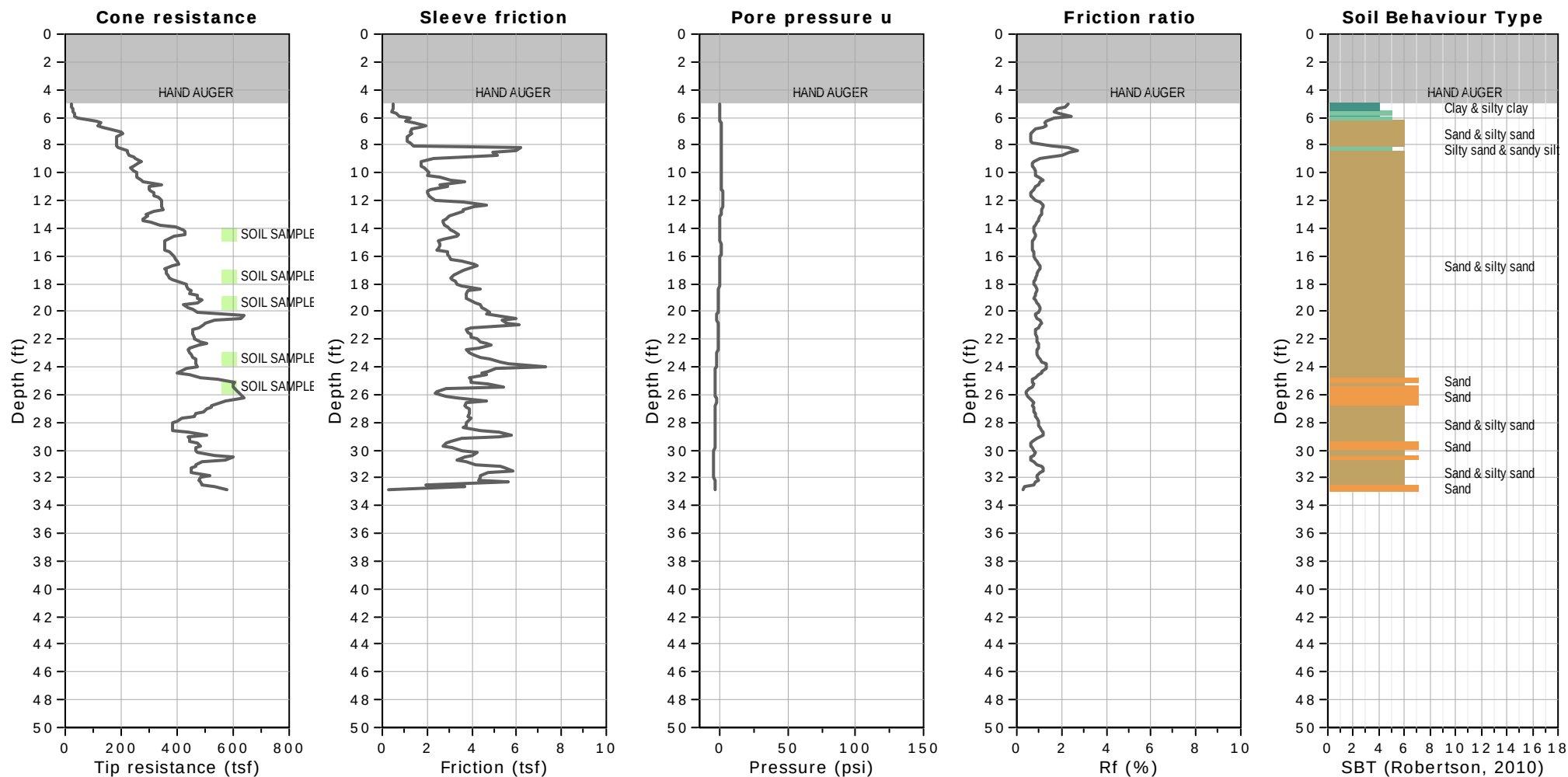


CLIENT: R.T. FRANKIAN & ASSOCIATES

SITE: CSC LID SITE X - SANTA CLARITA, CA

Field Rep: JIM

Total depth: 32.81 ft, Date: 10/31/2018



SBTn legend

- | | | |
|---------------------------|------------------------------|-----------------------------------|
| 1. Sensitive fine grained | 4. Clayey silt to silty clay | 7. Gravely sand to sand |
| 2. Organic material | 5. Silty sand to sandy silt | 8. Very stiff sand to clayey sand |
| 3. Clay to silty clay | 6. Clean sand to silty sand | 9. Very stiff fine grained |

WATER TABLE FOR ESTIMATING PURPOSES ONLY

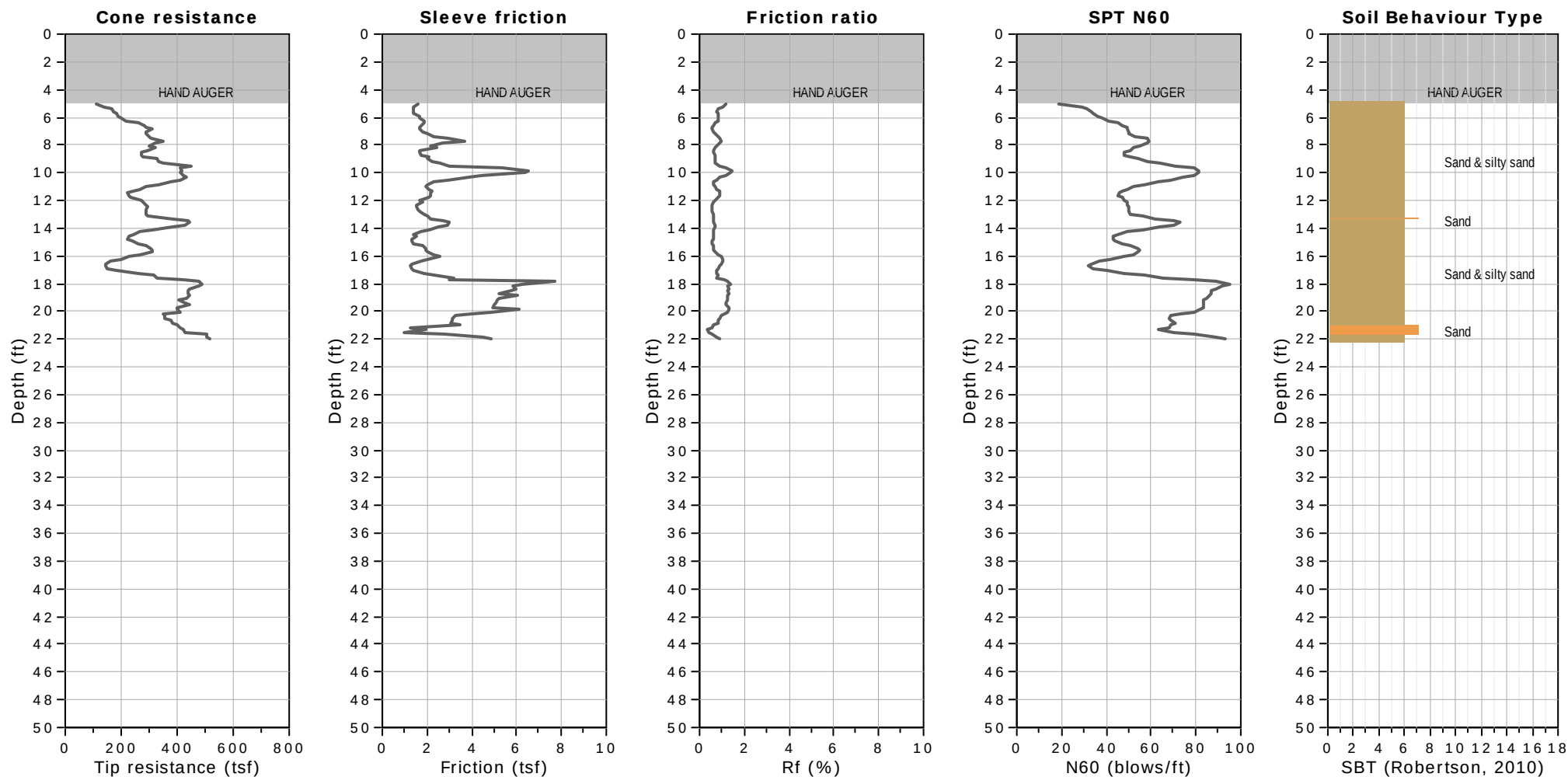


CLIENT: R.T. FRANKIAN & ASSOCIATES

SITE: CSC LID SITE X - SANTA CLARITA, CA

Field Rep: JIM

Total depth: 21.98 ft, Date: 10/31/2018



SBTn legend

- | | | |
|---------------------------|------------------------------|-----------------------------------|
| 1. Sensitive fine grained | 4. Clayey silt to silty clay | 7. Gravely sand to sand |
| 2. Organic material | 5. Silty sand to sandy silt | 8. Very stiff sand to clayey sand |
| 3. Clay to silty clay | 6. Clean sand to silty sand | 9. Very stiff fine grained |

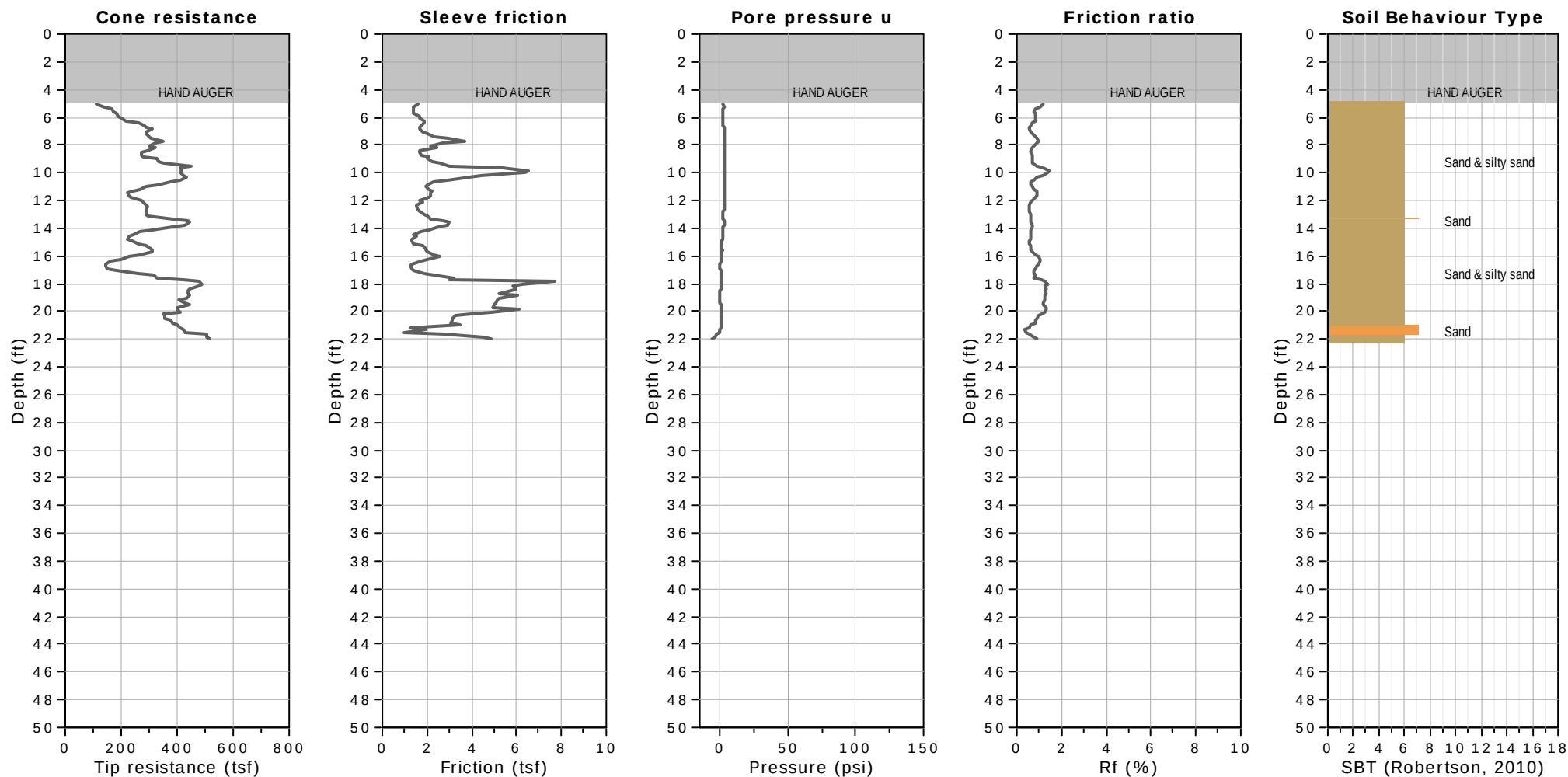


CLIENT: R.T. FRANKIAN & ASSOCIATES

SITE: CSC LID SITE X - SANTA CLARITA, CA

Field Rep: JIM

Total depth: 21.98 ft, Date: 10/31/2018



SBTn legend

- | | | |
|---------------------------|------------------------------|-----------------------------------|
| 1. Sensitive fine grained | 4. Clayey silt to silty clay | 7. Gravely sand to sand |
| 2. Organic material | 5. Silty sand to sandy silt | 8. Very stiff sand to clayey sand |
| 3. Clay to silty clay | 6. Clean sand to silty sand | 9. Very stiff fine grained |

WATER TABLE FOR ESTIMATING PURPOSES ONLY

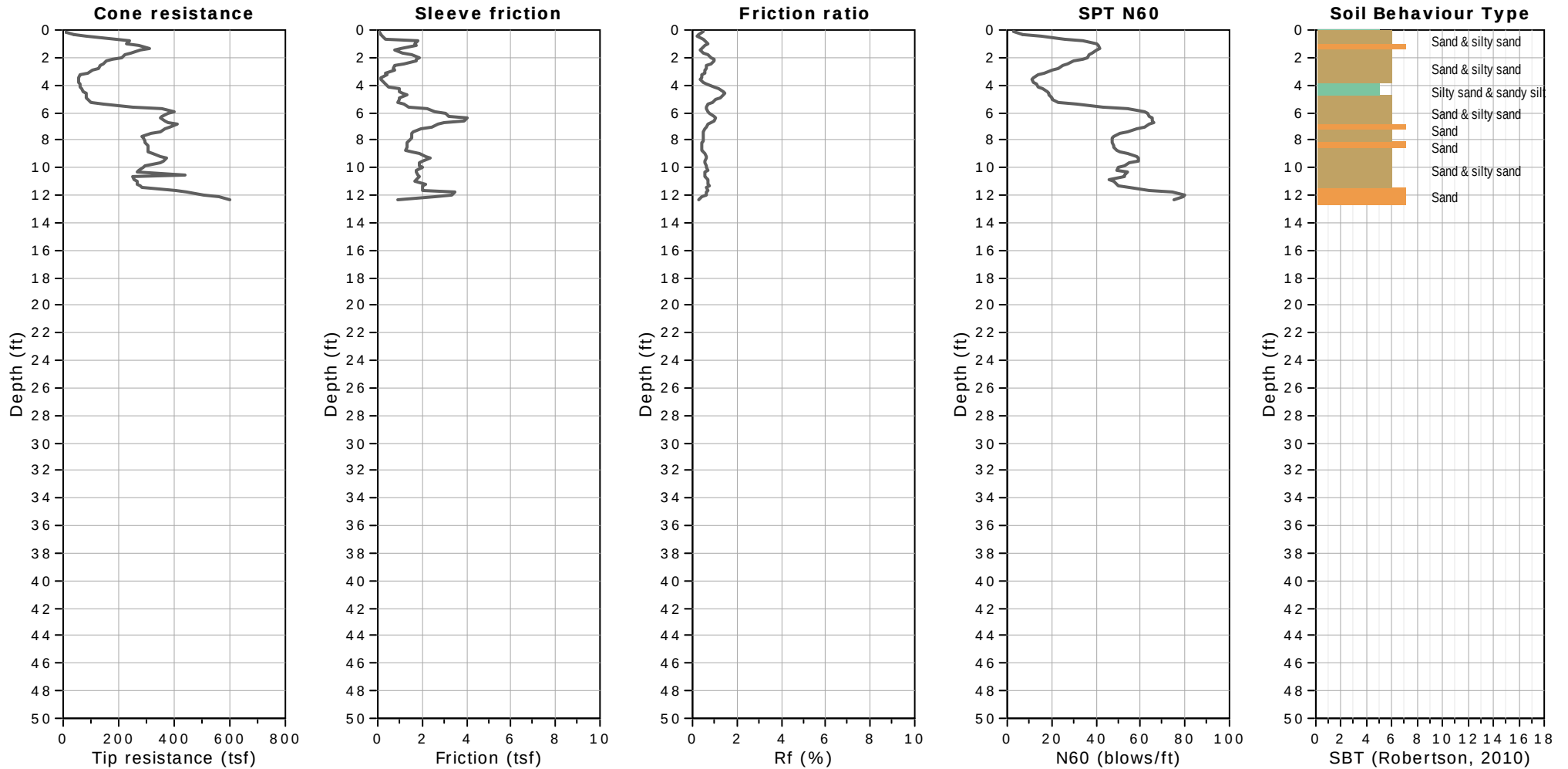


CLIENT: R.T. FRANKIAN & ASSOCIATES

SITE: CSC LID SITE X - SANTA CLARITA, CA

Field Rep: JIM

Total depth: 12.30 ft, Date: 11/1/2018



SBTn legend

- | | | |
|---------------------------|------------------------------|-----------------------------------|
| 1. Sensitive fine grained | 4. Clayey silt to silty clay | 7. Gravely sand to sand |
| 2. Organic material | 5. Silty sand to sandy silt | 8. Very stiff sand to clayey sand |
| 3. Clay to silty clay | 6. Clean sand to silty sand | 9. Very stiff fine grained |

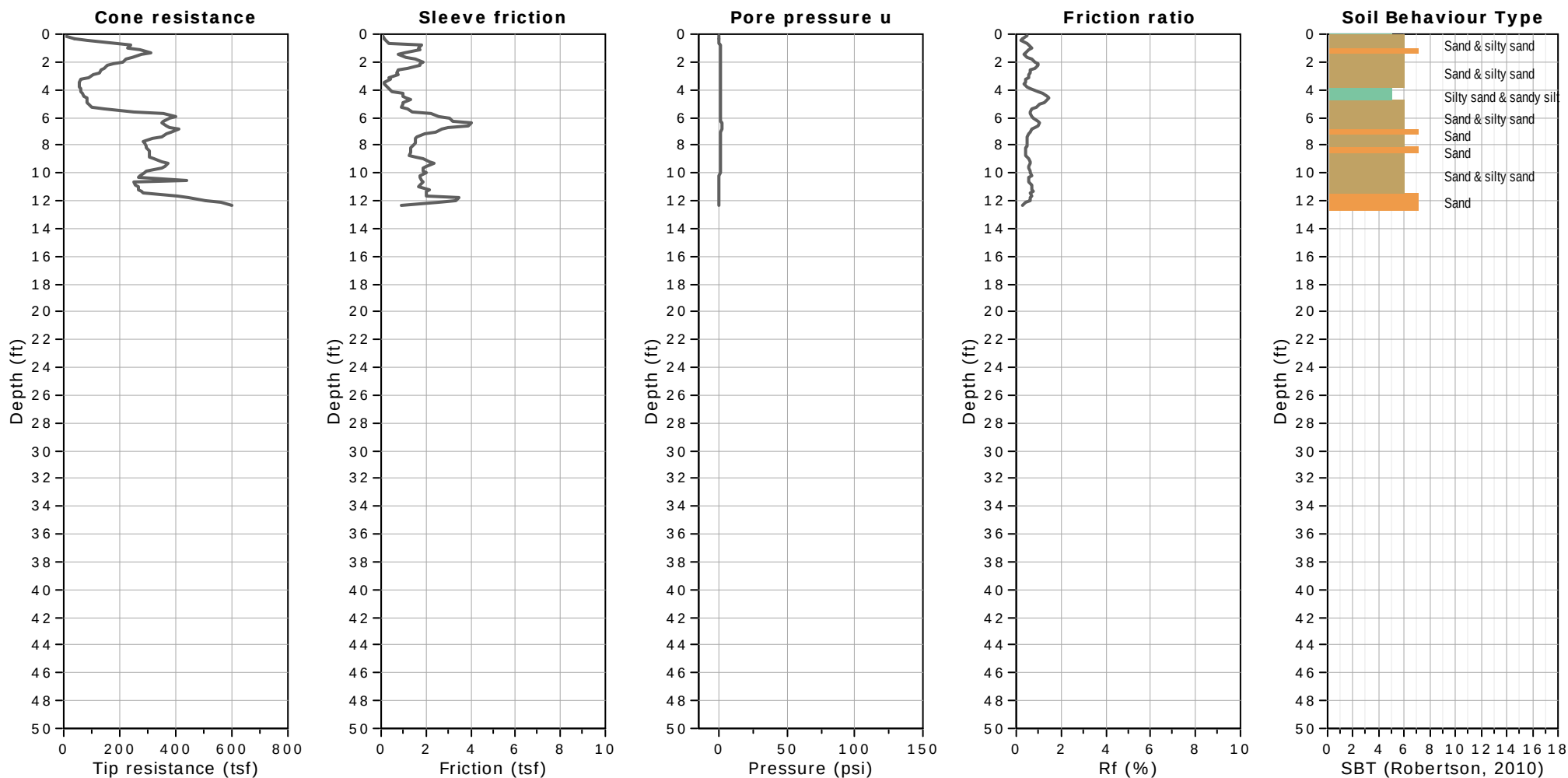


CLIENT: R.T. FRANKIAN & ASSOCIATES

SITE: CSC LID SITE X - SANTA CLARITA, CA

Field Rep: JIM

Total depth: 12.30 ft, Date: 11/1/2018



SBTn legend

- | | | |
|---------------------------|------------------------------|-----------------------------------|
| 1. Sensitive fine grained | 4. Clayey silt to silty clay | 7. Gravely sand to sand |
| 2. Organic material | 5. Silty sand to sandy silt | 8. Very stiff sand to clayey sand |
| 3. Clay to silty clay | 6. Clean sand to silty sand | 9. Very stiff fine grained |

WATER TABLE FOR ESTIMATING PURPOSES ONLY

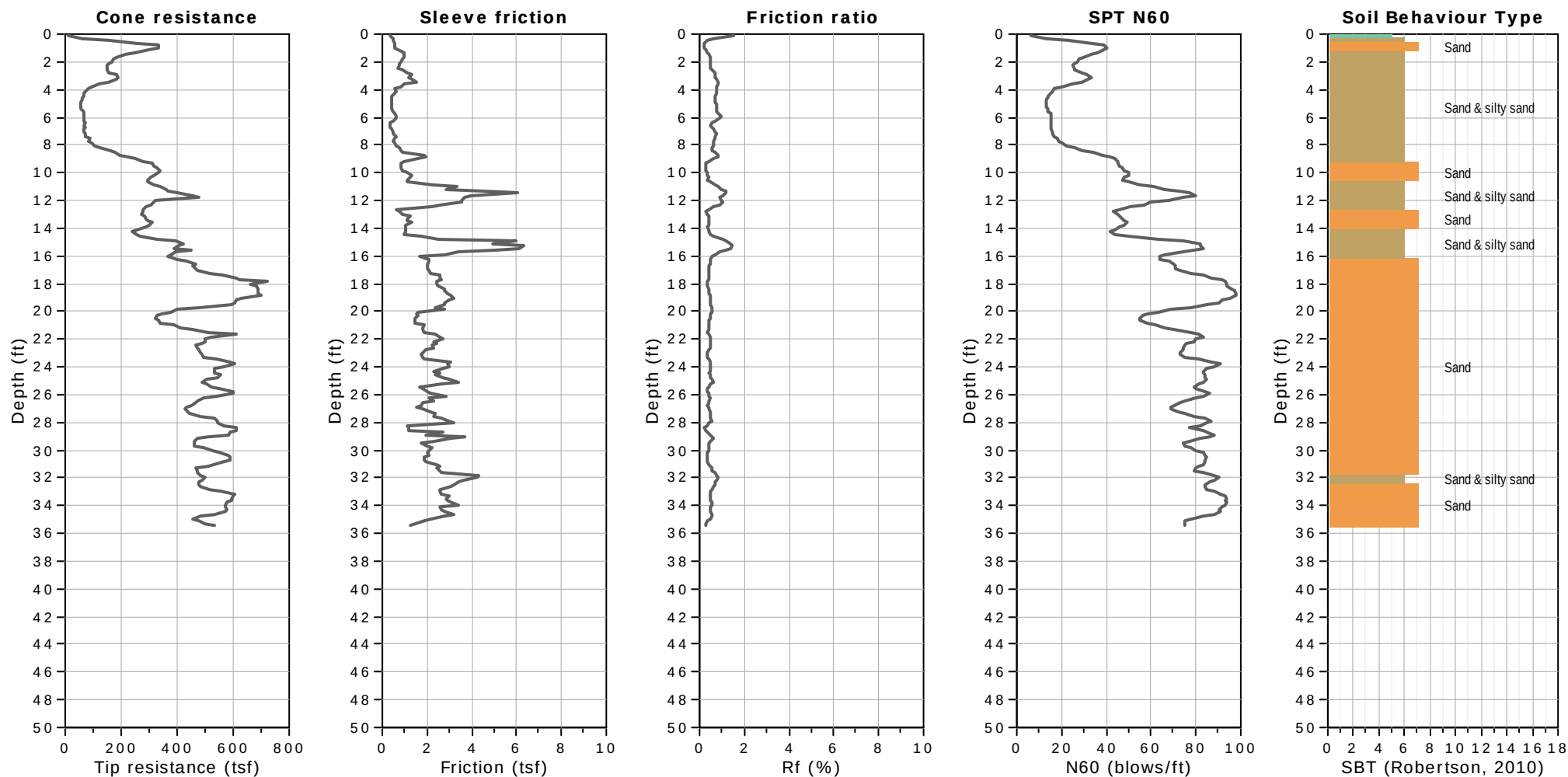


CLIENT: R.T. FRANKIAN & ASSOCIATES

SITE: CSC LID SITE X - SANTA CLARITA, CA

Field Rep: JIM

Total depth: 35.43 ft, Date: 11/1/2018



SBTn legend

- | | | |
|---------------------------|------------------------------|-----------------------------------|
| 1. Sensitive fine grained | 4. Clayey silt to silty clay | 7. Gravely sand to sand |
| 2. Organic material | 5. Silty sand to sandy silt | 8. Very stiff sand to clayey sand |
| 3. Clay to silty clay | 6. Clean sand to silty sand | 9. Very stiff fine grained |

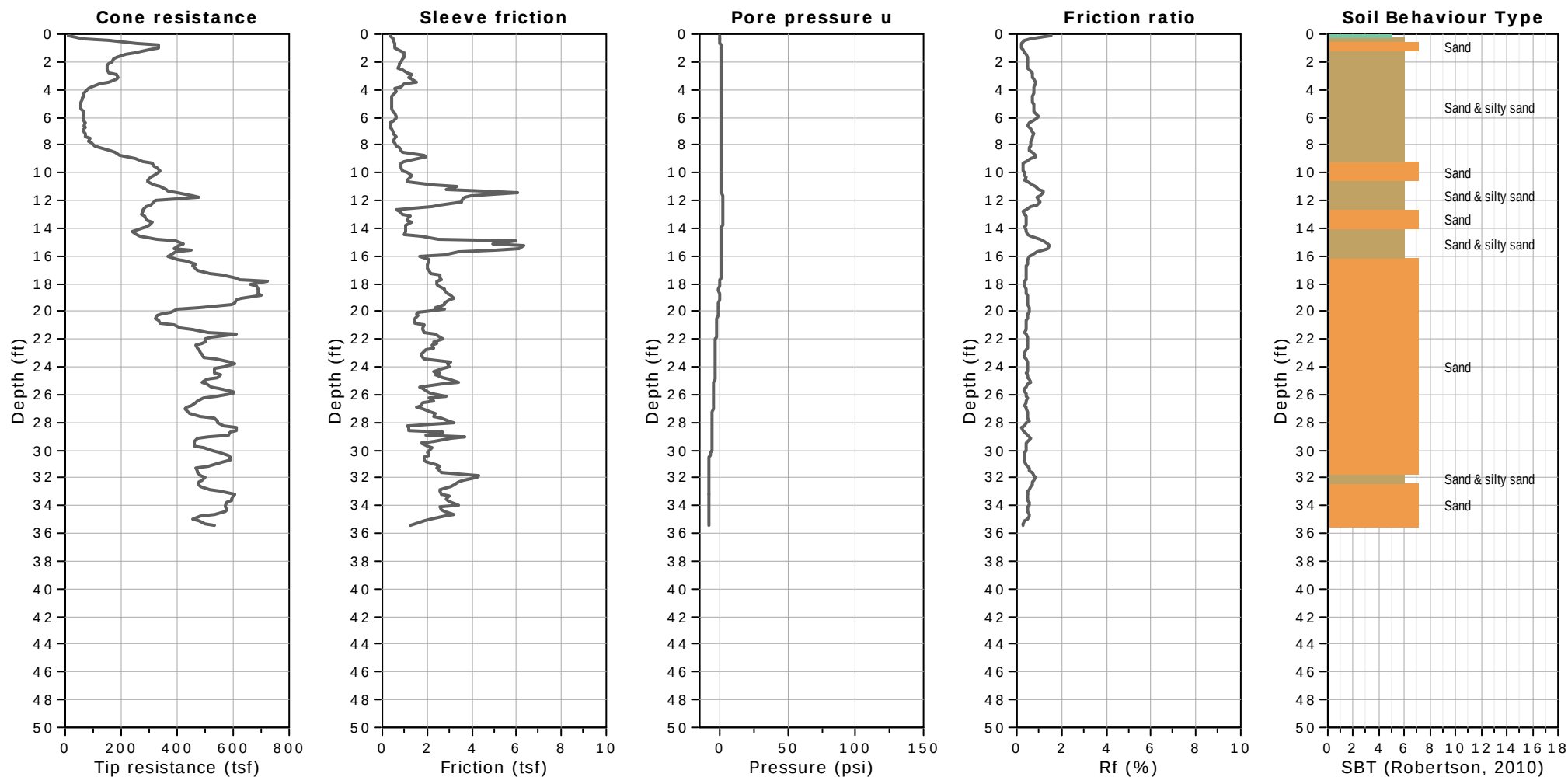


CLIENT: R.T. FRANKIAN & ASSOCIATES

SITE: CSC LID SITE X - SANTA CLARITA, CA

Field Rep: JIM

Total depth: 35.43 ft, Date: 11/1/2018



SBTn legend

- | | | |
|---------------------------|------------------------------|-----------------------------------|
| 1. Sensitive fine grained | 4. Clayey silt to silty clay | 7. Gravely sand to sand |
| 2. Organic material | 5. Silty sand to sandy silt | 8. Very stiff sand to clayey sand |
| 3. Clay to silty clay | 6. Clean sand to silty sand | 9. Very stiff fine grained |

WATER TABLE FOR ESTIMATING PURPOSES ONLY

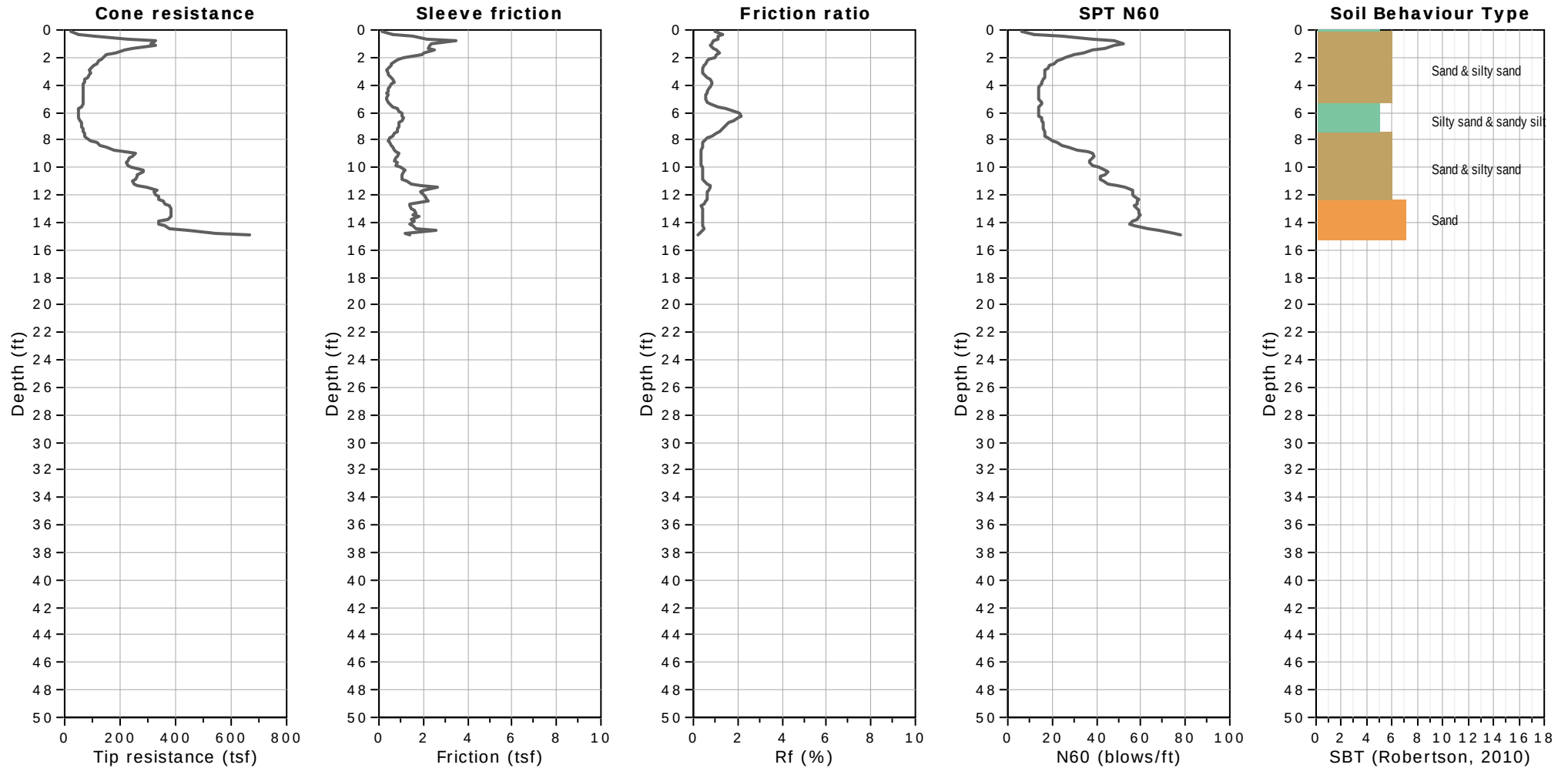


CLIENT: R.T. FRANKIAN & ASSOCIATES

SITE: CSC LID SITE X - SANTA CLARITA, CA

Field Rep: JIM

Total depth: 14.93 ft, Date: 11/1/2018



SBTn legend

- | | | |
|---------------------------|------------------------------|-----------------------------------|
| 1. Sensitive fine grained | 4. Clayey silt to silty clay | 7. Gravely sand to sand |
| 2. Organic material | 5. Silty sand to sandy silt | 8. Very stiff sand to clayey sand |
| 3. Clay to silty clay | 6. Clean sand to silty sand | 9. Very stiff fine grained |

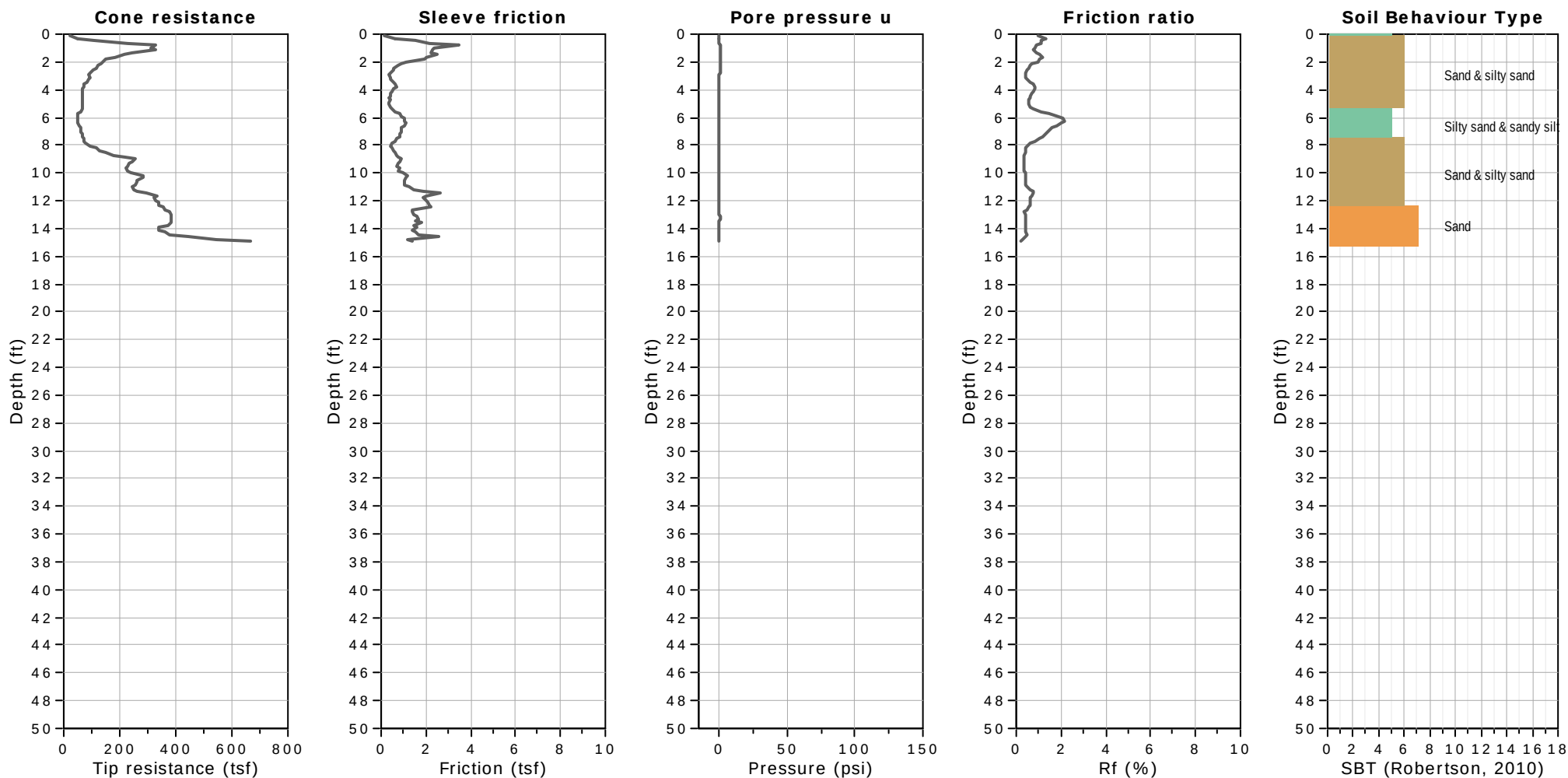


CLIENT: R.T. FRANKIAN & ASSOCIATES

SITE: CSC LID SITE X - SANTA CLARITA, CA

Field Rep: JIM

Total depth: 14.93 ft, Date: 11/1/2018



SBTn legend

- | | | |
|---------------------------|------------------------------|-----------------------------------|
| 1. Sensitive fine grained | 4. Clayey silt to silty clay | 7. Gravely sand to sand |
| 2. Organic material | 5. Silty sand to sandy silt | 8. Very stiff sand to clayey sand |
| 3. Clay to silty clay | 6. Clean sand to silty sand | 9. Very stiff fine grained |

WATER TABLE FOR ESTIMATING PURPOSES ONLY



APPENDIX D

Laboratory Testing

APPENDIX D

LABORATORY TESTING

Classification

Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D 2488. Soil classifications are indicated on the logs of the exploratory borings in Appendix A.

In-Place Moisture and Density Tests

The moisture content and dry density of relatively undisturbed samples obtained from the exploratory borings were evaluated in general accordance with ASTM D 2937. The test results are presented on the logs of the exploratory borings in Appendix A.

Gradation Analysis

Gradation analysis tests were performed on selected representative soil samples in general accordance with ASTM D 422. The grain-size distribution curves are shown on Figures D-1 through D-5. These test results were utilized in evaluating the soil classifications in accordance with the Unified Soil Classification System (USCS).

200 Wash

An evaluation of the percentage of particles finer than the No. 200 sieve in selected soil samples was performed in general accordance with ASTM D 1140. The results of the tests are presented on Figures D-6 and D-7.

Direct Shear Tests

Direct shear tests were performed on relatively undisturbed and remolded samples in general accordance with ASTM D 3080 to evaluate the shear strength characteristics of the selected materials. The samples were inundated during shearing to represent adverse field conditions. The results are shown on Figures D-8 through D-14.

Expansion Index Tests

The expansion index of selected materials was evaluated in general accordance with Uniform Building Code (UBC) Standard No. 18-2 (ASTM D 4829). Specimens were molded under a specified compactive energy at approximately 50 percent saturation (plus or minus 2 percent). The prepared 1-inch thick by 4-inch diameter specimens were loaded with a surcharge of 144 pounds per square foot and were inundated with distilled water. Readings of volumetric swell were made for a period of 24 hours. The results of these tests are presented on Figure D-15.

Proctor Density Tests

The maximum dry density and optimum moisture content of selected representative soil samples were evaluated using the Modified Proctor method in general accordance with ASTM D 698. The results of these tests are summarized on Figure D-16.

Sand Equivalent

Sand equivalent (SE) tests were performed on selected representative samples in general accordance with California Test (CT) 217. The SE value reported on Figure D-17 is the ratio of the coarse- to fine-grained particles in the selected samples.

R-Value

The resistance value, or R-value, for site soils was evaluated in general accordance with California Test (CT) 301. The sample was prepared and evaluated for exudation pressure and

expansion pressure. The equilibrium R-value is reported as the lesser or more conservative of the two calculated results. The test results are shown on Figure D-18.

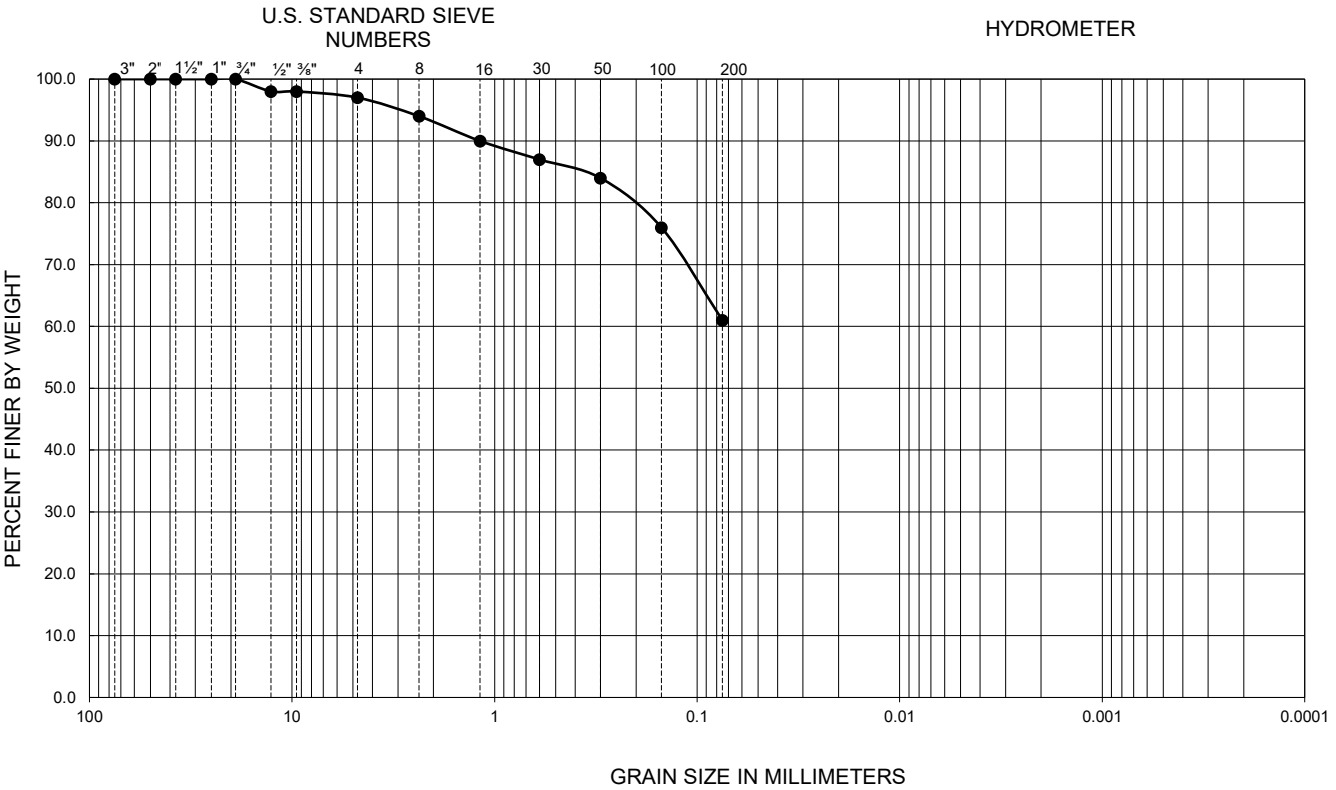
Soil Corrosivity Tests

Soil pH, and resistivity tests were performed on a representative sample in general accordance with CT 643. The soluble sulfate and chloride contents of selected samples were evaluated in general accordance with CT 417 and CT 422, respectively. The test results are presented on Figure D-19.

Unconfined Compression Tests

Unconfined compression tests were performed on remolded samples in general accordance with ASTM D 1633. The test results are presented on Figure D-20.

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY

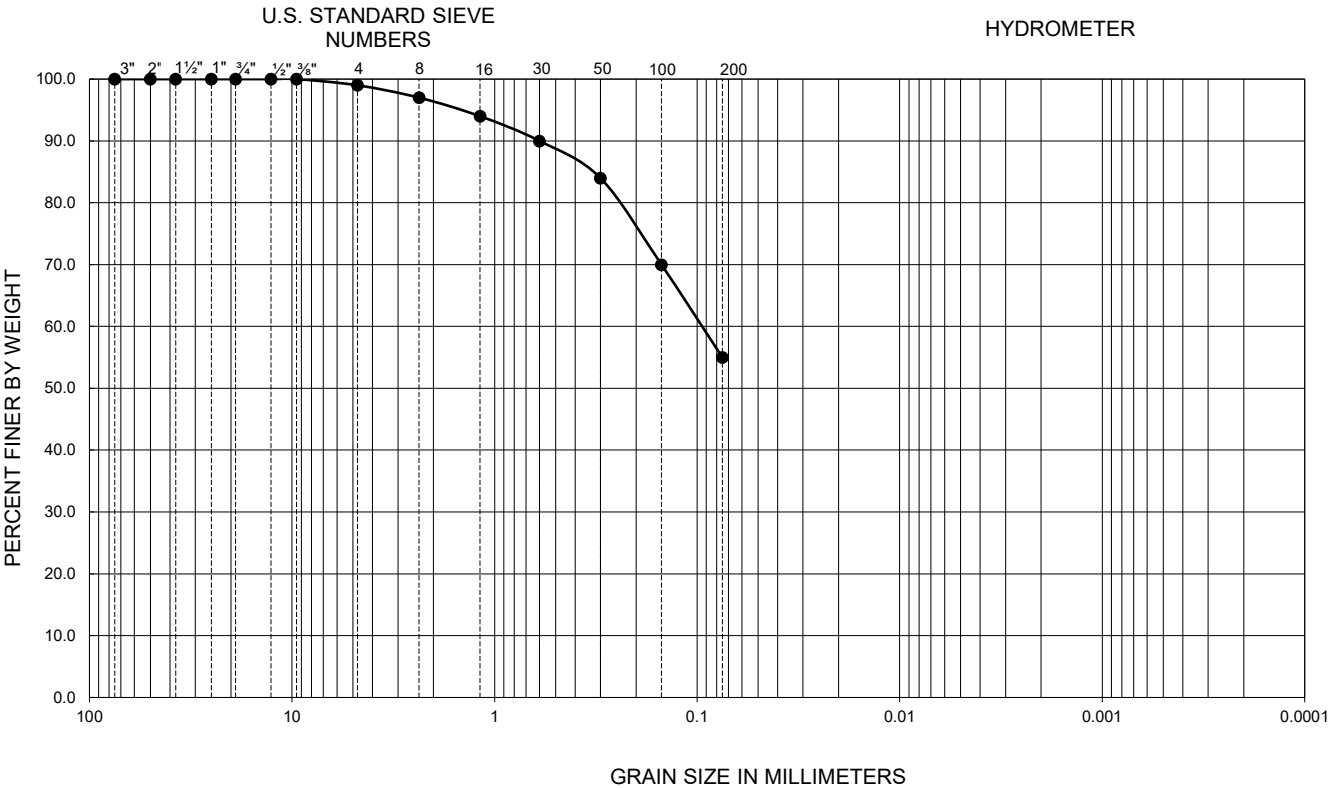


Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D ₁₀	D ₃₀	D ₆₀	C _u	C _c	Passing No. 200 (percent)	USCS
●	B-2	0.0-5.0	--	--	--	--	--	--	--	--	61	CL

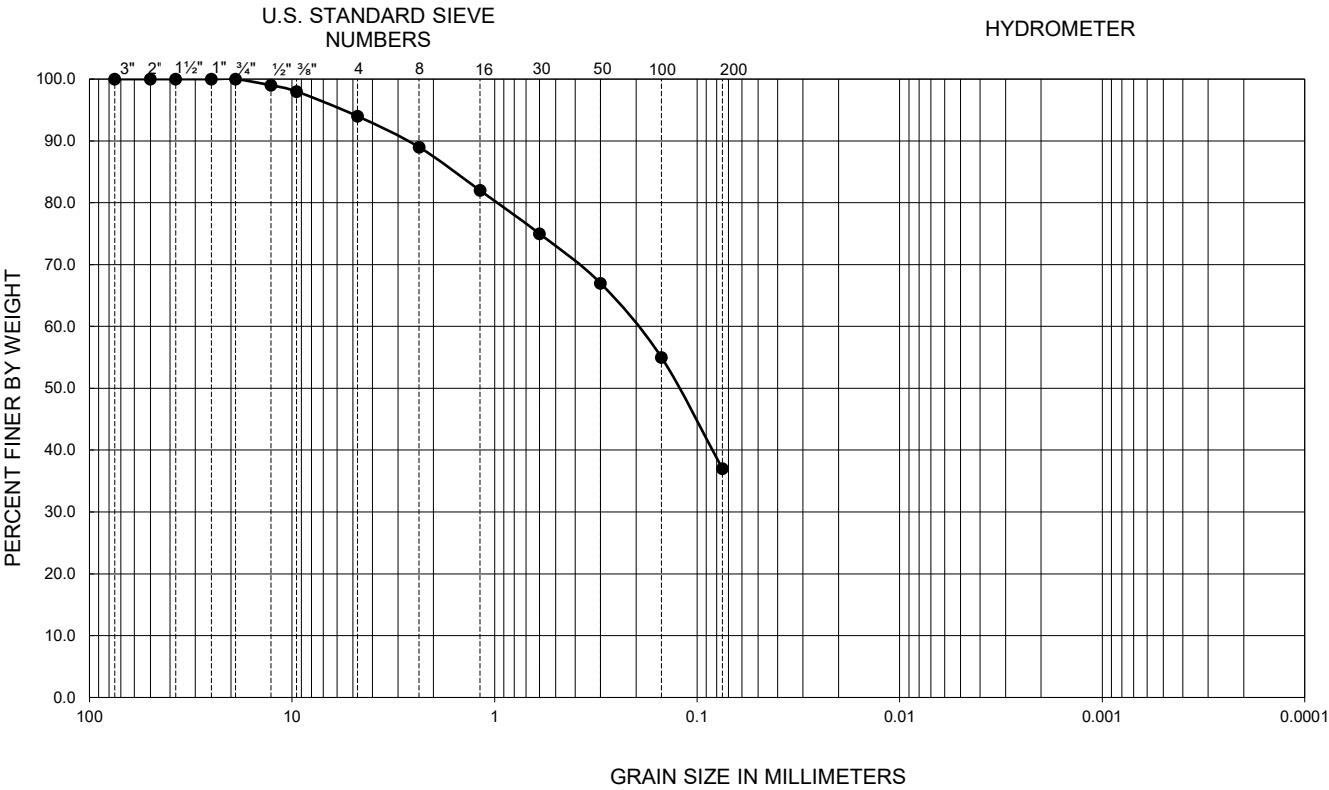
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE D-1

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY

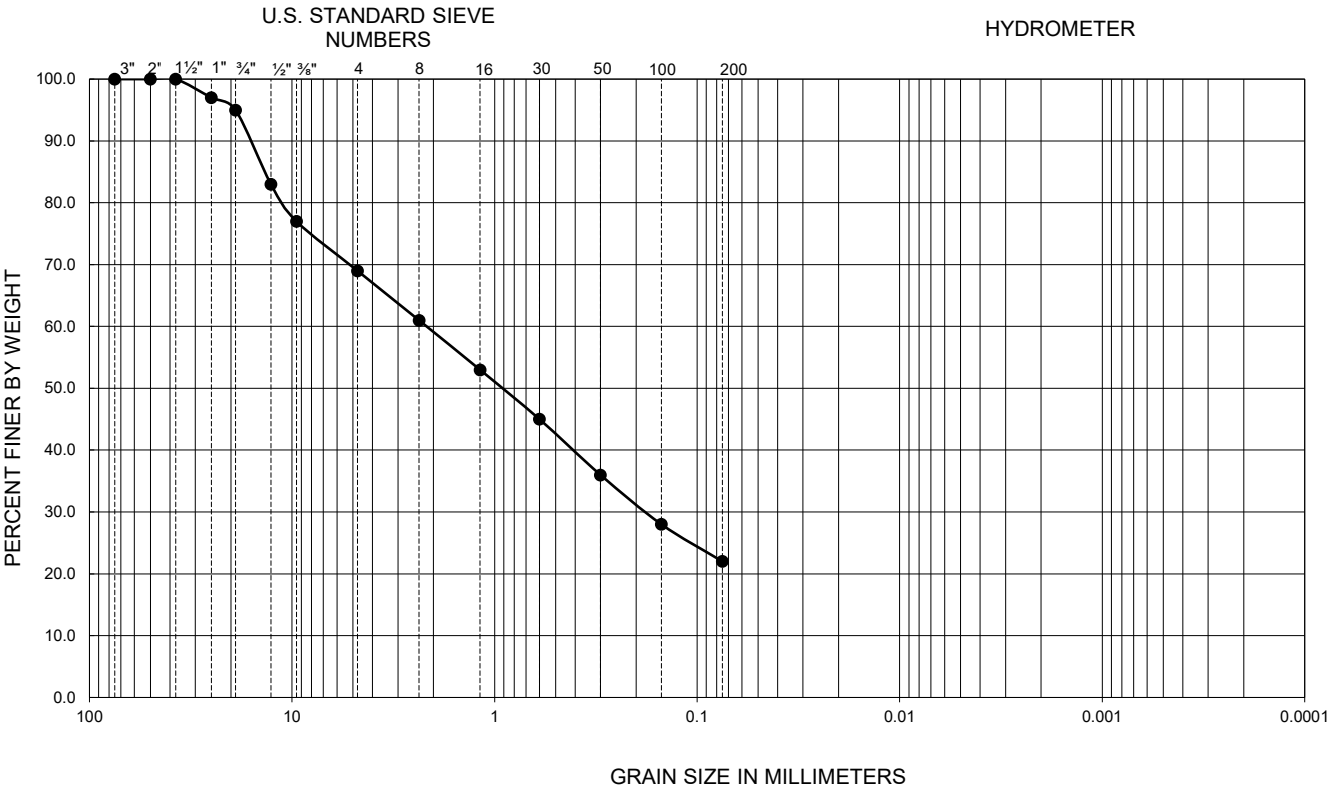


Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D ₁₀	D ₃₀	D ₆₀	C _u	C _c	Passing No. 200 (percent)	USCS
●	B-4	0.0-5.0	--	--	--	--	--	--	--	--	37	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE D-3

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY

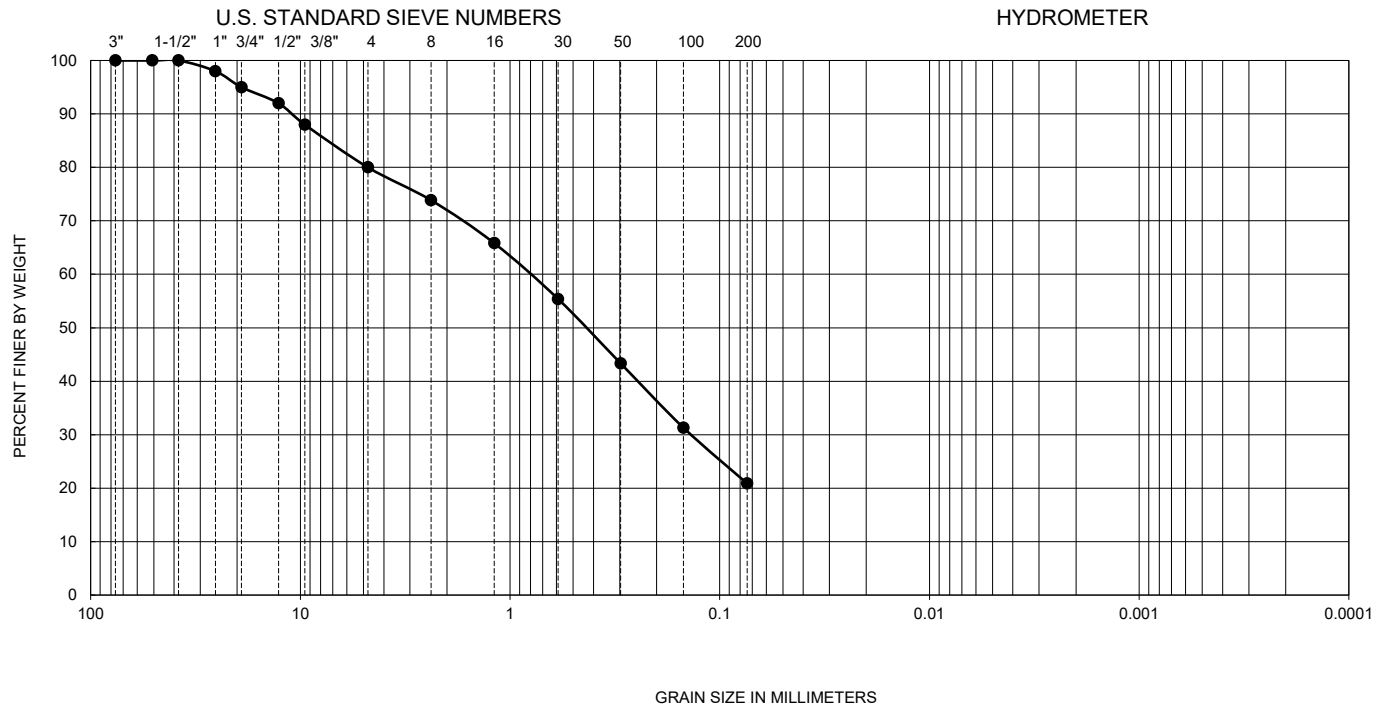


Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D ₁₀	D ₃₀	D ₆₀	C _u	C _c	Passing No. 200 (percent)	USCS
●	B-5	0.0-5.0	--	--	--	--	--	--	--	--	22	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE D-4

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Hole No.	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D ₁₀	D ₃₀	D ₆₀	C _u	C _c	Passing No. 200 (percent)	USCS
●	B-4 & B-5	0.0-5.0	--	--	--	--	--	--	--	--	21	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE D-5

GRADATION TEST RESULTS

VIA PRINCESSA PARK
SANTA CLARITA, CALIFORNIA

212109001 | 6/23

SAMPLE LOCATION	SAMPLE DEPTH (ft)	DESCRIPTION	PERCENT PASSING NO. 4	PERCENT PASSING NO. 200	USCS (TOTAL SAMPLE)
B-7	15.0-16.5	POORLY-GRADED SAND WITH SILT AND GRAVEL	84	5	SP-SM
B-9	10.0-11.5	POORLY-GRADED SAND WITH SILT AND GRAVEL	75	7	SP-SM
B-12	15.0-16.5	POORLY-GRADED SAND WITH SILT AND GRAVEL	71	11	SP-SM
B-12	40.0-41.5	POORLY-GRADED SAND WITH SILT AND GRAVEL	77	6	SP-SM
B-13	10.0-11.5	POORLY-GRADED SAND WITH GRAVEL	59	3	SP
B-16	20.0-21.5	POORLY-GRADED SAND WITH SILT AND GRAVEL	58	8	SP-SM
B-16	45.0-46.5	POORLY-GRADED SAND WITH SILT AND GRAVEL	77	9	SP-SM
B-16	90.0-91.5	POORLY-GRADED SAND	87	4	SP
B-18	20.0-21.5	SANDY SILT	100	52	ML

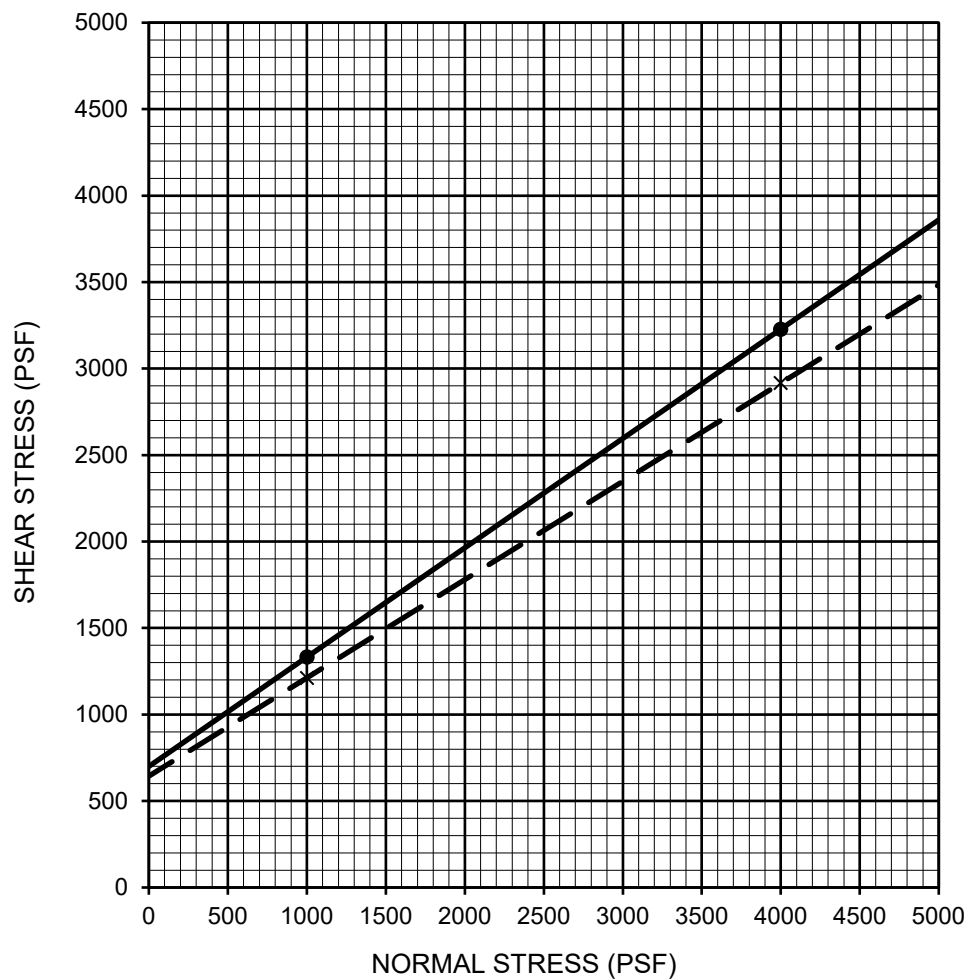
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 1140

FIGURE D-6

SAMPLE LOCATION	SAMPLE DEPTH (ft)	DESCRIPTION	PERCENT PASSING NO. 4	PERCENT PASSING NO. 200	USCS (TOTAL SAMPLE)
B-18	36.0-36.5	SILTY SAND	98	34	SM
B-18	60.0-61.5	POORLY-GRADED SAND WITH SILT	86	8	SP-SM
B-18	80.0-81.5	POORLY-GRADED SAND WITH SILT	93	5	SP-SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 1140

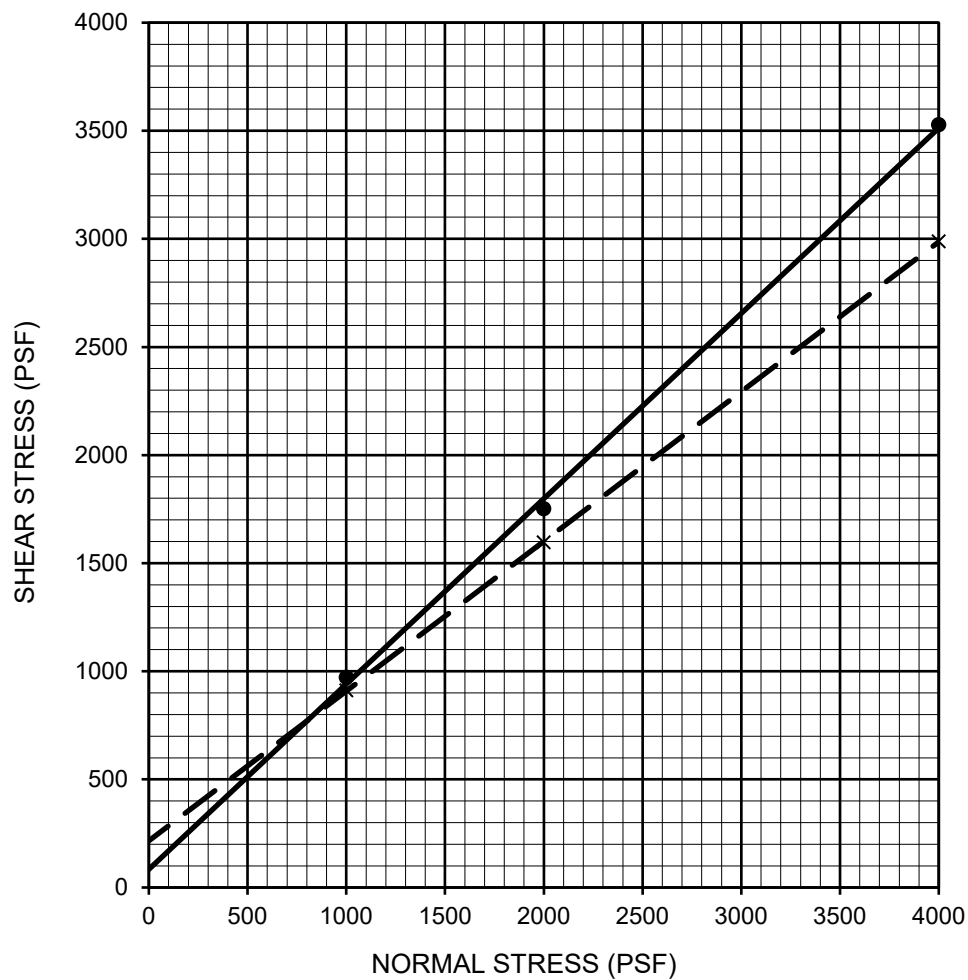
FIGURE D-7



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
SILTY SAND	—●—	B-2	15.0-16.5	Peak	700	32	SM
SILTY SAND	- - X - -	B-2	15.0-16.5	Ultimate	644	30	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

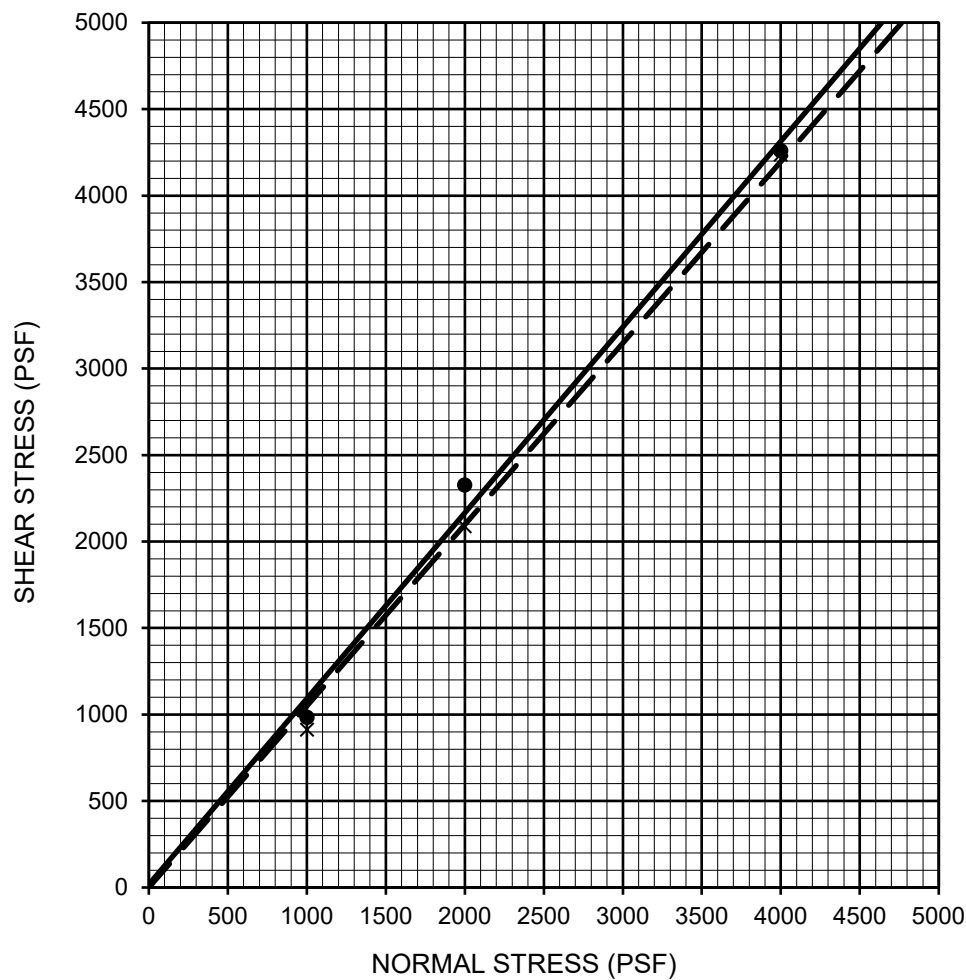
FIGURE D-8



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
POORLY-GRADED SAND WITH SILT	—●—	B-3	15.0-16.5	Peak	84	41	SP-SM
POORLY-GRADED SAND WITH SILT	- - X - -	B-3	15.0-16.5	Ultimate	216	35	SP-SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

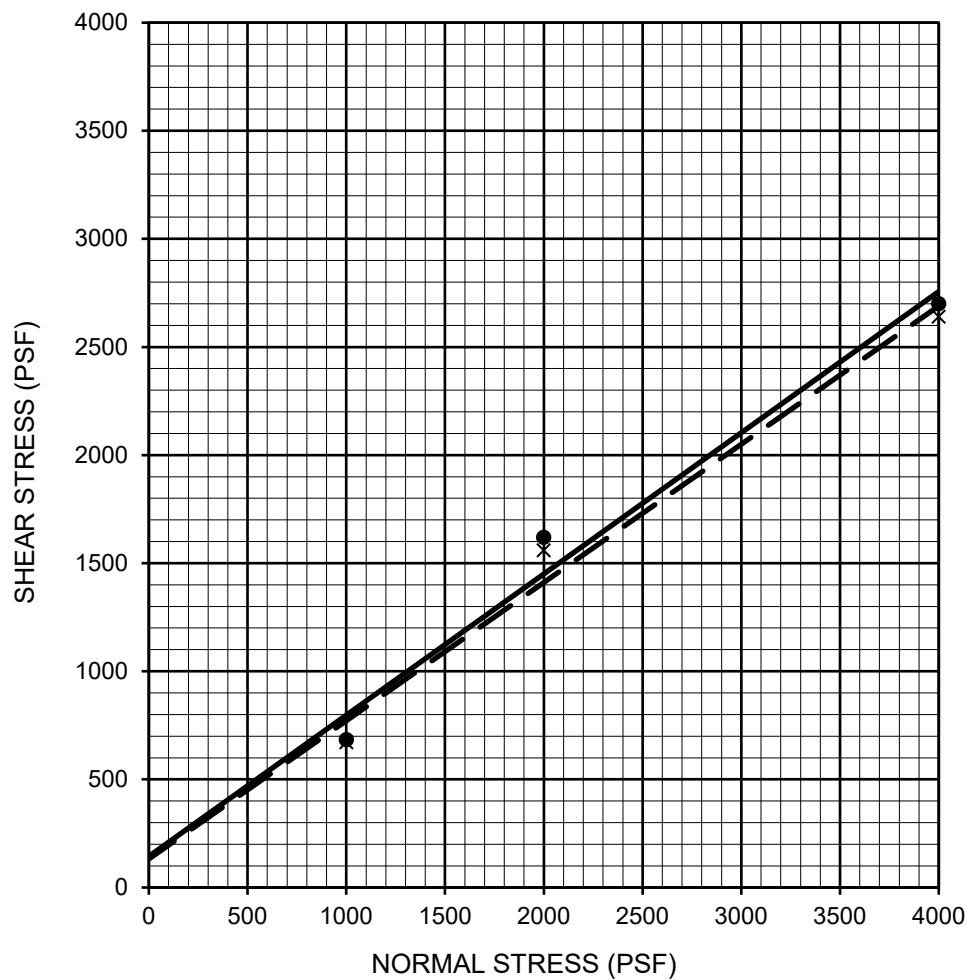
FIGURE D-9



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
POORLY-GRADED SAND	—●—	B-9	20.0-21.5	Peak	18	47	SP
POORLY-GRADED SAND	- - X - -	B-9	20.0-21.5	Ultimate	0	46	SP

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

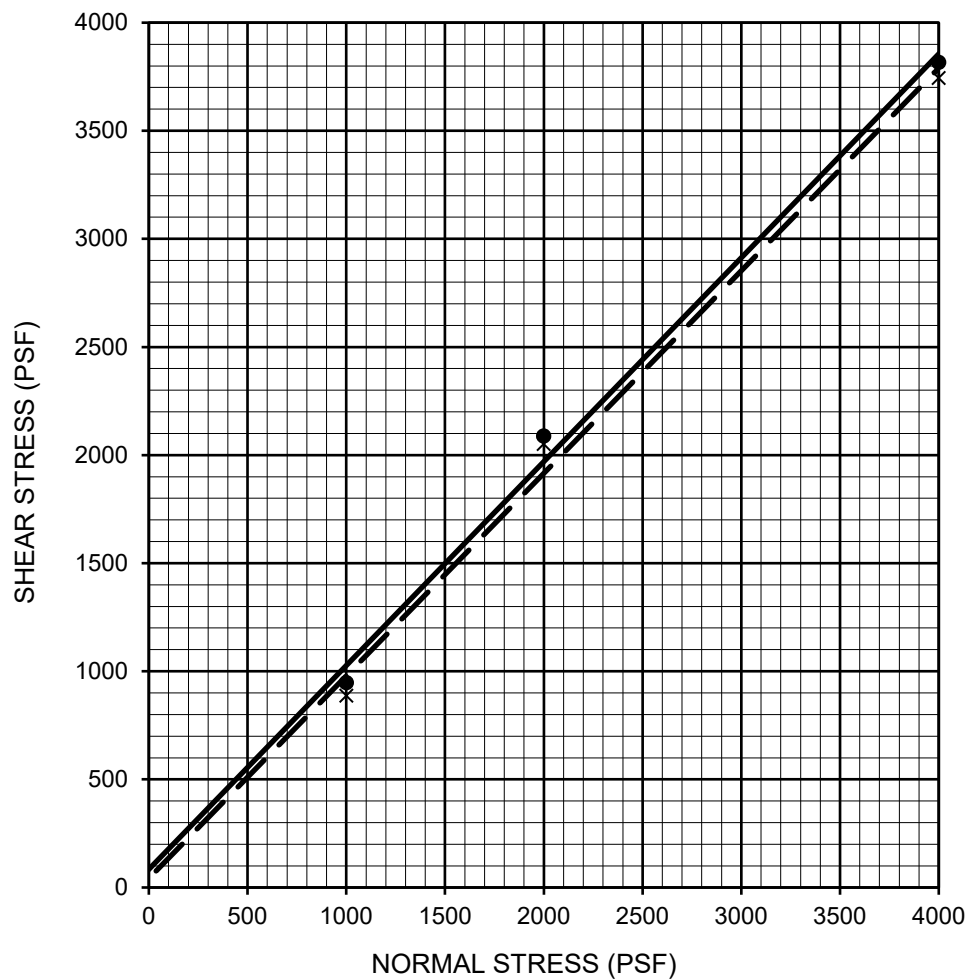
FIGURE D-10



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
SILTY SAND	—●—	B-12	10.0-11.5	Peak	144	33	SM
SILTY SAND	- - X - -	B-12	10.0-11.5	Ultimate	132	33	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

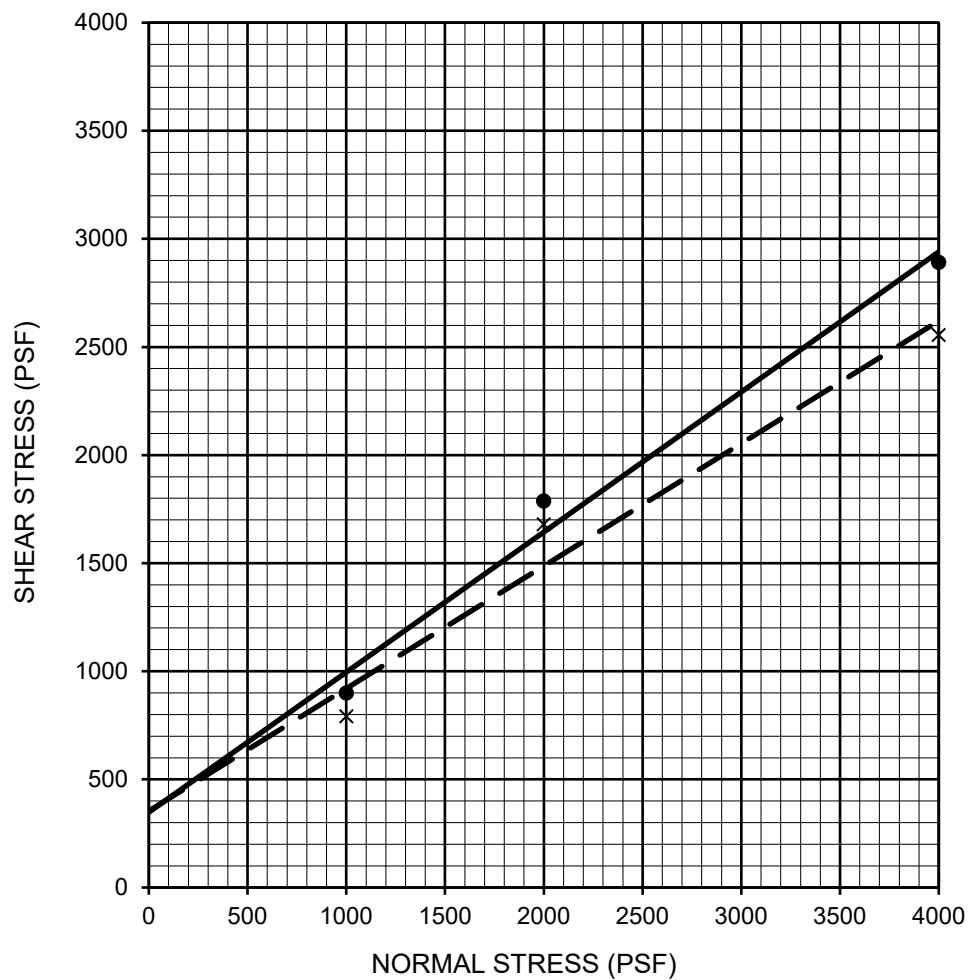
FIGURE D-11



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
POORLY-GRADED SAND	—●—	B-15	10.0-11.5	Peak	84	43	SP
POORLY-GRADED SAND	- - X - -	B-15	10.0-11.5	Ultimate	42	43	SP

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

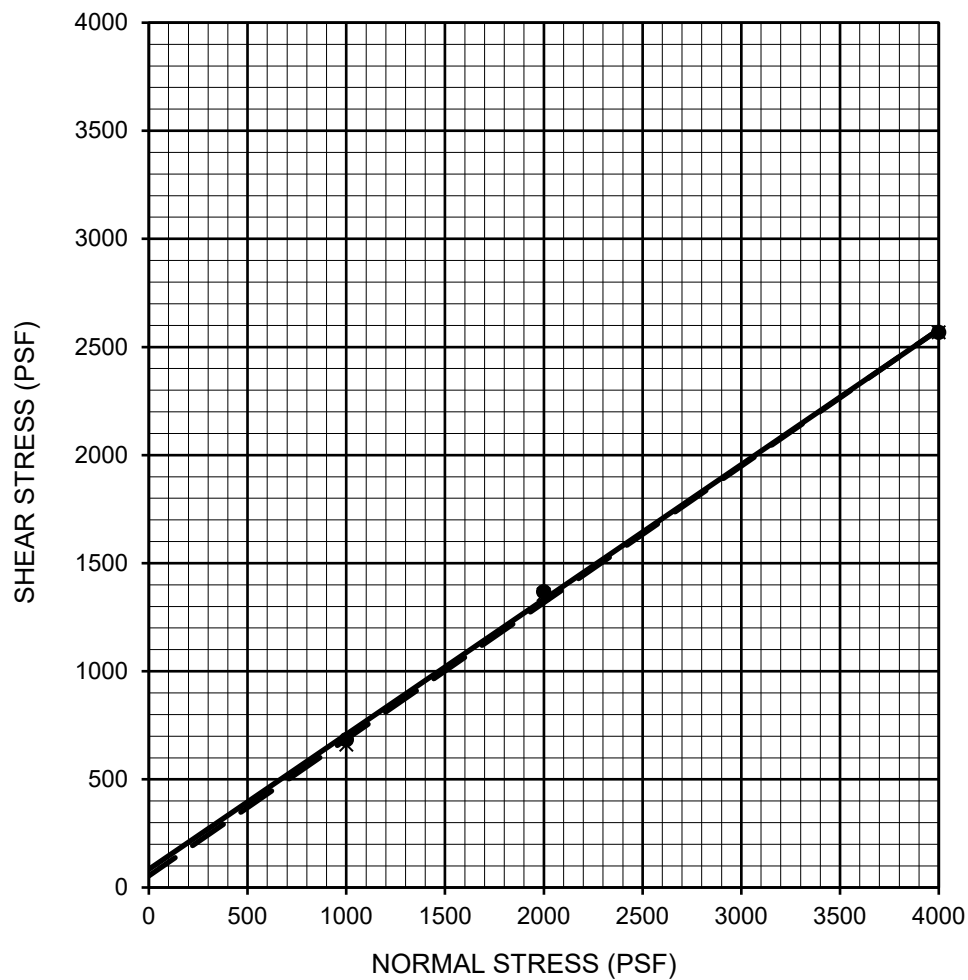
FIGURE D-12



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
CLAYEY SAND	—●—	B-16	5.0-6.5	Peak	348	33	SC
CLAYEY SAND	- - X - -	B-16	5.0-6.5	Ultimate	354	30	SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

FIGURE D-13



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
SILTY SAND	—●—	B-18	10.0-11.5	Peak	84	32	SM
SILTY SAND	- - X - -	B-18	10.0-11.5	Ultimate	54	32	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

FIGURE D-14

SAMPLE LOCATION	SAMPLE DEPTH (ft)	INITIAL MOISTURE (percent)	COMPACTED DRY DENSITY (pcf)	FINAL MOISTURE (percent)	VOLUMETRIC SWELL (in)	EXPANSION INDEX	POTENTIAL EXPANSION
B-14	0.0-3.0	10.5	108.5	23.5	0.033	33	Low
B-16	0.0-5.0	9.0	112.5	17.9	0.009	9	Very Low

PERFORMED IN GENERAL ACCORDANCE WITH

☐ UBC STANDARD 18-2

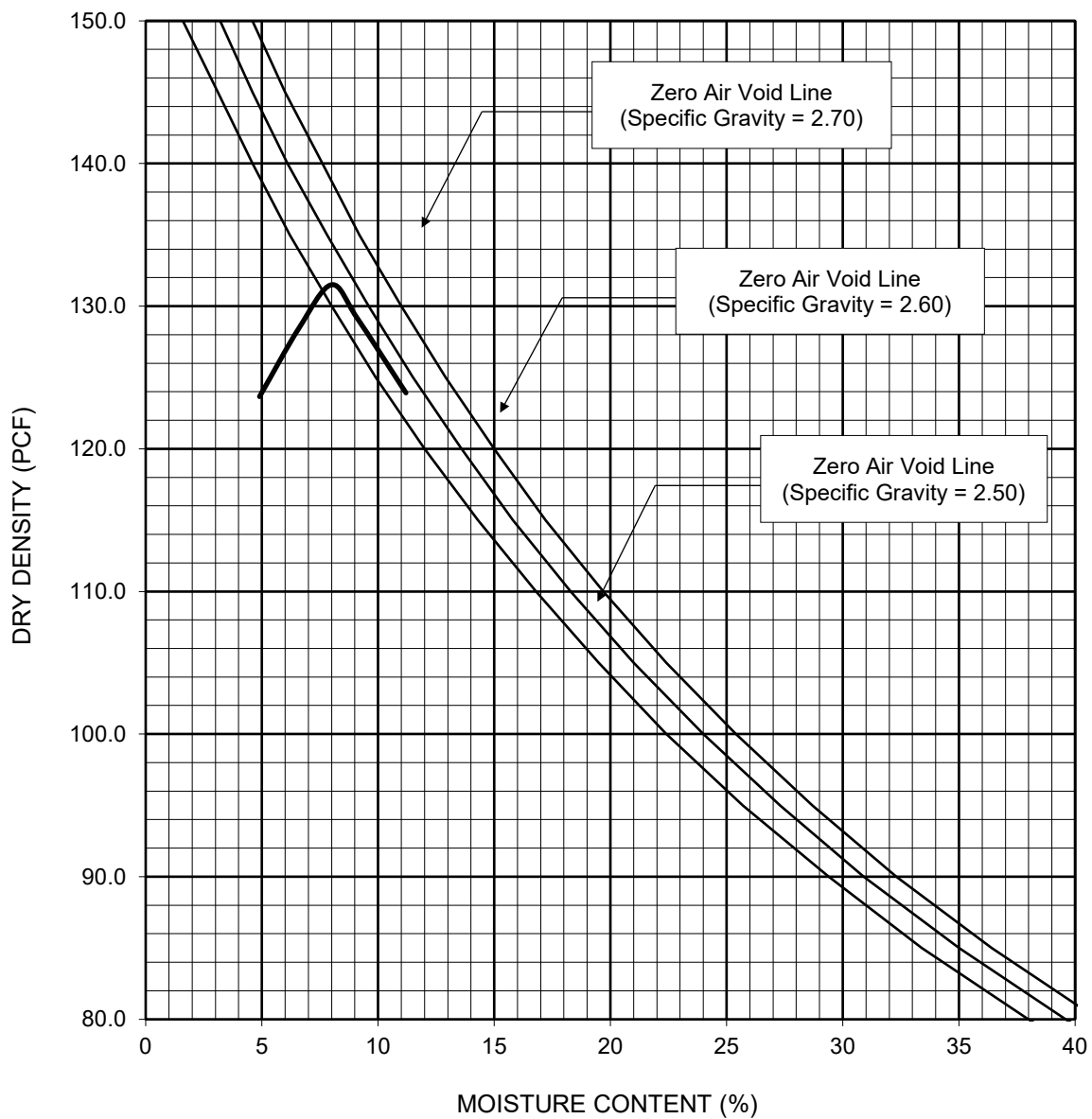
☒ ASTM D 4829

FIGURE D-15

EXPANSION INDEX TEST RESULTS

VIA PRINCESSA PARK
SANTA CLARITA, CALIFORNIA

212109001 | 6/23



Sample Location	Depth (ft)	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture Content (percent)
B-4 & B-5	0.0-5.0	Brown Silty SAND with Gravel	131.5	8.0
Dry Density and Moisture Content Values Corrected for Oversize (ASTM D 4718)			133	7.5

PERFORMED IN GENERAL ACCORDANCE WITH

☐ ASTM D 1557

☒ ASTM D 698

METHOD

☐ A

☐ B

☒ C

FIGURE D-16

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

PROCTOR DENSITY TEST RESULTS

VIA PRINCESSA PARK
SANTA CLARITA, CALIFORNIA

212109001 | 6/23

SAMPLE LOCATION	SAMPLE DEPTH (ft)	SOIL TYPE	SAND EQUIVALENT
B-18	1.0-5.0	SC	11

PERFORMED IN GENERAL ACCORDANCE WITH CT 217

FIGURE D-17

SAMPLE LOCATION	SAMPLE DEPTH (ft)	SOIL TYPE	R-VALUE
B-6	0.0-5.0	SM	56
B-17	0.0-5.0	SM	72
B-18	1.0-5.0	SC	47

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2844/CT 301

FIGURE D-18

R-VALUE TEST RESULTS

VIA PRINCESSA PARK
SANTA CLARITA, CALIFORNIA

SAMPLE LOCATION	SAMPLE DEPTH (ft)	pH ¹	RESISTIVITY ¹ (ohm-cm)	SULFATE CONTENT ²		CHLORIDE CONTENT ³ (ppm)
				(ppm)	(%)	
B-2	0.0-5.0	7.1	7,450	10	0.001	130
B-14	0.0-3.0	6.7	8,380	10	0.001	25

¹ PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 643

² PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 417

³ PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 422

FIGURE D-19

CORROSIVITY TEST RESULTS

VIA PRINCESSA PARK
SANTA CLARITA, CALIFORNIA

212109001 | 6/23



SOIL TYPE	PERCENT CEMENT ADDED BY DRY WEIGHT	DAYS OF CURING	UNCONFINED COMPRESSIVE STRENGTH (psi)
SM	3	3	350
SM	3	7	370
SM	3	28	370
SM	5	3	426
SM	5	7	527
SM	5	28	599
SM	7	3	539
SM	7	7	568
SM	7	28	600
SM	9	7	635

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 1633

FIGURE D-20

UNCONFINED COMPRESSION TEST RESULTS



APPENDIX E

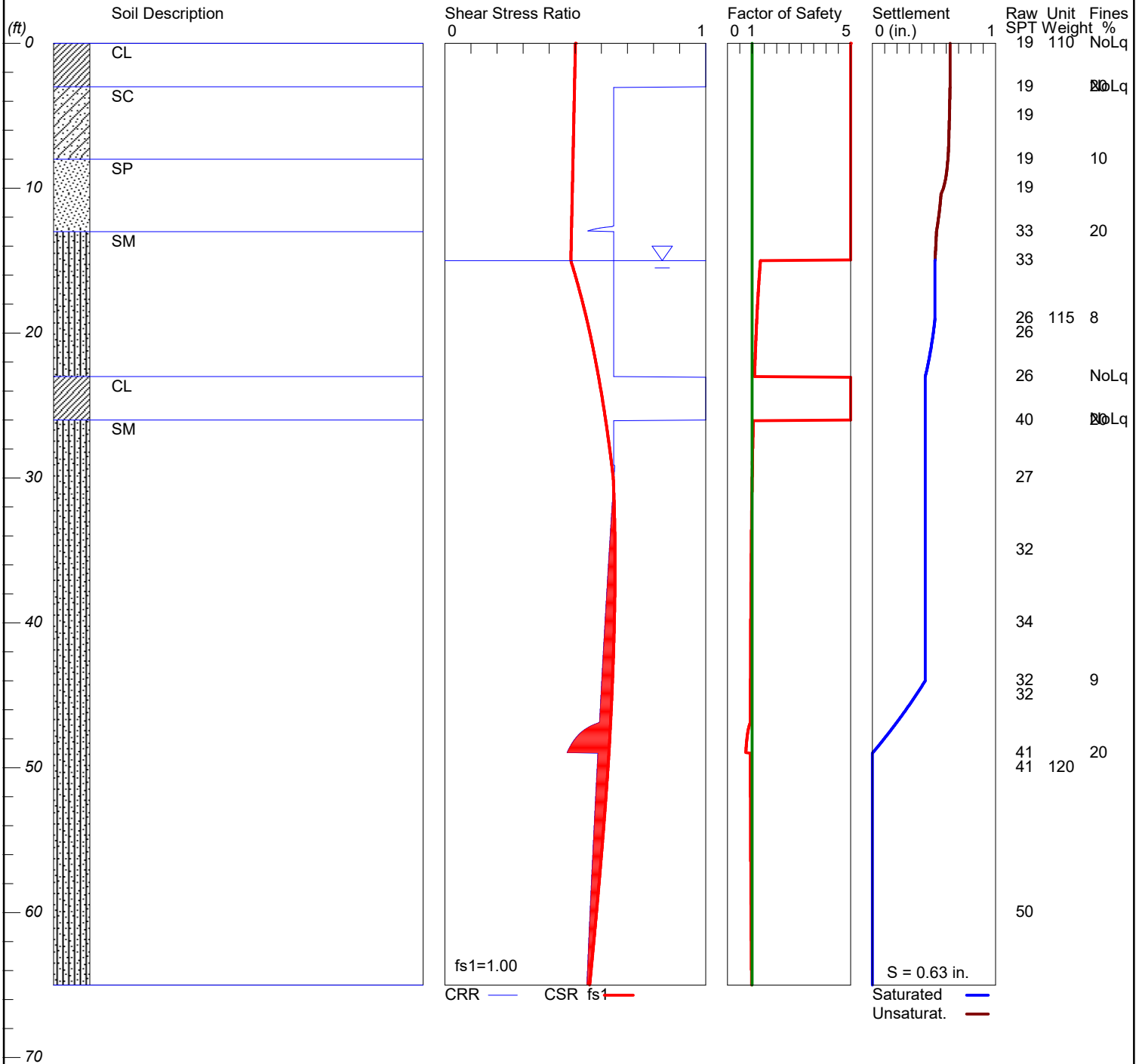
Liquefaction Analysis Results

LIQUEFACTION ANALYSIS

Via Princessa Park

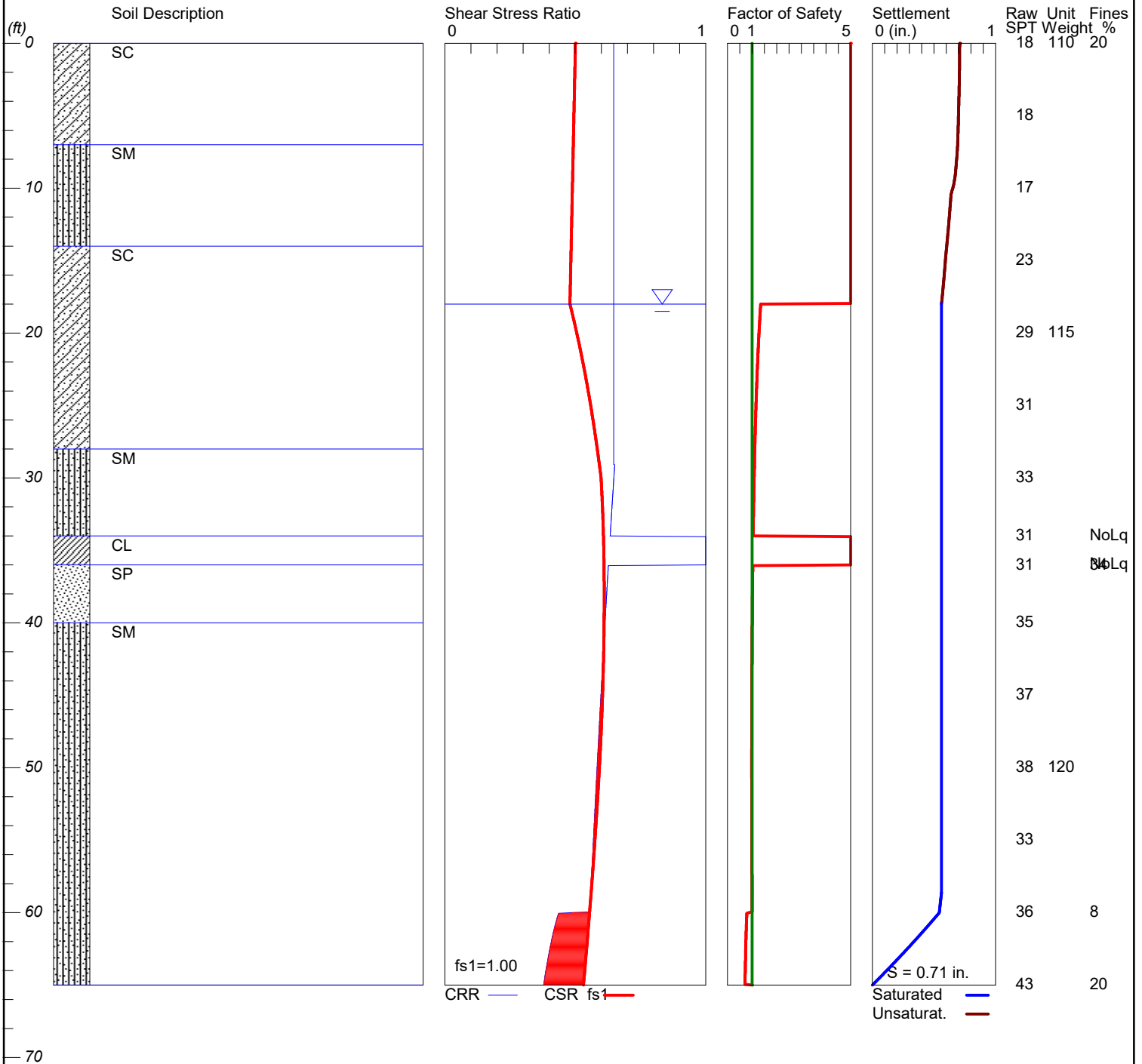
Hole No.=B-16 Water Depth=15 ft Surface Elev.=1390

Magnitude=6.78
Acceleration=0.77g



Via Princessa Park

Magnitude=6.78
Acceleration=0.77g

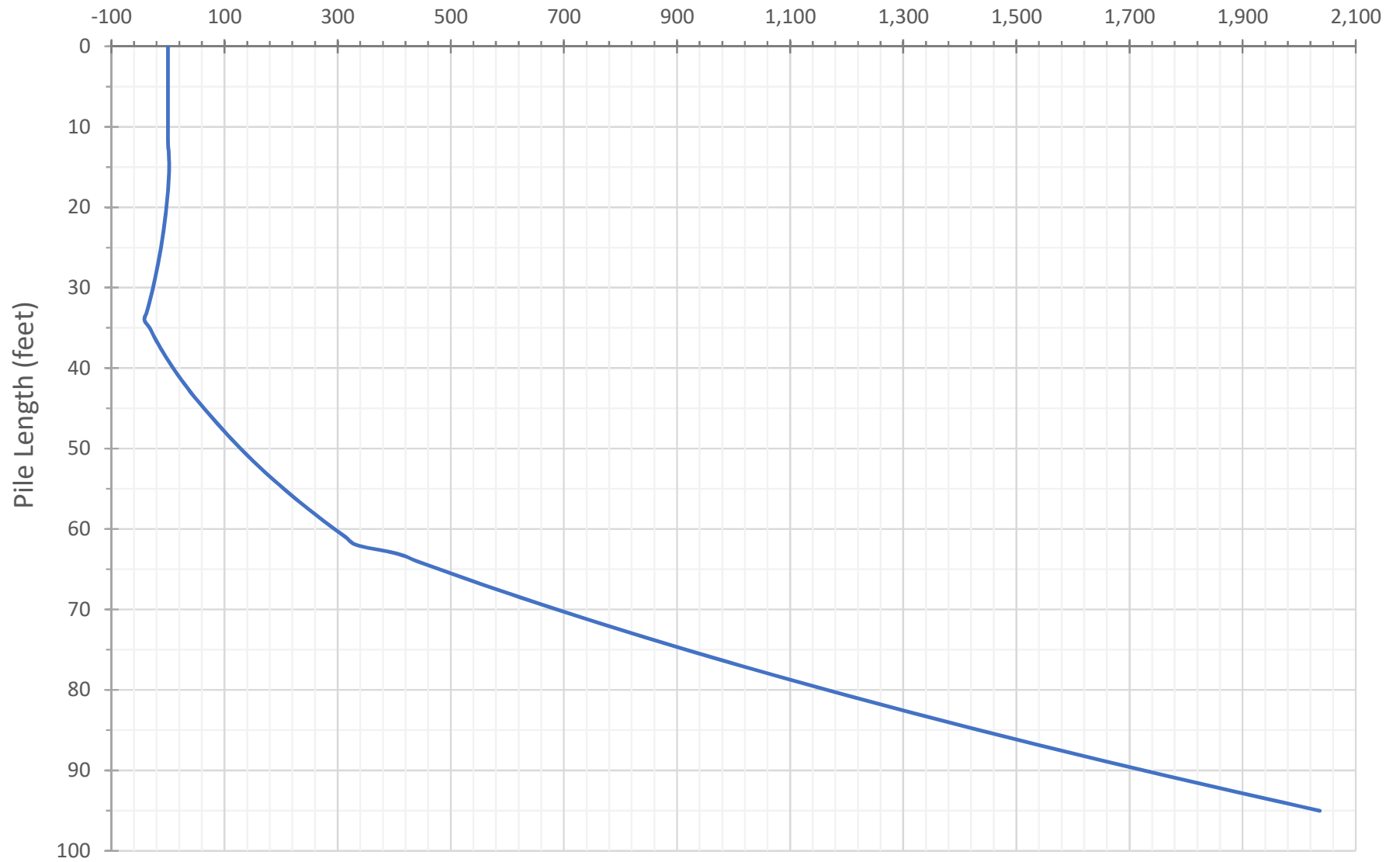




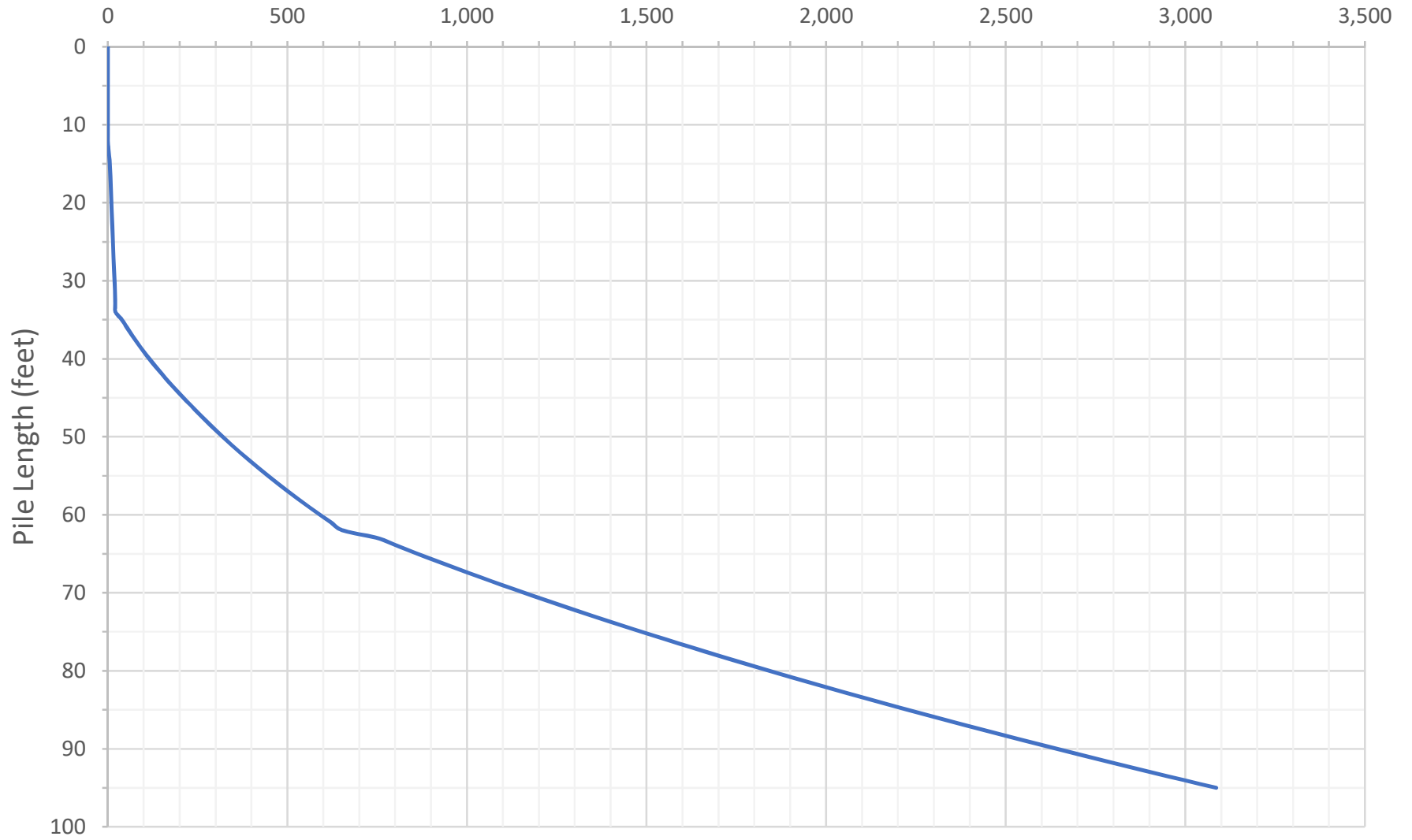
APPENDIX F

CISS Pile Analysis Results

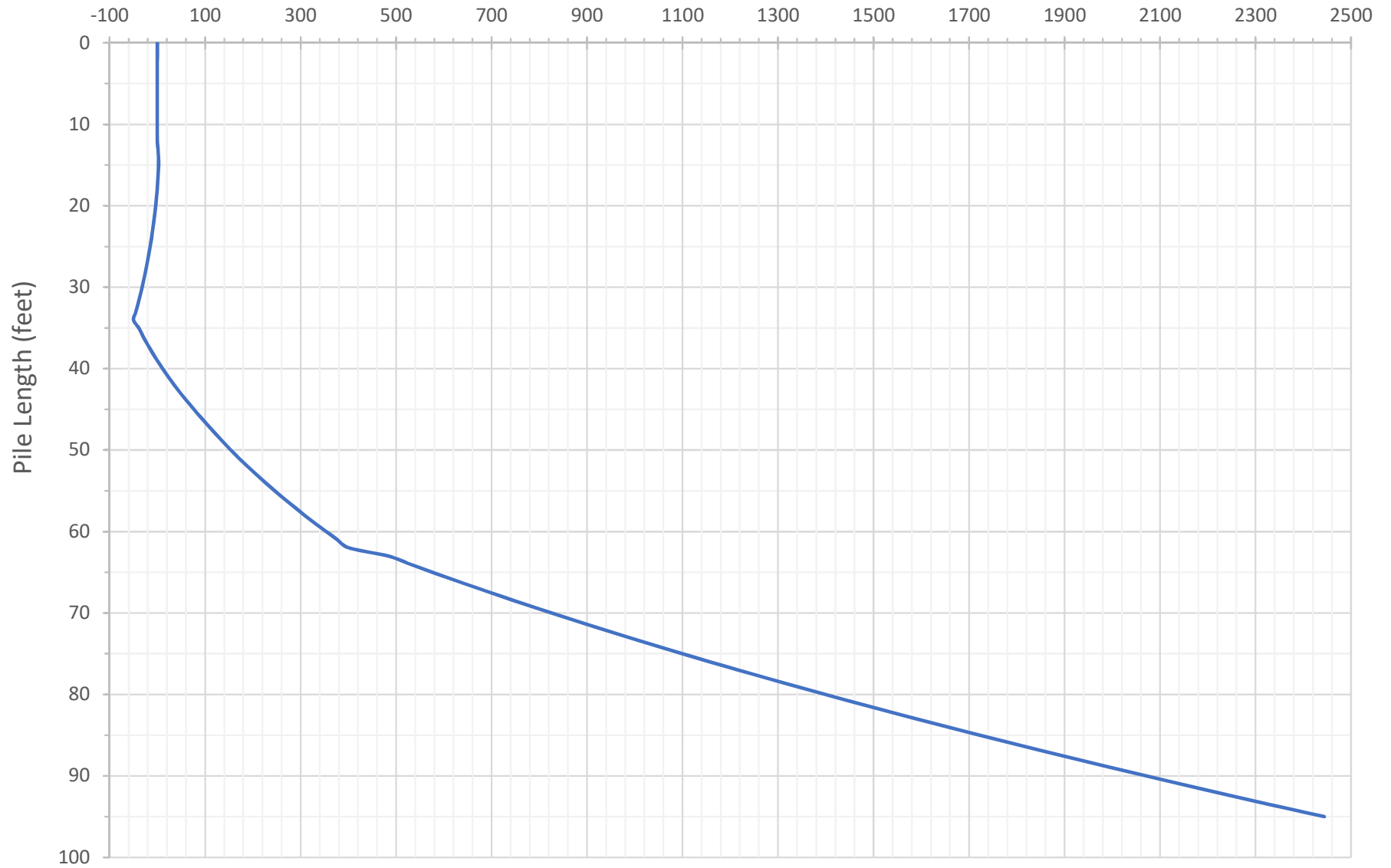
60-inch CISS
Axial Capacity (kips)
Service & Strength Limit States



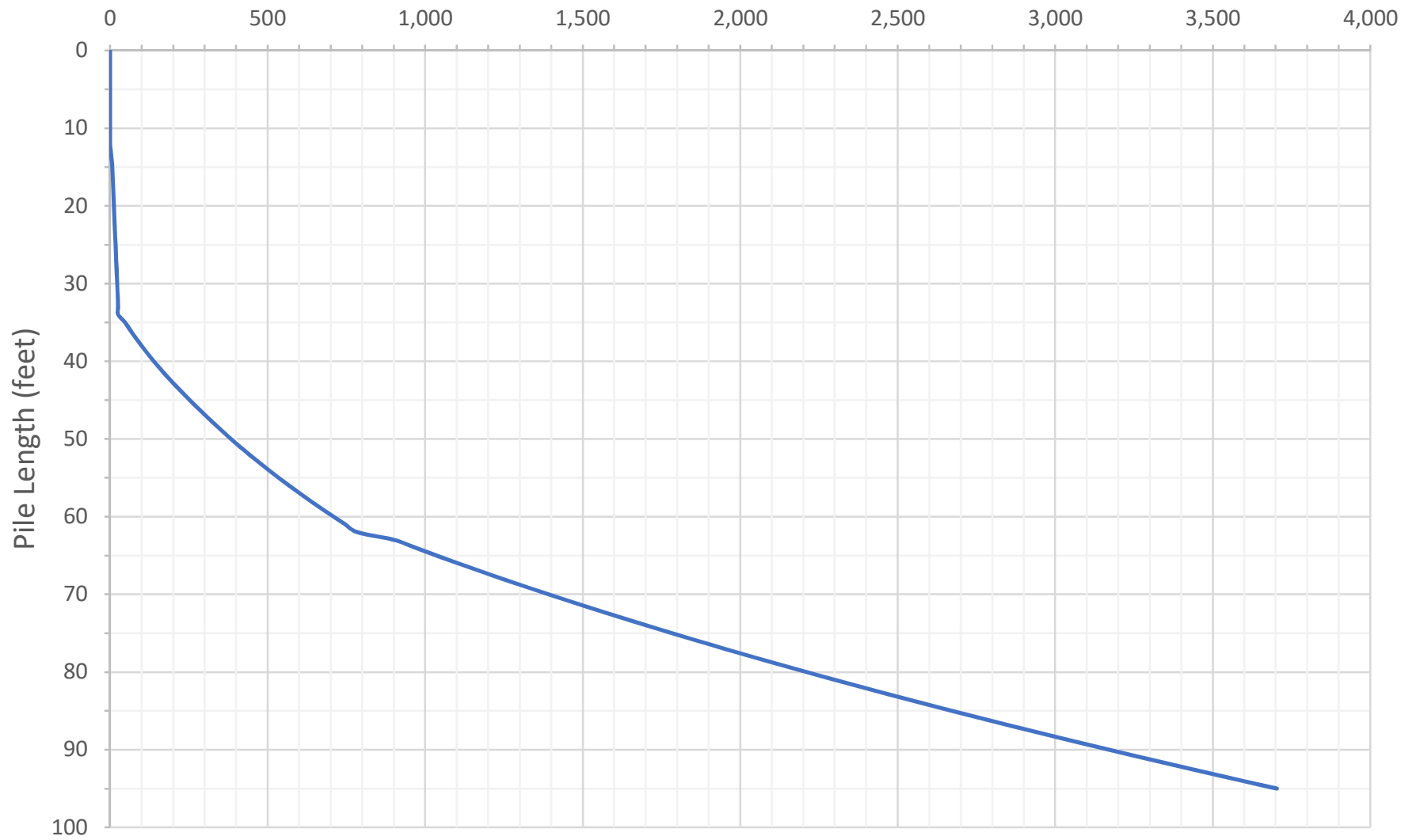
60-inch CISS
Axial Capacity (kips)
Extreme Limit State



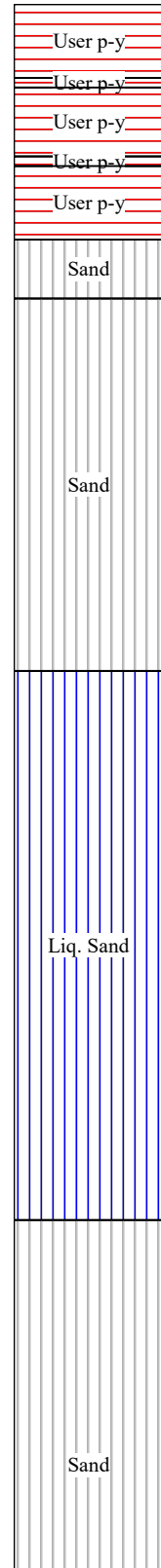
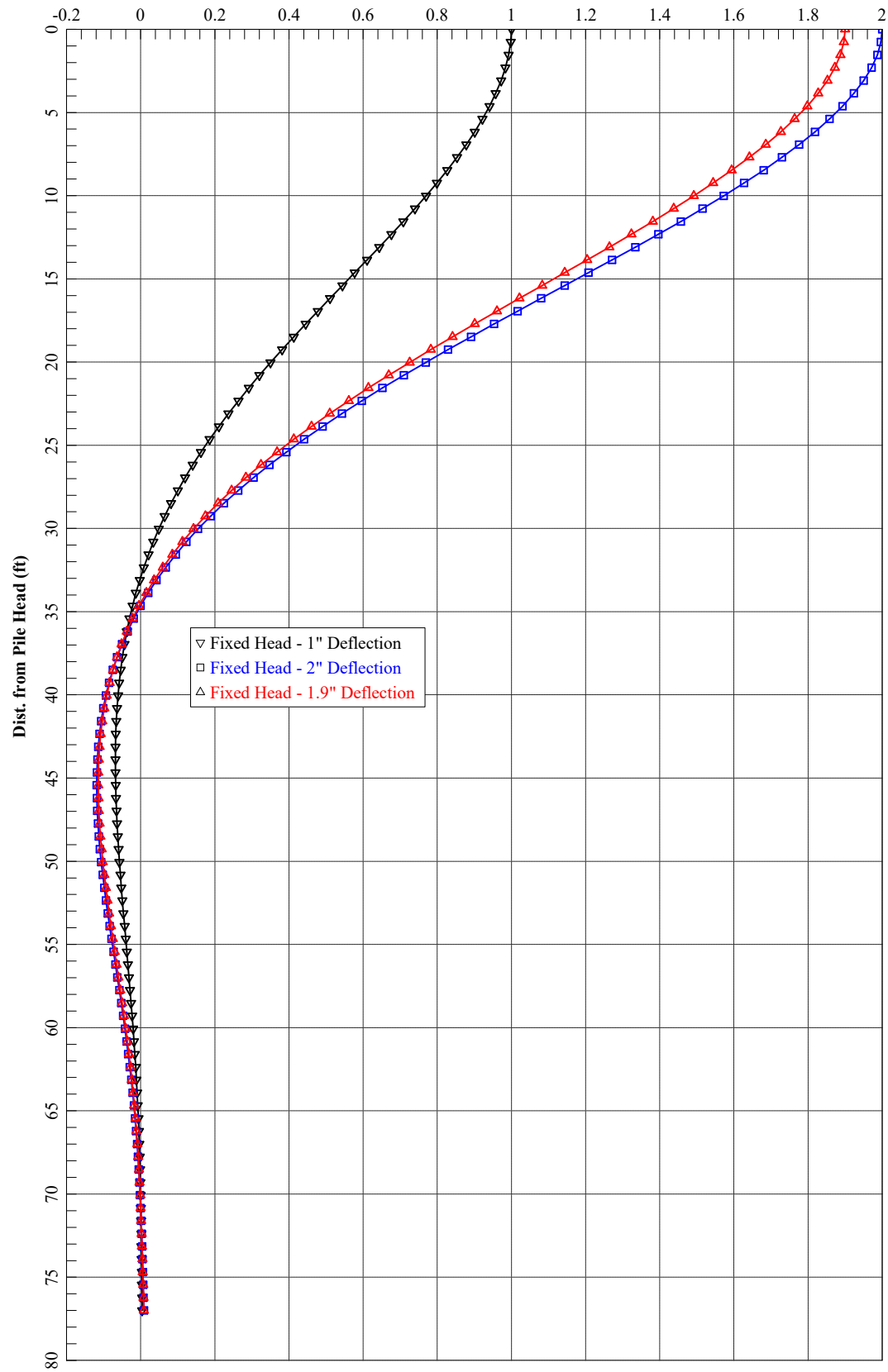
72-inch CISS
Axial Capacity (kips)
Service & Strength Limit States



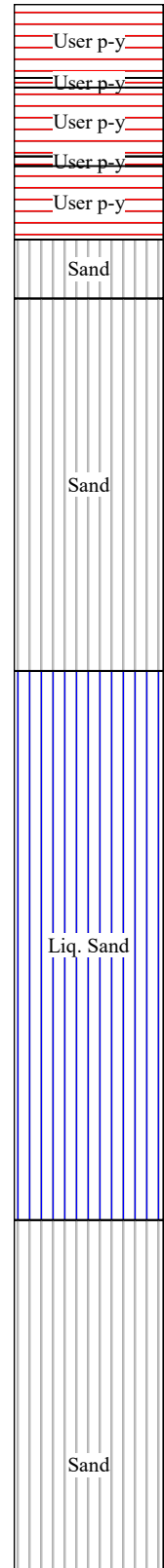
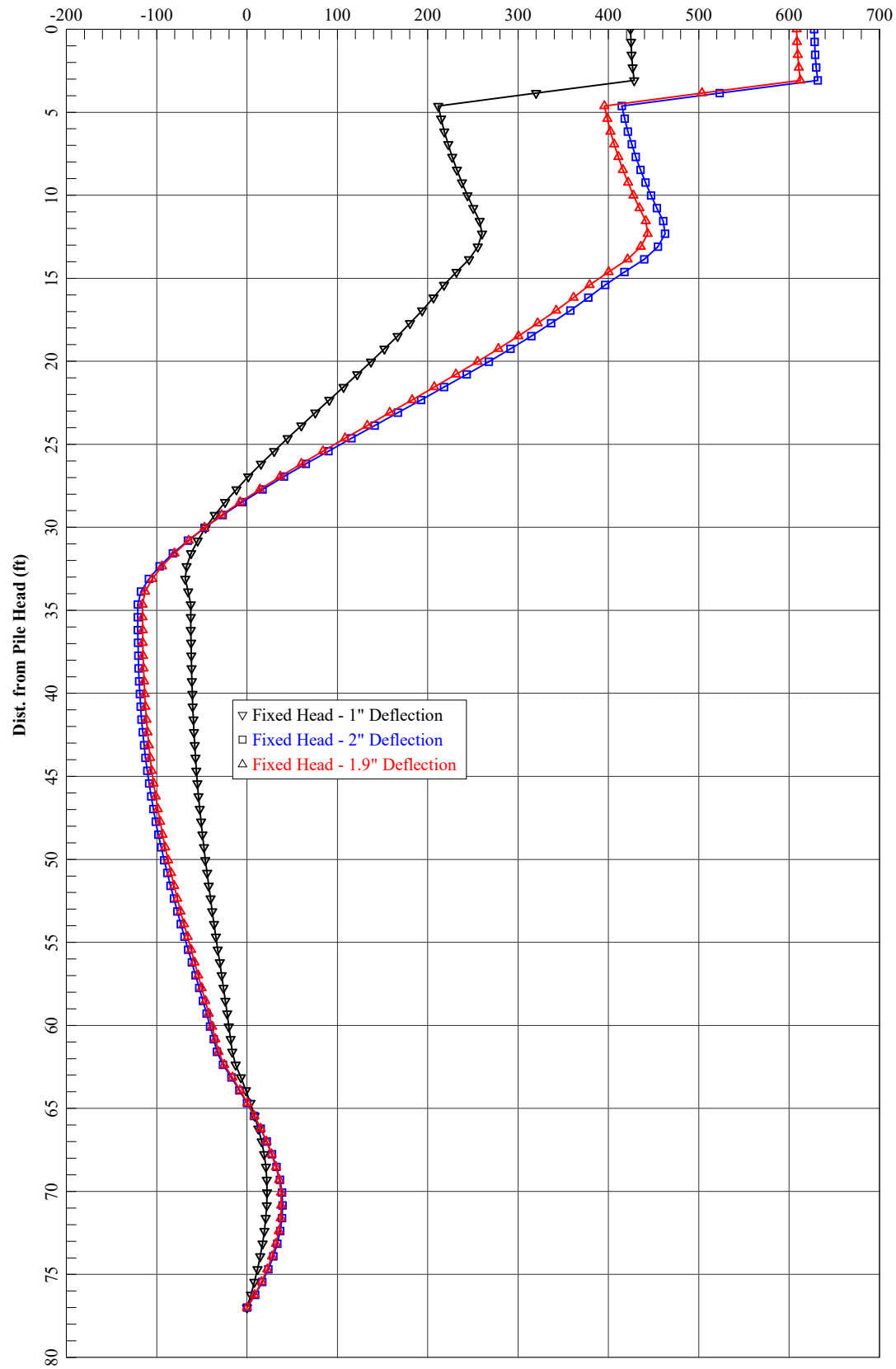
72-inch CISS
Axial Capacity (kips)
Extreme Limit State



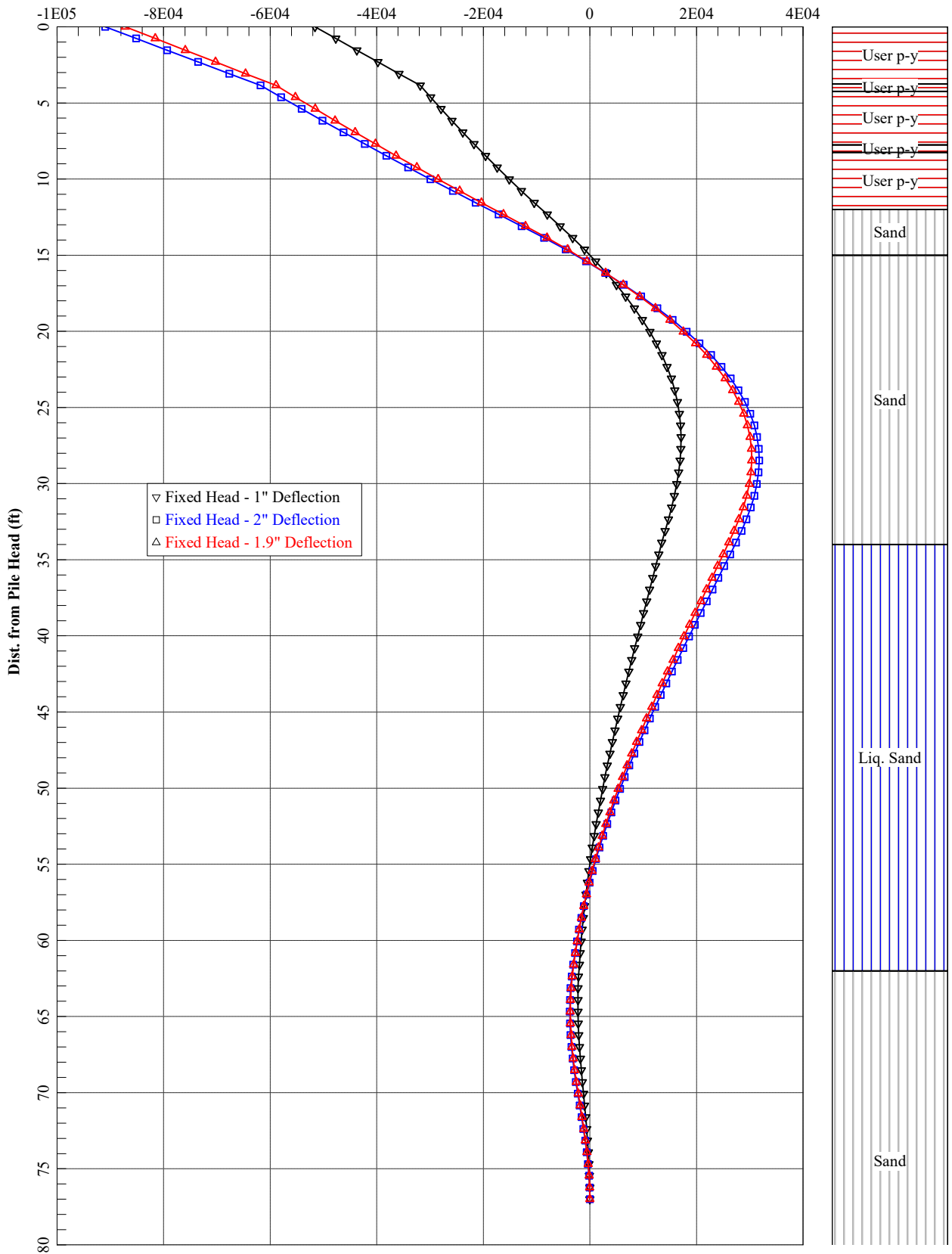
60-inch CISS Pile
Lateral Pile Deflection (inches)



60-inch CISS Pile
Shear Force (kips)



60-inch CISS Pile
Bending Moment (in-kips)



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LPile for Windows, Version 2022-12.005

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method
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Files Used for Analysis

Path to file locations:

\Projects\200000 - Irvine\212100 - 212149\212109\212109001\Electronic Project File\Data Analysis & Calculations\Pile\Lpile\

Name of input data file:

60in LPile_with tiebacks.lp12d

Name of output report file:

60in LPile_with tiebacks.lp12o

Name of plot output file:

60in LPile_with tiebacks.lp12p

Name of runtime message file:

60in LPile_with tiebacks.lp12r

Date and Time of Analysis

Date: June 15, 2023

Time: 15:34:12

Problem Title

Project Name: Via Principessa

Job Number: 212109001

Client:

Engineer:

Description:

Program Options and Settings

Computational Options:

- Conventional Analysis

Engineering Units Used for Data Input and Computations:

- US Customary System Units (pounds, feet, inches)

Analysis Control Options:

- | | | |
|--|---|---------------|
| - Maximum number of iterations allowed | = | 500 |
| - Deflection tolerance for convergence | = | 1.0000E-05 in |
| - Maximum allowable deflection | = | 300.0000 in |
| - Number of pile increments | = | 100 |

Loading Type and Number of Cycles of Loading:

- Static loading specified
- Use of p-y modification factors for p-y curves not selected
- Analysis uses layering correction (Method of Georgiadis)
- Analysis includes loading by one distributed lateral load acting on pile
- Loading by lateral soil movements acting on pile not selected
- Input of shear resistance at the pile tip not selected

- Input of moment resistance at the pile tip not selected
- Computation of pile-head foundation stiffness matrix not selected
- Push-over analysis of pile not selected
- Buckling analysis of pile not selected

Output Options:

- Output files use decimal points to denote decimal symbols.
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (nodal spacing of output points) = 1
- No p-y curves to be computed and reported for user-specified depths
- Print using wide report formats

----- Pile Structural Properties and Geometry -----

Number of pile sections defined	=	1
Total length of pile	=	77.000 ft
Depth of ground surface below top of pile	=	0.0000 ft

Pile diameters used for p-y curve computations are defined using 2 points.

p-y curves are computed using pile diameter values interpolated with depth over the length of the pile. A summary of values of pile diameter vs. depth follows.

Point No.	Depth Below Pile Head feet	Pile Diameter inches
1	0.000	60.0000
2	77.000	60.0000

Input Structural Properties for Pile Sections: -----

Pile Section No. 1:

Section 1 is an elastic pile	
Cross-sectional Shape	= Circular Pile
Length of section	= 77.000000 ft
Width of top of section	= 60.000000 in
Width of bottom of section	= 60.000000 in
Top Area	= 2827. sq. in

Bottom Area	=	2827. sq. in
Moment of Inertia at Top	=	318086. in^4
Moment of Inertia at Bottom	=	318086. in^4
Elastic Modulus	=	3600000. psi

Soil and Rock Layering Information

The soil profile is modelled using 9 layers

Layer 1 is modelled using user-specified p-y curves

Distance from top of pile to top of layer	=	0.0000 ft
Distance from top of pile to bottom of layer	=	3.750000 ft
Effective unit weight at top of layer	=	1.000000 pcf
Effective unit weight at bottom of layer	=	1.000000 pcf

User-specified p-y curve at top of layer 1 at depth = 0.0000 ft

Point No.	y inches	p lb/inch
-----	-----	-----
1	0.0000	0.0000
2	0.500000	0.0000
3	2.000000	0.0000

User-specified p-y curve at bottom of layer 1 at depth = 3.7500 ft

Point No.	y inches	p lb/inch
-----	-----	-----
1	0.0000	0.0000
2	0.500000	0.0000
3	2.000000	0.0000

Layer 2 is modelled using user-specified p-y curves

Distance from top of pile to top of layer	=	3.750000 ft
Distance from top of pile to bottom of layer	=	4.250000 ft
Effective unit weight at top of layer	=	1.000000 pcf
Effective unit weight at bottom of layer	=	1.000000 pcf

User-specified p-y curve at top of layer 2 at depth = 3.7500 ft

Point No.	y inches	p lb/inch
-----	-----	-----
1	0.0000	0.0000
2	0.500000	24000.
3	2.000000	24000.

User-specified p-y curve at bottom of layer 2 at depth = 4.2500 ft

Point No.	y inches	p lb/inch
-----	-----	-----
1	0.0000	0.0000
2	0.500000	24000.
3	2.000000	24000.

Layer 3 is modelled using user-specified p-y curves

Distance from top of pile to top of layer	=	4.250000 ft
Distance from top of pile to bottom of layer	=	7.750000 ft
Effective unit weight at top of layer	=	1.000000 pcf
Effective unit weight at bottom of layer	=	1.000000 pcf

User-specified p-y curve at top of layer 3 at depth = 4.2500 ft

Point No.	y inches	p lb/inch
-----	-----	-----
1	0.0000	0.0000
2	0.500000	0.0000
3	2.000000	0.0000

User-specified p-y curve at bottom of layer 3 at depth = 7.7500 ft

Point No.	y inches	p lb/inch
-----	-----	-----
1	0.0000	0.0000
2	0.500000	0.0000
3	2.000000	0.0000

Layer 4 is modelled using user-specified p-y curves

Distance from top of pile to top of layer	=	7.750000 ft
Distance from top of pile to bottom of layer	=	8.250000 ft
Effective unit weight at top of layer	=	1.000000 pcf
Effective unit weight at bottom of layer	=	1.000000 pcf

User-specified p-y curve at top of layer 4 at depth = 7.7500 ft

Point No.	y inches	p lb/inch
1	0.0000	0.0000
2	0.500000	24000.
3	2.000000	24000.

User-specified p-y curve at bottom of layer 4 at depth = 8.2500 ft

Point No.	y inches	p lb/inch
1	0.0000	0.0000
2	0.500000	24000.
3	2.000000	24000.

Layer 5 is modelled using user-specified p-y curves

Distance from top of pile to top of layer	=	8.250000 ft
Distance from top of pile to bottom of layer	=	12.000000 ft
Effective unit weight at top of layer	=	1.000000 pcf
Effective unit weight at bottom of layer	=	1.000000 pcf

User-specified p-y curve at top of layer 5 at depth = 8.2500 ft

Point No.	y inches	p lb/inch
1	0.0000	0.0000
2	0.500000	0.0000
3	2.000000	0.0000

User-specified p-y curve at bottom of layer 5 at depth = 12.0000 ft

Point No.	y inches	p lb/inch
-----	-----	-----
1	0.0000	0.0000
2	0.500000	0.0000
3	2.000000	0.0000

Layer 6 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer	=	12.000000 ft
Distance from top of pile to bottom of layer	=	15.000000 ft
Effective unit weight at top of layer	=	115.000000 pcf
Effective unit weight at bottom of layer	=	115.000000 pcf
Friction angle at top of layer	=	32.000000 deg.
Friction angle at bottom of layer	=	32.000000 deg.
Subgrade k at top of layer	=	160.000000 pci
Subgrade k at bottom of layer	=	160.000000 pci

Layer 7 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer	=	15.000000 ft
Distance from top of pile to bottom of layer	=	34.000000 ft
Effective unit weight at top of layer	=	52.600000 pcf
Effective unit weight at bottom of layer	=	52.600000 pcf
Friction angle at top of layer	=	32.000000 deg.
Friction angle at bottom of layer	=	32.000000 deg.
Subgrade k at top of layer	=	125.000000 pci
Subgrade k at bottom of layer	=	125.000000 pci

Layer 8 is liquefiable sand, by Rollins et al., 2004

Distance from top of pile to top of layer	=	34.000000 ft
Distance from top of pile to bottom of layer	=	62.000000 ft
Effective unit weight at top of layer	=	52.600000 pcf
Effective unit weight at bottom of layer	=	52.600000 pcf

Warning : The depth of this layer is deeper than the recommended depth limit for using the p-y criteria for liquefied sand.

Please consult the LPILE Technical Manual for additional background information regarding limitations on the use of the liquefied sand criteria.

Layer 9 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer	=	62.000000 ft
Distance from top of pile to bottom of layer	=	100.000000 ft
Effective unit weight at top of layer	=	62.600000 pcf
Effective unit weight at bottom of layer	=	62.600000 pcf
Friction angle at top of layer	=	36.000000 deg.
Friction angle at bottom of layer	=	36.000000 deg.
Subgrade k at top of layer	=	125.000000 pci
Subgrade k at bottom of layer	=	125.000000 pci

(Depth of the lowest soil layer extends 23.000 ft below the pile tip)

**** Warning - Possible Input Data Error ****

Values entered for effective unit weights of soil were outside the limits of 20 pcf to 140 pcf.

The minimum input value, in layer 1, for effective unit weight = 1.00 pcf

This data may be erroneous. Please check your data.

Summary of Input Soil Properties

Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Angle of Friction deg.	kpy pci
1	User Input	0.00	1.0000	--	--
	p-y Curves	3.7500	1.0000	--	--
2	User Input	3.7500	1.0000	--	--
	p-y Curves	4.2500	1.0000	--	--
3	User Input	4.2500	1.0000	--	--
	p-y Curves	7.7500	1.0000	--	--
4	User Input	7.7500	1.0000	--	--
	p-y Curves	8.2500	1.0000	--	--
5	User Input	8.2500	1.0000	--	--
	p-y Curves	12.0000	1.0000	--	--
6	Sand	12.0000	115.0000	32.0000	160.0000
	(Reese, et al.)	15.0000	115.0000	32.0000	160.0000
7	Sand	15.0000	52.6000	32.0000	125.0000
	(Reese, et al.)	34.0000	52.6000	32.0000	125.0000
8	Liquefied	34.0000	52.6000	--	--

9	Sand	62.0000	52.6000	--	--
	Sand	62.0000	62.6000	36.0000	125.0000
	(Reese, et al.)	100.0000	62.6000	36.0000	125.0000

Static Loading Type

Static loading criteria were used when computing p-y curves for all analyses.

Distributed Lateral Loading Used For All Load Cases

Distributed lateral load intensity defined using 2 points

Point No.	Depth X ft	Dist. Load lb/in
1	0.000	0.000
2	12.000	820.000

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 3

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length	Run Analysis
1	5	y = 1.000000 in	S = 0.0000 in/in	890000.	N.A.	Yes
2	5	y = 2.000000 in	S = 0.0000 in/in	890000.	N.A.	Yes
3	5	y = 1.900000 in	S = 0.0000 in/in	890000.	N.A.	Yes

V = shear force applied normal to pile axis

M = bending moment applied to pile head

y = lateral deflection normal to pile axis

S = pile slope relative to original pile batter angle

R = rotational stiffness applied to pile head

Values of top y vs. pile lengths can be computed only for load types with specified shear loading (Load Types 1, 2, and 3).

Thrust force is assumed to be acting axially for all pile batter angles.

Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness

Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 1

Pile Section No. 1:

Moment-curvature properties were derived from elastic section properties

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	0.00	0.00	N.A.	No	0.00	0.00
2	3.7500	3.7500	Yes	No	0.00	144000.
3	4.2500	4.2500	Yes	No	144000.	0.00
4	7.7500	7.7500	Yes	No	144000.	144000.
5	8.2500	8.2500	Yes	No	288000.	0.00
6	12.0000	72.1806	No	No	288000.	89657.
7	15.0000	24.2188	Yes	No	377657.	1861977.
8	34.0000	34.0000	No	No	2239634.	207362.
9	62.0000	34.9348	No	No	2446996.	N.A.

Notes: The F0 integral of Layer n+1 equals the sum of the F0 and F1 integrals for Layer n. Layering correction equivalent depths are computed only for soil types with both shallow-depth and deep-depth expressions for peak lateral load transfer. These soil types are soft and stiff clays, non-liquefied sands, and cemented c-phi soil.

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 1

Pile-head conditions are Displacement and Pile-head Rotation (Loading Type 5)
 Displacement of pile head = 1.000000 inches
 Rotation of pile head = 0.000E+00 radians
 Axial load on pile head = 890000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/inch	Soil Spr. Es*H lb/inch	Distrib. Lat. Load lb/inch
0.00	1.0000	-5.16E+07	424424.	0.00	5182.	1.15E+12	0.00	0.00	13.1542
0.7700	0.9981	-4.77E+07	424896.	-4.01E-04	4811.	1.15E+12	0.00	0.00	52.6167
1.5400	0.9926	-4.37E+07	425625.	-7.69E-04	4440.	1.15E+12	0.00	0.00	105.2333
2.3100	0.9839	-3.98E+07	426841.	-0.00111	4068.	1.15E+12	0.00	0.00	157.8500
3.0800	0.9722	-3.58E+07	428542.	-0.00141	3695.	1.15E+12	0.00	0.00	210.4667
3.8500	0.9578	-3.19E+07	319850.	-0.00168	3319.	1.15E+12	-24000.	231538.	263.0833
4.6200	0.9410	-2.99E+07	211644.	-0.00193	3135.	1.15E+12	0.00	0.00	315.7000
5.3900	0.9220	-2.79E+07	214804.	-0.00217	2947.	1.15E+12	0.00	0.00	368.3167
6.1600	0.9010	-2.59E+07	218451.	-0.00238	2757.	1.15E+12	0.00	0.00	420.9333
6.9300	0.8780	-2.38E+07	222583.	-0.00258	2563.	1.15E+12	0.00	0.00	473.5500
7.7000	0.8532	-2.17E+07	227202.	-0.00277	2365.	1.15E+12	0.00	0.00	526.1667
8.4700	0.8268	-1.96E+07	232307.	-0.00294	2163.	1.15E+12	0.00	0.00	578.7833
9.2400	0.7990	-1.74E+07	237898.	-0.00308	1956.	1.15E+12	0.00	0.00	631.4000
10.0100	0.7698	-1.51E+07	243975.	-0.00322	1743.	1.15E+12	0.00	0.00	684.0167
10.7800	0.7395	-1.28E+07	250538.	-0.00333	1525.	1.15E+12	0.00	0.00	736.6333
11.5500	0.7083	-1.05E+07	257588.	-0.00342	1301.	1.15E+12	0.00	0.00	789.2500
12.3200	0.6763	-8019010.	260190.	-0.00350	1071.	1.15E+12	-295.054	4031.	69.0333
13.0900	0.6436	-5595474.	255423.	-0.00355	842.5053	1.15E+12	-805.858	11569.	0.00
13.8600	0.6106	-3240369.	245769.	-0.00359	620.3857	1.15E+12	-1284.	19424.	0.00
14.6300	0.5773	-994641.	231861.	-0.00361	408.5818	1.15E+12	-1727.	27639.	0.00
15.4000	0.5440	1103712.	218146.	-0.00360	418.8687	1.15E+12	-1242.	21090.	0.00
16.1700	0.5107	3095983.	206301.	-0.00359	606.7680	1.15E+12	-1322.	23922.	0.00
16.9400	0.4777	4975158.	193712.	-0.00356	784.0007	1.15E+12	-1403.	27130.	0.00
17.7100	0.4450	6734255.	180424.	-0.00351	949.9084	1.15E+12	-1474.	30595.	0.00
18.4800	0.4129	8367096.	166525.	-0.00345	1104.	1.15E+12	-1535.	34352.	0.00
19.2500	0.3813	9868336.	152106.	-0.00337	1245.	1.15E+12	-1586.	38436.	0.00
20.0200	0.3505	1.12E+07	137260.	-0.00329	1374.	1.15E+12	-1627.	42894.	0.00
20.7900	0.3206	1.25E+07	122085.	-0.00319	1490.	1.15E+12	-1657.	47777.	0.00
21.5600	0.2915	1.35E+07	106681.	-0.00309	1592.	1.15E+12	-1677.	53146.	0.00
22.3300	0.2635	1.45E+07	91151.	-0.00297	1681.	1.15E+12	-1685.	59077.	0.00
23.1000	0.2365	1.53E+07	75602.	-0.00285	1755.	1.15E+12	-1681.	65661.	0.00
23.8700	0.2107	1.59E+07	60143.	-0.00273	1817.	1.15E+12	-1665.	73013.	0.00
24.6400	0.1861	1.64E+07	44886.	-0.00260	1865.	1.15E+12	-1637.	81279.	0.00
25.4100	0.1627	1.68E+07	29947.	-0.00246	1899.	1.15E+12	-1596.	90649.	0.00
26.1800	0.1406	1.70E+07	15446.	-0.00233	1921.	1.15E+12	-1542.	101377.	0.00

26.9500	0.1197	1.71E+07	1508.	-0.00219	1930.	1.15E+12	-1474.	113815.	0.00
27.7200	0.1001	1.71E+07	-11734.	-0.00205	1927.	1.15E+12	-1392.	128475.	0.00
28.4900	0.08178	1.69E+07	-24140.	-0.00191	1912.	1.15E+12	-1293.	146133.	0.00
29.2600	0.06472	1.67E+07	-35555.	-0.00178	1887.	1.15E+12	-1177.	168062.	0.00
30.0300	0.04891	1.63E+07	-45800.	-0.00165	1853.	1.15E+12	-1040.	196555.	0.00
30.8000	0.03430	1.59E+07	-54661.	-0.00152	1810.	1.15E+12	-877.615	236386.	0.00
31.5700	0.02089	1.53E+07	-61850.	-0.00139	1760.	1.15E+12	-678.486	300162.	0.00
32.3400	0.00861	1.47E+07	-66894.	-0.00127	1705.	1.15E+12	-413.386	443635.	0.00
33.1100	-0.00257	1.41E+07	-68215.	-0.00115	1646.	1.15E+12	127.5115	458905.	0.00
33.8800	-0.01269	1.35E+07	-64998.	-0.00104	1587.	1.15E+12	568.7162	414008.	0.00
34.6500	-0.02181	1.29E+07	-62342.	-9.35E-04	1534.	1.15E+12	6.2599	2652.	0.00
35.4200	-0.02997	1.24E+07	-62264.	-8.33E-04	1480.	1.15E+12	10.6906	3296.	0.00
36.1900	-0.03720	1.18E+07	-62141.	-7.35E-04	1427.	1.15E+12	15.9642	3965.	0.00
36.9600	-0.04356	1.12E+07	-61965.	-6.43E-04	1373.	1.15E+12	22.0372	4675.	0.00
37.7300	-0.04908	1.07E+07	-61730.	-5.54E-04	1320.	1.15E+12	28.8704	5436.	0.00
38.5000	-0.05380	1.01E+07	-61428.	-4.71E-04	1266.	1.15E+12	36.4224	6255.	0.00
39.2700	-0.05777	9526874.	-61054.	-3.91E-04	1213.	1.15E+12	44.6470	7141.	0.00
40.0400	-0.06103	8967548.	-60600.	-3.17E-04	1161.	1.15E+12	53.4916	8098.	0.00
40.8100	-0.06363	8412194.	-60062.	-2.47E-04	1108.	1.15E+12	62.8969	9134.	0.00
41.5800	-0.06559	7861652.	-59436.	-1.81E-04	1056.	1.15E+12	72.7961	10255.	0.00
42.3500	-0.06697	7316803.	-58715.	-1.20E-04	1005.	1.15E+12	83.1158	11467.	0.00
43.1200	-0.06781	6778565.	-57898.	-6.29E-05	954.0875	1.15E+12	93.7751	12779.	0.00
43.8900	-0.06814	6247884.	-56981.	-1.04E-05	904.0367	1.15E+12	104.6868	14197.	0.00
44.6600	-0.06800	5725725.	-55963.	3.79E-05	854.7898	1.15E+12	115.7575	15730.	0.00
45.4300	-0.06744	5213070.	-54842.	8.21E-05	806.4392	1.15E+12	126.8883	17386.	0.00
46.2000	-0.06648	4710903.	-53618.	1.22E-04	759.0777	1.15E+12	137.9751	19176.	0.00
46.9700	-0.06518	4220202.	-52293.	1.58E-04	712.7978	1.15E+12	148.9095	21110.	0.00
47.7400	-0.06356	3741936.	-50867.	1.90E-04	667.6905	1.15E+12	159.5801	23199.	0.00
48.5100	-0.06166	3277045.	-49345.	2.19E-04	623.8447	1.15E+12	169.8724	25455.	0.00
49.2800	-0.05952	2826441.	-47730.	2.43E-04	581.3464	1.15E+12	179.6706	27892.	0.00
50.0500	-0.05717	2390988.	-46028.	2.64E-04	540.2771	1.15E+12	188.8582	30525.	0.00
50.8200	-0.05464	1971502.	-44244.	2.82E-04	500.7135	1.15E+12	197.3192	33369.	0.00
51.5900	-0.05196	1568731.	-42385.	2.96E-04	462.7265	1.15E+12	204.9393	36444.	0.00
52.3600	-0.04917	1183353.	-40461.	3.07E-04	426.3800	1.15E+12	211.6068	39769.	0.00
53.1300	-0.04628	815963.	-38480.	3.15E-04	391.7299	1.15E+12	217.2144	43366.	0.00
53.9000	-0.04334	467064.	-36452.	3.20E-04	358.8239	1.15E+12	221.6602	47259.	0.00
54.6700	-0.04036	137059.	-34389.	3.23E-04	327.6997	1.15E+12	224.8491	51477.	0.00
55.4400	-0.03737	-173758.	-32303.	3.23E-04	331.1609	1.15E+12	226.6945	56050.	0.00
56.2100	-0.03440	-465208.	-30206.	3.20E-04	358.6488	1.15E+12	227.1199	61014.	0.00
56.9800	-0.03145	-737237.	-28113.	3.15E-04	384.3050	1.15E+12	226.0607	66408.	0.00
57.7500	-0.02857	-989917.	-26036.	3.08E-04	408.1362	1.15E+12	223.4659	72278.	0.00
58.5200	-0.02576	-1223452.	-23990.	2.99E-04	430.1619	1.15E+12	219.3007	78677.	0.00
59.2900	-0.02303	-1438182.	-21991.	2.89E-04	450.4140	1.15E+12	213.5484	85664.	0.00
60.0600	-0.02042	-1634585.	-20051.	2.76E-04	468.9375	1.15E+12	206.2139	93311.	0.00
60.8300	-0.01793	-1813273.	-18187.	2.62E-04	485.7903	1.15E+12	197.3262	101701.	0.00
61.6000	-0.01557	-1974993.	-16412.	2.47E-04	501.0429	1.15E+12	186.9429	110934.	0.00
62.3700	-0.01336	-2120622.	-12561.	2.31E-04	514.7777	1.15E+12	646.4289	447035.	0.00
63.1400	-0.01131	-2210920.	-6753.	2.13E-04	523.2940	1.15E+12	610.8506	499059.	0.00

63.9100	-0.00942	-2248917.	-1292.	1.95E-04	526.8778	1.15E+12	571.1796	560081.	0.00
64.6800	-0.00770	-2238000.	3784.	1.77E-04	525.8481	1.15E+12	527.5106	632678.	0.00
65.4500	-0.00615	-2181896.	8438.	1.59E-04	520.5567	1.15E+12	479.8375	720704.	0.00
66.2200	-0.00476	-2084681.	12632.	1.42E-04	511.3879	1.15E+12	427.9661	830338.	0.00
66.9900	-0.00353	-1950788.	16247.	1.26E-04	498.7600	1.15E+12	354.5453	928481.	0.00
67.7600	-0.00244	-1786496.	19031.	1.11E-04	483.2649	1.15E+12	247.9737	939154.	0.00
68.5300	-0.00148	-1600914.	20882.	9.69E-05	465.7619	1.15E+12	152.5795	949826.	0.00
69.3000	-6.48E-04	-1402198.	21898.	8.48E-05	447.0202	1.15E+12	67.3860	960498.	0.00
70.0700	8.33E-05	-1197636.	22169.	7.43E-05	427.7271	1.15E+12	-8.751	971170.	0.00
70.8400	7.25E-04	-993742.	21772.	6.55E-05	408.4970	1.15E+12	-77.088	981842.	0.00
71.6100	0.00129	-796364.	20774.	5.83E-05	389.8815	1.15E+12	-138.951	992515.	0.00
72.3800	0.00180	-610796.	19228.	5.26E-05	372.3798	1.15E+12	-195.680	1003187.	0.00
73.1500	0.00227	-441894.	17176.	4.83E-05	356.4500	1.15E+12	-248.587	1013859.	0.00
73.9200	0.00270	-294187.	14646.	4.54E-05	342.5191	1.15E+12	-298.911	1024531.	0.00
74.6900	0.00310	-171981.	11658.	4.35E-05	330.9933	1.15E+12	-347.772	1035203.	0.00
75.4600	0.00350	-79455.	8222.	4.25E-05	322.2668	1.15E+12	-396.124	1045876.	0.00
76.2300	0.00389	-20744.	4337.	4.21E-05	316.7296	1.15E+12	-444.713	1056548.	0.00
77.0000	0.00428	0.00	0.00	4.20E-05	314.7731	1.15E+12	-494.023	533610.	0.00

* The above values of total stress are combined axial and bending stresses.

Output Summary for Load Case No. 1:

Pile-head deflection	=	1.00000000 inches
Computed slope at pile head	=	0.000000 radians
Maximum bending moment	=	-51602901. inch-lbs
Maximum shear force	=	428542. lbs
Depth of maximum bending moment	=	0.000000 feet below pile head
Depth of maximum shear force	=	3.08000000 feet below pile head
Number of iterations	=	10
Number of zero deflection points	=	2

Computed Values of Pile Loading and Deflection for Lateral Loading for Load Case Number 2

Pile-head conditions are Displacement and Pile-head Rotation (Loading Type 5)

Displacement of pile head	=	2.000000 inches
Rotation of pile head	=	0.000E+00 radians
Axial load on pile head	=	890000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/inch	Soil Spr. Es*H lb/inch	Distrib. Lat. Load lb/inch
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0.00	2.0000	-9.10E+07	627597.	0.00	8895.	1.15E+12	0.00	0.00	13.1542
0.7700	1.9966	-8.52E+07	628063.	-7.11E-04	8348.	1.15E+12	0.00	0.00	52.6167
1.5400	1.9869	-7.94E+07	628792.	-0.00137	7799.	1.15E+12	0.00	0.00	105.2333
2.3100	1.9712	-7.35E+07	630007.	-0.00199	7250.	1.15E+12	0.00	0.00	157.8500
3.0800	1.9501	-6.77E+07	631709.	-0.00256	6698.	1.15E+12	0.00	0.00	210.4667
3.8500	1.9239	-6.18E+07	523017.	-0.00308	6145.	1.15E+12	-24000.	115267.	263.0833
4.6200	1.8931	-5.80E+07	414811.	-0.00357	5782.	1.15E+12	0.00	0.00	315.7000
5.3900	1.8580	-5.41E+07	417971.	-0.00402	5416.	1.15E+12	0.00	0.00	368.3167
6.1600	1.8188	-5.02E+07	421617.	-0.00444	5047.	1.15E+12	0.00	0.00	420.9333
6.9300	1.7759	-4.62E+07	425750.	-0.00483	4674.	1.15E+12	0.00	0.00	473.5500
7.7000	1.7296	-4.22E+07	430368.	-0.00519	4298.	1.15E+12	0.00	0.00	526.1667
8.4700	1.6801	-3.82E+07	435473.	-0.00551	3916.	1.15E+12	0.00	0.00	578.7833
9.2400	1.6278	-3.41E+07	441064.	-0.00580	3530.	1.15E+12	0.00	0.00	631.4000
10.0100	1.5729	-2.99E+07	447142.	-0.00606	3139.	1.15E+12	0.00	0.00	684.0167
10.7800	1.5158	-2.57E+07	453705.	-0.00628	2741.	1.15E+12	0.00	0.00	736.6333
11.5500	1.4568	-2.15E+07	460755.	-0.00647	2338.	1.15E+12	0.00	0.00	789.2500
12.3200	1.3961	-1.71E+07	462574.	-0.00663	1928.	1.15E+12	-464.418	3074.	69.0333
13.0900	1.3342	-1.28E+07	454892.	-0.00675	1522.	1.15E+12	-1267.	8778.	0.00
13.8600	1.2714	-8590088.	439711.	-0.00684	1125.	1.15E+12	-2018.	14669.	0.00
14.6300	1.2079	-4556810.	417836.	-0.00689	744.5447	1.15E+12	-2717.	20781.	0.00
15.4000	1.1440	-755160.	396278.	-0.00691	385.9953	1.15E+12	-1950.	15746.	0.00
16.1700	1.0801	2880094.	377632.	-0.00690	586.4066	1.15E+12	-2086.	17848.	0.00
16.9400	1.0165	6337022.	357737.	-0.00687	912.4438	1.15E+12	-2220.	20179.	0.00
17.7100	0.9533	9604003.	336663.	-0.00680	1221.	1.15E+12	-2342.	22697.	0.00
18.4800	0.8908	1.27E+07	314527.	-0.00671	1510.	1.15E+12	-2450.	25412.	0.00
19.2500	0.8292	1.55E+07	291457.	-0.00660	1779.	1.15E+12	-2544.	28346.	0.00
20.0200	0.7688	1.82E+07	267585.	-0.00646	2028.	1.15E+12	-2623.	31527.	0.00
20.7900	0.7098	2.06E+07	243049.	-0.00631	2256.	1.15E+12	-2688.	34985.	0.00
21.5600	0.6523	2.28E+07	217992.	-0.00613	2461.	1.15E+12	-2736.	38757.	0.00
22.3300	0.5965	2.47E+07	192560.	-0.00594	2645.	1.15E+12	-2769.	42886.	0.00
23.1000	0.5426	2.64E+07	166904.	-0.00573	2806.	1.15E+12	-2785.	47422.	0.00
23.8700	0.4906	2.79E+07	141181.	-0.00551	2945.	1.15E+12	-2783.	52427.	0.00
24.6400	0.4407	2.91E+07	115548.	-0.00528	3061.	1.15E+12	-2765.	57976.	0.00
25.4100	0.3929	3.01E+07	90169.	-0.00505	3154.	1.15E+12	-2728.	64163.	0.00
26.1800	0.3474	3.09E+07	65212.	-0.00480	3226.	1.15E+12	-2674.	71106.	0.00
26.9500	0.3042	3.14E+07	40850.	-0.00455	3276.	1.15E+12	-2600.	78960.	0.00
27.7200	0.2634	3.17E+07	17260.	-0.00429	3304.	1.15E+12	-2506.	87929.	0.00
28.4900	0.2249	3.18E+07	-5372.	-0.00404	3312.	1.15E+12	-2392.	98297.	0.00
29.2600	0.1888	3.17E+07	-26851.	-0.00378	3301.	1.15E+12	-2257.	110469.	0.00
30.0300	0.1550	3.13E+07	-46969.	-0.00353	3271.	1.15E+12	-2098.	125053.	0.00
30.8000	0.1236	3.09E+07	-65499.	-0.00328	3225.	1.15E+12	-1913.	143029.	0.00
31.5700	0.09446	3.02E+07	-82183.	-0.00303	3162.	1.15E+12	-1698.	166120.	0.00
32.3400	0.06759	2.94E+07	-96712.	-0.00279	3086.	1.15E+12	-1447.	197777.	0.00
33.1100	0.04291	2.85E+07	-108685.	-0.00256	2998.	1.15E+12	-1145.	246513.	0.00
33.8800	0.02035	2.74E+07	-117476.	-0.00233	2901.	1.15E+12	-758.002	344139.	0.00
34.6500	-1.62E-04	2.63E+07	-120978.	-0.00211	2797.	1.15E+12	0.01472	838.1099	0.00
35.4200	-0.01871	2.52E+07	-120950.	-0.00191	2693.	1.15E+12	6.0073	2966.	0.00

36.1900	-0.03539	2.41E+07	-120853.	-0.00171	2589.	1.15E+12	15.0251	3923.	0.00
36.9600	-0.05026	2.30E+07	-120663.	-0.00152	2485.	1.15E+12	26.1689	4811.	0.00
37.7300	-0.06342	2.19E+07	-120361.	-0.00134	2381.	1.15E+12	39.1608	5706.	0.00
38.5000	-0.07494	2.08E+07	-119931.	-0.00116	2277.	1.15E+12	53.8197	6636.	0.00
39.2700	-0.08492	1.97E+07	-119359.	-1.00E-03	2174.	1.15E+12	69.9928	7616.	0.00
40.0400	-0.09342	1.86E+07	-118632.	-8.45E-04	2071.	1.15E+12	87.5329	8658.	0.00
40.8100	-0.101	1.75E+07	-117736.	-6.99E-04	1968.	1.15E+12	106.2903	9769.	0.00
41.5800	-0.106	1.65E+07	-116663.	-5.62E-04	1867.	1.15E+12	126.1085	10957.	0.00
42.3500	-0.111	1.54E+07	-115402.	-4.34E-04	1766.	1.15E+12	146.8223	12230.	0.00
43.1200	-0.114	1.43E+07	-113946.	-3.14E-04	1666.	1.15E+12	168.2577	13595.	0.00
43.8900	-0.117	1.33E+07	-112290.	-2.02E-04	1568.	1.15E+12	190.2313	15058.	0.00
44.6600	-0.118	1.23E+07	-110429.	-9.94E-05	1471.	1.15E+12	212.5515	16629.	0.00
45.4300	-0.119	1.12E+07	-108361.	-4.58E-06	1376.	1.15E+12	235.0188	18315.	0.00
46.2000	-0.118	1.03E+07	-106086.	8.22E-05	1282.	1.15E+12	257.4274	20126.	0.00
46.9700	-0.117	9284965.	-103605.	1.61E-04	1190.	1.15E+12	279.5660	22070.	0.00
47.7400	-0.115	8337956.	-100922.	2.32E-04	1101.	1.15E+12	301.2195	24158.	0.00
48.5100	-0.113	7416112.	-98042.	2.96E-04	1014.	1.15E+12	322.1704	26400.	0.00
49.2800	-0.110	6521281.	-94972.	3.52E-04	929.8220	1.15E+12	342.2007	28810.	0.00
50.0500	-0.106	5655234.	-91723.	4.01E-04	848.1415	1.15E+12	361.0933	31401.	0.00
50.8200	-0.102	4819641.	-88306.	4.43E-04	769.3333	1.15E+12	378.6341	34186.	0.00
51.5900	-0.09806	4016056.	-84733.	4.79E-04	693.5438	1.15E+12	394.6139	37183.	0.00
52.3600	-0.09349	3245894.	-81021.	5.08E-04	620.9068	1.15E+12	408.8302	40407.	0.00
53.1300	-0.08867	2510423.	-77187.	5.31E-04	551.5414	1.15E+12	421.0895	43880.	0.00
53.9000	-0.08367	1810736.	-73249.	5.49E-04	485.5511	1.15E+12	431.2093	47622.	0.00
54.6700	-0.07853	1147745.	-69229.	5.61E-04	423.0217	1.15E+12	439.0204	51658.	0.00
55.4400	-0.07330	522160.	-65148.	5.68E-04	364.0202	1.15E+12	444.3692	56014.	0.00
56.2100	-0.06804	-65520.	-61029.	5.69E-04	320.9526	1.15E+12	447.1201	60721.	0.00
56.9800	-0.06278	-615022.	-56897.	5.67E-04	372.7784	1.15E+12	447.1579	65813.	0.00
57.7500	-0.05757	-1126306.	-52779.	5.60E-04	420.9996	1.15E+12	444.3910	71329.	0.00
58.5200	-0.05244	-1599574.	-48698.	5.49E-04	465.6355	1.15E+12	438.7540	77312.	0.00
59.2900	-0.04743	-2035276.	-44684.	5.34E-04	506.7284	1.15E+12	430.2110	83815.	0.00
60.0600	-0.04257	-2434113.	-40762.	5.16E-04	544.3443	1.15E+12	418.7596	90894.	0.00
60.8300	-0.03789	-2797035.	-36958.	4.95E-04	578.5730	1.15E+12	404.4350	98618.	0.00
61.6000	-0.03343	-3125242.	-33301.	4.71E-04	609.5276	1.15E+12	387.3151	107068.	0.00
62.3700	-0.02919	-3420174.	-26708.	4.45E-04	637.3438	1.15E+12	1040.	329080.	0.00
63.1400	-0.02521	-3626120.	-17311.	4.16E-04	656.7674	1.15E+12	994.4730	364491.	0.00
63.9100	-0.02150	-3746919.	-8359.	3.86E-04	668.1605	1.15E+12	943.1765	405334.	0.00
64.6800	-0.01807	-3786944.	91.1629	3.56E-04	671.9354	1.15E+12	885.8110	452945.	0.00
65.4500	-0.01492	-3751089.	7983.	3.26E-04	668.5538	1.15E+12	822.3776	509216.	0.00
66.2200	-0.01205	-3644773.	15260.	2.96E-04	658.5266	1.15E+12	752.7020	576972.	0.00
66.9900	-0.00946	-3473951.	21862.	2.67E-04	642.4157	1.15E+12	676.3000	660729.	0.00
67.7600	-0.00712	-3245157.	27722.	2.40E-04	620.8372	1.15E+12	592.1303	768410.	0.00
68.5300	-0.00502	-2965594.	32759.	2.15E-04	594.4704	1.15E+12	498.0768	915915.	0.00
69.3000	-0.00315	-2643309.	36573.	1.92E-04	564.0744	1.15E+12	327.4754	960498.	0.00
70.0700	-0.00147	-2292889.	38801.	1.72E-04	531.0249	1.15E+12	154.8174	971170.	0.00
70.8400	3.34E-05	-1929100.	39500.	1.55E-04	496.7144	1.15E+12	-3.550	981842.	0.00
71.6100	0.00140	-1565485.	38791.	1.41E-04	462.4205	1.15E+12	-149.947	992515.	0.00
72.3800	0.00264	-1214569.	36773.	1.30E-04	429.3241	1.15E+12	-286.819	1003187.	0.00

73.1500	0.00380	-888060.	33523.	1.21E-04	398.5297	1.15E+12	-416.634	1013859.	0.00
73.9200	0.00489	-597063.	29095.	1.15E-04	371.0846	1.15E+12	-541.775	1024531.	0.00
74.6900	0.00593	-352283.	23522.	1.12E-04	347.9983	1.15E+12	-664.444	1035203.	0.00
75.4600	0.00695	-164207.	16819.	1.10E-04	330.2601	1.15E+12	-786.553	1045876.	0.00
76.2300	0.00796	-43275.	8982.	1.09E-04	318.8545	1.15E+12	-909.615	1056548.	0.00
77.0000	0.00896	0.00	0.00	1.09E-04	314.7731	1.15E+12	-1035.	533610.	0.00

* The above values of total stress are combined axial and bending stresses.

Output Summary for Load Case No. 2:

Pile-head deflection	=	2.00000000 inches
Computed slope at pile head	=	0.000000 radians
Maximum bending moment	=	-90976888. inch-lbs
Maximum shear force	=	631709. lbs
Depth of maximum bending moment	=	0.000000 feet below pile head
Depth of maximum shear force	=	3.08000000 feet below pile head
Number of iterations	=	11
Number of zero deflection points	=	2

----- Computed Values of Pile Loading and Deflection for Lateral Loading for Load Case Number 3 -----

Pile-head conditions are Displacement and Pile-head Rotation (Loading Type 5)

Displacement of pile head	=	1.900000 inches
Rotation of pile head	=	0.000E+00 radians
Axial load on pile head	=	890000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/inch	Soil Spr. Es*H lb/inch	Distrib. Lat. Load lb/inch
0.00	1.9000	-8.72E+07	608224.	0.00	8537.	1.15E+12	0.00	0.00	13.1542
0.7700	1.8967	-8.16E+07	608690.	-6.81E-04	8007.	1.15E+12	0.00	0.00	52.6167
1.5400	1.8874	-7.59E+07	609419.	-0.00132	7475.	1.15E+12	0.00	0.00	105.2333
2.3100	1.8724	-7.03E+07	610635.	-0.00191	6943.	1.15E+12	0.00	0.00	157.8500
3.0800	1.8522	-6.46E+07	612336.	-0.00245	6408.	1.15E+12	0.00	0.00	210.4667
3.8500	1.8271	-5.89E+07	503644.	-0.00295	5872.	1.15E+12	-24000.	121369.	263.0833
4.6200	1.7977	-5.53E+07	395438.	-0.00341	5526.	1.15E+12	0.00	0.00	315.7000
5.3900	1.7641	-5.16E+07	398598.	-0.00384	5177.	1.15E+12	0.00	0.00	368.3167
6.1600	1.7267	-4.78E+07	402244.	-0.00424	4825.	1.15E+12	0.00	0.00	420.9333
6.9300	1.6858	-4.41E+07	406377.	-0.00461	4469.	1.15E+12	0.00	0.00	473.5500
7.7000	1.6415	-4.02E+07	410996.	-0.00495	4110.	1.15E+12	0.00	0.00	526.1667

8.4700	1.5943	-3.64E+07	416101.	-0.00526	3745.	1.15E+12	0.00	0.00	578.7833
9.2400	1.5443	-3.25E+07	421692.	-0.00554	3376.	1.15E+12	0.00	0.00	631.4000
10.0100	1.4919	-2.85E+07	427769.	-0.00578	3002.	1.15E+12	0.00	0.00	684.0167
10.7800	1.4374	-2.45E+07	434332.	-0.00600	2622.	1.15E+12	0.00	0.00	736.6333
11.5500	1.3811	-2.04E+07	441382.	-0.00618	2235.	1.15E+12	0.00	0.00	789.2500
12.3200	1.3232	-1.62E+07	443278.	-0.00633	1843.	1.15E+12	-447.828	3127.	69.0333
13.0900	1.2642	-1.21E+07	435880.	-0.00644	1453.	1.15E+12	-1223.	8936.	0.00
13.8600	1.2042	-8040048.	421233.	-0.00652	1073.	1.15E+12	-1948.	14945.	0.00
14.6300	1.1436	-4177102.	400120.	-0.00657	708.7328	1.15E+12	-2622.	21187.	0.00
15.4000	1.0828	-537766.	379304.	-0.00659	365.4920	1.15E+12	-1883.	16070.	0.00
16.1700	1.0219	2940825.	361290.	-0.00658	592.1344	1.15E+12	-2016.	18229.	0.00
16.9400	0.9612	6247105.	342064.	-0.00654	903.9633	1.15E+12	-2146.	20626.	0.00
17.7100	0.9009	9369788.	321699.	-0.00648	1198.	1.15E+12	-2263.	23205.	0.00
18.4800	0.8414	1.23E+07	300313.	-0.00639	1475.	1.15E+12	-2366.	25986.	0.00
19.2500	0.7828	1.50E+07	278032.	-0.00628	1732.	1.15E+12	-2456.	28993.	0.00
20.0200	0.7253	1.75E+07	254987.	-0.00615	1969.	1.15E+12	-2532.	32255.	0.00
20.7900	0.6691	1.98E+07	231311.	-0.00600	2186.	1.15E+12	-2593.	35804.	0.00
21.5600	0.6144	2.19E+07	207142.	-0.00583	2382.	1.15E+12	-2638.	39677.	0.00
22.3300	0.5614	2.38E+07	182626.	-0.00565	2556.	1.15E+12	-2668.	43919.	0.00
23.1000	0.5101	2.54E+07	157908.	-0.00545	2709.	1.15E+12	-2682.	48584.	0.00
23.8700	0.4607	2.68E+07	133141.	-0.00524	2840.	1.15E+12	-2679.	53736.	0.00
24.6400	0.4133	2.79E+07	108479.	-0.00502	2949.	1.15E+12	-2659.	59454.	0.00
25.4100	0.3679	2.89E+07	84083.	-0.00479	3036.	1.15E+12	-2622.	65837.	0.00
26.1800	0.3247	2.96E+07	60117.	-0.00455	3103.	1.15E+12	-2566.	73012.	0.00
26.9500	0.2838	3.00E+07	36748.	-0.00431	3148.	1.15E+12	-2492.	81143.	0.00
27.7200	0.2450	3.03E+07	14153.	-0.00407	3173.	1.15E+12	-2399.	90451.	0.00
28.4900	0.2086	3.04E+07	-7488.	-0.00382	3179.	1.15E+12	-2285.	101242.	0.00
29.2600	0.1744	3.02E+07	-27981.	-0.00358	3166.	1.15E+12	-2150.	113958.	0.00
30.0300	0.1424	2.99E+07	-47122.	-0.00334	3136.	1.15E+12	-1992.	129275.	0.00
30.8000	0.1127	2.94E+07	-64683.	-0.00310	3089.	1.15E+12	-1809.	148296.	0.00
31.5700	0.08516	2.88E+07	-80405.	-0.00286	3028.	1.15E+12	-1595.	173007.	0.00
32.3400	0.05978	2.80E+07	-93975.	-0.00263	2954.	1.15E+12	-1343.	207532.	0.00
33.1100	0.03648	2.71E+07	-104970.	-0.00241	2868.	1.15E+12	-1037.	262714.	0.00
33.8800	0.01520	2.61E+07	-112694.	-0.00220	2774.	1.15E+12	-634.757	385862.	0.00
34.6500	-0.00413	2.50E+07	-115623.	-0.00199	2675.	1.15E+12	0.8004	1789.	0.00
35.4200	-0.02160	2.40E+07	-115586.	-0.00179	2576.	1.15E+12	7.1615	3063.	0.00
36.1900	-0.03728	2.29E+07	-115479.	-0.00160	2477.	1.15E+12	16.0074	3967.	0.00
36.9600	-0.05126	2.19E+07	-115282.	-0.00142	2377.	1.15E+12	26.7925	4830.	0.00
37.7300	-0.06360	2.08E+07	-114976.	-0.00125	2278.	1.15E+12	39.2922	5709.	0.00
38.5000	-0.07439	1.98E+07	-114548.	-0.00109	2179.	1.15E+12	53.3493	6627.	0.00
39.2700	-0.08370	1.87E+07	-113984.	-9.33E-04	2080.	1.15E+12	68.8262	7598.	0.00
40.0400	-0.09162	1.77E+07	-113270.	-7.86E-04	1982.	1.15E+12	85.5875	8631.	0.00
40.8100	-0.09823	1.66E+07	-112397.	-6.47E-04	1884.	1.15E+12	103.4940	9735.	0.00
41.5800	-0.104	1.56E+07	-111353.	-5.17E-04	1787.	1.15E+12	122.3987	10918.	0.00
42.3500	-0.108	1.46E+07	-110131.	-3.96E-04	1691.	1.15E+12	142.1461	12185.	0.00
43.1200	-0.111	1.36E+07	-108723.	-2.82E-04	1595.	1.15E+12	162.5713	13545.	0.00
43.8900	-0.113	1.26E+07	-107124.	-1.76E-04	1501.	1.15E+12	183.5008	15005.	0.00
44.6600	-0.114	1.16E+07	-105331.	-7.88E-05	1409.	1.15E+12	204.7523	16572.	0.00

45.4300	-0.114	1.06E+07	-103340.	1.09E-05	1318.	1.15E+12	226.1363	18256.	0.00
46.2000	-0.114	9691366.	-101152.	9.29E-05	1229.	1.15E+12	247.4564	20064.	0.00
46.9700	-0.113	8766199.	-98768.	1.67E-04	1142.	1.15E+12	268.5114	22007.	0.00
47.7400	-0.111	7863376.	-96192.	2.34E-04	1056.	1.15E+12	289.0958	24094.	0.00
48.5100	-0.108	6984713.	-93429.	2.94E-04	973.5301	1.15E+12	309.0021	26338.	0.00
49.2800	-0.105	6131969.	-90486.	3.47E-04	893.1043	1.15E+12	328.0218	28749.	0.00
50.0500	-0.102	5306823.	-87372.	3.93E-04	815.2814	1.15E+12	345.9473	31342.	0.00
50.8200	-0.09816	4510861.	-84099.	4.33E-04	740.2110	1.15E+12	362.5736	34131.	0.00
51.5900	-0.09399	3745556.	-80679.	4.66E-04	668.0319	1.15E+12	377.7006	37133.	0.00
52.3600	-0.08954	3012249.	-77127.	4.94E-04	598.8708	1.15E+12	391.1342	40364.	0.00
53.1300	-0.08486	2312137.	-73459.	5.15E-04	532.8403	1.15E+12	402.6892	43845.	0.00
53.9000	-0.08002	1646252.	-69694.	5.31E-04	470.0379	1.15E+12	412.1905	47597.	0.00
54.6700	-0.07505	1015449.	-65852.	5.42E-04	410.5443	1.15E+12	419.4761	51646.	0.00
55.4400	-0.07000	420393.	-61953.	5.48E-04	354.4221	1.15E+12	424.3988	56017.	0.00
56.2100	-0.06493	-138457.	-58021.	5.49E-04	327.8315	1.15E+12	426.8285	60741.	0.00
56.9800	-0.05986	-660856.	-54078.	5.46E-04	377.1012	1.15E+12	426.6553	65854.	0.00
57.7500	-0.05485	-1146784.	-50149.	5.38E-04	422.9311	1.15E+12	423.7914	71394.	0.00
58.5200	-0.04992	-1596454.	-46259.	5.27E-04	465.3413	1.15E+12	418.1749	77406.	0.00
59.2900	-0.04511	-2010316.	-42434.	5.13E-04	504.3743	1.15E+12	409.7721	83942.	0.00
60.0600	-0.04044	-2389058.	-38699.	4.95E-04	540.0950	1.15E+12	398.5820	91060.	0.00
60.8300	-0.03596	-2733612.	-35081.	4.74E-04	572.5914	1.15E+12	384.6401	98831.	0.00
61.6000	-0.03168	-3045146.	-31603.	4.51E-04	601.9733	1.15E+12	368.0233	107335.	0.00
62.3700	-0.02763	-3325056.	-25258.	4.25E-04	628.3728	1.15E+12	1005.	336249.	0.00
63.1400	-0.02382	-3518904.	-16174.	3.98E-04	646.6554	1.15E+12	960.8624	372662.	0.00
63.9100	-0.02028	-3630482.	-7529.	3.69E-04	657.1788	1.15E+12	910.2998	414713.	0.00
64.6800	-0.01701	-3664100.	621.5224	3.39E-04	660.3495	1.15E+12	853.8412	463808.	0.00
65.4500	-0.01401	-3624577.	8223.	3.10E-04	656.6219	1.15E+12	791.4884	521942.	0.00
66.2200	-0.01128	-3517237.	15220.	2.81E-04	646.4982	1.15E+12	723.0669	592111.	0.00
66.9900	-0.00882	-3347930.	21555.	2.53E-04	630.5302	1.15E+12	648.0891	679133.	0.00
67.7600	-0.00660	-3123069.	27162.	2.27E-04	609.3226	1.15E+12	565.5016	791550.	0.00
68.5300	-0.00462	-2849720.	31960.	2.03E-04	583.5419	1.15E+12	473.1459	946748.	0.00
69.3000	-0.00285	-2535785.	35513.	1.81E-04	553.9334	1.15E+12	295.9189	960498.	0.00
70.0700	-0.00126	-2196417.	37495.	1.62E-04	521.9262	1.15E+12	132.9345	971170.	0.00
70.8400	1.53E-04	-1845554.	38033.	1.46E-04	488.8349	1.15E+12	-16.303	981842.	0.00
71.6100	0.00143	-1495960.	37247.	1.33E-04	455.8633	1.15E+12	-154.036	992515.	0.00
72.3800	0.00260	-1159418.	35229.	1.22E-04	424.1226	1.15E+12	-282.618	1003187.	0.00
73.1500	0.00369	-846929.	32055.	1.14E-04	394.6504	1.15E+12	-404.415	1013859.	0.00
73.9200	0.00471	-568911.	27776.	1.08E-04	368.4295	1.15E+12	-521.711	1024531.	0.00
74.6900	0.00568	-335398.	22425.	1.04E-04	346.4059	1.15E+12	-636.611	1035203.	0.00
75.4600	0.00663	-156215.	16014.	1.02E-04	329.5064	1.15E+12	-750.936	1045876.	0.00
76.2300	0.00757	-41135.	8544.	1.02E-04	318.6527	1.15E+12	-866.130	1056548.	0.00
77.0000	0.00851	0.00	0.00	1.01E-04	314.7731	1.15E+12	-983.141	533610.	0.00

* The above values of total stress are combined axial and bending stresses.

Output Summary for Load Case No. 3:

Pile-head deflection = 1.90000000 inches
 Computed slope at pile head = 0.000000 radians
 Maximum bending moment = -87182202. inch-lbs
 Maximum shear force = 612336. lbs
 Depth of maximum bending moment = 0.000000 feet below pile head
 Depth of maximum shear force = 3.08000000 feet below pile head
 Number of iterations = 11
 Number of zero deflection points = 2

 Summary of Pile-head Responses for Conventional Analyses

Definitions of Pile-head Loading Conditions:

Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
 Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
 Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
 Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
 Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

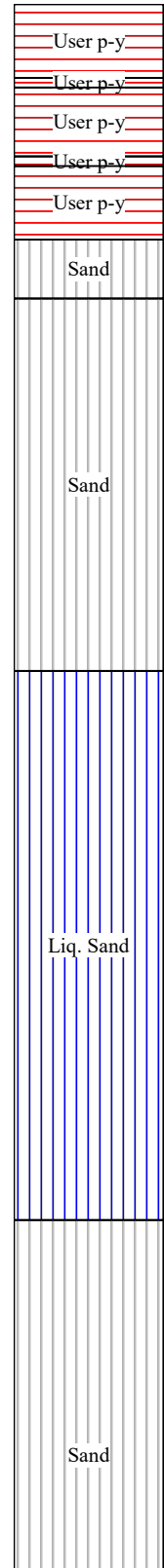
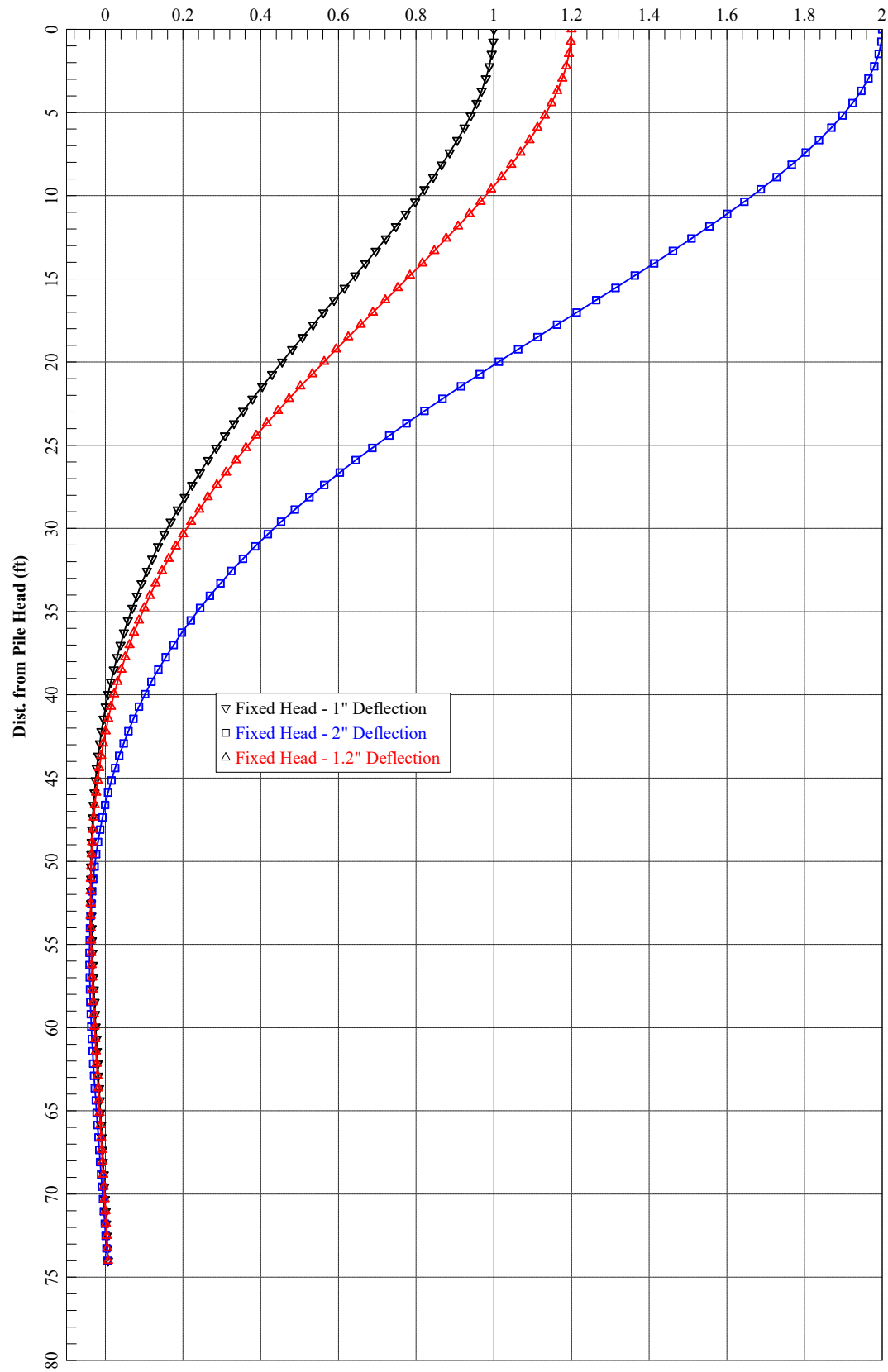
Load Case No.	Load Type 1	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	y, in	1.0000	S, rad	0.00	890000.	1.0000	0.00	428542.	-5.16E+07
2	y, in	2.0000	S, rad	0.00	890000.	2.0000	0.00	631709.	-9.10E+07
3	y, in	1.9000	S, rad	0.00	890000.	1.9000	0.00	612336.	-8.72E+07

Maximum pile-head deflection = 2.000000000 inches
 Maximum pile-head rotation = 0.000000000 radians = 0.000000 deg.

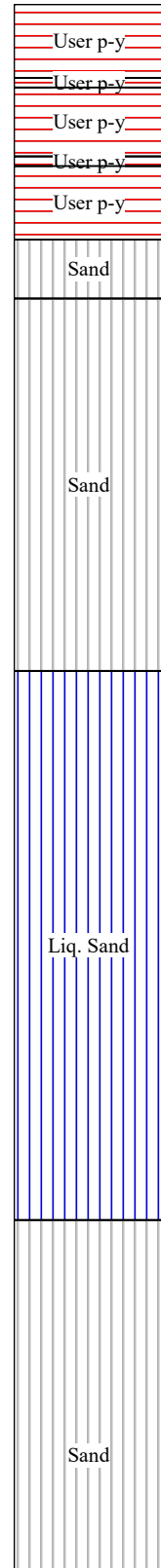
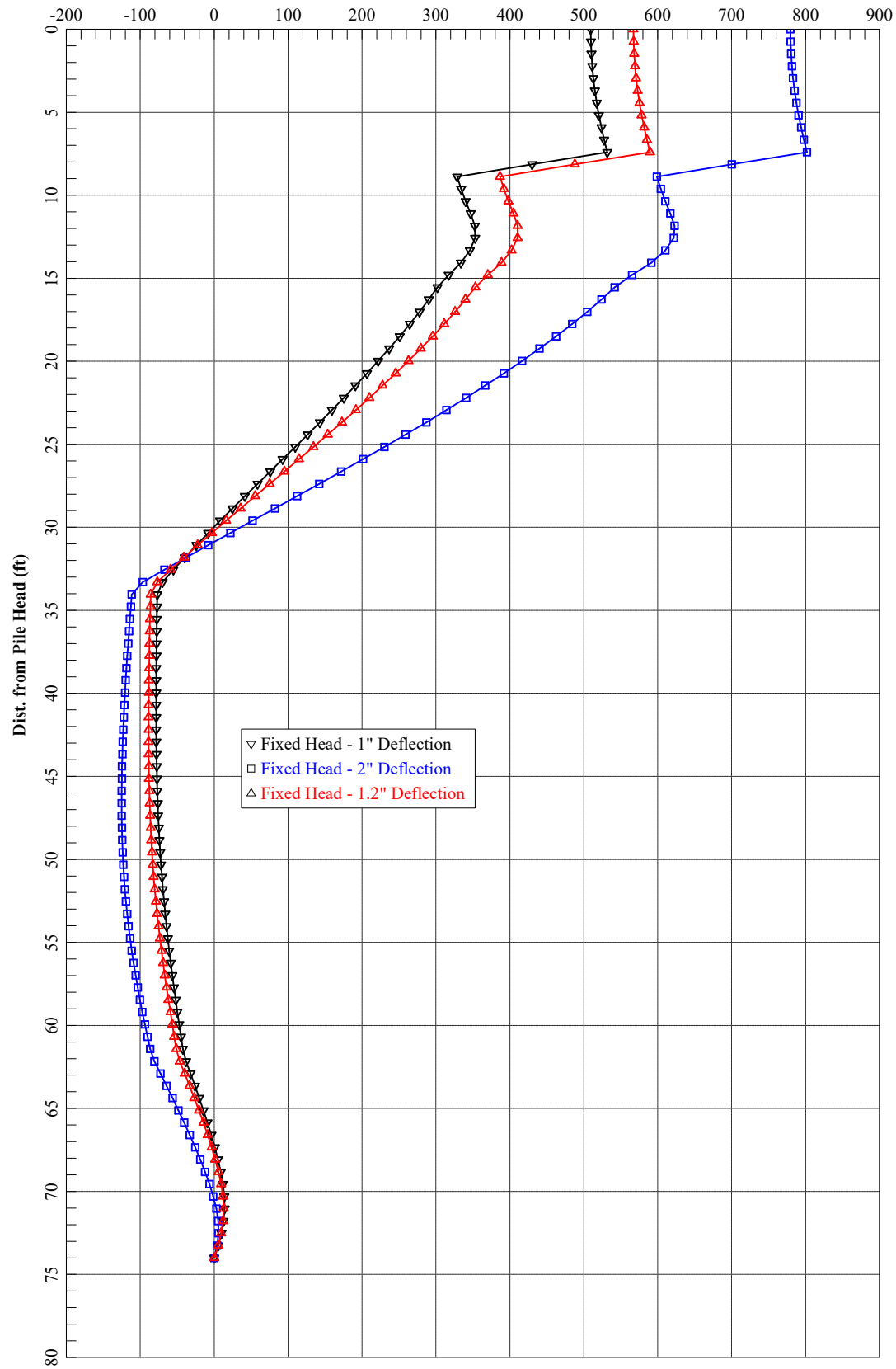
No error or warning messages were generated by this analysis.

The analysis ended normally.

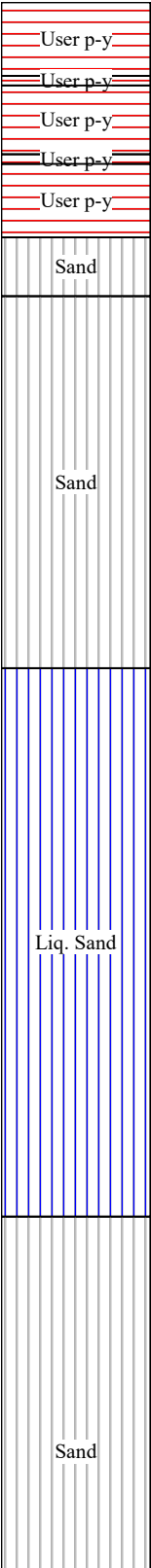
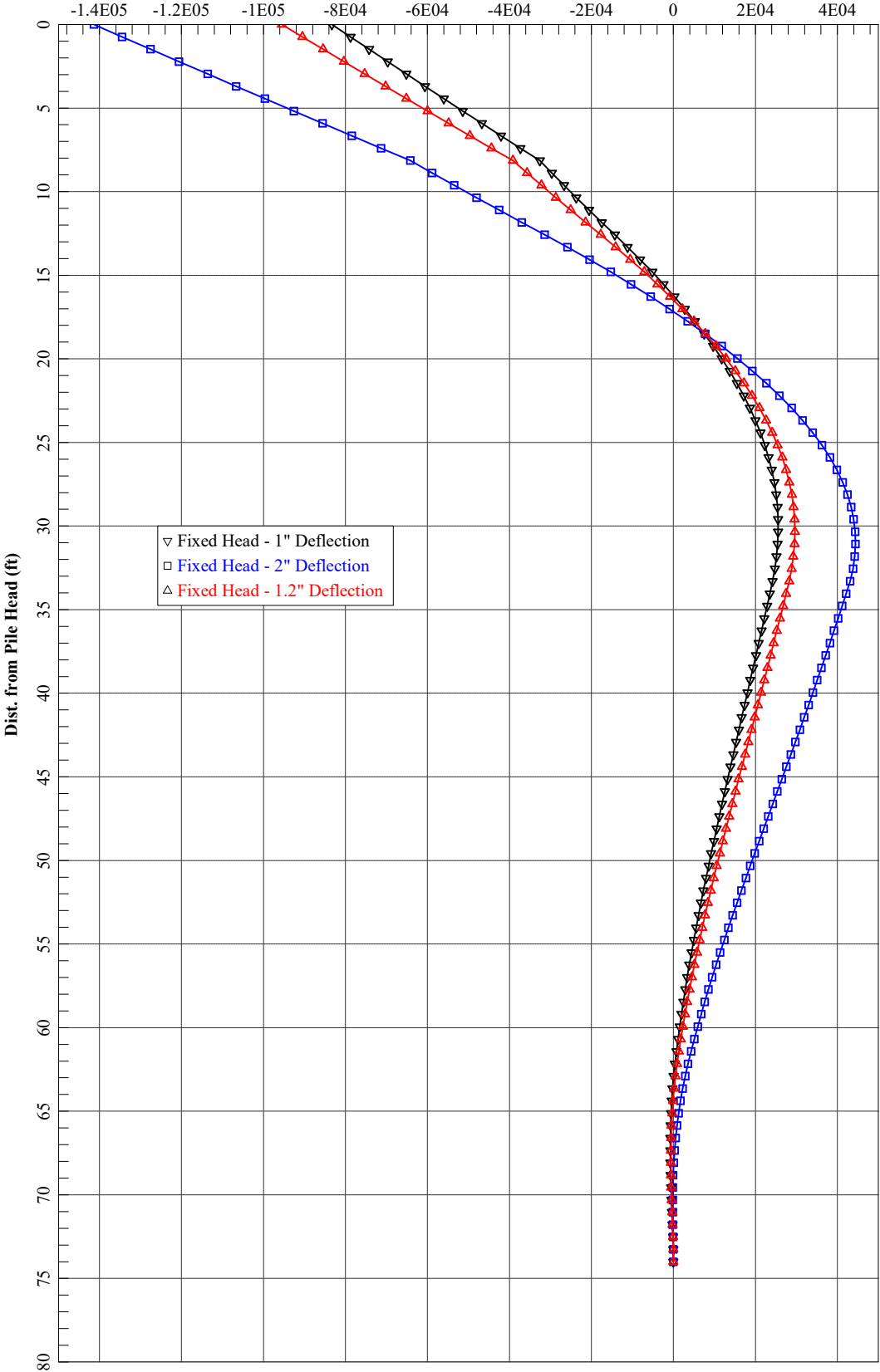
72-inch CISS Pile
Lateral Pile Deflection (inches)



72-inch CISS Pile
Shear Force (kips)



72-inch CISS Pile
Bending Moment (in-kips)



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LPILE for Windows, Version 2022-12.005

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method
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Files Used for Analysis

Path to file locations:

\Projects\200000 - Irvine\212100 - 212149\212109\212109001\Electronic Project File\Data Analysis & Calculations\Pile\Lpile\

Name of input data file:

72in LPILE_with tiebacks.lp12d

Name of output report file:

72in LPILE_with tiebacks.lp12o

Name of plot output file:

72in LPILE_with tiebacks.lp12p

Name of runtime message file:

72in LPILE_with tiebacks.lp12r

Date and Time of Analysis

Date: June 15, 2023

Time: 15:36:50

Problem Title

Project Name: Via Principessa

Job Number: 212109001

Client:

Engineer:

Description:

Program Options and Settings

Computational Options:

- Conventional Analysis

Engineering Units Used for Data Input and Computations:

- US Customary System Units (pounds, feet, inches)

Analysis Control Options:

- | | | |
|--|---|---------------|
| - Maximum number of iterations allowed | = | 500 |
| - Deflection tolerance for convergence | = | 1.0000E-05 in |
| - Maximum allowable deflection | = | 300.0000 in |
| - Number of pile increments | = | 100 |

Loading Type and Number of Cycles of Loading:

- Static loading specified
- Use of p-y modification factors for p-y curves not selected
- Analysis uses layering correction (Method of Georgiadis)
- Analysis includes loading by one distributed lateral load acting on pile
- Loading by lateral soil movements acting on pile not selected
- Input of shear resistance at the pile tip not selected

- Input of moment resistance at the pile tip not selected
- Computation of pile-head foundation stiffness matrix not selected
- Push-over analysis of pile not selected
- Buckling analysis of pile not selected

Output Options:

- Output files use decimal points to denote decimal symbols.
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (nodal spacing of output points) = 1
- No p-y curves to be computed and reported for user-specified depths
- Print using wide report formats

----- Pile Structural Properties and Geometry -----

Number of pile sections defined	=	1
Total length of pile	=	74.000 ft
Depth of ground surface below top of pile	=	0.0000 ft

Pile diameters used for p-y curve computations are defined using 2 points.

p-y curves are computed using pile diameter values interpolated with depth over the length of the pile. A summary of values of pile diameter vs. depth follows.

Point No.	Depth Below Pile Head feet	Pile Diameter inches
1	0.000	72.0000
2	74.000	72.0000

Input Structural Properties for Pile Sections: -----

Pile Section No. 1:

Section 1 is an elastic pile	
Cross-sectional Shape	= Circular Pile
Length of section	= 74.000000 ft
Width of top of section	= 72.000000 in
Width of bottom of section	= 72.000000 in
Top Area	= 4072. sq. in

Bottom Area	=	4072. sq. in
Moment of Inertia at Top	=	659583. in^4
Moment of Inertia at Bottom	=	659583. in^4
Elastic Modulus	=	3600000. psi

Soil and Rock Layering Information

The soil profile is modelled using 9 layers

Layer 1 is modelled using user-specified p-y curves

Distance from top of pile to top of layer	=	0.0000 ft
Distance from top of pile to bottom of layer	=	3.750000 ft
Effective unit weight at top of layer	=	1.000000 pcf
Effective unit weight at bottom of layer	=	1.000000 pcf

User-specified p-y curve at top of layer 1 at depth = 0.0000 ft

Point No.	y inches	p lb/inch
-----	-----	-----
1	0.0000	0.0000
2	0.500000	0.0000
3	2.000000	0.0000

User-specified p-y curve at bottom of layer 1 at depth = 3.7500 ft

Point No.	y inches	p lb/inch
-----	-----	-----
1	0.0000	0.0000
2	0.500000	0.0000
3	2.000000	0.0000

Layer 2 is modelled using user-specified p-y curves

Distance from top of pile to top of layer	=	3.750000 ft
Distance from top of pile to bottom of layer	=	4.250000 ft
Effective unit weight at top of layer	=	1.000000 pcf
Effective unit weight at bottom of layer	=	1.000000 pcf

User-specified p-y curve at top of layer 2 at depth = 3.7500 ft

Point No.	y inches	p lb/inch
-----	-----	-----
1	0.0000	0.0000
2	0.500000	24000.
3	2.000000	24000.

User-specified p-y curve at bottom of layer 2 at depth = 4.2500 ft

Point No.	y inches	p lb/inch
-----	-----	-----
1	0.0000	0.0000
2	0.500000	24000.
3	2.000000	24000.

Layer 3 is modelled using user-specified p-y curves

Distance from top of pile to top of layer	=	4.250000 ft
Distance from top of pile to bottom of layer	=	7.750000 ft
Effective unit weight at top of layer	=	1.000000 pcf
Effective unit weight at bottom of layer	=	1.000000 pcf

User-specified p-y curve at top of layer 3 at depth = 4.2500 ft

Point No.	y inches	p lb/inch
-----	-----	-----
1	0.0000	0.0000
2	0.500000	0.0000
3	2.000000	0.0000

User-specified p-y curve at bottom of layer 3 at depth = 7.7500 ft

Point No.	y inches	p lb/inch
-----	-----	-----
1	0.0000	0.0000
2	0.500000	0.0000
3	2.000000	0.0000

Layer 4 is modelled using user-specified p-y curves

Distance from top of pile to top of layer	=	7.750000 ft
Distance from top of pile to bottom of layer	=	8.250000 ft
Effective unit weight at top of layer	=	1.000000 pcf
Effective unit weight at bottom of layer	=	1.000000 pcf

User-specified p-y curve at top of layer 4 at depth = 7.7500 ft

Point No.	y inches	p lb/inch
1	0.0000	0.0000
2	0.500000	24000.
3	2.000000	24000.

User-specified p-y curve at bottom of layer 4 at depth = 8.2500 ft

Point No.	y inches	p lb/inch
1	0.0000	0.0000
2	0.500000	24000.
3	2.000000	24000.

Layer 5 is modelled using user-specified p-y curves

Distance from top of pile to top of layer	=	8.250000 ft
Distance from top of pile to bottom of layer	=	12.000000 ft
Effective unit weight at top of layer	=	1.000000 pcf
Effective unit weight at bottom of layer	=	1.000000 pcf

User-specified p-y curve at top of layer 5 at depth = 8.2500 ft

Point No.	y inches	p lb/inch
1	0.0000	0.0000
2	0.500000	0.0000
3	2.000000	0.0000

User-specified p-y curve at bottom of layer 5 at depth = 12.0000 ft

Point No.	y inches	p lb/inch
-----	-----	-----
1	0.0000	0.0000
2	0.500000	0.0000
3	2.000000	0.0000

Layer 6 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer	=	12.000000 ft
Distance from top of pile to bottom of layer	=	15.000000 ft
Effective unit weight at top of layer	=	115.000000 pcf
Effective unit weight at bottom of layer	=	115.000000 pcf
Friction angle at top of layer	=	32.000000 deg.
Friction angle at bottom of layer	=	32.000000 deg.
Subgrade k at top of layer	=	160.000000 pci
Subgrade k at bottom of layer	=	160.000000 pci

Layer 7 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer	=	15.000000 ft
Distance from top of pile to bottom of layer	=	34.000000 ft
Effective unit weight at top of layer	=	52.600000 pcf
Effective unit weight at bottom of layer	=	52.600000 pcf
Friction angle at top of layer	=	32.000000 deg.
Friction angle at bottom of layer	=	32.000000 deg.
Subgrade k at top of layer	=	125.000000 pci
Subgrade k at bottom of layer	=	125.000000 pci

Layer 8 is liquefiable sand, by Rollins et al., 2004

Distance from top of pile to top of layer	=	34.000000 ft
Distance from top of pile to bottom of layer	=	62.000000 ft
Effective unit weight at top of layer	=	52.600000 pcf
Effective unit weight at bottom of layer	=	52.600000 pcf

Warning : The depth of this layer is deeper than the recommended depth limit for using the p-y criteria for liquefied sand.

Please consult the LPILE Technical Manual for additional background information regarding limitations on the use of the liquefied sand criteria.

Layer 9 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer	=	62.000000 ft
Distance from top of pile to bottom of layer	=	100.000000 ft
Effective unit weight at top of layer	=	62.600000 pcf
Effective unit weight at bottom of layer	=	62.600000 pcf
Friction angle at top of layer	=	36.000000 deg.
Friction angle at bottom of layer	=	36.000000 deg.
Subgrade k at top of layer	=	125.000000 pci
Subgrade k at bottom of layer	=	125.000000 pci

(Depth of the lowest soil layer extends 26.000 ft below the pile tip)

**** Warning - Possible Input Data Error ****

Values entered for effective unit weights of soil were outside the limits of 20 pcf to 140 pcf.

The minimum input value, in layer 1, for effective unit weight = 1.00 pcf

This data may be erroneous. Please check your data.

Summary of Input Soil Properties

Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Angle of Friction deg.	kpy pci
1	User Input	0.00	1.0000	--	--
	p-y Curves	3.7500	1.0000	--	--
2	User Input	3.7500	1.0000	--	--
	p-y Curves	4.2500	1.0000	--	--
3	User Input	4.2500	1.0000	--	--
	p-y Curves	7.7500	1.0000	--	--
4	User Input	7.7500	1.0000	--	--
	p-y Curves	8.2500	1.0000	--	--
5	User Input	8.2500	1.0000	--	--
	p-y Curves	12.0000	1.0000	--	--
6	Sand	12.0000	115.0000	32.0000	160.0000
	(Reese, et al.)	15.0000	115.0000	32.0000	160.0000
7	Sand	15.0000	52.6000	32.0000	125.0000
	(Reese, et al.)	34.0000	52.6000	32.0000	125.0000
8	Liquefied	34.0000	52.6000	--	--

9	Sand	62.0000	52.6000	--	--
	Sand	62.0000	62.6000	36.0000	125.0000
	(Reese, et al.)	100.0000	62.6000	36.0000	125.0000

Static Loading Type

Static loading criteria were used when computing p-y curves for all analyses.

Distributed Lateral Loading Used For All Load Cases

Distributed lateral load intensity defined using 2 points

Point No.	Depth X ft	Dist. Load lb/in
1	0.000	0.000
2	12.000	820.000

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 3

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length	Run Analysis
1	5	y = 1.000000 in	S = 0.0000 in/in	890000.	N.A.	Yes
2	5	y = 2.000000 in	S = 0.0000 in/in	890000.	N.A.	Yes
3	5	y = 1.200000 in	S = 0.0000 in/in	890000.	N.A.	Yes

V = shear force applied normal to pile axis

M = bending moment applied to pile head

y = lateral deflection normal to pile axis

S = pile slope relative to original pile batter angle

R = rotational stiffness applied to pile head

Values of top y vs. pile lengths can be computed only for load types with specified shear loading (Load Types 1, 2, and 3).

Thrust force is assumed to be acting axially for all pile batter angles.

Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness

Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 1

Pile Section No. 1:

Moment-curvature properties were derived from elastic section properties

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	0.00	0.00	N.A.	No	0.00	0.00
2	3.7500	3.7500	Yes	No	0.00	144000.
3	4.2500	4.2500	Yes	No	144000.	0.00
4	7.7500	7.7500	Yes	No	144000.	144000.
5	8.2500	8.2500	Yes	No	288000.	0.00
6	12.0000	71.4290	No	No	288000.	107588.
7	15.0000	22.9366	Yes	No	395588.	1793545.
8	34.0000	34.0000	No	No	2189133.	227354.
9	62.0000	33.6676	No	No	2416486.	N.A.

Notes: The F0 integral of Layer n+1 equals the sum of the F0 and F1 integrals for Layer n. Layering correction equivalent depths are computed only for soil types with both shallow-depth and deep-depth expressions for peak lateral load transfer. These soil types are soft and stiff clays, non-liquefied sands, and cemented c-phi soil.

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 1

Pile-head conditions are Displacement and Pile-head Rotation (Loading Type 5)
 Displacement of pile head = 1.000000 inches
 Rotation of pile head = 0.000E+00 radians
 Axial load on pile head = 890000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/inch	Soil Spr. Es*H lb/inch	Distrib. Lat. Load lb/inch
0.00	1.0000	-8.33E+07	509207.	0.00	4766.	2.37E+12	0.00	0.00	12.6417
0.7400	0.9986	-7.88E+07	509648.	-3.03E-04	4519.	2.37E+12	0.00	0.00	50.5667
1.4800	0.9946	-7.43E+07	510322.	-5.89E-04	4271.	2.37E+12	0.00	0.00	101.1333
2.2200	0.9882	-6.97E+07	511444.	-8.58E-04	4024.	2.37E+12	0.00	0.00	151.7000
2.9600	0.9794	-6.52E+07	513016.	-0.00111	3775.	2.37E+12	0.00	0.00	202.2667
3.7000	0.9684	-6.06E+07	515036.	-0.00135	3525.	2.37E+12	0.00	0.00	252.8333
4.4400	0.9555	-5.60E+07	517506.	-0.00156	3275.	2.37E+12	0.00	0.00	303.4000
5.1800	0.9407	-5.14E+07	520425.	-0.00176	3022.	2.37E+12	0.00	0.00	353.9667
5.9200	0.9241	-4.67E+07	523793.	-0.00195	2769.	2.37E+12	0.00	0.00	404.5333
6.6600	0.9061	-4.20E+07	527609.	-0.00211	2513.	2.37E+12	0.00	0.00	455.1000
7.4000	0.8866	-3.73E+07	531875.	-0.00226	2255.	2.37E+12	0.00	0.00	505.6667
8.1400	0.8659	-3.26E+07	430030.	-0.00239	1995.	2.37E+12	-24000.	246132.	556.2333
8.8800	0.8441	-2.96E+07	328634.	-0.00251	1836.	2.37E+12	0.00	0.00	606.8000
9.6200	0.8213	-2.67E+07	334247.	-0.00261	1675.	2.37E+12	0.00	0.00	657.3667
10.3600	0.7977	-2.37E+07	340308.	-0.00271	1510.	2.37E+12	0.00	0.00	707.9333
11.1000	0.7732	-2.06E+07	346819.	-0.00279	1343.	2.37E+12	0.00	0.00	758.5000
11.8400	0.7481	-1.75E+07	352737.	-0.00286	1172.	2.37E+12	0.00	0.00	574.3278
12.5800	0.7224	-1.43E+07	352925.	-0.00292	998.1300	2.37E+12	-531.999	6540.	0.00
13.3200	0.6962	-1.11E+07	345756.	-0.00297	826.9507	2.37E+12	-1083.	13810.	0.00
14.0600	0.6696	-8094929.	333815.	-0.00301	660.4132	2.37E+12	-1607.	21307.	0.00
14.8000	0.6428	-5170129.	317343.	-0.00303	500.7778	2.37E+12	-2103.	29054.	0.00
15.5400	0.6158	-2411018.	302163.	-0.00304	350.1857	2.37E+12	-1316.	18974.	0.00
16.2800	0.5887	244406.	290275.	-0.00305	231.9321	2.37E+12	-1362.	20536.	0.00
17.0200	0.5617	2792459.	277821.	-0.00304	371.0047	2.37E+12	-1444.	22823.	0.00
17.7600	0.5347	5226598.	264673.	-0.00303	503.8598	2.37E+12	-1518.	25204.	0.00
18.5000	0.5079	7540914.	250905.	-0.00300	630.1751	2.37E+12	-1583.	27683.	0.00
19.2400	0.4813	9730153.	236591.	-0.00297	749.6636	2.37E+12	-1641.	30266.	0.00
19.9800	0.4551	1.18E+07	221807.	-0.00293	862.0755	2.37E+12	-1689.	32956.	0.00
20.7200	0.4293	1.37E+07	206632.	-0.00288	967.1988	2.37E+12	-1729.	35757.	0.00
21.4600	0.4039	1.55E+07	191148.	-0.00283	1065.	2.37E+12	-1759.	38672.	0.00
22.2000	0.3790	1.72E+07	175413.	-0.00277	1155.	2.37E+12	-1785.	41818.	0.00
22.9400	0.3547	1.87E+07	159385.	-0.00270	1237.	2.37E+12	-1825.	45680.	0.00
23.6800	0.3311	2.00E+07	143034.	-0.00263	1312.	2.37E+12	-1858.	49833.	0.00
24.4200	0.3081	2.12E+07	126420.	-0.00255	1378.	2.37E+12	-1884.	54306.	0.00
25.1600	0.2858	2.23E+07	109607.	-0.00247	1437.	2.37E+12	-1903.	59133.	0.00

25.9000	0.2642	2.32E+07	92658.	-0.00238	1487.	2.37E+12	-1915.	64352.	0.00
26.6400	0.2434	2.40E+07	75638.	-0.00230	1528.	2.37E+12	-1919.	70006.	0.00
27.3800	0.2234	2.46E+07	58612.	-0.00221	1562.	2.37E+12	-1916.	76147.	0.00
28.1200	0.2042	2.51E+07	41649.	-0.00211	1587.	2.37E+12	-1905.	82834.	0.00
28.8600	0.1859	2.54E+07	24813.	-0.00202	1604.	2.37E+12	-1887.	90137.	0.00
29.6000	0.1684	2.55E+07	8174.	-0.00192	1613.	2.37E+12	-1861.	98138.	0.00
30.3400	0.1517	2.56E+07	-8200.	-0.00183	1614.	2.37E+12	-1827.	106939.	0.00
31.0800	0.1359	2.54E+07	-24240.	-0.00173	1607.	2.37E+12	-1786.	116662.	0.00
31.8200	0.1210	2.52E+07	-39877.	-0.00164	1592.	2.37E+12	-1736.	127460.	0.00
32.5600	0.1068	2.47E+07	-55038.	-0.00154	1569.	2.37E+12	-1679.	139527.	0.00
33.3000	0.09352	2.42E+07	-69650.	-0.00145	1540.	2.37E+12	-1613.	153116.	0.00
34.0400	0.08102	2.35E+07	-76950.	-0.00136	1503.	2.37E+12	-31.324	3433.	0.00
34.7800	0.06931	2.29E+07	-77219.	-0.00128	1466.	2.37E+12	-29.330	3758.	0.00
35.5200	0.05835	2.22E+07	-77469.	-0.00119	1429.	2.37E+12	-26.958	4103.	0.00
36.2600	0.04812	2.15E+07	-77696.	-0.00111	1392.	2.37E+12	-24.200	4465.	0.00
37.0000	0.03862	2.08E+07	-77897.	-0.00103	1355.	2.37E+12	-21.052	4841.	0.00
37.7400	0.02980	2.01E+07	-78068.	-9.55E-04	1318.	2.37E+12	-17.520	5221.	0.00
38.4800	0.02165	1.94E+07	-78206.	-8.81E-04	1280.	2.37E+12	-13.620	5586.	0.00
39.2200	0.01415	1.88E+07	-78308.	-8.10E-04	1242.	2.37E+12	-9.392	5895.	0.00
39.9600	0.00727	1.81E+07	-78372.	-7.41E-04	1205.	2.37E+12	-4.930	6024.	0.00
40.7000	9.88E-04	1.74E+07	-78396.	-6.75E-04	1167.	2.37E+12	-0.568	5105.	0.00
41.4400	-0.00471	1.67E+07	-78382.	-6.11E-04	1129.	2.37E+12	3.9011	7348.	0.00
42.1800	-0.00986	1.60E+07	-78319.	-5.50E-04	1092.	2.37E+12	10.2212	9203.	0.00
42.9200	-0.01448	1.53E+07	-78195.	-4.91E-04	1054.	2.37E+12	17.7258	10871.	0.00
43.6600	-0.01859	1.46E+07	-78000.	-4.35E-04	1016.	2.37E+12	26.2420	12537.	0.00
44.4000	-0.02221	1.39E+07	-77725.	-3.82E-04	978.7610	2.37E+12	35.6658	14259.	0.00
45.1400	-0.02537	1.32E+07	-77363.	-3.31E-04	941.3204	2.37E+12	45.9129	16069.	0.00
45.8800	-0.02809	1.26E+07	-76906.	-2.83E-04	904.0560	2.37E+12	56.9038	17987.	0.00
46.6200	-0.03040	1.19E+07	-76349.	-2.37E-04	867.0163	2.37E+12	68.5584	20028.	0.00
47.3600	-0.03231	1.12E+07	-75686.	-1.94E-04	830.2525	2.37E+12	80.7929	22207.	0.00
48.1000	-0.03384	1.05E+07	-74912.	-1.53E-04	793.8184	2.37E+12	93.5186	24538.	0.00
48.8400	-0.03503	9878671.	-74023.	-1.15E-04	757.7697	2.37E+12	106.6413	27032.	0.00
49.5800	-0.03589	9226314.	-73017.	-7.95E-05	722.1641	2.37E+12	120.0608	29706.	0.00
50.3200	-0.03644	8583151.	-71890.	-4.62E-05	687.0603	2.37E+12	133.6708	32571.	0.00
51.0600	-0.03671	7950275.	-70642.	-1.53E-05	652.5180	2.37E+12	147.3592	35645.	0.00
51.8000	-0.03671	7328784.	-69273.	1.33E-05	618.5970	2.37E+12	161.0084	38942.	0.00
52.5400	-0.03647	6719773.	-67784.	3.95E-05	585.3572	2.37E+12	174.4950	42482.	0.00
53.2800	-0.03601	6124323.	-66175.	6.36E-05	552.8576	2.37E+12	187.6908	46281.	0.00
54.0200	-0.03535	5543492.	-64452.	8.54E-05	521.1559	2.37E+12	200.4631	50362.	0.00
54.7600	-0.03450	4978304.	-62618.	1.05E-04	490.3080	2.37E+12	212.6748	54747.	0.00
55.5000	-0.03348	4429740.	-60678.	1.23E-04	460.3674	2.37E+12	224.1850	59461.	0.00
56.2400	-0.03232	3898723.	-58640.	1.38E-04	431.3845	2.37E+12	234.8497	64530.	0.00
56.9800	-0.03103	3386109.	-56512.	1.52E-04	403.4061	2.37E+12	244.5218	69986.	0.00
57.7200	-0.02962	2892677.	-54302.	1.64E-04	376.4746	2.37E+12	253.0521	75862.	0.00
58.4600	-0.02812	2419114.	-52023.	1.74E-04	350.6275	2.37E+12	260.2893	82196.	0.00
59.2000	-0.02654	1966004.	-49686.	1.82E-04	325.8968	2.37E+12	266.0803	89030.	0.00
59.9400	-0.02489	1533818.	-47305.	1.88E-04	302.3081	2.37E+12	270.2711	96413.	0.00
60.6800	-0.02320	1122898.	-44894.	1.93E-04	279.8802	2.37E+12	272.7064	104400.	0.00

61.4200	-0.02146	733450.	-42470.	1.97E-04	258.6241	2.37E+12	273.2301	113055.	0.00
62.1600	-0.01970	365525.	-38172.	1.99E-04	238.5428	2.37E+12	694.8145	313159.	0.00
62.9000	-0.01793	52378.	-32042.	2.00E-04	221.4512	2.37E+12	685.7319	339590.	0.00
63.6400	-0.01616	-206697.	-26013.	1.99E-04	229.8739	2.37E+12	672.0972	369354.	0.00
64.3800	-0.01439	-412768.	-20128.	1.98E-04	241.1213	2.37E+12	653.5257	403212.	0.00
65.1200	-0.01264	-567294.	-14431.	1.96E-04	249.5553	2.37E+12	629.5254	442244.	0.00
65.8600	-0.01091	-672162.	-8974.	1.94E-04	255.2790	2.37E+12	599.4455	488034.	0.00
66.6000	-0.00920	-729741.	-3816.	1.91E-04	258.4216	2.37E+12	562.3916	543056.	0.00
67.3400	-0.00751	-742951.	977.2067	1.89E-04	259.1427	2.37E+12	517.0780	611455.	0.00
68.0800	-0.00585	-715366.	5322.	1.86E-04	257.6371	2.37E+12	461.5383	700917.	0.00
68.8200	-0.00421	-651365.	9114.	1.83E-04	254.1439	2.37E+12	392.4704	828033.	0.00
69.5600	-0.00259	-556397.	12058.	1.81E-04	248.9606	2.37E+12	270.4722	926539.	0.00
70.3000	-9.94E-04	-440085.	13724.	1.79E-04	242.6123	2.37E+12	104.8152	936396.	0.00
71.0400	5.90E-04	-315495.	13910.	1.78E-04	235.8121	2.37E+12	-62.833	946253.	0.00
71.7800	0.00216	-195849.	12597.	1.77E-04	229.2819	2.37E+12	-232.868	956110.	0.00
72.5200	0.00373	-94561.	9762.	1.76E-04	223.7536	2.37E+12	-405.688	965966.	0.00
73.2600	0.00529	-25260.	5481.	1.76E-04	219.9711	2.37E+12	-558.516	937026.	0.00
74.0000	0.00686	0.00	0.00	1.76E-04	218.5924	2.37E+12	-675.952	437777.	0.00

* The above values of total stress are combined axial and bending stresses.

Output Summary for Load Case No. 1:

Pile-head deflection = 1.00000000 inches
 Computed slope at pile head = 0.000000 radians
 Maximum bending moment = -83312327. inch-lbs
 Maximum shear force = 531875. lbs
 Depth of maximum bending moment = 0.000000 feet below pile head
 Depth of maximum shear force = 7.40000000 feet below pile head
 Number of iterations = 10
 Number of zero deflection points = 2

----- Computed Values of Pile Loading and Deflection for Lateral Loading for Load Case Number 2 -----

Pile-head conditions are Displacement and Pile-head Rotation (Loading Type 5)
 Displacement of pile head = 2.000000 inches
 Rotation of pile head = 0.000E+00 radians
 Axial load on pile head = 890000.0 lbs

Depth	Deflect.	Bending	Shear	Slope	Total	Bending	Soil Res.	Soil Spr.	Distrib.
X	y	Moment	Force	S	Stress	Stiffness	p	Es*H	Lat. Load
feet	inches	in-lbs	lbs	radians	psi*	lb-in^2	lb/inch	lb/inch	lb/inch

0.00	2.0000	-1.41E+08	779299.	0.00	7936.	2.37E+12	0.00	0.00	12.6417
0.7400	1.9977	-1.34E+08	779736.	-5.16E-04	7558.	2.37E+12	0.00	0.00	50.5667
1.4800	1.9908	-1.28E+08	780410.	-0.00101	7179.	2.37E+12	0.00	0.00	101.1333
2.2200	1.9798	-1.21E+08	781532.	-0.00147	6800.	2.37E+12	0.00	0.00	151.7000
2.9600	1.9647	-1.14E+08	783104.	-0.00191	6420.	2.37E+12	0.00	0.00	202.2667
3.7000	1.9459	-1.07E+08	785125.	-0.00232	6040.	2.37E+12	0.00	0.00	252.8333
4.4400	1.9235	-9.96E+07	787594.	-0.00271	5657.	2.37E+12	0.00	0.00	303.4000
5.1800	1.8979	-9.26E+07	790513.	-0.00306	5274.	2.37E+12	0.00	0.00	353.9667
5.9200	1.8691	-8.56E+07	793881.	-0.00340	4888.	2.37E+12	0.00	0.00	404.5333
6.6600	1.8375	-7.85E+07	797698.	-0.00370	4501.	2.37E+12	0.00	0.00	455.1000
7.4000	1.8033	-7.13E+07	801963.	-0.00398	4112.	2.37E+12	0.00	0.00	505.6667
8.1400	1.7667	-6.42E+07	700118.	-0.00424	3720.	2.37E+12	-24000.	120628.	556.2333
8.8800	1.7280	-5.88E+07	598722.	-0.00447	3430.	2.37E+12	0.00	0.00	606.8000
9.6200	1.6874	-5.35E+07	604335.	-0.00468	3136.	2.37E+12	0.00	0.00	657.3667
10.3600	1.6450	-4.80E+07	610397.	-0.00487	2840.	2.37E+12	0.00	0.00	707.9333
11.1000	1.6009	-4.25E+07	616908.	-0.00504	2540.	2.37E+12	0.00	0.00	758.5000
11.8400	1.5555	-3.70E+07	622826.	-0.00519	2238.	2.37E+12	0.00	0.00	574.3278
12.5800	1.5088	-3.14E+07	621656.	-0.00531	1932.	2.37E+12	-837.656	4930.	0.00
13.3200	1.4611	-2.59E+07	610358.	-0.00542	1630.	2.37E+12	-1707.	10374.	0.00
14.0600	1.4126	-2.05E+07	591513.	-0.00551	1336.	2.37E+12	-2537.	15952.	0.00
14.8000	1.3633	-1.53E+07	565469.	-0.00557	1052.	2.37E+12	-3328.	21679.	0.00
15.5400	1.3136	-1.03E+07	541888.	-0.00562	782.9306	2.37E+12	-1983.	13403.	0.00
16.2800	1.2635	-5561286.	523809.	-0.00565	522.1271	2.37E+12	-2089.	14683.	0.00
17.0200	1.2132	-947477.	504622.	-0.00566	270.3057	2.37E+12	-2232.	16339.	0.00
17.7600	1.1629	3490339.	484203.	-0.00566	409.0949	2.37E+12	-2367.	18072.	0.00
18.5000	1.1127	7741431.	462635.	-0.00564	641.1193	2.37E+12	-2491.	19882.	0.00
19.2400	1.0627	1.18E+07	440006.	-0.00560	862.4093	2.37E+12	-2605.	21770.	0.00
19.9800	1.0132	1.56E+07	416411.	-0.00555	1072.	2.37E+12	-2709.	23740.	0.00
20.7200	0.9642	1.93E+07	391949.	-0.00549	1271.	2.37E+12	-2801.	25795.	0.00
21.4600	0.9158	2.27E+07	366722.	-0.00541	1457.	2.37E+12	-2881.	27936.	0.00
22.2000	0.8681	2.59E+07	340813.	-0.00532	1631.	2.37E+12	-2954.	30219.	0.00
22.9400	0.8214	2.88E+07	314198.	-0.00521	1792.	2.37E+12	-3040.	32870.	0.00
23.6800	0.7755	3.15E+07	286857.	-0.00510	1940.	2.37E+12	-3117.	35694.	0.00
24.4200	0.7308	3.40E+07	258873.	-0.00498	2075.	2.37E+12	-3185.	38707.	0.00
25.1600	0.6871	3.62E+07	230327.	-0.00485	2195.	2.37E+12	-3244.	41923.	0.00
25.9000	0.6447	3.82E+07	201302.	-0.00471	2302.	2.37E+12	-3293.	45361.	0.00
26.6400	0.6035	3.99E+07	171882.	-0.00456	2394.	2.37E+12	-3333.	49039.	0.00
27.3800	0.5637	4.13E+07	142153.	-0.00441	2473.	2.37E+12	-3363.	52977.	0.00
28.1200	0.5252	4.25E+07	112202.	-0.00425	2536.	2.37E+12	-3383.	57201.	0.00
28.8600	0.4881	4.34E+07	82114.	-0.00409	2585.	2.37E+12	-3394.	61735.	0.00
29.6000	0.4525	4.40E+07	51976.	-0.00393	2619.	2.37E+12	-3394.	66609.	0.00
30.3400	0.4183	4.43E+07	21876.	-0.00376	2639.	2.37E+12	-3385.	71856.	0.00
31.0800	0.3856	4.44E+07	-8100.	-0.00360	2644.	2.37E+12	-3366.	77514.	0.00
31.8200	0.3544	4.43E+07	-37866.	-0.00343	2634.	2.37E+12	-3338.	83626.	0.00
32.5600	0.3247	4.38E+07	-67336.	-0.00327	2610.	2.37E+12	-3299.	90242.	0.00
33.3000	0.2964	4.31E+07	-96423.	-0.00311	2572.	2.37E+12	-3252.	97420.	0.00
34.0400	0.2695	4.21E+07	-111482.	-0.00295	2519.	2.37E+12	-140.261	4621.	0.00

34.7800	0.2441	4.12E+07	-112721.	-0.00279	2466.	2.37E+12	-138.808	5050.	0.00
35.5200	0.2200	4.02E+07	-113944.	-0.00264	2412.	2.37E+12	-136.584	5514.	0.00
36.2600	0.1972	3.92E+07	-115143.	-0.00249	2358.	2.37E+12	-133.556	6014.	0.00
37.0000	0.1758	3.82E+07	-116312.	-0.00235	2303.	2.37E+12	-129.691	6553.	0.00
37.7400	0.1556	3.72E+07	-117443.	-0.00220	2247.	2.37E+12	-124.959	7133.	0.00
38.4800	0.1366	3.61E+07	-118528.	-0.00207	2191.	2.37E+12	-119.336	7757.	0.00
39.2200	0.1188	3.51E+07	-119558.	-0.00193	2134.	2.37E+12	-112.800	8428.	0.00
39.9600	0.1023	3.40E+07	-120527.	-0.00180	2077.	2.37E+12	-105.336	9147.	0.00
40.7000	0.08680	3.30E+07	-121425.	-0.00168	2019.	2.37E+12	-96.933	9917.	0.00
41.4400	0.07244	3.19E+07	-122244.	-0.00156	1960.	2.37E+12	-87.587	10737.	0.00
42.1800	0.05913	3.08E+07	-122976.	-0.00144	1902.	2.37E+12	-77.303	11609.	0.00
42.9200	0.04685	2.98E+07	-123613.	-0.00133	1842.	2.37E+12	-66.093	12527.	0.00
43.6600	0.03556	2.87E+07	-124146.	-0.00122	1783.	2.37E+12	-53.986	13482.	0.00
44.4000	0.02522	2.76E+07	-124568.	-0.00111	1723.	2.37E+12	-41.030	14448.	0.00
45.1400	0.01579	2.65E+07	-124871.	-0.00101	1663.	2.37E+12	-27.314	15358.	0.00
45.8800	0.00725	2.54E+07	-125051.	-9.15E-04	1603.	2.37E+12	-13.039	15978.	0.00
46.6200	-4.57E-04	2.43E+07	-125105.	-8.22E-04	1543.	2.37E+12	0.7389	14347.	0.00
47.3600	-0.00736	2.32E+07	-125028.	-7.34E-04	1482.	2.37E+12	16.5379	19965.	0.00
48.1000	-0.01349	2.21E+07	-124799.	-6.49E-04	1422.	2.37E+12	35.1160	23124.	0.00
48.8400	-0.01888	2.09E+07	-124397.	-5.69E-04	1362.	2.37E+12	55.4844	26094.	0.00
49.5800	-0.02358	1.99E+07	-123807.	-4.92E-04	1302.	2.37E+12	77.2499	29087.	0.00
50.3200	-0.02763	1.88E+07	-123020.	-4.20E-04	1242.	2.37E+12	100.1254	32184.	0.00
51.0600	-0.03104	1.77E+07	-122025.	-3.52E-04	1183.	2.37E+12	123.8588	35428.	0.00
51.8000	-0.03388	1.66E+07	-120817.	-2.88E-04	1124.	2.37E+12	148.2110	38849.	0.00
52.5400	-0.03616	1.55E+07	-119392.	-2.28E-04	1066.	2.37E+12	172.9460	42473.	0.00
53.2800	-0.03792	1.45E+07	-117745.	-1.72E-04	1009.	2.37E+12	197.8270	46322.	0.00
54.0200	-0.03921	1.34E+07	-115879.	-1.20E-04	952.3009	2.37E+12	222.6144	50418.	0.00
54.7600	-0.04005	1.24E+07	-113793.	-7.12E-05	896.6578	2.37E+12	247.0647	54784.	0.00
55.5000	-0.04047	1.14E+07	-111493.	-2.66E-05	842.0580	2.37E+12	270.9306	59445.	0.00
56.2400	-0.04052	1.04E+07	-108985.	1.43E-05	788.6059	2.37E+12	293.9614	64424.	0.00
56.9800	-0.04022	9487178.	-106277.	5.16E-05	736.4021	2.37E+12	315.9029	69750.	0.00
57.7200	-0.03960	8555343.	-103381.	8.53E-05	685.5425	2.37E+12	336.4988	75453.	0.00
58.4600	-0.03870	7649789.	-100308.	1.16E-04	636.1174	2.37E+12	355.4912	81564.	0.00
59.2000	-0.03755	6772041.	-97075.	1.43E-04	588.2100	2.37E+12	372.6212	88122.	0.00
59.9400	-0.03617	5923475.	-93700.	1.66E-04	541.8954	2.37E+12	387.6302	95165.	0.00
60.6800	-0.03459	5105301.	-90202.	1.87E-04	497.2395	2.37E+12	400.2606	102741.	0.00
61.4200	-0.03285	4318538.	-86603.	2.05E-04	454.2980	2.37E+12	410.2566	110900.	0.00
62.1600	-0.03096	3563999.	-80721.	2.19E-04	413.1153	2.37E+12	914.5909	262310.	0.00
62.9000	-0.02896	2881473.	-72585.	2.31E-04	375.8631	2.37E+12	917.6762	281435.	0.00
63.6400	-0.02685	2271225.	-64447.	2.41E-04	342.5558	2.37E+12	915.2728	302674.	0.00
64.3800	-0.02467	1733083.	-56356.	2.48E-04	313.1840	2.37E+12	906.9983	326410.	0.00
65.1200	-0.02244	1266411.	-48367.	2.54E-04	287.7131	2.37E+12	892.4004	353149.	0.00
65.8600	-0.02016	870071.	-40538.	2.58E-04	266.0809	2.37E+12	870.9270	383581.	0.00
66.6000	-0.01786	542382.	-32933.	2.61E-04	248.1956	2.37E+12	841.8822	418680.	0.00
67.3400	-0.01553	281063.	-25624.	2.62E-04	233.9328	2.37E+12	804.3588	459883.	0.00
68.0800	-0.01320	83162.	-18691.	2.63E-04	223.1314	2.37E+12	757.1286	509420.	0.00
68.8200	-0.01086	-55038.	-12228.	2.63E-04	221.5964	2.37E+12	698.4487	571026.	0.00
69.5600	-0.00853	-138160.	-6349.	2.63E-04	226.1332	2.37E+12	625.6852	651589.	0.00

70.3000	-0.00620	-171940.	-1198.	2.62E-04	227.9769	2.37E+12	534.4628	765861.	0.00
71.0400	-0.00387	-163571.	3008.	2.61E-04	227.5201	2.37E+12	412.6760	946253.	0.00
71.7800	-0.00155	-122655.	5583.	2.61E-04	225.2869	2.37E+12	167.3048	956110.	0.00
72.5200	7.61E-04	-68543.	5958.	2.61E-04	222.3335	2.37E+12	-82.771	965966.	0.00
73.2600	0.00307	-20956.	4091.	2.60E-04	219.7362	2.37E+12	-337.736	975823.	0.00
74.0000	0.00539	0.00	0.00	2.60E-04	218.5924	2.37E+12	-583.692	481244.	0.00

* The above values of total stress are combined axial and bending stresses.

Output Summary for Load Case No. 2:

Pile-head deflection	=	2.00000000 inches
Computed slope at pile head	=	0.000000 radians
Maximum bending moment	=	-141388534. inch-lbs
Maximum shear force	=	801963. lbs
Depth of maximum bending moment	=	0.000000 feet below pile head
Depth of maximum shear force	=	7.40000000 feet below pile head
Number of iterations	=	10
Number of zero deflection points	=	2

----- Computed Values of Pile Loading and Deflection for Lateral Loading for Load Case Number 3 -----

Pile-head conditions are Displacement and Pile-head Rotation (Loading Type 5)

Displacement of pile head	=	1.200000 inches
Rotation of pile head	=	0.000E+00 radians
Axial load on pile head	=	890000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness lb-in^2	Soil Res. p lb/inch	Soil Spr. Es*H lb/inch	Distrib. Lat. Load lb/inch
0.00	1.2000	-9.55E+07	567003.	0.00	5432.	2.37E+12	0.00	0.00	12.6417
0.7400	1.1984	-9.05E+07	567444.	-3.48E-04	5157.	2.37E+12	0.00	0.00	50.5667
1.4800	1.1938	-8.54E+07	568117.	-6.77E-04	4882.	2.37E+12	0.00	0.00	101.1333
2.2200	1.1864	-8.04E+07	569240.	-9.87E-04	4606.	2.37E+12	0.00	0.00	151.7000
2.9600	1.1763	-7.53E+07	570812.	-0.00128	4329.	2.37E+12	0.00	0.00	202.2667
3.7000	1.1637	-7.02E+07	572832.	-0.00155	4052.	2.37E+12	0.00	0.00	252.8333
4.4400	1.1488	-6.51E+07	575302.	-0.00180	3773.	2.37E+12	0.00	0.00	303.4000
5.1800	1.1317	-6.00E+07	578221.	-0.00204	3492.	2.37E+12	0.00	0.00	353.9667
5.9200	1.1126	-5.48E+07	581588.	-0.00225	3210.	2.37E+12	0.00	0.00	404.5333
6.6600	1.0917	-4.96E+07	585405.	-0.00245	2927.	2.37E+12	0.00	0.00	455.1000
7.4000	1.0691	-4.44E+07	589671.	-0.00262	2641.	2.37E+12	0.00	0.00	505.6667

8.1400	1.0451	-3.91E+07	487826.	-0.00278	2353.	2.37E+12	-24000.	203923.	556.2333
8.8800	1.0198	-3.57E+07	386430.	-0.00292	2166.	2.37E+12	0.00	0.00	606.8000
9.6200	0.9933	-3.22E+07	392042.	-0.00305	1976.	2.37E+12	0.00	0.00	657.3667
10.3600	0.9657	-2.87E+07	398104.	-0.00316	1783.	2.37E+12	0.00	0.00	707.9333
11.1000	0.9372	-2.51E+07	404615.	-0.00326	1587.	2.37E+12	0.00	0.00	758.5000
11.8400	0.9078	-2.14E+07	410533.	-0.00335	1388.	2.37E+12	0.00	0.00	574.3278
12.5800	0.8777	-1.77E+07	410424.	-0.00342	1186.	2.37E+12	-598.887	6059.	0.00
13.3200	0.8471	-1.41E+07	402349.	-0.00348	987.1063	2.37E+12	-1220.	12788.	0.00
14.0600	0.8159	-1.05E+07	388889.	-0.00353	793.2126	2.37E+12	-1812.	19719.	0.00
14.8000	0.7844	-7118133.	370304.	-0.00356	607.0997	2.37E+12	-2374.	26872.	0.00
15.5400	0.7527	-3895196.	353253.	-0.00358	431.1920	2.37E+12	-1466.	17300.	0.00
16.2800	0.7209	-787780.	339971.	-0.00359	261.5894	2.37E+12	-1525.	18787.	0.00
17.0200	0.6890	2199401.	326004.	-0.00359	338.6356	2.37E+12	-1621.	20886.	0.00
17.7600	0.6572	5058729.	311227.	-0.00357	494.6976	2.37E+12	-1708.	23073.	0.00
18.5000	0.6256	7783255.	295716.	-0.00355	643.4020	2.37E+12	-1786.	25352.	0.00
19.2400	0.5942	1.04E+07	279548.	-0.00351	784.4072	2.37E+12	-1855.	27727.	0.00
19.9800	0.5632	1.28E+07	262807.	-0.00347	917.4107	2.37E+12	-1915.	30201.	0.00
20.7200	0.5326	1.51E+07	245575.	-0.00342	1042.	2.37E+12	-1966.	32777.	0.00
21.4600	0.5025	1.72E+07	227938.	-0.00336	1158.	2.37E+12	-2006.	35459.	0.00
22.2000	0.4729	1.92E+07	209963.	-0.00329	1266.	2.37E+12	-2042.	38343.	0.00
22.9400	0.4440	2.10E+07	191609.	-0.00321	1365.	2.37E+12	-2092.	41832.	0.00
23.6800	0.4158	2.26E+07	172847.	-0.00313	1455.	2.37E+12	-2134.	45573.	0.00
24.4200	0.3884	2.41E+07	153741.	-0.00305	1535.	2.37E+12	-2169.	49590.	0.00
25.1600	0.3617	2.54E+07	134360.	-0.00295	1606.	2.37E+12	-2196.	53912.	0.00
25.9000	0.3359	2.66E+07	114771.	-0.00286	1668.	2.37E+12	-2216.	58567.	0.00
26.6400	0.3110	2.75E+07	95044.	-0.00275	1720.	2.37E+12	-2227.	63591.	0.00
27.3800	0.2870	2.83E+07	75250.	-0.00265	1762.	2.37E+12	-2231.	69023.	0.00
28.1200	0.2640	2.89E+07	55458.	-0.00254	1795.	2.37E+12	-2227.	74908.	0.00
28.8600	0.2419	2.93E+07	35740.	-0.00243	1818.	2.37E+12	-2214.	81299.	0.00
29.6000	0.2207	2.96E+07	16169.	-0.00232	1832.	2.37E+12	-2194.	88258.	0.00
30.3400	0.2006	2.96E+07	-3184.	-0.00221	1836.	2.37E+12	-2165.	95855.	0.00
31.0800	0.1814	2.95E+07	-22245.	-0.00210	1831.	2.37E+12	-2128.	104179.	0.00
31.8200	0.1632	2.93E+07	-40943.	-0.00199	1816.	2.37E+12	-2083.	113333.	0.00
32.5600	0.1460	2.88E+07	-59204.	-0.00188	1793.	2.37E+12	-2030.	123445.	0.00
33.3000	0.1297	2.82E+07	-76952.	-0.00178	1760.	2.37E+12	-1968.	134675.	0.00
34.0400	0.1144	2.75E+07	-85903.	-0.00167	1720.	2.37E+12	-48.180	3739.	0.00
34.7800	0.1000	2.68E+07	-86321.	-0.00157	1679.	2.37E+12	-46.137	4096.	0.00
35.5200	0.08651	2.60E+07	-86720.	-0.00147	1637.	2.37E+12	-43.632	4479.	0.00
36.2600	0.07385	2.52E+07	-87094.	-0.00138	1596.	2.37E+12	-40.651	4888.	0.00
37.0000	0.06204	2.45E+07	-87440.	-0.00128	1554.	2.37E+12	-37.181	5322.	0.00
37.7400	0.05103	2.37E+07	-87752.	-0.00119	1512.	2.37E+12	-33.214	5779.	0.00
38.4800	0.04082	2.29E+07	-88028.	-0.00111	1470.	2.37E+12	-28.751	6255.	0.00
39.2200	0.03136	2.22E+07	-88261.	-0.00102	1428.	2.37E+12	-23.800	6738.	0.00
39.9600	0.02265	2.14E+07	-88448.	-9.42E-04	1385.	2.37E+12	-18.384	7209.	0.00
40.7000	0.01464	2.06E+07	-88586.	-8.63E-04	1343.	2.37E+12	-12.554	7616.	0.00
41.4400	0.00731	1.98E+07	-88670.	-7.88E-04	1300.	2.37E+12	-6.431	7809.	0.00
42.1800	6.47E-04	1.90E+07	-88700.	-7.15E-04	1258.	2.37E+12	-0.471	6460.	0.00
42.9200	-0.00539	1.83E+07	-88677.	-6.45E-04	1215.	2.37E+12	5.8531	9648.	0.00

43.6600	-0.01082	1.75E+07	-88587.	-5.79E-04	1172.	2.37E+12	14.3694	11798.	0.00
44.4000	-0.01566	1.67E+07	-88415.	-5.15E-04	1130.	2.37E+12	24.2566	13752.	0.00
45.1400	-0.01996	1.59E+07	-88151.	-4.54E-04	1087.	2.37E+12	35.2950	15705.	0.00
45.8800	-0.02372	1.51E+07	-87784.	-3.96E-04	1044.	2.37E+12	47.3441	17723.	0.00
46.6200	-0.02698	1.44E+07	-87306.	-3.41E-04	1002.	2.37E+12	60.2869	19839.	0.00
47.3600	-0.02977	1.36E+07	-86710.	-2.88E-04	960.1364	2.37E+12	74.0131	22077.	0.00
48.1000	-0.03210	1.28E+07	-85989.	-2.39E-04	918.3835	2.37E+12	88.4120	24454.	0.00
48.8400	-0.03401	1.21E+07	-85137.	-1.92E-04	876.9904	2.37E+12	103.3696	26987.	0.00
49.5800	-0.03552	1.13E+07	-84151.	-1.49E-04	836.0228	2.37E+12	118.7671	29690.	0.00
50.3200	-0.03665	1.06E+07	-83026.	-1.08E-04	795.5480	2.37E+12	134.4798	32579.	0.00
51.0600	-0.03744	9839559.	-81762.	-6.96E-05	755.6350	2.37E+12	150.3771	35670.	0.00
51.8000	-0.03789	9119850.	-80355.	-3.42E-05	716.3533	2.37E+12	166.3224	38979.	0.00
52.5400	-0.03804	8412986.	-78808.	-1.38E-06	677.7727	2.37E+12	182.1731	42523.	0.00
53.2800	-0.03792	7720239.	-77121.	2.88E-05	639.9626	2.37E+12	197.7813	46322.	0.00
54.0200	-0.03753	7042860.	-75297.	5.64E-05	602.9913	2.37E+12	212.9936	50395.	0.00
54.7600	-0.03691	6382068.	-73341.	8.15E-05	566.9253	2.37E+12	227.6522	54764.	0.00
55.5000	-0.03608	5739038.	-71257.	1.04E-04	531.8288	2.37E+12	241.5949	59454.	0.00
56.2400	-0.03506	5114890.	-69054.	1.24E-04	497.7628	2.37E+12	254.6559	64492.	0.00
56.9800	-0.03387	4510672.	-66739.	1.42E-04	464.7846	2.37E+12	266.6659	69906.	0.00
57.7200	-0.03253	3927348.	-64323.	1.58E-04	432.9468	2.37E+12	277.4534	75730.	0.00
58.4600	-0.03106	3365786.	-61818.	1.72E-04	402.2968	2.37E+12	286.8443	81999.	0.00
59.2000	-0.02948	2826743.	-59236.	1.83E-04	372.8759	2.37E+12	294.6630	88754.	0.00
59.9400	-0.02781	2310853.	-56593.	1.93E-04	344.7187	2.37E+12	300.7327	96042.	0.00
60.6800	-0.02605	1818608.	-53904.	2.01E-04	317.8520	2.37E+12	304.8754	103915.	0.00
61.4200	-0.02424	1350351.	-51187.	2.07E-04	292.2945	2.37E+12	306.9126	112434.	0.00
62.1600	-0.02238	906255.	-46491.	2.11E-04	268.0558	2.37E+12	750.8282	297891.	0.00
62.9000	-0.02049	521338.	-39855.	2.14E-04	247.0470	2.37E+12	743.7491	322267.	0.00
63.6400	-0.01859	195054.	-33303.	2.15E-04	229.2385	2.37E+12	731.8548	349618.	0.00
64.3800	-0.01668	-73526.	-26880.	2.15E-04	222.6055	2.37E+12	714.7534	380592.	0.00
65.1200	-0.01477	-285743.	-20635.	2.14E-04	234.1883	2.37E+12	691.9531	416091.	0.00
65.8600	-0.01287	-443387.	-14619.	2.13E-04	242.7925	2.37E+12	662.8152	457418.	0.00
66.6000	-0.01098	-548753.	-8895.	2.11E-04	248.5433	2.37E+12	626.4802	506557.	0.00
67.3400	-0.00912	-604701.	-3530.	2.09E-04	251.5970	2.37E+12	581.7422	566724.	0.00
68.0800	-0.00727	-614759.	1392.	2.07E-04	252.1459	2.37E+12	526.8141	643618.	0.00
68.8200	-0.00544	-583257.	5768.	2.05E-04	250.4266	2.37E+12	458.8311	748697.	0.00
69.5600	-0.00363	-515557.	9459.	2.03E-04	246.7315	2.37E+12	372.5742	910183.	0.00
70.3000	-0.00184	-418462.	11977.	2.01E-04	241.4321	2.37E+12	194.5536	936396.	0.00
71.0400	-6.89E-05	-306014.	12874.	1.99E-04	235.2947	2.37E+12	7.3448	946253.	0.00
71.7800	0.00170	-192977.	12095.	1.99E-04	229.1251	2.37E+12	-182.713	956110.	0.00
72.5200	0.00346	-94343.	9614.	1.98E-04	223.7417	2.37E+12	-375.993	965966.	0.00
73.2600	0.00521	-25354.	5488.	1.98E-04	219.9763	2.37E+12	-553.363	942652.	0.00
74.0000	0.00697	0.00	0.00	1.98E-04	218.5924	2.37E+12	-682.692	434990.	0.00

* The above values of total stress are combined axial and bending stresses.

Output Summary for Load Case No. 3:

Pile-head deflection = 1.20000000 inches
 Computed slope at pile head = 0.000000 radians
 Maximum bending moment = -95523202. inch-lbs
 Maximum shear force = 589671. lbs
 Depth of maximum bending moment = 0.000000 feet below pile head
 Depth of maximum shear force = 7.40000000 feet below pile head
 Number of iterations = 10
 Number of zero deflection points = 2

 Summary of Pile-head Responses for Conventional Analyses

Definitions of Pile-head Loading Conditions:

Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
 Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
 Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
 Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
 Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

Load Case No.	Load Type 1	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	y, in	1.0000	S, rad	0.00	890000.	1.0000	0.00	531875.	-8.33E+07
2	y, in	2.0000	S, rad	0.00	890000.	2.0000	0.00	801963.	-1.41E+08
3	y, in	1.2000	S, rad	0.00	890000.	1.2000	0.00	589671.	-9.55E+07

Maximum pile-head deflection = 2.000000000 inches
 Maximum pile-head rotation = 0.000000000 radians = 0.000000 deg.

No error or warning messages were generated by this analysis.

The analysis ended normally.



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