Geotechnical Engineering Report

SJCOE CODE STACK ACADEMY

201 N. California Street
Stockton, California
MPE No. 06357--01

MPE
MID PACIFIC ENGINEERING, INC.

April 18, 2024

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Geotechnical Engineering Report

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INTRODUCTION

As authorized, Mid Pacific Engineering, Inc. has prepared a *Geotechnical Engineering Report* for the proposed San Joaquin County Office of Education (SJCCOE) Code Stack Academy renovation project to be located at 201 N. California Street in Stockton, California.

SCOPE OF SERVICES

Our scope of services for this project included the following tasks:

1. site reconnaissance;
2. review of available historical aerial photographs, geologic maps, topographic maps, and groundwater information;
3. subsurface exploration, including the drilling and sampling of 6 soil borings to maximum depths of approximately 31½ feet to 51½ feet below the existing ground surface and the drilling and sampling of 2 soil borings within the basement of the existing building to an approximate depth of 19½ feet below the top of basement concrete slab, and 6 Cone Penetration Tests (CPTs) with one on the exterior of the building to 60 feet below existing grades, and 5 within the basement to a maximum depth of 50 feet;
4. bulk sampling of the near-surface soils;
5. laboratory testing;
6. engineering analyses; and,
7. preparation of this report.

To assist in the preparation of this report, we have reviewed the following documents:

- *Geotechnical Engineering Study, Ten Space Development, Open Window Project, Downtown Stockton*, prepared by Condor Earth Technologies (Project No. 7454A), dated April 24, 2017, herein referred to as the *Condor Geotechnical Engineering Study;*
- *Building Evaluation Report, San Joaquin County Office of Education,* prepared by Architechnica, dated November 10, 2021; and,
- *Building Site -Visit Summary*, prepared by Bevier Structural Engineering, dated October 26, 2021.
- *Code Stack Design Development Drawings* prepared by Architechnica, dated December 11, 2023.
- *Preliminary Building Load* summary prepared by Bevier Structural Engineering, dated July 2023.

FIGURES AND ATTACHMENTS

This report contains a Vicinity Map as Figure 1; a Site Plan, showing the approximate soil boring locations as Figure 2; and, the Logs of Soil Borings are presented as Figures 3 through 10. An explanation of the symbols and classification system used on the logs is included as Figure 11. Logs of Soil Borings from a previous investigation presented in the *Condor Geotechnical Engineering Study* are included as Figures 12 and 13. Figures 14 and 15 show the approximate extents of the former Miners Slough. Appendix A contains information of a general nature regarding project concepts, exploratory methods used during the field phase of our investigation, an explanation of laboratory testing accomplished, and laboratory test results. Appendix B contains *Earthwork Specifications* that may be used in the preparation of contract plans and documents. Appendix C contains the results of GeoSuite liquefaction and seismic settlement analyses. A copy of the Gregg Drilling Cone Penetration Testing Report is included as Appendix D.

PROJECT DESCRIPTION

We understand that the San Joaquin County Office of Education (SJCOE) has purchased the existing building at 201 N California Street (Parcel 139-25-004) to be renovated as the new home of SJCOE's Code Stack Academy. In addition, the vacant lot at 206 N Sutter Street

(Parcel 139-25-003), located to the west of the building site, has been purchased and will be developed for use as a new 50-space parking lot. The parking lot may also be used for possible carport solar development.

The existing building, formerly known as the State Hotel, was built in 1923 and is 3 stories tall with a full basement (5,774-square feet in size) that extends beneath the sidewalk along California Street. The building is constructed of unreinforced brick masonry, concrete, and dimensional redwood framing with structural steel framing present at the basement level and the ground floor. Interior framing on the 2nd and 3rd floors consists of load bearing and non-load bearing dimensional lumber.

The currently proposed project includes the replacement of the existing structural steel and interior wood framing system with a new three-story structural steel frame with concrete-filled metal deck which will be designed to support the anticipated building loads and to resist lateral wind and earthquake loads. The west wall of the existing building, including the basement wall and foundations, will be removed to allow for extension of basement and a three story building addition to the west of existing building. The existing unreinforced masonry walls will no longer be used to resist lateral loads. Parapet walls will be braced back into the new structural frame. Masonry walls are proposed to be tied into the structural steel frame. Preliminary column loading information from Bevier Structural Engineering indicates the planned 25 individual column loads will vary from about 10 kips to 315 kips dead plus live plus seismic. We understand that the new basement slab is proposed to be two feet lower than the existing slab to provide addition head room and useable space.

Associated development will include curb, gutter and sidewalk repairs and replacements, installation of underground utilities, and a landscaped parking lot located on the parcel extending west from the existing building. Future improvements to the west parking lot could include construction of solar canopies.

FINDINGS

SITE DESCRIPTION

The relatively irregular-shaped site consists of approximately 0.86 acres and currently supports two parcels: APN 139-25-004 and APN 139-25-003 located northwesterly of the intersection between North California Street and Channel Street in Stockton, California.

The site is bounded to the north by an asphalt concrete (AC) paved parking lot used for public parking and an existing single-story brick masonry building, beyond which is East Miner Avenue; to the east by North California Street; to the west by North Sutter Street; and, to the south by Channel Street.

At the time of our field investigation performed between April 13, 2023, and April 17, 2023 the perimeter of the site has been enclosed by a chain link fence. Parcel 139-25-004 located at 201 N California Street currently supports an existing 3 story building with a full basement, purchased by the SJCOE to be renovated as the new home of the SJCOE's Code Stack Academy. The remainder of the parcel supports deteriorated asphalt concrete pavements.

At the time of our field investigations performed between April 13, 2023, and October 30, 2023, Parcel 139-25-003 located at 206 N Sutter Street to the west of the building site, currently supports a vacant lot supporting a light to medium concentration of weeds and grasses. The exterior brick walls on the west side of the structure show signs of significant cracking and distress.

Topography of the site is relatively flat with an average surface elevation of approximately +15 feet relative to mean sea level (msl), based on our review of the topographic information presented in the *United States Geological Survey (USGS) 7.5 Minute Series Topographic Map of the Stockton West Quadrangle, San Joaquin County California (2021).*

SITE HISTORY

The project site history was compiled based on the review of historical aerial photographs of the site from Google Earth and HistoricAerials.com, taken in 1957, 1967, 1982, and from 1993 through 2022.

Review of an aerial photograph taken in 1957 indicates the site supports two buildings. Parcel 139-25-004 supports the existing building, formerly known as the State Hotel, constructed in 1923. Parcel 139-25-003 appears to support a building undergoing construction.

Review of an aerial photograph taken in 1967 indicates Parcel 139-25-003 supports a large building, which spans across a majority of the parcel footprint.

Review of aerial photographs taken in 1982, 1984, and between 1993 and 2016 indicate the site has remained relatively unchanged from 1967.

Review of an aerial photograph taken in 2017 indicates Parcel 139-25-003 has been cleared of the building observed between 1957 and 2016. The remainder of the site has remained relatively unchanged.

Review of aerial photographs taken between 2017 and 2022 indicate the site has remained relatively unchanged since 2017.

SITE GEOLOGY

The *Geologic Map of the San Francisco-San Jose Quadrangle, California* 1:250,000, Published in 1991, and compiled by D.L. Wagner, E.J. Bortugno, and R.D. McJunkin, indicates the project site is underlain by the Quaternary-aged Modesto Formation (Q_m) consisting of mainly arkosic alluvium.

HISTORIC OLD SLOUGH (MINER CHANNEL)

Based on previous work by Condor, fill and grading activities from historic urban development have altered the surface conditions over the past 150 years. This alteration includes the infilling of the historic Miner Channel which passes within the project (See Figure 12 and 13). The channel was replaced with a buried pipe and backfilled, however, we are not aware of any description of the backfill. The buried pipe is still operational and under control of the Municipal Utilities Department. The presence of the backfilled slough has significantly altered the subsurface conditions at the project site. The backfill of the slough is indicated to contain soft, compressible fine-grained soil deposits as well as loose sands.

Structures underlain these deposits have suffered distress and cracking due to (long-term) compression of these deposits, resulting in settlement (especially differential).

SOIL CONDITIONS

Based on historic review of the former slough, it appears that the at least the north end of the site and possibly adjacent areas of the building are underlain with the historic slough backfill. This has created variable soil conditions underlying the site. What we interpret as compressible slough deposits (very soft clays and very loose sands) were encountered in three of our boring located near the north end of the building at depths between 12 to 25 feet below the exterior grades of the building.

Four borings (Borings D1 through D4) were performed on Parcel 139-25-004 on April 13, 2023 and April 17, 2023, around the exterior of the existing building on the northern and western sides of the building to approximate depths ranging between 31½ feet to 51½ feet below existing site grades. The surface soils conditions encountered in all four borings consisted of 2 to 3 inches of AC over 4 to 9 inches of aggregate base (AB) materials. Undocumented fill soils were encountered in all four borings performed around the exterior of the building to depths ranging between approximately 5½ feet to 12½ feet below existing site grades (approximately +15 feet msl). The undocumented fill soils were generally underlain by very soft to very stiff sandy and silty clays, very stiff to hard clayey silts, and medium dense silty and clayey sands to the maximum depths explored of 31½ to 51½ feet below existing site grades. Soft, potentially compressible soils were generally exposed from depths of at least 12 to 25 feet below existing grades. Fragments of wood (timber) and asphalt were encountered in Boring D2 at approximate depths of 2 feet and 5 feet below existing site grades, respectively. Some gravel and fragments of red brick were also encountered in Boring D3 at approximate depths of 2 feet and 11 feet below existing site grades, respectively.

Two borings (Borings D5 and D6) were performed on Parcel 139-25-003 on April 17, 2023, in areas proposed to support the future AC paved parking lots and/or shade structures to approximate depths ranging between 15 feet to 20 feet below existing site grades. Undocumented fill soils were encountered in boring D5 and D6 to depths ranging between approximately 4 feet and 12 feet, below existing site grades (approximately +15 feet msl), respectively. The undocumented fill soils consisted of clayey gravel and medium stiff to very stiff sandy clays with gravel and asphalt or sandy clays with brick debris or concrete and asphalt debris. The undocumented fill soils were generally underlain by medium stiff to stiff silty clays and clayey silts to the maximum depths explored of 15 feet to 20 feet below existing site grades.

Two borings (Borings D7 and D8) were performed on Parcel 139-25-004 on April 14, 2023, inside the existing building at the basement floor level to an approximate depth of 19½ feet below the concrete slab level. Initially, the basement concrete slab was cored, prior to drilling, to determine the concrete slab thickness. The concrete slab thickness at borings D7 and D8 ranged between 4 inches and 5½ inches in thickness, respectively. The concrete slab was underlain by undocumented fill soils to approximate depths ranging between 1½ feet and 2½ feet below the top of the slab. The undocumented fill soils were underlain by variable clayey and sandy silts, clayey and silty sands, as well as silty and sandy clays to the maximum depth explored of 19½ feet below the top of the concrete slab.

Soil behavior types interpreted from CPT exploration performed on October 30, 2023, revealed the presence of variable, soft, potentially compressible clay and silt soils within the upper 25 to 30 feet below basement grades and with the upper 30 to 35 feet below exterior grades, underlain by a stiffer sandier soil layer of variable thicknesses in the range of depths between 25 to 40 feet.

GROUNDWATER

Groundwater was encountered in five of the six borings performed outside of the existing building and encountered in both borings performed inside the basement of the building.

Groundwater was initially encountered in Borings D1 through D5 at depths ranging between approximately 18 feet and 23 feet below existing exterior site grades. The final groundwater depths measured prior to backfill of the borings, ranged between approximately 16 feet and 17 feet below existing site grades.

Groundwater was initially encountered in Borings D7 and D8 at an approximate depth of 9 feet below the top of the basement concrete slab. The final groundwater depth measured prior to backfill of the borings was approximately 6½ feet below the top of the concrete slab.

Please note, the drilled borings may not have been left open long enough for groundwater to reach static equilibrium.

Based on our review of the *Condor Geotechnical Engineering Study*, which included groundwater information from a previous investigation performed in 2017, groundwater was encountered at an approximate depth of 25 feet below existing site grades.

To supplement our groundwater information, we have also reviewed available groundwater elevation data from the California Department of Water Resources (DWR) SGMA Data Viewer maps produced for the period between 2012 to 2022, which indicate the shallowest and deepest depths to groundwater ranged between 20 feet and 30 feet between 2017 and 2020, and 40 feet to 50 feet between 2022, respectively.

CONCLUSIONS

BEARING CAPACITY AND FOUNDATION SUPPORT

Based on the soil conditions encountered by our subsurface exploration and the results of our laboratory testing, the surface and near-surface soils to depths of approximately 25 to 40 feet below existing grades are variable with respect to composition, density and strength.

In our opinion, these soils are not considered capable of supporting the proposed structure without experiencing damaging differential settlements. Based on the confining size of the project site within an existing building, the relatively shallow depth to groundwater, and existing structure's sensitivity to vibration, systems such as helical anchor system would be feasible methods of improving support conditions at this site. Therefore, we will recommend the proposed structure be supported upon a rigid, structural mat which would act as a pile cap for the underlying deep foundation elements of helical piles to provide adequate building support and minimize the effects of differential settlements of both the existing and new structures and reduce influences of liquefaction settlements on the structure. The existing building itself is likely subject to future static and seismic settlements; therefore, underpinning with helical piles will help to provide uniformity of support and performance.

Our work also indicates that sub-excavated and recompacted engineered fill, which is properly placed and compacted in accordance with the recommendations of this report, will be capable of supporting the proposed improvements.

EFFECTS OF EXISTING DEVELOPMENT ON NEW CONSTRUCTION

We understand that the new mat slab is proposed to be two feet lower than the existing slab to provide addition head room and useable space. The existing brick building walls are showing signs of distress, and in our opinion, it will be important to limit the effects of adding new loads to the existing foundations to reduce the potential for additional settlements and building distress. In our opinion, all existing foundations to remain should be underpinned and stabilized using helical anchors similar to the new mat slab to provide uniform support between the two structures as well as help reduce future differential settlements between the new and existing structures.

EXPANSIVE SOIL

The results of our subsurface exploration and previous laboratory testing and work by others, indicate the native clays are potentially expansive. These clays, when present within the upper portion of the building pads, are capable of exerting significant expansion pressures on building slabs and foundations, and exterior flatwork when subjected to variations in soil moisture content, which must be considered in design and construction. Specific recommendations to reduce the effects of expansive soils are presented in this report.

SEISMIC SITE CLASS

The seismic design requirements for buildings and other structures are based on Seismic Design Category. Site Classification is required to determine the Seismic Design Category for a structure. The Site Classification is based on the upper 100 feet of the site profile defined by a weighted average value of either shear wave velocity, standard penetration resistance, or undrained shear strength in accordance with Section 20.4 of ASCE 7-16.

Subsurface explorations at this site were extended to a maximum depth of 51½ feet. Based on the results of our field investigation and laboratory testing, seismic CPT data, site geology, and our review of previous subsurface exploration in the immediate area by Condor, it is our professional opinion that most appropriate Seismic Site Classification for this site is Site Classification D.

We assumed that the soil properties below the boring depth to 100 feet are similar to that at maximum boring depth based on our experience and knowledge of geologic conditions of the general vicinity. Should more accurate site classification be required, additional deeper borings or geophysical testing may be performed to confirm the conditions below the current boring depth.

SEISMIC DESIGN PARAMETERS

The 2022 CBC Seismic Design Parameters have been generated using the ASCE 7 Hazard Tool (https://asce7hazardtool.online/). This web-based software application calculates seismic design parameters in accordance with ASCE 7-16 and 2022 CBC. The results indicate a mapped S1 value of 0.283. Per Section 11.4.8, a site-specific ground motion study should be performed in accordance with Section 21.2 of ASCE 7-16 for Site Class D sites with S1 value greater than or equal 0.2.

Supplement 3 to Section 11.4.8 of ASCE 7-16 includes an exception from such analysis for specific structures on Site Class D sites.

> **EXCEPTION**: A ground motion hazard analysis is not required where the value of the parameters S_{M1} determined by Eq (11.4-2) is increased by 50% for all applications of S_{M1} in this Standard. The resulting value of the parameter m determined by Eq. (11.4-4) shall be used for all applications of S_{D1}, in this Standard.

The commentary for Section 11 of ASCE 7-16 Supplement 3 states that "The Item 1 exception is intended as an acceptable way to address the inaccuracy of the spectral shape observed in the velocity domain for Site Class D sites subject to high ground motions. Increasing S_{M1} by 50% in Eq. (11.4-2) results in an increase in the value of S_{D1} determined by Eq. (11.4-4) by 50 percent. These increased values of SM1 and SD1 are to be used for all applications of these parameters throughout the Standard, including for the formulation of the design response spectrum where a design response spectrum is needed per this standard. It should be noted that the 50% increase in S_{D1} also increases T_s by 50% resulting in an extension of the acceleration-controlled plateau of the design response spectrum."

Based on this exception, the spectral response accelerations presented below were calculated using the site coefficients (F_a and F_v) from Tables 1613.2.3(1) and 16132.3(2) presented in Section 1613.2.3 of the 2022 CBC.

Table 1 - 2022 CBC/ASCE 7-16 Seismic Design Parameters

Description	Value
Site Location	Latitude: 37.9557° / Longitude: -121.2854°
Site Classification	D
Mapped MCE_R ground motion 1)	S_S =0.725 and S_1 = 0.283
Site Coefficients	F_a = 1.22 and F_v = 2.034 2)
Site-modified spectral acceleration	S_{MS} = 0.885 and S_{M1} = 0.863 2)
Numeric seismic design value	S_{DS} = 0.590 and S_{D1} = 0.576 2)
Site modified peak ground acceleration	PGA_M = 0.393 g
Mode de-aggregated Magnitude 3)	5.5
Closest Distance, r_{Rup} 3)	110.84 km

1) These values were obtained using ASCE 7 Hazard Tool (https://asce7hazardtool.online/) accessed at 7/24/2023.
2) The value of the parameters, S_{M1}, determined by Eq. (11.4-2) of ASCE 7-16 is increased by 50% for all applications of S_{M1} per ASCE 7-16 Supplement 3.
3) This value was obtained using on-line Unified Hazard Tool by the USGS (https://earthquake.usgs.gov/hazards/interactive/) for return period of 2% in 50 years accessed at July 24, 2023.

The commentary for Section 11 of ASCE 7-16 (Page 534 of Section C11.4.8 of ASCE 7-16) states that "In general, this exception effectively limits the requirements for site-specific hazard analysis to very tall and or flexible structures at Site Class D sites." Based on our understanding of the proposed structure, it is our assumption that the exception in Section 11.4.8 applies to the proposed structure. However, the structural engineer should verify the applicability of this exception.

Site-specific ground response and ground motion hazard analyses, and/or time history analyses were not part of our work scope.

Typically, a site-specific ground motion study will generate less conservative coefficients and acceleration values which may reduce construction costs. We recommend consulting with a structural engineer to evaluate the need for such study and its potential impact on construction costs. MPE should be contacted if a site-specific ground motion study is desired.

LIQUEFACTION POTENTIAL

Liquefaction is a process in which strong ground shaking causes saturated soils to lose their strength and behave as a fluid. Ground failure associated with liquefaction can result in severe damage to structures. Soil types susceptible to liquefaction include sand, silty sand, sandy silt and silt, as well as soils having a plasticity index (PI) less than 7 (Boulanger and Idriss, 2006). Loose soils with a PI less than 12 and moisture content greater than 85 percent of the liquid limit are also susceptible to liquefaction (Bray and Sancio, 2006). For liquefiable soils, the geologic conditions for increased susceptibility to liquefaction are: 1) shallow groundwater (generally less than 50 feet in depth), 2) the presence of unconsolidated sandy alluvium, typically Holocene in age, and 3) strong ground shaking. All three of these conditions must be present for liquefaction to occur.

Liquefaction potential of the site was evaluated based on Boulanger and Idriss (2010-1016)[1] method using an estimated high groundwater level of 6 feet below the basement slab level. The site modified peak ground acceleration, PGA_M, of 0.393g and a mode-deaggregated earthquake magnitude (Mw) of 5.5 were utilized as input into the liquefaction analysis program GeoSuite©, version 3.3 (Yi, 2023)[2]. The theory and methodology of liquefaction potential and seismic settlement evaluations are described in the appended *Theory and Methodology of Liquefaction and Seismic Settlement* section of this report.

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[1] Idriss, I. M. and Boulanger, R. W., (2010). "SPT based liquefaction triggering procedures." Rep. No. UCD/CGM-10/02, Univ. of California, Davis, CA

[2] Yi, F., 2023, GeoSuite©, version 3.3 - A Comprehensive Package for Geotechnical and Civil Engineers, GeoAdvanced, http://geoadvanced.com/.

Liquefaction potential was evaluated for the soil behavior type profiles encountered at five of the six CPT locations. Data from CPT 4 was excluded. The results of liquefaction potential evaluations are included in Appendix C.

SEISMIC SETTLEMENT

Liquefaction-induced settlement was evaluated following the procedures described by Idriss and Boulanger (2008)[3]. The seismic settlement of dry sands was evaluated following the procedures described by Yi (2022)[4] which incorporated UCLA volumetric strain material model (VSMM) for fine contents correction (Duku et al. 2008[5]; Yee et al. 2014[6]; Stewart, 2014[7]).

Seismic settlement was estimated for the same soil profiles utilized in the liquefaction analyses. The results of liquefaction potential and seismic settlement evaluations are shown in Appendix C. Total seismic settlements varied from about 0.6 to 1.8 inches. Figure C8 shows the distribution of seismic settlements at each location.

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[3] Idriss, I. M., and Boulanger, R. W., 2008, Soil Liquefaction During Earthquake, Earthquake Engineering Research Institute, EERI Publication MNO-12.

[4] Yi, F., 2022, "Procedures to Evaluate Seismic Settlement in Dry Sand Based on CPT Data – An Update", Proceedings of the 5th International Symposium on Cone Penetration Testing (CPT'22), 8-10 June 2022, Bologna, Italy (Cone Penetration Testing 2022)

[5] Duku, PM, JP Stewart, DH Whang, and E Yee (2008). Volumetric strains of clean sands subject to cyclic loads, J. Geotech. & Geoenv. Engrg., ASCE, 134 (8), 1073-1085.

[6] Yee, E., Duku, P. M., and Stewart, J. P. (2014). Cyclic volumetric strain behavior of sands with fines of low plasticity, J. Geotech. & Geoenv. Engrg., ASCE, 140(4), 04013042 (10 pages).

[7] Stewart, J. P. (2014), Notes on Seismic Compression of Compacted Soils, C&EE 225 – Geotechnical Earthquake Engineering, University of California, Los Angeles.

SURFACE MANIFESTATION OF LIQUEFACTION

Both modified liquefaction potential index (LPI_{ISH})[8] and liquefaction severity number (LSN)[9] were calculated for all soil profiles. The LPI_{ISH} indicates that the liquefaction risk of the site is "very low." The site exhibits little expression of liquefaction as per the LSN index. A little expression of liquefaction means "little to no expression of liquefaction" at ground surface, during or after earthquake shaking per Tonlin & Taylor (2013).

SUITABILITY OF ON-SITE SOILS FOR USE AS FILL

In our opinion, the on-site soils encountered in our test borings are considered suitable for use as engineered fill materials provided they are free of rubble, debris and organics, and are at a suitable moisture content to achieve the desired degree of compaction. Based on the subsurface conditions which exposed undocumented fills containing rubble and debris (reference Borings D5 and 6 within the future parking lot and solar canopy), hand picking of rubble and debris may be required prior to re-use as engineered fill. Additionally, use and re-use of the on-site soils, especially within the proposed west parking lot area, should consider the potential environmental issues identified from previous work by others.

EXCAVATION CONDITIONS

Based on our field investigation, the native soils on the site should be readily excavatable with conventional earthmoving and trenching equipment typically used in the area.

Excavations likely will stand at a near-vertical inclination for short periods of time, unless zones or pockets of clean cohesionless sands are encountered or the construction is performed during the rainy season. Excavations encountering perched water, saturated soils, or excavations exposing granular, silty sand soils may slough or cave if left open for an extended period of time requiring sloped excavations and other stabilization methods.

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[8] https://geoadvanced.com/support/tech-notes/surface-manifestation/
Maurer, B. W., Green, R. A., and Taylor, O.S., 2015, Moving towards an improved index for assessing liquefaction hazard: Lessons from historical data, Soils and Foundations 2015; 55(4): 778-787, The Japanese Geotechnical Society

[9] Tonkin & Taylor Ltd, 2013, Liquefaction vulnerability study, Report prepared for Earthquake Commission, New Zealand

Excavations deeper than five feet that will be entered by workers should be sloped and/or braced in accordance with current OSHA regulations. The contractor must provide an adequately constructed and braced shoring system in accordance with federal, state and local safety regulations for individuals working in an excavation that may expose them to the danger of moving ground. If material is stored or heavy equipment is operated near an excavation, stronger shoring would be needed to resist the extra pressure due to the superimposed loads.

GROUNDWATER

Groundwater was initially encountered in Borings D1 through D5 at depths ranging between approximately 18 feet and 23 feet below existing exterior site grades with final groundwater depths measured prior to backfill of the borings, ranged between approximately 16 feet and 17 feet below existing site grades. Groundwater was initially encountered in the basement level at an approximate depth of 9 feet below the top of the basement concrete slab with final groundwater depth measured at approximately 6½ feet below the top of the concrete slab.

Based on the proposed development and assumed shallow depth of utilities, groundwater should not be a factor in the design or construction of the planned parking lot improvements. If deeper excavations, on the order of 15 feet or deeper are planned then groundwater and/or wet soils could be a factor. Excavations deeper than 5 to 10 feet below basement could encounter groundwater and/or wet soils. The need for dewatering of excavations, if required, can best be determined during site work when subsurface conditions are fully exposed.

SEASONAL WATER

The near-surface soils also may be in a near-saturated condition during and for a significant time following the rainy season. If grading operations are to proceed shortly after the rainy season, and before prolonged periods of warm dry weather, the near-surface soils may be at moisture contents where significant aeration or chemical-treatment may be required to dry the soils to a moisture content where the specified degree of compaction can be achieved.

Furthermore, soils located beneath existing pavements, slabs, and flatwork, or within or adjacent to landscaped areas will likely be at elevated moisture contents regardless of the time of year of construction and require drying. Wet soils should be anticipated and considered in the construction schedule.

Seasonal moisture and landscape irrigation will result in high soil moisture contents below interior floor slabs throughout their lifetime. Moisture vapor penetration resistance should be a significant consideration in design and construction of interior floor slabs.

SOIL CORROSION POTENTIAL

Six samples of the sub-surface soils was submitted to Sunland Analytical to determine soil pH, minimum resistivity, chloride and sulfate concentrations to help evaluate potential for corrosive attack upon reinforced concrete and exposed buried metal. The results of the corrosivity testing are summarized in Table 2.

Table 2 – Soil Corrosivity Testing

Sample Identification	CA DOT Test #643 Modified (Sm. Cell)		CA DOT 417	CA DOT 422
	pH	Minimum Resistivity	Chloride	Sulfate
D1 (1’ – 1½’)	7.67	1,150 Ω-cm	19.0 ppm	61.4 ppm
D3-7 (25’ – 26’)	7.54	1,660 Ω-cm	8.1 ppm	10.2 ppm
D2-7 (25’ – 26’)	7.55	1,770 Ω-cm	15.7 ppm	19.0 ppm
D2-3 (8½’ – 10’)	7.20	460 Ω-cm	54.4 ppm	218.5 ppm
D6-2 (3½’ – 5’)	7.81	880 Ω-cm	21.1 ppm	55.8 ppm
D5-3 (8½’ – 10’)	7.18	560 Ω-cm	135.7 ppm	37.9 ppm

* = Small cell method   Ω-cm = Ohm-centimeters   ppm = Parts per million

The California Department of Transportation Corrosion Technology Section, Office of Materials and Foundations, Corrosion Guidelines Version 3.0, March 2018, considers a site to be corrosive to foundation elements if one or more of the following conditions exists for the representative soil and/or water samples taken:

- a minimum resistivity value for soil of less 1,100 ohm-cm,
- *Chloride concentration is 500 ppm or greater,*
- *sulfate concentration is 1500 ppm or greater, or*
- *the pH is 5.5 or less.*

Based on these criteria, in general, the on-site soils are not considered highly corrosive to steel reinforcement properly embedded within Portland cement concrete for the samples tested. The exceptions to this are the low resistivities such as those at Borings D2, D5 and D6 should be considered to represent highly corrosive soils.

Table 19.3.1.1 – Exposure Categories and Classes, American Concrete Institute (ACI) 318-19, Section 19.3, as referenced in Section 1904.1 of the 2022 CBC, indicates the severity of sulfate exposure for the samples tested is not a concern. Ordinary Type I-II Portland cement is considered suitable for use on this project, assuming a minimum concrete cover is maintained over the reinforcement. However, based on the tested sulfates in Boring D2, the soils in Boring D2 have a Class S1 designation due to higher sulfates and need to be considered in the design.

Underground Metallic Pipelines

According to Pierre R. Roberge[10], Section 5.2.3, Table 5.3 the resistivity values of the onsite soils indicated the onsite soils are rated “highly corrosive” to “extremely corrosive” to ferrous metals including ductile/cast iron, steel, and dielectric coated steel. Additionally, review of Table 5.4 indicates a point count higher than 10, based on the tested soils, therefore corrosion protective measures such as cathodic protection (CP) are recommended.

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[10] R. Roberge (2006), Corrosion Basics: An Introduction, Third Edition

Mid Pacific Engineering, Inc. are not corrosion engineers. Therefore, to further define the soil corrosion potential at the site, or to determine the need or design parameters for cathodic protection or grounding systems, a corrosion engineer should be consulted.

Import fills, if used for construction, should be sampled and tested to verify the materials have corrosion characteristics within acceptable limits and generally should be similar to the tested on-site soils.

PAVEMENT SUBGRADE QUALITY & SUPPORT

We anticipate the soils exposed at pavement subgrade elevation will consist of poor quality clay soils. Based on our work and considering the variability of soils anticipated to be exposed, it is our opinion that a Resistance ("R") value of 5 is considered appropriate for design of pavements at this site.

Of concern for the west parking lot is the presence of undocumented fills and the possible remnants remaining from former structures. Our borings exposed between 4 and 12 feet of fills some which contained asphalt, concrete and brick rubble and debris. It may not be feasible to remove and recompact the entire depth of fills due to cost and also considering the potential environmental issues that it may expose. If the fills are not completely removed and recompacted, there would be an increased risk of surface distress, cracking and increased maintenance and repairs over the life of the pavements. If an increased risk is acceptable, one option might be to remove a portion of the upper fills (say the upper two feet), scarify and recompact the bottom, removing exposed remnants and then lime-treat the upper soils to provide an increase in near surface support. It may such that the soils to be lime-treated will require screening or additional handpicking to remove over-size debris or rubble that would interfere or damage the rotary mixer.

Our experience indicates that lime treatment of clay soils can result in a substantial improvement to the support characteristics of the clays, and reduce the required thickness of the base materials. Chemical treatment also can be used to reduce the moisture content of near-saturated soils to facilitate grading operations, and reduce expansion potential of expansive soils.

Additional testing is needed to determine the minimum spread rates of the various products and the resulting subgrade support achieved. It will be important that the subgrade soils be tested and evaluated after initial grading to determine the most appropriate treatment options based on the exposed soil conditions. An experienced soil stabilization contractor must be retained to help facilitate selecting the most appropriate products for treatment for the exposed soils and subgrade conditions.

The performance of chemically stabilized soils is very dependent on adequate and uniform mixing of the selected products into the subgrade soils, and providing a proper curing period following compaction. An experienced soil stabilization contractor combined with a comprehensive quality control program is essential to achieve the best results with chemically treated subgrades.

Preliminary recommendations for chemical-treatment are presented in this report. Additional laboratory testing to further evaluate the feasibility of chemical-treatment is needed prior to final design.

RECOMMENDATIONS

Grading and improvement plans were not available at the time we prepared this report; therefore, it is essential that our office review grading plans when they become available to verify the applicability of the recommendations of this report, or provide modified or revised recommendations, as needed. This is an essential requirement. Our office should also review foundation plans, and project specifications to verify compliance with the recommendations provided in this report or provide modified recommendations, as needed.

We anticipate grading operations will be limited to the parking areas, underground utilities and excavations for the new basement.

The recommendations presented below are appropriate for typical construction in the late spring through fall months. The on-site soils likely will be saturated by rainfall in the winter and spring months, and will not be compactable without drying by aeration or the addition of lime (or a similar product) to dry the soils. Should the construction schedule require work to continue during the wet months, additional recommendations should be provided by the Geotechnical Engineer retained to provide services during construction.

Due to the presence of pavements, undocumented fills, and remnants from former structures, the contractor should anticipate additional excavation, backfilling and reworking of areas that may contain remnants of pre-existing and former structures. We recommend construction bid documents contain a unit price (price per cubic yard) for additional excavation of unsuitable materials and replacement with engineered fill.

We consider the undocumented fills we encountered to be suspect with regards to structural support requiring additional investigation. Based on this, we recommend that additional subsurface exploration (potholing) be performed during the initial phase of site work to explore the suspect areas to determine the presence of old fills and remnants from former development. We should coordinate with the Contractor on areas to be explored. The Contractor should be prepared to provide a backhoe and operator to perform the excavations.

SITE CLEARING

Initially, the site should be cleared of existing structures designated for removal, including but not limited to: foundations, slabs, concrete pavements, utilities to be relocated or abandoned including all associated backfill, fences, trees, vegetation, landscaping, demolition debris, rubbish, rubble, and other deleterious materials. Where practical, site clearing operations should extend a minimum of five feet beyond the limits of the proposed structural areas of the site. Existing underground utilities located within the proposed building pad should be completely removed and/or rerouted as necessary. Removal of underground utilities also should include all associated trench backfill. Utilities located outside of the building area should be properly abandoned (i.e., fully grouted provided the abandoned utility is situated at least 2½ feet below the final subgrade level to reduce the potential for localized “hard spots”). Any tree removal should include the rootball and roots larger than ½” in diameter. Adequate removal of roots, rubble or debris may require laborers and handpicking to clear the subgrade soils to the satisfaction of our representative, prior to further site preparation. Demolition debris should be hauled off site. The contractor should anticipate additional excavation, backfilling and reworking of the areas containing existing or former structures. We recommend construction bid documents contain a unit price (price per cubic yard) for additional excavation of unsuitable materials and replacement with engineered fill.

Our review of available literature and historical photographs provides a limited site history. Therefore, unknown buried structures (foundations, basements, utility lines, etc.) may be present on-site and may be encountered during the construction. If encountered, these structures should be removed and the resulting cavities or holes should be backfilled with properly moisture conditioned and compacted engineered fill as described in this report.

Use of concrete rubble in engineered fill construction is up to the Owner/Developer. If concrete rubble is approved for used in engineered fill, it should be pulverized to a maximum particle size of three inches and blended with native, or imported, soils to create a compactable mixture. It is important to note the use of such materials are not conducive to lime-treatment, where proposed.

Additional potholing should be performed in coordination with our office to explore areas of former remnants requiring additional excavation and recompaction. All exposed remnants including old fills should be completely removed and properly backfilled with engineered fill.

We anticipate the exposed soils underlying the existing slabs and pavements will be at an elevated moisture content. These soils should be appropriately aerated to bring the soils to a compactable moisture content, or the soils may be removed and replaced with drier on-site or imported soils, or lime-treated.

Excavations resulting from the clearing and site preparation operations should be cleaned out to expose firm, undisturbed soil and the excavations properly backfilled with engineered fill in accordance with the recommendations of this report. During grading operations, the exposed subgrades should be evaluated by our representative. Any other loose, disturbed, soft or otherwise unstable materials should be removed to expose a firm base for the support of the fill needed to restore the areas back to the required grades.

It is very important that our representative be present during clearing and grading operations to verify adequate removal of existing pavements, trees, and debris. If clearing and removal of structures takes place without our direct observation, additional over excavation of the areas will be required. It is important that excavations resulting from clearing operations be left as shallow dish-shaped depressions for proper location and to allow proper access with compaction equipment during grading operations.

To prevent potential damage to the existing structure and associated foundations, we recommend that subexcavation operations are not performed within three feet of the existing structure.

It is very important that our representative be present grading operations to verify adequate removal of remaining rubble and debris as well as existing undocumented fills and determine the need for additional sub-excavation based on exposed conditions. Sub-excavations deeper than the recommended minimum depths may be needed to fully expose firm and stable, undisturbed native soils.

SITE PREPARATION AND ENGINEERED FILL CONSTRUCTION

Following clearing, all structural areas designated to receive fill, remain at-grade, or achieved by excavation, should be ripped and cross-ripped to a depth of at least 12 inches, thoroughly moisture conditioned to at least two percent above the optimum moisture content, and uniformly compacted to not less than 90 percent of the ASTM D1557 maximum dry density. Thorough and uniform compaction of the existing surface soils is crucial to support of the planned structures therefore full time observation and testing by the Geotechnical Engineer’s representative is recommended during grading.

SUB-EXCAVATION – WEST PARKING LOT AREA

Over excavation within the west parking lot should extend to a depth of at least 24 inches below final subgrade or existing grade, whichever is deeper. The zone of over excavation should extend laterally at least 5 feet beyond the perimeter of the proposed improvements, where possible. The exposed grades should be ripped and cross-ripped to a depth of 12 inches and exposed remnants from former development removed to expose firm and stable conditions, as identified by our office. The grades then should be moisture conditioned and compacted. As noted earlier, screening or hand-picking may be needed to remove rubble, debris and over sized materials to allow proper lime-treatment and fil construction.

Following compaction, the excavations should be backfilled with properly lime-treated soils to construction a minimum 24-inch thick layer of lime-treatment. Lime-treatment may consist of spreading, mixing and compaction in maximum 12 inch layers within the excavation, or alternately, or in addition to, a "mixing table" could be used whereby the soils that have been stockpiled can be spread out in a thin lift in a convenient area adjacent to the excavation, and a standard spreader machine and rotary mixer used to treat the soils. The soils also could be placed as a thin lift within the excavation and treated. Once treated, mixed and remixed, the soils can then be placed and compacted as engineered fill, as recommended. The method selected will depend on the prevailing site conditions, weather and the contractors means and methods. A combination of methods could be used.

If soft or yielding soils are exposed by this processing, excavation should continue until stiff, non-yielding soils are encountered. The depth and extent of required over excavations should be approved in the field by the Geotechnical Engineer of Record prior to placement of fill or improvements.

Compaction must be performed in the presence of the Geotechnical Engineer, or their representative, who will evaluate the performance of the subgrade under compactive loads and identify any loose or unstable soil conditions that could require additional excavation. If unstable areas are exposed during compaction operations, those areas experiencing instability should be removed to a firm base and backfilled with engineered fill. Compaction should be achieved using a heavy, self-propelled, sheepsfoot compactor.

Difficulty in achieving subgrade compaction or unusual soil instability may be indications of loose fills or backfill associated with past subsurface items such as cisterns, burn pits, dump pits or utility lines. Should these conditions exist, the materials should be excavated to check for subsurface structures and the excavations backfilled with engineered fill. We recommend construction bid documents contain a unit price (price per cubic yard) for all excess excavation due to unsuitable materials and replacement with engineered fill.

On-site soils are considered suitable for use in engineered fill construction, if free of rubble, rubbish, debris, or concentrations of organics and are at a compactable moisture content.

Imported fill materials, if required, should be granular with a Plasticity Index of 15 or less; Expansion Index of 20 or less; and, free of particles greater than three inches in maximum dimension. Imported soils should be free of contamination with proper documentation to be provided by the Contractor to the Owner and Owner's environmental consultant. Imported soils should be approved by the Geotechnical Engineer office prior to being transported to the site.

Engineered fill should be placed in horizontal lifts not exceeding six inches in compacted thickness. Each layer should be thoroughly moisture conditioned to at least the optimum moisture content and uniformly compacted to at least 90 percent of the maximum dry density, as defined above. Fill materials should be uniformly and thoroughly moisture conditioned to the full depth of each lift. Compactive effort should be applied uniformly across the full width of the fill. Engineered fills should be properly benched into excavation side slopes to remove loose soils and promote uniformity of fill construction and support.

Backfill Beneath Existing Sidewalk

The portion of the basement underneath the existing sidewalk along the east side of the existing building will not be included in the new development and will be backfilled. Since the slab may act as an impermeable barrier and potentially trap water (the future consequences of which are not known), we recommend the bottom slab be removed (if it will not adversely affect existing of new construction) or at least have holes cut into the bottom to along some drainage, if needed. A geotextile fabric over the cut outs of holes may be used to reduce potential settlements, depending on exposed conditions. In our opinion, since the area is with the City right-of-way, the area should be backfilled with Class 2 aggregate base compacted to at least 95 percent relative compaction.

The upper 12 inches of final building pad (basement) subgrades should be brought to at least the optimum moisture content and uniformly compacted to not less than 90 percent of the maximum dry density and must be stable.

The upper 12 inches of final untreated pavement subgrades should be uniformly moisture conditioned to at least the optimum moisture content, processed, and uniformly compacted to at least 95 percent of the maximum dry density, regardless of whether final grade is completed by excavation, filling, or left at existing grade. Final subgrade preparation and compaction should be performed just prior to placement of aggregate base, and must be stable under construction traffic.

Permanent excavation and fill slopes should be constructed no steeper than two horizontal to one vertical (2:1) and should be vegetated as soon as practical following grading to minimize erosion. As a minimum, erosion control measures including placement of straw bale sediment barriers or construction of silt filter fences in areas where surface run-off may be concentrated would be prudent. Slopes should be over-built and cutback to design grades and inclinations.

Site preparation should be accomplished in accordance with the recommendations of this section and appended *Guide Earthwork Specifications*. A representative of the Geotechnical Engineer must be present during site preparation and grading operations to observe and test the fill to verify compliance with the recommendations of this report.

PRELIMINARY LIME-TREATMENT

Lime treatment can be an effective way to reduce the moisture content of near-saturated or unstable soils to facilitate grading operations. Lime treatment likely will only be economical for treatment of large areas. Lime treated soils will not support landscaping and should be removed from within all planter areas and replaced with suitable landscape soils. Typically, lime treatment should be at least 12 inches thick; however, deeper mixing depths (16 to 18 inches) could be needed in areas of very wet and deeper instabilities, or where additional support is needed. The actual amount of product (spread rate) and mixing depth needed for stabilization can only be determined at the time of construction based upon the prevailing site and soil moisture conditions. The contractor should include an add/deduct unit price for lime to account for variations in the quantities of product used.

In our experience lime, Portland cement or a combination of such products have been used to stabilize variable subgrades, depending on soil types, and soil moisture and stability conditions; therefore; before a decision is made to use any product or combination of products a qualified stabilization contractor must be retained and assist in determining the most effective treatment. It is crucial that the selected stabilization contractor determine the actual product and amount of product to add and the proper mixing depth to achieve the desired results.

Due to wet weather, construction activities and equipment traffic, and the potential for variable subgrade conditions, some isolated areas of instability may be exposed following treatment requiring remedial work and repairs. Such areas may require sub-excavation and use of layers of geogrid and additional thicknesses of aggregate base, or slurry backfill, to stabilize the final grade prior to further pavement construction. Construction equipment and vehicle traffic over the treated subgrade, prior to a proper and adequate curing period, will tend to de-stabilize the grades.

Special care and consideration should be given to those areas where shallow utilities are present. The depth of stabilization may be limited in those areas, depending on the depth of utilities, and since the full depth of treatment may not be achieved, some instabilities may remain, requiring additional stabilization. In the case where shallow utilities are present and/or for isolated areas that are difficult to heal, use of a 2-sack sand-cement slurry may be considered. Selection of a sand-cement slurry mix should consider whether future excavations will be needed. The supplier should be consulted for additional information on anticipated slurry mix strengths.

The native clay soils are anticipated to react well with the addition of quicklime (high-calcium or dolomitic) and could enhance the support characteristics of the subgrade and allow for a reduction in the aggregate base section. Chemical treatment of subgrade soils as part of the pavement section should be performed in accordance with Section 24 of the Caltrans *Standard Specifications*. For preliminary estimating purposes only, we recommend a minimum spread rate of at least 5 pounds of quicklime per square foot of mixing depth (at least 12 inches) based on 4½ percent by dry weight of soil to be treated assumed to have a dry unit weight of 110 pcf. Lime-treated subgrades should be compacted to not less than 95 percent of the ASTM D1557 maximum dry density, at a moisture content of at least two percent above the optimum moisture content. As noted above, a mixing table may be needed to provide the recommended minimum 24 inch lime-treated section.

It should be noted that the surface and near-surface soils across the site may vary; therefore, it will be important that the subgrade soils be tested and evaluated after initial grading to determine the most appropriate treatment options based on the exposed soil conditions. An experienced soil stabilization contractor should be retained to help facilitate selecting the most appropriate products for treatment.

If chemical treatment alternates are selected for use at this site, additional testing should be performed prior to and during construction to verify that the design parameters are achieved in the field. Samples of the field-mixed soil and lime should be collected and tested for minimum unconfined compressive strength of 300 pounds per square inch (psi) when tested in accordance with California Test 373 and a minimum Resistance value of 50 when tested in accordance with California Test 301. This additional testing will either verify the design parameters, including the use of lime and/or cement; provide the opportunity to modify the pavement sections; or, modify the spread rate based upon the test results.

UTILITY TRENCH BACKFILL

We recommend only native soils (in lieu of select sand backfill) be used as backfill for utility trenches located within building footprints and extend at least five feet beyond to perimeter foundations to minimize water transmission beneath the structures. Trench backfill should be thoroughly moisture conditioned to at least the optimum moisture content and mechanically compacted to 90 percent of the ASTM D1557 maximum dry density. The upper 12 inches of trench backfill within pavement subgrades should be compacted to at least 95 percent of the ASTM D1557 maximum dry density. We recommend trenches be constructed prior to lime-treatment; however, if utility trenches are excavated through lime-treatment then the upper portion of the trench backfill should be backfilled wit Class 2 aggregate base compacted to 95 percent relative compaction. Thickness of the AB backfill should match the thickness of the lime-treated section.

We recommend that underground utility trenches that are aligned nearly parallel with foundations be at least three feet from the outer edge of foundations, wherever possible. As a general rule, trenches should not encroach into the zone extending outward at a 1:1 inclination below the foundations. Additionally, trenches near foundations should not remain open longer than 72 hours to prevent drying and potential shrinkage cracks. The intent of these recommendations is to prevent loss of both lateral and vertical support of foundations, resulting in possible settlement.

FOUNDATION DESIGN

The current concept is to use a rigid mat supported upon helical piles bearing into the more competent layers between depths of 25 to 35 feet (some piles may need to extend to 40 feet if capacity is not developed shallower). Total seismic settlements based on our analyses varied from about 0.6 to 1.8 inches with the seismic settlements below the bottom of anticipated helical pile depths to vary between roughly ½ inch to 1 inch.

We are providing design soil values for the analysis of the foundations, and suggested minimums, but only from a Geotechnical Engineering perspective. The project Structural Engineer should determine final foundation design width and depth dimensions and reinforcing requirements, based on their specific structural design, which should include an appropriate factor of safety applied to the overall design.

Effects of Existing Building & New Construction

We understand that the new mat slab is proposed to be two feet lower than the existing slab to provide addition head room and useable space. The existing brick building walls are showing signs of distress, and in our opinion, it will be important to limit the effects of adding new loads to the existing foundations to reduce the potential for additional settlements and building distress. In our opinion, all existing foundations to remain should be underpinned and stabilized using helical anchors similar to the new mat slab to provide uniform support between the two structures as well as help reduce future differential settlements between the structures. We recommend that additional exploration be performed to determine the foundation depth and width dimensions, where possible, of the existing foundations, especially at the perimeter wall locations. This information will be important since new foundations will be constructed close to, or adjacent to, the existing foundations and may help to reduce design and construction conflicts. And also would be essential information to obtain if underpinning and stabilizing of existing foundations

Helical Piles

To our knowledge, helical piles have been used in the immediate area of the site due to the effects of the former miner's slough creating soft and non-uniform soils conditions. Based on available information, it appears that prior helical anchors have been installed extending to depths below basement level varying from approximately 8 and 70 feet with the majority of helical piles extending to between roughly 15 and 55 feet. This data shows the variability of the subsurface conditions in the area due to the old slough and effects on soil support conditions, as well as the overall variability of soils support quality within the area.

We recommend coordinating with a qualified design-build installation contractor early in the design process to verify anticipated anchor capacities at the site based on their experience with similar soil conditions as well as to coordinate shaft sizes, helix configurations, and hardware requirements for the required capacities.

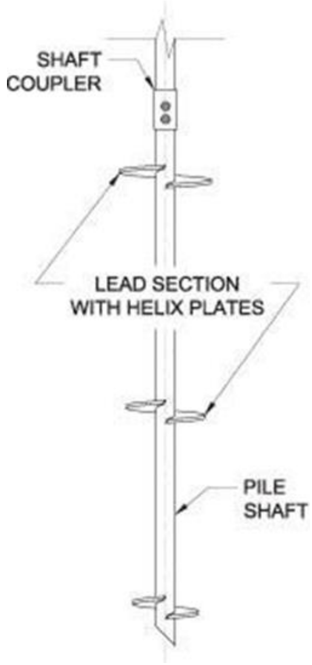
The Structural Engineer should verify the size and adequacy of the center shaft based on the anticipated structural loadings as well as the structural connections of anchors to foundations and anchor spacings based on allowable mat spans. Close coordination between the Structural Engineer and the helical pile designer will be needed.

The number of anchors and anchor spacing will depend on the structural loadings and should be determined by the Structural Engineer in coordination with helical pile designer.

Final depth of each anchor as well as the size and capacity of the center shaft and the size and number of helixes per shaft will depend on the required structural capacity and the depth at which the required torque is achieved that produces the design axial anchor capacity. Appropriate factors of safety should be applied when developing the design anchor capacities. We recommend minimum factors of a safety of at least 3 for dead load, 2 for dead plus live load and 1.5 for total loads conditions including seismic and wind forces. Actual factors of safety should be determined by the Structural Engineer and helical pile designer, based on the specific design requirements and performance expectations.

Helical piles should extend through the soft compressible miner's slough sediments to bear into the more competent layers between depths of 25 to 35 feet (as deep as 40 feet if capacity is not developed shallower), below basement level. Based on our work, anchor depths have the potential to vary significantly depending on the variations in soil conditions encountered across the site and lead section used. In our opinion, load testing of the helical anchors must be performed to verify the maximum and allowable bearing capacities and factors of safety.

Typical Helical Pile



Mat Slab

A system of foundation support consisting of a rigid, reinforced concrete mat slab supported upon helical anchors is proposed.

The subgrades for the proposed mat slab should be prepared and compacted in accordance with the recommendations in this report. Mat foundation thickness and reinforcing, and detailing for incorporating the helical anchors should be determined by the Structural Engineer based on their specific design and analysis. We recommend that all foundations be adequately reinforced to provide structural continuity and spanning between helical anchorage points, mitigate cracking and permit spanning of local soil irregularities.

In our opinion, the mat slab should be no less than 18 inches in minimum thickness and thickened as needed to for accommodating the helical anchor connection details, supporting structural columns, or wherever additional rigidity and integrity is needed. Final concrete slab thickness, compressive strength, reinforcement and detailing should be determined by the Structural Engineer based on anticipated slab loading. Temporary loads exerted during construction from vehicle traffic, cranes, forklifts, and storage of palletized construction materials should be considered in the design of the slab-on-grade floors.

Resistance to lateral displacement of concrete foundations may be computed using an allowable friction factor of 0.20, which may be multiplied by the effective vertical load on each foundation. Additional resistance can be achieved by considering *passive* lateral earth pressure against the vertical projection of the foundations extending below grade equal to an equivalent fluid pressure 150 pounds per cubic foot (pcf). These two modes of resistance (friction and *passive* pressure) should not be added unless the frictional component is reduced by 50 percent due to the mobilization of the resistive forces occurring at different degrees of horizontal movement. Passive resistance should only be used where such forces do not detrimentally affect the existing building and its foundations.

Floor slabs may be underlain by a layer of free-draining crushed rock, serving as a deterrent to migration of capillary moisture. The crushed rock layer should be at least four inches thick and graded such that 100 percent passes a one-inch sieve and none passes a No. 4 sieve. Moisture vapor protection for areas where moisture vapor through the slab is a concern may be provided by placing a plastic water vapor retarder (at least 10-mils thick) directly over the prepared subgrade. The plastic water vapor retarder should meet or exceed the minimum specifications as outlined in ASTM E1745. Consideration should be given to using a thicker, higher quality membrane for additional moisture protection such as a 15-mil thick vapor barrier or other product. The membrane should be installed so that there are no holes or uncovered areas. All seams should overlap and be sealed with manufacturer-approved tape, continuous at the laps to create vapor tight conditions. All perimeter edges of the membrane, such as pipe penetrations, interior and exterior footings, joints, etc., should be sealed or caulked per manufacturer's recommendations. An optional, thin layer of clean sand above the membrane is acceptable, as an aid to curing of the slab concrete.

Floor slab construction over the past 25 years or more has included placement of a thin layer of sand over the vapor retarder membrane. The intent of the sand is to aid in the proper curing of the slab concrete. However, recent debate over excessive moisture vapor emissions from floor slabs includes concern for water trapped within the sand. As a consequence, we consider the use of the sand layer as optional. The concrete curing benefits should be weighed against efforts to reduce slab moisture vapor transmission.

If heavier floor loads are anticipated and/or increased support is desired, the crushed rock section (if used) beneath interior slab-on-grade floors could be replaced with a thicker section of Class 2 aggregate base compacted to at least 95 percent of the maximum dry density as determined by ASTM D1557.

The recommendations presented above are intended to mitigate any significant soils-related cracking of the slab-on-grade floors. More important to the performance and appearance of a Portland cement concrete slab is the quality of the concrete, the workmanship of the concrete contractor, the curing techniques utilized and the spacing of control joints.

FLOOR SLAB MOISTURE PENETRATION RESISTANCE

It is considered likely that floor slab subgrade soils will become wet to near-saturated at some time during the life of the structures. This is a certainty when slabs are constructed during the wet seasons or when constantly wet ground or poor drainage conditions exist adjacent to structures. For this reason, it should be assumed that all slabs in occupied areas, as well as those intended for moisture-sensitive floor coverings or materials, require protection against moisture or moisture vapor penetration. Standard practice includes the gravel and water vapor retarder as suggested above. However, the gravel and plastic membrane offer only a limited, first-line of defense against soil-related moisture. Recommendations contained in this report concerning foundation and floor slab design are presented as *minimum* requirements, only from the Geotechnical Engineering standpoint.

It is emphasized that the use of a water vapor retarder will not "moisture proof" the slab, nor does it assure that slab moisture transmission levels will be low enough to prevent damage to floor coverings or other building components. If increased protection against moisture vapor penetration of slabs is desired, a concrete moisture protection specialist should be consulted. The architect and design team should consider all available measures for slab moisture protection. It is commonly accepted that maintaining the lowest practical water-cement ratio in the slab concrete is an effective way to help reduce future moisture vapor penetration of the completed slabs.

BASEMENT WALL – RETAINING WALL DESIGN PARAMETERS

The new basement walls and other below grade structures should be designed to resist "at-rest" lateral earth pressures equal to an equivalent fluid pressure of 75 psf per foot of wall backfill for the design condition of walls fixed at the top with no sloping backfill or surcharge pressures. Surcharge pressures developed by vehicles, foundations and slabs near the top of the retaining walls also must be included. An appropriate seismic increment should be included in the wall design as required by Code.

Our recommendations assume full drainage behind retaining walls to prevent the build-up of hydrostatic pressure behind the wall. Retaining walls should be provided with a drainage blanket (Class 2 permeable material, Caltrans Specification Section 68-1.025) at least one-foot wide extending from the base of wall to within one foot of the top of the wall, with the top foot above the drainage layer consisting of compacted on-site materials. Perforated rigid pipe should be provided near the base of the wall to drain accumulated water. Drain pipes should slope to discharge at no less than a one percent fall to a suitable drainage discharge point. Open-graded 1/2-inch to 3/4-inch crushed rock may be used in lieu of the Class 2 permeable material, if the rock and drain pipe are completely enveloped in an approved nonwoven geotextile filter fabric.

Structural backfill materials for retaining walls should consist of granular on-site or imported soils free of significant quantities of rubbish, rubble, organics and rock over six inches in size. Structural backfill should be placed in lifts not exceeding six inches in compacted thickness, and should be mechanically compacted to at least 90 percent relative compaction. Structural backfill supporting at-grade structures or pavements should be compacted to at least 95 percent.

Construction of basement and below grade walls should include proper water and moisture–proofing methods to reduce moisture related interior problems.

FUTURE SOLAR CANOPY PIER FOUNDATIONS

These are preliminary recommendations, Mid Pacific Engineering, Inc. should be retained to review the final plans and specifications as they are developed to verify that the intent of our recommendations has been implemented in those documents, and provide revised recommendations as needed.

We are providing design soil values for the analysis of the foundations, and suggested minimums , but only from a Geotechnical Engineering perspective. The project Structural Engineer should determine the final foundation design width and depth dimensions, concrete strength and reinforcing requirements, based on their specific structural design which should include an appropriate factor of safety applied to the overall design.

In our opinion, drilled pier foundations bearing on firm undisturbed ground, engineered fill that is placed and compacted in accordance with the recommendations of our report, or a combination of these materials, as confirmed by our representative, can be utilized for the solar canopy structure foundations proposed for the site.

In general, we recommend the proposed piers consist of drilled, cast-in-drilled hold (CIDH) reinforced concrete piers. Piers for support of the carport structure should be at least 36 inches in diameter and extend through the existing fills to a depth of at least 15 feet below lowest adjacent soil grade. Drilled pier foundations should be structurally isolated from any adjacent concrete flatwork by a felt strip or similar material.

Drilled piers may be sized utilizing a maximum allowable vertical bearing capacity of 3,000 psf. This value may be increased by one-third to include short-term wind or seismic forces. The weight of foundation concrete below grade may be disregarded in sizing computations.

Uplift resistance of pier foundations may be computed using the following resisting forces, where applicable: 1) weight of the pier concrete (150 pounds per cubic foot) and, 2) the allowable skin friction of 100 psf applied over the shaft area of the pier. Increased uplift resistance can be achieved by increasing the diameter of the pier or increasing the depth.

The upper 12 inches of skin friction should be neglected unless the pier is completely surrounded by slab concrete or pavements for a distance of at least three feet from the edge of the foundation pier.

Sizing of piers to resist lateral loads can be evaluated using Section 1807.3.2 of the 2022 CBC. A value of 100 pcf as defined in Table 1806.2 of the CBC may be used for the lateral bearing pressure of the on-site soils, as the coefficients S1 and S3 for the non-constrained and constrained conditions, respectively. Per Section 1806.1 of the 2022 CBC, an increase of 1/3 is permitted when using the alternate load combinations in Section 1605.2 that include wind or earthquake loads. The upper 18 inches of the subgrade should be neglected due to presence of expansive soils and undocumented fills.

Reinforcement and concrete should be placed in the pier excavations as soon as possible after excavation is completed to minimize the chances of sidewall caving into the excavations. Although we do not anticipate excessive sloughing of the sidewalls during pier construction, we recommend that the pier contractor be prepared to case the pier holes if conditions require.

To minimize the amount of sidewall caving, we recommend that a maximum elapsed time of 48 hours between completion of the pier excavation and the start of concrete placement. The bottom of the pier excavations should be free of loose or disturbed soils prior to placement of the concrete. Cleaning of the bearing surface should be verified by the geotechnical engineer prior to concrete placement.

To reduce lateral movement of the drilled shafts, it is necessary to place the concrete for the drilled shafts in intimate contact with the surrounding soil. Any voids or enlargements in the shafts due to excavation or temporary casing installation shall be filled with concrete at the time shaft concrete is placed.

We estimate total settlement for drilled pier foundations using the recommended maximum net allowable bearing pressure and skin friction presented above, should be less than one inch. The settlement estimate is based on the available soil information, our experience with similar structures and soil conditions, and field verification of suitable bearing soils during foundation construction.

It is considered essential that our representative be present during pier drilling to verify adequate depth of penetration into competent bearing soils. Concrete reinforcing steel should not be placed in any pier excavation until approved by our representative.

EXTERIOR FLATWORK (NON-PAVEMENT AREAS)

Areas to receive exterior concrete flatwork (i.e., sidewalks, etc.) should be supported on a minimum 18 inch layer of properly compacted imported non-expansive engineered fill. One option would be to provide 4 inches of Class 2 Aggregate Base (AB) over 14 inches of non-expansive select import engineered fill (or Class 2 AB). Alternatively, to the imported non-expansive soil layer the soil subgrades could be lime-treated to reduce the expansion potential of the soils. All engineered fill placed under flatwork should be compacted as recommended in this report (i.e., compacted to at least 90 percent of the maximum ASTM D1557 dry unit weight at no less than two percent above the optimum moisture content).

Expansion joints should be provided to allow for minor vertical movement of the flatwork. Exterior flatwork should be constructed independent of perimeter building foundations and isolated column foundations by the placement of a layer of felt material between the flatwork and the foundation.

Consideration should be given to thickening the outer edges of sidewalks to at least twice the slab thickness. Thorough moisture conditioning of subgrade soils is important to reduce the risk of non-uniform moisture withdrawal from the concrete and the possibility of plastic shrinkage cracks. Practices recommended by the Portland Cement Association (PCA) for proper placement and curing of concrete, as well as for joint spacing and construction, should be followed during exterior concrete slab construction.

The Architect or Structural Engineer should determine the final thickness, strength, reinforcement, and joint spacing of exterior slab-on-grade concrete; however, we offer the following suggested minimum guidelines. Exterior flatwork should be at least four inches thick and be constructed independent of perimeter building foundations and isolated column foundations by the placement of a layer of felt material between the flatwork and the foundation. Reinforcement should consist of at least heavy duty welded wire fabric (flat sheets), or equivalent steel reinforcing bars, placed mid-depth of the slab. Thicker slabs constructed where light wheeled traffic or intermittent light loading is expected over the slabs. Public sidewalk design, thickness and construction should conform to local jurisdiction requirements. Slabs receiving wheeled or vehicular traffic should be thickened and designed as pavements.

SITE DRAINAGE

Site drainage should be accomplished to provide positive drainage of surface water away from buildings and prevent ponding of water adjacent to structures. The grade adjacent to the structures should be sloped away from foundations at a minimum two percent. Proper control of surface water drainage is essential to the performance of foundations, slabs-on-grade and pavements. We recommend using full-roof gutters, with downspouts from roof drains connected to rigid non-perforated piping directed to an appropriate drainage point away from the structures, or discharging onto paved surfaces leading away from the structures and foundations. Concentrated storm water discharge collected from roof downspouts or surface drains should not be allowed to drain on unprotected slopes adjacent to structures. Finished grades should be graded to drain positively away from all pavement and building structures. Ponding of surface water should be avoided near foundations and pavements. Landscape berms, if planned, should not be constructed in such a manner as to promote drainage toward buildings.

All excavations should be protected from concentrated storm water run-off to minimize potential erosion. Ponding of surface water or allowing sheet flow of water over any open excavation must be avoided.

PAVEMENT DESIGN

The pavement sections have been calculated for a range of traffic indices using the design procedures contained in Chapters 600 to 670 of the 6th Edition of the *California Highway Design Manual.* The project Civil Engineer should determine the appropriate traffic index based on anticipated traffic conditions. We can provide additional section thicknesses for other Traffic Indices, as needed.

Table 3 - Pavement Design Alternatives

Traffic Index (TI)	Traffic Condition	Untreated Pavement Subgrade R-value = 5		Lime-treated Pavement Subgrade (a)	
		Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
5.0	Parking Areas	2½	11	2½*	5
		3	10	3	4
6.0	Drive Aisle	2½	15	2½	8
		3	14	3	7

(a) = Lime-treated subgrade should be at least 12 inches thick and possess a minimum unconfined compressive strength of 300 pounds per square inch (psi) when tested in accordance with California Test 373 and a minimum R-value of 50 when testing in accordance with CTM 301.

We emphasize that the performance of a pavement is critically dependent upon uniform compaction of the subgrade soils, as well as all engineered fill and utility trench backfill within the limits of the pavements. Materials used for pavement construction should conform to the appropriate sections of the most recent editions of the City of Stockton Engineering Standards and the Caltrans *Standard Specifications.*

It has been our experience that pavement failures may occur where a non-uniform or disturbed subgrade soil condition is created. Subgrade disturbances can result if pavement subgrade preparation is performed prior to underground utility construction and/or if a significant time period passes between subgrade preparation and placement of aggregate base. Therefore, we recommend that pavement subgrade preparation, i.e., scarification, moisture conditioning and compaction, be performed just prior to aggregate base placement.

The upper 12 inches of final pavement subgrades should be uniformly moisture conditioned to at least the optimum moisture content and compacted to at least 95 percent relative compaction. Subgrades must be properly compacted and stable prior to placing AB. Pavement subgrades should be proof-rolled with a loaded water truck and must be stable under construction traffic prior to placement of aggregate base. All aggregate base (AB) should be compacted to at least 95 percent of the maximum dry density and density testing performed to verify compaction. In addition, we recommend the AB be proof rolled with a loaded water truck just prior to paving to verify stability. Any areas of observed instability should be stabilized and recompacted as necessary to achieve the compaction requirements above.

Earthwork construction within the limits of the pavements should be performed in accordance with the recommendation contained within this report. Materials quality and construction of the structural section should conform to the applicable provisions of the Caltrans Standard Specifications, latest editions.

Portland Cement Concrete Pavements

In the summer heat, high axle loads coupled with shear stresses induced by sharply turning tire movements can lead to failure in asphalt concrete pavements. Therefore, we recommend that consideration be given to using a Portland cement concrete (PCC) section in areas subjected to concentrated heavy wheel loading, such as entry driveways, truck maneuvering areas, and in front of trash enclosures. At the time this report was prepared, the need for, and locations of, PCC pavements had not yet been determined. Therefore, when more information is available regarding uses, loading and potential subgrade conditions, we should review the information and provide specific thicknesses as applicable.

For preliminary purposes, it may be assumed that Portland cement concrete slabs in areas of entry driveways and in front of trash enclosures should be at least 6 inches thick and be underlain by at least 6 inches of 95 percent compacted Class 2 aggregate base. Thicker slabs will be needed in areas of frequent bus traffic, in heavy duty areas, or areas subjected to high traffic frequencies by heavy trucks or equipment. In these areas, Portland cement concrete slabs with a minimum thickness of 7 inches and underlain by at least 6 inches of 95 percent compacted Class 2 aggregate base may be needed. These sections are preliminary and subject to revision based on review of additional information regarding loadings and traffic frequencies.

We suggest the concrete slabs be constructed with thickened edges in accordance with American Concrete Institute (ACI) design standards. The concrete pavements should contain steel reinforcement. The project design engineer should determine such reinforcing requirements, as well as joint spacing and details. Construction of Portland cement concrete pavements should be performed in accordance with applicable American Concrete Institute (ACI) or PCA standards. Portland cement concrete utilized in pavements should attain a compressive strength of at least 3500 psi at 28 days.

PAVEMENT DRAINAGE

Efficient drainage of all surface water to avoid infiltration and saturation of the supporting aggregate base and subgrade soils is important to pavement performance. Consideration should be given to using full-depth curbs between landscaped areas and pavements to serve as a cut off for water that could migrate into the pavement base materials or subgrade soils. Geotextile water barriers also could be used to prevent migration of water into pavement base materials, if extruded curbs are used. Proprietary geotextile moisture barriers and curb details should be reviewed and approved by our office prior to construction. Weep holes are recommended in parking lot drop inlets to allow accumulating water moving through the aggregate base to drain from beneath the pavements.

Earthwork construction within the limits of the pavements should be performed in accordance with the recommendation contained within this report. Materials used for pavement construction should conform to the applicable sections of the Caltrans *Standard Specifications* and the *City of Stockton Standards*, latest editions, where appropriate.

CONSTRUCTION TESTING AND OBSERVATION

Site preparation should be accomplished in accordance with the recommendations of this report and the appended *Guide Earthwork Specifications*. Representatives of Mid Pacific Engineering, Inc. must be present during site preparation and all grading operations to observe and test the fills to verify compliance with our recommendations and the job specifications. In the event that MPE is not retained to provide geotechnical engineering observation and testing services during construction, the Geotechnical Engineer retained to provide this service should indicate in writing that they agree with the recommendations of this report, and prepare supplemental recommendations as necessary.

A final report by the "Geotechnical Engineer" should be prepared upon completion of the project indicating compliance with or deviations from this report and the project plans and specifications. Please be aware that the title Geotechnical Engineer is restricted in the State of California to a Civil Engineer authorized by the State of California to use the title "Geotechnical Engineer."

LIMITATIONS

Our recommendations are based upon the information provided regarding the proposed construction, combined with our analysis of site conditions revealed by the field exploration and laboratory testing programs. We have used our best engineering judgment based upon the information provided and the data generated from our investigation. This report has been prepared in accordance with generally accepted standards of practice existing in northern California at the time of the report. No warranty, either express or implied, is provided.

If the proposed construction is modified or re-sited; or, if it is found during construction that subsurface conditions differ from those we encountered at the boring and CPT locations, we should be afforded the opportunity to review the new information or changed conditions to determine if our conclusions and recommendations must be modified. Mid Pacific Engineering, Inc. should be retained to review the final plans and specifications to verify that the intent of our recommendations has been implemented in those documents.

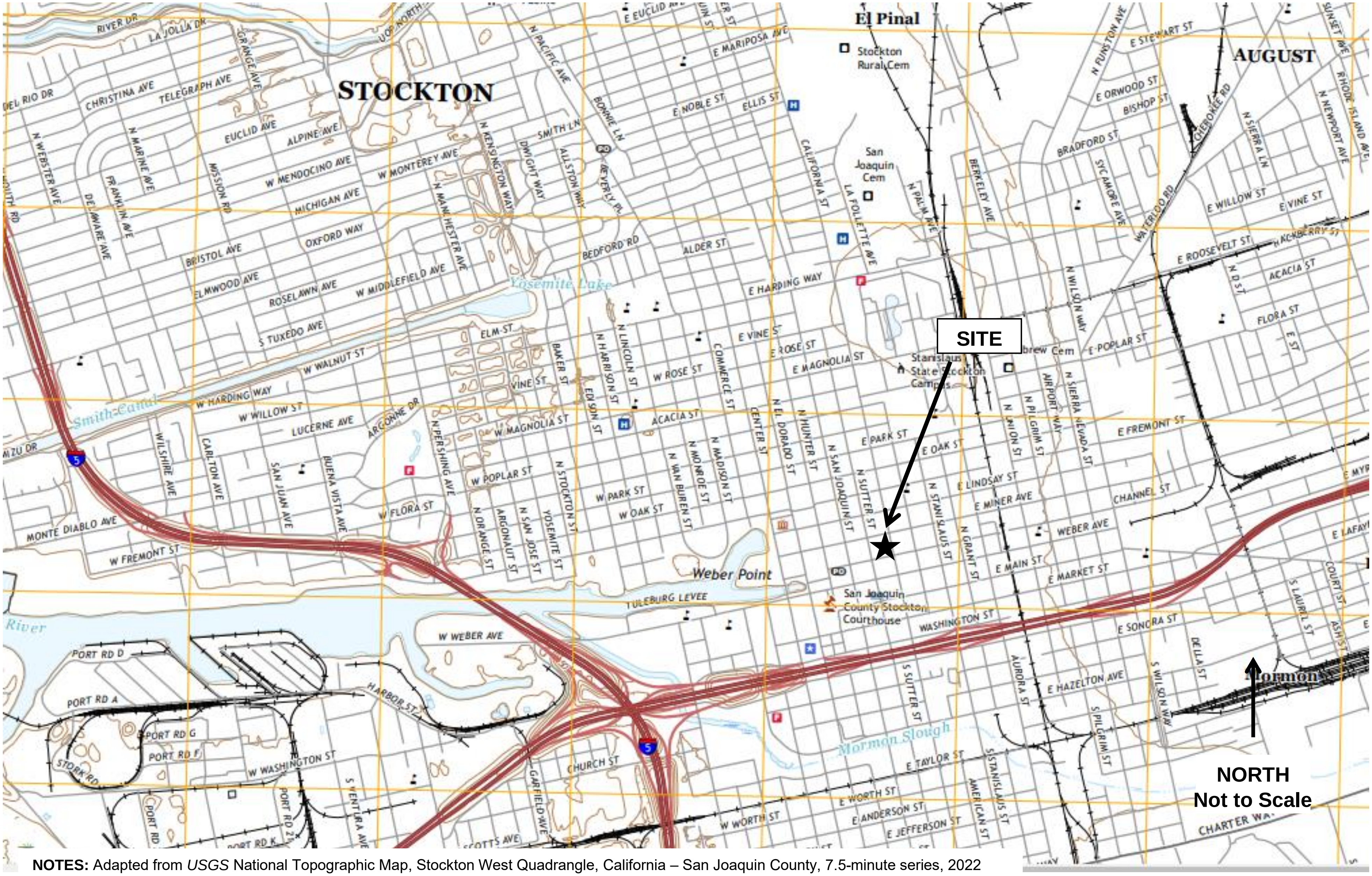
We emphasize that this report is applicable only to the proposed construction and the investigated site and should not be utilized for construction on any other site. The conclusions and recommendations of this report are considered valid for a period of two years. If design is not completed and construction has not started within two years of the date of this report, the report must be reviewed and updated, as necessary.

Mid Pacific Engineering, Inc.

Daniel C. Smith
Principal Engineer

DANIEL C. SMITH
No. 2530
Exp. 6/30/25

FIGURES



NOTES: Adapted from *USGS* National Topographic Map, Stockton West Quadrangle, California – San Joaquin County, 7.5-minute series, 2022

MID PACIFIC ENGINEERING, INC.

VICINITY MAP

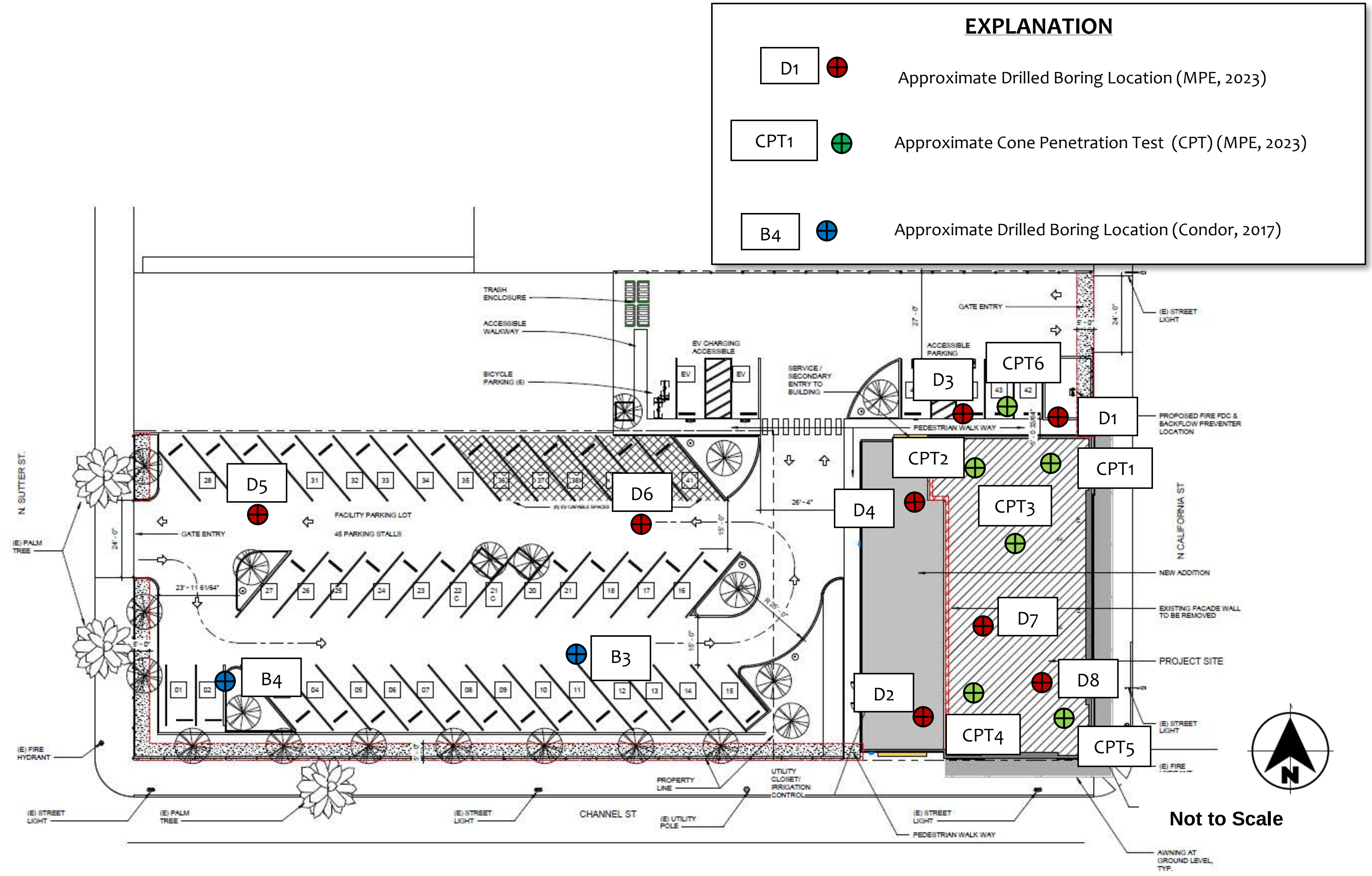
SJCOE CODESTACK ACADEMY

201 N California Street
Stockton, California

FIGURE 1

Date: 05/23

MPE No. 06357-01



NOTES: Adapted from Architechnica *Site Plan*, Sheet A2.1, dated April 9, 2024.

MID PACIFIC ENGINEERING, INC.

SITE PLAN

SJCOE CODESTACK ACADEMY

201 N California Street
Stockton, California

FIGURE 2

Date: 04/24

MPE No. 06357-01

Project: SJCOE Codestack Academy Project Location: 201 N California Street MPE Number: 06357-01	LOG OF SOIL BORING D1 Sheet 1 of 2

Date(s) Drilled	4/13/2023	Logged By	DR	Checked By	DCS
Drilling Method	Hollow Stem Augers	Drilling Contractor	V&W Drilling, Inc.	Total Depth of Drill Hole	51½ Feet
Drill Rig Type	CME-75	Diameter(s) of Hole, inches	7⅝ Inches	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	Initial = 20', Final = 16'	Sampling Method(s)	140 Lb Hammer/30" Drop	Drill Hole Backfill	Neat Cement

Remarks (N) = Converted to SPT Blow Counts

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA			TEST DATA		
				SAMPLE	SAMPLE NUMBER	BLOWS PER FOOT	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
		■	3" Asphalt Concrete (AC) / 9" Aggregate Base (AB)						
		▩	Dark brown, moist, medium stiff, slightly sandy, lean clay (FILL)	▥	D1-1	9 (7)	23.0	88	
	5	▩	light brown, red mottling, soft	▥	D1-2	3 (2)	23.9	78	UCC 0.3 tsf
		▩	brown, light brown, red mottling	▥	D1-3	4 (3)	26.8	86	UCC 0.3 tsf
	10	▩	dark brown, stiff, with AC fragment	▥	D1-4	11 (8)	38.9	58	
		▨	Dark brown, moist, very soft/ soft, slightly sandy, lean clay (CL)	▥	D1-5	2 (2)	33.3	85	UCC 0.2 tsf
	15	▨	▼						
		▨	dark brown, gray, soft	▥	D1-6	4 (3)			
	20	▨	▽						
		▨	dark brown, wet, stiff fine sandy clay	▥	D1-7	17 (13)	19.1	105	UCC 1.0 tsf
		▦	Dark brown, wet, medium dense, clayey fine sand (SC)						
		▨▥	Light brown, wet, stiff, silty lean clay / clayey silt (CL/ML)						
	25	▨▥		▥	D1-8	13 (10)	29.2	90	UCC 1.0 tsf

Project: SJCOE Codestack Academy

Project Location: 201 N California Street

MPE Number: 06357-01

LOG OF SOIL BORING D1

Sheet 2 of 2

Date(s) Drilled	4/13/2023	Logged By	DR	Checked By	DCS
Drilling Method	Hollow Stem Augers	Drilling Contractor	V&W Drilling, Inc.	Total Depth of Drill Hole	51½ Feet
Drill Rig Type	CME-75	Diameter(s) of Hole, inches	7⅝ Inches	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	Initial = 20', Final = 16'	Sampling Method(s)	140 Lb Hammer/30" Drop	Drill Hole Backfill	Neat Cement
Remarks	(N) = Converted to SPT Blow Counts				

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA			TEST DATA		
				SAMPLE	SAMPLE NUMBER	BLOWS PER FOOT	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Light brown, black mottling, wet, very stiff, lean clay (CL)						
	30			▥	D1-9	21 (16)	22.8	102	UCC 1.6 tsf
	35		Brown, orangish brown, wet, dense, clayey fine sand (SC)	▥	D1-10	55 (42)			
	40		Brown, wet, medium dense, slightly clayey, silty fine sand (SM)	▥	D1-11	32 (25)			
			Light brown, wet, stiff/very stiff, lean clay/clayey silt (CL/ML)						
	45			▥	D1-12	18 (14)			
			Orangish brown, wet, medium dense, clayey fine sand / silty fine sand (SC/SM)						
	50		Gray, blueish gray, white cemented veins, wet, very stiff lean clay (CL)	▥	D1-13	40 (31)			

MPE *Mid Pacific Engineering, Inc .*

FIGURE 3

Project: SJCOE Codestack Academy Project Location: 201 N California Street MPE Number: 06357-01	LOG OF SOIL BORING D2 Sheet 1 of 2

Date(s) Drilled	4/13/2023	Logged By	DR	Checked By	DCS
Drilling Method	Hollow Stem Augers	Drilling Contractor	V&W Drilling, Inc.	Total Depth of Drill Hole	51½ Feet
Drill Rig Type	CME-75	Diameter(s) of Hole, inches	7⅝ Inches	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	Initial = 22', Final = 16'	Sampling Method(s)	140 Lb Hammer/30" Drop	Drill Hole Backfill	Neat Cement

Remarks (N) = Converted to SPT Blow Counts

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA			TEST DATA		
				SAMPLE	SAMPLE NUMBER	BLOWS PER FOOT	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			2" Asphalt Concrete (AC) / 4" Aggregate Base (AB)						
			Dark brown, moist, stiff, lean clay with wood fragments (FILL)	▥	D2-1	14 (11)			
	5		with asphalt						
			Dark brown, moist, very stiff, lean clay (CL)	▥	D2-2	22 (17)	22.2	88	
	10		Brown, black mottling, moist, stiff, silty lean clay/ clayey silt (CL/ML)	▥	D2-3	16 (12)	23.4	89	UCC 1.6 tsf
			Brown, moist, stiff, very stiff, lean clay (CL)	▥	D2-4	19 (15)	24.2	88	UCC 0.3 tsf
			Brown, orangish brown, moist, medium dense, silty fine sand (SM)						
	15		Brown, orangish brown, moist, medium dense, silty fine sand / fine sandy silt (SM/ML)	▥	D2-5	22 (17)			
	20		Brown, orangish browN, medium dense, slightly clayey, fine sandy silt (ML)	▥	D2-6	17 (13)	32.1	85	
	25		Light brown, wet, very stiff, silty lean clay / clayey silt (CL/ML)	▥	D2-7	20 (15)	30.1	90	

Mid Pacific Engineering, Inc .

FIGURE 4

Project: SJCOE Codestack Academy

Project Location: 201 N California Street

MPE Number: 06357-01

LOG OF SOIL BORING D2

Sheet 2 of 2

Date(s) Drilled	4/13/2023	Logged By	DR	Checked By	DCS
Drilling Method	Hollow Stem Augers	Drilling Contractor	V&W Drilling, Inc.	Total Depth of Drill Hole	51½ Feet
Drill Rig Type	CME-75	Diameter(s) of Hole, inches	7⅝ Inches	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	Initial = 22', Final = 16'	Sampling Method(s)	140 Lb Hammer/30" Drop	Drill Hole Backfill	Neat Cement

Remarks (N) = Converted to SPT Blow Counts

				SAMPLE DATA			TEST DATA		
ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	BLOWS PER FOOT	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
		▨▥							
		▨	Light brown, black mottling, stiff, lean clay (CL)						
	30	▨	slightly sandy	▥	D2-8	17 (13)	25.3	94	
	35	▩	Brown, orangish brown, wet, medium dense, clayey fine sand (SC)	▥	D2-9	27 (21)			
	40	▨▥	Light brown, wet, very stiff, silty lean clay / clayey silt (CL/ML)	▥	D2-10	20 (15)			
	45	▩	Brown, wet, medium dense, slightly clayey silty fine sand / clayey fine sand (SC)	▥	D2-11	16 (12)			
	50	▨	Light gray, blueish gray, white cemented veins, wet, very stiff, lean clay (CL)	▥	D2-12	26 (20)			

Project: SJCOE Codestack Academy	LOG OF SOIL BORING D3
Project Location: 201 N California Street	
MPE Number: 06357-01	Sheet 1 of 2

Date(s) Drilled: 4/17/2023	Logged By: DR	Checked By: DCS
Drilling Method: Hollow Stem Augers	Drilling Contractor: V&W Drilling, Inc.	Total Depth of Drill Hole: 31½ Feet
Drill Rig Type: CME-75	Diameter(s) of Hole, inches: 7⅝ Inches	Approx. Surface Elevation, ft MSL:
Groundwater Depth [Elevation], feet: 23' Initial, 17' Final	Sampling Method(s): 140 Lb Hammer/30" Drop	Drill Hole Backfill: Neat Cement

Remarks: (N) = Converted to SPT Blow Counts

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE NUMBER	BLOWS PER FOOT	TEST DATA: MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			3" Asphalt Concrete (AC) / 4" Aggregate Base (AB)						
			Dark brown, brown, moist, stiff sandy lean clay with some gravel (FILL)	▥	D3-1	11 (8)	17.4	75	
	5			▥	D3-2	12 (9)			
	10		with red brick fragments	▥	D3-3	11 (8)	28.8	74	
			Brown, moist, loose, silty fine sand (SM)	▥	D3-4	9 (7)	19.2	88	
	15		Dark brown, brown, moist, medium stiff, lean clay (CL) ▼	▥	D3-5	7 (5)	29.7	87	UCC 0.6 tsf
	20				D3-6				
			dark brown, sandy clay ▽						
	25		Light brown, orange mottling, white veins, wet, very stiff, silty lean clay, clayey silt (CL/ML)	▥	D3-7	25 (19)	28.2	89	UCC 1.6 tsf

Project: SJCOE Codestack Academy Project Location: 201 N California Street MPE Number: 06357-01	LOG OF SOIL BORING D3 Sheet 2 of 2

Date(s) Drilled	4/17/2023	Logged By	DR	Checked By	DCS
Drilling Method	Hollow Stem Augers	Drilling Contractor	V&W Drilling, Inc.	Total Depth of Drill Hole	31½ Feet
Drill Rig Type	CME-75	Diameter(s) of Hole, inches	7⅝ Inches	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	23' Initial, 17' Final	Sampling Method(s)	140 Lb Hammer/30" Drop	Drill Hole Backfill	Neat Cement
Remarks	(N) = Converted to SPT Blow Counts				

				SAMPLE DATA			TEST DATA		
ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	BLOWS PER FOOT	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
	30		light brown, wet, very stiff	▥	D3-8	33 (25)	21.1	103	UCC 2.1 tsf
	35								
	40								
	45								
	50								

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FIGURE 5

Project: SJCOE Codestack Academy

Project Location: 201 N California Street, Stockton, CA

MPE Number: 06357-01

LOG OF SOIL BORING D4

Sheet 1 of 2

Date(s) Drilled	4/17/2023	Logged By	DR	Checked By	DCS
Drilling Method	Hollow Stem Augers	Drilling Contractor	V&W Drilling, Inc.	Total Depth of Drill Hole	31½ Feet
Drill Rig Type	CME-75	Diameter(s) of Hole, inches	7⅝ Inches	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	18' Initial, 16' Final	Sampling Method(s)	140 Lb Hammer/30" Drop	Drill Hole Backfill	Neat Cement

Remarks (N) = Converted to SPT Blow Counts

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA			TEST DATA		
				SAMPLE	SAMPLE NUMBER	BLOWS PER FOOT	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
		■	2½" Asphalt Concrete (AC) / 5½ Aggregate Base (AB)						
		▩	Brown, light brown, black, moist, stiff, fine sandy clay (FILL)						
		▩		▥	D4-1	14 (11)			
	5	▩	brown, orangish brown, medium stiff	▥	D4-2	7 (5)	23.6 (8)	78	
		▩		▥	D4-3	6 (5)	23.1	79	UCC 0.4 tsf
	10	▩							
		▩		▥	D4-4	6 (5)	29.7	86	UCC 0.7 tsf
		▨	Dark brown, moist, medium stiff, lean clay (CL)						
	15	▨							
		▨	▼ soft	❖	D4-5				
		▨	▽ wet						
		▨		▥	D4-6	3 (2)			
	20	▨		▥	D4-7	2 (2)			
		▨		❖	D4-8				
	25	▥	Light brown, orange mottling, wet, clayey silt (ML)	▥	D4-9	19 (15)			UCC 1.8 tsf

MPE *Mid Pacific Engineering, Inc .*

FIGURE 6

Project: SJCOE Codestack Academy
Project Location: 201 N California Street, Stockton, CA
MPE Number: 06357-01

LOG OF SOIL BORING D4

Sheet 2 of 2

Date(s) Drilled	4/17/2023	Logged By	DR	Checked By	DCS
Drilling Method	Hollow Stem Augers	Drilling Contractor	V&W Drilling, Inc.	Total Depth of Drill Hole	31½ Feet
Drill Rig Type	CME-75	Diameter(s) of Hole, inches	7⅝ Inches	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	18' Initial, 16' Final	Sampling Method(s)	140 Lb Hammer/30" Drop	Drill Hole Backfill	Neat Cement

Remarks (N) = Converted to SPT Blow Counts

				SAMPLE DATA			TEST DATA		
ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	BLOWS PER FOOT	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
		▥							
	30	▨	Light brown, orangish brown, moist, fine sandy clay (CL)	▥	D4-10	13 (10)	22.4	101	UCC 0.8 tsf
	35								
	40								
	45								
	50								

MPE *Mid Pacific Engineering, Inc .*

FIGURE 6

Project: SJCOE Codestack Academy	LOG OF SOIL BORING D5
Project Location: 201 N California Street, Stockton, CA	
MPE Number: 06357-01	Sheet 1 of 1

Date(s) Drilled	4/17/2023	Logged By	DR	Checked By	DCS
Drilling Method	Hollow Stem Augers	Drilling Contractor	V&W Drilling, Inc.	Total Depth of Drill Hole	20 Feet
Drill Rig Type	CME-75	Diameter(s) of Hole, inches	7⅝ Inches	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	18' Initial, 17' Final	Sampling Method(s)	140 Lb Hammer/30" Drop	Drill Hole Backfill	Neat Cement

Remarks (N) = Converted to SPT Blow Counts

				SAMPLE DATA			TEST DATA		
ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	BLOWS PER FOOT	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
		⊠	Dark brown, moist, clayey gravel (FILL)						
		⊠	Brown, orangish brown, moist, fine sandy clay with fine to coarse gravel and asphalt fragments (FILL)	▥	D5-1	20 (15)	11.4	79	
		⊠		▥	D5-2	22 (17)	18.4	97	
	5	▨	Dark brown, moist, very stiff, lean clay (CL)						
	10	▨	brown, moist, very stiff, fine sandy clay	▥	D5-3	29 (22)			
		▨▥	Light brown, orange mottling, moist, stiff, lean clay / clayey silt (CL/ML)						
	15	▨▥		▥	D5-4	10 (8)	28.5	86	UCC 1.1 tsf
		▨▥	▼ ▽ D5						
	20	▨▥		▥	D5-5	17 (13)	31.5	88	UCC 0.8 tsf
	25								

MPE *Mid Pacific Engineering, Inc .*

FIGURE 7

Project: SJCOE Codestack Academy
Project Location: 201 N California Street, Stockton, CA
MPE Number: 06357-01

LOG OF SOIL BORING D6

Sheet 1 of 1

Date(s) Drilled	4/17/2023	Logged By	DR	Checked By	DCS
Drilling Method	Hollow Stem Augers	Drilling Contractor	V&W Drilling, Inc.	Total Depth of Drill Hole	15 Feet
Drill Rig Type	CME-75	Diameter(s) of Hole, inches	7⅝ Inches	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	Not Encountered	Sampling Method(s)	140 Lb Hammer/30" Drop	Drill Hole Backfill	Neat Cement

Remarks (N) = Converted to SPT Blow Counts

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA			TEST DATA		
				SAMPLE	SAMPLE NUMBER	BLOWS PER FOOT	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Brown, moist, fine sandy clay with brick debris (FILL)						
					D6-1	25 (19)			
			with black asphalt fragments		D6-2	20 (15)	17.1	90	
	5								
			brown, dark brown, with concrete and asphalt fragments		D6-3	13 (10)	21.1	88	
	10								
			Dark brown, moist, medium stiff, lean clay (CL)						
					D6-4	5 (4)	31.2	85	UCC 0.5 tsf
	15								
	20								
	25								

Project: SJCOE Codestack Academy	LOG OF SOIL BORING D7
Project Location: 201 N California Street, Stockton, CA	
MPE Number: 06357-01	Sheet 1 of 1

Date(s) Drilled	4/14/2023	Logged By	DR	Checked By	DCS
Drilling Method	Solid Flight Augers	Drilling Contractor	West Coast Exploration, Inc.	Total Depth of Drill Hole	19½ Feet
Drill Rig Type	Mobile Minute Man	Diameter(s) of Hole, inches	3 Inches	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	9' Initial, 6½' Final	Sampling Method(s)	70 Lb Hammer/30" Drop	Drill Hole Backfill	Neat Cement
Remarks					

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA			TEST DATA		
				SAMPLE	SAMPLE NUMBER	BLOWS PER FOOT	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			4" Concrete Slab (PCC)						
			Dark brown, brown, moist, lean clay with some wood fragments (FILL)						
			Brown, orangish brown, moist, slightly clayey fine sandy silt / silty fine sand (ML/SM)	▤	D7-1	47	17.9	107	UCC 2.2 tsf
				▤	D7-2	50/4"	21.1	104	UCC 2.5 tsf
	5		Brown, black mottling, very moist, slightly clayey, fine sandy silt (ML) ▼	▤	D7-3	45	24.2	100	UCC 1.0 tsf
	10		▽ Brown, black mottling, wet, silty clay / clayey silt (CL/ML)	▤	D7-4	60			
			Brown, orangish brown, wet, clayey fine sand (SC)	▤	D7-5	50/3"			
	15		Light brown, red mottling, wet, silty clay / clayey silt (CL/ML)	▤	D7-6	50/6"			
			with white veins	▤	D7-8	60			
	20								
	25								

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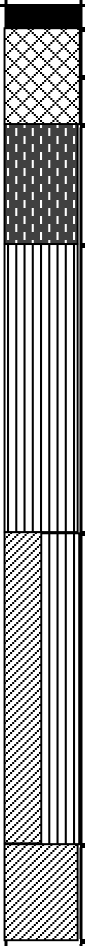
FIGURE 9

Project: SJCOE Codestack Academy
Project Location: 201 N California Street, Stockton, CA
MPE Number: 06357-01

LOG OF SOIL BORING D8

Sheet 1 of 1

Date(s) Drilled	4/14/2023	Logged By	DR	Checked By	DCS
Drilling Method	Solid Flight Augers	Drilling Contractor	West Coast Exploration, Inc.	Total Depth of Drill Hole	19½ Feet
Drill Rig Type	Mobile Minute Man	Diameter(s) of Hole, inches	3 Inches	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	9' Initial, 6½' Final	Sampling Method(s)	70 Lb Hammer/30" Drop	Drill Hole Backfill	Neat Cement
Remarks					



ELEVATION, feet	DEPTH, feet	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA			TEST DATA		
			SAMPLE	SAMPLE NUMBER	BLOWS PER FOOT	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
		4" Concrete Slab (PCC)						
		Brown, orangish brown, moist, silty fine to medium sand (FILL)	▤	D8-1	45	8.4	147	
		Brown, moist, fine sandy gravel (FILL)						
		Brown, orangish brown, moist, cemented, silty fine sand (SM)	▤	D8-2	50/4"	18.0	97	UCC 0.6 tsf
	5	Light brown, brown, moist, slightly clayey, very fine sandy silt (ML) ▼	▤	D8-3	65	25.0	102	UCC 0.9 tsf
	10	▽ wet, white veins	▤	D8-4	40/6"	24.1	100	UCC 0.8 tsf
		Brown, wet, fine sandy clay / clayey silt (CL/ML)	▤	D8-5	70	18.3	113	UCC 1.4 tsf
	15	light brown, reddish veins, wet	▤	D8-6	50/4"			
		Light brown, light gray, white veins, wet, lean clay (CL)	▤	D8-7	63			
	20							
	25							

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FIGURE 10

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS		SYMBOL	CODE	TYPICAL NAMES
COARSE GRAINED SOILS (More than 50% of soil > no. 200 sieve size)	GRAVELS (More than 50% of coarse fraction > no. 4 sieve size)	GW	░	Well graded gravels or gravel - sand mixtures, little or no fines
		GP	▓	Poorly graded gravels or gravel - sand mixtures, little or no fines
		GM	▥	Silty gravels, gravel - sand - silt mixtures
		GC	▧	Clayey gravels, gravel - sand - silt mixtures
	SANDS (50% or more of coarse fraction < no. 4 sieve size)	SW	░	Well graded sands or gravelly sands, little or no fines
		SP	░	Poorly graded sands or gravelly sands, little or no fines
		SM	▓	Silty sands, sand - silt mixtures
		SC	▓	Clayey sands, sand clay mixtures
FINE GRAINED SOILS (More than 50% of soil < no. 200 sieve size)	SILTS & CLAYS LL< 50	ML	▥	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
		CL	▨	Inorganic clays of low to medium plasticity, gravely clays, sandy clays, silty clays, lean clays
		OL	▤	Organic silts and organic silty clays of low plasticity
	SILTS & CLAYS LL ≥ 50	MH	▥	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
		CH	▧	Inorganic clays of high plasticity, fat clays
		OH	▒	Organic clays of medium to high plasticity, organic silty clays, organic silts
HIGHLY ORGANIC SOILS		Pt	▒	Peat and other highly organic soils
ROCK		RX	▓	Rocks, weathered to fresh
FILL		FILL	▩	Artificially placed fill material

OTHER SYMBOLS

▥ = Drive Sample: 2-1/2" O.D. Modified California sampler

▤ = Drive Sample: 3" O.D. Modified California sampler

▧ =SPT Sampler

∇ = Initial Water Level

▼ = Final Water Level

— — — — = Estimated or gradational material change line

——— = Observed material change line

Laboratory Tests
PI = Plasticity Index
EI =Expansive Index
UCC = Unconfined Compression Test
TR = Triaxial Compression Test
GR = Gradation Analysis (Sieve)
K = Permeability Test

GRAIN SIZE CLASSIFICATION

CLASSIFICATION	RANGE OF GRAIN SIZES	
	U.S. Standard Sieve Size	Grain Size in Millimeters
BOULDERS	Above 12"	Above 305
COBBLES	12" to 3"	305 to 76.2
GRAVEL coarse (c) fine (f)	3" to No. 4 3" to 3/4" 3/4" to No. 4	76.2 to 4.76 76.2 to 19.1 19.1 to 4.76
SAND coarse (c) Medium (m) fine (f)	No. 4 to No. 200 No. 4 to No. 10 No. 10 to No. 40 No. 40 to No. 200	4.76 to 0.074 4.76 to 2.00 2.00 to 0.420 0.420 to 0.074
SILT & CLAY	Below No. 200	Below 0.074

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UNIFIED SOIL CLASSIFICATION SYSTEM
SJCOE CODESTACK ACADEMY
201 N. California Street
Stockton, California

FIGURE 11
Date: 04/23
MPE No. 06357-01

PREVIOUS BORING LOGS BY OTHERS

CONDOR EARTH
TECHNOLOGIES, INC.
CONDOR®
209-234-0518
FAX 209-234-0538

LOG OF BORING
No. B-3

PROJECT: Open Window Mixed Use Project - Downtown Stockton **PROJECT NO.:** 7454A

CLIENT: Ten Space Development

PROJECT LOCATION: Downtown Stockton, California

LOCATION: APN: 139-250-03, Northeast corner of Parcel, 50' West of Adjacent **ELEVATION:**

DRILLER: West Coast Exploration **LOGGED BY:** N. Garnica

DRILLING METHOD: Solid-Stem Auger **DATE:** 02/16/17

DEPTH TO - WATER> INITIAL: ▽ - **AFTER DRILLING:** ▼ - **CAVING>** -

Depth (feet)	Sample Type	USCS	Description	Graphic	Sample No.	Blow Counts	N Value	Moisture Content (%)	Dry Density (pcf)	Plasticity Index	Liquid Limit	% < #200	Misc. Tests
0													
	■	CL	Light Brown, Lean Clay with Sand, Moist, Hard	▨	1	22 25 30	37	20.8	96.4				
	◪	CL	Light Brown, Lean Clay with Sand, Moist, Hard	▨	2	15 18 18	36						
5	■	CL	Light Brown, Lean Clay with Sand, Moist, Hard	▨	3	30 50/6	50	20.1	98.8				
10			Boring Terminated at 10 ft. Attempted to go deeper, but encountered concrete on two separate attempts.										
15													
20													
25													
30													
35													

This information pertains only to this boring and should not be interpreted as being indicitive of the site.

CONDOR EARTH TECHNOLOGIES, INC.
209-234-0518
FAX 209-234-0538

LOG OF BORING
No. B-4

PROJECT: Open Window Mixed Use Project - Downtown Stockton **PROJECT NO.:** 7454A
CLIENT: Ten Space Development
PROJECT LOCATION: Downtown Stockton, California
LOCATION: APN: 139-250-03, Southwest Corner of Parcel, 30' Sutter and **ELEVATION:**
DRILLER: West Coast Exploration **LOGGED BY:** N. Garnica
DRILLING METHOD: Solid-Stem Auger **DATE:** 02/17/17
DEPTH TO - WATER> INITIAL: ▽ 25' **AFTER DRILLING:** ▼ - **CAVING>** -

This information pertains only to this boring and should not be interpreted as being indicitive of the site.

Depth (feet)	Sample Type	USCS	Description	Graphic	Sample No.	Blow Counts	N Value	Moisture Content (%)	Dry Density (pcf)	Plasticity Index	Liquid Limit	% < #200	Misc. Tests
0		GM	0-1.0' Gravelly Fill										
		CL	Brown, Gravelly Lean Clay, Moist, Medium Stiff	1.0	1	12 12 8	13	17.7	97.5				
		CL	Brown, Sandy Lean Clay with Gravel, Moist, Medium Stiff		2	10 7 7	14						
5		SM	Light Brown, Silty Sand, Moist, Medium Stiff	5.0	3	9 8 8	11	12.6	88.1				
10		CL	Light Brown, Sandy, Lean Clay, Moist, Hard	10.0	4	15 20 22	42						
15		CL	Light Brown, Lean Clay, Moist, Hard		5	30 30 30	40	23.6	91.9				
20		CL	Olive, Lean Clay, Moist, Very Stiff		6	10 12 18	30						
25	▽												
30		CL	Grayish Green, Lean Clay, Wet, Very Stiff*		7	8 8 10	18						
35			Grayish Green, Sandy, Lean Clay, Wet Very Stiff*		8	15 17	30						

** Samples had a smell of gasoline starting at about 30' down to 50'.*

CONDOR EARTH TECHNOLOGIES, INC.
209-234-0518
FAX 209-234-0538

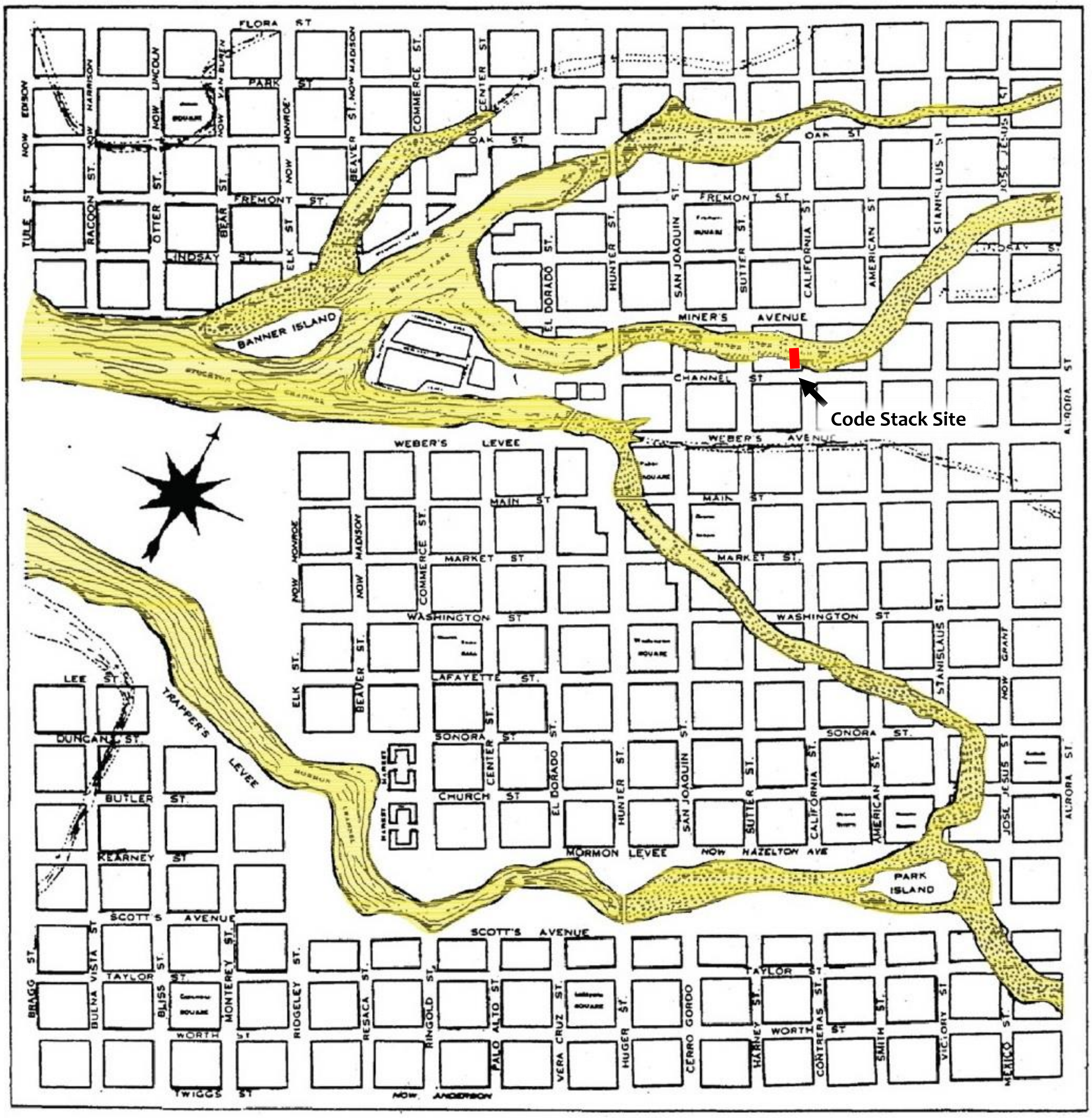
LOG OF BORING
No. B-4

PROJECT:	Open Window Mixed Use Project - Downtown Stockton	**PROJECT NO.:**	7454A
CLIENT:	Ten Space Development		
PROJECT LOCATION:	Downtown Stockton, California		
LOCATION:	APN: 139-250-03, Southwest Corner of Parcel, 30' Sutter and	**ELEVATION:**	
DRILLER:	West Coast Exploration	**LOGGED BY:**	N. Garnica
DRILLING METHOD:	Solid-Stem Auger	**DATE:**	02/17/17
DEPTH TO - WATER> INITIAL:	▽ 25'	**AFTER DRILLING:** ▼ -	**CAVING>** -

Depth (feet)	Sample Type	USCS	Description	Graphic	Sample No.	Blow Counts	N Value	Moisture Content (%)	Dry Density (pcf)	Plasticity Index	Liquid Limit	% < #200	Misc. Tests
	■			▨		28							
40	◩	CL	Grayish Green, Lean Clay, Wet, Very Stiff*	▨	9	10 12 15	27						
45	■	CL	Grayish Green, Lean Clay, Wet, Very Stiff*	▨	10.0	20 20 20	27						
50			Boring Terminated at 50 ft.										
55													
60													
65													
70													

** Samples had a smell of gasoline starting at about 30' down to 50'.*

This information pertains only to this boring and should not be interpreted as being indicitive of the site.



NOTES: Adapted from Figure 4, Condor Earth , Report No. 7454A, dated April 2017.

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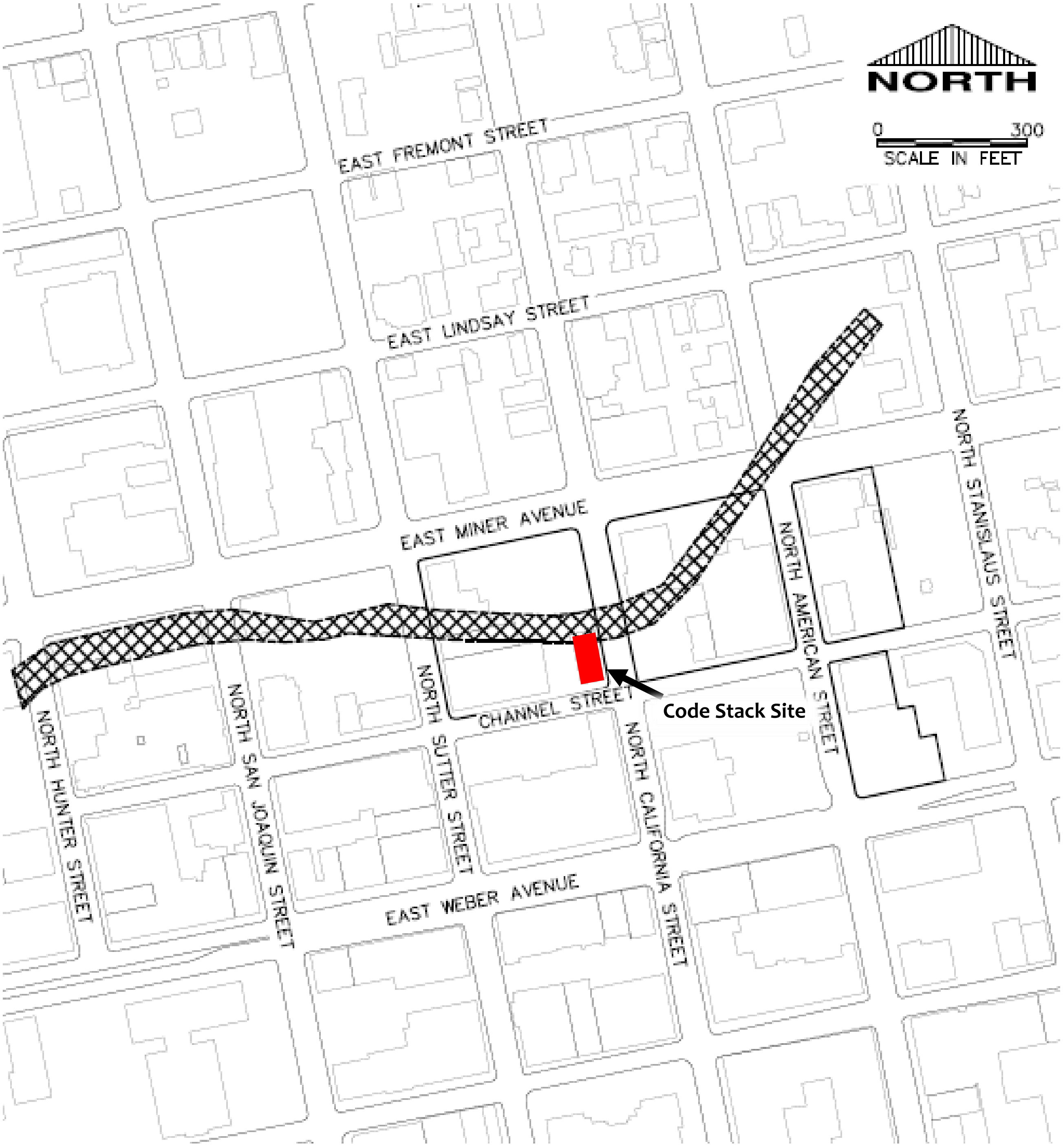
HISTORIC MINER CHANNEL - OVERALL EXTENT
SJCOE CODE STACK
201 N. California Street
Stockton, California

FIGURE 14

Date: 4/24

MPE No. 06357-01



LEGEND

▨ = APPROXIMATE FORMER MINER CHANNEL BOUNDARY

☐ = APPROXIMATE PROJECT BOUNDARY

APPROXIMATE BOUNDARIES OF FORMER MINER CHANNEL DIGITIZED FROM 1895 SANBORN FIRE INSURANCE MAPS
BUILDING AND EDGE OF STREET BOUNDARIES FROM CITY OF STOCKTON GIS DEPARTMENT

NOTES: Adapted from Figure 4, Condor Earth , Report No. 7454A, dated April 2017.

MPE
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HISTORIC MINER CHANNEL
SJCOE CODE STACK
201 N. California Street
Stockton, California

FIGURE 15

Date: 4/24

MPE No.06357-01

APPENDIX A

APPENDIX A

A. GENERAL INFORMATION

The performance of a Geotechnical Engineering Investigation for the proposed Codestack building renovation project to be constructed at 201 North California Street in Stockton, California, was authorized by Tim Dearborn, AIA, of Architechnica on February 24, 2023. Authorization was for an investigation as described in our proposal letter of December 13, 2022, sent to Mr. Dearborn with Architechnica whose mailing address is 555 W Benjamin Holt Drive, Suite 423, Stockton, CA 95207; email: tim@architechnica.net and phone number: 209-952-5850.

B. FIELD EXPLORATION

On April 17, 2023, six (6) soil borings were drilled at the approximate locations indicated on Figure 3, utilizing a CME-75 truck-mounted drill rig equipped with 7-5/8-inch O.D hollow-stem augers to the maximum depth of 51½ feet below ground surface (bgs).

At various intervals, relatively undisturbed soil samples were recovered with a 2½-inch O.D., 2-inch I.D. Modified California sampler (ASTM D3550), or with a 2-inch O.D., 1⅜-inch I.D. SPT sampler (ASTM D1586) driven by a 140-pound hammer freely falling 30 inches. The number of blows of the hammer required to drive the 18-inch long sampler each 6-inch interval was recorded with the sum of the blows required to drive the sampler the lower 12-inch interval, or portion thereof, being designated the penetration resistance or "blow count" for that particular drive.

The samples obtained with the modified California sampler were retained in 2-inch diameter by 6-inch long, thin-walled brass tubes contained within the sampler. Immediately after recovery, the field engineer visually classified the soil in the tubes or SPT- sampler. The ends of the tubes were sealed to preserve the natural moisture contents. Disturbed bulk samples of the surface materials also were obtained at various locations and depths. Soil samples were taken to our laboratory for additional classification (ASTM D2488) and selection of samples for testing.

On April 14, 2023, two (2) additional borings were performed in the basement area of the building using a Mobile Minute-Man portable drill rig equipped with 3-inch diameter, solid flight augers. Borings were drilled to a maximum depth of 19½ feet below basement slab level. At various intervals, relatively undisturbed soil samples were recovered with a 2½-inch O.D., 2-inch I.D. Modified California sampler (ASTM D3550), hand driven by a 70-pound hammer.

On October 31, 2023, six (6) Cone Penetration Tests (CPTs) were performed to a maximum depth of approximately 50 feet below basement grade and 60 feet below exterior site grades.

The Logs of Soil Borings, Figures 3 through 10, contain descriptions of the soils encountered in each boring. A Boring Legend explaining the Unified Soil Classification System and the symbols used on the logs is contained on Figure 11. Logs of previous borings by others are presented on Figures 12 and 13. The Logs of CPTs with Soil Behavior Charts and seismic velocity results are contained in the CPT report in Appendix D of the report. The approximate locations of borings and CPTs are indicated on Figure 2.

C. LABORATORY TESTING

Selected undisturbed samples of the soils were tested to determine dry unit weight (ASTM D2937), natural moisture content (ASTM D2216), and unconfined compressive strength (ASTM 2166). The results of these tests are included on the boring logs at the depth each sample was obtained.

Six samples of near-surface soils were submitted to Sunland Analytical in Rancho Cordova, California, for corrosivity testing in accordance with No. 643 (Modified Small Cell), CT 532, CT 422, and CT 417. The analytical results are presented in the text of the report.

APPENDIX B

APPENDIX B

GUIDE EARTHWORK SPECIFICATIONS

SJCOE CODE STACK ACADEMY
201 N. California Street
Stockton, California

PART 1: GENERAL

1.1 SCOPE

A. General Description

This item shall include clearing of all surface and subsurface structures associated with previous development of the site, existing structures, septic systems, leach lines, concrete slabs, foundations, asphalt concrete, utilities to be relocated or abandoned including all associated backfill, trees, demolition debris, rubbish, rubble, rubbish and associated items; preparation of surfaces to be filled, filling, spreading, compaction, observation and testing of the fill; and all subsidiary work necessary to complete the grading of the building areas to conform with the lines, grades and slopes as shown on the accepted Drawings.

B. Related Work Specified Elsewhere

1. Trenching and backfilling for sanitary sewer system: Section ______.
2. Trenching and backfilling for storm drain system: Section ______.
3. Trenching and backfilling for underground water, natural gas, and electric supplies: Section ______.

C. Geotechnical Engineer

Where specific reference is made to "Geotechnical Engineer" this designation shall be understood to include either him or his representative.

1.2 PROTECTION

A. Adequate protection measures shall be provided to protect workers and passers-by at the site. Streets and adjacent property shall be fully protected throughout the operations.

B. In accordance with generally accepted construction practices, the Contractor shall be solely and completely responsible for working conditions at the job site, including safety of all persons and property during performance of the work. This requirement shall apply continuously and shall not be limited to normal working hours.

C. Any construction review of the Contractor's performance conducted by the Geotechnical Engineer is not intended to include review of the adequacy of the Contractor's safety measures, in, on or near the construction site.

D. Adjacent streets and sidewalks shall be kept free of mud, dirt or similar nuisances resulting from earthwork operations.

E. Surface drainage provisions shall be made during the period of construction in a manner to avoid creating a nuisance to adjacent areas.

F. The site and adjacent influenced areas shall be watered as required to suppress dust nuisance.

1.3 GEOTECHNICAL REPORT

A. A Geotechnical Engineering Report (MPE No. 06357-01; dated April 18, 2024) has been prepared for this site by Mid Pacific Engineering, Inc., Geotechnical Engineers. A copy is available for review at the office of Mid Pacific Engineering, Inc., 840 Embarcadero Drive, Suite 20, West Sacramento, California 95605.

B. The information contained in this report was obtained for design purposes only. The Contractor is responsible for any conclusions he/she may draw from this report; should the Contractor prefer not to assume such risk, he/she should employ their own experts to analyze available information and/or to make additional borings upon which to base their conclusions, all at no cost to the Owner.

1.4 EXISTING SITE CONDITIONS

The Contractor shall be acquainted with all site conditions. If unshown active utilities are encountered during the work, the Architect shall be promptly notified for instructions. Failure to notify will make the Contractor liable for damage to these utilities arising from Contractor's operations subsequent to the discovery of such unshown utilities.

1.5 SEASONAL LIMITS

Fill material shall not be placed, spread or rolled during unfavorable weather conditions. When the work is interrupted by heavy rains, fill operations shall not be resumed until field tests indicate that the moisture contents of the subgrade and fill materials are satisfactory.

PART 2: PRODUCTS

2.1 MATERIALS

A. All fill shall be of approved local materials from required excavations, supplemented by imported fill, if necessary. Approved local materials are defined as local soils with a maximum particle size of approximately three inches (3"); free from significant quantities of rubble, rubbish and vegetation; and, having been tested and approved by the Geotechnical Engineer prior to use.

B. Imported fill materials shall be approved by the Geotechnical Engineer; shall meet the above requirements; shall have plasticity indices not exceeding fifteen (15), when tested in accordance with ASTM D4318; shall have a maximum Expansion Index not exceeding twenty (20) when tested in accordance with ASTM D4829; and, shall be of three-inch (3") maximum particle size. Import fill shall be clean of contamination with appropriate documentation. All imported materials shall be approved by the Geotechnical Engineer prior to being transported to the site.

C. Asphalt concrete, aggregate base, aggregate subbase, and other paving products shall comply with the appropriate provisions of the *State of California (Caltrans) Standard Specifications* Standards, latest editions.

PART 3: EXECUTION

3.1 LAYOUT AND PREPARATION

Lay out all work, establish grades, locate existing underground utilities, set markers and stakes, set up and maintain barricades and protection of utilities-all prior to beginning actual earthwork operations.

3.2 CLEARING, GRUBBING AND PREPARING BUILDING PADS AND PAVEMENT AREAS

A. The site shall be cleared of existing structures designated for removal including but not limited to, foundations, slabs-on-grade, exterior flatwork, pavements, utilities to be relocated or abandoned including all associated backfill, demolition debris, rubbish, rubble and other unsuitable materials. Subsurface utilities to be relocated or abandoned shall be removed from within and to at least five feet beyond the perimeter of the proposed structural areas; remaining piping beyond the structure that is not removed shall be plugged. Excavations and depressions resulting from the removal of such items, as well as any existing excavations or loose soil deposits, as determined by the Geotechnical Engineer, shall be cleaned out to firm, undisturbed soil and backfilled with suitable materials in accordance with these specifications.

B. Subgrades shall be sub-excavated in depth and lateral extent, as required by the Geotechnical Engineer.

C. The upper twelve inches (12") of soil subgrades within areas of removed flatwork, pavements, and utilities as well as sub-excavated and disturbed areas shall be ripped and cross-ripped to expose any remaining remnants, roots, rubble and debris. All exposed rubble and debris shall be removed from the subgrades. Hand picking of exposed rubble and debris shall be performed by the Contractor to adequately clear the grades.

D. The surfaces upon which fill is to be placed, as well as at-grade areas or areas achieved by excavation, shall be plowed or scarified to a depth of at least twelve inches (12") until the surface is free from ruts, hummocks or other uneven features which would tend to prevent uniform compaction by the selected equipment.

E. Subgrade preparation and compaction shall extend at least five feet (5') beyond the proposed structure lines, or as required by the Geotechnical Engineer based on the exposed soil and site conditions.

F. When the moisture content of the subgrade is below that required to achieve the specified density, and that minimum content recommended in the geotechnical report, water shall be added until the proper moisture content is achieved.

G. When the moisture content of the subgrade is too high to permit the specified compaction to be achieved, the subgrade shall be aerated by blading or other methods until the moisture content is satisfactory for compaction.

H. After the foundations for fill have been cleared, plowed or scarified, they shall be disced or bladed until uniform and free from large clods, brought to the proper moisture content and compacted to not less than ninety percent (90%) for all structural areas of the maximum dry density as determined by the ASTM D1557-91 Compaction Test. Soils compaction shall be performed using a heavy, self-propelled sheepsfoot compactor (Caterpillar 815 or equivalent size compactor) capable of providing compaction to the full depth of soils scarification/ripping. Compaction operations shall be performed in the presence of the Geotechnical Engineer who will evaluate the performance of the materials under compactive load. Unstable soil deposits, as determined by the Geotechnical Engineer, shall be excavated to expose a firm base and grades restored with engineered fill in accordance with these specifications and the Geotechnical Engineering Report.

3.3 PLACING, SPREADING AND COMPACTING FILL MATERIAL

a. The selected soil fill material shall be placed in layers which when compacted shall not exceed six inches (6") in thickness. Each layer shall be spread evenly and shall be thoroughly mixed during the spreading to promote uniformity of material in each layer.

b. When the moisture content of the fill material is below that required to achieve the specified density, water shall be added until the proper moisture content of at least the optimum is achieved.

c. When the moisture content of the fill material is too high to permit the specified degree of compaction to be achieved, the fill material shall be aerated by blading or other methods until the moisture content is satisfactory.

d. After each layer has been placed, mixed and spread evenly, it shall be thoroughly compacted to at least ninety percent (90%) of the ASTM D1557 maximum dry density. Compaction shall be undertaken with a heavy, self-propelled sheepsfoot compactor (Caterpillar 815 or equivalent size compactor) capable of achieving the specified density and shall be accomplished while the fill material is at the required moisture content. Each layer shall be compacted over its entire area until the desired density has been obtained.

e. The filling operations shall be continued until the fills have been brought to the finished slopes and grades as shown on the accepted Drawings.

3.5 FINAL SUBGRADE PREPARATION

The upper twelve inches (12") of final building pad subgrades and all final subgrades supporting pavement sections shall be brought to a uniform moisture content, and shall be uniformly compacted to not less than:

building pad & flatwork	90%
pavement areas	95%

regardless of whether final subgrade elevations are attained by filling, excavation or are left at existing grades.

Flatwork shall be underlain by eighteen inches (18") of Class 2 Aggregate Base compacted to at least ninety percent (90%) of ASTM D1557 maximum dry density.

3.6 TRENCH BACKFILL

Utility trench backfill shall be placed in lifts of no more than six inches (6") in compacted thickness. Each lift shall be compacted to at least ninety percent (90%) compaction, as defined by ASTM D1557, except that backfill supporting sidewalks, streets or other public pavement shall be compacted to comply with applicable County of Sacramento Standards, latest editions. The upper twelve inches in pavement areas, the minimum compaction should be ninety-five (95%) percent of ASTM D1557. If lime-treated, the upper 12 inches of trench backfill (or to the depth of the treatment, whichever is deeper) should consist of ninety-five percent (95%) compacted Class 2 Aggregate Base material.

3.7 TESTING AND OBSERVATION

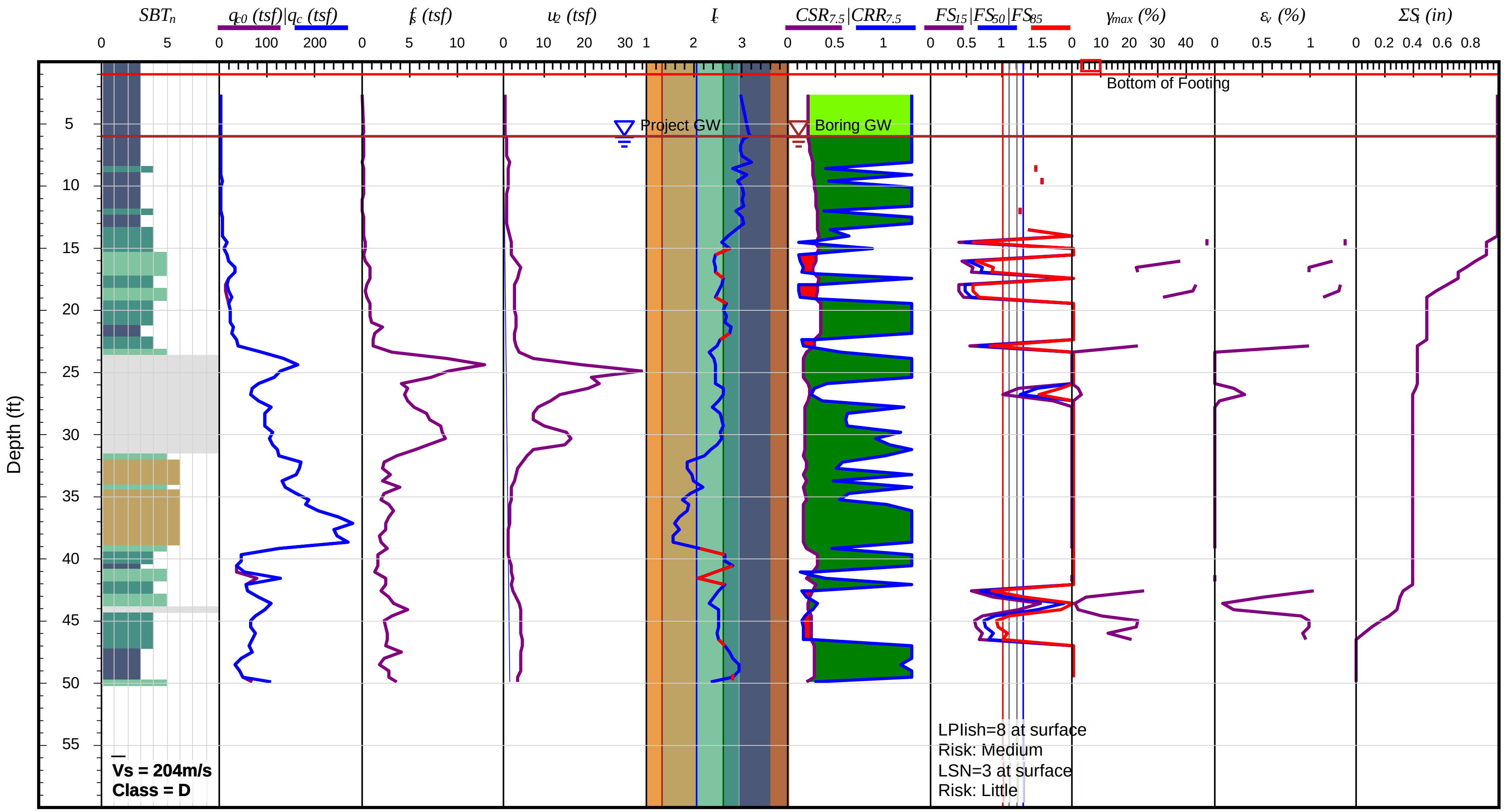
a. Grading operations shall be observed by the Geotechnical Engineer, serving as the representative of the Owner.

b. Field density tests shall be made by the Geotechnical Engineer after compaction of each layer of fill. Additional layers of fill shall not be spread until the field density tests indicate that the minimum specified density has been obtained.

c. Earthwork shall not be performed without the notification or approval of the Geotechnical Engineer. The Contractor shall notify the Geotechnical Engineer at least two (2) working days prior to commencement of any aspect of the site earthwork.

d. If the Contractor should fail to meet the technical or design requirements embodied in this document and on the applicable plans, he shall make the necessary readjustments until all work is deemed satisfactory, as determined by the Geotechnical Engineer and the Architect/Engineer. No deviation from the specifications shall be made except upon written approval of the Geotechnical Engineer or Architect/Engineer.

APPENDIX C



🟥 Sensitive fine grained
🟫 Organic soils - peats
🟦 Clay to silty clay
🟩 Silty clay to clayey silt

🟩 Sandy silt to silty sand
🟨 Silty sand to clean sand
🟧 Dense sand to gravelly sand
⬛ Clayey sand to very stiff sand

⬜ Very stiff fine grained *

* Overconsolidated or cemented

Silt Correction:
UCLA method

Earthquake & Groundwater Information:
Magnitude = 5.5
Max. Acceleration = 0.393 g
Project GW = 6 ft
Maximum Settlement = 0.99 in
Settl. at Bottom of Footing = 0.99 in

Liquefaction: Boulanger & Idriss (2010-16)
Settl.: [dry] Yi (2022); [sat] Idriss & Boulanger (2008)
Lateral spreading: Idriss & Boulanger (2008)
M correction: [Sand; Clay] Boulanger & Idriss(2004)
σv correction: Idriss & Boulanger (2008)
Stress reduction: Idriss & Boulanger (2008)

MPE
MID PACIFIC ENGINEERING, INC.

Seismic Settlement Potential - CPT Data

Project:	Code Stack Academy Renovation Project				
Location:	201 N. California Street, Stockton, California				
Project No.:	06357-01	CPT No.:	CPT-1	Figure:	C-3

R:\06300-00 through 06399-00\06357-01 - SJCOE Codestack Academy - GER\4 - GER & Correspondence\Data & Analysis\GeoSuite\GeoSuite_06357-01_CPT-1.csv

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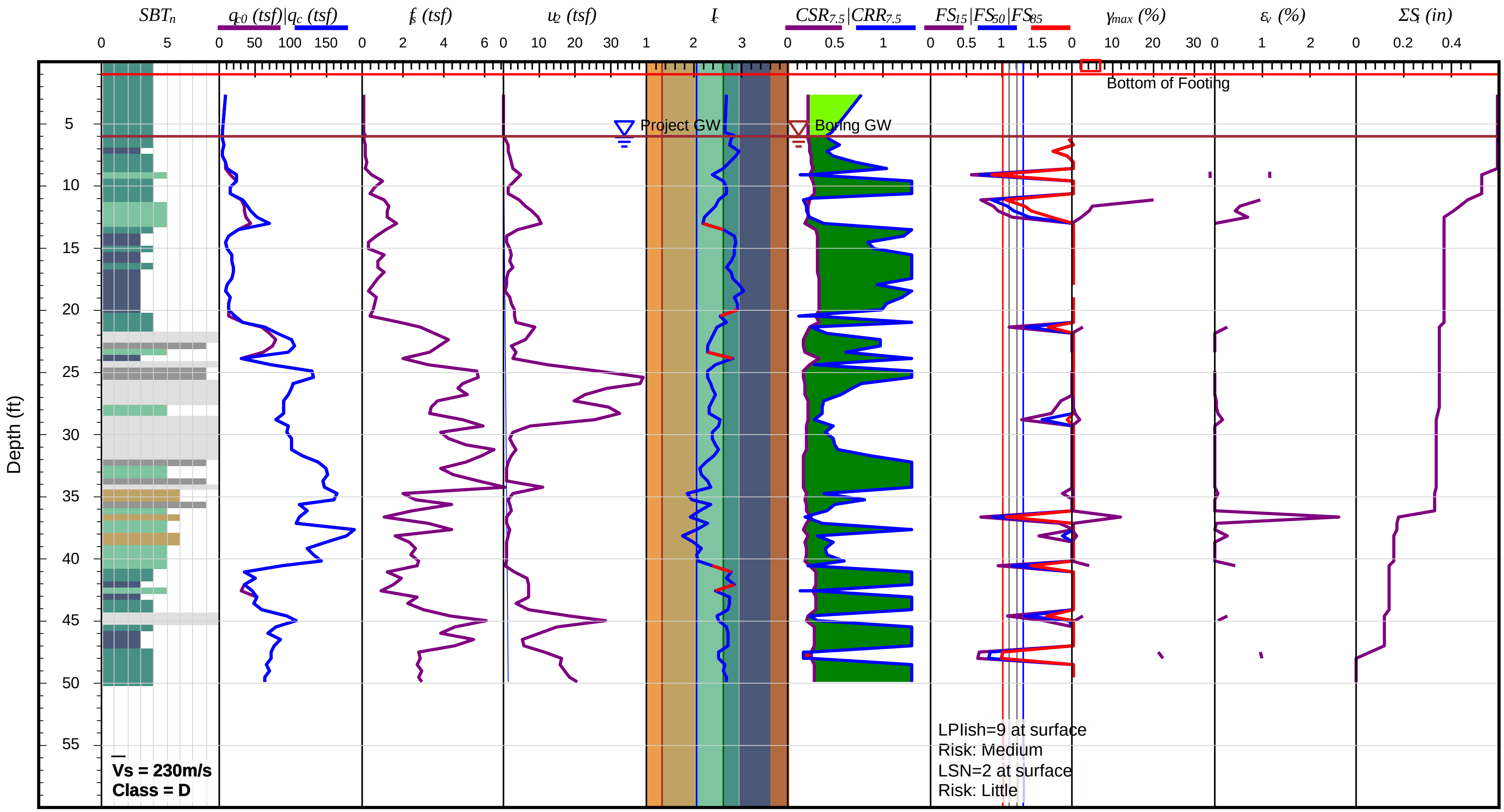

Vs = 230m/s
Class = D

LPIish=9 at surface
Risk: Medium
LSN=2 at surface
Risk: Little

🟥 Sensitive fine grained
🟫 Organic soils - peats
⬛ Clay to silty clay
🟩 Silty clay to clayey silt
🟩 Sandy silt to silty sand
🟨 Silty sand to clean sand
🟧 Dense sand to gravelly sand
⬜ Clayey sand to very stiff sand
⬜ Very stiff fine grained *

* Overconsolidated or cemented

Silt Correction:
UCLA method

Earthquake & Groundwater Information:
Magnitude = 5.5
Max. Acceleration = 0.393 g
Project GW = 6 ft
Maximum Settlement = 0.59 in
Settl. at Bottom of Footing = 0.59 in

Liquefaction: Boulanger & Idriss (2010-16)
Settl.: [dry] Yi (2022); [sat] Idriss & Boulanger (2008)
Lateral spreading: Idriss & Boulanger (2008)
M correction: [Sand; Clay] Boulanger & Idriss(2004)
σv correction: Idriss & Boulanger (2008)
Stress reduction: Idriss & Boulanger (2008)

MPE
MID PACIFIC ENGINEERING, INC.

Seismic Settlement Potential - CPT Data

Project:	Code Stack Academy Renovation Project				
Location:	201 N. California Street, Stockton, California				
Project No.:	06357-01	CPT No.:	CPT-2	Figure:	C-4

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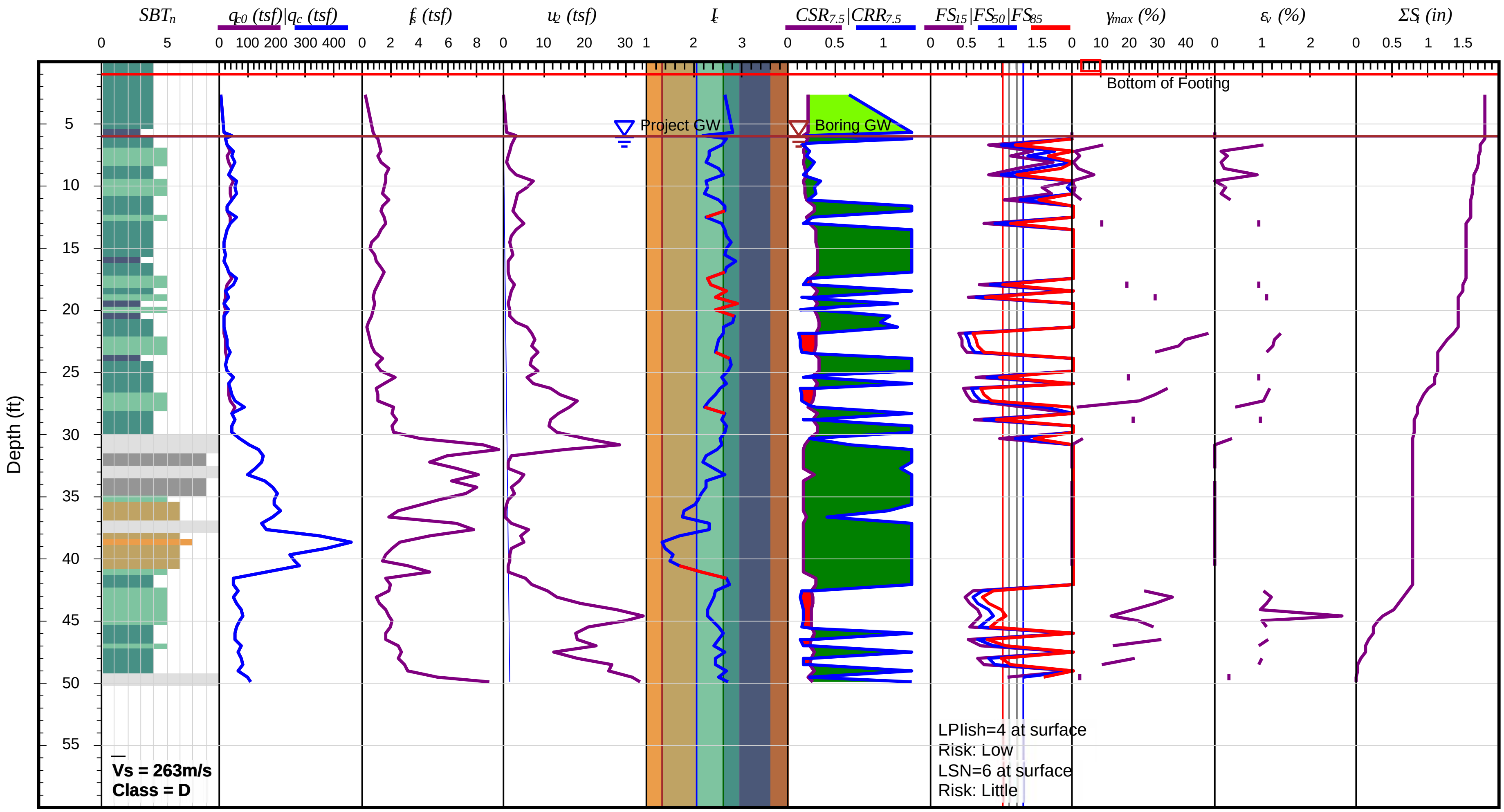

🟥 Sensitive fine grained
🟫 Organic soils - peats
⬛ Clay to silty clay
🟩 Silty clay to clayey silt

🟩 Sandy silt to silty sand
🟨 Silty sand to clean sand
🟧 Dense sand to gravelly sand
⬜ Clayey sand to very stiff sand

⬜ Very stiff fine grained *

* Overconsolidated or cemented

Silt Correction:
UCLA method

Earthquake & Groundwater Information:
Magnitude = 5.5
Max. Acceleration = 0.393 g
Project GW = 6 ft
Maximum Settlement = 1.80 in
Settl. at Bottom of Footing = 1.80 in

Liquefaction: Boulanger & Idriss (2010-16)
Settl.: [dry] Yi (2022); [sat] Idriss & Boulanger (2008)
Lateral spreading: Idriss & Boulanger (2008)
M correction: [Sand; Clay] Boulanger & Idriss(2004)
σv correction: Idriss & Boulanger (2008)
Stress reduction: Idriss & Boulanger (2008)

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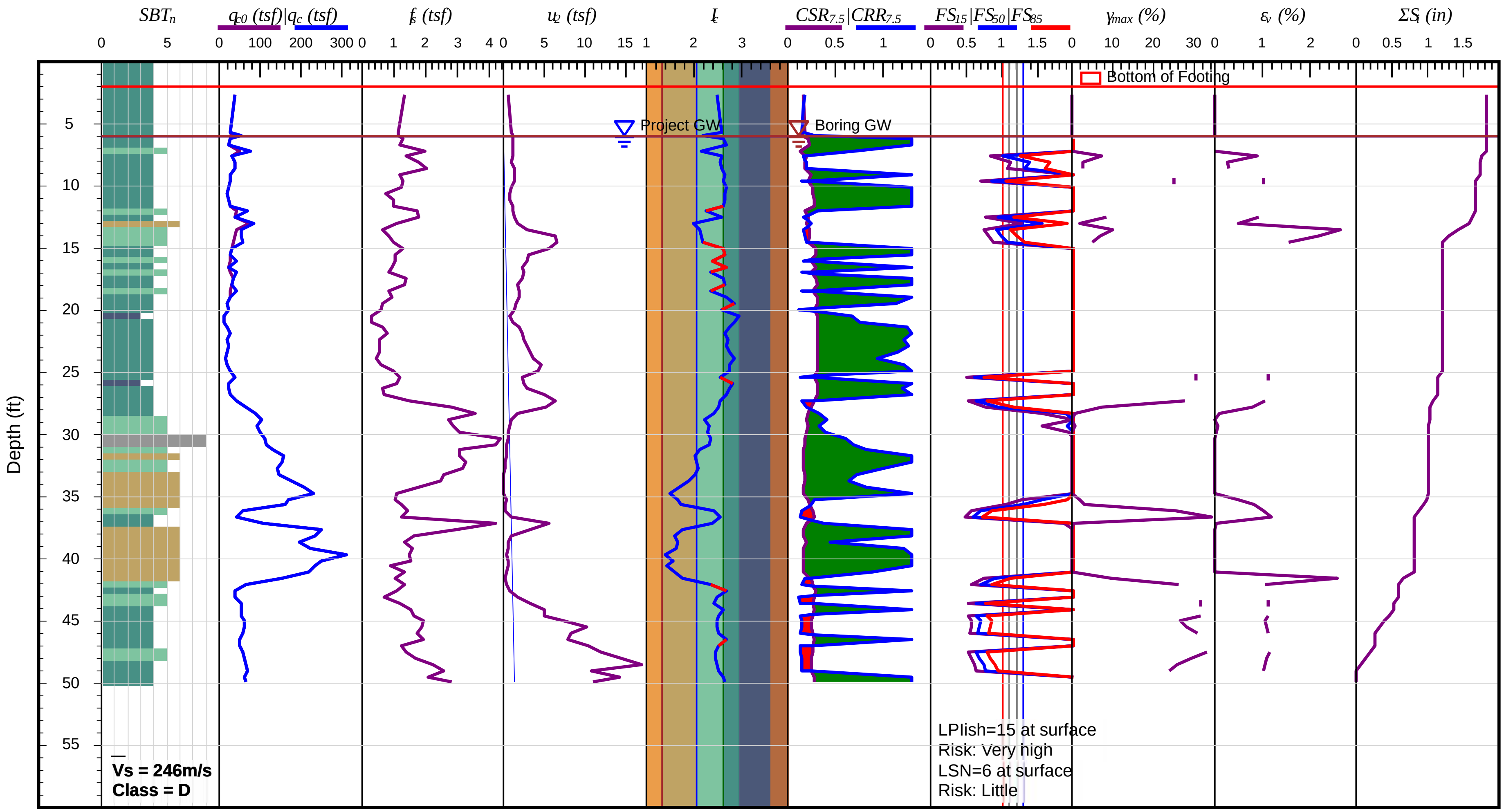
MID PACIFIC ENGINEERING, INC.

Seismic Settlement Potential - CPT Data

Project:	Code Stack Academy Renovation Project				
Location:	201 N. California Street, Stockton, California				
Project No.:	06357-01	CPT No.:	CPT-03	Figure:	C-5



🟥 Sensitive fine grained
🟫 Organic soils - peats
⬛ Clay to silty clay
🟩 Silty clay to clayey silt

🟩 Sandy silt to silty sand
🟨 Silty sand to clean sand
🟧 Dense sand to gravelly sand
⬜ Clayey sand to very stiff sand

⬜ Very stiff fine grained *

* Overconsolidated or cemented

Silt Correction:
UCLA method

Earthquake & Groundwater Information:
Magnitude = 5.5
Max. Acceleration = 0.393 g
Project GW = 6 ft
Maximum Settlement = 1.81 in
Settl. at Bottom of Footing = 1.81 in

Liquefaction: Boulanger & Idriss (2010-16)
Settl.: [dry] Yi (2022); [sat] Idriss & Boulanger (2008)
Lateral spreading: Idriss & Boulanger (2008)
M correction: [Sand; Clay] Boulanger & Idriss(2004)
σv correction: Idriss & Boulanger (2008)
Stress reduction: Idriss & Boulanger (2008)

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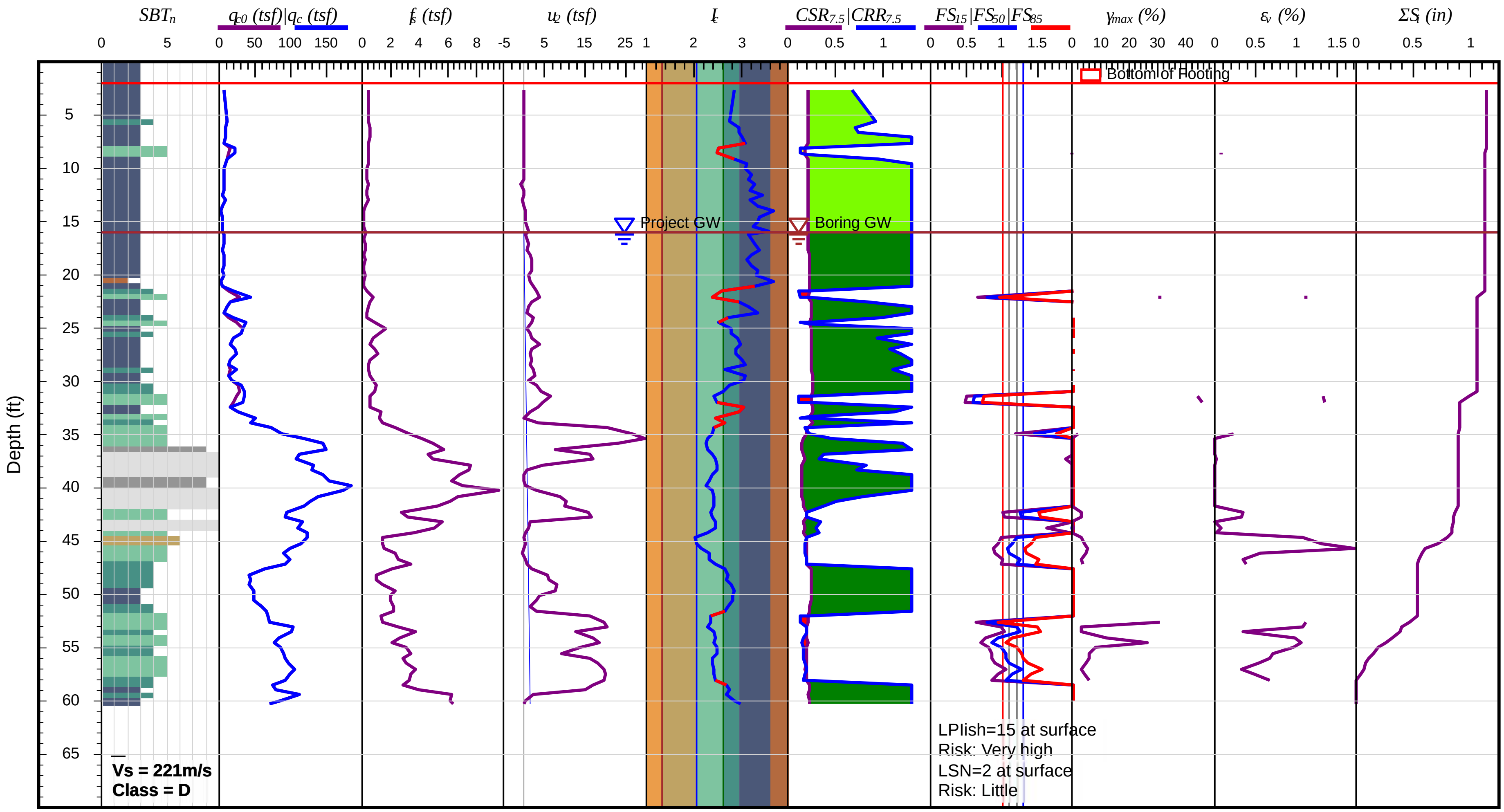
MPE
MID PACIFIC ENGINEERING, INC.

Seismic Settlement Potential - CPT Data

Project: Code Stack Academy Renovation Project
Location: 201 N. California Street, Stockton, California
Project No.: 06357-01 CPT No.: CPT-05 Figure: C-6



🟥 Sensitive fine grained
🟫 Organic soils - peats
⬛ Clay to silty clay
🟩 Silty clay to clayey silt

🟩 Sandy silt to silty sand
🟨 Silty sand to clean sand
🟧 Dense sand to gravelly sand
⬛ Clayey sand to very stiff sand

⬜ Very stiff fine grained *

* Overconsolidated or cemented

Silt Correction:
UCLA method

Earthquake & Groundwater Information:
Magnitude = 5.5
Max. Acceleration = 0.393 g
Project GW = 16 ft
Maximum Settlement = 1.13 in
Settl. at Bottom of Footing = 1.13 in

Liquefaction: Boulanger & Idriss (2010-16)
Settl.: [dry] Yi (2022); [sat] Idriss & Boulanger (2008)
Lateral spreading: Idriss & Boulanger (2008)
M correction: [Sand; Clay] Boulanger & Idriss(2004)
σv correction: Idriss & Boulanger (2008)
Stress reduction: Idriss & Boulanger (2008)

MPE
MID PACIFIC ENGINEERING, INC.

Seismic Settlement Potential - CPT Data

Project:	Code Stack Academy Renovation Project				
Location:	201 N. California Street, Stockton, California				
Project No.:	06357-01	CPT No.:	CPT-06	Figure:	C-7

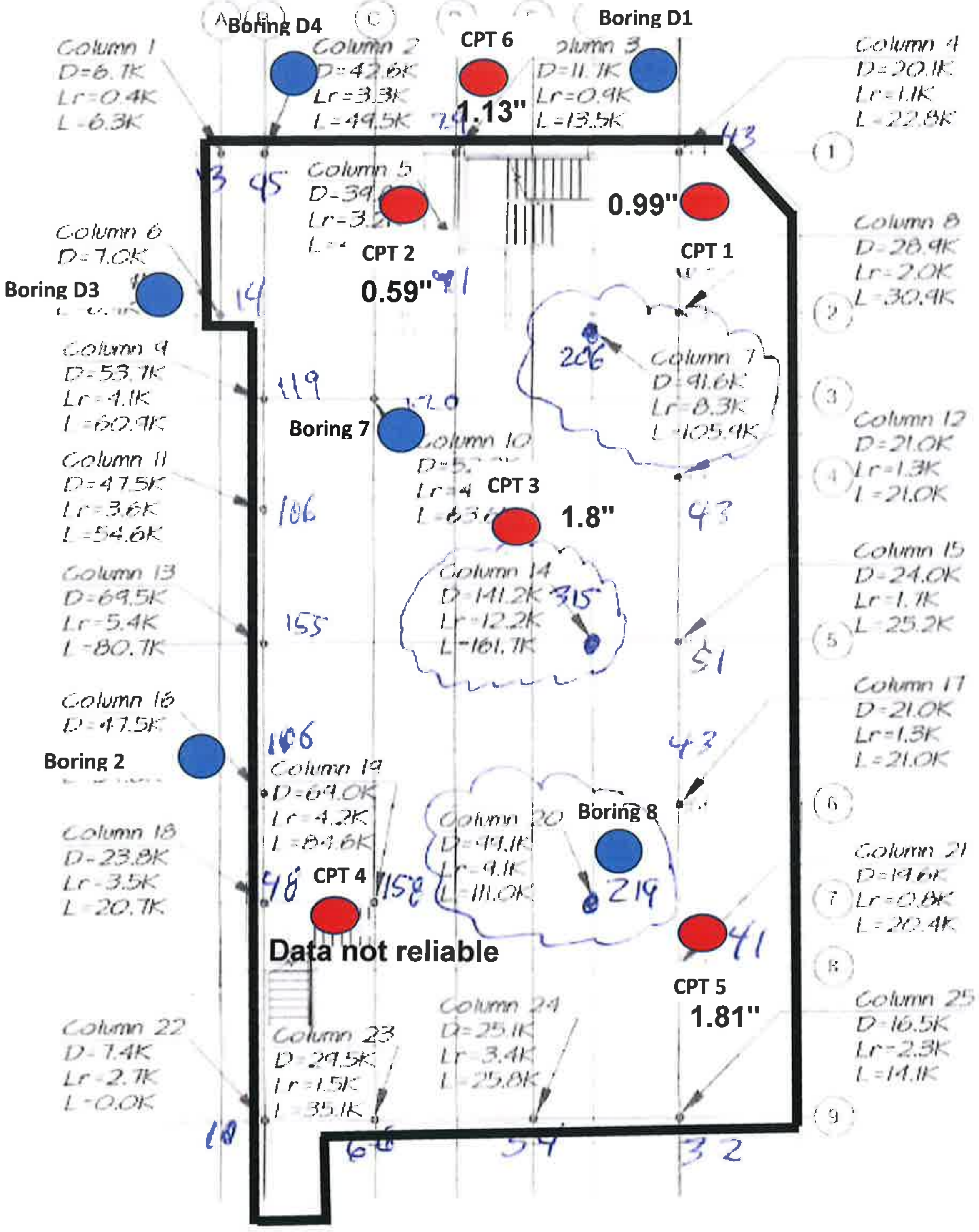
R:\06300-00 through 06399-00\06357-01 - SJCOE Codestack Academy - GER\4 - GER & Correspondence\Data & Analysis\GeoSuite\GeoSuite_06357-01_CPT-6.csv

GeoSuite© Version 4.0.0.2. Developed by Fred Yi, PhD, PE, GE, F. ASCE

Prepared at 11/16/2023 4:02:03 PM

🔵 **Test Boring**

🔴 **CPT** (Total Seismic Settlement (inches))



Column Load Summary at Foundation

THEORY AND METHODOLOGY OF LIQUEFACTION AND SEISMIC SETTLEMENT

LIQUEFACTION POTENTIAL

Liquefaction is a process in which strong ground shaking causes saturated soils to lose their strength and behave as a fluid. Ground failure associated with liquefaction can result in severe damage to structures. Soil types susceptible to liquefaction include sand, silty sand, sandy silt and silt, as well as soils having a plasticity index (PI) less than 7 (Boulanger and Idriss, 2006). Loose soils with a PI less than 12 and moisture content greater than 85 percent of the liquid limit are also susceptible to liquefaction (Bray and Sancio, 2006). For sandy soils, the geologic conditions for increased susceptibility to liquefaction are: 1) shallow groundwater (generally less than 50 feet in depth), 2) the presence of unconsolidated sandy alluvium, typically Holocene in age, and 3) strong ground shaking. All three of these conditions must be present for liquefaction to occur.

For clayey soils, recent studies indicate that deposits of clays and plastic silts (i.e., cohesive soils) have also experienced failure during earthquakes (Idriss and Boulanger, 2008). This kind of failure is called cyclic softening. "The term cyclic softening is used in reference to strength loss and deformation in clays and plastic silts, while the term liquefaction is used in reference to strength loss and deformation in saturated sands and other cohesionless soils. As such, the terms cyclic softening and liquefaction can also be used in reference to the engineering procedures that have been developed for these respective soil types" (Idriss and Boulanger, 2008).

Liquefaction potential can usually be evaluated based on the SPT, CPT or shear wave velocity data and using the simplified procedure described by Seed and Idriss (1971, 1982), Seed and others (1985), modified in the 1996 National Center for Earthquake Engineering Research (NCEER) and 1998 NCEER/National Science Foundation (NSF) workshops (Youd and Idriss, 2001), and as recently summarized by Idriss and Boulanger (2008). The method of evaluating liquefaction potential consists of comparing the cyclic stress ratio (CSR) developed in the soil by the earthquake motion to cyclic resistance ratio (CRR), which will cause liquefaction of the soil for a given number of cycles. In the simplified procedure, the CSR developed in the soil is calculated from a formula that incorporates ground surface acceleration, total and effective stresses in the soil at different depths (which in turn are related to the location of the groundwater table), non-rigidity of the soil column and a number of simplifying assumptions.

For sandy soils, the CRR that will cause liquefaction is related to the relative density of the soil, expressed in terms of SPT blowcounts (N1)60 (Seed and Idriss, 1982; Seed and others, 1985; Youd and Idriss, 2001; Idriss and Boulanger, 2008), cone penetration resistance (qc1N) (Robertson and Wride, 1998; Youd and Idriss, 2001; Idriss and Boulanger, 2008) or shear wave velocity (Vs1) (Andrus and Stokoe, 2000; Youd and Idriss, 2001; Andrus and others, 2004), all normalized for an effective overburden pressure of 1 ton per square foot and corrected to equivalent clean sand resistance. For clayey soils, the CRR is related to cyclic undrained shear strength ratio, s_u/σ_{vc}' (Idriss and Boulanger, 2008). All of these methods are incorporated into a liquefaction and seismic settlement program, GeoSuite©, version 2.4 (Yi, 2018).

SEISMIC SETTLEMENT

Prediction of seismic-induced settlement is also very important. Seismic-induced settlement includes settlement that occurs both in dry sands and saturated sands (California Geological Survey, 2008). Severe seismic shaking may cause dry sands to densify, resulting in settlement expressed at the ground surface. Seismic settlement in dry soils generally occurs in loose sands and silty sands, with cohesive and fine-grained soils being less prone to significant settlement. For saturated soils, significant settlement is anticipated if the soils exhibit liquefaction during seismic shaking.

The methods for evaluating seismic settlement in saturated sands can generally be classified into two groups. The method for the first group was developed during the 1970s and 1980s, generally based on the relationship between cyclic stress ratio, $(N_1)_{60}$, and volumetric strain (Silver and Seed, 1971; Lee and Albaisa, 1974; and Tokimatsu and Seed, 1987). The method for the second group was developed in the early 1990s with the paper by Ishihara and Yoshimine (1992) as the first publication in the category, modified and improved by various researchers (Robertson and Wride, 1998; Yoshimine et al., 2006; Idriss and Boulanger, 2008; and Yi, 2010), and is generally based on the relationship between volumetric strain and the factor of safety for liquefaction. Idriss and Boulanger (2008) modified the methods to incorporate both SPT and CPT data. Yi (2010) modified the methods to incorporate shear wave velocity data.

Research related to the estimation of dry sand settlement during earthquake excitation was initiated in the early 1970s by Silver and Seed (1971), followed by the works of several researchers (Seed and Silver, 1972; Pyke et al., 1975; Tokimatsu and Seed, 1987; and Pradel, 1998). A simplified method of evaluating earthquake-induced settlements in dry, sandy soils based on the Tokimatsu and Seed procedure has been developed by Pradel (1998) and is recommended by Martin and Lew (1999) as one of the standard methods for the estimation of earthquake-induced settlements of dry sands in California.

In recent years, serious research was performed by the University of California, Los Angeles (Duku et al. 2008; Yee et al. 2014; Stewart, 2014), and a new volumetric strain material model (VSMM) was proposed. The new UCLA VSMM was developed based on a series of laboratory test results and is able to consider the effects of overburden pressure, fines contents and degree of saturation. This new model was utilized for a new based-isolated new hospital, Loma Linda University Medical Center Campus Transformation Project, and approved by California's Office of Statewide Health Planning and Development (OSHPD). All of these methods generally utilize SPT data. Utilizing the test results of Silver and Seed (1971), Yi extended the application of the procedures for both CPT (Yi, 2010a) and V_s data (Yi, 2010b, 2010c). These methods are also incorporated into a liquefaction and seismic settlement program, GeoSuite©, version 2.5 (Yi, 2020).

SURFACE MANIFESTATION OF LIQUEFACTION

Ishihara (1985) published a paper containing observations on the protective effect that an upper layer of non-liquefied material had against the manifestation of liquefaction at the ground surface. The paper contained graphs that plotted thickness of the upper non-liquefied layer (H_1) and the thickness of underlying liquefied material (H_2). The maximum acceleration is 400 to 500 gal in Ishihara's graph. The term "surface manifestation" is utilized to describe liquefaction-induced surface damage.

A quantitative method using an index called the liquefaction potential index (LPI) was developed and presented by Iwasaki (1978, 1982). The LPI is defined as:

$$\mathrm{LPI} = \int_0^{20} \mathrm{F_1\,W(z)dz}$$

where $W(z) = 10 – 0.5z$, $F_1 = 1 - FS$ for $FS < 1.0$, $F_1 = 0$ for $FS > 1.0$ and z is the depth below the ground surface in meters. The LPI presents the risk of liquefaction damage as a single value with the following indicators of liquefaction-induced damage:

LPI Range and Damage	
LPI Range	Damage
LPI = 0	Liquefaction risk is very low.
0 < LPI ≤ 5	Liquefaction risk is low.
5 < LPI ≤ 10	Liquefaction risk is medium.
10 < LPI ≤ 15	Liquefaction risk is high.
LPI > 15	Liquefaction risk is very high.

The original liquefaction potential index (LPI) was improved by Maurer et al (2015) by assessing liquefaction hazard utilizing the Ishihara (1985) boundary curves for liquefaction surface effects. The new index is named Ishihara-inspired index, $\mathrm{LPI}_{\mathrm{ISH}}$.

$$\mathrm{LPI}_{ISH} = \int_0^{20} F(FS) \frac{25.56}{z} \mathrm{dz}$$

where

$$F(FS) = \begin{cases} 1 - FS \; if \; FS \leq 1 \;\cap\; H_1 \cdot m(FS) \leq 3 \\ 0 \; otherwise \end{cases}$$

and

$$m(FS) = \exp\left(\frac{5}{25.56(1 - FS)}\right) - 1$$

The most recent development for quantitative descriptions of liquefaction-induced surface damage, called "liquefaction vulnerability," was made by Tonkin & Taylor (2013) after the Christchurch earthquakes occurred between 2010 and 2011 and was based on field observations and analyses of approximately 7,500 cone penetrometer test (CPT) investigations. A new index, the liquefaction severity number (LSN), was proposed and defined as:

$$\mathrm{LSN} = \int \frac{\varepsilon_{\mathrm{v}}}{\mathrm{z}} \mathrm{dz}$$

where ε_v is the calculated volumetric densification strain in the subject layer from Zhang et al. (2002) and z is the depth to the layer of interest in meters below the ground surface. The typical behaviors of sites with a given LSN are summarized in following table.

LSN Ranges and Observed Land Effects	
LSN Range	**Predominant Performance**
0 – 10	Little to no expression of liquefaction, minor effects
10 – 20	Minor expression of liquefaction, some sand boils
20 – 30	Moderate expression of liquefaction, with sand boils and some structural damage
30 – 40	Moderate to severe expression of liquefaction, settlement can cause structural damage
40 – 50	Major expression of liquefaction, undulations and damage to ground surface, severe total and differential settlement of structures
>50	Severe damage, extensive evidence of liquefaction at surface, severe total and differential settlements affecting structures, damage to services

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APPENDIX D

Gregg Drilling CPT Report

CONE PENETRATION TESTING (CPT) REPORT

Gregg Drilling LLC

Prepared for: Mid Pacific Engineering
Project D2239070
November 3, 2023

Prepared by: Eleni Pateras
epateras@greggdrilling.com

Table of Contents

November 3, 2023

Mid Pacific Engineering, Inc.
Attn: Daniel Rivera

Subject: CPT Site Investigation
SJCOE Codestack
Stockton, CA
GREGG Project Number: D2239070

Dear Daniel:

The following report presents the results of Gregg Drilling'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	Groundwater Samples	(GWS)	☐
5	Soil Samples	(SS)	☐
6	Vapor Samples	(VS)	☐

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 me at 562-427-6899.

Sincerely,

CPT Reports Team
Gregg Drilling, LLC.

Cone Penetration Testing (CPT) Procedure

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 5cm 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.

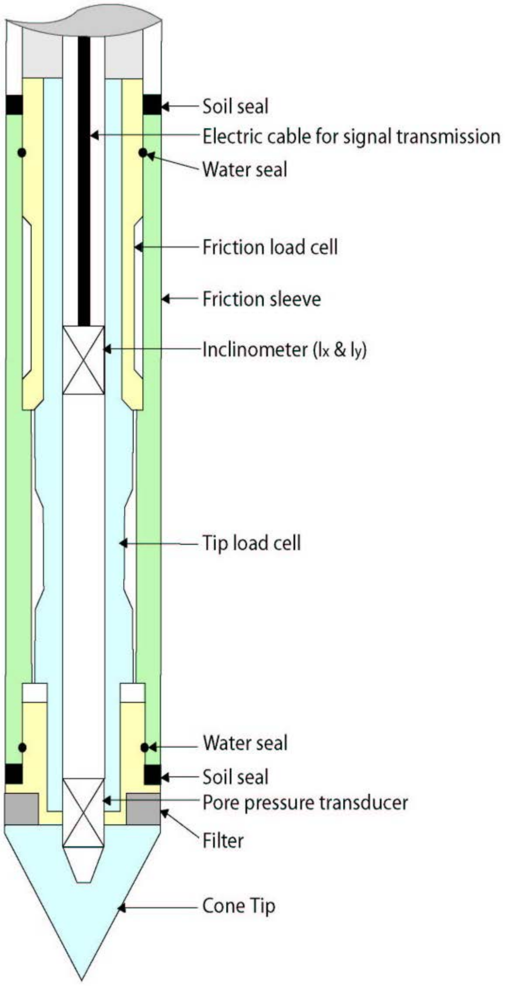


Figure CPT

15cm^2 Standard Cone Specifications

Dimensions	
Cone base area	15 cm^2
Sleeve surface area	225 cm^2
Cone net area ratio	0.85
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)

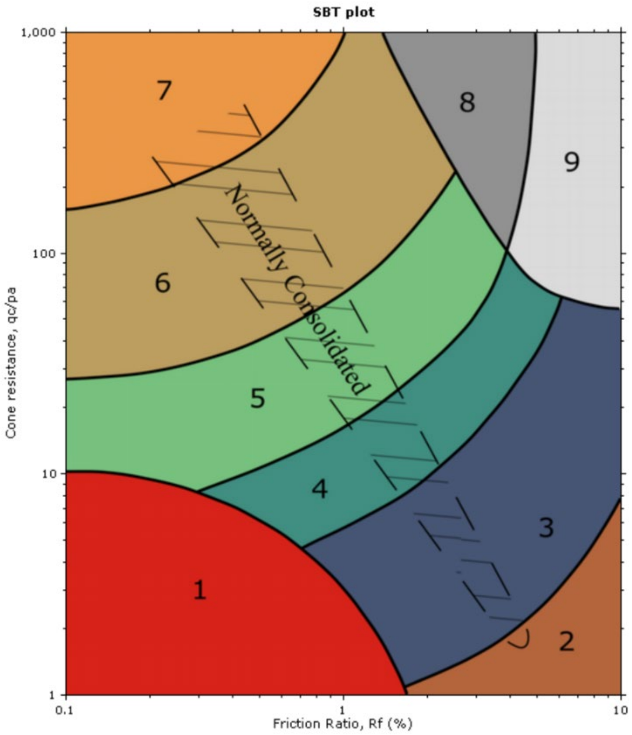
Note: The repeatability during field use will depend somewhat on ground conditions, abrasion, maintenance and zero load stability.

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 (2010) (Figure SBT). Typical plots display SBT based on the non-normalized charts of Robertson (2010) or normalized data (2009 and 2016). For CPT soundings deeper than 30m, we recommend the use of the normalized charts of Robertson (2009 and 2016) which can be displayed as SBTn. The report also includes spreadsheet output of computer calculations of basic interpretation in terms of SBT and SBTn 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 Robertson and Cabal (Guide to Cone Penetration Testing 7th Edition, 2022). The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed. Gregg Drilling LLC does not warrant 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.



SBTn legend

🟥 1. Sensitive fine grained
🟫 2. Organic material
🟦 3. Clay to silty clay
🟩 4. Clayey silt to silty clay
🟩 5. Silty sand to sandy silt
🟫 6. Clean sand to silty sand
🟧 7. Gravelly sand to sand
⬛ 8. Very stiff sand to clayey sand
⬜ 9. Very stiff fine grained

Figure SBT (After Robertson 2010) – Note: Colors may vary slightly compared to plots

Pore Pressure Dissipation Tests (PPDTs)

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.

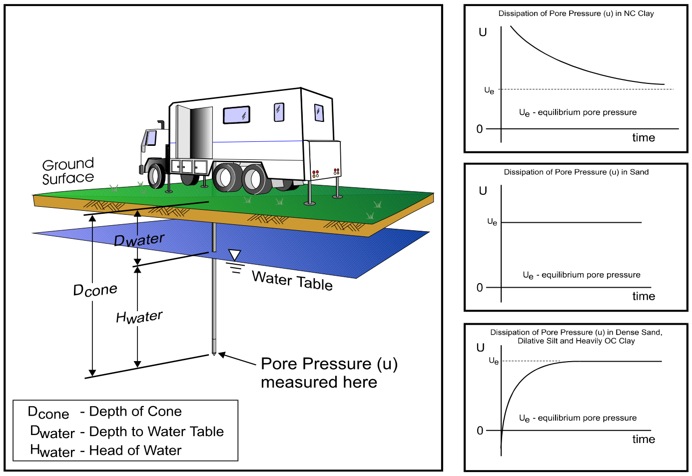
Pore pressure dissipation data can be interpreted to provide estimates of:

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

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 completed for this project is included in Table 1.



Water Table Calculation

$$D_{water} = D_{cone} - H_{water}$$

where $H_{water} = U_e$ (depth units)

Useful Conversion Factors:
1psi = 0.704m = 2.31 feet (water)
1tsf = 0.958 bar = 13.9 psi
1m = 3.28 feet

Figure PPDT

Seismic Cone Penetration Tests (SCPT)

Seismic Cone Penetration Testing (SCPT) can be conducted at various intervals during the Cone Penetration Test. Shear wave velocity (Vs) 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 Drilling'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 = Δd/Δt

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

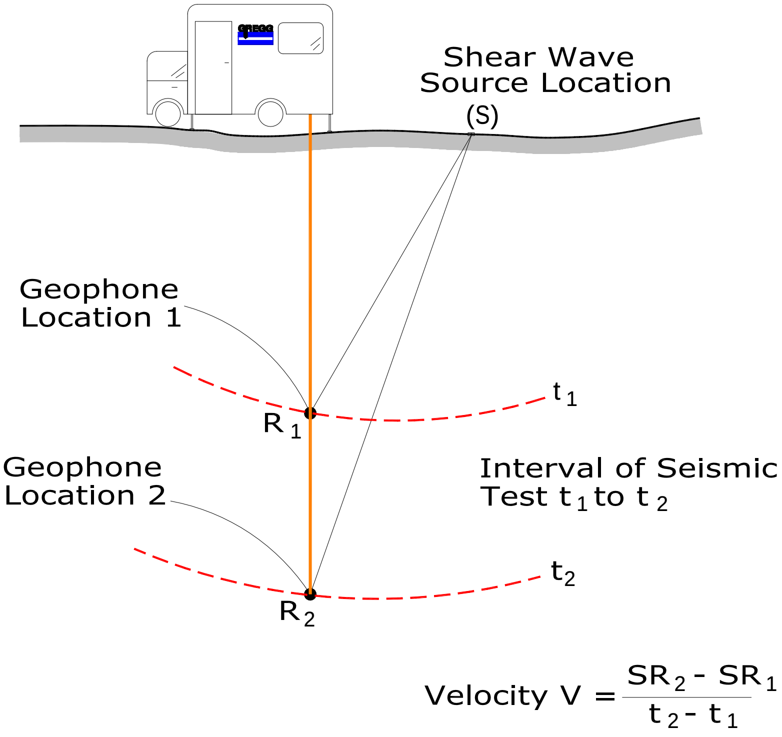


Figure SCPT

Soil Sampling

Gregg Drilling 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.

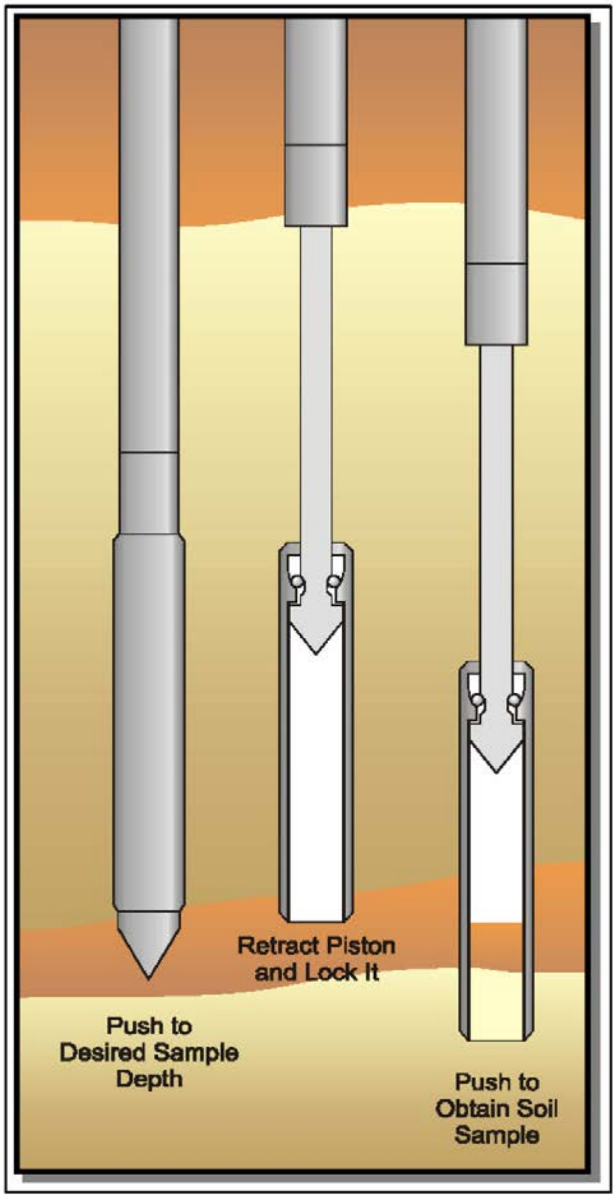


Figure SS

Ground Water Sampling

Gregg Drilling 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.

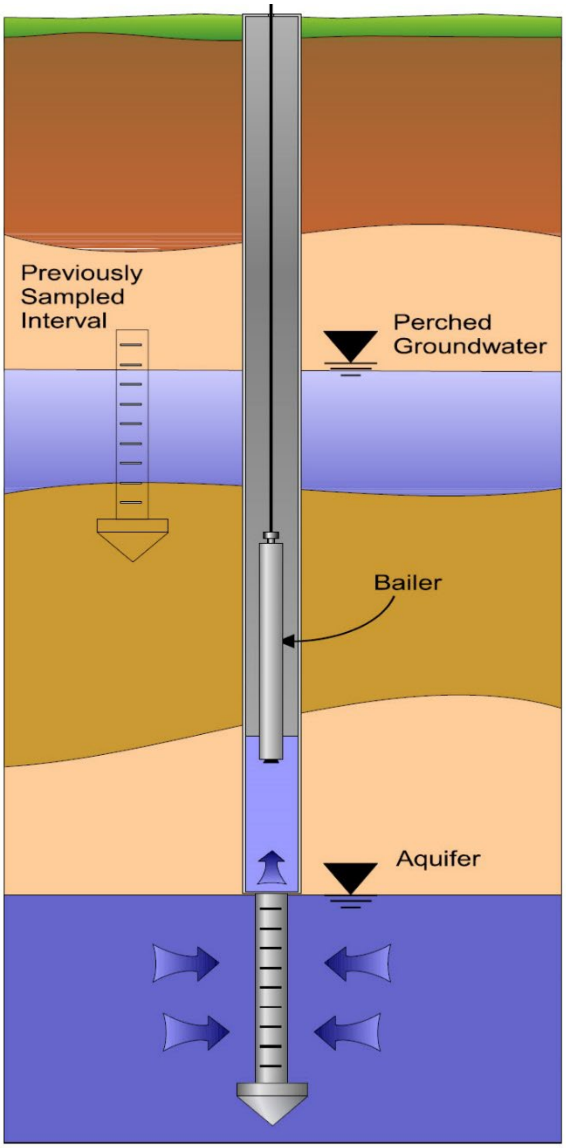


Figure GWS

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

References

Lunne, T., Robertson, P.K. and Powell, J.J.M., "Cone Penetration Testing in Geotechnical Practice" E & FN Spon. ISBN 0 419 23750, 1997

Roberston, P.K., "Soil Classification using the Cone Penetration Test", Canadian Geotechnical Journal, Vol. 27, 1990 pp. 151-158.

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

Table 1: Cone Penetration Testing Summary

CPT Sounding Identification	Date	Termination Depth (ft)	Depth of Soil Samples (ft)	Depth of Groundwater Samples (ft)	Depth of Pore Pressure Dissipation Tests (ft)
CPT-1	10/30/2023	50.20	-	-	33.79
CPT-2	10/31/2023	50.20	-	-	-
CPT-3	10/31/2023	50.20	-	-	36.42
CPT-4	10/31/2023	22.64	-	-	-
SCPT-5	11/2/2023	50.20	-	-	-
SCPT-6	11/2/2023	60.37	-	-	46.10

APPENDIX A:

CPT PLOTS

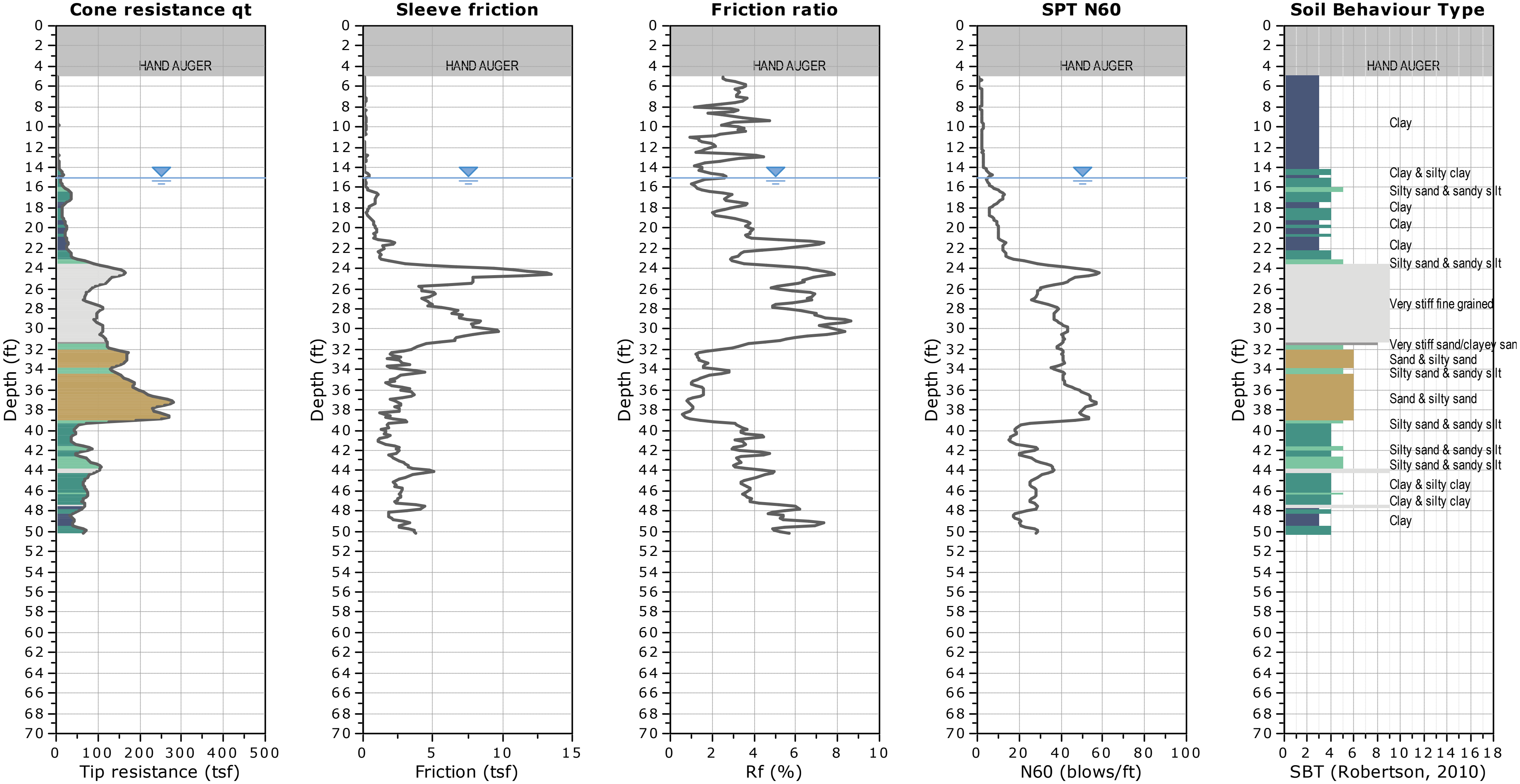
GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-1

FIELD REP: DANIEL RIVERA
Cone ID: GDC-97

CLIENT: MID PACIFIC ENGINEERING
SITE: SJCOE CODESTACK, STOCKTON, CA

Total depth: 50.20 ft, Date: 10/30/2023

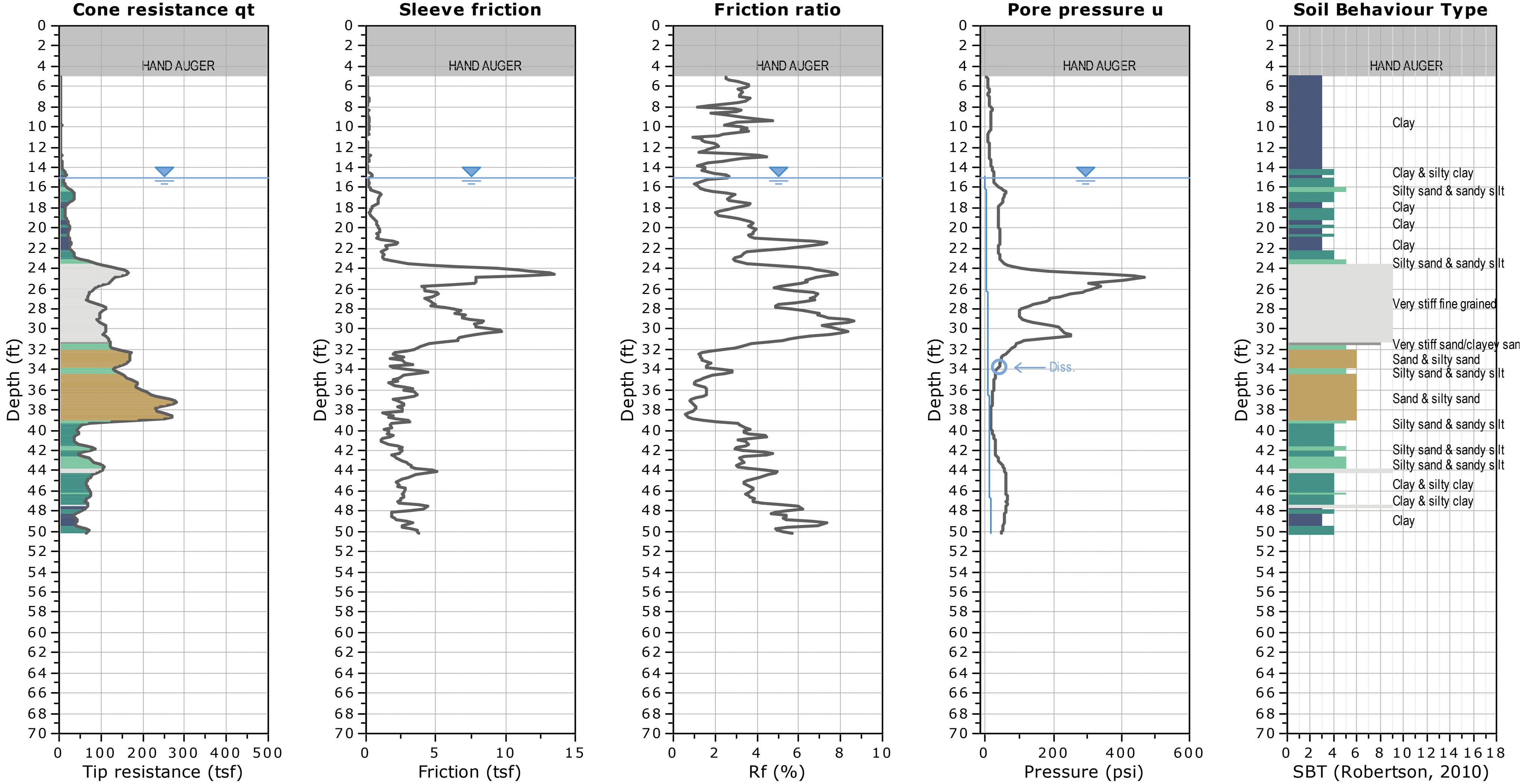


GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-1

CLIENT: MID PACIFIC ENGINEERING
SITE: SJCOE CODESTACK, STOCKTON, CA

FIELD REP: DANIEL RIVERA
Cone ID: GDC-97
Total depth: 50.20 ft, Date: 10/30/2023



WATER TABLE FOR ESTIMATING PURPOSES ONLY

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

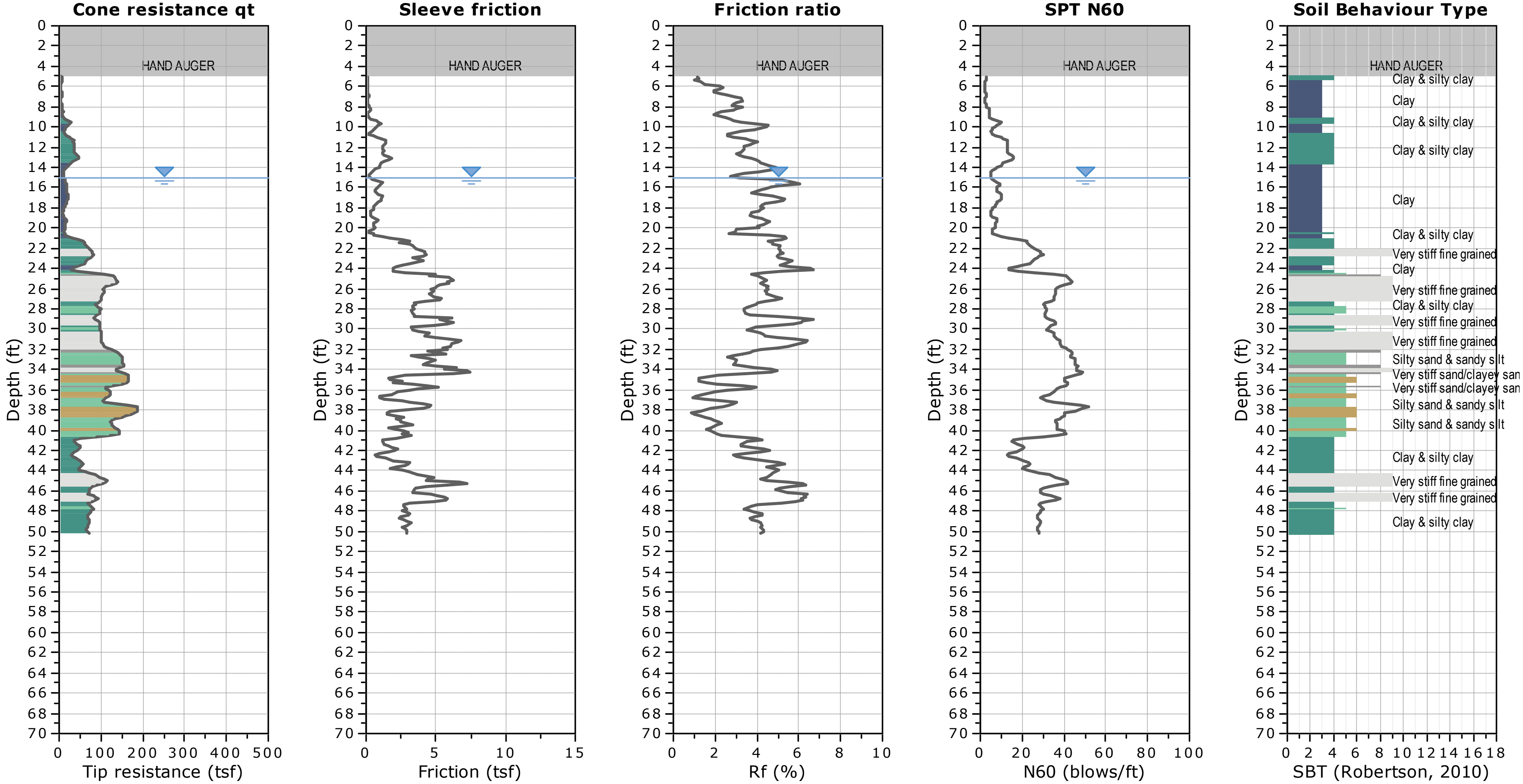
CPT: CPT-2

CLIENT: MID PACIFIC ENGINEERING

SITE: SJCOE CODESTACK, STOCKTON, CA

FIELD REP: DANIEL RIVERA
Cone ID: GDC-97

Total depth: 50.20 ft, Date: 10/31/2023

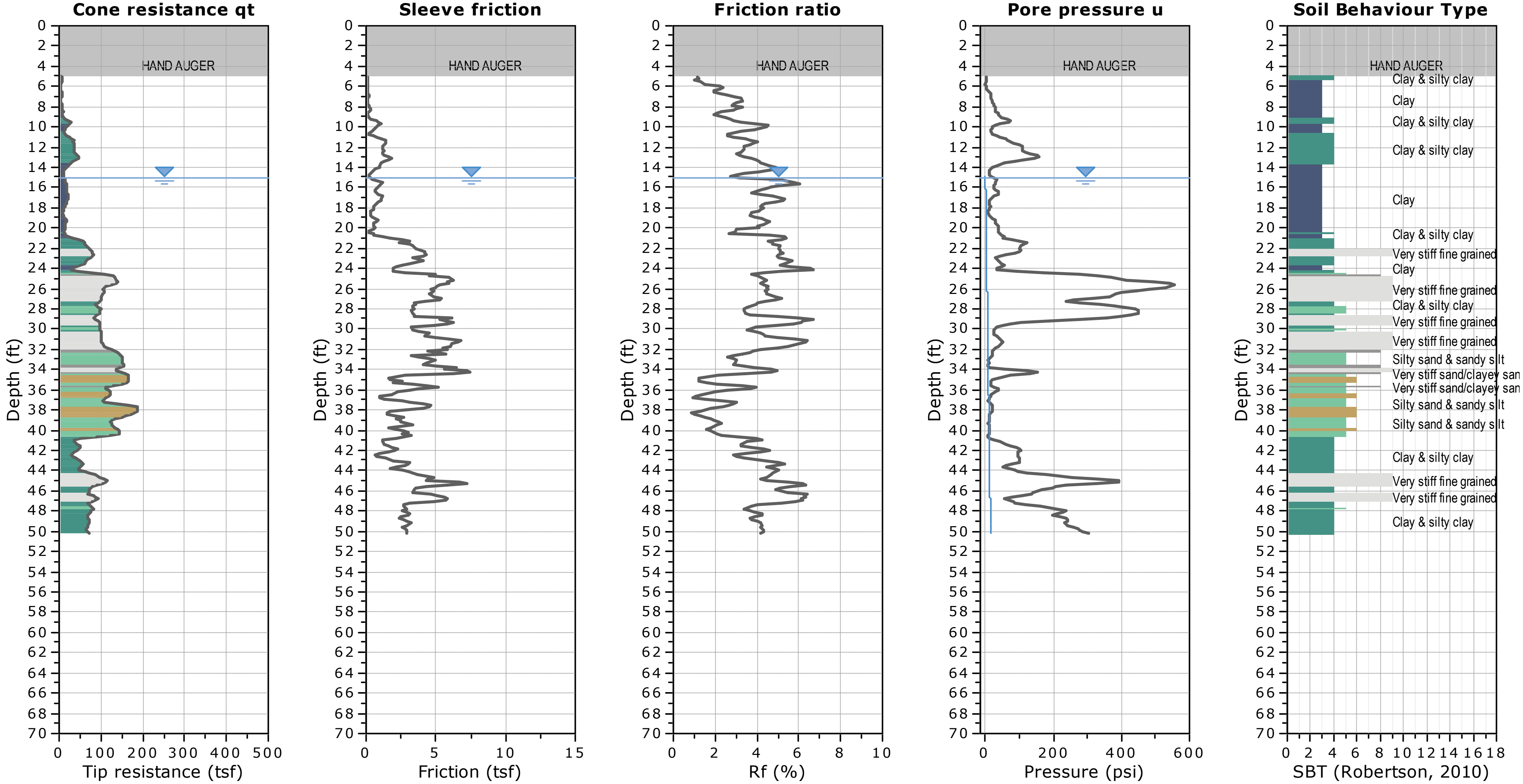


GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-2

FIELD REP: DANIEL RIVERA
Cone ID: GDC-97
Total depth: 50.20 ft, Date: 10/31/2023

CLIENT: MID PACIFIC ENGINEERING
SITE: SJCOE CODESTACK, STOCKTON, CA



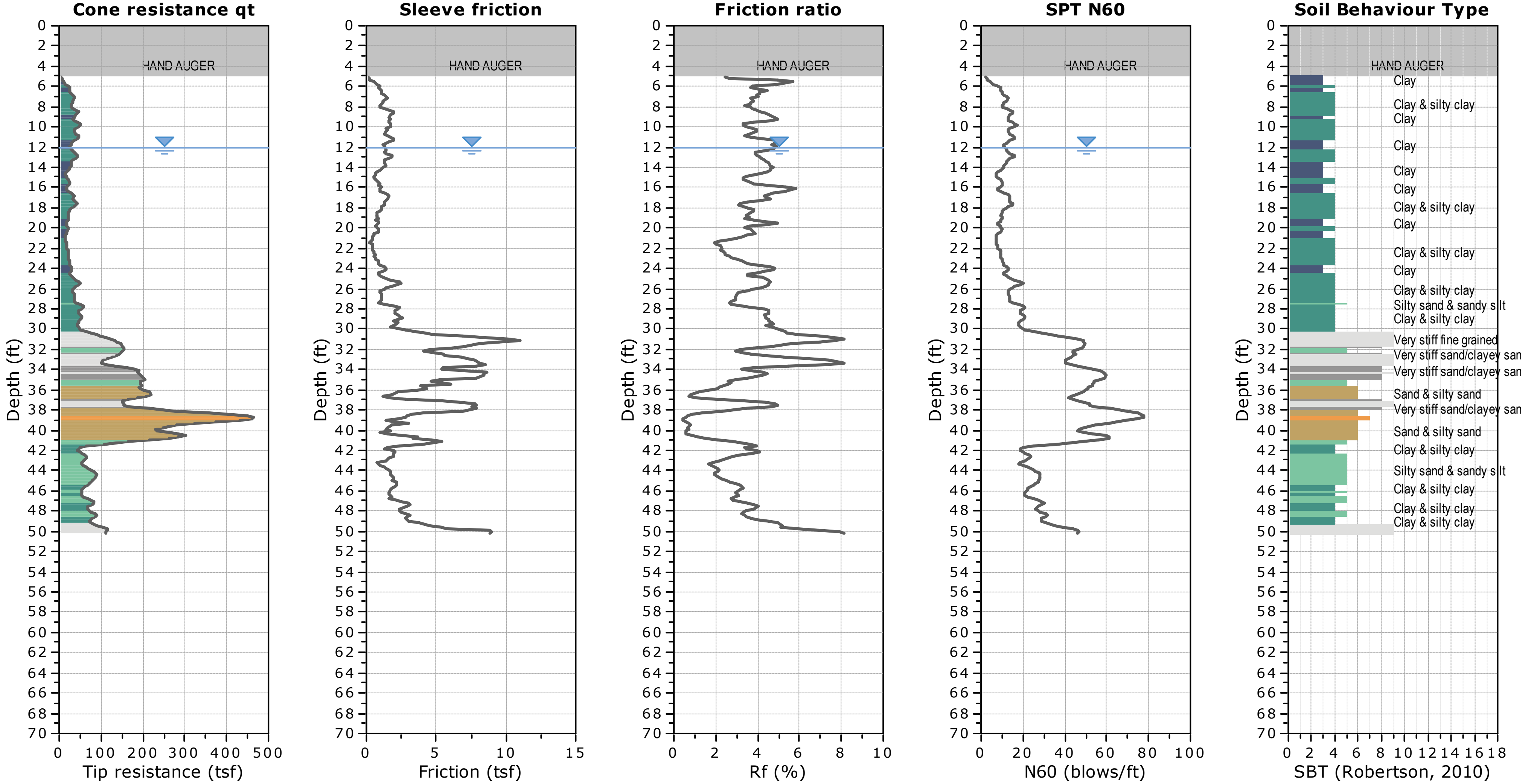
WATER TABLE FOR ESTIMATING PURPOSES ONLY

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-3

CLIENT: MID PACIFIC ENGINEERING
SITE: SJCOE CODESTACK, STOCKTON, CA

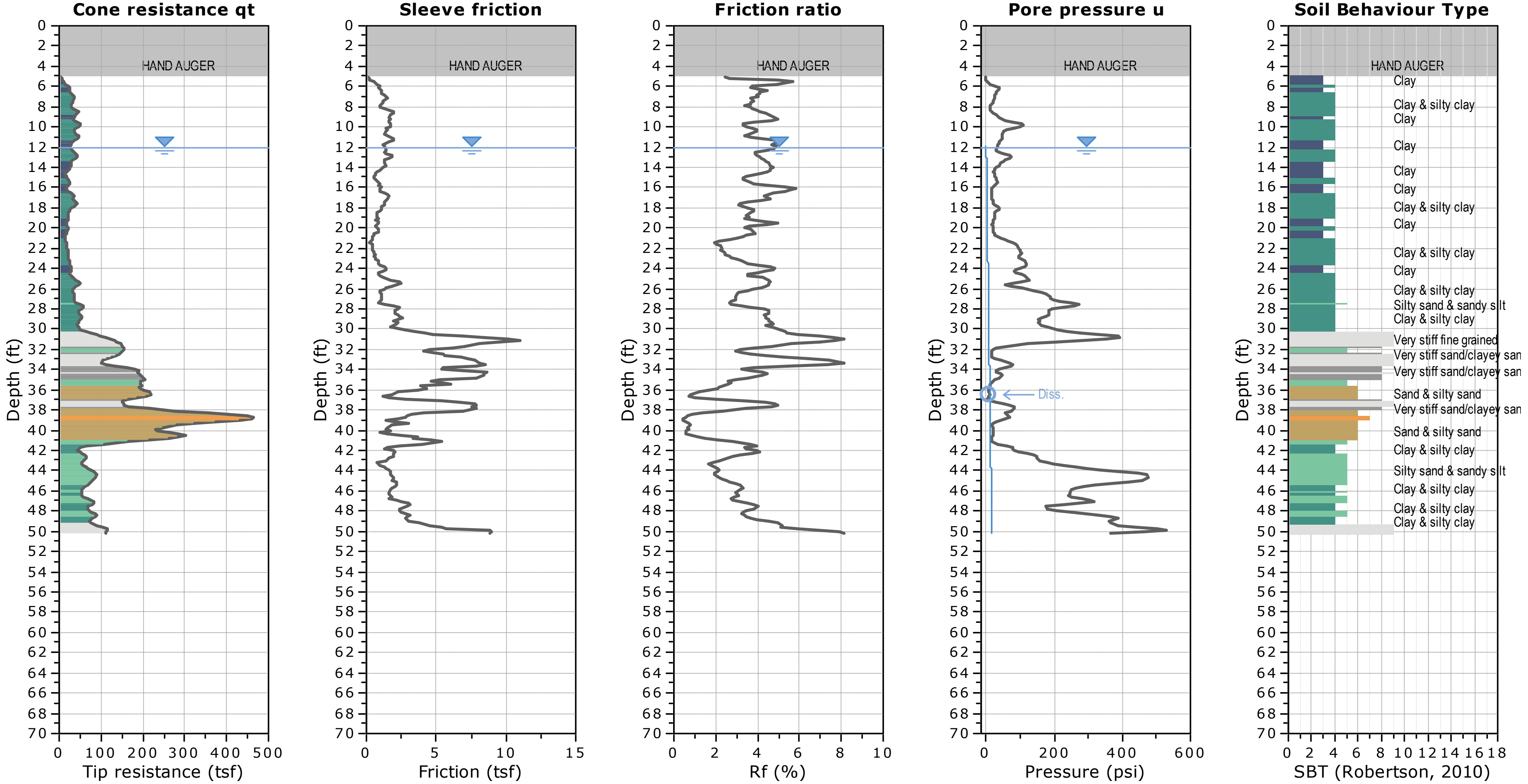
FIELD REP: DANIEL RIVERA
Cone ID: GDC-97
Total depth: 50.20 ft, Date: 10/31/2023



CPT: CPT-3

CLIENT: MID PACIFIC ENGINEERING
SITE: SJCOE CODESTACK, STOCKTON, CA

FIELD REP: DANIEL RIVERA
Cone ID: GDC-97
Total depth: 50.20 ft, Date: 10/31/2023



WATER TABLE FOR ESTIMATING PURPOSES ONLY

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

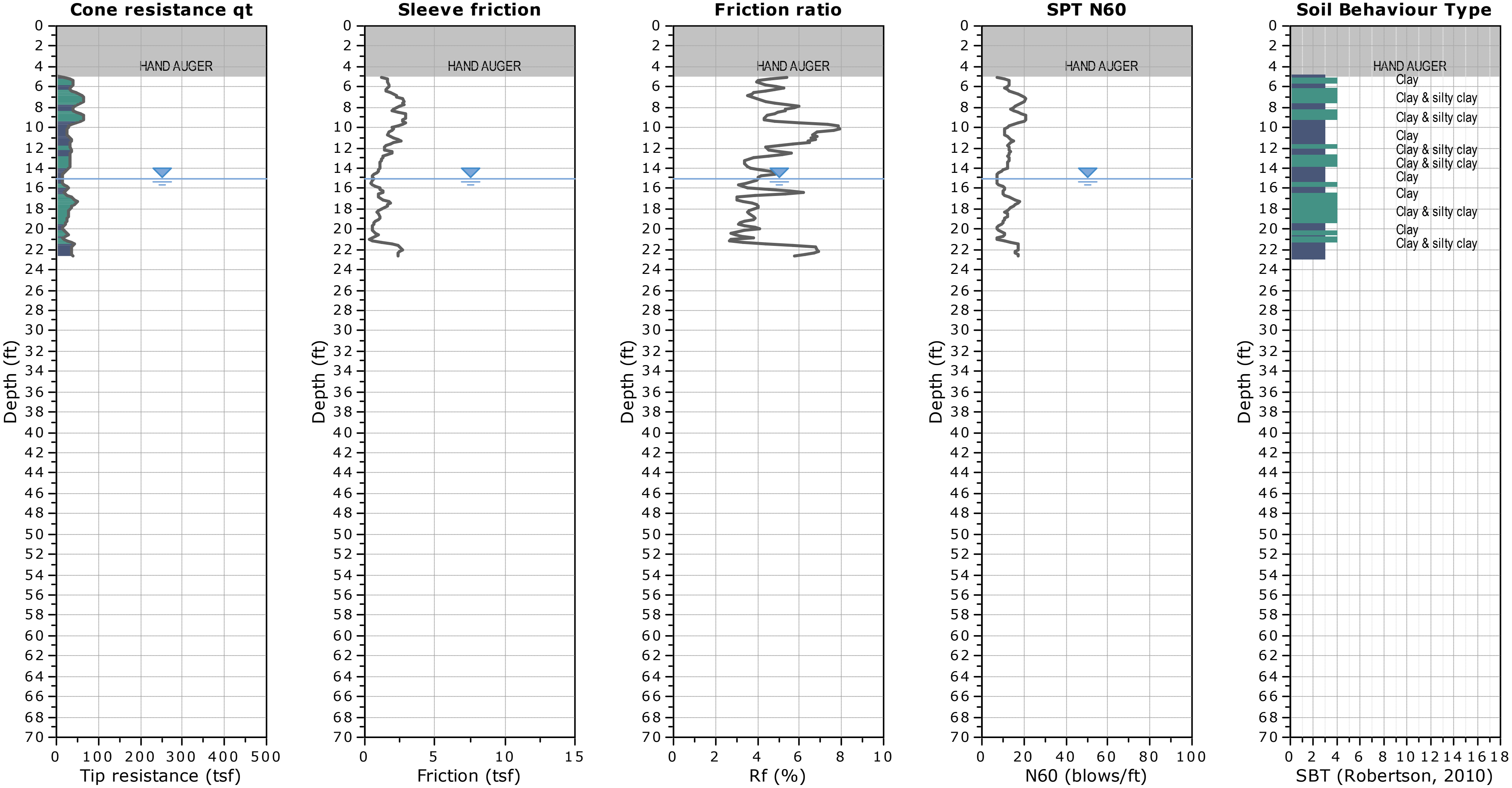
CPT: CPT-4

CLIENT: MID PACIFIC ENGINEERING

SITE: SJCOE CODESTACK, STOCKTON, CA

FIELD REP: DANIEL RIVERA
Cone ID: GDC-97

Total depth: 22.64 ft, Date: 10/31/2023



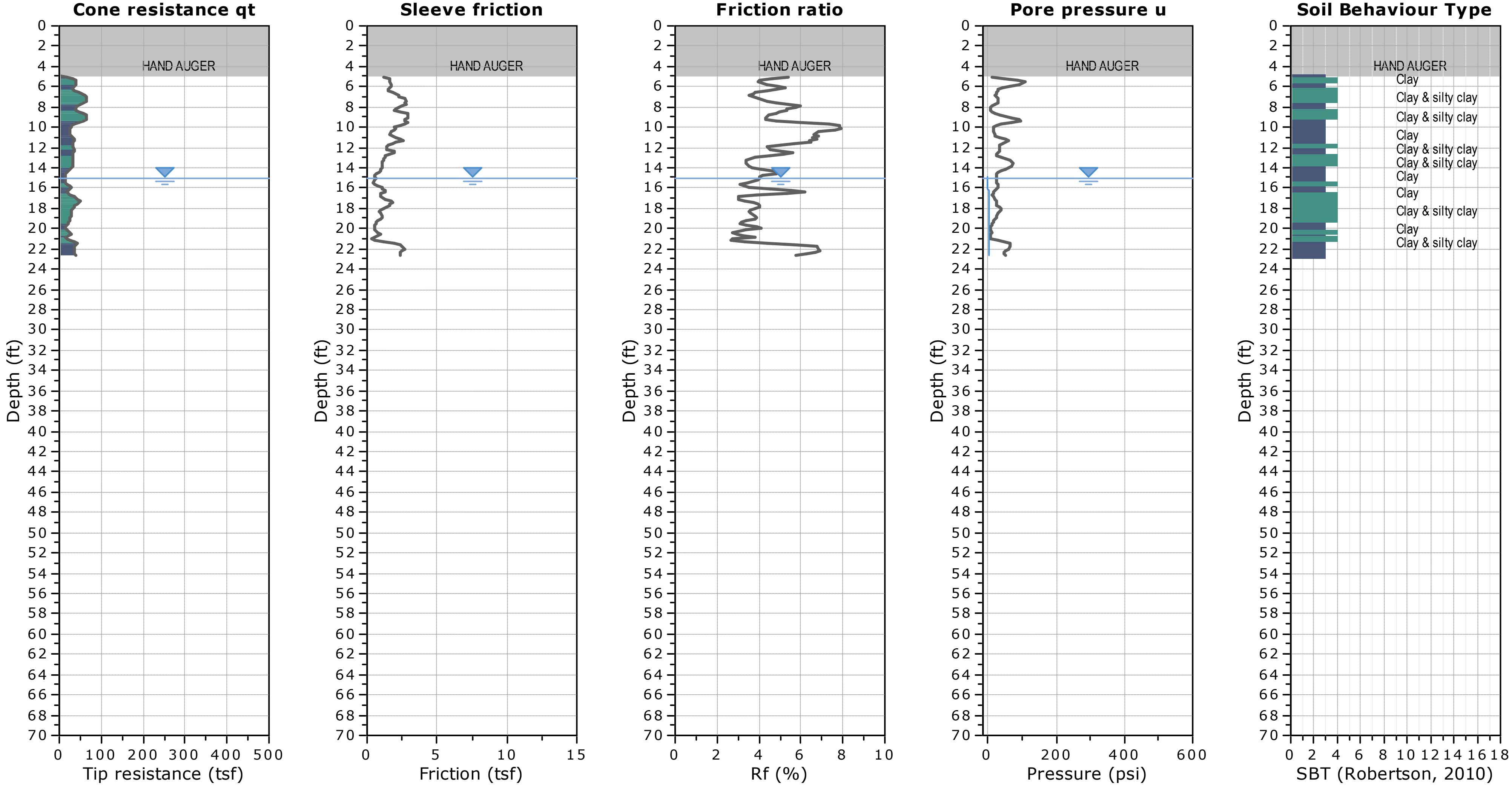
GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-4

FIELD REP: DANIEL RIVERA
Cone ID: GDC-97

CLIENT: MID PACIFIC ENGINEERING
SITE: SJCOE CODESTACK, STOCKTON, CA

Total depth: 22.64 ft, Date: 10/31/2023



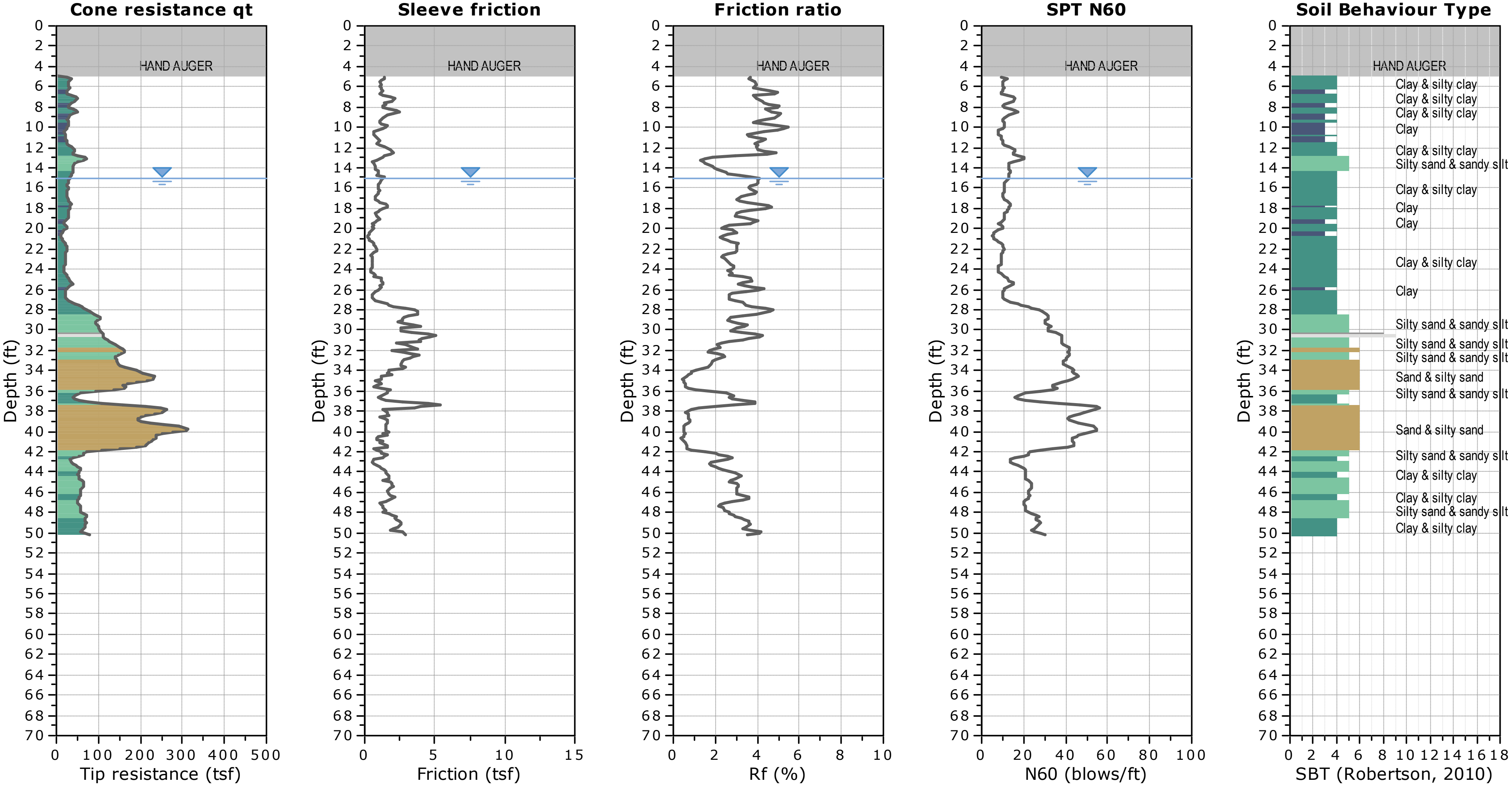
WATER TABLE FOR ESTIMATING PURPOSES ONLY

CPT: SCPT-5

FIELD REP: DANIEL RIVERA
Cone ID: GDC110

CLIENT: MID PACIFIC ENGINEERING
SITE: SJCOE CODESTACK, STOCKTON, CA

Total depth: 50.20 ft, Date: 11/02/2023



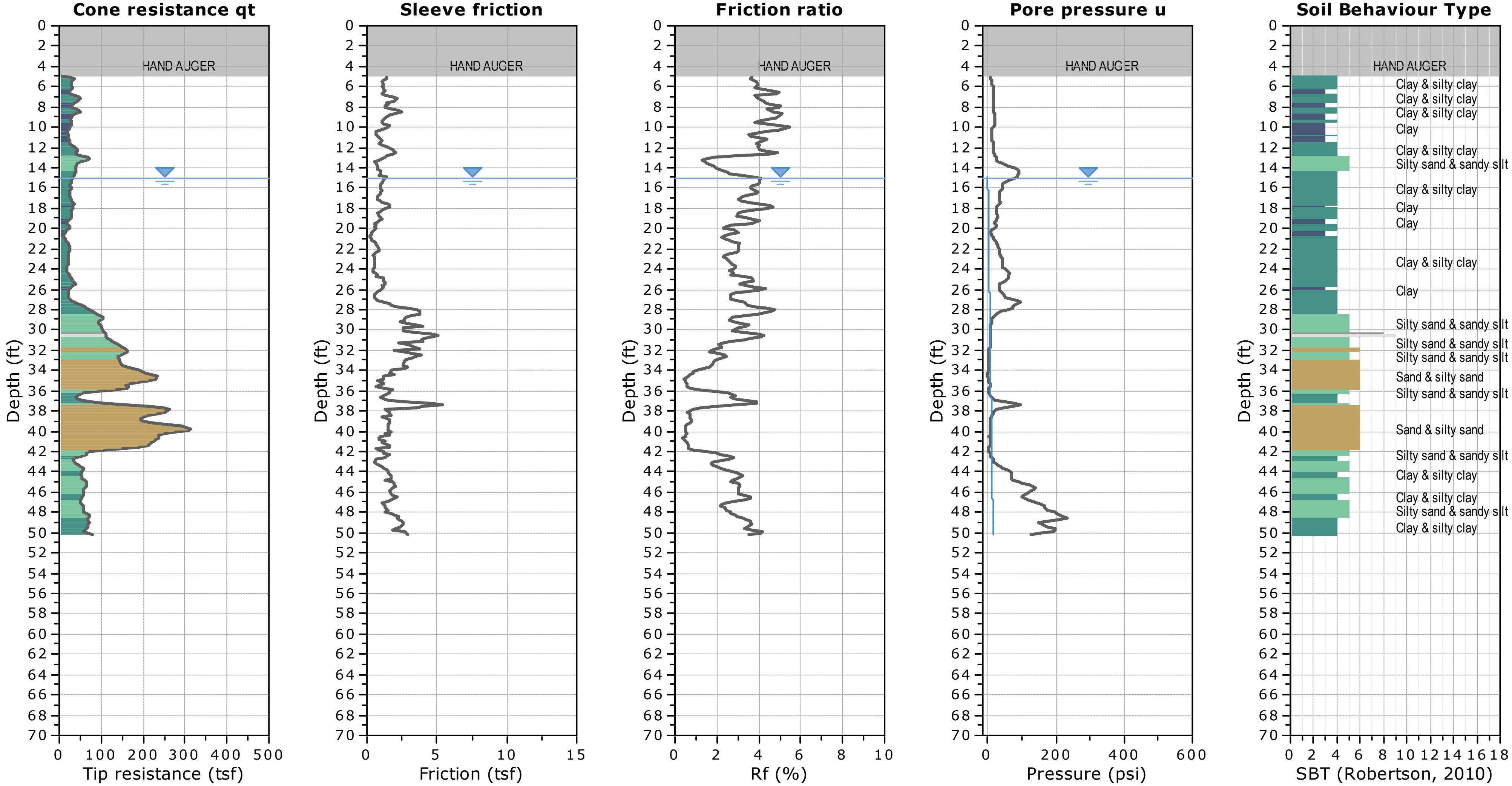
GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: SCPT-5

FIELD REP: DANIEL RIVERA
Cone ID: GDC110

CLIENT: MID PACIFIC ENGINEERING
SITE: SJCOE CODESTACK, STOCKTON, CA

Total depth: 50.20 ft, Date: 11/02/2023



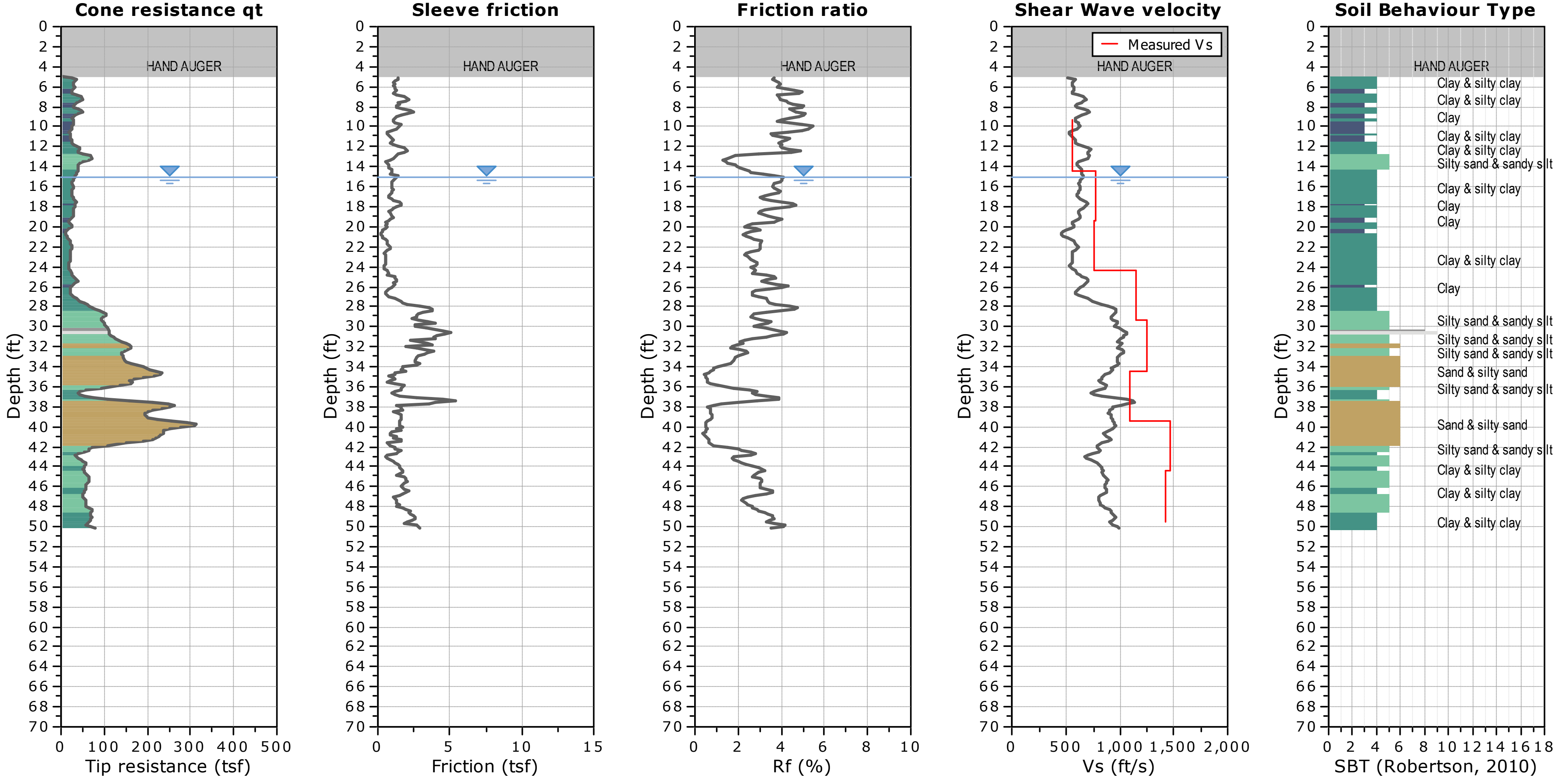
WATER TABLE FOR ESTIMATING PURPOSES ONLY

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: SCPT-5

FIELD REP: DANIEL RIVERA
Cone ID: GDC110
Total depth: 50.20 ft, Date: 11/02/2023

CLIENT: MID PACIFIC ENGINEERING
SITE: SJCOE CODESTACK, STOCKTON, CA



SBTn legend

🟥 1. Sensitive fine grained
🟫 2. Organic material
🟦 3. Clay to silty clay
🟩 4. Clayey silt to silty clay
🟩 5. Silty sand to sandy silt
🟨 6. Clean sand to silty sand
🟧 7. Gravelly sand to sand
⬜ 8. Very stiff sand to clayey sand
⬜ 9. Very stiff fine grained

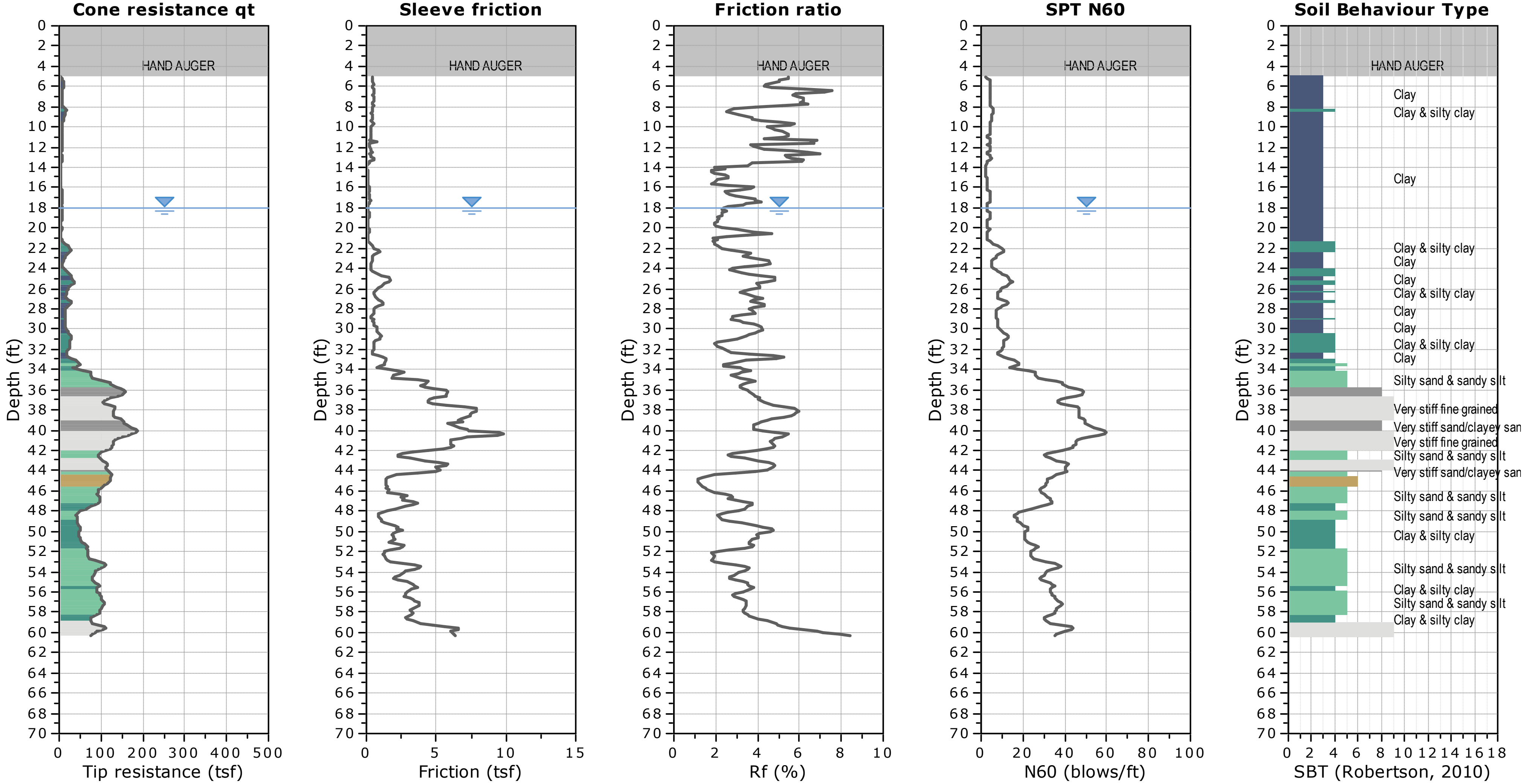
CPT: SCPT-6

CLIENT: MID PACIFIC ENGINEERING

SITE: SJCOE CODESTACK, STOCKTON, CA

FIELD REP: DANIEL RIVERA
Cone ID: GDC110

Total depth: 60.37 ft, Date: 11/02/2023

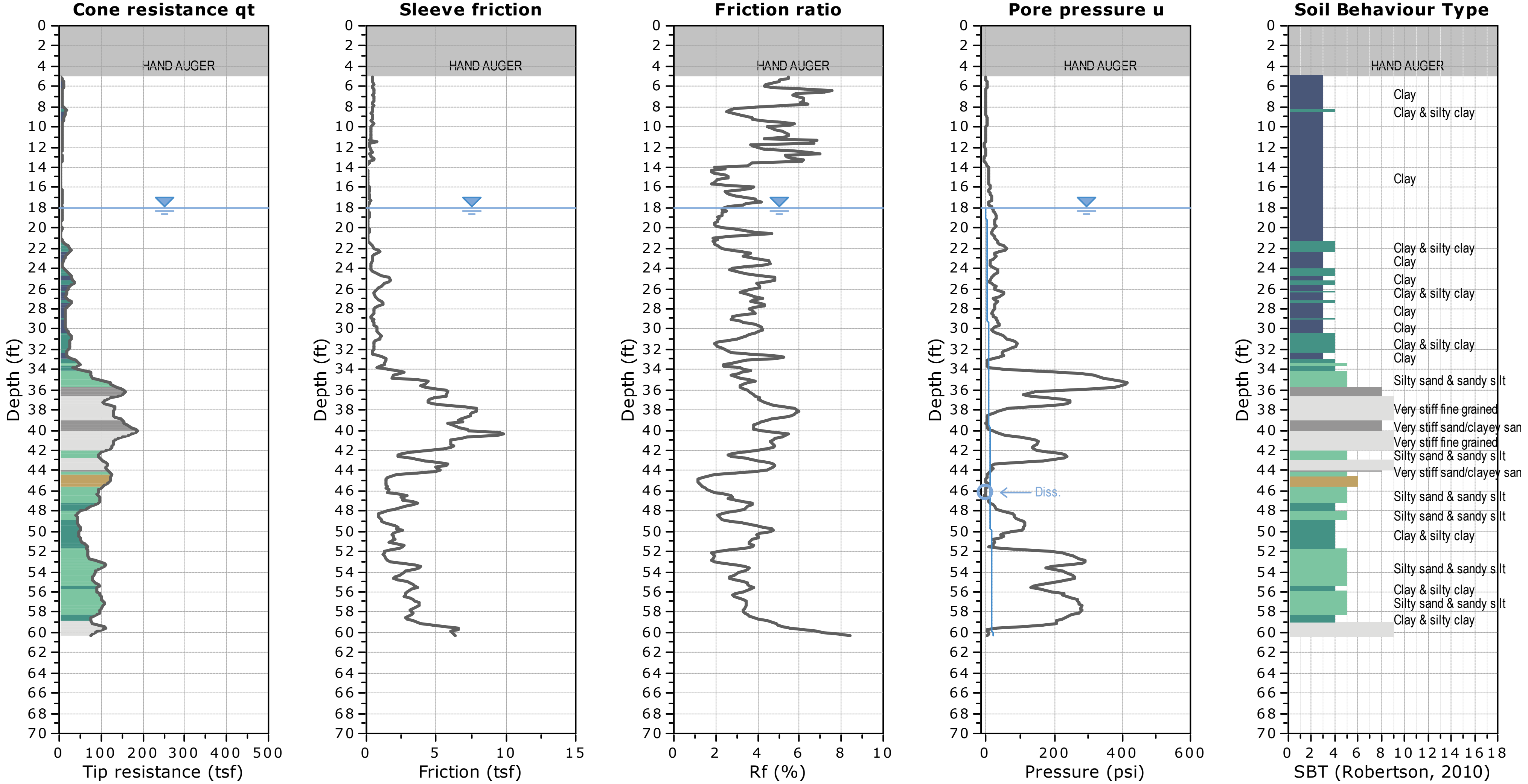


GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: SCPT-6

CLIENT: MID PACIFIC ENGINEERING
SITE: SJCOE CODESTACK, STOCKTON, CA

FIELD REP: DANIEL RIVERA
Cone ID: GDC110
Total depth: 60.37 ft, Date: 11/02/2023



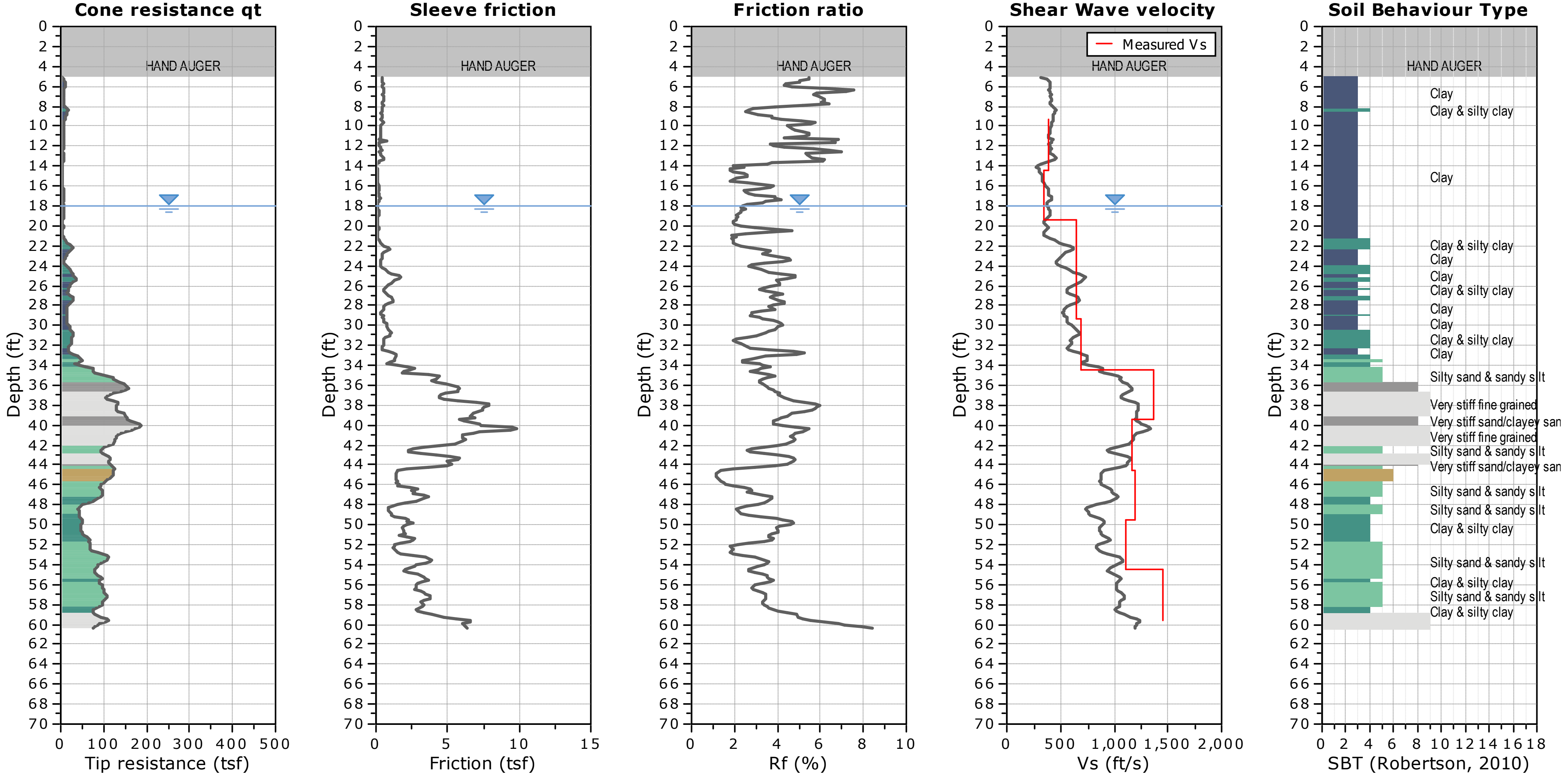
WATER TABLE FOR ESTIMATING PURPOSES ONLY

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: SCPT-6

CLIENT: MID PACIFIC ENGINEERING
SITE: SJCOE CODESTACK, STOCKTON, CA

FIELD REP: DANIEL RIVERA
Cone ID: GDC110
Total depth: 60.37 ft, Date: 11/02/2023



SBTn legend

🟥 1. Sensitive fine grained
🟫 2. Organic material
🟦 3. Clay to silty clay
🟩 4. Clayey silt to silty clay
🟩 5. Silty sand to sandy silt
🟨 6. Clean sand to silty sand
🟧 7. Gravelly sand to sand
⬛ 8. Very stiff sand to clayey sand
⬜ 9. Very stiff fine grained

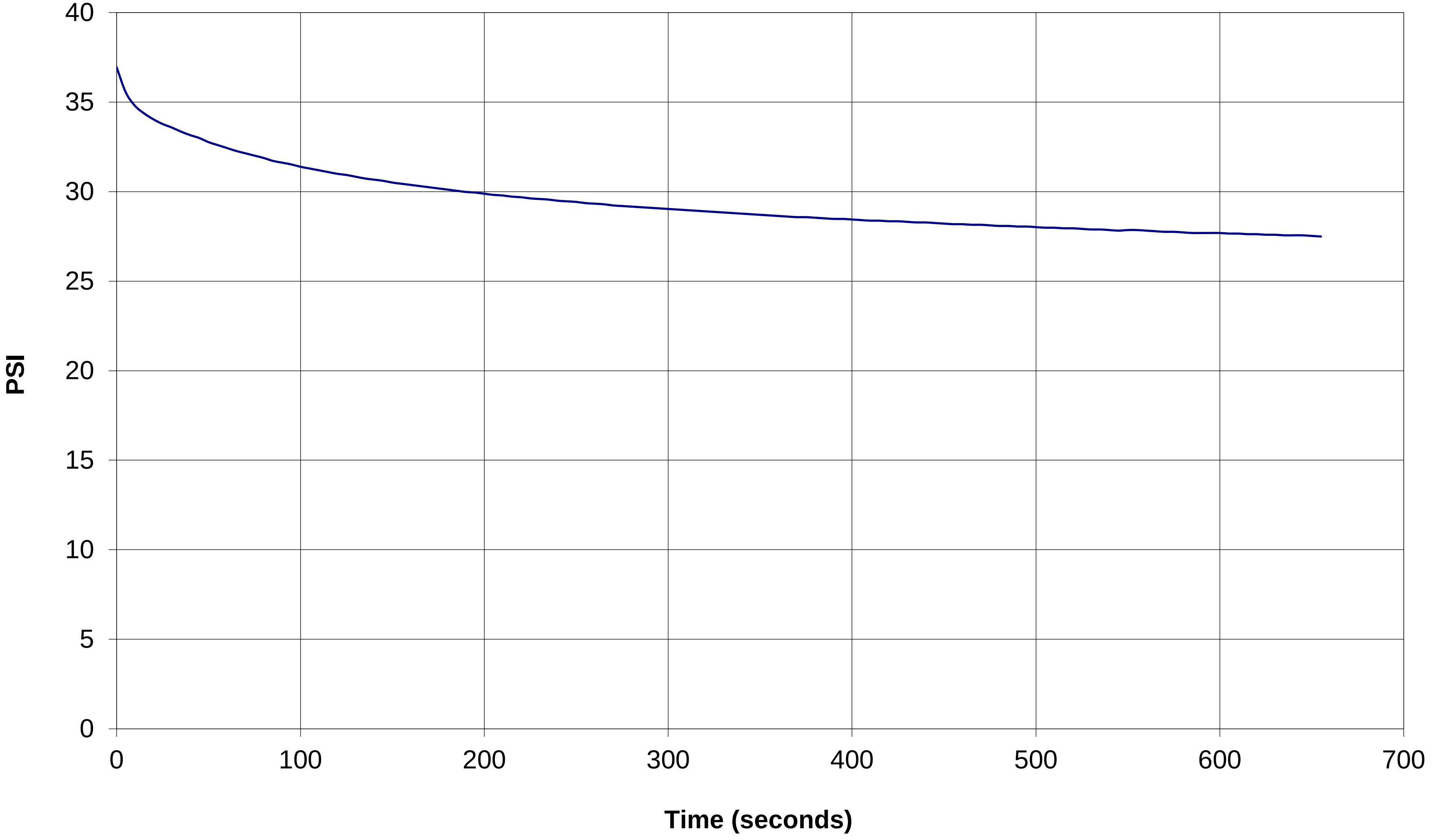
APPENDIX B:

PORE PRESSURE DISSIPATION TEST PLOTS

GREGG DRILLING, LLC

Pore Pressure Dissipation Test

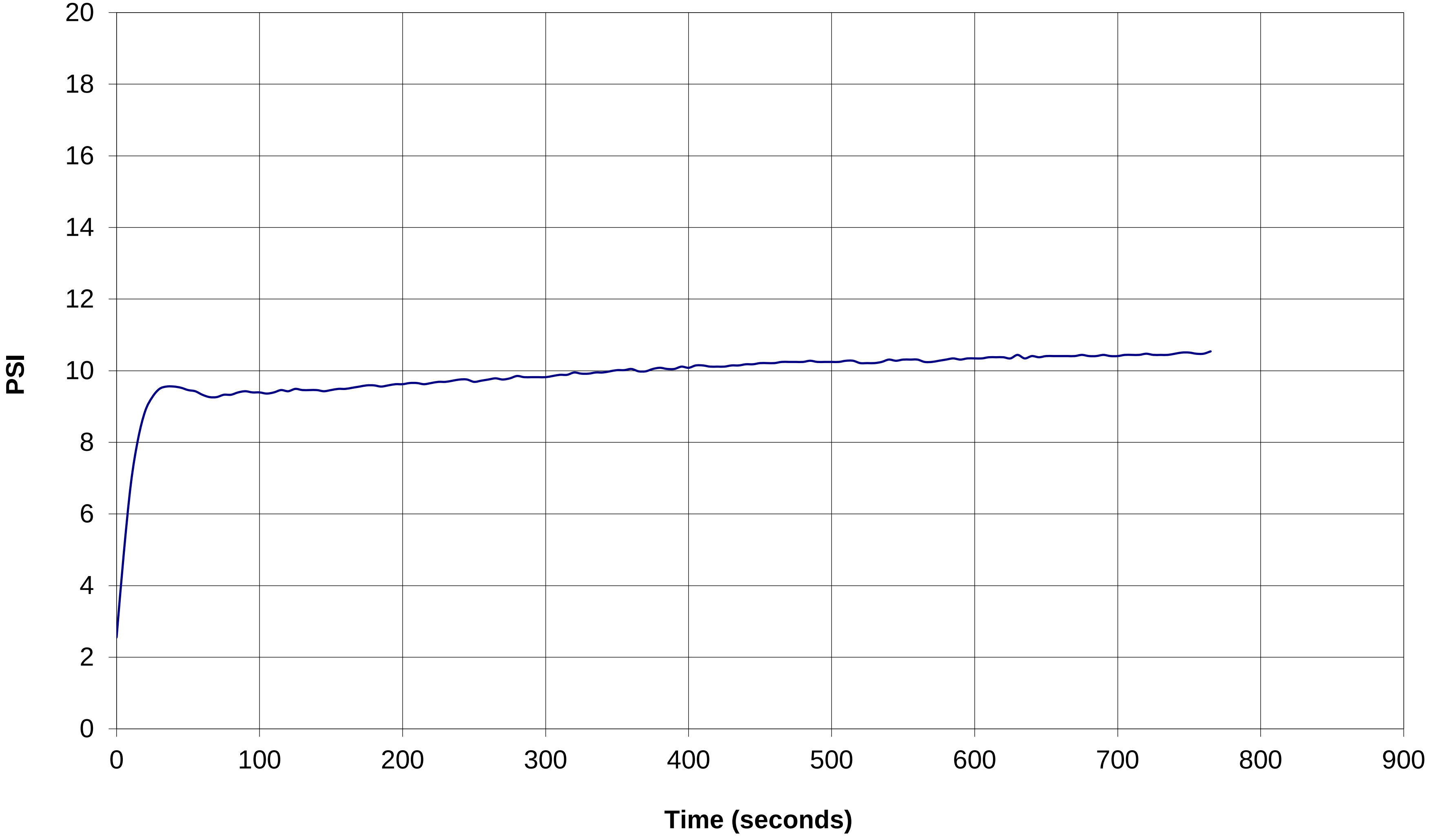
Sounding: CPT-1
Depth (ft): 33.79
Site: 201 N California
Engineer: Daniel Rivera



GREGG DRILLING, LLC

Pore Pressure Dissipation Test

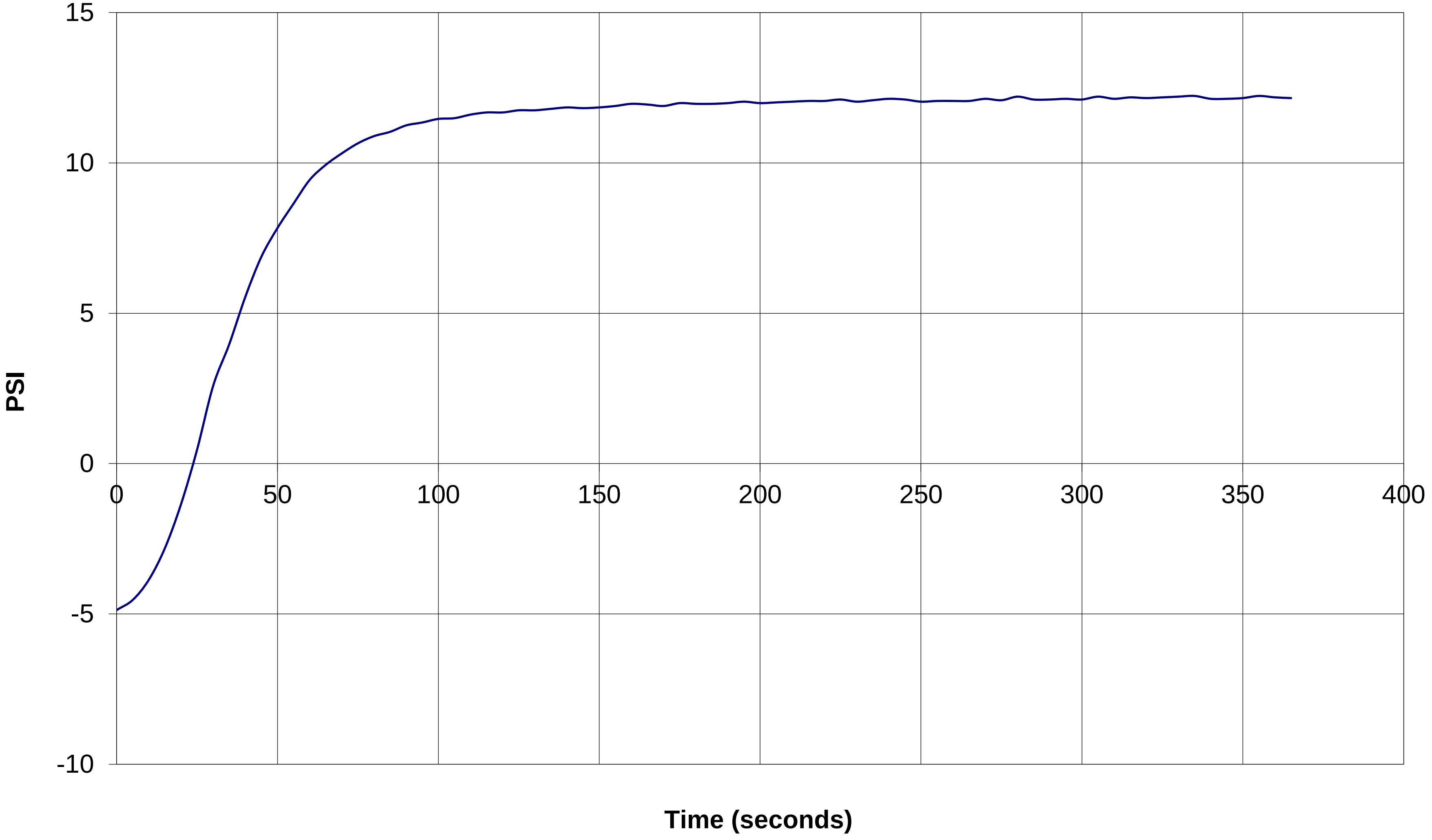
Sounding: CPT-3
Depth (ft): 36.42
Site: 201 N California
Engineer: Daniel Rivera



GREGG DRILLING, LLC

Pore Pressure Dissipation Test

Sounding: SCPT-6
Depth (ft): 46.10
Site: 201 N. California
Engineer: Daniel Rivera



APPENDIX C:

SEISMIC

PLOTS & TABLES

Shear Wave Velocity Calculations

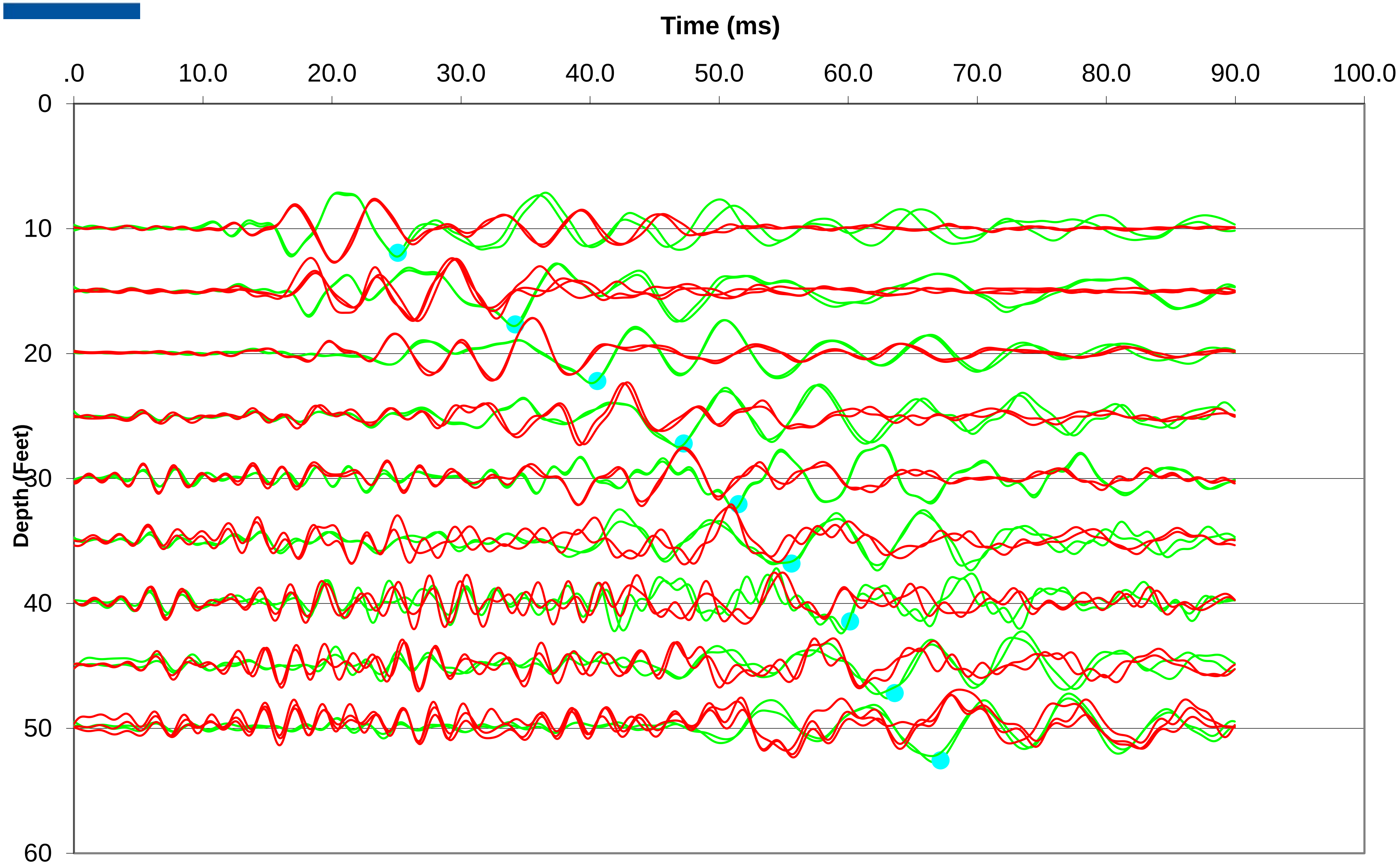
201 N California St.
SCPT-5

Geophone Offset: 0.66 Feet
Source Offset: 1.67 Feet

11/02/23

Test Depth (Feet)	Geophone Depth (Feet)	Waveform Ray Path (Feet)	Incremental Distance (Feet)	Characteristic Arrival Time (ms)	Incremental Time Interval (ms)	Interval Velocity (Ft/Sec)	Interval Depth (Feet)
10.01	9.35	9.49	9.49	24.9500			
15.09	14.43	14.53	5.03	34.0000	9.0500	556.2	11.89
20.01	19.35	19.42	4.90	40.3000	6.3000	777.3	16.89
25.10	24.44	24.50	5.07	46.9500	6.6500	762.5	21.90
30.02	29.36	29.41	4.91	51.2000	4.2500	1155.7	26.90
35.10	34.44	34.49	5.08	55.2500	4.0500	1253.9	31.90
40.03	39.37	39.40	4.92	59.7500	4.5000	1092.5	36.91
45.11	44.45	44.48	5.08	63.2000	3.4500	1472.8	41.91
50.20	49.54	49.56	5.08	66.7500	3.5500	1431.6	46.99

Waveforms for Sounding SCPT-5



Shear Wave Velocity Calculations

201 N. California St.
SCPT-6

Geophone Offset: 0.66 Feet
Source Offset: 1.67 Feet

11/02/23

Test Depth (Feet)	Geophone Depth (Feet)	Waveform Ray Path (Feet)	Incremental Distance (Feet)	Characteristic Arrival Time (ms)	Incremental Time Interval (ms)	Interval Velocity (Ft/Sec)	Interval Depth (Feet)
10.01	9.35	9.49	9.49	21.7500			
15.09	14.43	14.53	5.03	34.6000	12.8500	391.7	11.89
20.01	19.35	19.42	4.90	48.9500	14.3500	341.2	16.89
25.10	24.44	24.50	5.07	56.8000	7.8500	645.9	21.90
30.02	29.36	29.41	4.91	64.4000	7.6000	646.3	26.90
35.10	34.44	34.49	5.08	71.7500	7.3500	690.9	31.90
40.03	39.37	39.40	4.92	75.3500	3.6000	1365.6	36.91
45.28	44.62	44.65	5.25	79.8500	4.5000	1165.6	41.99
50.03	49.37	49.40	4.75	83.8500	4.0000	1188.5	46.99
55.12	54.46	54.48	5.08	88.4500	4.6000	1104.9	51.92
60.37	59.71	59.73	5.25	92.0500	3.6000	1457.5	57.08

Waveforms for Sounding SCPT-6