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APPENDIX H

Geotechnical Report

GEOTECHNICAL INVESTIGATION
Advanced Manufacturing Building
330 W. Trimble Road
San Jose, California

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GEOTECHNICAL INVESTIGATION
Advanced Manufacturing Building
330 W. Trimble Road
San Jose, California

1.0 INTRODUCTION

This report presents the results of the geotechnical investigation by Langan Engineering and Environmental Services for the proposed Advanced Manufacturing Building (AMB) development at 330 W. Trimble Road in San Jose, California. The approximate location of the site is shown on Figure 1. The current AMB project replaces a previously proposed development at the site, formerly referred to as the North Town development. We performed a geotechnical investigation and summarized the results of our findings for the North Town development in a report dated 15 November 2019. Using the subsurface information from the previous geotechnical investigation, this report presents our revised geotechnical conclusions and recommendations for the updated AMB development.

Our understanding of the AMB project is based on our review of drawings titled "Advanced Manufacturing Building, 330 W. Trimble Road, San Jose, California, 95131," dated 16 December 2022 by Ware Malcomb (project architect), Paradigm Structural Engineers (project structural engineer), and HMH (project civil engineer), and our recent correspondence with the project team. We understand that the AMB project will be designed and permitted using the 2019 California Building Code (CBC).

The approximately 10.2-acre AMB site is bounded by W. Trimble Road on the northwest, Orchard Parkway on the east and northeast, and an internal access road between W. Trimble Road and Orchard Parkway (part of completed North Town roadway extension project) and the 350 W. Trimble Road development on the south; see Figure 2 for additional details.

The proposed AMB will have an approximately rectangular footprint with maximum plan dimensions of about 330 by 700 feet, a total area of about 207,350 square feet, and a finished floor at Elevation 28 feet[1]. The single-story building will be at-grade, have a 36-foot clear interior height, and a maximum exterior height of 50 feet. The proposed column spacing is typically 50 to 60 feet. According to our correspondence with Paradigm Structural Engineers, the column dead

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[1] All elevations reference the North American Vertical Datum of 1988 (NAVD88).

plus live loads will typically be between about 250 and 400 kips, with perimeter wall loads of about 10.5 kips per foot.

Site improvements associated with the AMB project include at-grade parking areas, access driveways, stormwater management features, new utilities, site lights, low site retaining walls, and hardscape and landscape areas. According to our correspondence with HMH and the available site grading plans, we understand that site grading will be minor, with cut and fills on the order of two feet or less.

2.0 SCOPE OF SERVICES

Our geotechnical services were performed in general accordance with the scope of services outlined in our proposal dated 2 August 2022. Our services included using the results of our subsurface exploration and laboratory test programs completed at the project site in 2019 and performing supplemental engineering analyses to update our past conclusions and recommendations regarding the following for the AMB project:

- 2019 CBC site classification, mapped values S_S and S_1, modification factors F_a and F_v, and S_{MS} and S_{M1}, as appropriate;
- site seismicity and potential for seismic hazards including liquefaction, lateral spreading, and fault rupture;
- appropriate foundation type(s) including shallow and deep foundations and/or ground improvement, as necessary;
- design parameters for the recommended foundation type(s), including vertical and lateral capacities and associated estimated settlements;
- subgrade preparation for slabs-on-grade, shallow foundations (if appropriate), exterior slabs and flatwork, including sidewalks;
- site preparation, grading, and excavation, including criteria for fill quality and compaction;
- flexible pavement for driveway and site access road areas; and
- construction considerations.

3.0 FIELD EXPLORATION AND LABORATORY TESTING

We drilled six borings and performed seven cone penetration tests (CPTs) at the site in 2019 for the previously proposed North Town development. The approximate locations of the borings and

CPTs are shown on Figure 2. Before performing our field exploration, we obtained a soil boring/monitoring well permit from the Santa Clara Valley Water District (SCVWD), notified Underground Service Alert (USA), and checked the exploration locations for underground utilities using a private utility locator. Details of the field exploration and laboratory test programs are discussed in the remainder of this section.

3.1 Borings

Six borings, designated B-1 through B-6, were completed at the site from 4 through 7 March 2019. Borings B-1 and B-4 through B-6 were drilled using a truck-mounted, drill rig operated by Exploration Geoservices, Inc.; these borings were drilled with a hollow stem auger to depths of approximately 45 to 60 feet below ground surface (bgs). Borings B-2 and B-3 were drilled using a truck-mounted, rotary wash drill rig operated by Pitcher Services, LLC; these borings were drilled to depths of approximately 81½ feet bgs.

Our field engineer or geologist logged the borings and obtained samples of the material encountered for visual classification and laboratory testing. Logs of the borings are presented in Appendix A as Figures A-1 through A-6. The soil encountered in the borings was classified in accordance with the Classification Chart presented on Figure A-7. Soil samples were obtained using three different types of samplers: two driven, split-barrel samplers and a piston thin-walled sampler. The sampler types are as follows:

- Sprague & Henwood (S&H) split-barrel sampler with a 3.0-inch outside diameter and 2.5-inch inside diameter, lined with steel or brass tubes with an inside diameter of 2.43 inches

- Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and 1.5-inch inside diameter, without liners

- Shelby Tube (ST) a piston, thin-walled sampler with a 3-inch outside diameter and a 2.93-inch inside diameter

The sampler type was chosen based on the soil type being sampled and desired sample quality for laboratory testing. In general, the S&H sampler was used to obtain samples in medium stiff to very stiff cohesive soil and the SPT sampler was used to evaluate the penetration resistance of sandy soil. The ST sampler was used to selectively obtain relatively undisturbed samples of soft to medium stiff cohesive soil.

The SPT and S&H samplers were driven with a 140-pound, above-ground, automatic safety hammer (Borings B-2 and B-3) and a downhole, wireline hammer (Borings B-1 and B-4 through B-6) falling 30 inches. The samplers were driven up to 18 inches and the hammer blows required to drive the samplers every 6 inches of penetration were recorded and are presented on the boring logs. A "blow count" is defined as the number of hammer blows per 6 inches of penetration or 50 blows for 6 inches or less of penetration. The driving of samplers was discontinued if the observed (recorded) blow count was 50 for 6 inches or less of penetration. The blow counts required to drive the S&H and SPT samplers were converted to approximate SPT N-values using factors of 0.5 and 0.9, respectively, for Borings B-1 and B-4 through B-6, respectively, and factors of 0.7 and 1.2, respectively, for Borings B-2 and B-3 to account for sampler type and hammer energy, and are shown on the boring logs. The last two blow counts were used for the conversion to SPT N-values.

The ST sampler is pushed hydraulically into the soil; the piston pressure required to advance the sampler, if noted, is shown on the boring logs, measured in pounds per square inch (psi).

The soil cuttings from the borings were collected in 55-gallon drums, which were stored temporarily at the site, tested, and transported off-site for proper disposal. Upon completion, the boreholes were backfilled with cement grout in accordance with the requirements of the SCVWD.

3.2 Cone Penetration Tests

Seven CPTs, designated as CPT-1 through CPT-7, were performed on 4 and 5 March 2019 by ConeTec Inc. at the approximate locations shown on Figure 2. The CPTs were advanced to depths of approximately 61.7 to 101.1 feet bgs.

The CPTs were performed by hydraulically pushing a 1.7-inch-diameter, cone-tipped probe, with a projected area of 15 square centimeters, into the ground. The cone tip measures tip resistance, and a friction sleeve behind the cone tip measures frictional resistance. Electrical strain gauges or load cells within the cone continuously measured the cone tip resistance and frictional resistance during the entire depth of each probing. Accumulated data was processed by computer to provide engineering information, such as the types and approximate strength characteristics of the soil encountered. The CPT logs, showing tip resistance, side friction and friction ratio by depth, as well as interpreted SPT N-Values and interpreted soil classification, are presented in Appendix B. Soil types were estimated using the classification chart shown at the end of Appendix B.

Pore-pressure dissipation tests (PPDTs) were performed during the advancement of all the CPTs at various depths. The PPDTs were conducted at various depths to measure hydrostatic water pressures and to determine the approximate depth of the groundwater level. The variation of pore pressure with time is measured behind the tip of the cone and recorded. For our investigation, the duration of the tests range from approximately 215 to 635 seconds. The results of the seven PPDTs are presented in Appendix B.

Upon completion of the field investigation, the CPT holes were backfilled with cement-bentonite grout in accordance with the requirements of SCVWD.

3.3 Laboratory Testing

The soil samples collected from the field exploration program were reexamined in the office for soil classifications, and representative samples were selected for laboratory testing. The laboratory test program was designed to evaluate engineering properties of the soil at the site. Samples were tested to measure moisture content, dry density, plasticity (Atterberg Limits), percent fines, shear strength, compressibility, R-value and corrosivity, where appropriate. Results of the laboratory testing are included on the boring logs and in Appendix C on Figures C-1 through C-8.

3.4 Soil Corrosivity Testing

To evaluate the corrosivity of the soil near the foundation subgrade, we performed corrosivity tests on samples obtained from the upper 3 feet. The corrosivity of the soil samples was evaluated by CERCO Analytical using the following ASTM Test Methods:

- Redox - ASTM D1498
- pH - ASTM D4972
- Resistivity (100 percent Saturation) – ASTM G57
- Sulfide – ASTM D4658M
- Chloride – ASTM D4327
- Sulfate – ASTM D4327

The laboratory corrosion test results and a brief corrosivity evaluation are presented in Appendix D.

4.0 SITE AND SUBSURFACE CONDITIONS

The site was previously occupied by paved parking areas, landscape areas, and several below-grade utilities. In addition, a depressed stormwater management area was located in the eastern part of the site; see Figure 2 for the approximate location according to a record survey by Level 10. The stormwater depression (typically about 4 to 6 feet deep) was backfilled using undocumented fill in March 2018. The parking areas, utilities, and majority of the landscape areas were demolished in 2021, after which the site was used as a construction yard and laydown area. The site was rough graded in 2022 to facilitate stormwater management until it is redeveloped for the AMB project. It is relatively flat, with ground surface elevations typically from approximately Elevation 25½ to 28 feet. Isolated areas slope up to about Elevation 33 feet in the northern part of the site (HMH, 2019).

4.1 Subsurface Conditions

In our 2019 exploration, the surface material typically consisted of approximately 2½ inches of asphalt concrete (AC). Beneath the pavement section, the borings and CPTs encountered alluvial deposits. The near surface clay (i.e., within 7½ to 10 feet of the existing ground surface) consisted of stiff to very stiff clay, with a layer of silty sand and gravel at Boring B-3. Laboratory test results indicate the upper clay has very high expansion potential[2] with a plasticity index (PI) of about 54 to 57.

The near surface clay layer is underlain by soft to hard clay, sandy clay, clay with sand layers, and loose to very dense sand with varying types and amount of fines layers to the maximum depth explored. Where tested, the undrained shear strengths of the clay varied from 360 to 1,680 pounds per square foot (psf). Laboratory test results indicate that the clay has a compression ratio of 0.11 to 0.15, is overconsolidated[3] with overconsolidation ratios (OCRs) of about 1.6 to 3.8. In addition, laboratory test results indicate that the clay layers below a depth of about 7 feet have low expansion potential with a PI ranging of 8 to 10, where tested. The sand and gravel layers contain about 9 to 22½ percent fines, where tested.

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[2] Very highly expansive soil undergoes very large volume changes with changes in moisture content.

[3] An overconsolidated clay has experienced a pressure greater than its current load.

4.2 Groundwater

The California Geological Survey reported the historic high groundwater level in the site vicinity as approximately 10 feet bgs as part of the Seismic Hazards Zone Report (San Jose West Quadrangle).

Groundwater was encountered in the borings at depths of approximately 10 to 15 feet bgs, corresponding to approximately Elevation 17 to 14 feet. The groundwater levels were measured at the time of drilling and likely do not represent the stabilized groundwater level. Seasonal fluctuation in rainfall influence groundwater levels and could cause several feet of variation.

The PPDTs conducted at the CPTs were performed in the sand layers between depths of approximately 20.8 and 68.1 feet bgs. The potentiometric surface of the groundwater in these sand layers was calculated to be approximately 4.7 to 8.5 feet bgs, corresponding to approximately Elevation 21.6 to 17.5 feet. The hydrostatic water pressure measured during the PPDTs may not represent static groundwater conditions due to increased hydrostatic water pressure from recent rainfalls at the time of drilling and may represent and artesian condition in the sand layers. A summary of the potentiometric surface levels from the PPDTs is summarized in Table B-1, included in Appendix B.

5.0 SEISMIC AND GEOLOGIC HAZARDS

Information about seismic and geologic hazards are included in the following subsections.

5.1 Regional Seismicity

The project site is in a seismically active region. Numerous earthquakes have been recorded in the region in the past, and moderate to large earthquakes should be anticipated during the service life of the proposed development. The major active faults in the area are the San Andreas, San Gregorio, Hayward, and Calaveras faults. These and other faults of the region are shown on Figure 3. For each of the active faults within 50 kilometers (km) of the site, the distance from the site and estimated mean Moment magnitude[4] [2014 Working Group on California Earthquake Probabilities (WGCEP) (2015) and Uniform California Earthquake Rupture Forecast Version 3 (UCERF3) as detailed in the United States Geological Survey Open File Report 2013-1165] are

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[4] Moment magnitude is an energy-based scale and provides a physically meaningful measure of the size of a faulting event. Moment magnitude is directly related to average slip and fault rupture area.

summarized in Table 1. The mean Moment magnitude presented on Table 1 was computed assuming full rupture of the segment using Hanks and Bakun (2008) relationship.

TABLE 1
Regional Faults and Seismicity

Fault Segment[1]	**Approx. Distance from fault (km)**	**Direction from Site**	**Mean Moment Magnitude**[2]
Silver Creek	1.5	East	6.7
Total Hayward-Rodgers Creek Healdsburg	9	Northeast	7.6
Total Calaveras	13	East	7.5
Mission (connected)	13	Northeast	6.1
Monte Vista - Shannon	14	Southwest	7.0
San Andreas 1906 event	20	Southwest	8.1
Pilarcitos	22	West	6.7
Butano	24	Southwest	6.7
Sargent	27	South	6.8
Greenville	37	East	7.1
Mount Diablo Thrust	40	North	6.6
Total San Gregorio	41	West	7.6

Notes:
1. This table is a summary and does not include all the fault segmentation, alternate traces and low activity faults included in the UCERF3 model.
2. Mean Moment Magnitude based on entire fault length rupturing using Hanks and Bakun (2008)

Figure 3 also shows the earthquake epicenters for events with magnitude greater than 5.0 from January 1800 through August 2014. Since 1800, four major earthquakes have been recorded on the San Andreas fault. In 1836 an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) scale (Figure 4) occurred east of Monterey Bay on the San Andreas fault (Toppozada and Borchardt 1998). The estimated Moment magnitude, $M_{w,}$ for this earthquake is about 6.25. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to a M_w of about 7.5. The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage. This earthquake created a surface rupture along the San Andreas fault from Shelter Cove to San Juan Bautista approximately 470 kilometers in length. It had a maximum intensity of XI (MM), a M_w of about 7.9, and was felt 560 kilometers away in Oregon, Nevada, and Los Angeles.

The Loma Prieta Earthquake occurred on 17 October 1989, in the Santa Cruz Mountains with a M_w of 6.9, approximately 39 km from the site.

In 1868 an earthquake with an estimated maximum intensity of X on the MM scale occurred on the southern segment (between San Leandro and Fremont) of the Hayward fault. The estimated M_w for the earthquake is 7.0. In 1861, an earthquake of unknown magnitude (probably a M_w of about 6.5) was reported on the Calaveras Fault. The most recent significant earthquake on this fault was the 1984 Morgan Hill earthquake ($M_w = 6.2$).

In 2016, the U.S. Geologic Survey (USGS) predicted a 72 percent chance of a magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Area in 30 years (Aagaard et al. 2016). More specific estimates of the probabilities for different faults in the Bay Area are presented in Table 2.

TABLE 2
Estimates of 30-Year Probability (2014 to 2043) of a Magnitude 6.7 or Greater Earthquake

Fault	Probability (percent)
Hayward-Rodgers Creek	33
Calaveras	26
N. San Andreas	22
San Gregorio	16
Mount Diablo Thrust	16
Greenville	6

5.2 Geologic Hazards

The site is in a seismically active area and will likely be subjected to very strong shaking during a major earthquake. Strong ground shaking during an earthquake can result in ground failure such as that associated with soil liquefaction[5], lateral spreading[6], and seismic densification[7]. Each of these conditions has been evaluated based on our literature review, field investigation, and analyses, and is discussed in this section.

5.2.1 Liquefaction and Associated Hazards

As shown on Figure 5, the site is within a zone designated with the potential for liquefaction, as identified by the California Division of Mines and Geology (CDMG), known now as the California Geologic Survey, in a map titled "State of California Seismic Hazard Zones, San Jose West 7.5-Minute Quadrangle, Santa Clara County" prepared by the CDMG (7 February 2002). Specifically, the map shows the site is in an area "where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693 (c) would be required."

We performed our liquefaction analysis in accordance with the State of California Special Publication 117A, Guidelines for Evaluation and Mitigation of Seismic Hazards in California and following the procedures in Boulanger and Idriss (2014) to evaluate the liquefaction potential at the site. The Boulanger and Idriss (2014) procedures are updates of the Idriss and Boulanger (2008) procedures and the simplified procedures developed by Seed and Idriss (1971) and later by the 1996 NCEER and the 1998 NCEER/NSF workshops on the Evaluation of Liquefaction Resistance of Soils (Youd and Idriss 2001). To estimate volumetric strain and associated liquefaction-induced settlement, we used the procedure developed by Tokimatsu and Seed (1987) for the borings and Zhang et al. (2002) for the CPTs. We also used the procedure developed by Cetin et al. (2009) to apply a depth weighting factor to the liquefiable layers, which is based on the rationale that the contribution of deeper potentially-liquefiable soil layers to the overall ground settlement is lower than the contribution of shallower layers.

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[5] Liquefaction is a transformation of soil from a solid to a liquefied state during which saturated soil temporally loses strength resulting from the buildup of excess pore water pressure, especially during earthquake-induced cyclic loading. Soil susceptible to liquefaction includes loose to medium dense sand and gravel, low-plasticity silt, and some low-plasticity clay deposits.

[6] Lateral spreading is a phenomenon in which surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. Upon reaching mobilization, the surficial blocks are transported downslope or in the direction of a free face by earthquake and gravitational forces.

[7] Seismic densification is a phenomenon in which non-saturated, cohesionless soil is compacted by earthquake vibrations, causing ground surface settlement.

These methods are used to estimate a factor of safety against liquefaction triggering by taking the ratio of soil strength (resistance of the soil to cyclic shaking) to the seismic demand that can be expected from a design level seismic event. Specifically, two distinct terms are used in the liquefaction triggering analyses:

- Cyclic Resistance Ratio (CRR), which quantifies the soil's resistance to cyclic shaking; a function of soil depth, density, depth of groundwater, earthquake magnitude, and overall soil behavior

- Cyclic Stress Ratio (CSR), which quantifies the stresses that may develop during cyclic shaking

The factor of safety (FS) against liquefaction triggering can be expressed as the ratio of CRR over CSR. For our analyses, if the FS for a soil layer is less than 1.3, it is considered possible that the soil layer could liquefy during a large seismic event. For our calculations of estimated liquefaction-induced settlement, we assumed layers with a FS equal to or greater than 1.3 will not experience liquefaction-induced settlement.

The primary design parameters used in our liquefaction triggering calculations are summarized in Table 3.

TABLE 3
Primary Input Parameters Used in Liquefaction Evaluation

Parameter	Value
Depth to historic high groundwater	Approximately 10 feet bgs
Peak Ground Acceleration (PGA_M)*	0.579g
Predominant Earthquake Moment Magnitude (Mw)	8.1
Factor of Safety for Liquefaction Triggering	1.3
Conversion for S&H and SPT sampler blow count to SPT N-values**	0.7 and 1.2, respectively (to account for the automatic hammer)
CPT conversion factor for tip resistance to SPT N-value	4 to 5

Notes:
* Values for liquefaction analysis based on the site-specific response spectra per ASCE 7-16 and 2019 California Building Code; see Appendix E for details.
** Refer to Section 3.1 for additional details about sampler conversions.

In our analyses, soil that has significant amount of plastic fines, I_c greater than 2.6 were considered too cohesive to liquefy; a corrected cone tip resistance q_{c1N} greater 160 tons per square foot (tsf) were considered too dense to liquefy. Because the predominant earthquake is a moment magnitude 8.1, the cyclic resistance ratio (CRR) has been scaled to a moment magnitude of 7.5 using magnitude scaling factors developed by Boulanger and Idriss (2014).

In our assessment of the liquefaction potential for the CPTs, we considered the approach for soil classification and behavior presented in Robertson (2016). In this approach, CPT data is used to determine dilative and contractive behavior. The soil classification and behavior chart uses the normalized CPT tip resistance and friction ratio to separate material into clayey, sandy, and transitional soil types. The chart further uses another parameter, CD, to divide the dilative and contractive behavior of these soil types. A CD value of 70 or higher separates the soil between contractive and dilative tendencies. To capture transitional and borderline material, we used a CD cut-off value of 80. The CPTs indicate that many of the medium dense sand and low-plasticity silt layers below the groundwater level are potentially liquefiable, but will likely exhibit dilative behavior and thus not be prone to settlement during earthquake shaking.

Layers of medium dense sand with varying amounts of clay and silt, varying in thickness from several inches to approximately 4½ feet, were encountered below the groundwater level to a depth of approximately 43 feet bgs. Below this depth, the sands are dense to very dense. On the basis of the results of our analyses, we conclude that some of the medium dense layers could potentially liquefy during a major earthquake and may experience liquefaction-induced settlement. A summary of the data where a potentially liquefiable layer was encountered in the rotary wash borings (B-2 and B-3) and CPT-1 through CPT-7, as well as other pertinent parameters regarding liquefaction triggering and associated settlement, are presented in Tables 4 (borings) and 5 (CPTs). The data from the hollow-stem auger borings (B-1 and B-4 through B-6) were not included in our liquefaction analysis because of the potential for stress relief and disturbance at the bottom of the borehole when drilling in granular soils below groundwater.

TABLE 4
Summary of Liquefaction Potential and Estimate Settlement from Boring Data

Boring Number	Approx. Depth to Layer (feet)	Elevation of top of layer (feet)	Layer Thickness (feet)	$(N_1)_{60\text{-}CS}$	PGA_M	CSR_{EQ}	$CRR_{7.5}$	Factor of Safety	Volumetric Strain ε_v (percent)	Estimated Vertical Settlement (inches)
B-2	12	14.3	1	28	0.58	0.41	0.34	0.8	0.8	0.10
	30	-3.7	4	33	0.58	0.53	0.54	1.0	0.0	-
	34	-7.7	4.5	27	0.58	0.54	0.26	0.5	0.4	0.24
Total Settlement at B-2										**0.34**
B-3	14	13.1	4.5	18	0.58	0.45	0.17	0.4	1.2	0.67
	25	2.1	2	27	0.58	0.51	0.30	0.6	0.6	0.14
	35.5	-8.4	1	8	0.58	0.54	0.10	0.2	1.1	0.13
Total Settlement at B-3										**0.94**

TABLE 5
Summary of Liquefaction Potential and Estimate Settlement from CPT Data

CPT Number	Approx. Depth (feet)	Layer Thickness (feet)	I_C	$(q_{c1N})_{CS}$ (tsf)	N_{160}	CSR_{EQ}	$CRR_{7.5}$	Factor of Safety	Volumetric Strain ε_V (percent)	Estimated Vertical Settlement (inches)
CPT-3	12.6	0.6	2.6	70	14	0.54	0.11	0.20	2.45	0.14
	19.6	0.4	2.4	85	17	0.67	0.12	0.18	1.79	0.10
Total Settlement at CPT-3										**0.24**

We conclude that several discontinuous layers are potentially liquefiable during a major earthquake and estimate that up to 1 inch of liquefaction-induced settlements may occur at the project site. Because the layers appear discontinuous, differential settlement may be up to 1 inch over a horizontal distance of about 30 feet.

5.2.2 Seismic Densification

Seismic densification refers to seismically-induced differential compaction of non-saturated granular material (sand and gravel above the groundwater table) caused by earthquake vibrations. The borings and CPTs indicate that the materials above the water table are sufficiently clayey, and therefore the potential for seismic densification is low.

5.2.3 Lateral Spreading

Lateral spreading is a phenomenon in which a surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. The surficial blocks are transported downslope or in the direction of a free face, such as a channel, by earthquake and gravitational forces. Lateral spreading is generally the most pervasive and damaging type of liquefaction-induced ground failure generated by earthquakes.

We used the results of the laboratory tests performed on soil samples from the rotary wash borings, the CPT data, and the Revised Multilinear Regression Equations for Prediction of Lateral Spread Displacements (Youd et al. 2001) to evaluate the potential for lateral spreading. These regression equations indicate that sandy soil layers with $(N_1)_{60}$ values greater than 15 blows per foot may be moderately susceptible to soil liquefaction, but are sufficiently dense to resist the potential for lateral spreading (Youd et al 2001). Tables 4 and 5 indicate there are several layers with $(N_1)_{60}$ values less than 15; however, these layers appear to be discontinuous. In addition, the Guadalupe River (i.e., closest free face) is approximately 800 feet northwest of the site. Considering these conditions, we judge the potential for lateral spreading to be low.

5.2.4 Fault Rupture

Historically, ground surface ruptures closely follow the trace of geologically young faults. The site is not within an Earthquake Fault Zone, as defined by the Alquist-Priolo Earthquake Fault Zoning Act and no known active or potentially active faults exist on the site. Therefore, we conclude the risk of fault offset through the site from a known active fault is low. In a seismically active area, the remote possibility exists for future faulting in areas where no faults previously existed; however, we conclude that the risk of surficial ground deformation from faulting at the site is low.

5.2.5 Tsunami

Recent published maps (California Emergency Agency, 2009) indicate the project site is not within the tsunami inundation zone; therefore, we conclude the potential risk by inundation from tsunami to be low within the project site. However, the project civil engineer should evaluate the impact of sea level rise on the potential risk of inundation from a tsunami.

6.0 DISCUSSION AND CONCLUSIONS

From a geotechnical standpoint, the proposed project is feasible provided the site conditions and geotechnical issues discussed below are properly addressed during the design and construction of the proposed building. The primary geotechnical issues include:

- the presence of near surface expansive soil

- the presence of moderately compressible alluvial deposits
- the presence of shallow groundwater
- the potential for liquefaction-induced settlement

These issues and their impact on the geotechnical aspects of the project are discussed in the following subsections.

6.1 Expansive Soil Considerations

The near-surface soil encountered during our 2019 subsurface exploration has very high expansion potential. Moisture fluctuations in the near-surface expansive soil could cause the soil to expand or contract resulting in movement and potential damage to improvements that overlie them. Potential causes of moisture fluctuations include drying during construction, and subsequent wetting from rain, capillary rise, landscape irrigation, and type of plant selection.

The volume changes from expansive soils can cause cracking of foundations, floor slabs, and exterior flatwork. Any new foundations, exterior slabs, and concrete flatwork areas should be designed and constructed to resist the effects of the expansive soil. These effects can be mitigated by moisture conditioning the expansive soil, providing select, non-expansive fill below flatwork and other at-grade improvements, and providing additional reinforcing steel. In addition new foundations can be deepened to reduce the effects of expansive soil.

An alternative to importing select fill includes lime treatment of the near-surface soil. Lime treatment can reduce the swell potential and increase the shear strength of the soil. Lime stabilization of the at-grade building pad and the subgrade of exterior flatwork and pavement may be a cost-effective means of improving on-site soils for use as non-expansive fill within the building pad.

Furthermore, if the surface soil becomes wet, it may be difficult to compact during the winter. If required, the soil can be mixed with lime to aid in compaction.

The degree to which lime will react with soil depends on such variables as type of soil, minerals present, quantity and type of lime, and the length of time the lime-soil mixture is cured. The quantity of lime added generally ranges from 5 percent to 7 percent by weight and should be determined by laboratory testing. If lime treatment is intended to reduce swelling potential and/or increase the strength of the soil, the lime treatment contractor should collect a bulk sample of the soil and perform laboratory tests to determine if the lime will react with the soil, the amount of lime required and the resulting plasticity index. If implemented, we should be provided with the results to evaluate the effectiveness of the lime treatment.

6.2 Settlement and Foundations

The primary considerations related to the selection of appropriate foundation systems are:

- the presence of the very highly expansive near-surface material,
- moderately compressible soil,
- potentially liquefiable sand layers, and
- anticipated building settlements.

If a shallow foundation such as footings is selected the footings will need to be deepened to reduce the effects of highly expansive soil. A continuous perimeter footing should be used to reduce moisture changes beneath the building. In addition during construction foundation excavations should be kept moist to avoid shrinkage cracks forming in the soil. If shrinkage cracks form, the soil will need to be moisture conditioned, which may not be practical or overexcavated, thereby enlarging footing excavations.

The proposed building site is susceptible to the following potential sources of settlement:

- compression of undocumented fill at the former stormwater management area in the eastern part of the site,
- consolidation of the underlying alluvial deposits under the weight of new building loads or new fill, and
- liquefaction-induced settlement as discussed in Section 5.2.

A stormwater depression was formerly located in the eastern part of the proposed building's footprint; see Figure 2 for the approximate location. According to available record survey data by Level 10, we estimate the depression extended approximately 3 to 6½ feet below existing site grades. Documentation of the fill placement and compaction is not available; therefore, we do not know if the fill was properly placed. Consequently, assuming that the fill was not engineered in place, we estimate up to one inch of settlement could occur in the fill where shallow footings are resting on the undocumented fill in the former stormwater depression area.

Based on the available site grading and drainage plan (HMH, 2022), we understand that site grading will typically be minor with fill placements on the order of about two feet or less. If new fill is placed to grade the site, we estimate approximately ¼ and ½ inch of consolidation settlement could occur for one and two feet of new fill, respectively.

Based on the provided structural loads and building layout, we conclude that a shallow foundation system consisting of continuous perimeter footings and isolated spread footing bearing on natural or improved soils is feasible provided that the estimated static settlements in this section and seismic settlements per Section 5.2 are acceptable. The amount of settlement associated with a shallow foundation system will depend on the design bearing pressure, bottom of footing elevation, the type and condition of the underlying soils, and the footing sizes.

We estimate that total static settlements for shallow foundations bearing on natural (i.e., unimproved) ground with allowable bearing pressure of 3,000 pounds per square foot (psf) could be up to about 1¾ inch; the estimated static settlements are in addition to the consolidation settlement from site grading and seismically-induced settlements previously discussed.

Based on our discussion with the project team, we understand the project team would like to limit total static settlements to about one inch. To limit static settlement, the footings supported on native, unimproved soil would need to be enlarged (i.e., to lower the bearing pressure) or the soil below the footings be improved.

We understand that the project team prefers to support the building on shallow footings bearing on improved ground after reviewing preliminary construction logistics and premiums. Ground improvement systems can also be used for slab support, however, we understand that other measures (i.e., site grading and subgrade preparation, thickened slab, etc.) are preferred. An additional discussion about ground improvement for foundation support is included in the following subsection.

6.2.1 Ground Improvement

Two common alternatives to reduce the static settlement of the proposed structure are to use cement-treated, compacted aggregate piers (CAPs) or drilled displacement columns (DDCs) to strengthen the compressible soil. The CAPs or DDCs strengthen the soil matrix with a grid of shafts filled with compacted select aggregate material or controlled low-strength material (CLSM), respectively. Shallow foundations can then be used on top of the CAPs or DDCs. CAP and DDC systems are installed under design-build contracts by specialty contractors.

CAPs are constructed using a drilling tool to remove or displace the compressible soil and replace it with select material, such as crushed rock, that is compacted in lifts. Cement-treatment can be performed to preclude bulging of the aggregate pier and limit additional settlement. Installation of CAPs produces soil cuttings if the soil is removed from the shaft during column installation and not displaced laterally. Typically, CAPs are 18 to 30 inches in diameter.

DDCs are constructed using a displacement auger to create a shaft that is filled with CLSM injected under pressure as the displacement auger is withdrawn. Installation of DDCs produces minimal soil cuttings because the soil is displaced during column installation, and some densification occurs in the soil between the columns. Typically, DDCs are 16 to 24 inches in diameter. Because DDCs inject the CLSM under pressure, there is the potential for soil heave near the DDC element. To eliminate the potential to damage nearby improvements, DDCs may need to be set back a horizontal distance from adjacent improvements, including buildings, slabs, or utilities.

Because the CAP and DDC systems are installed by specialty design-build contractors, we do not provide specific design recommendations or settlement estimates for these systems. We understand that a DDC ground improvement system is likely quicker and more economical to install, and therefore is preferred to support the shallow foundations for this project. Based on discussions with a local specialty design-build contractor, shallow foundations will likely be supported on DDCs installed to depths of about 15 to 25 feet below existing ground surface, which will limit total static settlements to less than one inch, with differential settlements of less than ½ inch over a distance of 30 feet. The DDC ground improvement system would not target improvement of the potentially liquefiable layers, but would likely provide some benefit. The amount of seismic settlement reduction will depend on the depth and spacing of the DDCs.

Additional information about preliminary bearing pressures for the improved ground and subgrade modulus recommendations (by others) are included in Section 7.2.

6.3 Groundwater and Dewatering Considerations

The historic high groundwater level in the project vicinity has been observed as high as approximately 10 feet bgs (California Geological Survey, 2002). Based on groundwater measurements during our investigation, we judge static groundwater levels range from approximately 10 to 15 feet bgs, corresponding to approximately Elevation 17 to 14 feet. Therefore, we conclude a design groundwater elevation of Elevation 17 feet should be used.

6.4 Corrosion Potential

CERCO Analytical performed tests on two soil samples from the site in 2019 to evaluate corrosion potential to buried metals and concrete. The results of the tests are presented in Appendix D and summarized in Table 6.

TABLE 6
Summary of Corrosivity Test Results

Test Boring	Sample Depth (feet)	pH	Sulfate (mg/kg)	Resistivity (ohms-cm)	Redox (mV)	Chloride (ppm)
B-1	1 to 4	8.45	140	740	280	N.D
B-5	6	8.37	37	1,200	220	N.D.

N.D. = None Detected

Based upon resistivity measurements, the soil samples tested are classified as "corrosive" to buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron. All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion. For more detailed recommendations regarding the corrosion protection of buried metals and concrete, a licensed corrosion consultant should be retained.

A brief evaluation of the corrosivity of the soil samples is presented in Appendix D. For more detailed recommendations regarding the corrosion protection of buried metals and concrete, a licensed corrosion consultant should be retained.

7.0 RECOMMENDATIONS

From a geotechnical standpoint, the site can be developed as planned, provided the estimated static settlements discussed in Section 6.2 and liquefaction-induced settlements discussed in Section 5.2.1 are tolerable and the recommendations presented in this section of the report are incorporated into the design and contract documents. Criteria for foundation design, together with recommendations for site preparation, floor slabs, site retaining walls, fill placement, utilities, pavement sections, and seismic design are presented in this section of the report.

7.1 Site Preparation and Earthwork

Existing pavements, old building foundations, abandoned utilities, and other obstructions should be removed from areas to receive improvements. We anticipate the excavations for this project can be made using conventional earth-moving equipment except where old foundations and other buried obstructions are encountered. These may require hoe rams or jackhammers to remove. Any parts of existing buried foundations or walls that could interfere with the proposed improvements should be broken off and removed.

Where utilities to be removed extend off site, they should be capped or plugged with grout at the property line. It may be feasible to abandon utilities in-place, outside the proposed building footprint provided they will not interfere with future utilities or building foundations. If utilities are abandoned in-place, they should be completely filled with flowable cement grout over their entire length. Existing utility lines, where encountered, should be addressed on a case-by-case basis.

From a geotechnical standpoint, asphalt and concrete removed from the site may be crushed and reused provided it is free of organic material and rocks or lumps greater than three inches in greatest dimension. The acceptability of using crushed asphalt at the site should be verified by the property owner, architect, and environmental consultant. Where crushed asphalt pavement materials are used, particles between 1½ and 3 inches in greatest dimension should comprise no more than 20 percent of the fill by weight.

Where used, sand containing less than 10 percent fines (particles passing the No. 200 sieve) should also be compacted to at least 95 percent relative compaction. Samples of on-site and proposed import fill materials should be submitted to Langan for approval at least three business days prior to use at the site.

7.1.1 Site Grading for Expansive Soil

Where highly expansive clay is encountered, the near surface soil should be moisture conditioned to three to five percent above the optimum moisture content and compacted to between 88 and 93 percent relative compaction[8]. To reduce the effects of expansive soil, we recommend at least 24 inches of imported (select) material or lime treated soil be placed beneath the building footprint; the select fill should extend at least five feet beyond building footprint. Prior to placement of select fill in building areas, the on-site soil exposed by stripping should be scarified

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[8] Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D1557 (latest edition) laboratory compaction procedure.

to a depth of at least 12 inches, moisture-conditioned to between three to five percent above optimum moisture content, and compacted to between 88 and 93 percent relative compaction. The soil subgrade should be kept moist until it is covered by select fill.

If site grading occurs in late summer or in fall, the surface soil may be dry to depths exceeding 12 inches. Therefore, prior to grading, we should perform moisture content tests on the upper three feet of soil in building areas. Surface soil that has a moisture content of less than 20 percent (i.e., the approximate plastic limit of the soil) should be excavated, moisture-conditioned to at least three to five percent above optimum moisture content, and compacted to between 88 percent and 93 percent relative compaction to reduce its expansion potential. Based on our experience in the project area, we judge the maximum depth of required excavation for moisture conditioning will be approximately two feet.

All select fill placed beneath improvements should meet the following criteria:

- be free of organic matter
- contain no rocks or lumps larger than three inches in greatest dimension
- have a low expansion potential (defined by a liquid limit of less than 40 and plasticity index lower than 12)
- have a low corrosion potential[9]
- be approved by the geotechnical engineer.

In addition, the select fill should contain at least 20 percent fines (particles passing the No. 200 sieve) to reduce the potential for surface water to infiltrate beneath slabs. The on-site soils encountered in our 2019 exploration do not meet the requirements of select fill.

Select fill should be placed in lifts not exceeding eight inches in loose thickness, moisture-conditioned to near optimum moisture content, and compacted to at least 90 percent relative compaction. The subgrade should be rolled to a firm, non-yielding surface. If the compacted subgrade is disturbed during utility trench or foundation excavations, the subgrade should be re-rolled to provide a smooth, firm surface for concrete slab support.

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[9] Low corrosion potential is defined as a minimum resistivity of 2,000 ohms-cm and maximum sulfate and chloride concentrations of 250 parts per million.

Where utility trenches backfilled with sand or gravel enter the building pad, an impermeable plug consisting of native clay or lean concrete, at least five feet in length, should be installed at the building line. Further, where sand- or gravel-backfilled trenches cross planter areas and pass below asphalt or concrete pavements, a similar plug should be placed at the edge of the pavement. The purpose of these plugs is to reduce the potential for water to become trapped in trenches beneath the building or pavements. This trapped water can cause heaving of soils beneath slabs and softening of subgrade soil beneath pavements.

7.1.2 Site Grading for the Former Stormwater Management Area

The former stormwater management area in the eastern part of the site was filled with undocumented fill. As discussed in Section 6.2, the fill could settle excessively and is not suitable for foundation or slab support. Therefore, the former stormwater management area should be overexcavated until competent native material is exposed. The subgrade should then be scarified and compacted as recommended herein before placing and compacting engineered fill in lifts. The existing, on-site material can be reused as engineered fill provided it has a low expansion potential, contains no rocks or lumps larger than three inches in the largest dimension, and is approved by the geotechnical engineer. We should review the on-site soil during construction to check its suitability.

7.2 DDC-Supported Shallow Foundations

As discussed in Section 6.2, if the estimated total and differential settlements are tolerable, the proposed structures can be supported on shallow foundations supported on DDCs. DDCs are designed by a specialty design-build contractor; therefore, we cannot provide specific design recommendations or settlement estimates for these systems. Our geotechnical report should be provided to the design-build contractor to provide final foundation design plans and we should be retained to provide technical input and review of the design prior to construction.

We reviewed the site and planned development with a local specialty design-build subcontractor to obtain preliminary DDC lengths, allowable bearing pressures, and estimated modulus of subgrade reaction. The final design values should be determined by the selected design-build subcontractor. Per the local design-build subcontractor, allowable bearing pressures for shallow foundations supported on about 15- to 25-foot-long DDC ground improvement elements will likely be on the order of 5,000 psf for dead plus live loads. The anticipated allowable bearing pressure includes a factor of safety 2, and can be increased by one-third for total (i.e., wind or seismic) loads. The modulus of subgrade reaction will likely be on the order of 35 pounds per cubic inch (pci) for dead plus live conditions and 70 pci for total loads based on the design-build subcontractor's estimate.

The ultimate resistance of DDCs should be verified by at least three load tests in compression. The test locations should be selected by the ground improvement contractor and approved by the geotechnical engineer and the structural engineer. The compression load tests should be performed in accordance with ASTM D1143, latest edition, *Standard Test Method for Piles Under Static Axial Compressive Load*. Equipment used for the test (load frame, jacks, and reaction piles) should be capable of applying at least 2 times the allowable dead plus live design load, at least 1.5 times the total load, and at least 1.1 times the calculated ultimate resistance. At the conclusion of load testing, the ultimate resistance from the load tests should be used to verify the design ultimate resistance and other design parameters.

The footings should be at least 18 inches wide for continuous footings and 24 inches wide for isolated spread footings. To reduce the potential for movement of the footings due to shrink and swell of the expansive clay, we recommend that a continuous perimeter footing be used, and that perimeter and interior footings be deepened to bottom at least 36 and 30 inches, respectively, below the lowest adjacent soil subgrade (not including the capillary moisture break, if used). Footings adjacent to utility trenches or other footings should bear below an imaginary 1.5H:1V (horizontal to vertical) plane projected upward from the bottom edge of the utility trench or adjacent footings.

Lateral forces can be resisted by a combination of friction along the base of the footing and passive resistance against the embedded vertical faces of the foundation. To provide a uniform distribution of the foundation loads, a load transfer pad, typically consisting of about 12 inches of compacted, open-graded angular crushed rock should be placed above the ground improvement elements. The load transfer platform and its thickness should be designed by the design/build contractor. To calculate the passive resistance against the vertical faces of the footings, we recommend a uniform pressure (i.e., rectangular distribution) of 1,200 psf. The upper foot of passive resistance should be ignored unless confined by a concrete slab. The value for passive pressures includes a factor of safety of 1.5. Frictional resistance against the base of the footings should be calculated based on parameters provided by the design-build subcontractor.

The exposed subgrade for the footings should be free of standing water, debris, and disturbed materials prior to constructing the footing. We should check the footing subgrade after cleaning, but prior to placement of reinforcing steel to confirm bearing, moisture condition, and that loose and disturbed material has been removed. If loose or disturbed material is observed in the footing excavation, it should be overexcavated to firm, competent material and replaced with lean concrete or engineered fill. Maintaining proper moisture will likely require wetting the excavations periodically until the concrete is placed; if the soil becomes desiccated and cracks form, it may be necessary to overexcavate to remove the desiccated soil.

Positive surface drainage should be provided around the building to direct surface water away from the foundations. In addition, roof downspouts should be discharged into controlled drainage facilities to keep the water away from the foundations. As discussed in Section 7.13, unlined bioretention systems should be set back a minimum of 5 feet away from building foundations. If subdrains are used to keep water away from foundations, they should be placed above the groundwater table.

7.3 Site Retaining Wall Design

If needed, we recommend site retaining walls be designed to resist lateral pressures imposed by the adjacent soil and vehicles. Walls that are free to rotate (active condition) or restrained (at-rest condition) and backfilled with select fill may be designed using the equivalent fluid pressures presented in Table 7. Because the site is in a seismically active area, the design should also be checked for seismic conditions for structures assigned to Seismic Design Category D, E, or F and retaining more than 6 feet of backfill height[10]. Under seismic loading conditions, there will be a seismic pressure increment that should be added to active earth pressures. We used the procedures outlined by Sitar (2012) and the peak ground acceleration based on the Design Earthquake ground motion level to compute the seismic pressure increment. For seismic conditions, retaining walls should be designed for the more critical loading condition of restrained (at-rest) pressure or total pressures (active plus seismic increment) using the equivalent fluid weights and pressures presented in Table 7.

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[10] California Building Code (2019) Section 1803.5.12.

TABLE 7

Retaining Wall Design Earth Pressures (Select Fill and Drained Conditions)

	Static Conditions		Seismic Conditions[2]
Condition	**Unrestrained Walls (Active)**	**Restrained Walls (At-rest)**	**Total Pressure – Active Plus Seismic Pressure Increment**
Above Groundwater[1]	35 pcf	50 pcf	55 pcf
Below Groundwater[1]	75 pcf	80 pcf	90 pcf

Notes:

1. Recommended design groundwater elevation is Elevation 17 feet (NAVD88 datum).
2. The more critical condition of either at-rest pressure for static conditions or active pressure plus a seismic pressure increment for seismic conditions should be checked.
3. Assumes backfill behind retaining wall is select fill; criteria for select fill is presented in Section 7.1.

If the retaining wall will support native soil or native soil is used as backfill then we should provide additional earth pressures for design.

Where traffic will pass within 10 feet of retaining walls, temporary traffic loads should be considered in the design of the walls. Traffic loads may be modeled by a uniform pressure of 100 psf applied in the upper 10 feet of the walls. If heavy trucks will operate within 10 feet of retaining walls, the temporary traffic load should be increased and modeled as a uniform pressure of 250 psf applied in the upper 10 feet of the walls.

The retaining walls should be supported on shallow, spread footings bearing on firm, native soil or engineered fill. The bottom of the footings should be embedded at least 36 inches below the lowest adjacent soil subgrade and should be at least 18 inches wide for continuous footings and 24 inches for isolated spread footings. Footings adjacent to utility trenches (or other footings) should bear below an imaginary 1.5H:1V (horizontal to vertical) plane projected upward from the bottom edge of the utility trench (or adjacent footings).

For the recommended minimum embedment, the retaining wall footings bearing on firm native soil may be designed for an allowable bearing pressure of 3,000 pounds per square foot (psf) for dead plus live loads, with a one-third increase for total loads, including wind and/or seismic loads.

Lateral loads on retaining wall footings can be resisted by a combination of passive resistance acting against the vertical faces of the footings and friction along the bases of the footings. Passive resistance may be calculated using a uniform pressure of 1,200 psf; the upper foot of soil should be ignored unless confined by a concrete slab or pavement. Frictional resistance should be computed using a base friction coefficient of 0.30. This value includes a factor of safety of about 1.5.

The lateral earth pressures given assume the walls are properly backdrained above the water table to prevent the buildup of hydrostatic pressure. If the walls are not drained, they should be designed for an equivalent fluid weight of 90 pounds per cubic foot (pcf) to account for hydrostatic pressure. One acceptable method for backdraining the walls is to place a prefabricated drainage panel against the back side of the wall. The drainage panel should extend to a perforated PVC collector pipe. The pipe should be surrounded on all sides by at least 4 inches of Caltrans Class 2 permeable material and should be sloped to drain into an appropriate outlet. We should check the manufacturer's specifications for the proposed drainage panel material to verify it is appropriate for its intended use.

If backfill is required behind retaining walls, the walls should be braced or hand-compaction equipment used to prevent unwanted surcharges on the walls.

7.4 Floor Slabs

Concrete floor slabs supported at-grade should be at least 6 inches thick, reinforced, and designed to accommodate the anticipated static settlements from site grading (up to about ½ inch). We understand that the project team does not intend to mitigate the potential for seismic settlements below the floor slabs, and consequently, the floor slab may need to be repaired following a major earthquake on a nearby fault. Increasing the floor slab thickness and adding reinforcing steel should improve the performance and resiliency of the slab.

Because expansive soil is present near the existing ground surface, the slabs on-grade should be underlain by 24 inches of select fill (or lime treated soil) and the subgrade should be prepared in accordance with Section 7.1. If the subgrade is disturbed during excavation for footings and utilities, it should be re-rolled. Where soft or loose soil is present at the subgrade elevation prior to placing select fill, the weak soil should be removed and replaced with engineered fill or lean concrete.

Moisture is likely to condense on the underside of the ground floor slabs, even though they will be above the design groundwater level. Consequently, a moisture barrier should be considered if movement of water vapor through the slabs would be detrimental to its intended use. A typical moisture barrier consists of a capillary moisture break and a water vapor retarder.

The capillary moisture break should consist of at least four inches of clean, free-draining gravel or crushed rock. The vapor retarder should meet the requirements for Class C vapor retarders stated in ASTM E1745-97. The vapor retarder should be placed in accordance with the requirements of ASTM E1643-98. These requirements include overlapping seams by six inches, taping seams, and sealing penetrations in the vapor retarder. The particle size of the gravel/crushed rock should meet the gradation requirements presented in Table 8.

TABLE 8
Gradation Requirements for Capillary Moisture Break

Sieve Size	Percentage Passing Sieve
Gravel or Crushed Rock	
1 inch	90 – 100
3/4 inch	30 – 100
1/2 inch	5 – 25
3/8 inch	0 – 6

Concrete mixes with high water/cement (w/c) ratios result in excess water in the concrete, which increases the cure time and results in excessive vapor transmission through the slab. Therefore, concrete for the floor slab should have a low w/c ratio - less than 0.45. The slab should be properly cured. Before the floor covering is placed, the contractor should check that the concrete surface and the moisture emission levels (if emission testing is required) meet the manufacturer's requirements.

7.5 Seismic Design

For seismic design in accordance with the provisions of 2019 California Building Code (CBC), a site-specific response analysis is required to be performed for Site Class D, unless the structural exceptions in Section 11.4.8 of ASCE 7-16 are met. If the exceptions are met, the following parameters may be used for seismic design:

- Risk-Targeted Maximum Considered Earthquake (MCE_R) S_s and S_1 of 1.500g and 0.600g, respectively.
- Site Class D
- Site Coefficients Fa and Fv of 1.0 and 1.7

- MCER spectral response acceleration parameters at short periods, S_{MS}, and at one-second period, S_{M1}, of 1.500g and 1.020g, respectively.
- Design Earthquake (DE) spectral response acceleration parameters at short period, S_{DS}, and at one-second period, S_{D1}, of 1.000g and 0.680g, respectively.
- PGAM is 0.645g.

If the exceptions are not met, a site-specific response analysis is required. At the request of the project team, we performed probabilistic seismic hazard analysis (PSHA) and deterministic analysis to develop recommended horizontal spectra at the ground surface for the MCE_R and DE consistent with ASCE 7-16 and 2019 CBC. Additional details about our site-specific seismic analysis are presented in Appendix E.

The recommended spectra are presented on Figure E-8 for 5 percent damping; digitized values of the MCE_R and DE spectra, respectively, for damping ratio of 5 percent are presented in Table 9.

TABLE 9
Recommended MCE_R, and DE Spectra
Spectral Acceleration (g's)

Period (seconds)	MCE_R (5 percent damping)	DE (5 percent damping)
0.01	0.689	0.459
0.10	1.050	0.700
0.20	1.484	0.989
0.30	1.733	1.155
0.40	1.805	1.203
0.50	1.789	1.193
0.75	1.556	1.037
1.00	1.431	0.954
1.50	1.076	0.717
2.00	0.843	0.562
3.00	0.575	0.383
4.00	0.407	0.271
5.00	0.299	0.200

Note:

1. DE and MCE_R correspond to the Design Earthquake and Risk-Targeted Maximum Considered Earthquake, respectively, per CBC 2019/ASCE 7-16.

Because site-specific procedure was used to determine the recommended response spectra, the corresponding values of S_{MS}, S_{M1}, S_{DS} and S_{D1} per Section 21.4 of ASCE 7-16 should be used, as shown in Table 10.

TABLE 10
Design Spectral Acceleration Value

Parameter	Spectral Acceleration Value (g's)
S_{MS}	1.624[11]
S_{M1}	1.724[12]
S_{DS}	1.083[11]
S_{D1}	1.149[12]

7.6 Utilities and Utility Backfill

Utility trenches should be excavated a minimum of 4 inches below the bottom of pipes or conduits and have clearances of at least 4 inches on all sides. Where necessary, trench excavations should be shored and braced to prevent cave-ins and/or in accordance with safety regulations. If trenches extend below the groundwater level, it will be necessary to temporarily dewater them to allow for placement of the pipe and/or conduits and backfill.

To provide uniform support, pipes or conduits should be bedded on a minimum of 4 inches of sand or fine gravel. After pipes and conduits are tested, inspected (if required), and approved, they should be covered to a depth of 6 inches with sand or fine gravel, which should then be mechanically tamped. Backfill should be placed in lifts of 8 inches or less, moisture-conditioned, and compacted to at least 90 percent relative compaction. If fill with less than 10 percent fines is used, the entire depth of the fill should be compacted to at least 95 percent relative compaction. Jetting of trench backfill should not be permitted. Special care should be taken when backfilling utility trenches in pavement areas. Poor compaction may cause excessive settlements resulting in damage to the pavement section.

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[11] S_{MS} and S_{DS} are based on the site-specific response spectra and are based on 90 percent of the maximum spectral acceleration within the period range of 0.2 to 5 seconds; they are governed by 90 percent of the spectral acceleration at a period of 0.4 second.

[12] S_{M1} and S_{D1} are based on the site-specific response spectra and are the maximum of the product of period, T, and spectral acceleration, Sa, for periods from 1.0 to 5.0 seconds; they are governed by the product of the period and spectral acceleration at a period of 3.0 seconds.

As discussed in Section 7.1.1, where utility trenches backfilled with sand or gravel enter the building pad, an impermeable plug consisting of native clay or lean concrete, at least 5 feet in length, should be installed at the building line. Further, where sand- or gravel-backfilled trenches cross planter areas and pass below asphalt or concrete pavements, a similar plug should be placed at the edge of the pavement. The purpose of these plugs is to reduce the potential for water to become trapped in trenches beneath the building or pavements. This trapped water can cause heaving of soils beneath slabs and softening of subgrade soil beneath pavements.

The corrosivity results provided in Appendix D of this report should be reviewed and corrosion protection measures used, if needed. We recommend a corrosion engineer be retained when detailed corrosion protection recommendations are needed.

7.7 Asphalt Pavements

The State of California flexible pavement design method was used to develop the recommended asphalt concrete pavement sections. We expect the final soil subgrade in asphalt-paved areas will generally consist of on-site soil. On the basis of the laboratory test results we selected an R-value of 12 for design.

For our calculations, we assumed a Traffic Index (TI) of 4 for automobile parking areas with occasional trucks, and 5 and 6 for driveways and truck-use areas; these TIs should be confirmed by the project civil engineer. Table 11 presents our recommendations for asphalt pavement sections.

TABLE 11
Pavement Section Design

TI	Asphalt Concrete (inches)	Class 2 Aggregate Base R = 78 (inches)
4	2.5	7
5	3	8.5
6	3.5	11.5

Pavement components should conform to the current Caltrans Standard Specifications. The upper 6 inches of the soil subgrade in pavement areas should be moisture-conditioned to above optimum and compacted to at least 95 percent relative compaction and rolled to provide a smooth non-yielding surface. Aggregate base (AB) should be compacted to at least 95 percent relative compaction.

7.8 Concrete Pavements (Vehicular)

Concrete pavement design is based on a maximum single-axle load of 20,000 pounds and a maximum tandem axle of 32,000 pounds. According to past correspondence with HMH, concrete pavements will be designed for a TI of 13, the recommended rigid pavement section for these axle loads is 6 inches of Portland cement concrete over 6 inches of Caltrans Class 2 AB. The concrete pavement section should rest on at least 12 inches of select fill; the upper 4 inches of select fill can consist of the AB.

The modulus of rupture of the concrete should be at least 500 psi at 28 days. Contraction joints should be constructed at 15-foot spacing. Because the near surface soils are highly expansive, we recommend construction and expansion joints be dowelled. Where the outer edge of a concrete pavement meets asphalt pavement, the concrete slab should be thickened by 50 percent at a taper not to exceed a slope of 1 in 10. For loading docks, we recommend the slab be reinforced with a minimum of No. 4 bars at 16-inch-spacing in both directions. Recommendations for subgrade preparation and AB compaction for concrete pavement are the same as those we have described for asphalt pavement in Section 7.7.

7.9 Concrete Flatwork (Non-Vehicular)

We recommend new sidewalks and concrete flatwork (in non-vehicular traffic area) be underlain by at least 4 inches of Class 2 AB material (or the minimum thickness per City of San Jose Standards) that has been compacted to at least 90 percent relative compaction. To further reduce the potential for shrink/swell cracking, exterior slabs should be underlain by 12 inches of select fill; the upper 4 inches of select fill can consist of the AB. The select fill should extend at least 2 feet beyond the edge of slabs. Even with 12 inches of select fill, these slabs may experience some cracking due to shrinking and swelling of the underlying expansive soil. Thickening the slabs and adding additional reinforcement will control this cracking to some degree. In addition, where slabs provide access to buildings, it would be prudent to dowel the entrance to the building to permit rotation of the slab as the exterior ground shrinks and swells and to prevent a vertical offset at the entries.

7.10 Pavers

Interlocking pavers (assumed to have minimum thickness of 2.375 inch) should be placed on 2 inches of sand overlying a concrete sub-slab (where required) and Class 2 AB. In addition, the paver section should rest on at least 12 inches of select fill. For pavers used in pedestrian walkways, the pavers should be placed on two inches of sand overlying four inches of Class 2 AB.

For vehicular traffic, the required thickness of the concrete sub-slab and Class 2 AB are presented in Table 12.

TABLE 12
Interlocking Paver Section Design for Vehicular Use

TI	Concrete Sub-Slab Thickness (inches)	Class 2 Aggregate Base R = 78 (inches)
4	0	7
5	3½	8
6	5	9

Where a concrete sub-slab is recommended, the concrete slab should have minimal reinforcement (such as No. 3 steel reinforced bars placed 18 inches on center in both horizontal directions). Because the near surface soils are highly expansive, we recommend that the construction and expansion joints be dowelled.

We recommend the paver manufacturer be consulted to confirm the pavers selected are rated for heavy traffic loads. The paver manufacturer should also confirm whether or not pavers should be flush at the joints and whether mortar should be used if the pavers will be subject to heavy traffic loading.

The upper 6 inches of the soil subgrade in pavement areas should be moisture-conditioned to above optimum and compacted to at least 95 percent relative compaction. AB should conform to current Caltrans Standard Specifications. All AB should be compacted to at least 95 percent relative compaction.

7.11 Site Drainage

Positive surface drainage should be provided around the building to direct surface water away from building foundations. To reduce the potential for water ponding adjacent to the building, we recommend the ground surface within a horizontal distance of 5 feet from the buildings be designed to slope down and away from the building with a surface gradient of at least 2 percent in unpaved areas and 1 percent in paved areas. In addition, roof downspouts should be discharged into controlled drainage facilities to keep the water away from the foundations.

7.12 Landscaping

The use of water-intensive landscaping around the perimeter of the buildings should be avoided to reduce the amount of water introduced to the subgrade. Irrigation of landscaping around the building should be limited to drip or bubbler-type systems. Trees with large roots or that have high water demand should also be avoided since they can dry out the soil beneath foundations and cause settlement. The purpose of these recommendations is to avoid large differential moisture changes adjacent to the foundations, which have been known to cause significant differential movement over short horizontal distances in expansive soil, resulting in cracking of slabs and architectural damage.

To reduce the potential for irrigation water entering the pavement section, vertical curbs adjacent to landscaped areas should extend through any aggregate base and at least 6 inches into the underlying soil. In heavily watered areas, such as lawns, it may also be necessary to install a subdrain behind the curb to intercept excess irrigation water.

7.13 Bioretention Systems

Bioretention areas are landscaping features used to treat stormwater runoff within a development site. They are commonly located in parking lot islands and landscape areas. Surface runoff is directed into shallow, landscaped depressions, which usually include mulch and a prepared soil mix. Typically, the filtered runoff is collected in a perforated underdrain beneath the bioretention system and returned to the storm drain system. For larger storms, runoff will generally overflow the bioretention areas and is diverted to the storm drain system.

The soil within a bioretention system should typically have an infiltration rate sufficient to draw down any pooled water within 48 hours after a storm event. Bioretention soil should be installed in accordance with the Santa Clara County's C.3 stormwater technical guidelines and include an underdrain system with a waterproof liner on the sides and bottom of the bioretention swale.

Underdrains are typically at the invert of the bioretention system to intercept water that does not infiltrate into the surrounding soils. Underdrains consist of a perforated PVC pipe surrounded by two to three inches of Class 2 Permeable material (Caltrans Standard Specifications Section 68-2.02F(3)). The perforated PVC pipe cross-section area should be determined based on the desired hydraulic conductivity of the underdrain. Underdrains should be installed in accordance with the Santa Clara County's C.3 stormwater technical guidelines.

Because of the presence of near surface expansive soil, unlined bioretention systems should be set back a minimum of 5 feet from building foundations, slabs, concrete flatwork or pavements. If bioretention systems are closer than 5 feet, passive resistance of foundation elements should be neglected. Overflow from bioretention areas should be directed to the storm drain system away from building foundations and slabs.

Typically, the bottom of the bioretention system is recommended to be a minimum of 2 feet or more above the groundwater table.

8.0 ADDITIONAL GEOTECHNICAL SERVICES

Prior to construction, we should review the project plans and specifications to check their conformance with the intent of our geotechnical recommendations. During construction, we should observe the installation of ground improvement elements and shallow foundations, and the preparation of the building pad subgrade. We should also observe the subgrade preparation and any fill placement and perform field density tests to check that adequate moisture conditioning and fill compaction has been achieved beneath proposed sidewalk and pavement areas. These observations will allow us to compare the actual with the anticipated soil conditions and to check that the contractor's work conforms with the geotechnical aspects of the plans and specifications.

9.0 LIMITATIONS

The conclusions and recommendations presented in this report apply to the site and construction conditions as we have described them and are the result of engineering studies and our interpretations of the existing geotechnical conditions. Actual subsurface conditions may vary. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that described in this report, Langan should be notified so that supplemental recommendations can be developed. Our scope of services relates solely to the geotechnical aspects of the project and does not address environmental concerns.

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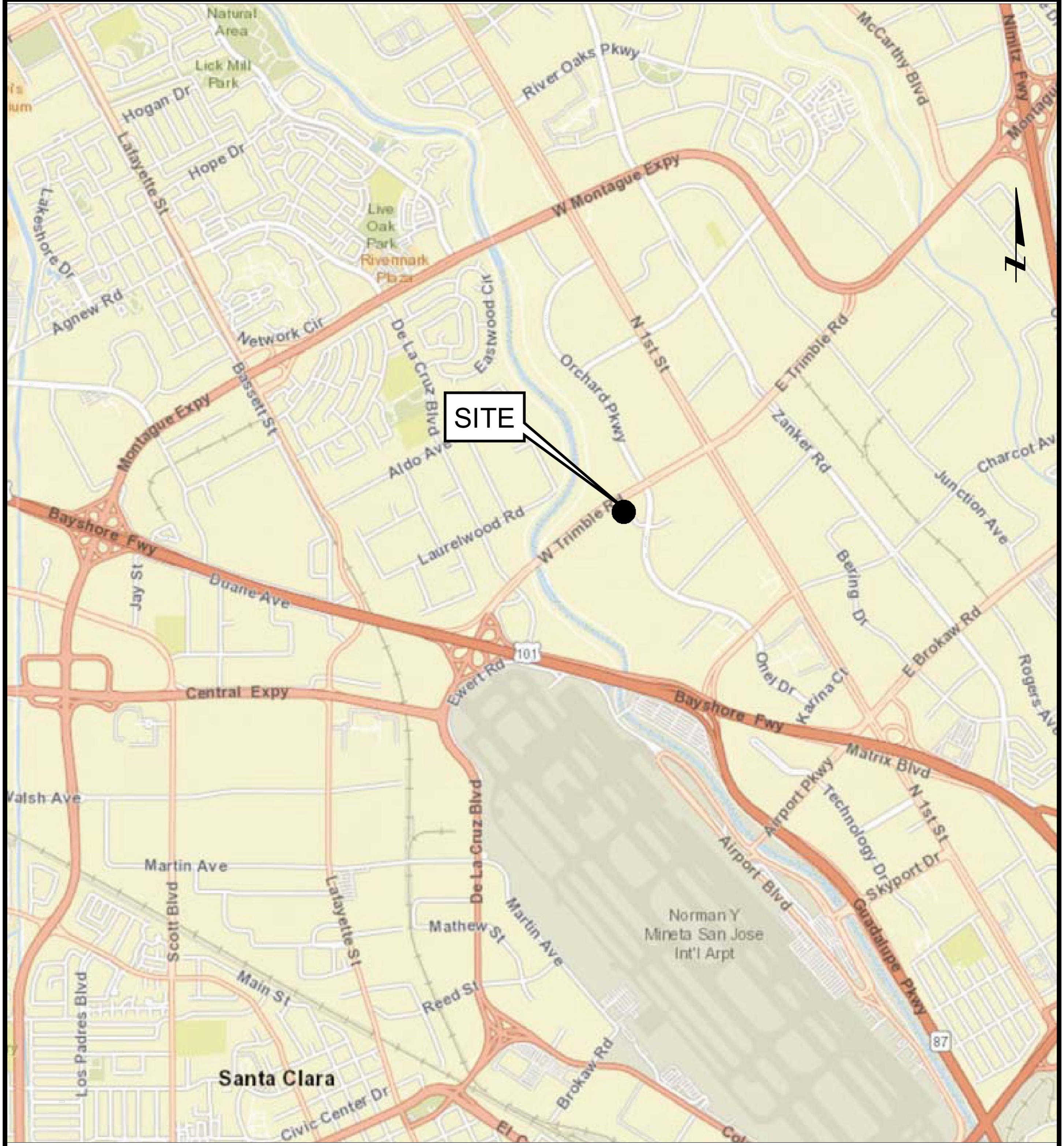
FIGURES



Note:
Topographic base map is provided through Langan's Esri
Arc GIS software licensing and Arc GIS online.
Credits: Sources: Esri, DeLorme, NAVTEQ, USGS, Intermap, iPC, NRCAN.

0 2,500 Feet

Approximate scale

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Project

ADVANCED
MANUFACTURING
BUILDING

330 W. TRIMBLE ROAD, SAN JOSE

SANTA CLARA COUNTY CALIFORNIA

Figure Title

SITE LOCATION
MAP

Project No.
770651906

Date
01/03/2023

Drawn By
JDF

Checked By
TL

Figure

1

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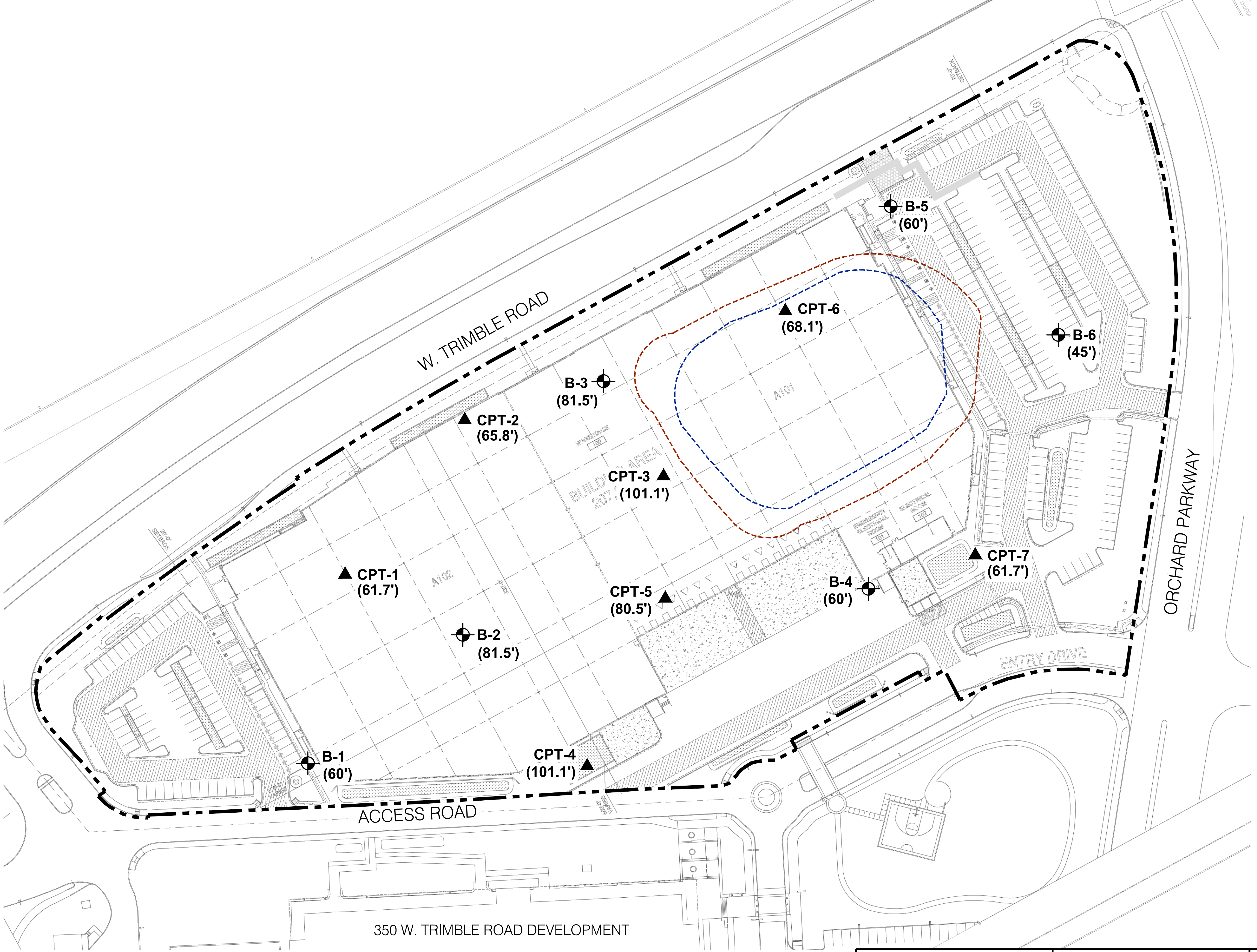

EXPLANATION

B-1 ⌖ **(60')**	Approximate location and depth of boring by Langan, March 2019
CPT-1 ▲ **(61.7')**	Approximate location and depth of cone penetration test by Langan, March 2019
▬ · ▬ ·	Approximate extent of project
- - - -	Approximate top of slope for former stormwater depression
- - - -	Approximate bottom of slope for former stormwater depression

Notes:

1. Reference 1: Base map by Ware Malcomb Sheet A100 "Advanced Manufacturing Building", 12/16/2022.
2. Reference 2: "Survey of Depression" by Level 10, October 2017.

0 100 Feet

Approximate scale

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Figure Title
SITE PLAN

Project No. 770651906
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Figure
2

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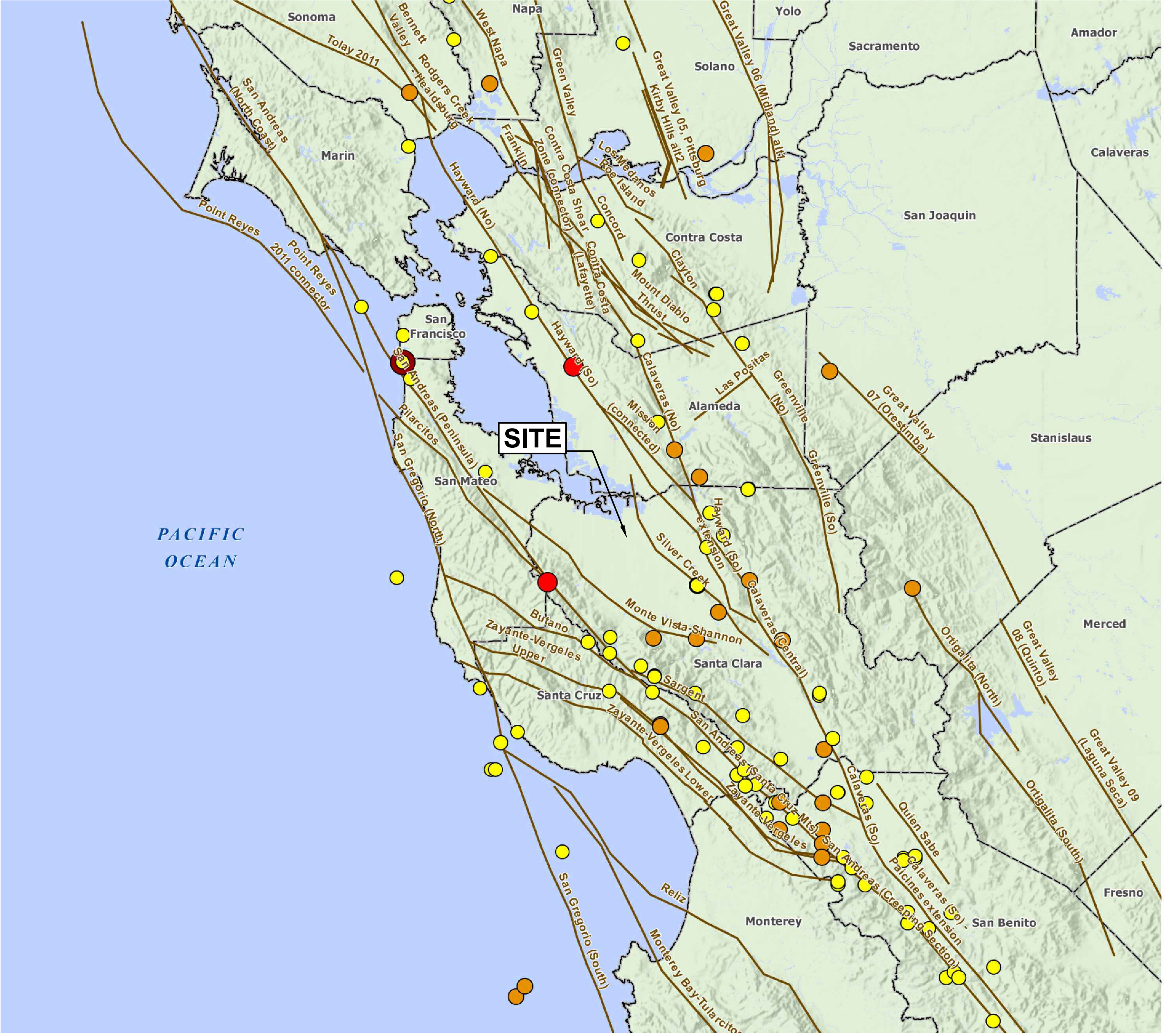

LEGEND

⬚	County Boundary
—	Fault

Earthquake Epicenter Magnitude

●	Magnitude 5 to 5.9
●	Magnitude 6 to 6.9
●	Magnitude 7 to 7.4
●	Magnitude 7.5 to 8

0 20 Miles

Approximate scale

Notes:

1. Quaternary fault data displayed are provided by the CGS Map Sheet 48: Fault based seismic sources used in the Uniform California Earthquake Rupture Forecast, Version (UCERF3).
2. The Earthquake Epicenter (Magnitude) data is provided by the U.S Geological Survey (USGS) and is current through 2015.
3. Basemap hillshade and County boundaries provided by USGS and California Department of Transportation.
4. Map displayed in California State Coordinate System, California (Teale) Albers, North American Datum of 1983 (NAD83), Meters.

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Figure Title
MAP OF MAJOR FAULTS AND EARTHQUAKE EPICENTERS IN THE SAN FRANCISCO BAY AREA

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Figure
3

I Not felt by people, except under especially favorable circumstances. However, dizziness or nausea may be experienced.

Sometimes birds and animals are uneasy or disturbed. Trees, structures, liquids, bodies of water may sway gently, and doors may swing very slowly.

II Felt indoors by a few people, especially on upper floors of multi-story buildings, and by sensitive or nervous persons.

As in Grade I, birds and animals are disturbed, and trees, structures, liquids and bodies of water may sway. Hanging objects swing, especially if they are delicately suspended.

III Felt indoors by several people, usually as a rapid vibration that may not be recognized as an earthquake at first. Vibration is similar to that of a light, or lightly loaded trucks, or heavy trucks some distance away. Duration may be estimated in some cases.

Movements may be appreciable on upper levels of tall structures. Standing motor cars may rock slightly.

IV Felt indoors by many, outdoors by a few. Awakens a few individuals, particularly light sleepers, but frightens no one except those apprehensive from previous experience. Vibration like that due to passing of heavy, or heavily loaded trucks. Sensation like a heavy body striking building, or the falling of heavy objects inside.

Dishes, windows and doors rattle; glassware and crockery clink and clash. Walls and house frames creak, especially if intensity is in the upper range of this grade. Hanging objects often swing. Liquids in open vessels are disturbed slightly. Stationary automobiles rock noticeably.

V Felt indoors by practically everyone, outdoors by most people. Direction can often be estimated by those outdoors. Awakens many, or most sleepers. Frightens a few people, with slight excitement; some persons run outdoors.

Buildings tremble throughout. Dishes and glassware break to some extent. Windows crack in some cases, but not generally. Vases and small or unstable objects overturn in many instances, and a few fall. Hanging objects and doors swing generally or considerably. Pictures knock against walls, or swing out of place. Doors and shutters open or close abruptly. Pendulum clocks stop, or run fast or slow. Small objects move, and furnishings may shift to a slight extent. Small amounts of liquids spill from well-filled open containers. Trees and bushes shake slightly.

VI Felt by everyone, indoors and outdoors. Awakens all sleepers. Frightens many people; general excitement, and some persons run outdoors.

Persons move unsteadily. Trees and bushes shake slightly to moderately. Liquids are set in strong motion. Small bells in churches and schools ring. Poorly built buildings may be damaged. Plaster falls in small amounts. Other plaster cracks somewhat. Many dishes and glasses, and a few windows break. Knickknacks, books and pictures fall. Furniture overturns in many instances. Heavy furnishings move.

VII Frightens everyone. General alarm, and everyone runs outdoors.

People find it difficult to stand. Persons driving cars notice shaking. Trees and bushes shake moderately to strongly. Waves form on ponds, lakes and streams. Water is muddied. Gravel or sand stream banks cave in. Large church bells ring. Suspended objects quiver. Damage is negligible in buildings of good design and construction; slight to moderate in well-built ordinary buildings; considerable in poorly built or badly designed buildings, adobe houses, old walls (especially where laid up without mortar), spires, etc. Plaster and some stucco fall. Many windows and some furniture break. Loosened brickwork and tiles shake down. Weak chimneys break at the roofline. Cornices fall from towers and high buildings. Bricks and stones are dislodged. Heavy furniture overturns. Concrete irrigation ditches are considerably damaged.

VIII General fright, and alarm approaches panic.

Persons driving cars are disturbed. Trees shake strongly, and branches and trunks break off (especially palm trees). Sand and mud erupts in small amounts. Flow of springs and wells is temporarily and sometimes permanently changed. Dry wells renew flow. Temperatures of spring and well waters varies. Damage slight in brick structures built especially to withstand earthquakes; considerable in ordinary substantial buildings, with some partial collapse; heavy in some wooden houses, with some tumbling down. Panel walls break away in frame structures. Decayed pilings break off. Walls fall. Solid stone walls crack and break seriously. Wet grounds and steep slopes crack to some extent. Chimneys, columns, monuments and factory stacks and towers twist and fall. Very heavy furniture moves conspicuously or overturns.

IX Panic is general.

Ground cracks conspicuously. Damage is considerable in masonry structures built especially to withstand earthquakes; great in other masonry buildings - some collapse in large part. Some wood frame houses built especially to withstand earthquakes are thrown out of plumb, others are shifted wholly off foundations. Reservoirs are seriously damaged and underground pipes sometimes break.

X Panic is general.

Ground, especially when loose and wet, cracks up to widths of several inches; fissures up to a yard in width run parallel to canal and stream banks. Landsliding is considerable from river banks and steep coasts. Sand and mud shifts horizontally on beaches and flat land. Water level changes in wells. Water is thrown on banks of canals, lakes, rivers, etc. Dams, dikes, embankments are seriously damaged. Well-built wooden structures and bridges are severely damaged, and some collapse. Dangerous cracks develop in excellent brick walls. Most masonry and frame structures, and their foundations are destroyed. Railroad rails bend slightly. Pipe lines buried in earth tear apart or are crushed endwise. Open cracks and broad wavy folds open in cement pavements and asphalt road surfaces.

XI Panic is general.

Disturbances in ground are many and widespread, varying with the ground material. Broad fissures, earth slumps, and land slips develop in soft, wet ground. Water charged with sand and mud is ejected in large amounts. Sea waves of significant magnitude may develop. Damage is severe to wood frame structures, especially near shock centers, great to dams, dikes and embankments, even at long distances. Few if any masonry structures remain standing. Supporting piers or pillars of large, well-built bridges are wrecked. Wooden bridges that "give" are less affected. Railroad rails bend greatly and some thrust endwise. Pipe lines buried in earth are put completely out of service.

XII Panic is general.

Damage is total, and practically all works of construction are damaged greatly or destroyed. Disturbances in the ground are great and varied, and numerous shearing cracks develop. Landslides, rock falls, and slumps in river banks are numerous and extensive. Large rock masses are wrenched loose and torn off. Fault slips develop in firm rock, and horizontal and vertical offset displacements are notable. Water channels, both surface and underground, are disturbed and modified greatly. Lakes are dammed, new waterfalls are produced, rivers are deflected, etc. Surface waves are seen on ground surfaces. Lines of sight and level are distorted. Objects are thrown upward into the air.

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Figure Title
MODIFIED MERCALLI INTENSITY SCALE

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Figure
4

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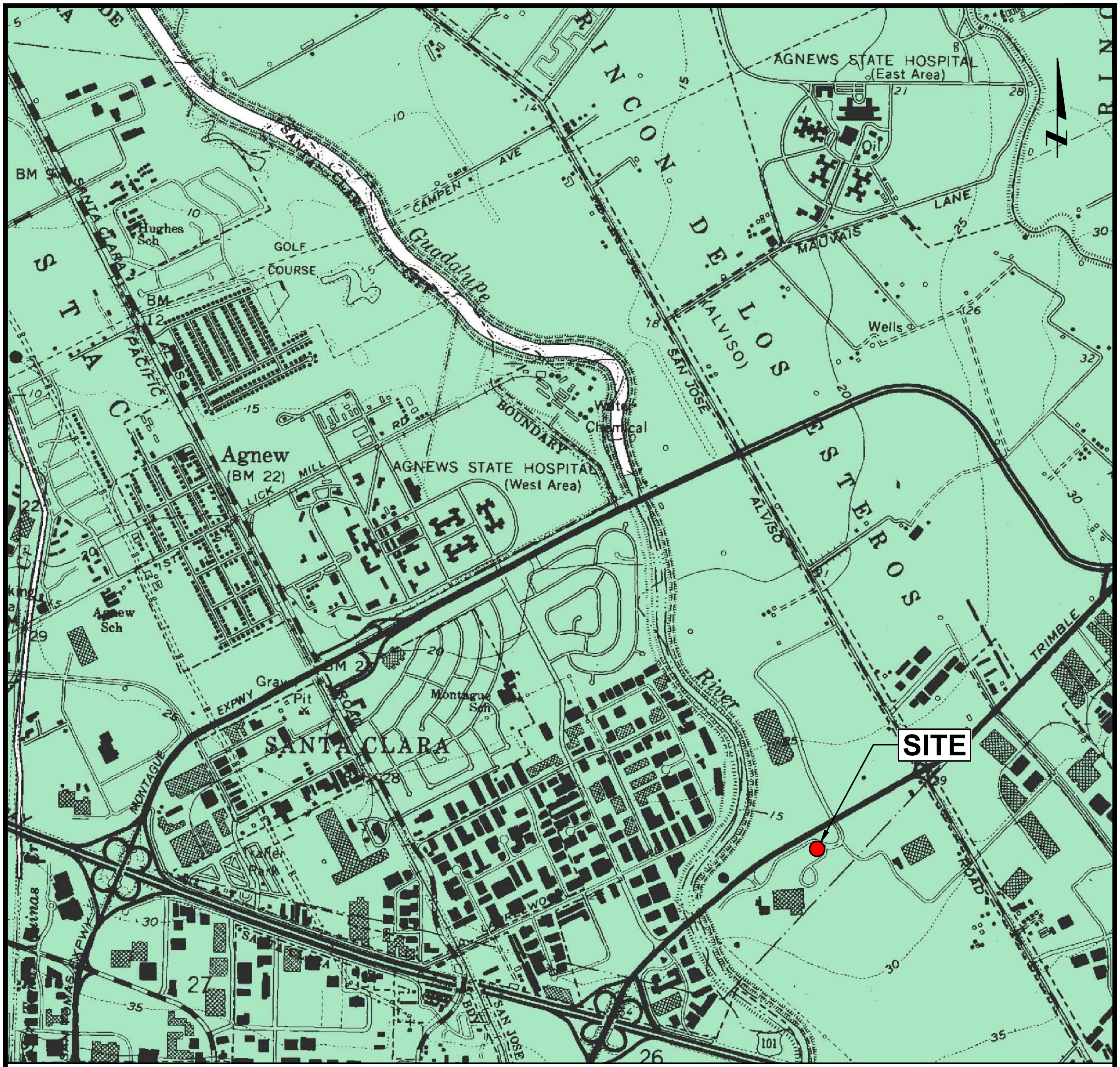

EXPLANATION

🟩	**Liquefaction**; Areas where historic occurrence of liquefaction, or local topographic, geological, geotechnical, and subsurface water conditions indicate a potential for permanent ground displacements.

Note:
State of California Seismic Hazard Zones Milpitas Quadrangle Official Map
Released: October 19, 2004

0 2,000 Feet

Approximate scale

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Project
ADVANCED
MANUFACTURING
BUILDING
330 W. TRIMBLE ROAD, SAN JOSE
SANTA CLARA COUNTY CALIFORNIA

Figure Title
REGIONAL SEISMIC HAZARD ZONES MAP

Project No. 770651906
Date 01/03/2023
Drawn By JDF
Checked By TL

Figure
5

Filename: \\langan.com\data\SJO\data9\770651903\Project Data\CAD\03\2D-DesignFiles\770651906\770651906-B-GI0101.dwg Date: 1/5/2023 Time: 11:06 User: agekas Style Table: Langan.stb Layout: Fig 5 Haz Map

APPENDIX A

LOGS OF TEST BORINGS

PROJECT: **ADVANCED MANUFACTURING BUILDING**
330 W. Trimble Road
San Jose, California

Log of Boring B-1

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2

Date started: 3/6/19

Date finished: 3/6/19

Drilling method: Hollow Stem Auger (B-53 RED)

Hammer weight/drop: 140 lbs./30 inches

Hammer type: Automatic Safety

Samplers: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

Logged by: T. Toledo

LABORATORY TEST DATA

Ground Surface Elevation: 26.1 feet[2]

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						2.5 inches asphalt concrete (AC)						
1		☒				CLAY (CH) dark brown, very stiff, moist						
2		☒										
3	BULK	☒				LL = 77, PL = 23, PI = 54, see Figure C-7						
4		☒										
5					CH							
6	S&H	■	6 17 26	22		brown, trace fine sand						
7												
8												
9	S&H	■	8 11 15	13		light brown						
10						SANDY CLAY (CL) olive with gray mottling, stiff, moist, fine sand						
11					CL							
12												
13					▽	CLAY (CL) (03/06/19, 9:30 a.m.) olive, very stiff, wet seam of fine to coarse sand						
14	SPT	◩	6 8 10	16								
15												
16												
17					CL							
18												
19	S&H	■	13 26 18	22								
20	SPT	◩	13 18 20	34		hard						
21						SILTY SAND (SM) gray-brown, dense, wet, fine-grained						
22												
23												
24	S&H	■	18 30 42	36	SM	gray						
25												
26												
27												
28						SAND (SP) gray-brown, very dense, wet, fine-grained						
29	SPT	◩	18 27 38	59	SP							
30												
31												

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Project No.: 770651906

Figure: A-1a

PROJECT: **ADVANCED MANUFACTURING BUILDING**
330 W. Trimble Road
San Jose, California

Log of Boring B-1

PAGE 2 OF 2

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft	LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft	LABORATORY TEST DATA: Fines %	LABORATORY TEST DATA: Natural Moisture Content, %	LABORATORY TEST DATA: Dry Density Lbs/Cu Ft
						SAND (SP) (continued)						
32												
33												
34	SPT	◩	13 18 38	50		dark brown, dense to very dense, fine- to coarse-grained						
35												
36												
37												
38												
39	SPT	◩	25 29 50/ 1"	71/ 7"		yellow-brown, some fine subrounded gravel, trace clay						
40												
41					SP							
42												
43												
44	SPT	◩	33 50/ 6"	45/ 6"		clay seam, increase in coarse sand						
45												
46												
47												
48												
49												
50						medium dense, fine- to medium-grained						
51	SPT	◩	18 12 14	23		CLAY (CL) olive, very stiff, wet						
52												
53												
54	SPT	◩	31 37 40	69		olive with gray and orange, hard						
55					CL							
56												
57												
58												
59	S&H	■	5 12 17	15	SP	SAND (SP) yellow-brown, medium dense, wet, fine-to coarse-grained						
60												
61												
62												

Boring terminated at a depth of 60 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 13.5 feet below ground surface during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.5 and 0.9, respectively to account for sampler type and hammer energy.

[2] Elevations reference North American Vertical Datum of 1988 (NAVD88) and is based on a topographic survey provided by HMH dated 22 May 2019.

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Project No.: 770651906

Figure: A-1b

PROJECT: **ADVANCED MANUFACTURING BUILDING**
330 W. Trimble Road
San Jose, California

Log of Boring B-2

PAGE 1 OF 3

Boring location: See Site Plan, Figure 2

Logged by: T. Toledo

Date started: 3/5/19

Date finished: 3/5/19

Drilling method: Rotary Wash

Hammer weight/drop: 140 lbs./30 inches

Hammer type: Automatic Safety

Samplers: Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube (ST)

DEPTH (feet)	SAMPLES				LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA					
	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]		Ground Surface Elevation: 26.3 feet[2]	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						2.5 inches asphalt concrete (AC)						
1						CLAY (CH) dark brown, stiff, moist						
2												
3												
4					CH							
5						dark brown with gray						
6	S&H	▨	14 9 11	14								
7												
8												
9						SANDY CLAY (CL) brown, very soft to soft, wet, fine sand, trace fine gravel						
10					CL							
11	S&H	▨	1 1 2	2								
12	SPT	▨	3 6 9	18	SC	CLAYEY SAND (SC) olive-gray, medium dense, wet, trace fine gravel						
13						SAND with CLAY (SP-SC) brown, medium dense, wet, fine- to coarse-grained, trace subrounded gravel						
14	SPT	▨	10 8 12	24	SP-SC							
15												
16						SAND with GRAVEL (SP) yellow-brown, dense, wet, fine- to coarse-grained, fine to coarse subrounded gravel						
17												
18												
19					SP							
20												
21	SPT	▨	10 16 15	37								
22												
23						SAND with CLAY and GRAVEL (SP-SC) brown, medium dense, wet						
24												
25												
26	SPT	▨	16 13 11	29	SP-SC					5.5	9.1	
27												
28												
29												
30												

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Figure: A-2a

Log of Boring B-2

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value¹	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31	SPT		4 9 11	24	SP	SAND with CLAY and GRAVEL (SP-SC) (continued) SAND (SP) brown, medium dense, wet, trace fine gravel						
32												
33												
34												
35					SM	SILTY SAND (SM) gray, medium dense, wet, fine-grained, trace wood fragments						
36	SPT		9 7 8	18						20.1	21.2	
37												
38												
39					CL	CLAY (CL) gray, wet						
40												
41	ST		300 psi									
42					SP	CLAYEY SAND with GRAVEL (SC) yellow-brown, very dense, fine- to coarse-grained, fine gravel						
43	SPT		19 35 50/ 5"	102/ 11"								
44												
45												
46												
47					CL	CLAY (CL) brown, wet						
48												
49					SP	SAND (SP) brown, dense, fine- to coarse-grained, trace fine subrounded to subangular gravel						
50	SPT		16 20 19	47								
51												
52					CL	CLAY with SAND (CL) yellow-brown, very stiff, wet, fine sand						
53												
54												
55												
56	S&H		6 9 14	16		Consolidation Test, see Figure C-1					23.4	102
57												
58												
59												
60												

PROJECT: **ADVANCED MANUFACTURING BUILDING**
330 W. Trimble Road
San Jose, California

Log of Boring B-2

PAGE 3 OF 3

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
61	S&H	▨	25 30 29	41		SAND with GRAVEL (SP) brown, dense, wet, fine- to coarse-grained, fine to coarse subrounded gravel						
62	SPT	◩	9 22 29	61		very dense						
63												
64												
65					SP							
66	SPT	◩	22 25 31	67								
67												
68												
69												
70						SILTY SAND (SM) brown, dense, wet, fine-grained						
71	SPT	◩	10 12 24	43								
72					SM							
73												
74						SAND with GRAVEL (SP) yellow-brown with orange, very dense, wet, fine- to coarse-grained, fine to coarse subrounded to subangular gravel						
75												
76	SPT	◩	22 23 33	67								
77					SP							
78												
79												
80						SAND (SP) yellow-brown, very dense, wet, fine- to coarse-grained, trace fine to coarse subrounded to subangular gravel						
81	SPT	◩	22 24 24	58	SP							
82												
83												
84												
85												
86												
87												
88												
89												
90												

Boring terminated at a depth of 81.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater obscure by drilling method.

---

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.

[2] Elevations reference North American Vertical Datum of 1988 (NAVD88) and is based on a topographic survey provided by HMH dated 22 May 2019.

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Figure: A-2c

PROJECT: **ADVANCED MANUFACTURING BUILDING**
330 W. Trimble Road
San Jose, California

Log of Boring B-3

PAGE 1 OF 3

Boring location: See Site Plan, Figure 2

Logged by: C. Leege

Date started: 3/4/19

Date finished: 3/4/19

Drilling method: Rotary Wash

Hammer weight/drop: 140 lbs./30 inches

Hammer type: Automatic Safety

Samplers: Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube (ST)

Ground Surface Elevation: 27.1 feet^2

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value^1	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						2.5 inches asphalt concrete (AC)						
1					CH	CLAY (CH) dark brown, moist						
2												
3					SM	SILTY SAND with GRAVEL (SM) olive-gray, moist, fine-grained, coarse subrounded to subangular gravel						
4						CLAY (CH) dark brown, stiff, moist						
5												
6	S&H	▨	3 6 8	10								
7					CH							
8												
9												
10						SANDY CLAY (CL) olive, medium stiff, wet, fine sand						
11	S&H	▨	2 2 5	5								
12					CL							
13												
14						CLAYEY SAND (SC) olive-gray, medium dense, wet, fine-grained, trace fine gravel						
15												
16	SPT	▨	5 4 5	11	SC	LL = 28, PL = 18, PI = 10, see Figure C-7				21.3	14.1	
17						grades with increase gravel content						
18												
19						SAND (SP) yellow-brown, medium dense, wet, fine-grained, trace fine to coarse subrounded gravel, trace clay						
20												
21	SPT	▨	8 10 11	25								
22					SP							
23												
24												
25												
26	SPT	▨	12 8 8	19	SM	SILTY SAND (SM) yellow-brown, medium dense, wet, fine-grained						
27												
28					CL	CLAY (CL) gray, stiff, wet						
29												
30												

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Project No.: 770651906

Figure: A-3a

PROJECT: **ADVANCED MANUFACTURING BUILDING**
330 W. Trimble Road
San Jose, California

Log of Boring B-3

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						CLAY (CL) (continued)						
31	ST					Consolidation Test, see Figure C-2					29.6	92
32							TxUU	2,400	1,580		27.4	98
33					CL							
34												
35						medium stiff						
36	S&H		0 4 4	6	SC	CLAYEY SAND (SC) gray, loose, wet, fine- to medium-grained, trace fine gravel						
37	SPT		2 2 4	7		CLAY (CL) gray with olive mottling, medium stiff, wet, trace fine sand						
38												
39												
40						increased sand content						
41	S&H		2 3 4	5								
42												
43					CL							
44												
45						yellow-brown, stiff, trace fine sand						
46	ST			100 psi			TxUU	3,200	1,680		23.5	104
47												
48												
49												
50						SANDY CLAY (CL) olive with red-yellow mottling, stiff, wet, fine to medium sand, trace fine subangular gravel						
51	S&H		4 6 7	9								
52					CL							
53												
54												
55						CLAY (CL) olive with red-yellow mottling, very stiff, wet						
56	S&H		11 20 21	29								
57					CL							
58												
59												
60												

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Project No.: 770651906

Figure: A-3b

PROJECT: **ADVANCED MANUFACTURING BUILDING**
330 W. Trimble Road
San Jose, California

Log of Boring B-3

PAGE 3 OF 3

DEPTH (feet)	SAMPLES: Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
61	S&H	■	13 25 35	42		SAND wIth GRAVEL (SP) brown, dense, wet, fine-grained, fine to coarse subangular gravel						
62												
63												
64												
65												
66	SPT	◩	23 30 28	70		very dense, fine- to coarse-grained, subangular gravel						
67												
68												
69												
70												
71	SPT	◩	17 26 28	65	SP							
72												
73												
74												
75						dense, trace clay						
76	SPT	◩	17 20 20	48								
77												
78												
79												
80						very dense						
81	SPT	◩	19 20 23	52								
82												
83												
84												
85												
86												
87												
88												
89												
90												

Boring terminated at a depth of 81.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater obscure by drilling method.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.

[2] Elevations reference North American Vertical Datum of 1988 (NAVD88) and is based on a topographic survey provided by HMH dated 22 May 2019.

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Project No.: 770651906

Figure: A-3c

PROJECT: **ADVANCED MANUFACTURING BUILDING**
330 W. Trimble Road
San Jose, California

Log of Boring B-4

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2

Date started: 3/7/19 Date finished: 3/7/19

Drilling method: Hollow Stem Auger (B-53 RED)

Hammer weight/drop: 140 lbs./30 inches Hammer type: Automatic Safety

Samplers: Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube (ST)

Logged by: T. Toledo

LABORATORY TEST DATA

Ground Surface Elevation: 29 feet^2

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value^1	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0						2.5 inches asphalt concrete (AC)						
1					CH	CLAY (CH) dark brown, very stiff, moist						
6	S&H	■	8 17 24	21								
8					CL	CLAY (CL) yellow-brown with orange, stiff, moist, trace fine to coarse sand						
9	S&H	■	12 13 13	13								
10						Consolidation Test, see Figure C-3					26.7	95
13					SC	CLAYEY SAND (SC) gray-brown with orange, medium dense, wet, fine-grained, trace fine gravel						
14	S&H	■	11 15 13	14								
15					▽	(03/07/19, 10:00 a.m.) at 14.5 feet: LL: = 26, PL = 18, PI = 8				40.5	21.5	
17					SM	SILTY SAND (SM) light gray, very dense, fine-grained, wet, trace fine to coarse gravel						
19	SPT	◪	26 40 41	73	SP	SAND (SP) gray-brown, very dense, wet, fine- to coarse-grained, trace fine to coarse subangular gravel						
24	SPT	◪	31 50/ 6"	45/ 6"								
24.5					GP	GRAVEL with SAND (GP) gray-brown, very dense, wet, fine- to coarse-grained, subrounded to subangular, fine to coarse sand						
28					SM	SILTY SAND (SM) light gray, dense, wet, fine-grained, trace fine subrounded gravel, some clay						
29	SPT	◪	20 18 23	37								
31												

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Project No.: 770651906

Figure: A-4a

PROJECT: **ADVANCED MANUFACTURING BUILDING**
330 W. Trimble Road
San Jose, California

Log of Boring B-4

PAGE 2 OF 2

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft	LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft	LABORATORY TEST DATA: Fines %	LABORATORY TEST DATA: Natural Moisture Content, %	LABORATORY TEST DATA: Dry Density Lbs/Cu Ft
						SILTY SAND (SM) (continued)						
32												
33					SM							
34	SPT	◪	7 8 10	16								
35						CLAY (CL) brown, very stiff, wet						
36					CL							
37												
38						CLAY (CL) yellow-brown with gray, very stiff, wet, trace coarse sand						
39	S&H	■	11 15 22	19								
40												
41					CL							
42												
43												
44	S&H	■	12 20 27	24								
45						SILTY SAND (SM) gray, dense, wet, fine-grained						
46												
47												
48												
49	SPT	◪	12 20 25	41	SM	red-yellow to light brown						
50												
51												
52												
53						SAND (SP) yellow-brown, very dense, wet, fine-grained						
54	SPT	◪	17 31 42	66								
55												
56					SP							
57												
58												
59	SPT	◪	12 18 22	36		dense						
60												
61												
62												

Boring terminated at a depth of 60 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 15 feet below ground surface during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.5 and 0.9, respectively to account for sampler type and hammer energy.

[2] Elevations reference North American Vertical Datum of 1988 (NAVD88) and is based on a topographic survey provided by HMH dated 22 May 2019.

Project No.: 770651906

Figure: A-4b

PROJECT: **ADVANCED MANUFACTURING BUILDING**
330 W. Trimble Road
San Jose, California

Log of Boring B-5

Boring location: See Site Plan, Figure 2
Date started: 3/7/19
Date finished: 3/7/19
Drilling method: Hollow Stem Auger (B-53 RED)
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Automatic Safety
Samplers: Sprague & Henwood (S&H), Standard Penetration Test (SPT)
Logged by: T. Toledo

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 27.1 feet[2]	LABORATORY TEST DATA Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						2.5 inches asphalt concrete (AC)						
1						CLAY (CH) dark brown, very stiff, moist						
2	BULK	☒										
3						LL = 78, PL = 21, PI = 57, see Figure C-7 R-value Test, see Figure C-8						
4					CH							
5												
6	S&H	■	13 22 32	27								
7												
8						SANDY CLAY (CL) yellow-brown, very stiff, moist, fine to coarse sand						
9	S&H	■	11 15 17	16								
10					CL							
11												
12						CLAYEY SAND (SC) yellow-brown to gray-brown, medium dense, wet,						
13						∇ fine-grained (03/07/19)						
14	SPT	◩	7 11 13	22						31.2	22.4	
15					SC							
16												
17												
18												
19	SPT	◩	9 11 20	28		CLAY (CL) yellow-brown, very stiff, wet						
20												
21												
22												
23					CL							
24	SPT	◩	7 12 19	28		gray, trace wood fragments						
25												
26												
27												
28												
29	S&H	■	12 15 23	19		SILTY SAND (SM) gray-brown, medium dense, wet, fine-grained						
30					SM							
31												

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Project No.: 770651906
Figure: A-5a

PROJECT: **ADVANCED MANUFACTURING BUILDING**
330 W. Trimble Road
San Jose, California

Log of Boring B-5

PAGE 2 OF 2

DEPTH (feet)	SAMPLES Sampler Type	 Sample	 Blows/ 6"	 SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA Type of Strength Test	 Confining Pressure Lbs/Sq Ft	 Shear Strength Lbs/Sq Ft	 Fines %	 Natural Moisture Content, %	 Dry Density Lbs/Cu Ft
					SM	SILTY SAND (SM) (continued)						
32						CLAY (CL) yellow-brown, hard, wet						
33												
34	SPT	◩	12 17 22	35								
35												
36												
37												
38					CL							
39	S&H	■	11 13 26	20		yellow-brown with gray-brown, very stiff						
40												
41												
42												
43												
44	S&H	■	13 21 26	24		SILTY SAND (SM) yellow-brown, medium dense, wet, fine-grained						
45												
46					SM							
47												
48												
49	SPT	◩	17 50/ 5"	45/ 5"		SAND (SP) brown, very dense, wet, fine-grained						
50					SP							
51												
52						SANDY CLAY (CL) yellow-brown, hard, wet, fine- to medium-grained sand						
53												
54	SPT	◩	14 26 37	57								
55					CL							
56												
57												
58						CLAY (CL) gray, very stiff, wet						
59	S&H	■	14 20 27	24	CL							
60												
61												
62												

Boring terminated at a depth of 60 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 13.5 feet below ground surface during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.5 and 0.9, respectively to account for sampler type and hammer energy.

[2] Elevations reference North American Vertical Datum of 1988 (NAVD88) and is based on a topographic survey provided by HMH dated 22 May 2019.

LANGAN

Project No.: 770651906

Figure: A-5b

PROJECT: **ADVANCED MANUFACTURING BUILDING**
330 W. Trimble Road
San Jose, California

Log of Boring B-6

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2

Date started: 3/6/19 | Date finished: 3/6/19

Drilling method: Hollow Stem Auger (B-53 RED)

Hammer weight/drop: 140 lbs./30 inches | Hammer type: Automatic Safety

Samplers: Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube (ST)

Logged by: T. Toledo

LABORATORY TEST DATA

Ground Surface Elevation: 26.9 feet^2

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value^1	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						2.5 inches asphalt concrete (AC)						
1		☒				CLAY (CH) dark brown, very stiff, moist						
2	BULK	☒				R-value Test, see Figure C-8						
3		☒										
4					CH							
5		■	11			brown						
6	S&H	■	20	27								
			34									
7												
8												
9	S&H	■	6 6 6	6		SANDY CLAY (CL) yellow-brown, soft, moist, fine- to coarse-grained sand						
10						▽ (03/06/19, 3:30 p.m.)	TxUU	1,100	360		19.3	108
11					CL							
12												
13						CLAY with SAND (CL) gray-brown, stiff to very stiff, wet, with fine sand						
14	S&H	■	11 13 17	15								
15					CL							
16												
17												
18						SAND (SP) yellow-brown, medium dense, wet, fine-grained						
19	S&H	■	11 10 27	19	SP							
20	SPT	◪	16 17 19	32		brown, dense						
21					SC	CLAYEY SAND (SC) gray, dense, wet, fine-grained						
22						CLAY (CL) gray, hard, wet, trace fine sand						
23												
24	SPT	◪	13 18 32	36								
25					CL							
26												
27												
28						SILTY SAND (SM) gray-brown, medium dense, wet, trace fine subrounded gravel						
29	S&H	■	25 22 26	24	SM							
30												

LANGAN

Project No.: 770651906

Figure: A-6a

PROJECT: **ADVANCED MANUFACTURING BUILDING**
330 W. Trimble Road
San Jose, California

Log of Boring B-6

PAGE 2 OF 2

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft	LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft	LABORATORY TEST DATA: Fines %	LABORATORY TEST DATA: Natural Moisture Content, %	LABORATORY TEST DATA: Dry Density Lbs/Cu Ft
						CLAY (CL) gray with yellow-brown mottling, hard, wet, trace fine sand						
31												
32												
33												
34	S&H	■	16 29 32	31								
35												
36												
37					CL							
38												
39	S&H	■	9 13 14	14		yellow-brown, stiff						
40												
41	ST	◧		200 psi								
42												
43												
44	S&H	■	14 12 30	21								
45					SM	SILTY SAND (SM) yellow-brown, dense, wet, fine-grained						
46												
47												
48												
49												
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 45 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 10 feet below ground surface during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.5 and 0.9, respectively to account for sampler type and hammer energy.
[2] Elevations reference North American Vertical Datum of 1988 (NAVD88) and is based on a topographic survey provided by HMH dated 22 May 2019.

LANGAN

Project No.: 770651906
Figure: A-6b

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions		Symbols	Typical Names
Coarse-Grained Soils (more than half of soil > no. 200 sieve size	**Gravels** (More than half 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-clay mixtures
	Sands (More than half 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 half of soil < no. 200 sieve size)	**Silts and Clays** LL = < 50	**ML**	Inorganic silts and clayey silts of low plasticity, sandy silts, gravelly silts
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays
		OL	Organic silts and organic silt-clays of low plasticity
	Silts and Clays LL = > 50	**MH**	Inorganic silts of high plasticity
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic silts and clays of high plasticity
Highly Organic Soils		**PT**	Peat and other highly organic soils

GRAIN SIZE CHART

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	3" to No. 4	76.2 to 4.76
coarse	3" to 3/4"	76.2 to 19.1
fine	3/4" to No. 4	19.1 to 4.76
Sand	No. 4 to No. 200	4.76 to 0.075
coarse	No. 4 to No. 10	4.76 to 2.00
medium	No. 10 to No. 40	2.00 to 0.420
fine	No. 40 to No. 200	0.420 to 0.075
Silt and Clay	Below No. 200	Below 0.075

▽ Unstabilized groundwater level

▼ Stabilized groundwater level

SAMPLE DESIGNATIONS/SYMBOLS

⬒	Sample taken with Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter. Darkened area indicates soil recovered
◩	Classification sample taken with Standard Penetration Test sampler
◨	Undisturbed sample taken with thin-walled tube
⊠	Disturbed sample
⊙	Sampling attempted with no recovery
◫	Core sample
◉	Analytical laboratory sample
▥	Sample taken with Direct Push or Drive sampler

SAMPLER TYPE

C	Core barrel
CA	California split-barrel sampler with 2.5-inch outside diameter and a 1.93-inch inside diameter
D&M	Dames & Moore piston sampler using 2.5-inch outside diameter, thin-walled tube
O	Osterberg piston sampler using 3.0-inch outside diameter, thin-walled Shelby tube
PT	Pitcher tube sampler using 3.0-inch outside diameter, thin-walled Shelby tube
S&H	Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter
SPT	Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and a 1.5-inch inside diameter
ST	Shelby Tube (3.0-inch outside diameter, thin-walled tube) advanced with hydraulic pressure

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Project
ADVANCED MANUFACTURING BUILDING
330 W. TRIMBLE ROAD, SAN JOSE
SANTA CLARA COUNTY CALIFORNIA

Figure Title
CLASSIFICATION CHART

Project No. 770651906
Date 01/03/2023
Drawn By JDF
Checked By TL

Figure
A-7

APPENDIX B

CONE PENETRATION TEST RESULTS

TABLE B-1
Cone Penetration Test (CPT) Summary

Location	Ground Surface Elevation[1] (feet)	Depth of PPDT[2] (feet)	Interpreted Potentiometric Surface Depth from PPDT (feet)	Interpreted Potentiometric Surface Elevation from PPDT (feet)
CPT-1	26	20.8	8.5	17.5
CPT-2	26.7	65.8	7.1	19.6
CPT-3	26.7	38.1	5.1	21.6
CPT-4	26.3	24.6	7.8	18.5
CPT-4	26.3	63.9	4.7	21.6
CPT-5	25.8	25.1	7.7	18.1
CPT-6	27.4	38.6	8.8	18.6
CPT-6	27.4	68.1	7.2	20.2
CPT-7	27.4	54.5	6.3	21.1

Notes:

[1] Elevations reference North American Vertical Datum of 1988 (NAVD) and is based on a topographic survey provided by HMH dated 22 May 2019.

[2] PPDT = pore pressure dissipation test

PRESENTATION OF SITE INVESTIGATION RESULTS

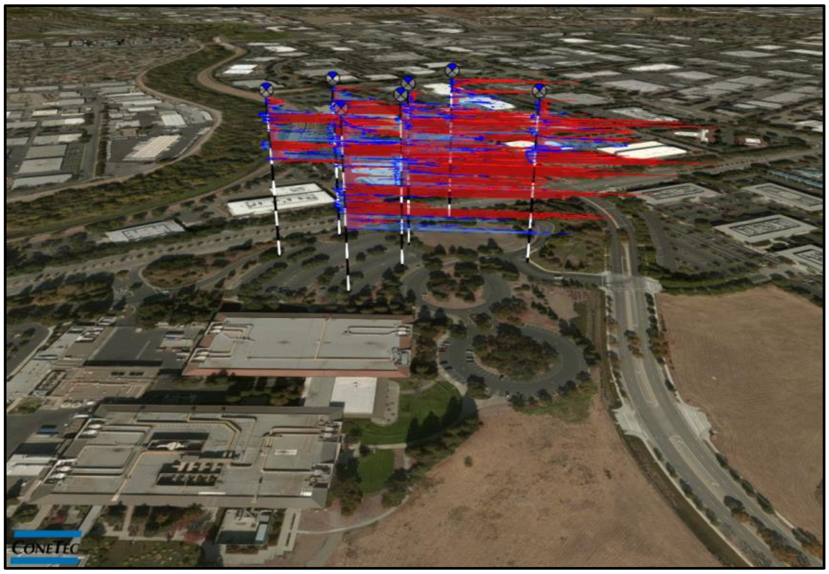
North Town

Prepared for:

Langan Engineering

ConeTec Inc. Job No: 19-56026

Project Start Date: 04-Mar-2019
Project End Date: 05-Mar-2019
Report Date: 06-Mar-2019



Prepared by:

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www.conetecdataservices.com

Introduction

The enclosed report presents the results of the site investigation program conducted by ConeTec Inc. for Langan Engineering at the corner of West Trimble Road and Orchard Parkway, San Jose, CA. The program consisted of seven cone penetration tests (CPT).

Project Information

Project	
Client	Langan Engineering
Project	North Town
ConeTec project number	19-56026

An image from Google Earth including the CPT test locations is presented below.

Rig Description	Deployment System	Test Type
CPT truck rig (C17)	30 ton rig cylinder	CPT

Coordinates		
Test Type	Collection Method	EPSG Number
CPT	Consumer grade GPS	32610

Cone Penetration Test (CPT)	
Depth reference	Depths are referenced to the existing ground surface at the time of each test.
Tip and sleeve data offset	0.1 meter This has been accounted for in the CPT data files.
Additional plots	Standard plots with expanded scales, Advanced plots with Ic, Su(Nkt), Phi and N1(60)Ic, as well as Soil Behavior Type (SBT) scatter plots have been included in the data release package.

Cone Penetrometers Used for this Project						
Cone Description	Cone Number	Cross Sectional Area (cm^2)	Sleeve Area (cm^2)	Tip Capacity (bar)	Sleeve Capacity (bar)	Pore Pressure Capacity (psi)
483:T1500F15U500	483	15	225	1500	15	500
Cone 483 was used for all CPT soundings.						

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

Limitations

This report has been prepared for the exclusive use of Langan Engineering (Client) for the project titled “North Town”. The report’s contents may not be relied upon by any other party without the express written permission of ConeTec Inc. (ConeTec). ConeTec has provided site investigation services, prepared the factual data reporting and provided geotechnical parameter calculations consistent with current best practices. No other warranty, expressed or implied, is made.

The information presented in the report document and the accompanying data set pertain to the specific project, site conditions and objectives described to ConeTec by the Client. In order to properly understand the factual data, assumptions and calculations, reference must be made to the documents provided and their accompanying data sets, in their entirety.

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

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

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

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

All ConeTec piezocones can record pore pressure at various locations. Unless otherwise noted, the pore pressure filter is located directly behind the cone tip in the "u_2" position (ASTM Type 2). The filter is 6 mm thick, made of porous plastic (polyethylene) having an average pore size of 125 microns (90-160 microns). The function of the filter is to allow rapid movements of extremely small volumes of water needed to activate the pressure transducer while preventing soil ingress or blockage.

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

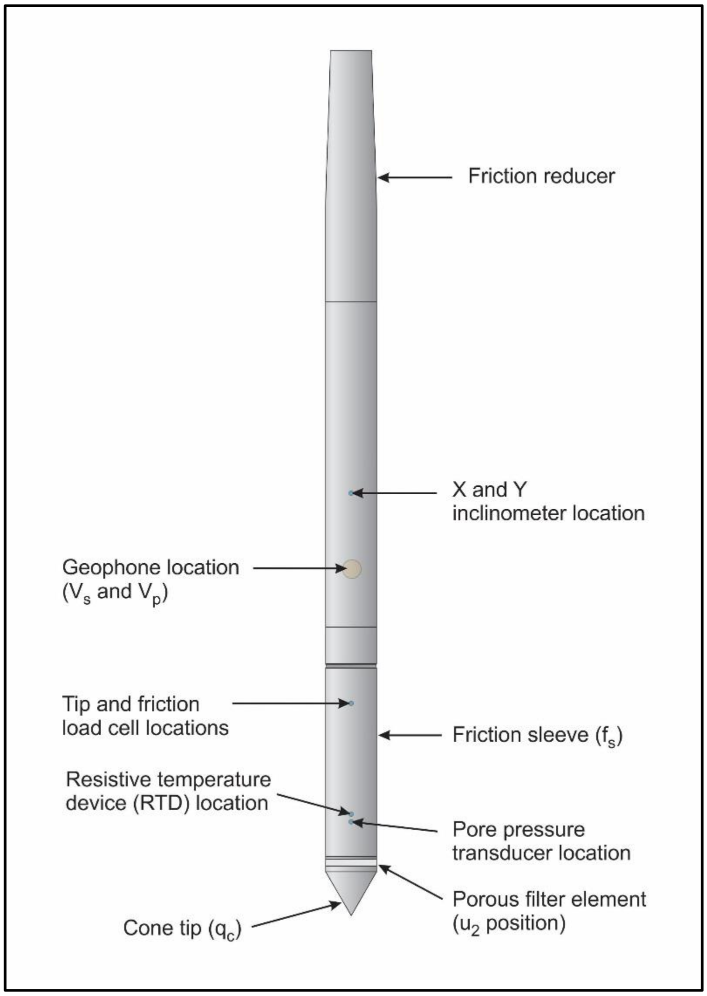

Figure CPTu. Piezocone Penetrometer (15 cm^2)

The ConeTec data acquisition systems consist of a Windows based computer and a signal conditioner and power supply interface box with a 16 bit (or greater) analog to digital (A/D) converter. The data is recorded at fixed depth increments using a depth wheel attached to the push cylinders or by using a spring loaded rubber depth wheel that is held against the cone rods. The typical recording interval is 2.5 cm; custom recording intervals are possible.

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

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

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

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

The CPTu is conducted at a steady rate of 2 cm/s, within acceptable tolerances. Typically one meter length rods with an outer diameter of 1.5 inches are added to advance the cone to the sounding termination depth. After cone retraction final baselines are recorded.

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

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

The interpretation of piezocone data for this report is based on the corrected tip resistance (q_t), sleeve friction (f_s) and pore water pressure (u). The interpretation of soil type is based on the correlations developed by Robertson et al. (1986) and Robertson (1990, 2009). It should be noted that it is not always possible to accurately identify a soil behavior based on these parameters. In these situations, experience, judgment and an assessment of other parameters may be used to infer soil behavior type.

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

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

where: q_t is the corrected tip resistance
q_c is the recorded tip resistance
u_2 is the recorded dynamic pore pressure behind the tip (u_2 position)
a is the Net Area Ratio for the piezocone (0.8 for ConeTec probes)

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

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

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

A summary of the CPTu soundings along with test details and individual plots are provided in the appendices. A set of files with calculated geotechnical parameters were generated for each sounding based on published correlations and are provided in Excel format in the data release folder. Information regarding the methods used is also included in the data release folder.

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

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

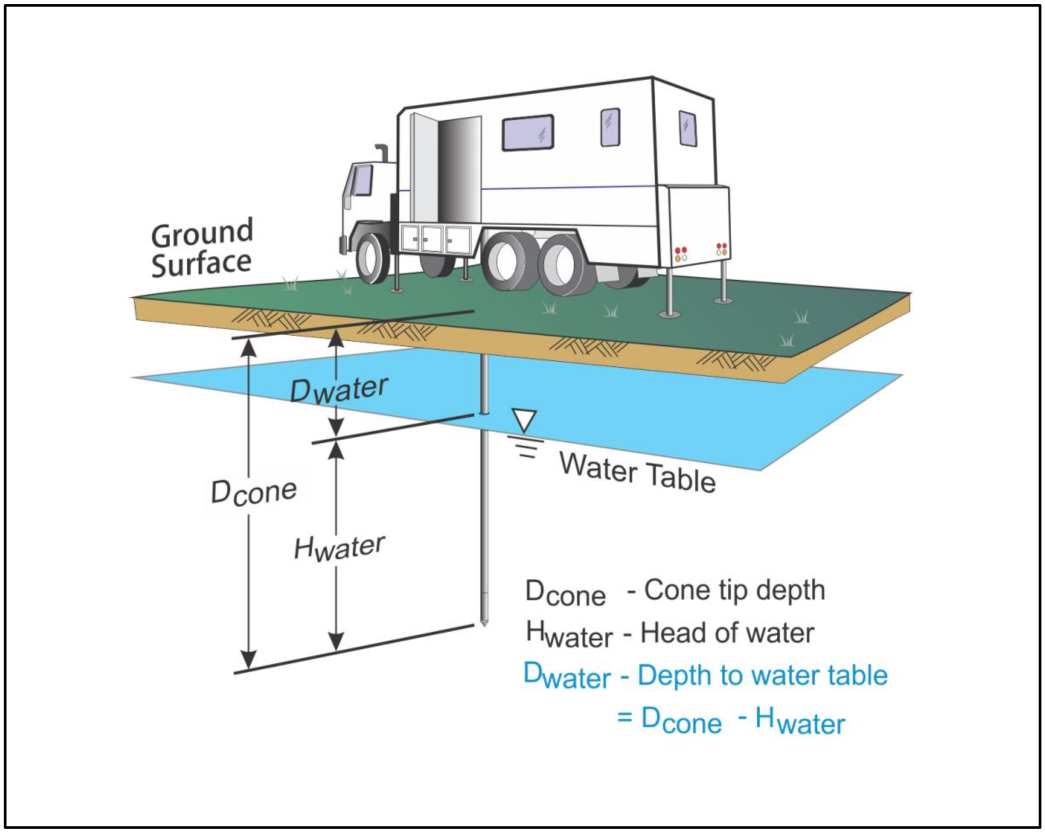

Figure PPD-1. Pore pressure dissipation test setup

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

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

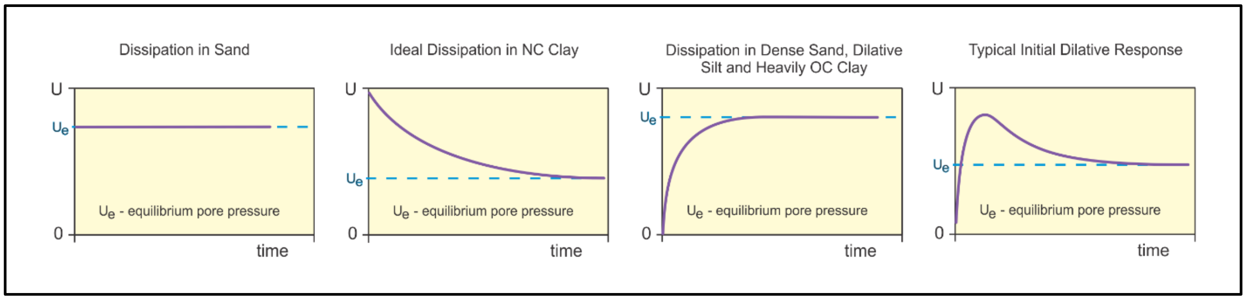

Figure PPD-2. Pore pressure dissipation curve examples

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

In fine grained deposits the point at which 100% of the excess pore pressure has dissipated is known as t_{100}. In some cases this can take an excessive amount of time and it may be impractical to take the dissipation to t_{100}. A theoretical analysis of pore pressure dissipations by Teh and Houlsby (1991) showed that a single curve relating degree of dissipation versus theoretical time factor (T*) may be used to calculate the coefficient of consolidation (c_h) at various degrees of dissipation resulting in the expression for c_h shown below.

$$c_h = \frac{T^* \cdot a^2 \cdot \sqrt{I_r}}{t}$$

Where:

T^* is the dimensionless time factor (Table Time Factor)
a is the radius of the cone
I_r is the rigidity index
t is the time at the degree of consolidation

Table Time Factor. T* versus degree of dissipation (Teh and Houlsby (1991))

Degree of Dissipation (%)	20	30	40	50	60	70	80
T* (u_2)	0.038	0.078	0.142	0.245	0.439	0.804	1.60

The coefficient of consolidation is typically analyzed using the time (t_{50}) corresponding to a degree of dissipation of 50% (u_{50}). In order to determine t_{50}, dissipation tests must be taken to a pressure less than u_{50}. The u_{50} value is half way between the initial maximum pore pressure and the equilibrium pore pressure value, known as u_{100}. To estimate u_{50}, both the initial maximum pore pressure and u_{100} must be known or estimated. Other degrees of dissipations may be considered, particularly for extremely long dissipations.

At any specific degree of dissipation the equilibrium pore pressure (u at t_{100}) must be estimated at the depth of interest. The equilibrium value may be determined from one or more sources such as measuring the value directly (u_{100}), estimating it from other dissipations in the same profile, estimating the phreatic surface and assuming hydrostatic conditions, from nearby soundings, from client provided information, from site observations and/or past experience, or from other site instrumentation.

For calculations of c_h (Teh and Houlsby (1991)), t_{50} values are estimated from the corresponding pore pressure dissipation curve and a rigidity index (I_r) is assumed. For curves having an initial dilatory response in which an initial rise in pore pressure occurs before reaching a peak, the relative time from the peak value is used in determining t_{50}. In cases where the time to peak is excessive, t_{50} values are not calculated.

Due to possible inherent uncertainties in estimating I_r, the equilibrium pore pressure and the effect of an initial dilatory response on calculating t_{50}, other methods should be applied to confirm the results for c_h.

Additional published methods for estimating the coefficient of consolidation from a piezocone test are described in Burns and Mayne (1998, 2002), Jones and Van Zyl (1981), Robertson et al. (1992) and Sully et al. (1999).

A summary of the pore pressure dissipation tests and dissipation plots are presented in the relevant appendix.

ASTM D5778-12, 2012, "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils", ASTM, West Conshohocken, US.

Burns, S.E. and Mayne, P.W., 1998, “Monotonic and dilatory pore pressure decay during piezocone tests”, Canadian Geotechnical Journal 26 (4): 1063-1073.

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Jones, G.A. and Van Zyl, D.J.A., 1981, “The piezometer probe: a useful investigation tool”, Proceedings, 10th International Conference on Soil Mechanics and Foundation Engineering, Vol. 3, Stockholm: 489-495.

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

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

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

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

Robertson, P.K., 1990, “Soil Classification Using the Cone Penetration Test”, Canadian Geotechnical Journal, Volume 27: 151-158.

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

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

Robertson, P.K., Sully, J.P., Woeller, D.J., Lunne, T., Powell, J.J.M. and Gillespie, D.G., 1992, “Estimating coefficient of consolidation from piezocone tests”, Canadian Geotechnical Journal, 29(4): 551-557.

Sully, J.P., Robertson, P.K., Campanella, R.G. and Woeller, D.J., 1999, “An approach to evaluation of field CPTU dissipation data in overconsolidated fine-grained soils”, Canadian Geotechnical Journal, 36(2): 369-381.

Teh, C.I., and Houlsby, G.T., 1991, “An analytical study of the cone penetration test in clay”, Geotechnique, 41(1): 17-34.

The appendices listed below are included in the report:

- Cone Penetration Test Summary and Standard Cone Penetration Test Plots
- Standard Cone Penetration Test Plots with Expanded Scales
- Advanced Cone Penetration Test Plots with Ic, Su(Nkt), Phi and N1(60)Ic
- Soil Behavior Type (SBT) Scatter Plots
- Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

Cone Penetration Test Summary and Standard Cone Penetration Test Plots

Job No: 19-56026
Client: Lagan Engineering
Project: North Town
Start Date: 04-Mar-2019
End Date: 05-Mar-2019

CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Cone	Assumed Phreatic Surface[1] (ft)	Final Depth (ft)	Northing[2] (m)	Easting (m)	Refer to Notation Number
CPT-01	19-56026_CP01	04-Mar-2019	483:T1500F15U500	8.5	61.68	4137837	594358	
CPT-02	19-56026_CP02	04-Mar-2019	483:T1500F15U500	7.1	65.78	4137883	594393	
CPT-03	19-56026_CP03	05-Mar-2019	483:T1500F15U500	5.1	101.05	4137871	594449	
CPT-04	19-56026_CP04	05-Mar-2019	483:T1500F15U500	7.8	101.05	4137781	594420	
CPT-05	19-56026_CP05	05-Mar-2019	483:T1500F15U500	7.7	80.54	4137830	594449	
CPT-06	19-56026_CP06	04-Mar-2019	483:T1500F15U500	8.8	68.08	4137932	594476	
CPT-07	19-56026_CP07	04-Mar-2019	483:T1500F15U500	6.3	61.68	4137849	594538	

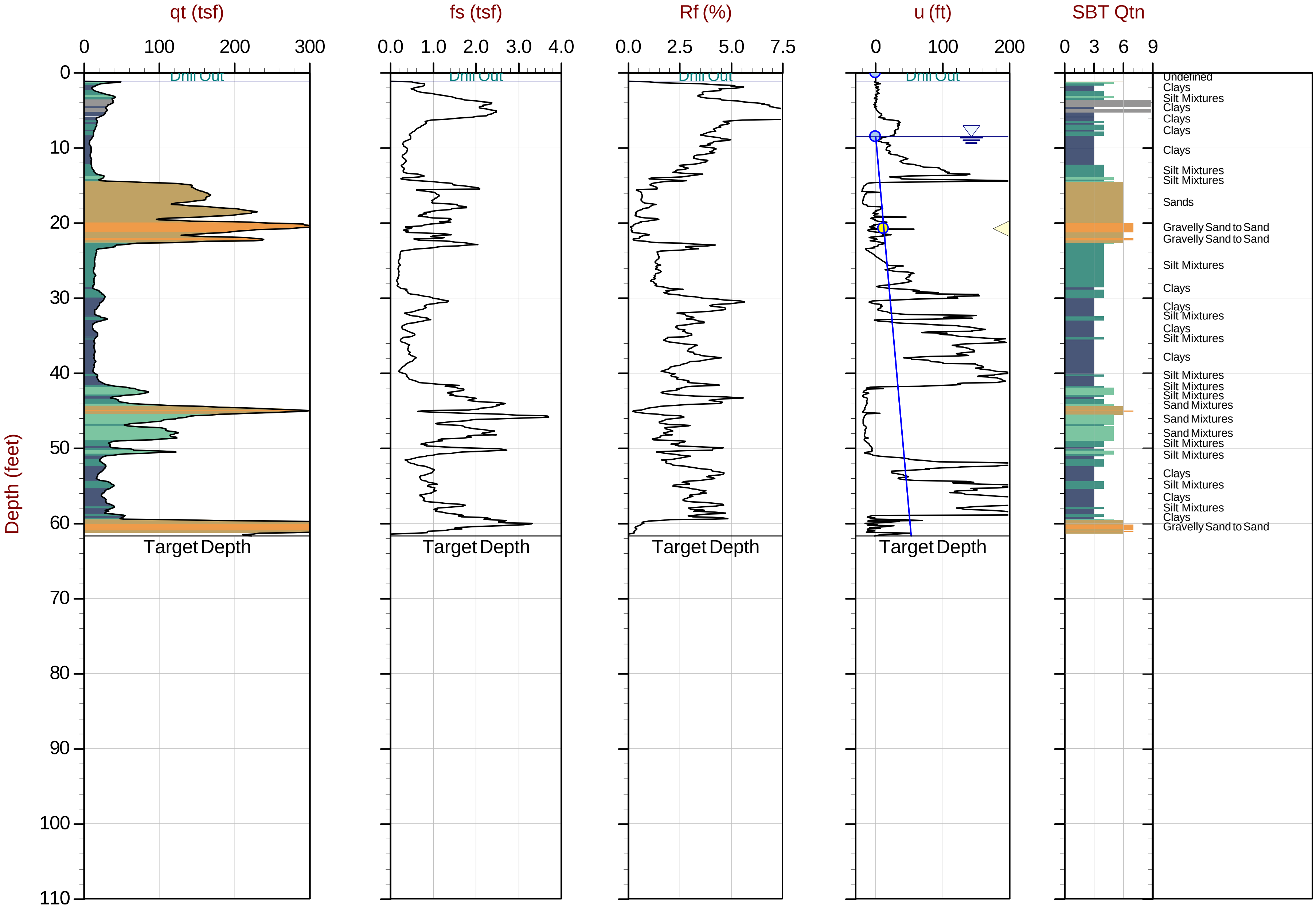
1. The assumed phreatic surface was based on pore pressure dissipation tests, unless otherwise noted. Hydrostatic conditions were assumed for the calculated parameters.
2. The coordinates were acquired using consumer grade GPS equipment in datum: WGS84 / UTM Zone 10 North.

CONETEC

Langan Engineering

Job No: 19-56026
Date: 2019-03-04 07:46
Site: North Town

Sounding: CPT-01
Cone: 483:T1500F15U500



Max Depth: 18.800 m / 61.68 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56026_CP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4137837m E: 594358m
Sheet No: 1 of 1

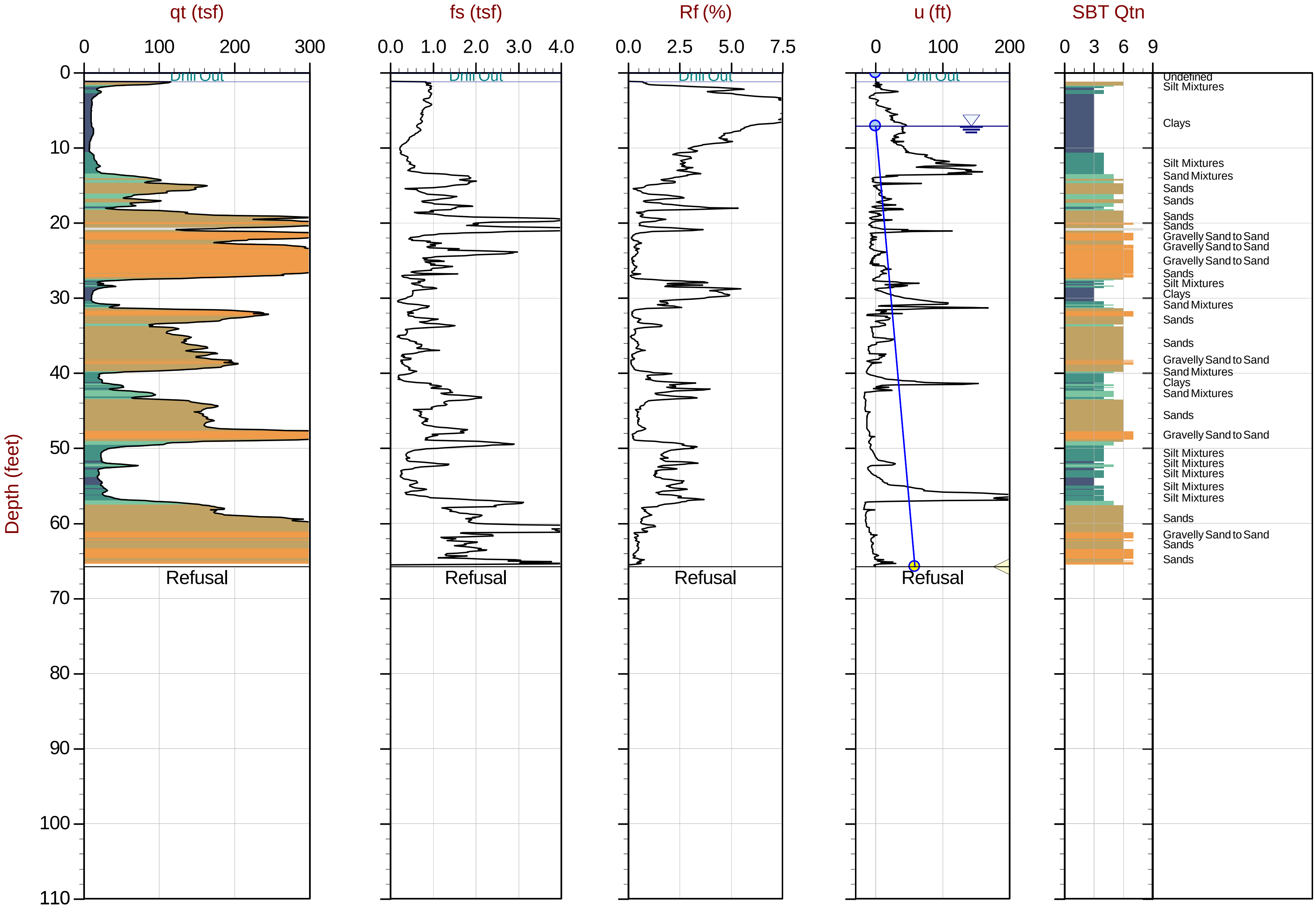
Overplot Item: ● Ueq ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved ◁ Dissipation, Ueq assumed — Hydrostatic Line

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

Langan Engineering

Job No: 19-56026
Date: 2019-03-04 09:00
Site: North Town

Sounding: CPT-02
Cone: 483:T1500F15U500



Max Depth: 20.050 m / 65.78 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56026_CP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4137883m E: 594393m
Sheet No: 1 of 1

Overplot Item: ● Ueq ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved ◁ Dissipation, Ueq assumed — Hydrostatic Line

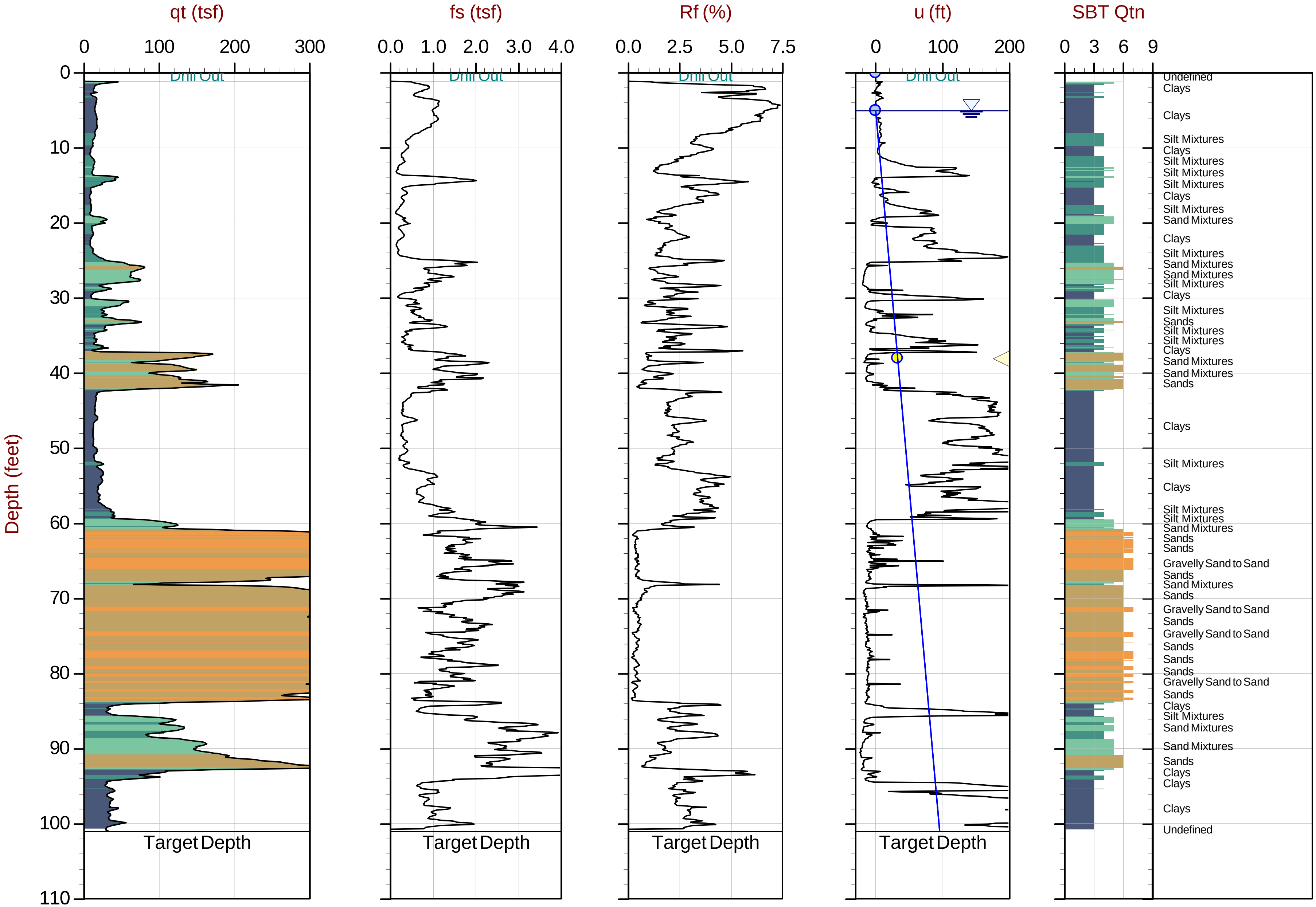
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Langan Engineering

Job No: 19-56026
Date: 2019-03-05 07:37
Site: North Town

Sounding: CPT-03
Cone: 483:T1500F15U500



Max Depth: 30.800 m / 101.05 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56026_CP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4137871m E: 594449m
Sheet No: 1 of 1

Overplot Item: ● Ueq ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved ◁ Dissipation, Ueq assumed — Hydrostatic Line

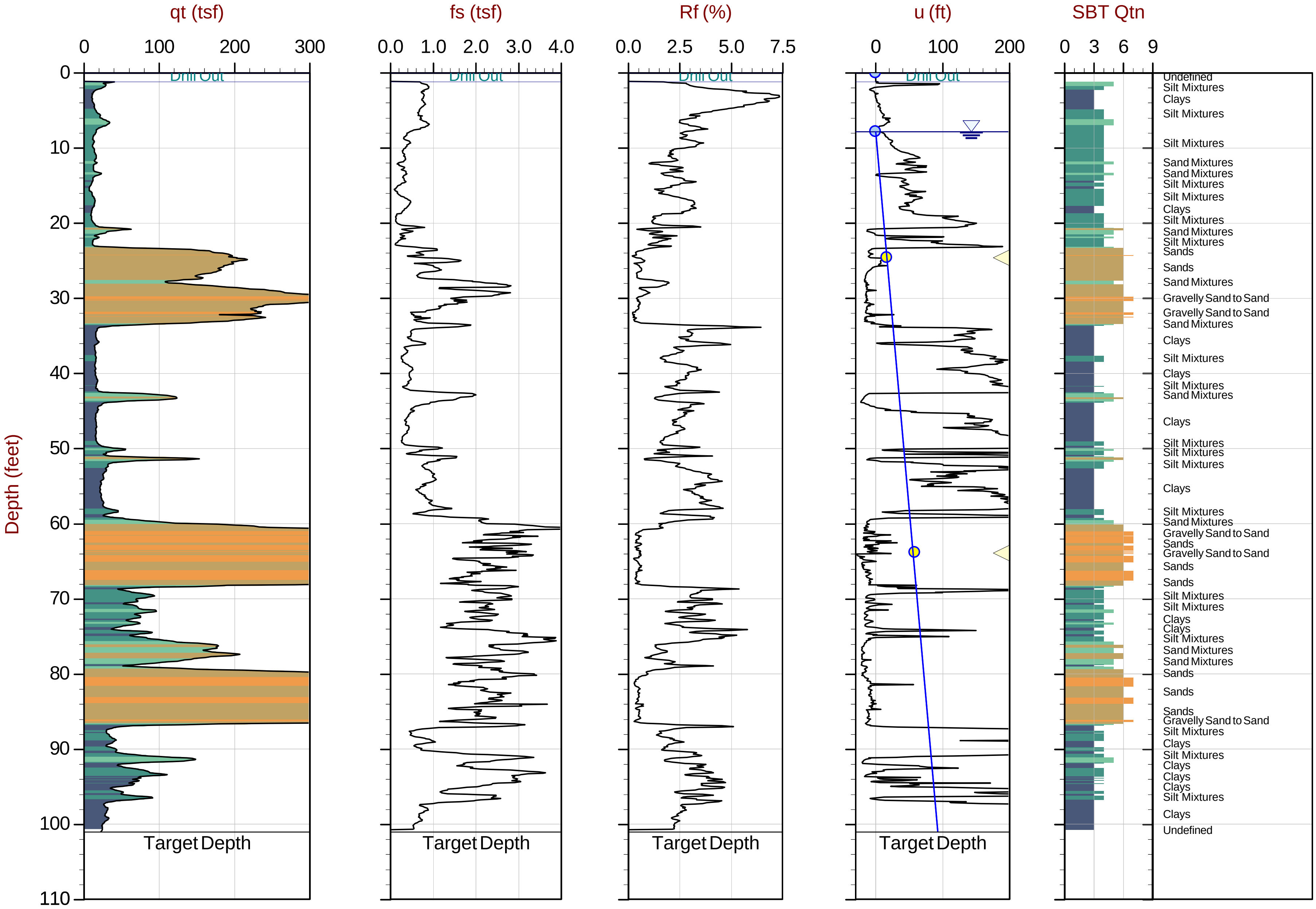
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Langan Engineering

Job No: 19-56026
Date: 2019-03-05 09:39
Site: North Town

Sounding: CPT-04
Cone: 483:T1500F15U500



Max Depth: 30.800 m / 101.05 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56026_CP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4137781m E: 594420m
Sheet No: 1 of 1

Overplot Item: ● Ueq ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved ◁ Dissipation, Ueq assumed — Hydrostatic Line

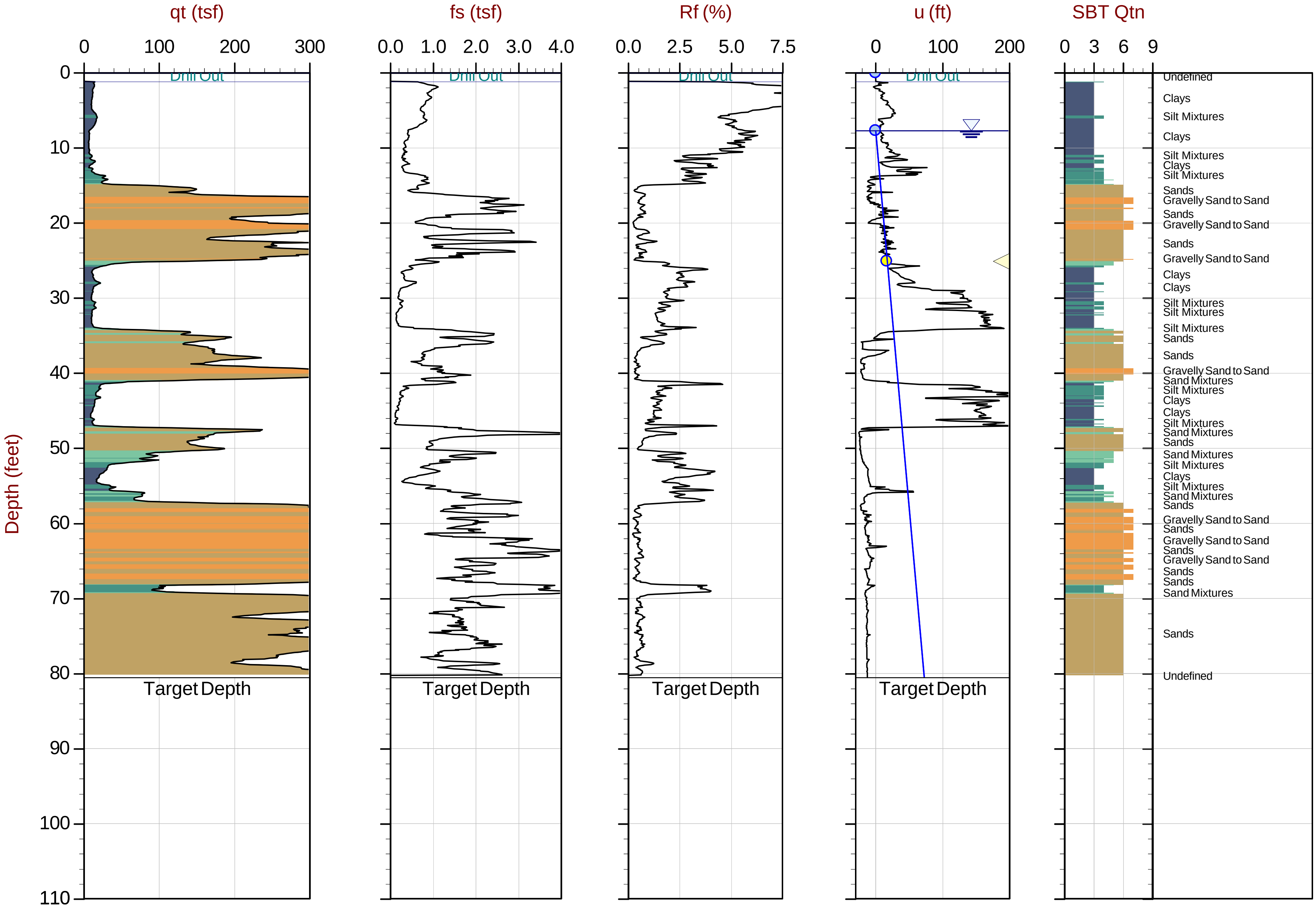
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Langan Engineering

Job No: 19-56026
Date: 2019-03-05 08:38
Site: North Town

Sounding: CPT-05
Cone: 483:T1500F15U500



Max Depth: 24.550 m / 80.54 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56026_CP05.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4137830m E: 594449m
Sheet No: 1 of 1

Overplot Item: ● Ueq ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved ◁ Dissipation, Ueq assumed — Hydrostatic Line

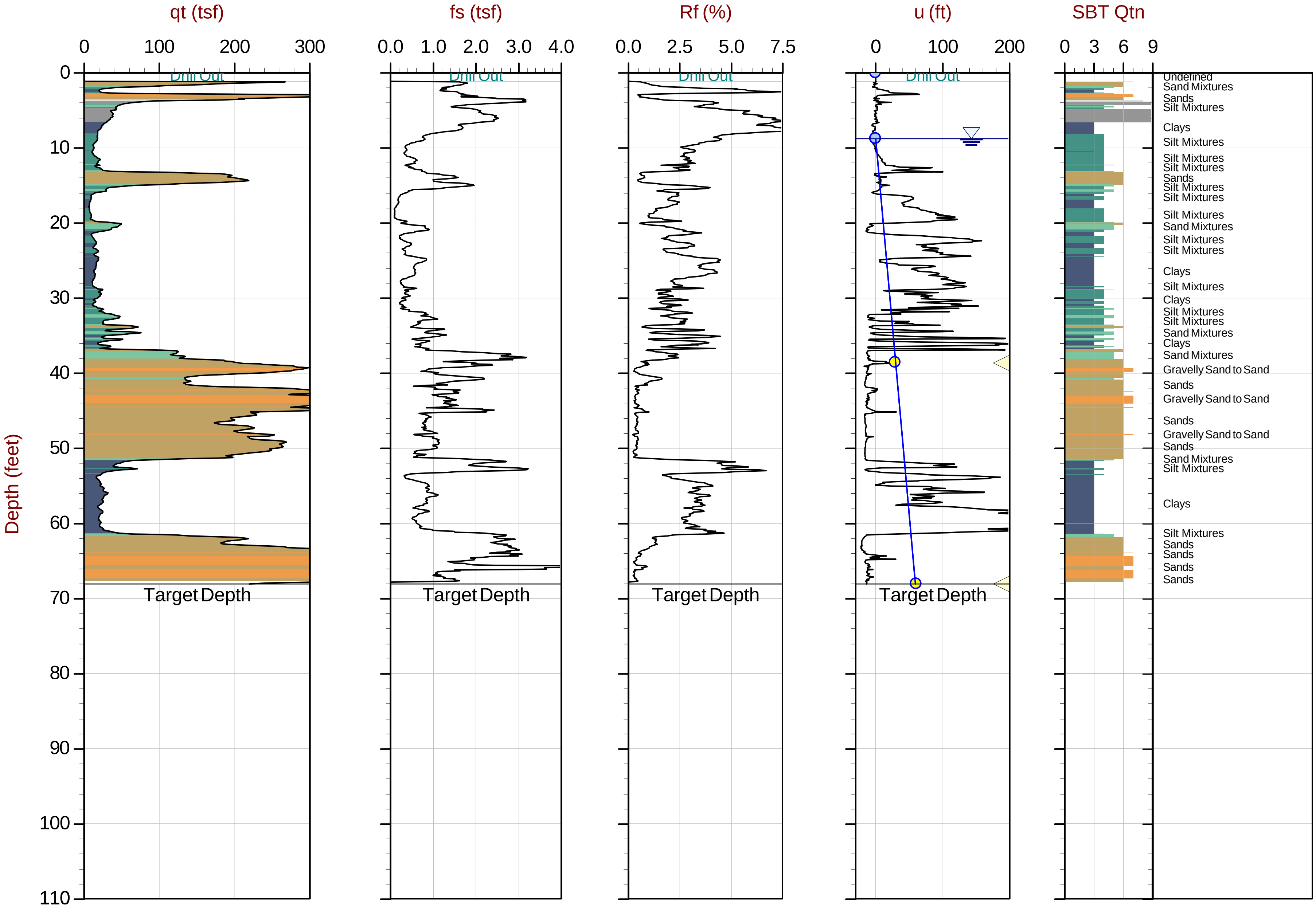
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Langan Engineering

Job No: 19-56026
Date: 2019-03-04 10:12
Site: North Town

Sounding: CPT-06
Cone: 483:T1500F15U500



Max Depth: 20.750 m / 68.08 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56026_CP06.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4137932m E: 594476m
Sheet No: 1 of 1

Overplot Item: ● Ueq ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved ◁ Dissipation, Ueq assumed — Hydrostatic Line

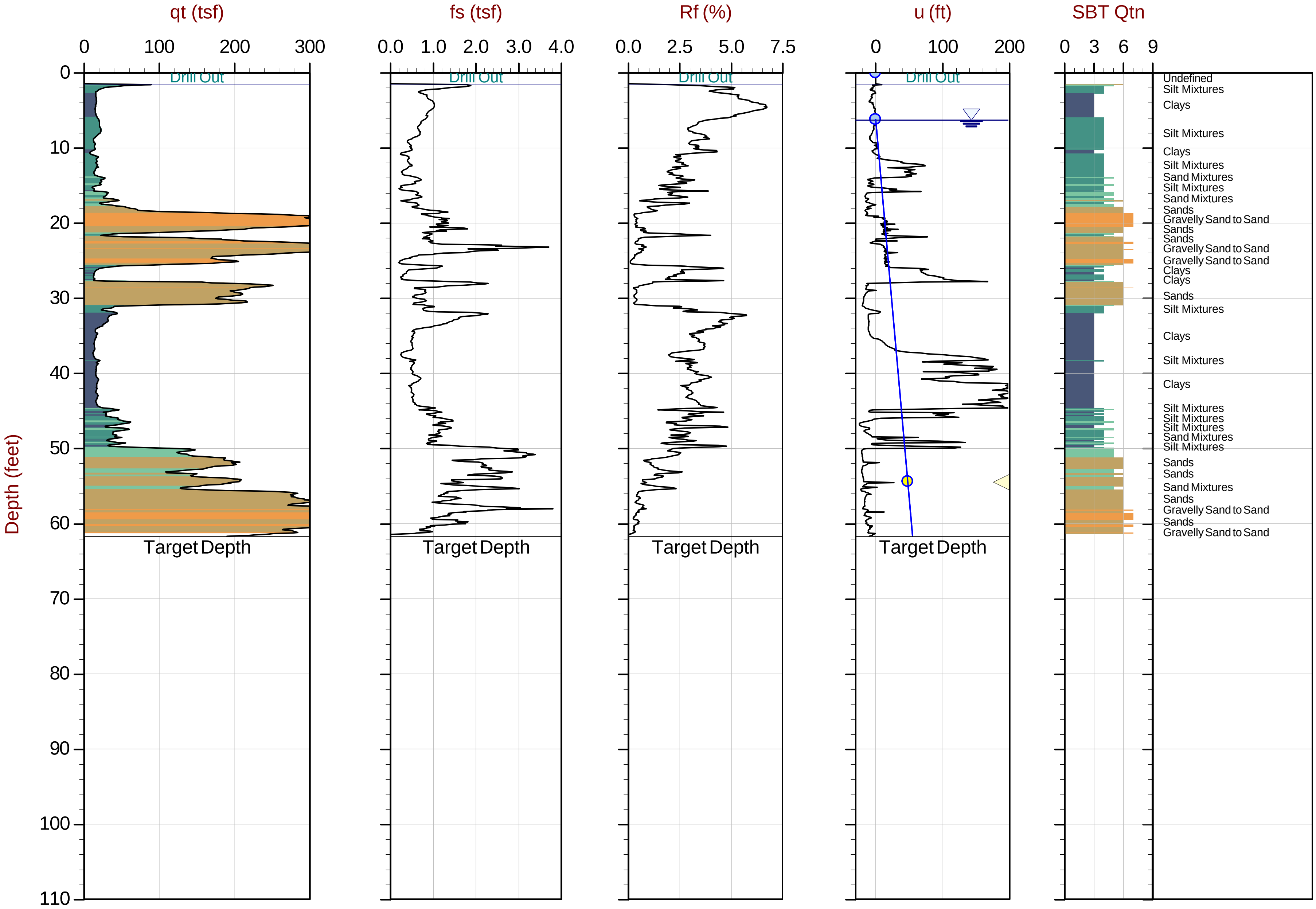
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Langan Engineering

Job No: 19-56026
Date: 2019-03-04 11:10
Site: North Town

Sounding: CPT-07
Cone: 483:T1500F15U500



Max Depth: 18.800 m / 61.68 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56026_CP07.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4137849m E: 594538m
Sheet No: 1 of 1

Overplot Item: 🟡 Ueq ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved ◁ Dissipation, Ueq assumed — Hydrostatic Line

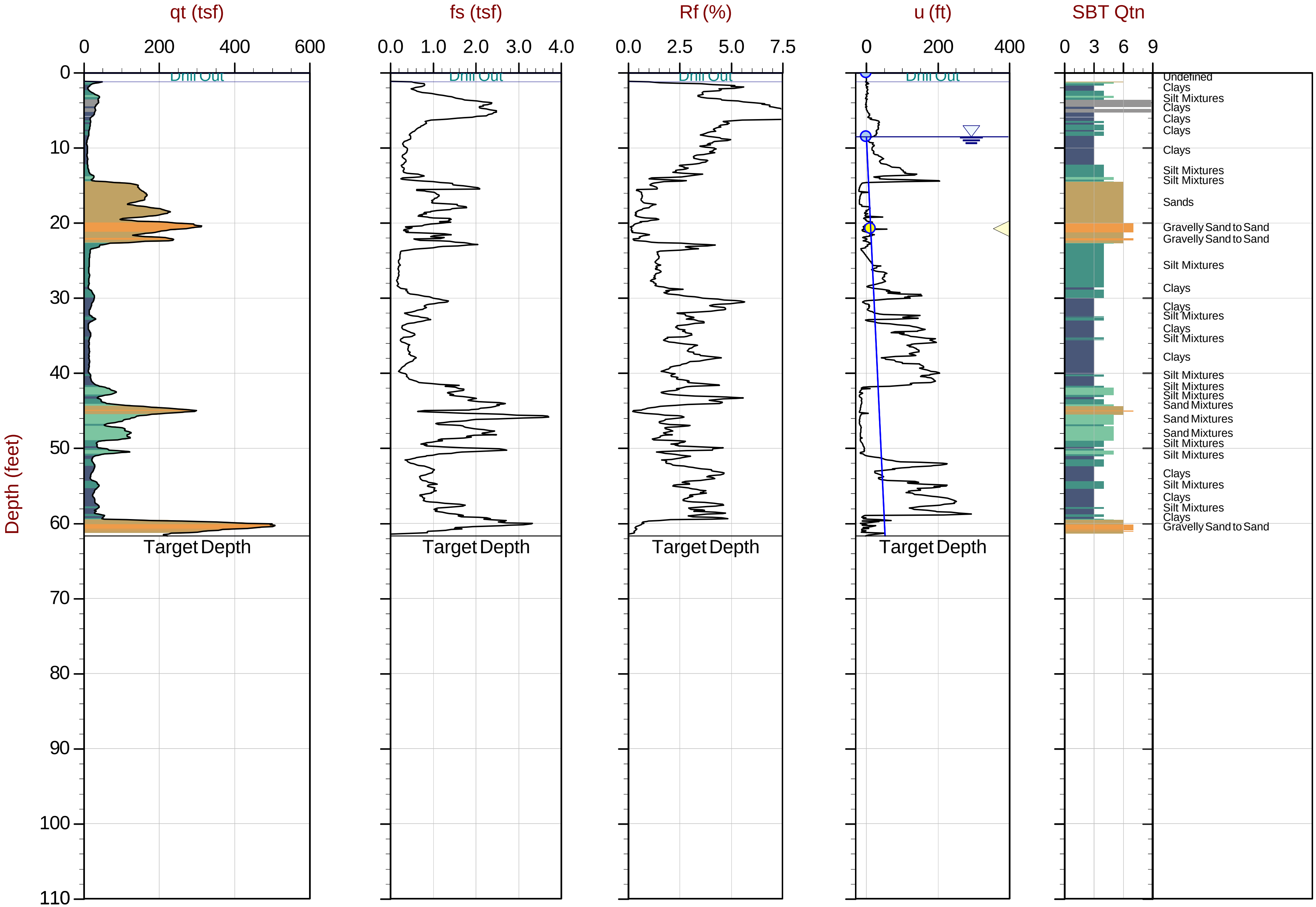
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Standard Cone Penetration Test Plots with Expanded Scales

Langan Engineering

Job No: 19-56026
Date: 2019-03-04 07:46
Site: North Town

Sounding: CPT-01
Cone: 483:T1500F15U500



Max Depth: 18.800 m / 61.68 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56026_CP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4137837m E: 594358m
Sheet No: 1 of 1

Overplot Item: ● Ueq ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved ◁ Dissipation, Ueq assumed — Hydrostatic Line

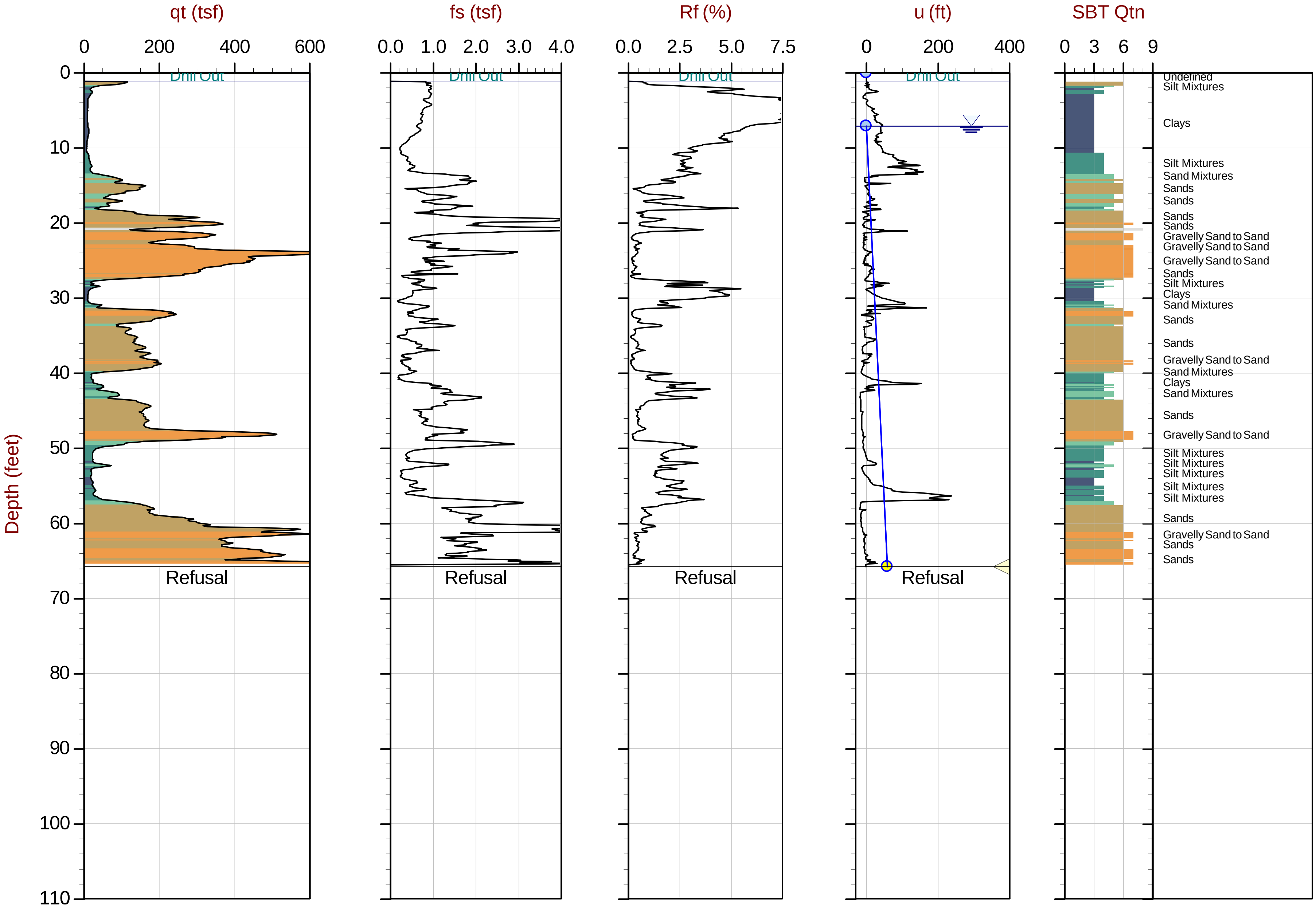
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Langan Engineering

Job No: 19-56026
Date: 2019-03-04 09:00
Site: North Town

Sounding: CPT-02
Cone: 483:T1500F15U500



Max Depth: 20.050 m / 65.78 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56026_CP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4137883m E: 594393m
Sheet No: 1 of 1

Overplot Item: ● Ueq ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved ◁ Dissipation, Ueq assumed — Hydrostatic Line

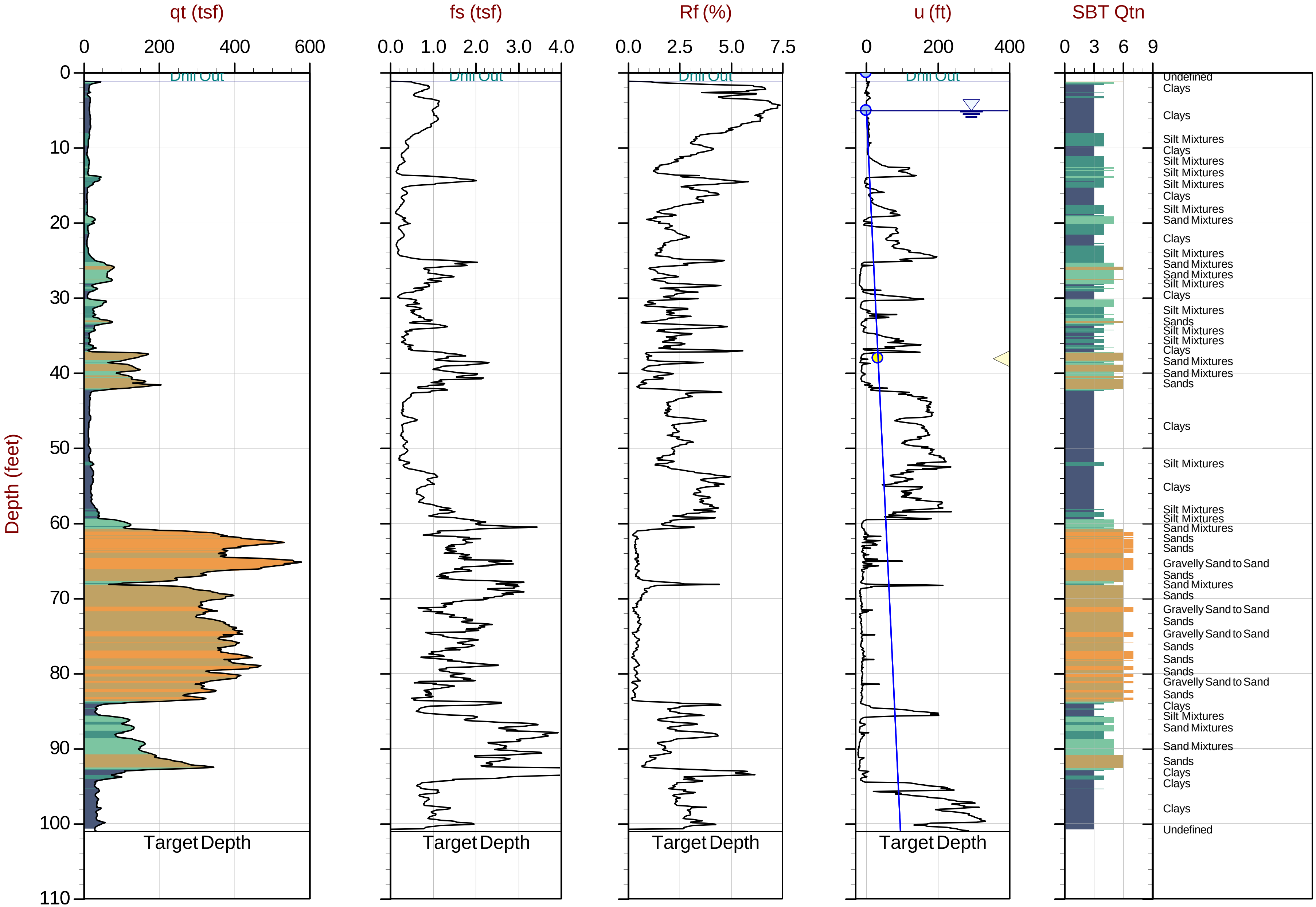
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Langan Engineering

Job No: 19-56026
Date: 2019-03-05 07:37
Site: North Town

Sounding: CPT-03
Cone: 483:T1500F15U500



Max Depth: 30.800 m / 101.05 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56026_CP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4137871m E: 594449m
Sheet No: 1 of 1

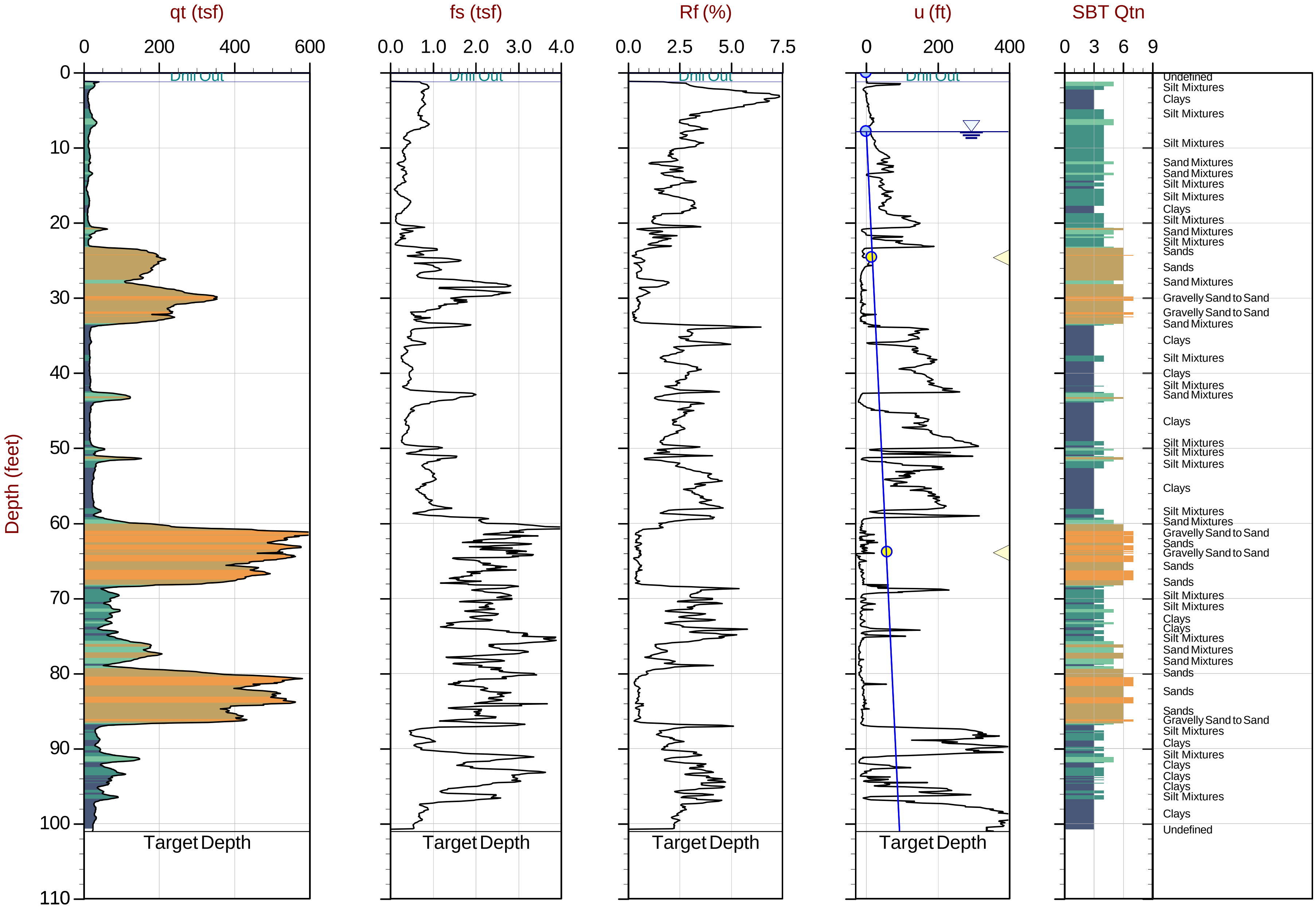
Overplot Item: ● Ueq ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved ◁ Dissipation, Ueq assumed — Hydrostatic Line

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

Langan Engineering

Job No: 19-56026
Date: 2019-03-05 09:39
Site: North Town

Sounding: CPT-04
Cone: 483:T1500F15U500



Max Depth: 30.800 m / 101.05 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56026_CP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4137781m E: 594420m
Sheet No: 1 of 1

Overplot Item: ● Ueq ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved ◁ Dissipation, Ueq assumed — Hydrostatic Line

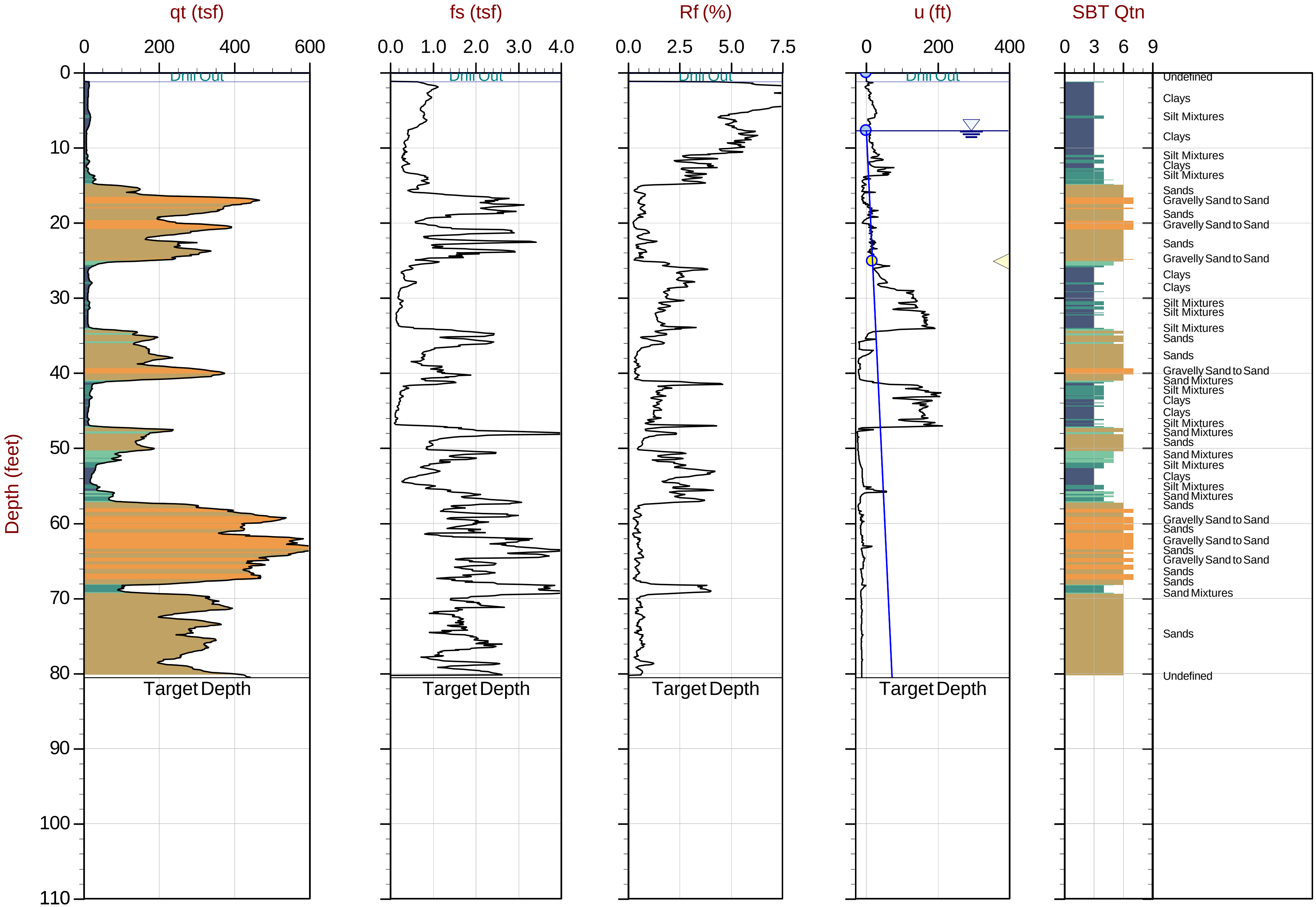
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Langan Engineering

Job No: 19-56026
Date: 2019-03-05 08:38
Site: North Town

Sounding: CPT-05
Cone: 483:T1500F15U500



Max Depth: 24.550 m / 80.54 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56026_CP05.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4137830m E: 594449m
Sheet No: 1 of 1

Overplot Item: ● Ueq ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved ◁ Dissipation, Ueq assumed — Hydrostatic Line

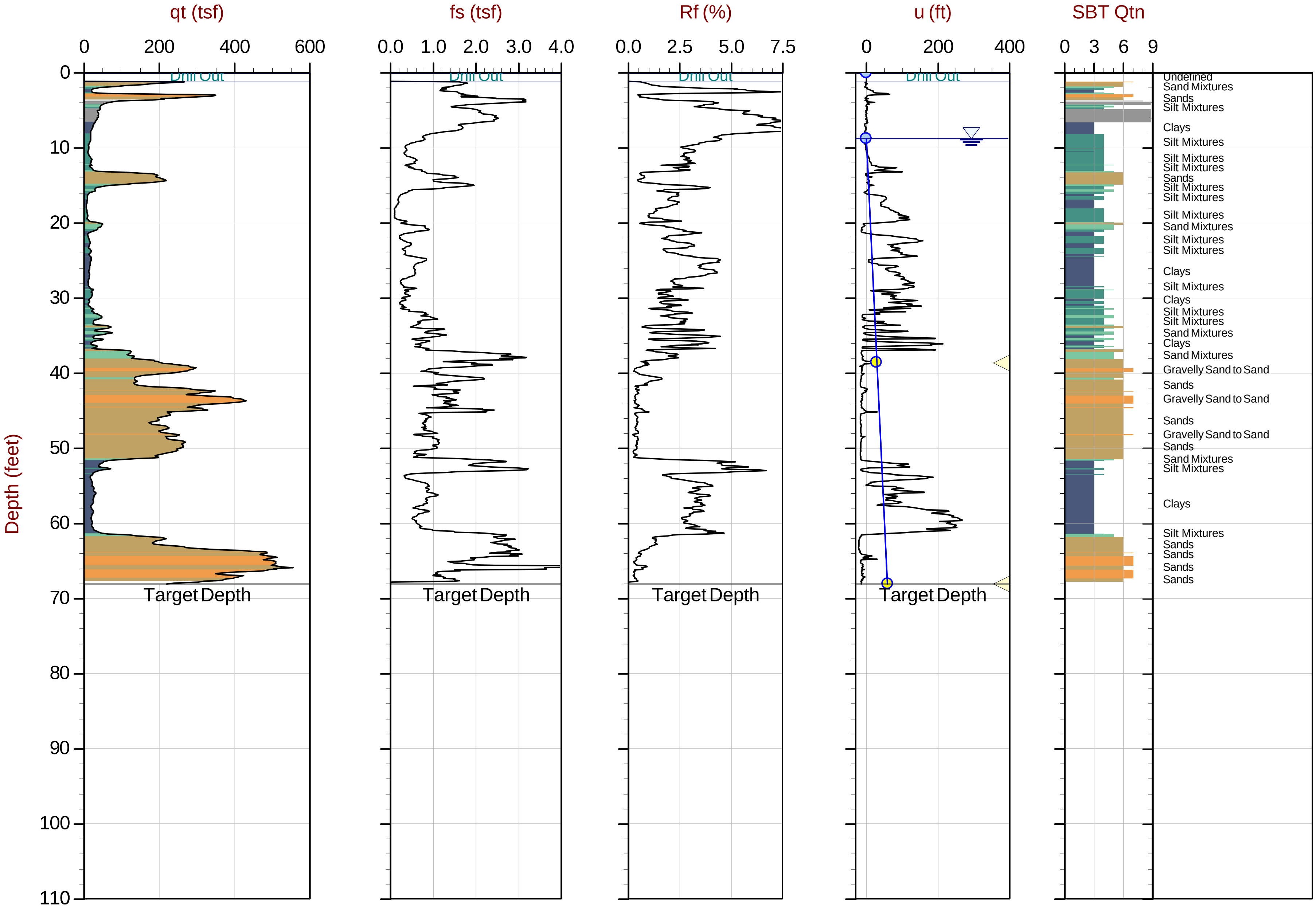
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Langan Engineering

Job No: 19-56026
Date: 2019-03-04 10:12
Site: North Town

Sounding: CPT-06
Cone: 483:T1500F15U500



Max Depth: 20.750 m / 68.08 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56026_CP06.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4137932m E: 594476m
Sheet No: 1 of 1

Overplot Item: ● Ueq ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved ◁ Dissipation, Ueq assumed — Hydrostatic Line

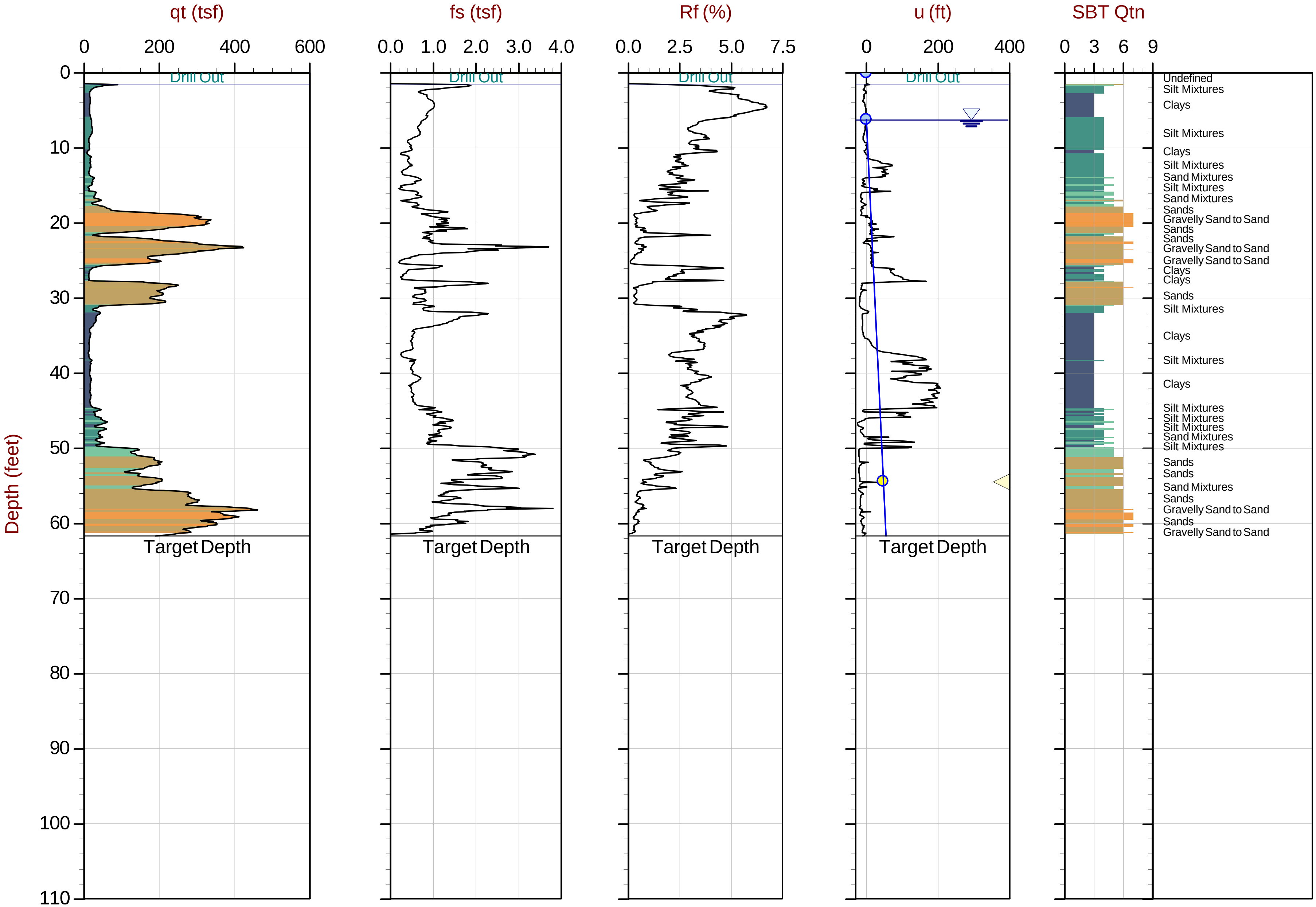
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Langan Engineering

Job No: 19-56026
Date: 2019-03-04 11:10
Site: North Town

Sounding: CPT-07
Cone: 483:T1500F15U500



Max Depth: 18.800 m / 61.68 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56026_CP07.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4137849m E: 594538m
Sheet No: 1 of 1

Overplot Item: ● Ueq ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved ◁ Dissipation, Ueq assumed — Hydrostatic Line

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

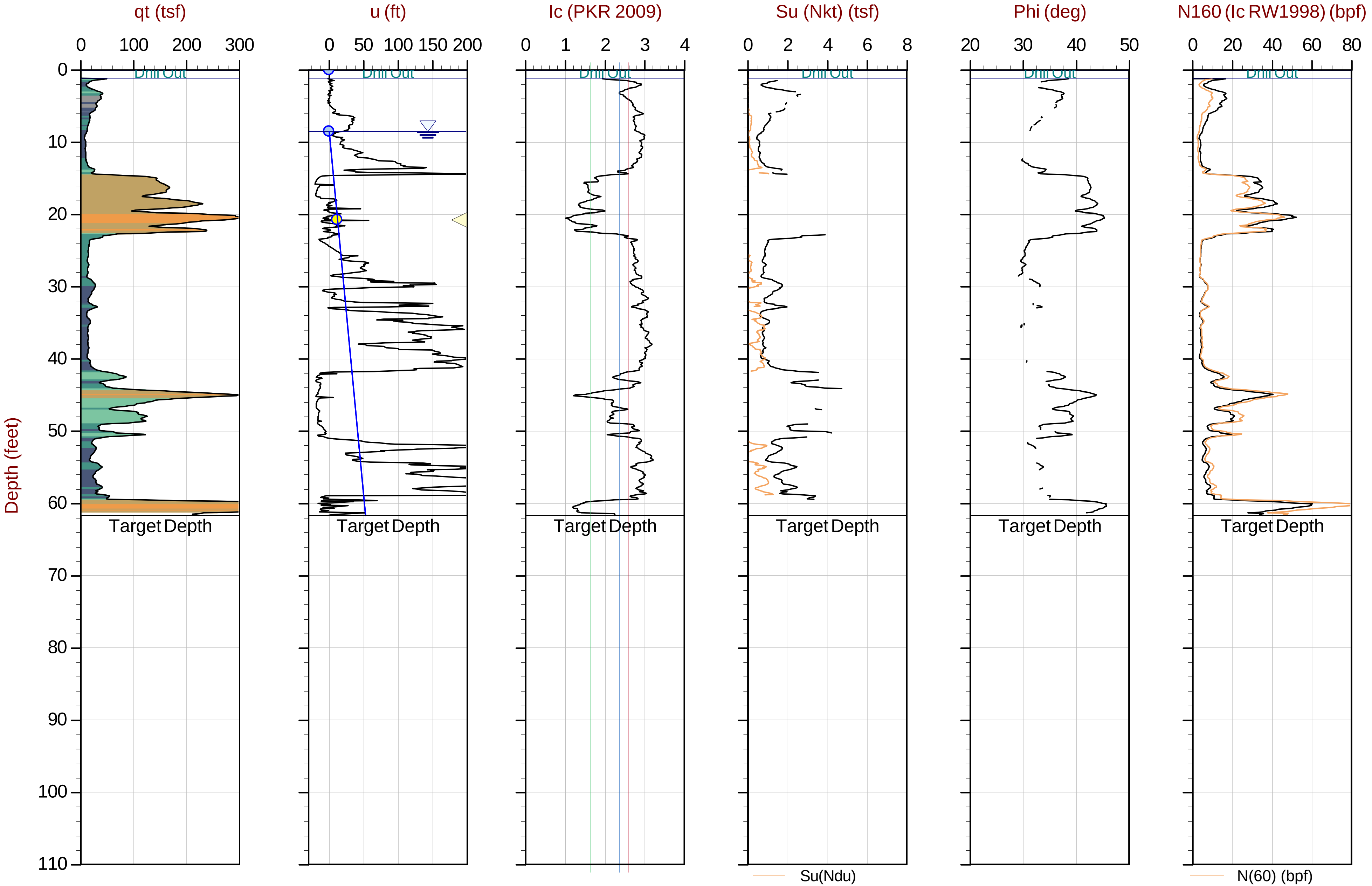
Advanced Cone Penetration Test Plots with Ic, Su(Nkt), Phi and N1(60)Ic

CONETEC

Langan Engineering

Job No: 19-56026
Date: 2019-03-04 07:46
Site: North Town

Sounding: CPT-01
Cone: 483:T1500F15U500



Max Depth: 18.800 m / 61.68 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56026_CP01.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4137837m E: 594358m
Sheet No: 1 of 1

Overplot Item: ● Ueq ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved ◁ Dissipation, Ueq assumed — Hydrostatic Line

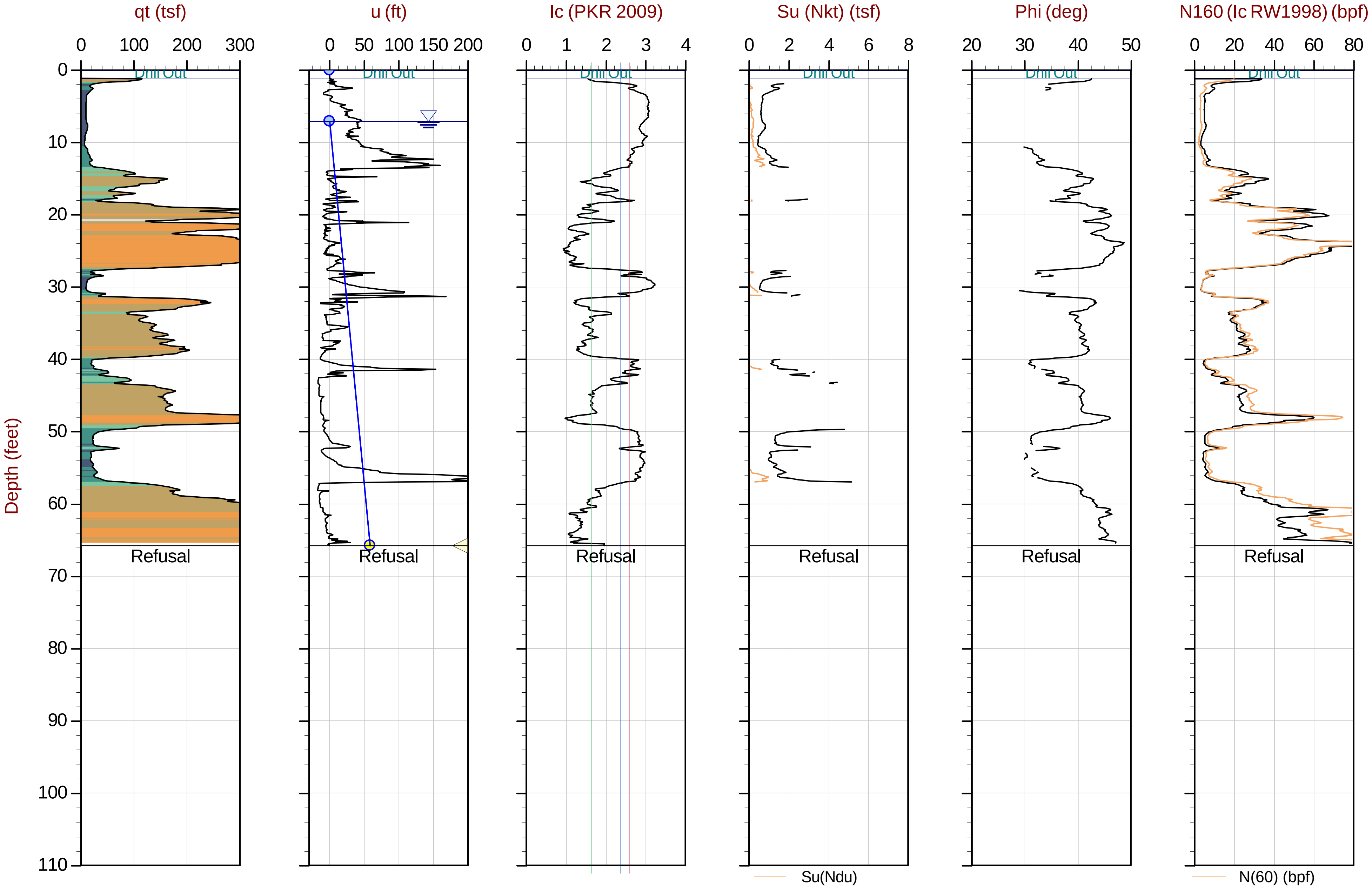
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Langan Engineering

Job No: 19-56026
Date: 2019-03-04 09:00
Site: North Town

Sounding: CPT-02
Cone: 483:T1500F15U500



Max Depth: 20.050 m / 65.78 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56026_CP02.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4137883m E: 594393m
Sheet No: 1 of 1

Overplot Item: ● Ueq ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved ◁ Dissipation, Ueq assumed — Hydrostatic Line

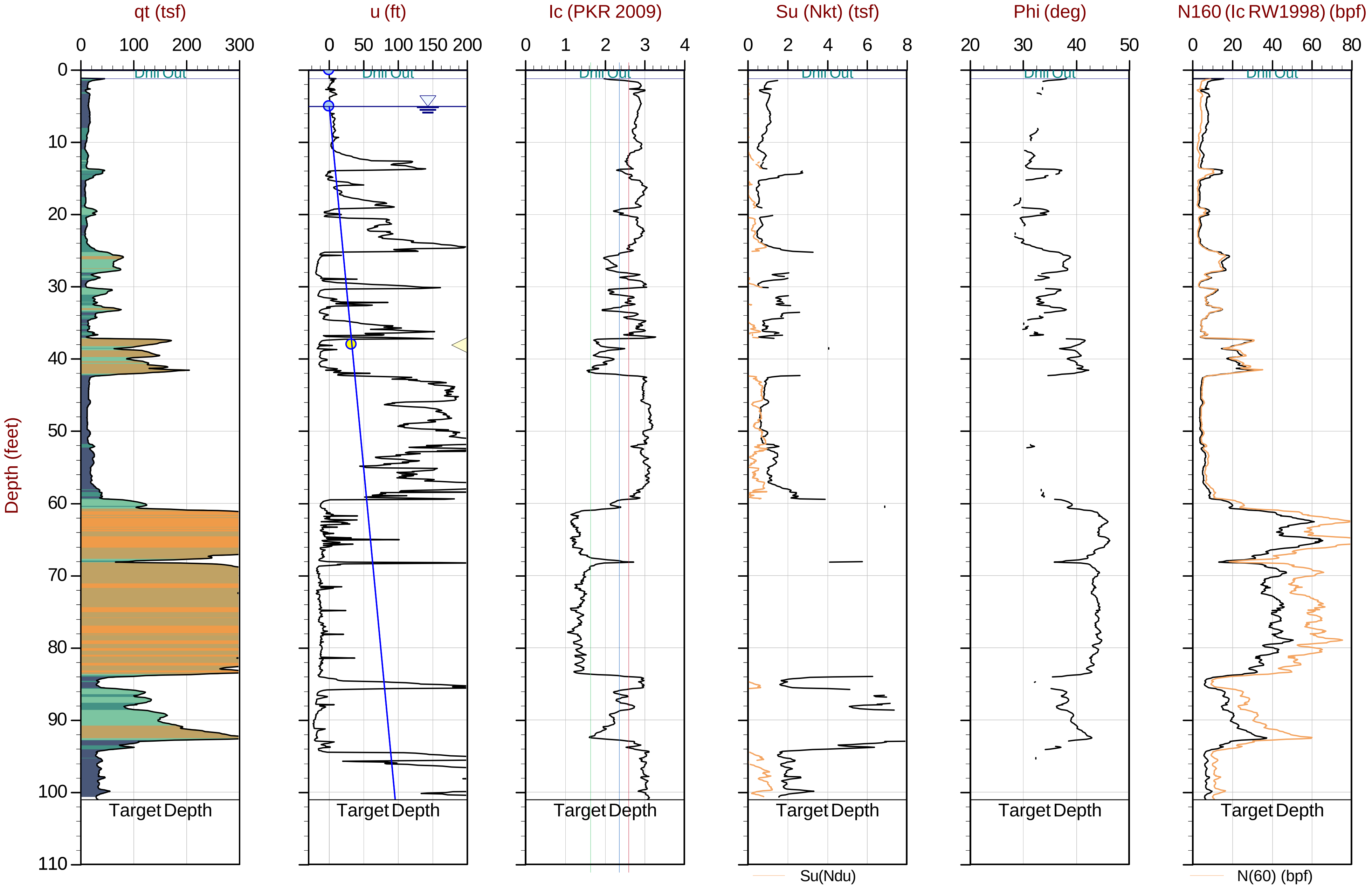
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Langan Engineering

Job No: 19-56026
Date: 2019-03-05 07:37
Site: North Town

Sounding: CPT-03
Cone: 483:T1500F15U500



Max Depth: 30.800 m / 101.05 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56026_CP03.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4137871m E: 594449m
Sheet No: 1 of 1

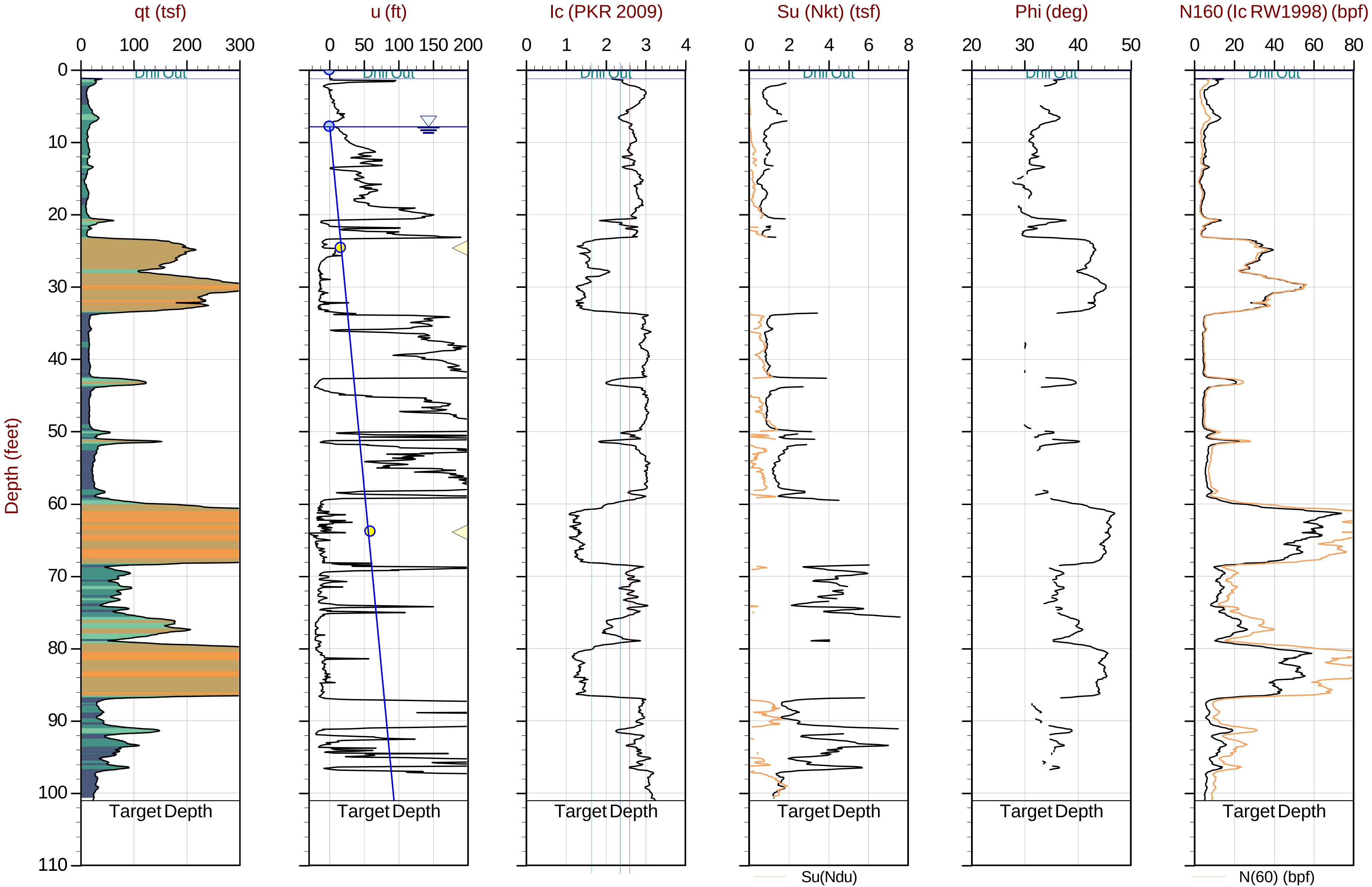
Overplot Item: ● Ueq ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved ◁ Dissipation, Ueq assumed — Hydrostatic Line

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

Langan Engineering

Job No: 19-56026
Date: 2019-03-05 09:39
Site: North Town

Sounding: CPT-04
Cone: 483:T1500F15U500



Max Depth: 30.800 m / 101.05 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56026_CP04.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4137781m E: 594420m
Sheet No: 1 of 1

Overplot Item: ● Ueq ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved ◁ Dissipation, Ueq assumed — Hydrostatic Line

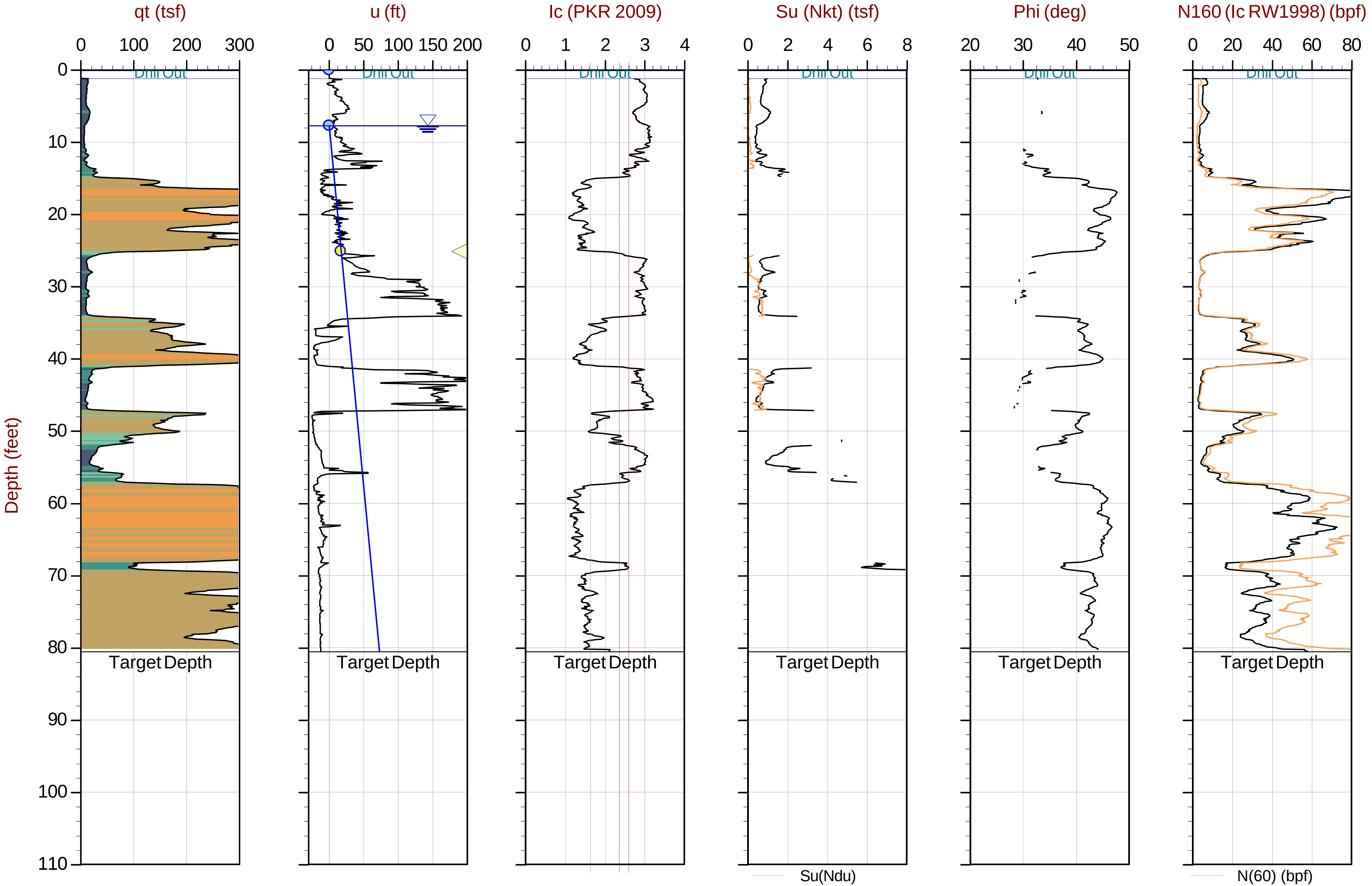
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Langan Engineering

Job No: 19-56026
Date: 2019-03-05 08:38
Site: North Town

Sounding: CPT-05
Cone: 483:T1500F15U500



Max Depth: 24.550 m / 80.54 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56026_CP05.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4137830m E: 594449m
Sheet No: 1 of 1

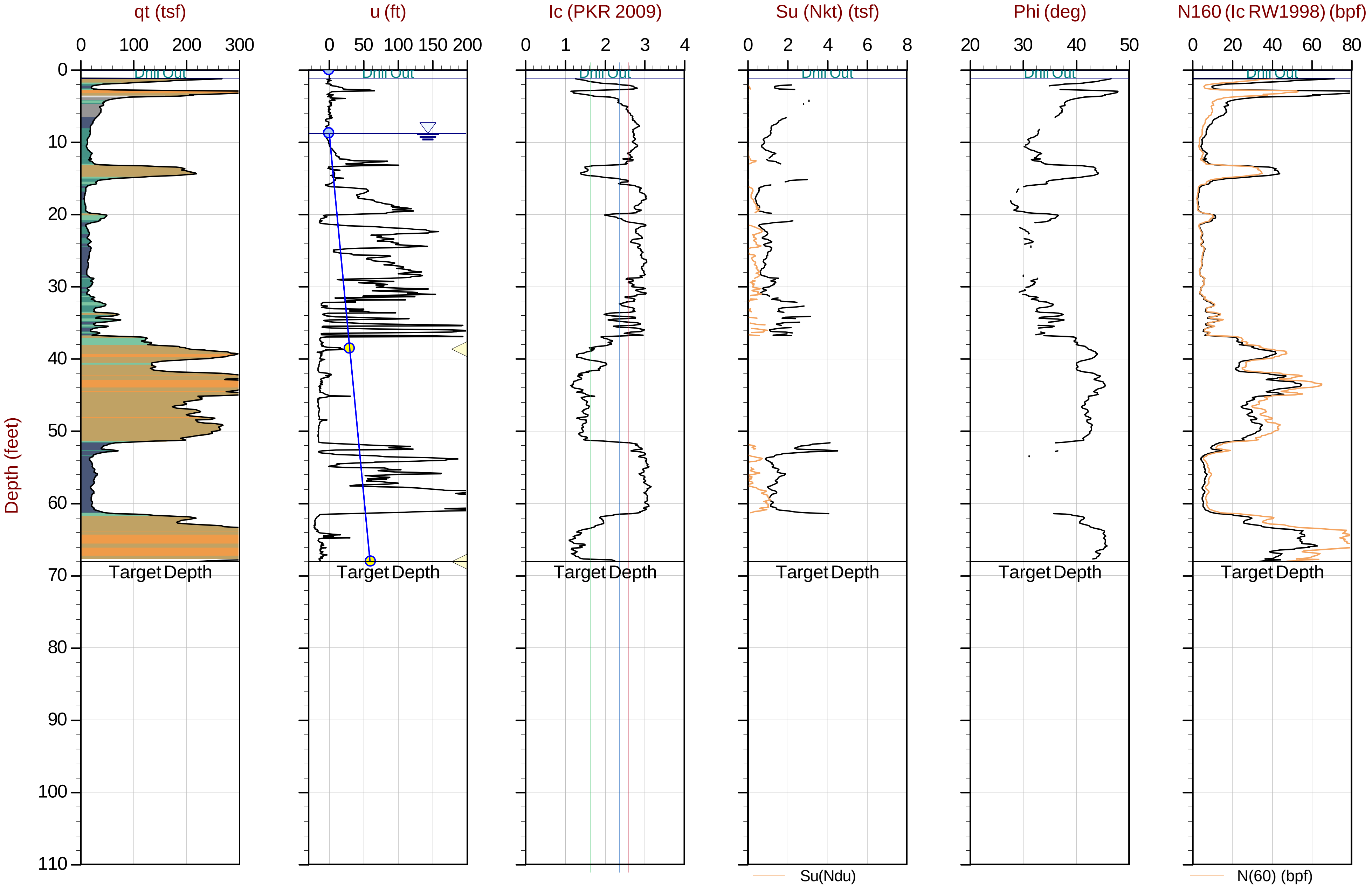
Overplot Item: ● Ueq ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved ◁ Dissipation, Ueq assumed — Hydrostatic Line

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

Langan Engineering

Job No: 19-56026
Date: 2019-03-04 10:12
Site: North Town

Sounding: CPT-06
Cone: 483:T1500F15U500



Max Depth: 20.750 m / 68.08 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56026_CP06.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4137932m E: 594476m
Sheet No: 1 of 1

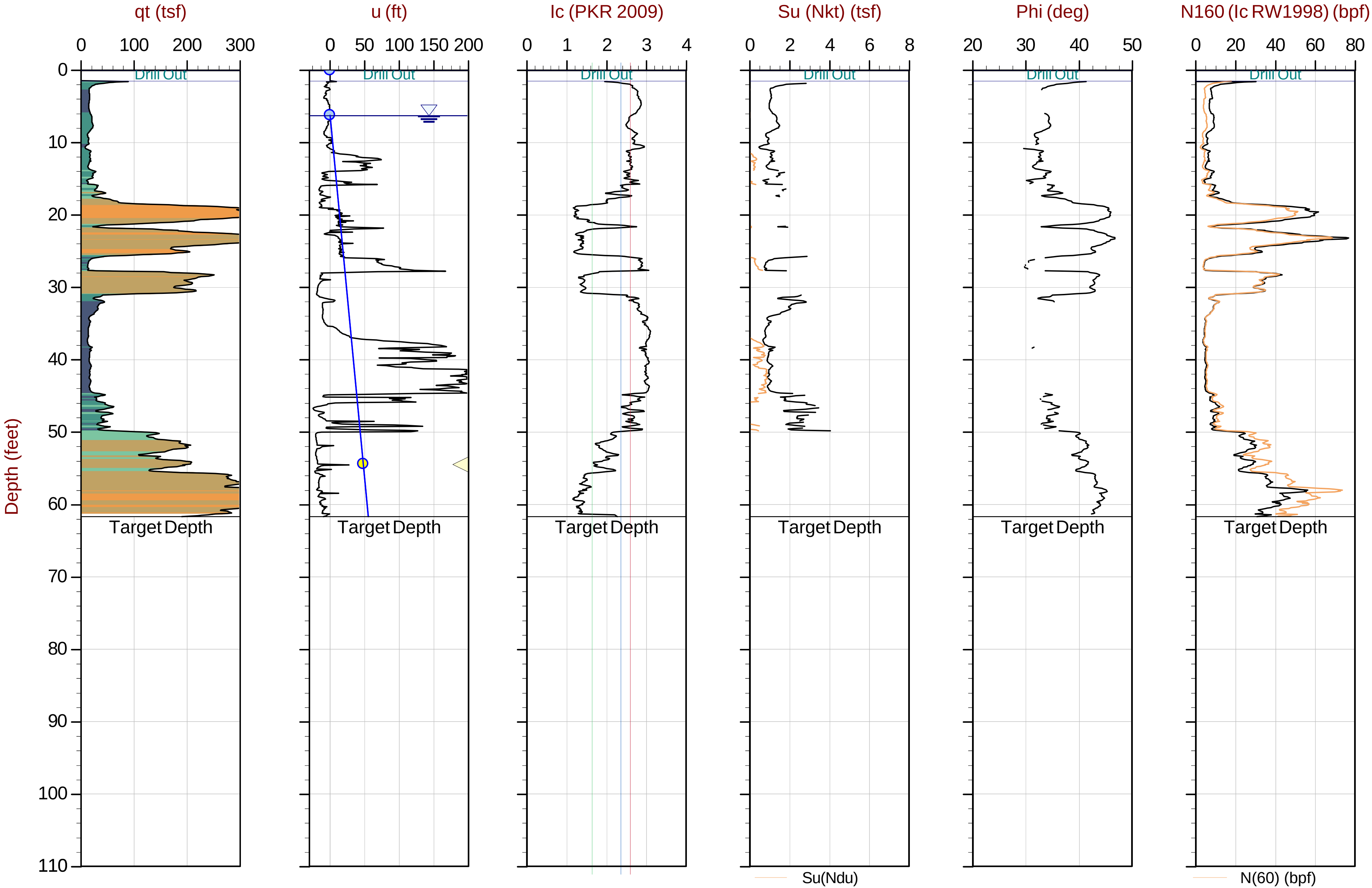
Overplot Item: ● Ueq ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved ◁ Dissipation, Ueq assumed — Hydrostatic Line

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

Langan Engineering

Job No: 19-56026
Date: 2019-03-04 11:10
Site: North Town

Sounding: CPT-07
Cone: 483:T1500F15U500



Max Depth: 18.800 m / 61.68 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 19-56026_CP07.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4137849m E: 594538m
Sheet No: 1 of 1

Overplot Item: ● Ueq ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved ◁ Dissipation, Ueq assumed — Hydrostatic Line

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

Soil Behavior Type (SBT) Scatter Plots

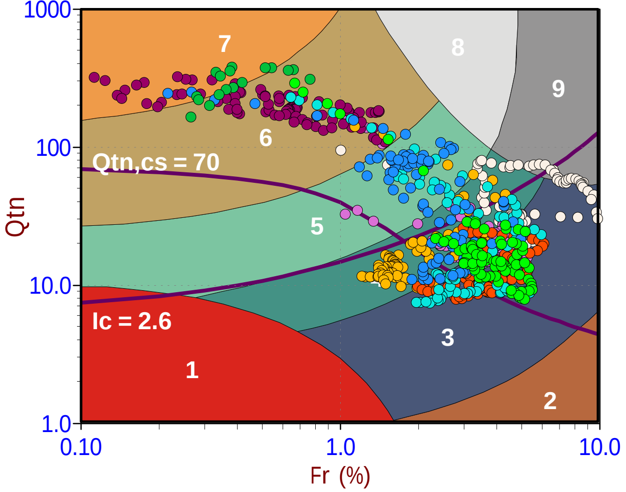
CONETEC

Langan Engineering

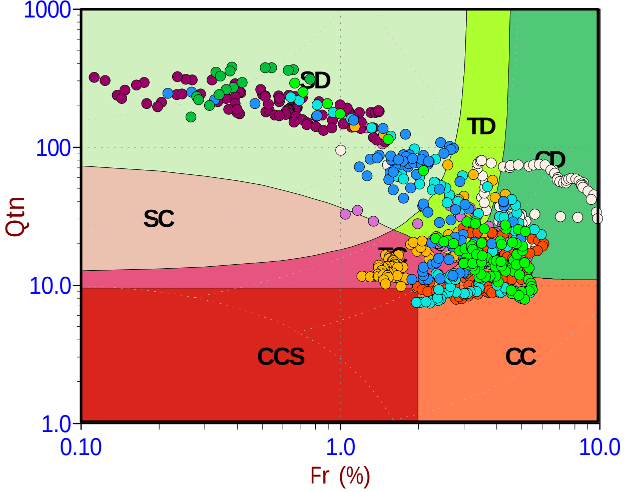
Job No: 19-56026
Date: 2019-03-04 07:46
Site: North Town

Sounding: CPT-01
Cone: 483:T1500F15U500

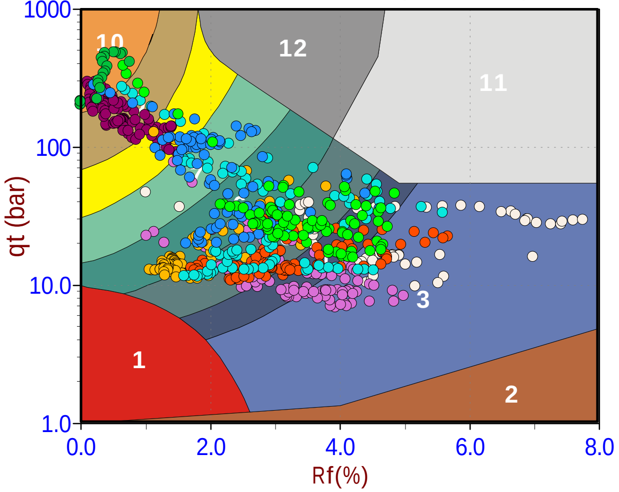
Qtn Chart (PKR 2009)



Modified SBTn (PKR 2016)



Standard SBT Chart (UBC 1986)



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

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

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

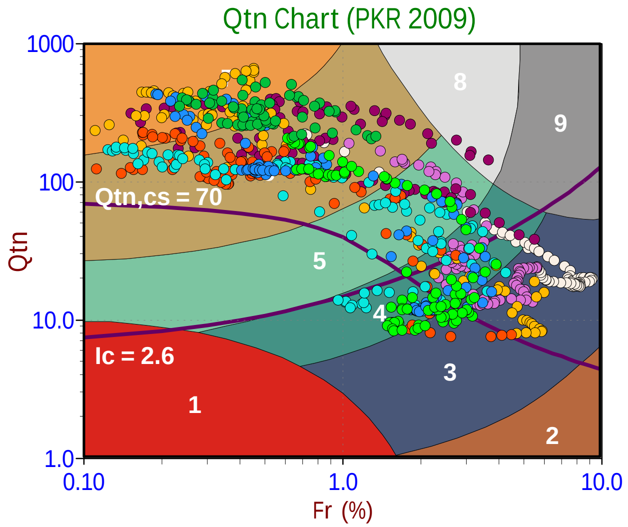
Legend
- 🟥 Sensitive Fines
- 🟫 Organic Soil
- 🟦 Clay
- ■ Silty Clay
- ■ Clayey Silt
- ■ Silt
- ■ Sandy Silt
- 🟨 Silty Sand/Sand
- ■ Sand
- 🟧 Gravelly Sand
- ■ Stiff Fine Grained
- ■ Cemented Sand

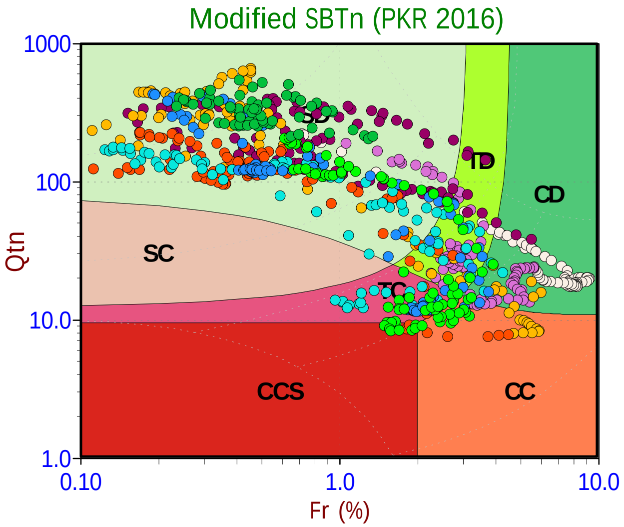
CONETEC

Langan Engineering

Job No: 19-56026
Date: 2019-03-04 09:00
Site: North Town

Sounding: CPT-02
Cone: 483:T1500F15U500


Qtn Chart (PKR 2009)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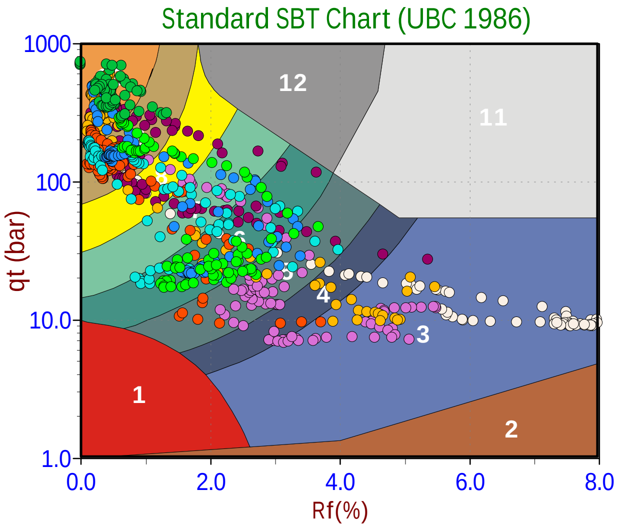

Standard SBT Chart (UBC 1986)

Depth Ranges

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

Legend

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

Legend

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

Legend

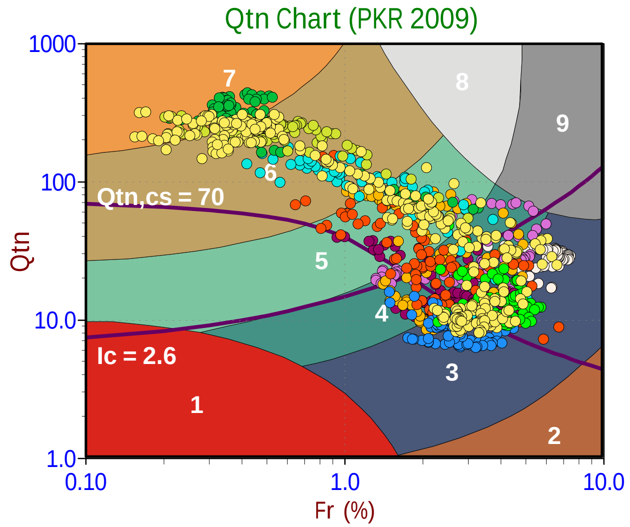
- ■ Sensitive Fines
- ■ Organic Soil
- ■ Clay
- ■ Silty Clay
- ■ Clayey Silt
- ■ Silt
- ■ Sandy Silt
- ■ Silty Sand/Sand
- ■ Sand
- ■ Gravelly Sand
- ■ Stiff Fine Grained
- ■ Cemented Sand

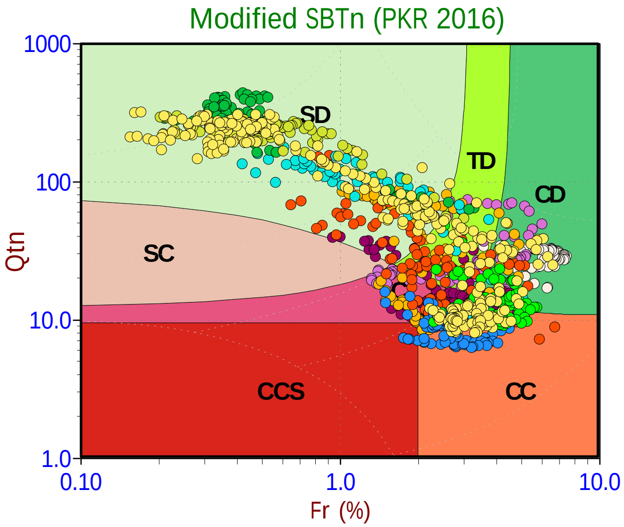
CONETEC

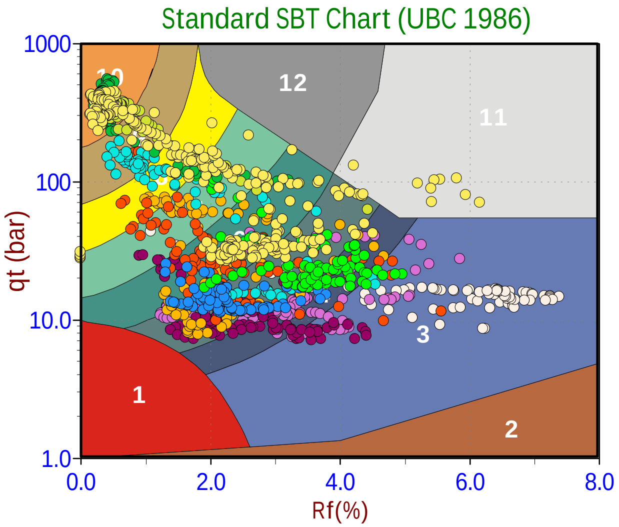
Langan Engineering

Job No: 19-56026
Date: 2019-03-05 07:37
Site: North Town

Sounding: CPT-03
Cone: 483:T1500F15U500


Qtn Chart (PKR 2009)


Modified SBTn (PKR 2016)


Standard SBT Chart (UBC 1986)

Depth Ranges

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

Legend

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

Legend

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

Legend

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

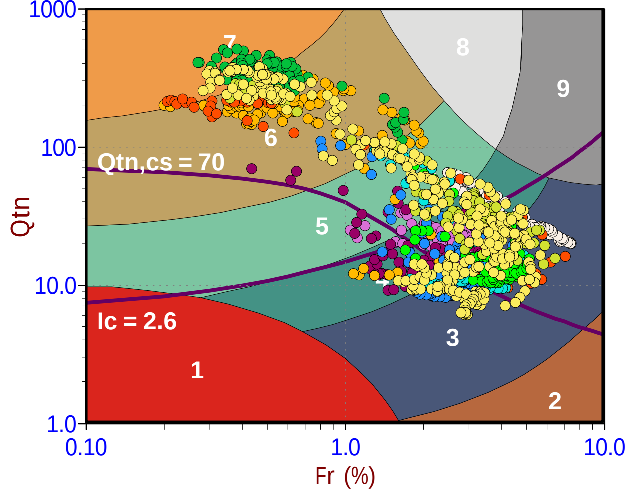
CONETEC

Langan Engineering

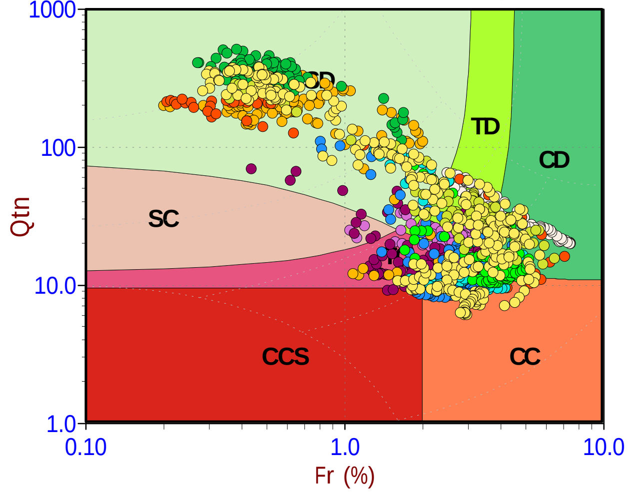
Job No: 19-56026
Date: 2019-03-05 09:39
Site: North Town

Sounding: CPT-04
Cone: 483:T1500F15U500

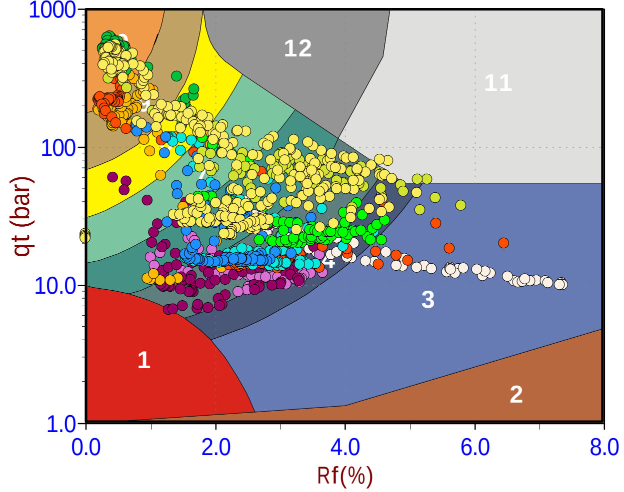
Qtn Chart (PKR 2009)



Modified SBTn (PKR 2016)



Standard SBT Chart (UBC 1986)



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

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

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

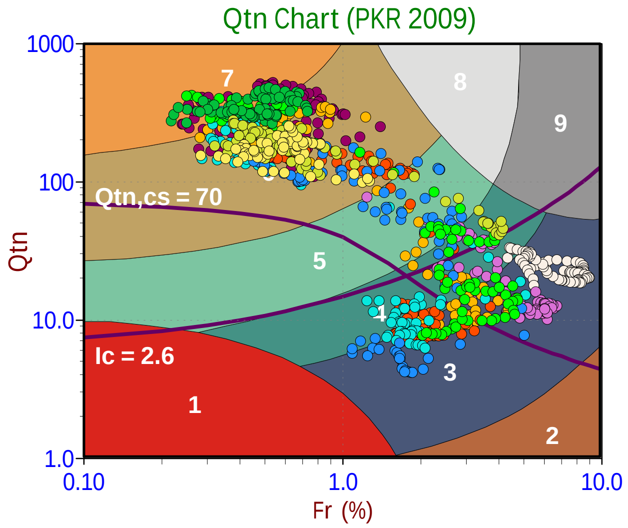
Legend
- ■ Sensitive Fines
- ■ Organic Soil
- ■ Clay
- ■ Silty Clay
- ■ Clayey Silt
- ■ Silt
- ■ Sandy Silt
- ■ Silty Sand/Sand
- ■ Sand
- ■ Gravelly Sand
- ■ Stiff Fine Grained
- ■ Cemented Sand

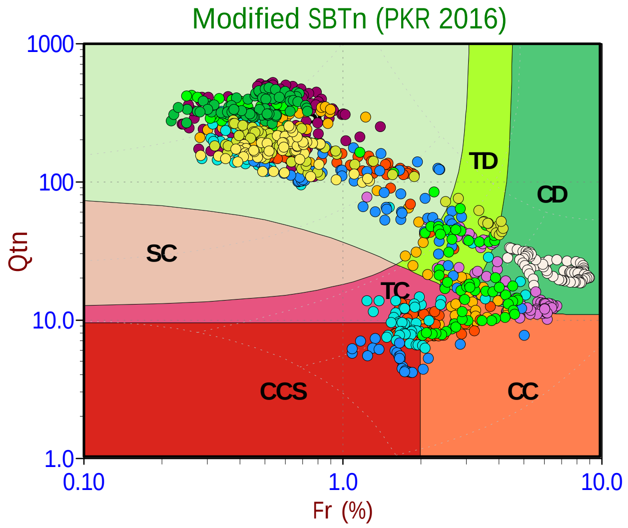
CONETEC

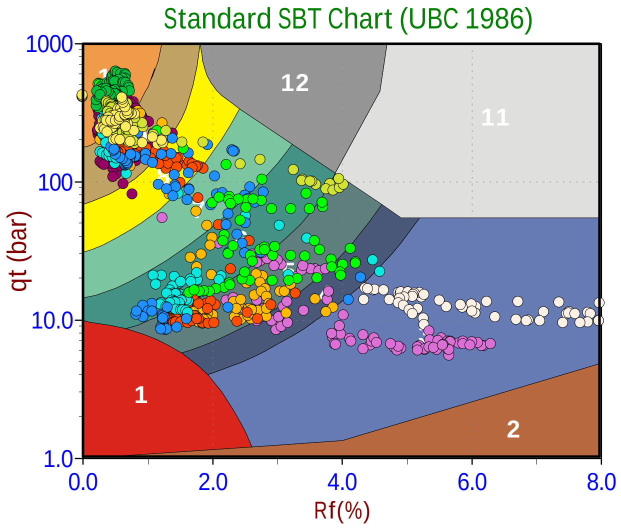
Langan Engineering

Job No: 19-56026
Date: 2019-03-05 08:38
Site: North Town

Sounding: CPT-05
Cone: 483:T1500F15U500


Qtn Chart (PKR 2009)


Modified SBTn (PKR 2016)


Standard SBT Chart (UBC 1986)

Depth Ranges

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

Legend

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

Legend

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

Legend

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

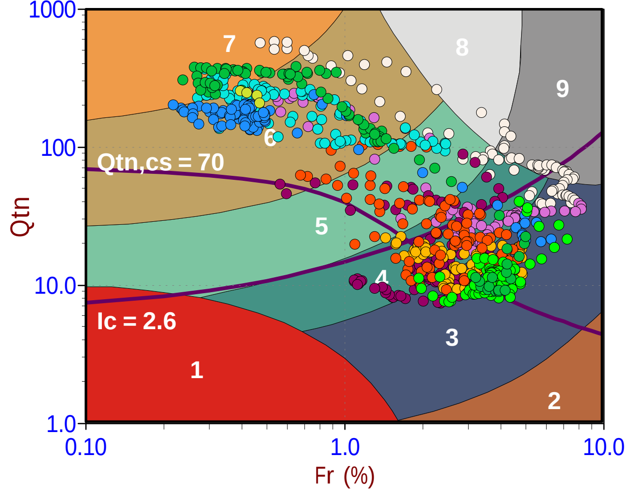
CONETEC

Langan Engineering

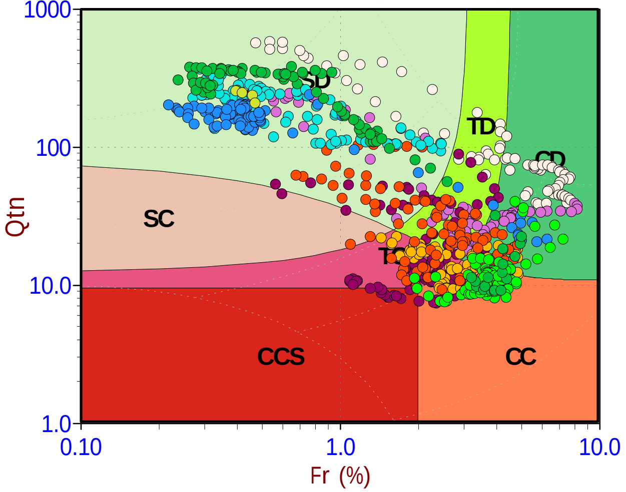
Job No: 19-56026
Date: 2019-03-04 10:12
Site: North Town

Sounding: CPT-06
Cone: 483:T1500F15U500

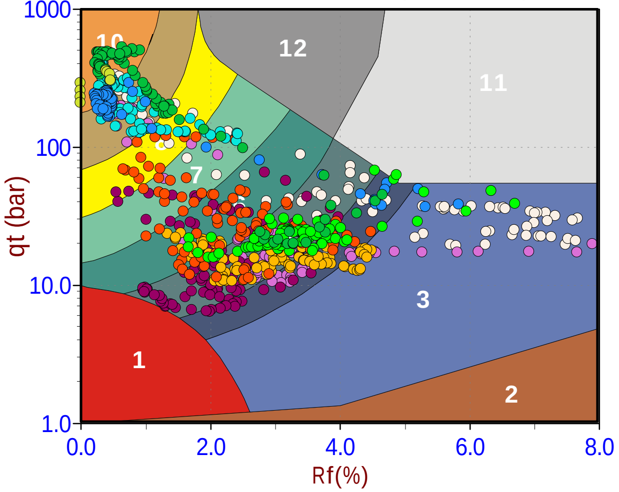
Qtn Chart (PKR 2009)



Modified SBTn (PKR 2016)



Standard SBT Chart (UBC 1986)



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

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

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

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

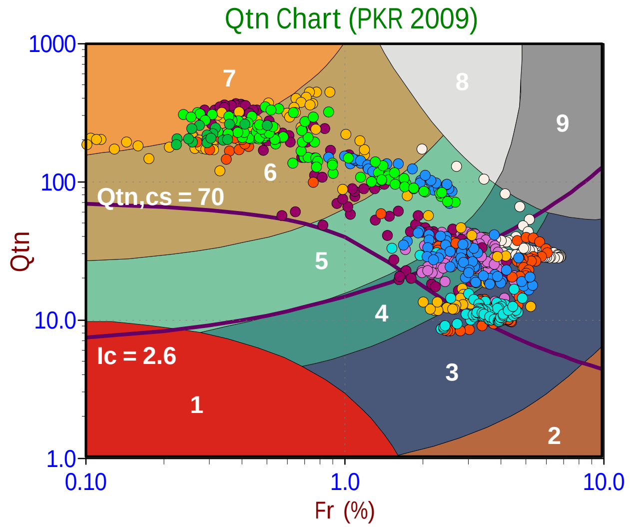
CONETEC

Langan Engineering

Job No: 19-56026
Date: 2019-03-04 11:10
Site: North Town

Sounding: CPT-07
Cone: 483:T1500F15U500

Qtn Chart (PKR 2009)



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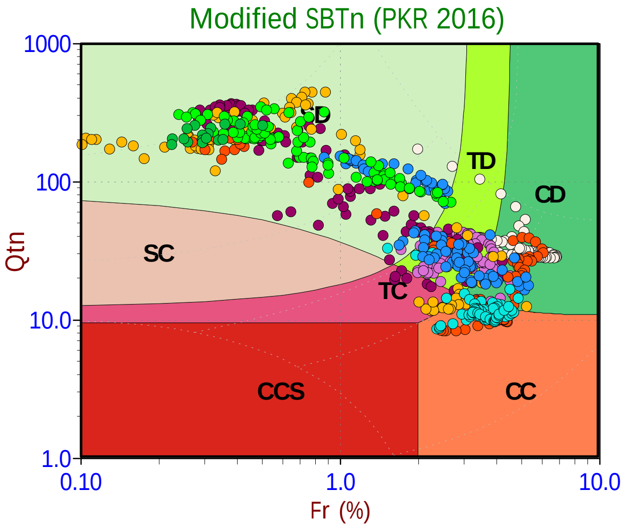
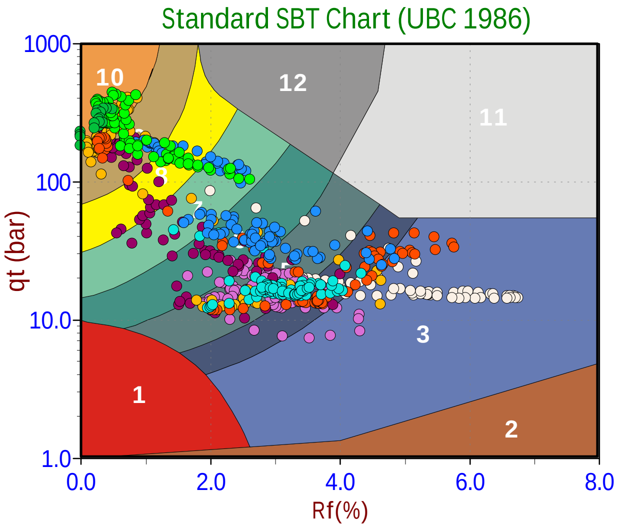


Standard SBT Chart (UBC 1986)



Depth Ranges

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

Legend

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

Legend

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

Legend

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

Pore Pressure Dissipation Summary and
Pore Pressure Dissipation Plots

Job No: 19-56026
Client: Lagan Engineering
Project: North Town
Start Date: 04-Mar-2019
End Date: 05-Mar-2019

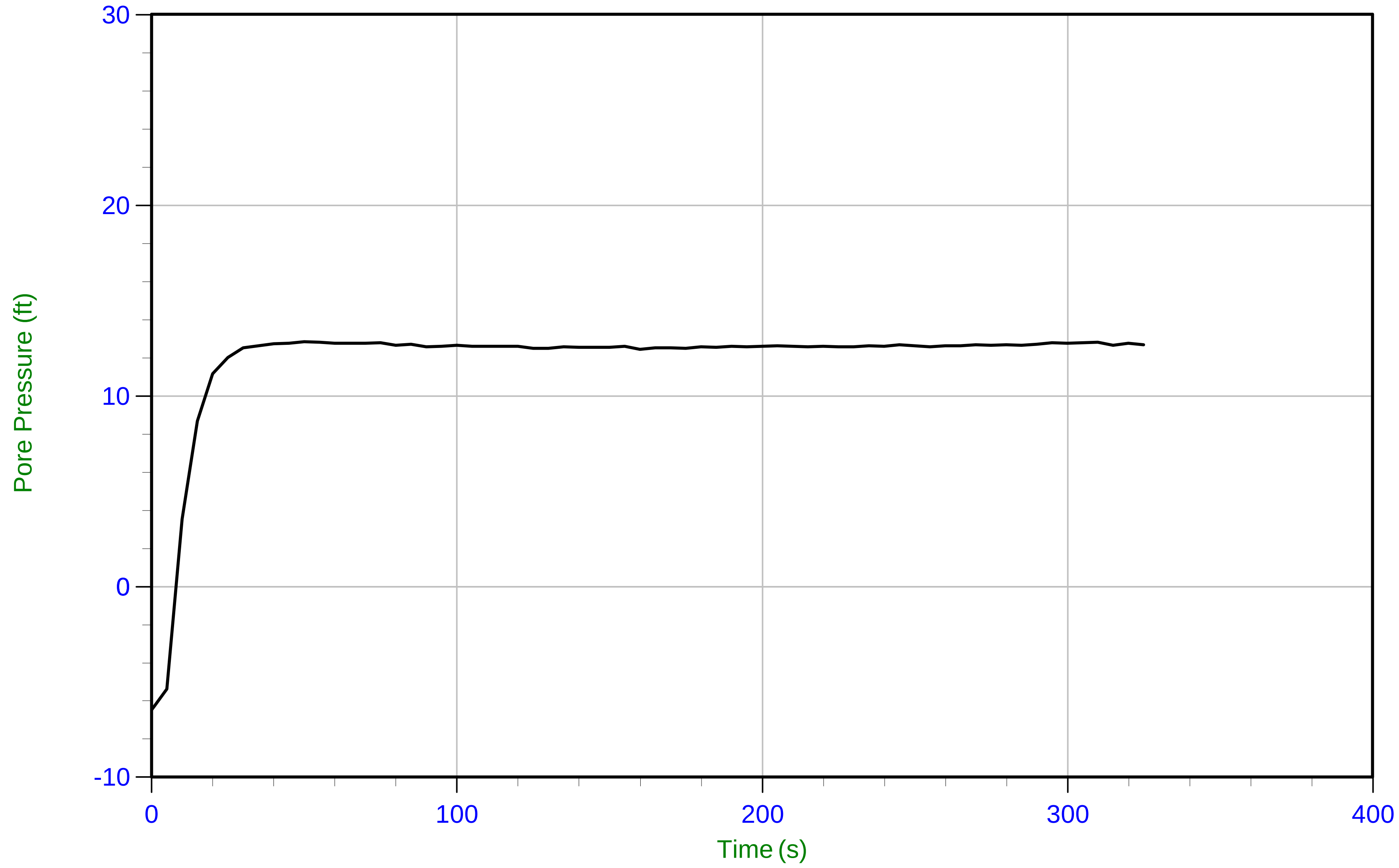
CPTu PORE PRESSURE DISSIPATION SUMMARY

Sounding ID	File Name	Cone Area (cm^2)	Duration (s)	Test Depth (ft)	Estimated Equilibrium Pore Pressure U_{eq} (ft)	Calculated Phreatic Surface (ft)
CPT-01	19-56026_CP01	15	325	20.75	12.2	8.5
CPT-02	19-56026_CP02	15	635	65.78	58.7	7.1
CPT-03	19-56026_CP03	15	245	38.06	33.0	5.1
CPT-04	19-56026_CP04	15	505	24.61	16.8	7.8
CPT-04	19-56026_CP04	15	505	63.89	59.2	4.7
CPT-05	19-56026_CP05	15	535	25.10	17.4	7.7
CPT-06	19-56026_CP06	15	240	38.63	29.8	8.8
CPT-06	19-56026_CP06	15	215	68.08	60.9	7.2
CPT-07	19-56026_CP07	15	300	54.46	48.2	6.3

Langan Engineering

Job No: 19-56026
Date: 03/04/2019 07:46
Site: North Town

Sounding: CPT-01
Cone: 483:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 19-56026_CP01.PPF
Depth: 6.325 m / 20.751 ft
Duration: 325.0 s

u Min: -6.5 ft
u Max: 12.8 ft
u Final: 12.7 ft

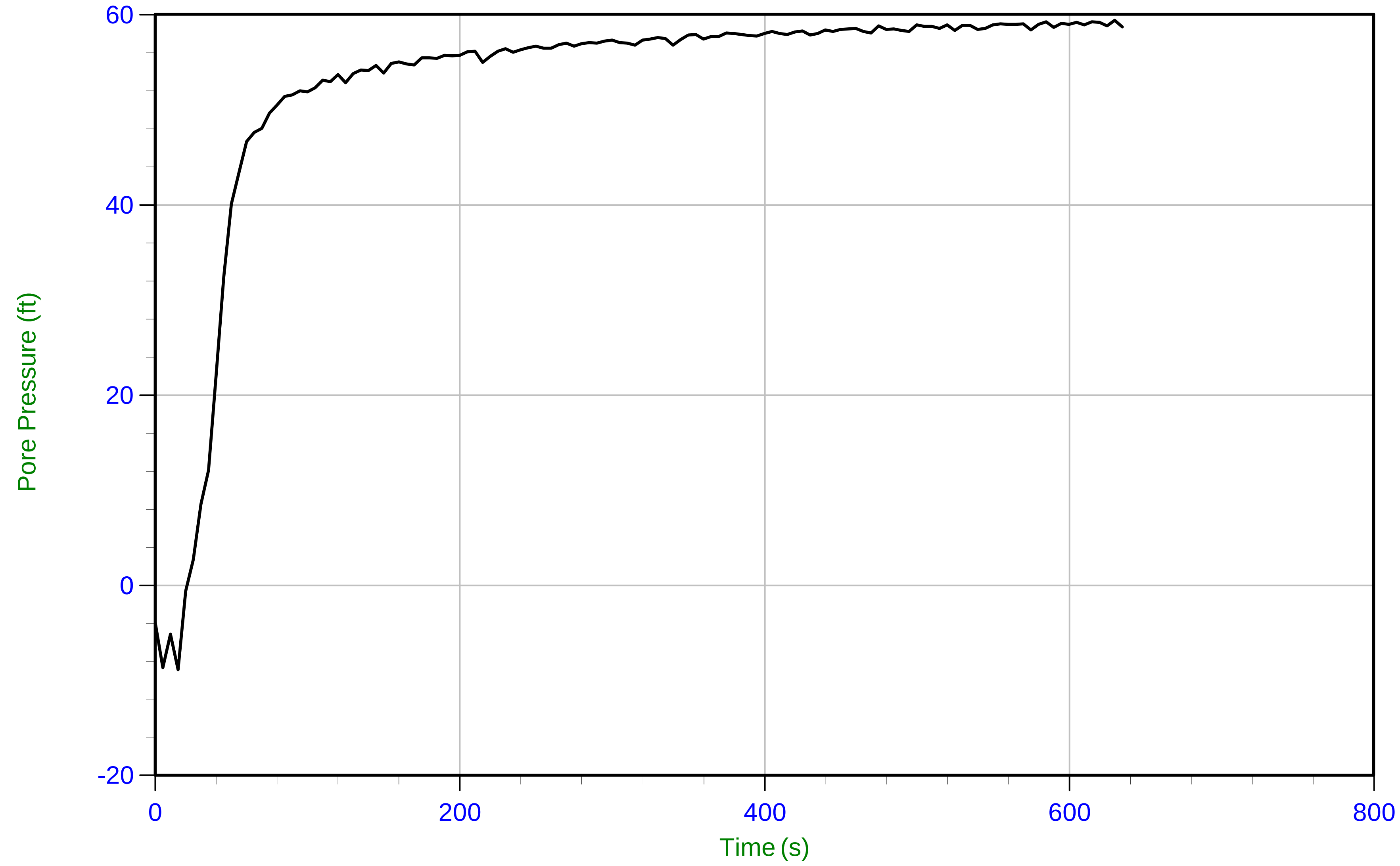
WT: 2.600 m / 8.530 ft
Ueq: 12.2 ft

CONETEC

Langan Engineering

Job No: 19-56026
Date: 03/04/2019 09:00
Site: North Town

Sounding: CPT-02
Cone: 483:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 19-56026_CP02.PPF
Depth: 20.050 m / 65.780 ft
Duration: 635.0 s

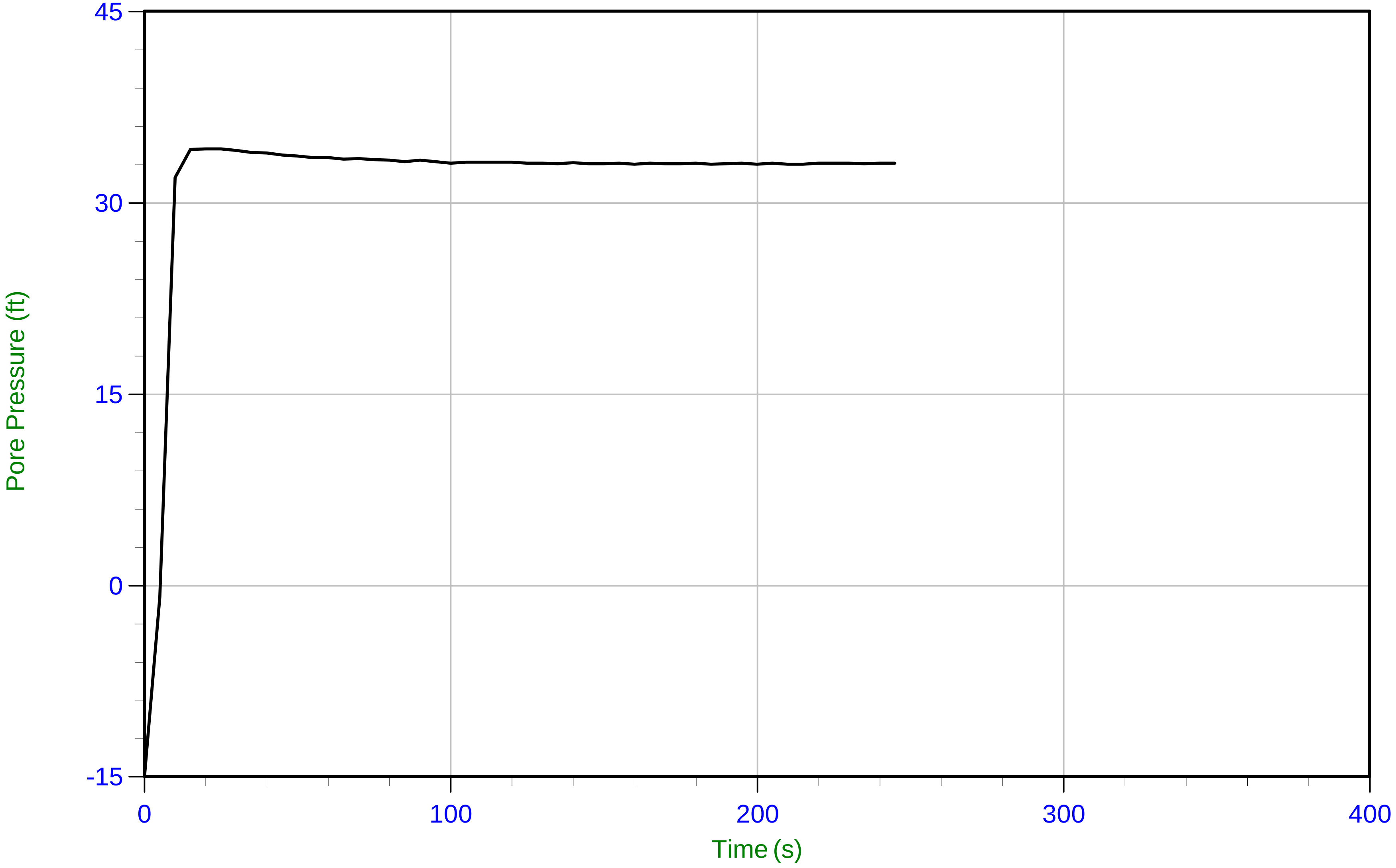
u Min: -8.9 ft
u Max: 59.4 ft
u Final: 58.7 ft

WT: 2.162 m / 7.093 ft
Ueq: 58.7 ft

Langan Engineering

Job No: 19-56026
Date: 03/05/2019 07:37
Site: North Town

Sounding: CPT-03
Cone: 483:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 19-56026_CP03.PPF
Depth: 11.600 m / 38.057 ft
Duration: 245.0 s

u Min: -15.0 ft
u Max: 34.2 ft
u Final: 33.1 ft

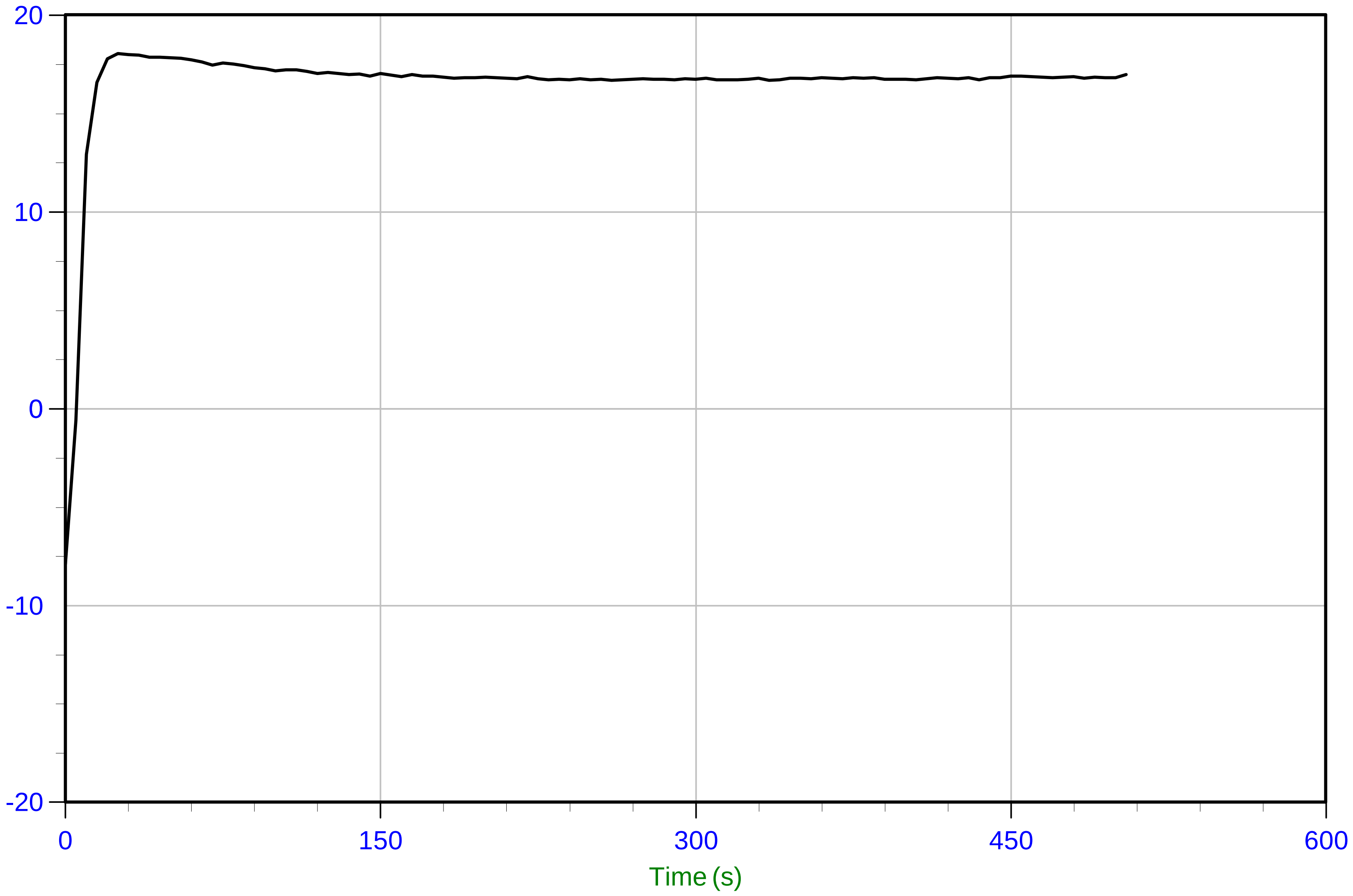
WT: 1.542 m / 5.059 ft
Ueq: 33.0 ft

CONETEC

Langan Engineering

Job No: 19-56026
Date: 03/05/2019 09:39
Site: North Town

Sounding: CPT-04
Cone: 483:T1500F15U500 Area=15 cm²



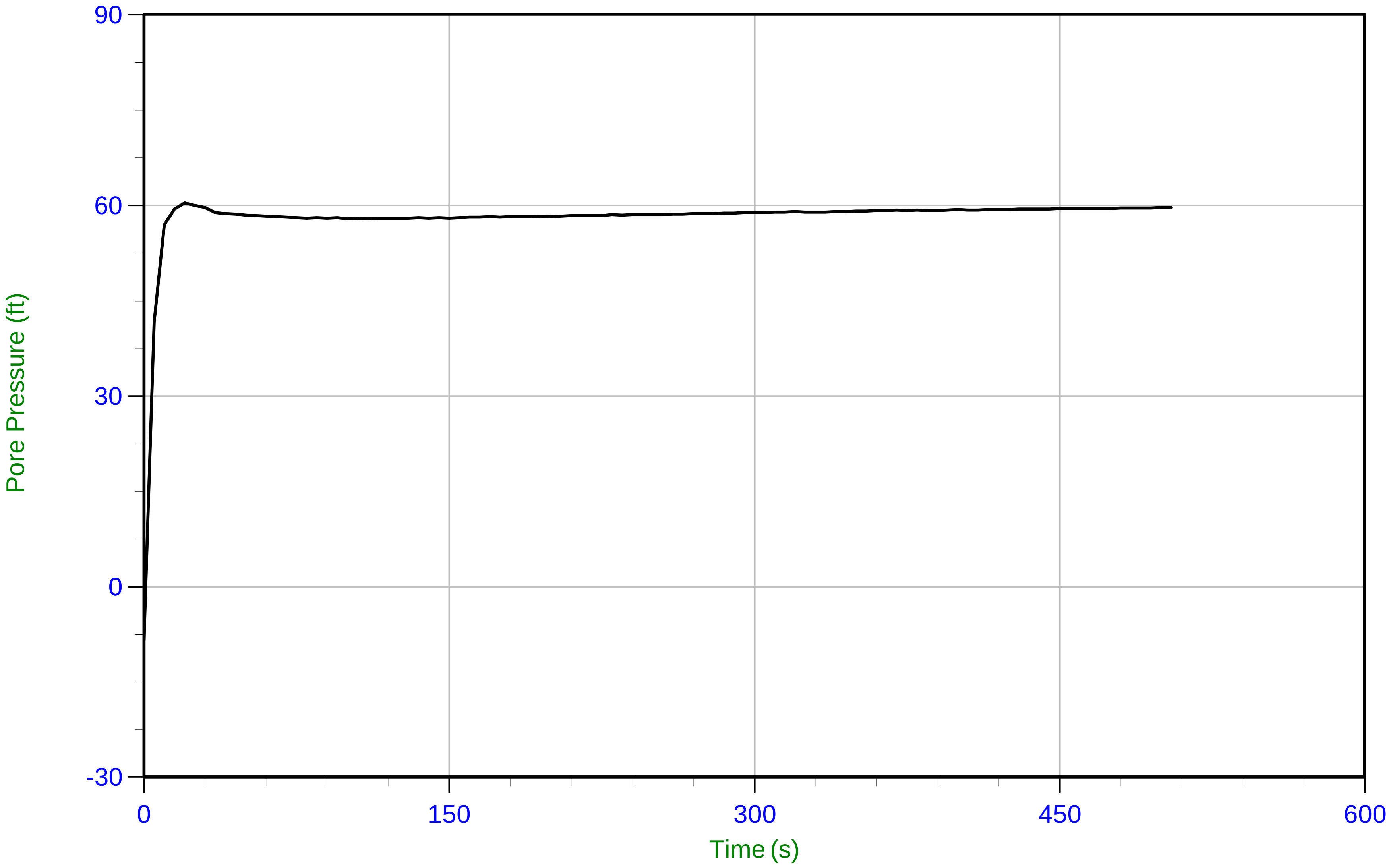
Trace Summary:

Filename: 19-56026_CP04.PPF
Depth: 7.500 m / 24.606 ft
Duration: 505.0 s

u Min: -7.9 ft
u Max: 18.0 ft
u Final: 17.0 ft

WT: 2.389 m / 7.838 ft
Ueq: 16.8 ft

Langan Engineering	Job No: 19-56026 Date: 03/05/2019 09:39 Site: North Town	Sounding: CPT-04 Cone: 483:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 19-56026_CP04.PPF
Depth: 19.475 m / 63.894 ft
Duration: 505.0 s

u Min: -8.5 ft
u Max: 60.4 ft
u Final: 59.7 ft

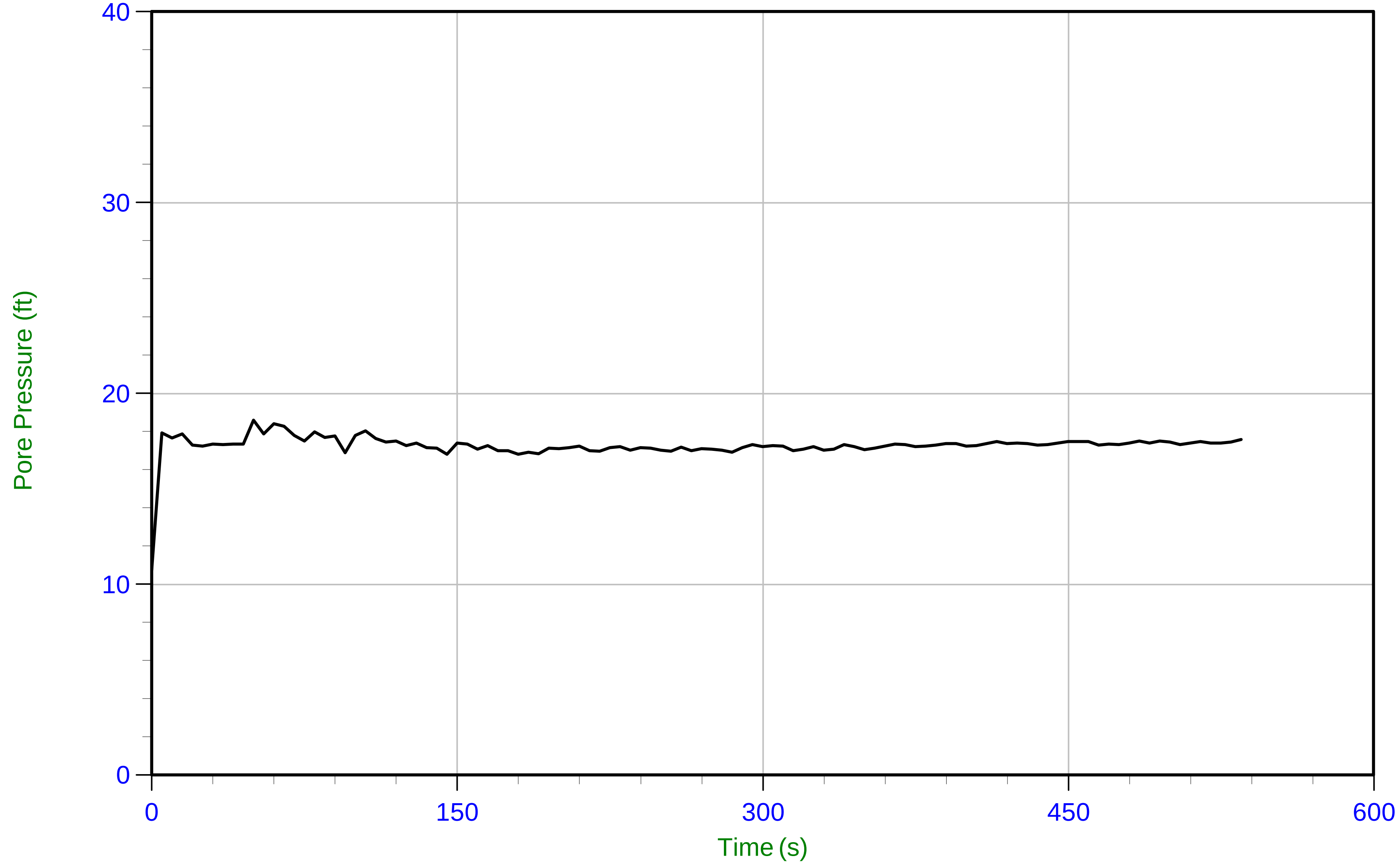
WT: 1.427 m / 4.682 ft
Ueq: 59.2 ft

CONETEC

Langan Engineering

Job No: 19-56026
Date: 03/05/2019 08:38
Site: North Town

Sounding: CPT-05
Cone: 483:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 19-56026_CP05.PPF
Depth: 7.650 m / 25.098 ft
Duration: 535.0 s

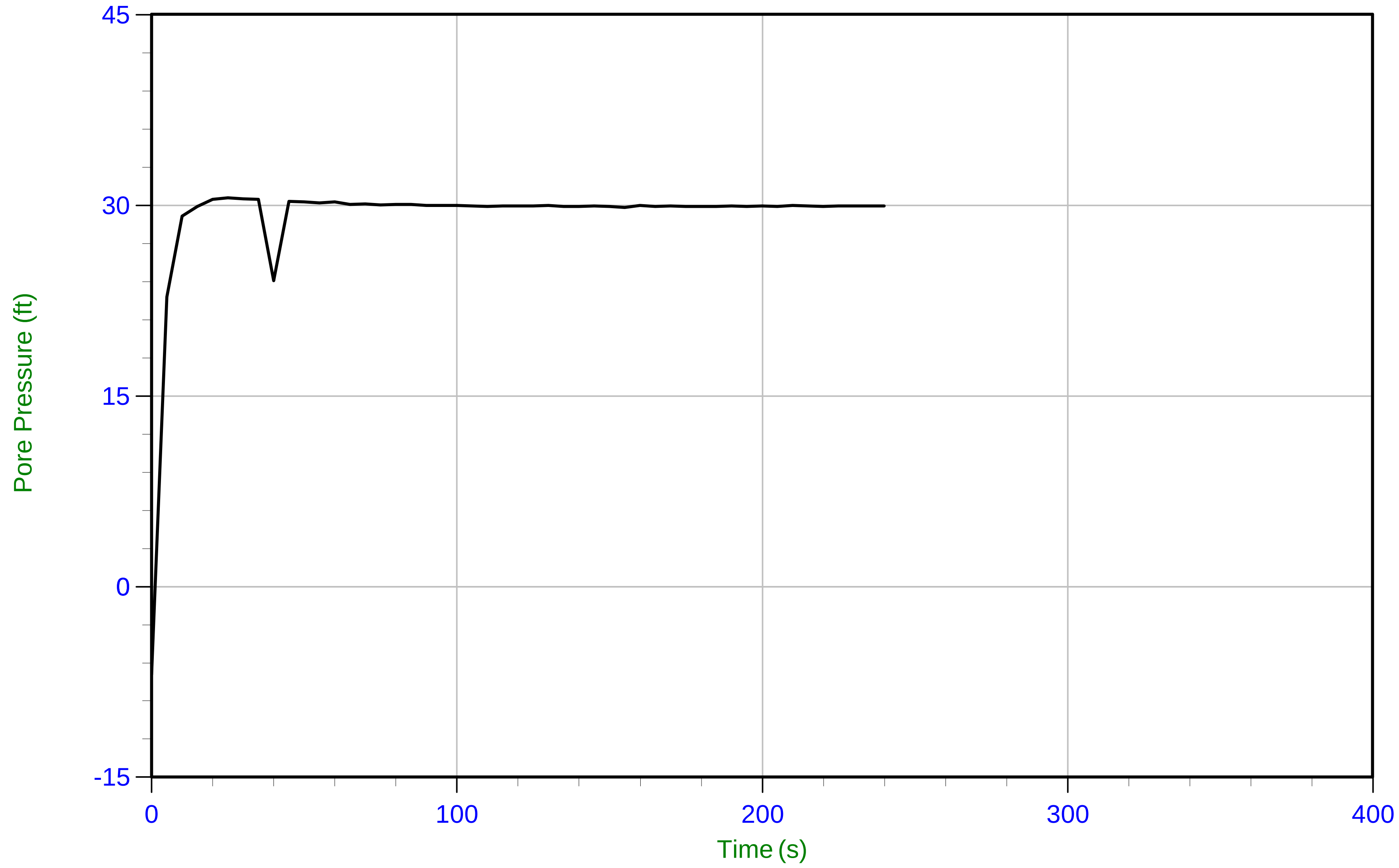
u Min: 10.7 ft
u Max: 18.6 ft
u Final: 17.6 ft

WT: 2.354 m / 7.723 ft
Ueq: 17.4 ft

Langan Engineering

Job No: 19-56026
Date: 03/04/2019 10:12
Site: North Town

Sounding: CPT-06
Cone: 483:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 19-56026_CP06.PPF
Depth: 11.775 m / 38.631 ft
Duration: 240.0 s

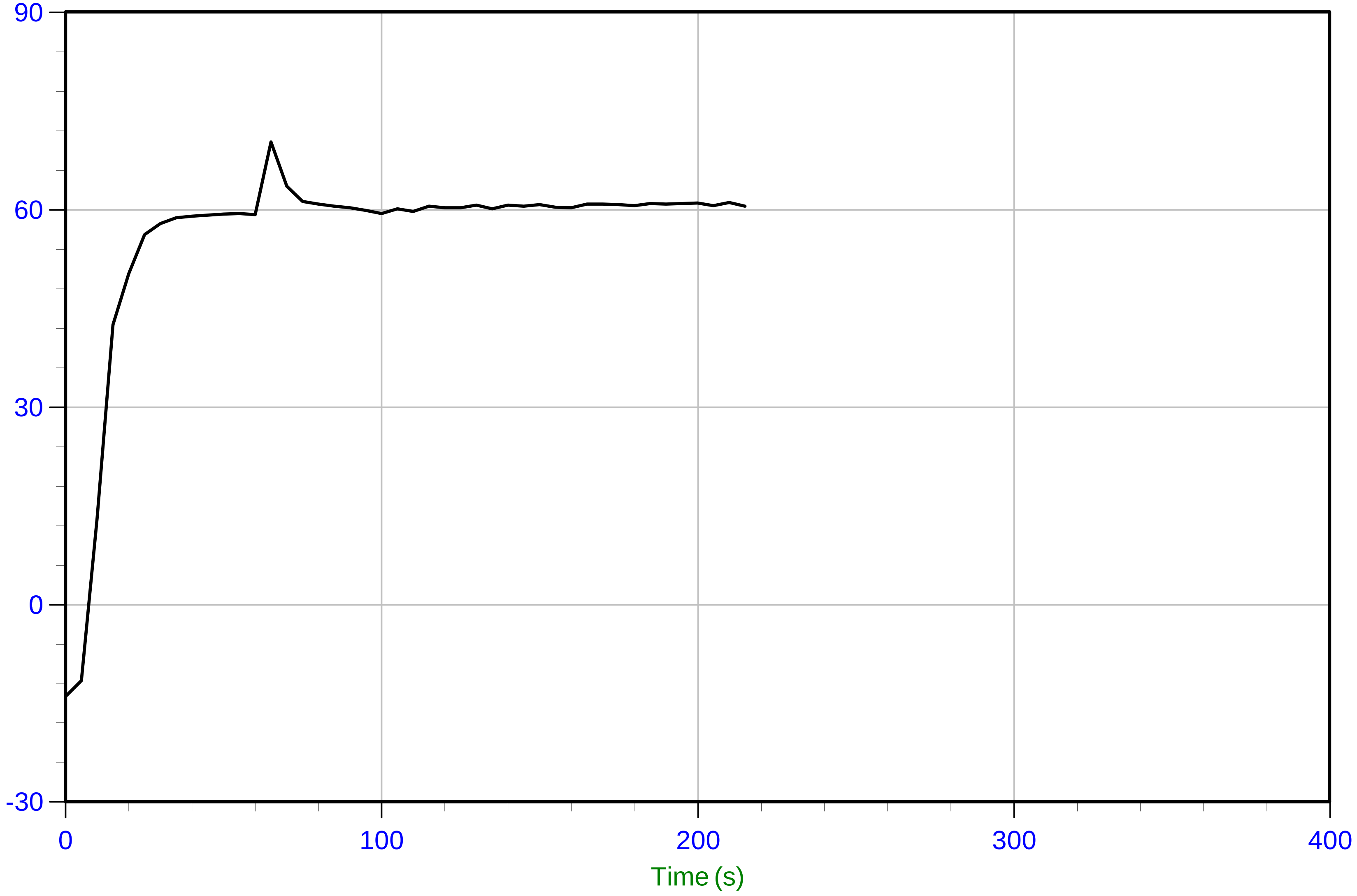
u Min: -6.9 ft
u Max: 30.6 ft
u Final: 29.9 ft

WT: 2.677 m / 8.783 ft
Ueq: 29.8 ft

Langan Engineering

Job No: 19-56026
Date: 03/04/2019 10:12
Site: North Town

Sounding: CPT-06
Cone: 483:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 19-56026_CP06.PPF
Depth: 20.750 m / 68.077 ft
Duration: 215.0 s

u Min: -14.0 ft
u Max: 70.3 ft
u Final: 60.5 ft

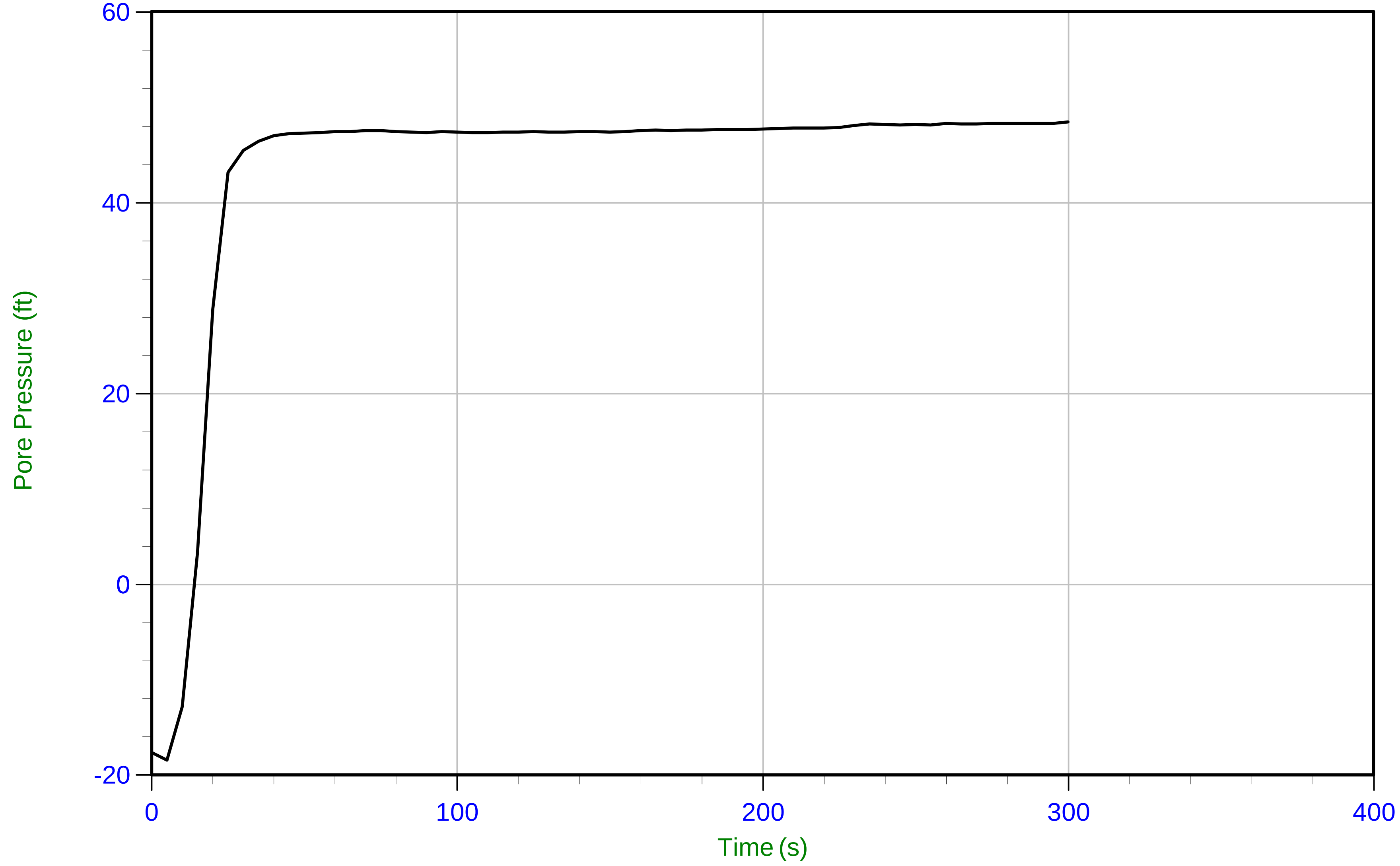
WT: 2.185 m / 7.169 ft
Ueq: 60.9 ft

CONETEC

Langan Engineering

Job No: 19-56026
Date: 03/04/2019 11:10
Site: North Town

Sounding: CPT-07
Cone: 483:T1500F15U500 Area=15 cm²



Trace Summary:

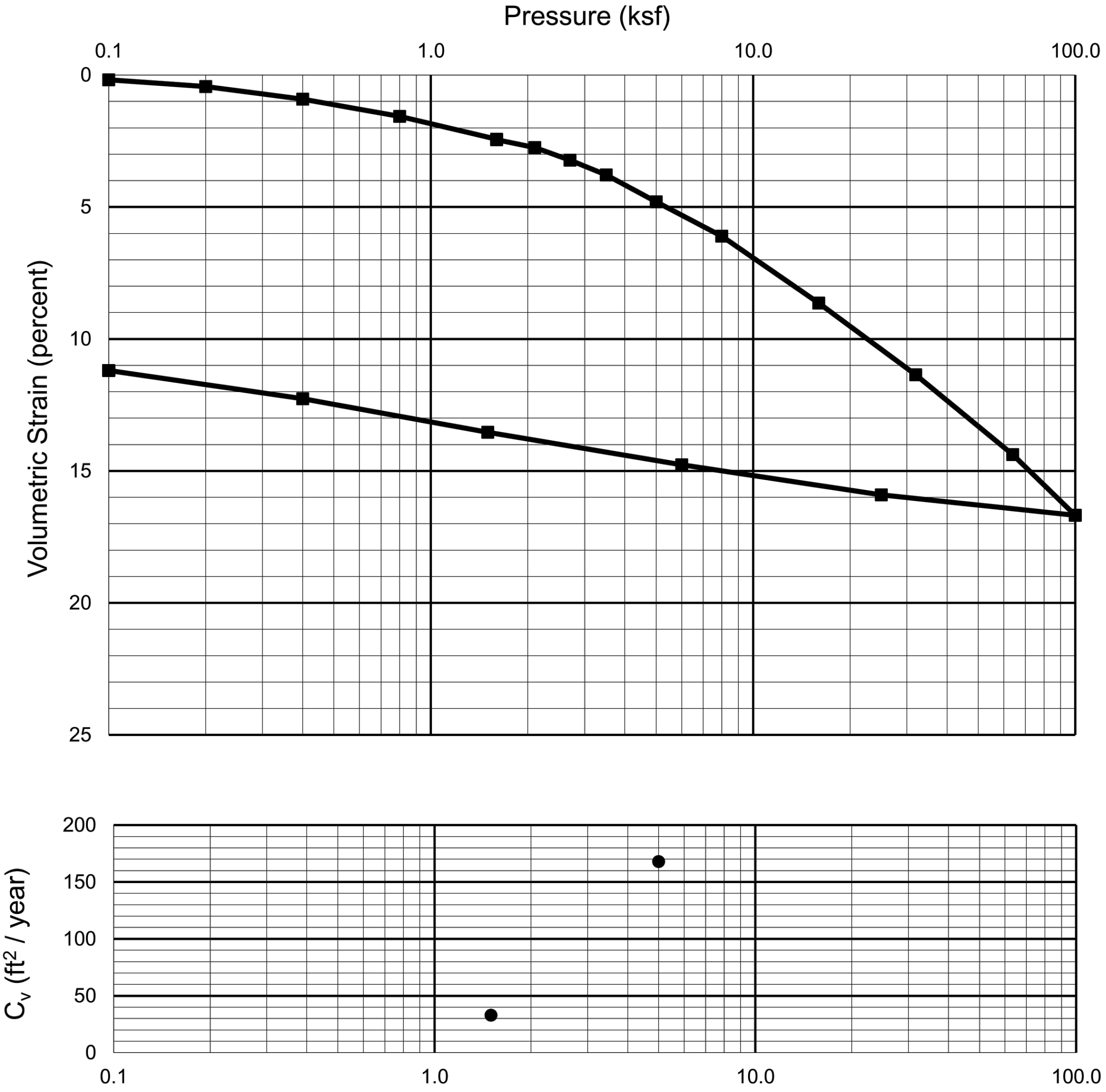
Filename: 19-56026_CP07.PPF
Depth: 16.600 m / 54.461 ft
Duration: 300.0 s

u Min: -18.4 ft
u Max: 48.5 ft
u Final: 48.5 ft

WT: 1.908 m / 6.260 ft
Ueq: 48.2 ft

APPENDIX C

LABORATORY TEST RESULTS



Sampler Type: Sprague & Henwood				Condition		Before Test		After Test	
Diameter (in) 2.42	Height (in) 1.00			Water Content	w_o	23.4 %	w_f	17.4 %	
Overburden Pressure, p_o	3,900	psf		Void Ratio	e_o	0.66	e_f	0.47	
Preconsol. Pressure, p_c	8,100	psf		Saturation	S_o	97 %	S_f	100 %	
Compression Ratio, $C_{\varepsilon c}$	0.11			Dry Density	γ_d	102 pcf	γ_d	115 pcf	
LL - -	PL - -		PI - -				G_s	2.70	(assumed)
Classification CLAY with SAND (CL), yellow-brown				Source		B-2 at 56 feet			

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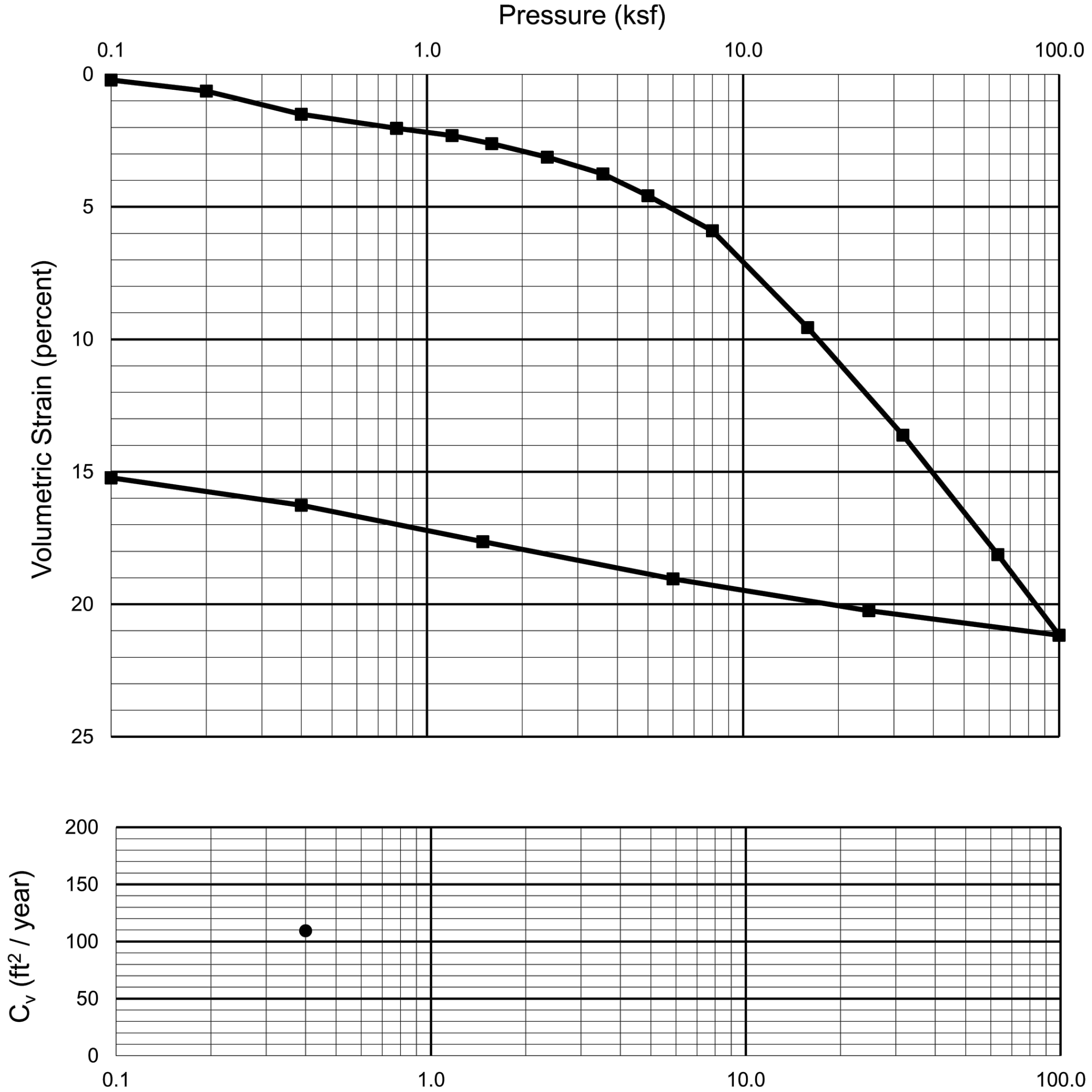
SANTA CLARA COUNTY CALIFORNIA

Figure Title
CONSOLIDATION TEST REPORT

Project No. 770651906
Date 01/03/2023
Drawn By JDF
Checked By TL

Figure
C-1



Sampler Type: Shelby Tube		Condition		Before Test		After Test
Diameter (in) 2.42	Height (in) 1.00	Water Content	w_o	29.6 %	w_f	20.4 %
Overburden Pressure, p_o	2,350 psf	Void Ratio	e_o	0.83	e_f	0.55
Preconsol. Pressure, p_c	9,000 psf	Saturation	S_o	96 %	S_f	100 %
Compression Ratio, $C_{\varepsilon c}$	0.15	Dry Density	γ_d	92 pcf	γ_d	109 pcf
LL - -	PL - -	PI - -		G_s	2.70	(assumed)
Classification CLAY (CL), gray		Source		B-3 at 30 feet		

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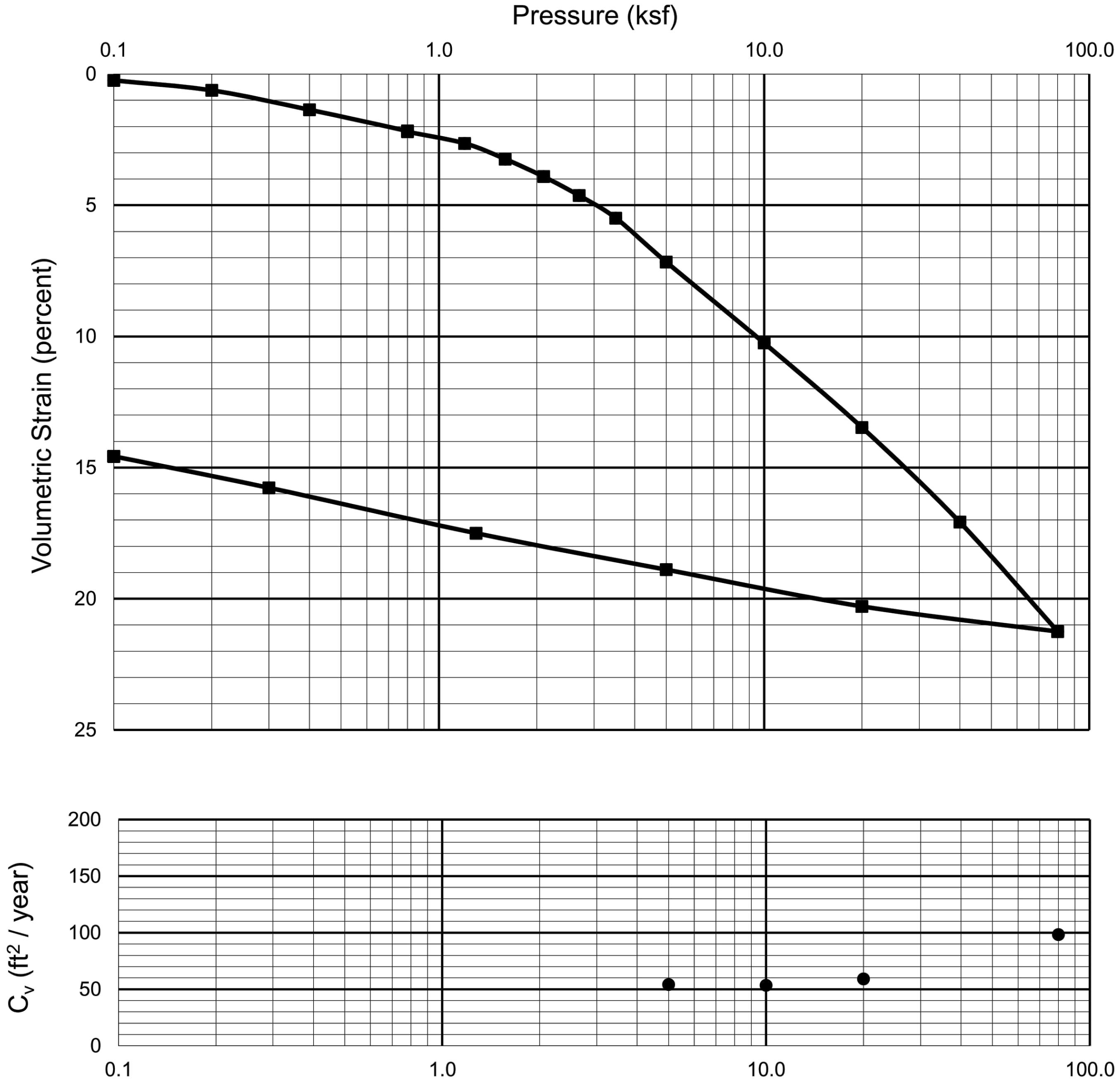
Figure
C-2

Filename: \\langan.com\data\SJO\data9\770651903\Project Data\CAD\03\2D-DesignFiles\770651906\770651906-B-GI0101.dwg Date: 1/3/2023 Time: 15:32 User: agekas Style Table: Langan.stb Layout: Fig C-2 Consol B-3 @ 30ft



Sampler Type: Sprague & Henwood			Condition		Before Test		After Test
Diameter (in) 2.42	Height (in) 1.00		Water Content	w_o	26.7 %	w_f	18.9 %
Overburden Pressure, p_o	1,140	psf	Void Ratio	e_o	0.77	e_f	0.51
Preconsol. Pressure, p_c	3,300	psf	Saturation	S_o	94 %	S_f	100 %
Compression Ratio, $C_{\varepsilon c}$	0.13		Dry Density	γ_d	95 pcf	γ_d	112 pcf
LL - -	PL - -		PI - -		G_s	2.70	(assumed)
Classification CLAY (CL), yellow-brown with orange			Source B-4 at 9.5 feet				

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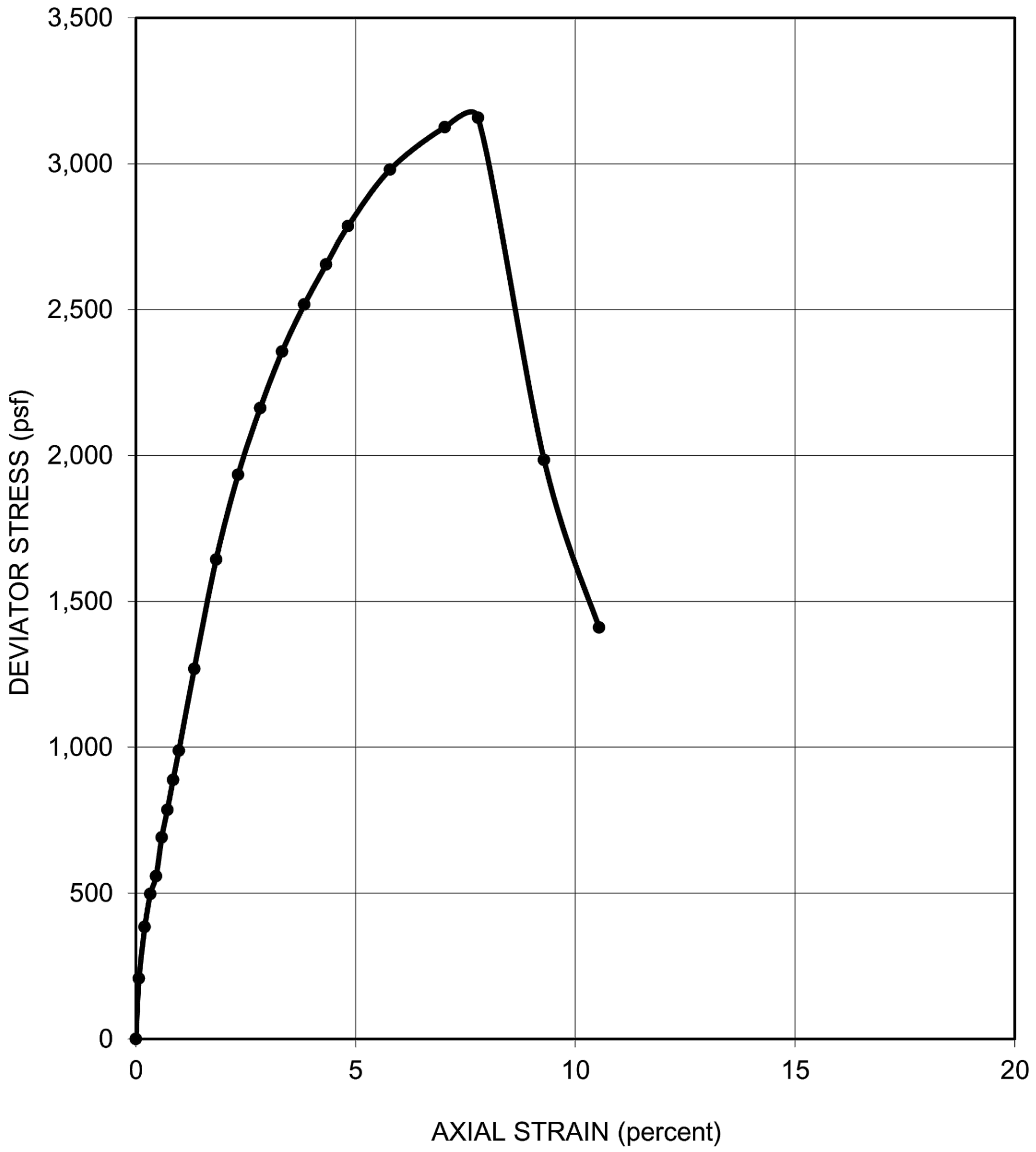
Figure
C-3



SAMPLER TYPE	Shelby Tube			SHEAR STRENGTH	1,580	psf
DIAMETER (in.)	2.86	HEIGHT (in.) 6.1		STRAIN AT FAILURE	7.8	%
MOISTURE CONTENT		27.4	%	CONFINING PRESSURE	2,400	psf
DRY DENSITY		98	pcf	STRAIN RATE	0.50	% / min
DESCRIPTION	CLAY (CL), gray			SOURCE	B-3 at 30 feet	

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Figure Title
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Project No. 770651906
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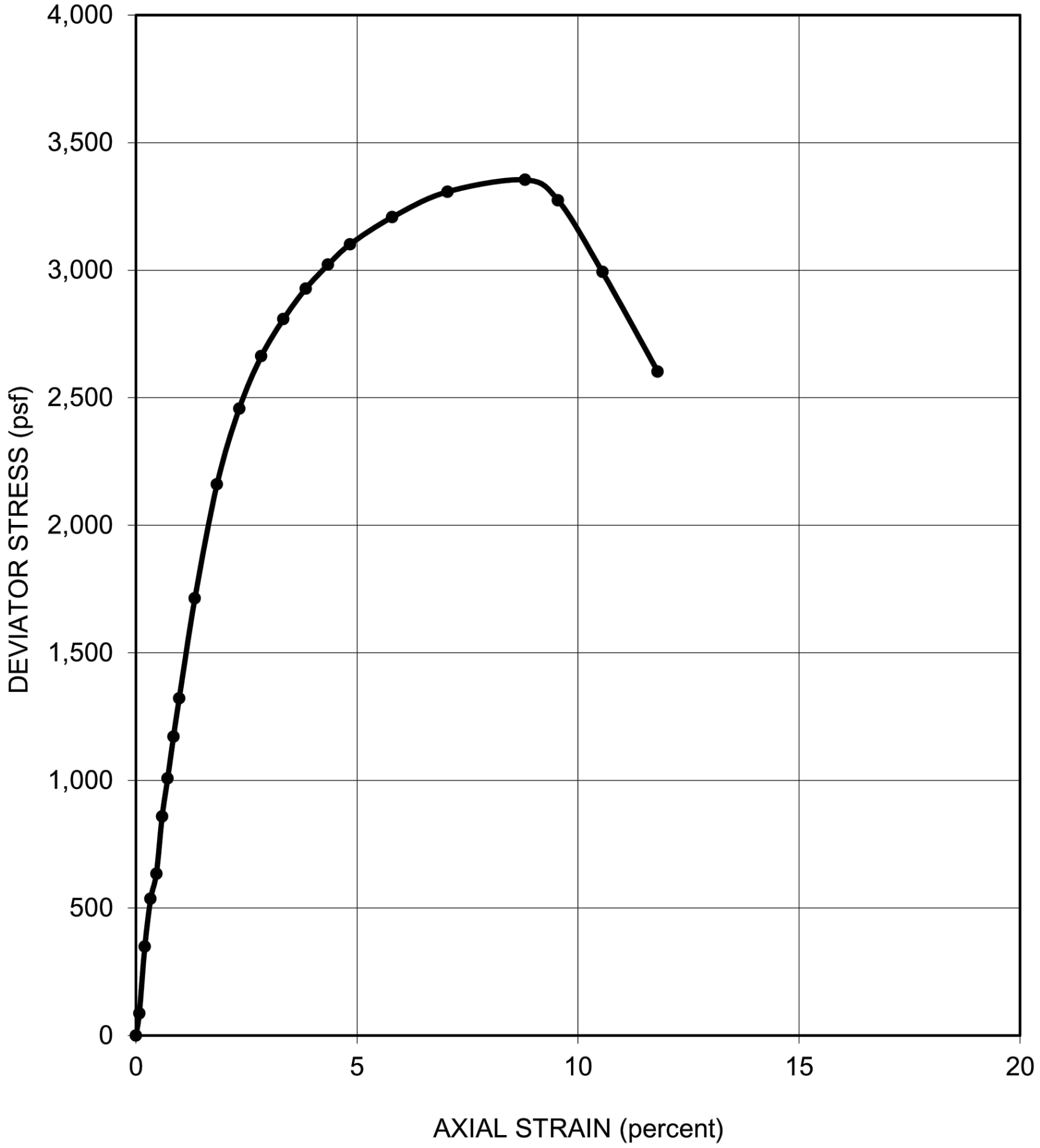
Figure
C-4



SAMPLER TYPE	Shelby Tube			SHEAR STRENGTH	1,680	psf
DIAMETER (in.)	2.86	HEIGHT (in.) 6.1		STRAIN AT FAILURE	8.8	%
MOISTURE CONTENT		23.5	%	CONFINING PRESSURE	3,200	psf
DRY DENSITY		104	pcf	STRAIN RATE	0.50	% / min
DESCRIPTION	CLAY (CL), yellow-brown			SOURCE	B-3 at 45 feet	

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Figure Title
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Project No. 770651906
Date 01/03/2023
Drawn By JDF
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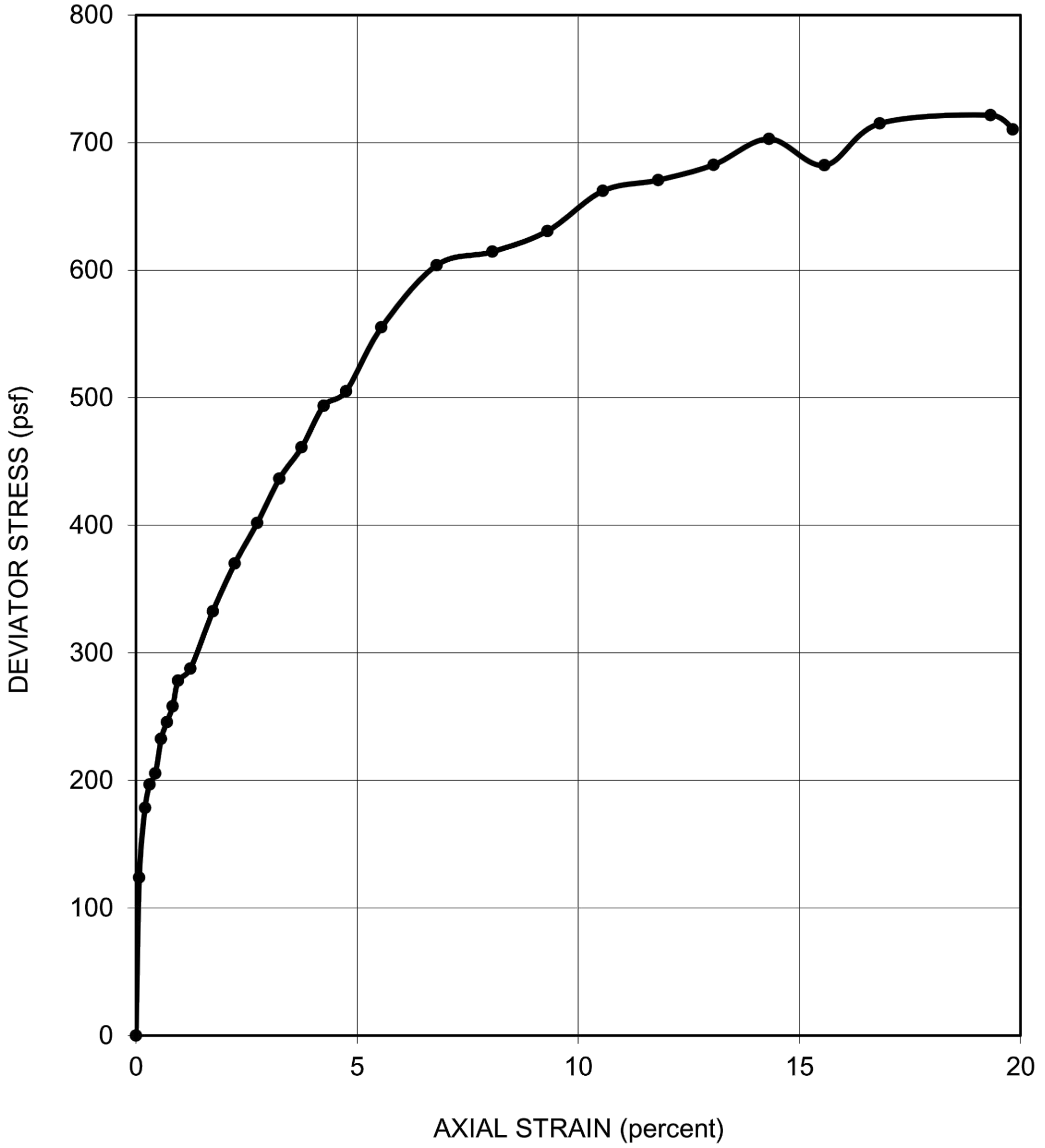
Figure
C-5



SAMPLER TYPE	Sprague & Henwood			SHEAR STRENGTH	360	psf
DIAMETER (in.)	2.40	HEIGHT (in.)	5.81	STRAIN AT FAILURE	19.3	%
MOISTURE CONTENT		19.3	%	CONFINING PRESSURE	1,100	psf
DRY DENSITY		108	pcf	STRAIN RATE	0.50	% / min
DESCRIPTION	SANDY CLAY (CL), yellow-brown				SOURCE	B-6 at 9.5 feet

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Figure Title

UNCONSOLIDATED-UNDRAINED
TRIAXIAL COMPRESSION TEST

Project No.
770651906

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JDF

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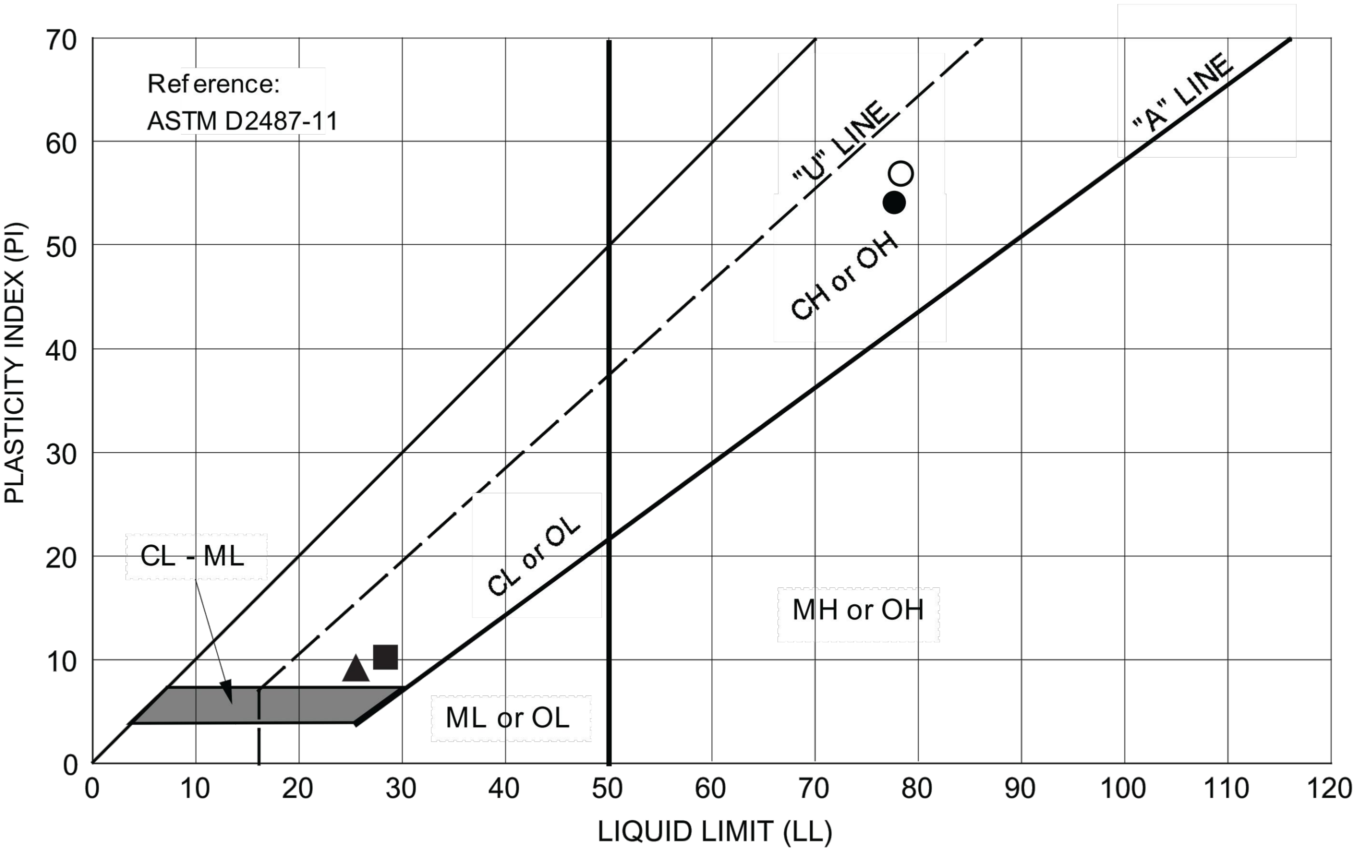
TL

Figure

C-6



Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	B-1 at 1 - 5 feet	CLAY (CH), dark brown	--	77	54	--
■	B-3 at 15 feet	CLAYEY SAND (SC), olive-gray	14.1	28	10	21.3
▲	B-4 at 14.5 feet	CLAYEY SAND (SC), gray-brown with orange	21.5	26	8	40.5
○	B-5 at 1 - 4 feet	CLAY (CH), dark brown	--	78	57	--

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Figure Title
PLASTICITY CHART

Project No. 770651906
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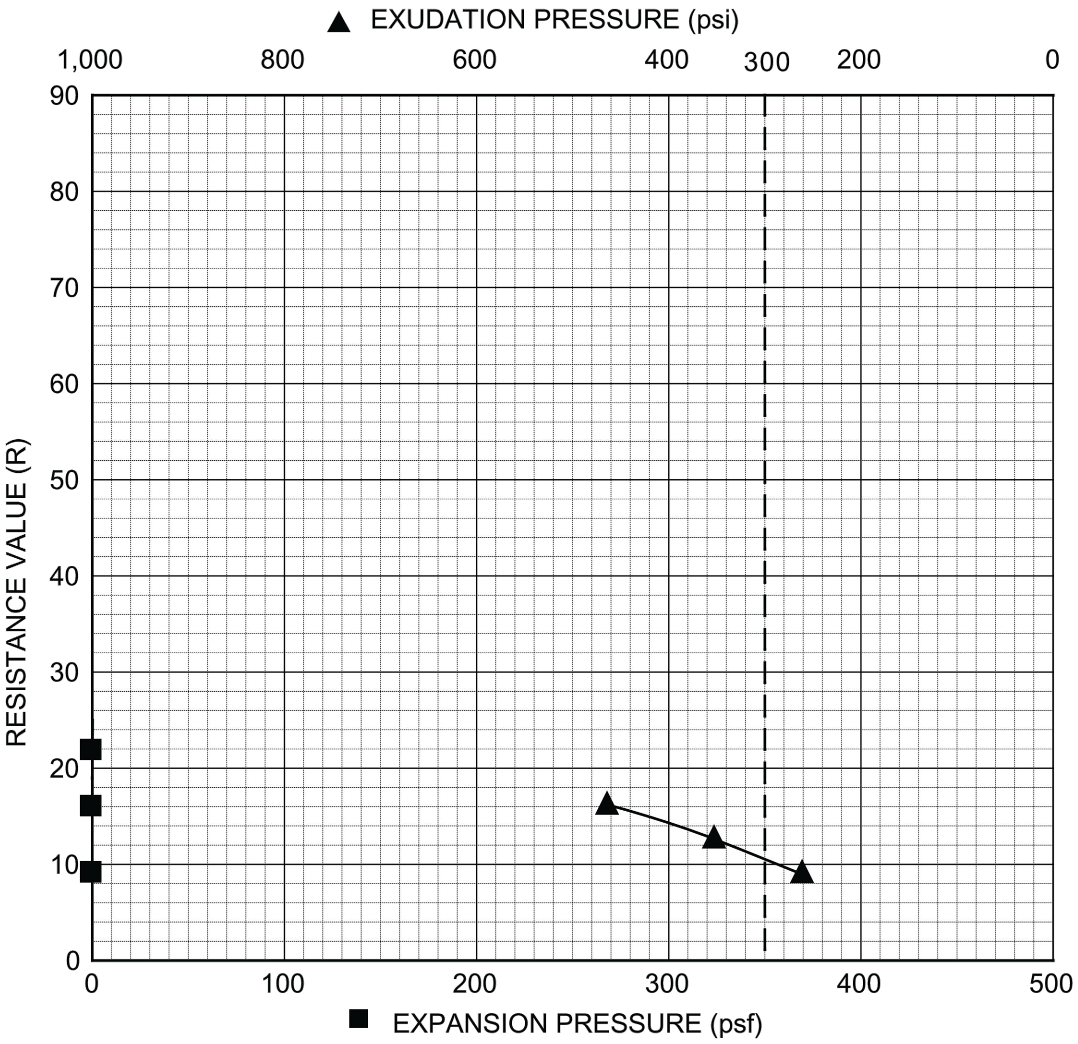
Figure
C-7



Specimen ID:	A	B	C	D
Water Content (%)	27.0	25.2	23.0	--
Dry Density (pcf)	93.1	95.6	102.7	--
Exudation Pressure (psi)	260	356	464	--
Expansion Pressure (psf)	0.00	0.00	0.00	--
Resistance Value (R)	9	13	17	--

Sample Source	Sample Description	Sand Equivalent	Expansion Pressure	R value
B-5/B-6 at 1 - 4 feet	CLAY (CH), dark brown	--	--	12

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Figure Title
RESISTANCE VALUE TEST DATA

Project No. 770651906
Date 01/03/2023
Drawn By JDF
Checked By TL

Figure **C-8**

Filename: \\langan.com\data\SJO\data9\770651903\Project Data\CAD\03\2D-DesignFiles\770651906\770651906-B-GI0101.dwg Date: 1/3/2023 Time: 15:39 User: agekas Style Table: Langan.stb Layout: Fig C-8 R Value Test Data

APPENDIX D

CORROSIVITY ANALYSIS – ASTM TEST METHODS

California State Certified Laboratory No. 2153

CERCO
analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

20 March, 2019

Job No. 1903076
Cust. No. 12242

Mr. John Gouchon
Langan
1 Almaden Blvd., Suite 590
San Jose, CA 95113

Subject: Project No.: 770651903.700.022
Project Name: North Town
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Gouchon:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on March 12, 2019. Based on the analytical results, a brief evaluation is enclosed for your consideration.

Based upon the resistivity measurements, both samples are classified as "corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentrations reflect none detected with a reporting limit of 15 mg/kg.

The sulfate ion concentrations are 37 & 140 mg/kg and are determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations.

The pH of the soils are 8.37 & 8.45, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potentials are 220 & 280-mV. Both samples are indicative of potentially "slightly corrosive" soils resulting from anaerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call *JDH Corrosion Consultants, Inc. at (925) 927-6630.*

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

California State Certified Laboratory No. 2153

CERCO
analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 **462 2771** Fax. 925 **462 2775**
www.cercoanalytical.com

Client: Langan
Client's Project No.: 770651903.700.022
Client's Project Name: North Town
Date Sampled: 03/06-07/19
Date Received: 12-Mar-19
Matrix: Soil
Authorization: Chain of Custody

Date of Report: 20-Mar-2019

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
1903076-001	B-1 SA @ 1-4'	280	8.45	-	740	-	N.D.	140
1903076-002	B5, S1 @ 6'	220	8.37	-	1,200	-	N.D.	37

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
Date Analyzed:	19-Mar-2019	19-Mar-2019	-	19-Mar-2019 & 20-Mar-2019	-	19-Mar-2019	19-Mar-2019

Cheryl McMillen
Laboratory Director

* Results Reported on "As Received" Basis

N.D. - None Detected

Quality Control Summary - **All laboratory quality control parameters were found to be within established limits**

12601

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CHAIN OF CUSTODY RECORD

Page 1 of 1

CA1903076

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☐ 501 14th Street, Third Floor, Oakland, CA 94612
☐ 3320 Data Drive, Suite 350, Rancho Cordova, CA 95670-7982
☒ 1 Almaden Boulevard, Suite 590, San Jose, CA 95113

Site Name: North Town
Job Number: 770651903-700-082
Project Manager\Contact: John Gouchon JGouchon@langan.com
Samplers: Chad Lecas
Recorder (Signature Required): Chad

Turnaround Time
Standard

					Matrix				No. Containers & Preservative						Analysis Requested														
	Field Sample Identification No.	Date	Time	Lab Sample No.	Soil	Water	Air	Other	HCL	H_2SO_4	HNO_3	Ice			Corrosion Bv	Brief Fuel											Silica gel clean-up	Hold	Remarks
001	B-1 SA F4'	3/6/19		1	X										X	X													
002	B5 S1 6'	3/7/19		2	X										X	X													TUBE

Relinquished by: (Signature) Chad	Date: 3/9/19	Time	Received by: (Signature)	Date 3/12/19	Time 0930
Relinquished by: (Signature)	Date:	Time	Received by: (Signature)	Date	Time
Relinquished by: (Signature)	Date:	Time	Received by Lab: (Signature)	Date	Time

Sent to Laboratory (Name): Cerco Analytical
Laboratory Comments/Notes:

Method of Shipment ☐ Lab courier ☐ Fed Ex ☐ Airborne ☐ UPS
☐ Hand Carried ☒ Private Courier (Co. Name) GSO

White Copy - Original Yellow Copy - Laboratory Pink Copy - Field COC Number:

APPENDIX E

SITE-SPECIFIC RESPONSE SPECTRA

APPENDIX E

SITE-SPECIFIC RESPONSE SPECTRA

This appendix presents the details of our estimation of the level of ground shaking at the site during future earthquakes. To develop site-specific response spectra in accordance with 2019 California Building Code (CBC) criteria, and by reference ASCE 7-16, we performed probabilistic seismic hazard analysis (PSHA) and deterministic analysis to develop smooth, site-specific horizontal spectra for two levels of shaking, namely:

- Risk-Targeted Maximum Considered Earthquake (MCE_R), which corresponds to the lesser of the risk-targeted two percent probability of exceedance in 50 years (2,475-year return period) or 84th percentile of the controlling deterministic event both considering the maximum direction as described in ASCE 7-16, with appropriate lower limit checks.

- Design Earthquake (DE), which corresponds to 2/3 of the MCE_R.

E1.0 PROBABILISTIC SEISMIC HAZARD ANALYSIS

Because the location, recurrence interval, and magnitude of future earthquakes are uncertain, we performed a PSHA, which systematically accounts for these uncertainties. The results of a PSHA define a uniform hazard for a site in terms of a probability that a particular level of shaking will be exceeded during the given life of the structure.

To perform a PSHA, information regarding the seismicity, location, and geometry of each source, along with empirical relationships that describe the rate of attenuation of strong ground motion with increasing distance from the source, are needed. The assumptions necessary to perform the PSHA are that:

- the geology and seismic tectonic history of the region are sufficiently known, such that the rate of occurrence of earthquakes can be modeled by historic or geologic data

- the level of ground motion at a particular site can be expressed by an attenuation relationship that is primarily dependent upon earthquake magnitude and distance from the source of the earthquake

- the earthquake occurrence can be modeled as a Poisson process with a constant mean occurrence rate.

As part of the development of the site-specific spectra, we performed a PSHA to develop a site-specific response spectrum for 2 percent probability of exceedance in 50 years. The ground surface spectrum was developed using the OpenSHA Hazard Spectrum Application 1.5.2. The approach used in PSHA is based on the probabilistic seismic hazard model developed by Cornell (1968) and McGuire (1976). Our analysis modeled the faults in the Bay Area as linear sources, and earthquake activities were assigned to the faults based on historical and geologic data. The levels of shaking were estimated using ground motion prediction equations (attenuation relationships) that are primarily dependent upon the magnitude of the earthquake and the distance from the site to the fault, as well as the average shear wave velocity of the upper 30 meters, V_{S30}.

E1.1 Probabilistic Model

In probabilistic models, the occurrence of earthquake epicenters on a given fault is assumed to be uniformly distributed along the fault. This model considers ground motions arising from the portion of the fault rupture closest to the site rather than from the epicenter. Fault rupture lengths were modeled using fault rupture length-magnitude relationships given by Wells and Coppersmith (1994).

The probability of exceedance, $P_e(Z)$, at a given ground-motion, Z, at the site within a specified time period, T, is given as:

$$P_e(Z) = 1 - e^{-V(z)T}$$

where V(z) is the mean annual rate of exceedance of ground motion level Z. V(z) can be calculated using the total-probability theorem.

$$V(z) = \sum_i v_i \iint P[Z > z \mid m,r] f_{M_i}(m) f_{R_i|M_i}(r;m) dr\, dm$$

where:

v_i = the annual rate of earthquakes with magnitudes greater than a threshold M_{oi} in source i

P [Z > z | m,r] = probability that an earthquake of magnitude m at distance r produces ground motion amplitude Z higher than z

f_{Mi} (m) and $f_{Ri|Mi}$ (r;m) = probability density functions for magnitude and distance

Z represents peak ground acceleration, or spectral acceleration values for a given frequency of vibration. The peak accelerations are assumed to be log-normally distributed about the mean with a standard error that is dependent upon the magnitude and attenuation relationship used.

E1.2 Source Modeling and Characterization

The segmentation of faults, maximum magnitudes, and recurrence rates were modeled using the data presented in the Uniform California Earthquake Rupture Forecast Version 3 (UCERF3) as detailed in the United States Geological Survey Open File Report 2013-1165. These and other faults of the region are shown on Figure 3. Table E-1 presents the distance and direction from the site to the fault, mean moment magnitude, mean slip rate, and fault length for individual fault segments in UCERF3 source model. The mean moment magnitude presented in Table E-1 was computed assuming full rupture of the segment using Hanks and Bakun (2008) relationship.

TABLE E-1
Source Zone Parameters

Fault Name	Approx. Distance from Fault (km)	Direction from Site	Mean Moment Magnitude[1]	Mean Slip Rate (mm/yr)	Fault Length (km)
Silver Creek	1.5	East	6.7	0.1	48
Hayward (So)	9	Northeast	6.9	9.8	54
Total Hayward-Rodgers Creek Healdsburg	9	Northeast	7.6	7.3	213
Hayward (So) extension	12	East	6.1	4.3	23
Calaveras (Central)	13	East	6.7	10.2	52
Mission (connected)	13	Northeast	6.1	0.8	28
Calaveras (No)	13	Northeast	6.8	4.8	48
Monte Vista - Shannon	14	Southwest	7.0	0.8	60
San Andreas (Peninsula)	20	Southwest	7.2	15.1	100
San Andreas 1906 event	20	Southwest	8.1	17.2	464
Pilarcitos	22	West	6.7	0.7	51
San Andreas (Santa Cruz Mts)	24	Southwest	7.0	18.6	63
Butano	24	Southwest	6.7	0.7	46
Las Positas	27	Northeast	6.3	0.4	15
Sargent	27	South	6.8	1.7	57
Zayante-Vergeles 2011 CFM	32	South	7.1	0.1	90
Zayante-Vergeles	33	South	6.9	0.1	58
Greenville (No)	37	East	6.9	2.6	51
Greenville (So)	37	East	6.5	1.8	29
Mount Diablo Thrust	40	North	6.6	1.6	25
Mount Diablo Thrust South	40	Northeast	6.2	1.5	11
San Gregorio (North)	41	West	7.3	4.6	129
Mount Diablo Thrust North CFM	43	North	6.4	1.8	19
Hayward (No)	49	Northwest	6.8	8.3	53
Calaveras (So)	52	Southeast	6.4	11.6	26
Reliz	52	Southwest	7.3	0.3	127
Franklin	52	North	6.7	1.1	38
Great Valley 07 (Orestimba)	53	Northeast	6.8	0.5	66
Clayton	54	North	6.4	0.7	16
Contra Costa (Lafayette)	54	North	6.1	0.8	8
Monterey Bay-Tularcitos	54	Southwest	7.2	0.6	86
Contra Costa (Larkey)	54	North	6.0	0.8	8
Ortigalita (North)	57	East	6.6	1.8	40
Great Valley 06 (Midland) alt1	57	Northeast	7.1	0.3	69
Contra Costa (Reliez Valley)	58	North	5.9	0.2	6
Concord	58	North	6.4	3.4	18
Great Valley 06 Midland alt2	60	Northeast	6.7	0.3	33
Contra Costa Shear Zone (connector)	61	North	6.6	0.9	30

---

[1] Mean Moment Magnitude based on entire fault length rupturing using Hanks and Bakun (2008)

San Gregorio (South)	62	Southwest	7.1	2.1	90
Contra Costa (Briones)	63	North	6.0	0.4	9
Contra Costa (Southampton)	64	North	6.2	0.1	11
Los Medanos - Roe Island	66	North	6.4	0.2	21
Point Reyes 2011 connector	67	West	6.5	0.1	34
Quien Sabe	70	Southeast	6.4	0.9	25
Great Valley 05 Pittsburg Kirby Hills alt2	71	North	6.8	1.0	32
Great Valley 05 Pittsburg - Kirby Hills alt1	73	North	6.3	1.0	21
Contra Costa (Dillon Point)	73	North	6.1	0.7	11
Contra Costa (Ozal - Columbus)	74	North	6.1	0.4	9
Green Valley	75	North	6.8	3.8	43
Great Valley 08 (Quinto)	76	East	6.0	0.3	19
San Andreas (Creeping Section)	76	Southeast	7.3	18.7	121
Ortigalita (South)	76	East	6.9	1.2	62
Calaveras (So) - Paicines extension	77	Southeast	6.9	7.1	60
Contra Costa (Vallejo)	85	North	5.6	0.6	4
Contra Costa (Lake Chabot)	85	North	5.6	0.7	4
San Andreas (North Coast)	86	Northwest	7.4	18.0	171
Great Valley 09 (Laguna Seca)	89	East	6.6	1.6	39
West Napa	91	North	6.8	1.3	44
Rodgers Creek - Healdsburg	98	Northwest	7.1	5.7	82
Point Reyes	100	Northwest	6.7	0.1	63

Note: The table above is a summary and does not include all the fault segmentation, alternate traces and low activity faults included in the UCERF3 model.

E1.3 Attenuation Relationships

Based on the subsurface conditions, the site is classified as a stiff soil profile, Site Class D. Using the subsurface information available at the site, we estimated the shear wave velocity of the upper 100 feet (30 meters), V_{S30}, is approximately 820 feet per second (250 meters per second). Furthermore, NGAW-2 database indicates that depths Z_1 and $Z_{2.5}$ at close by recording stations are about 600 meters and 1.0 kilometers, respectively. These values were used in the development of site-specific spectra.

The Pacific Earthquake Engineering Research Center (PEER) embarked on the NGA-West 2 project to update the previously developed ground motion prediction equations (attenuation relationships), which were mostly published in 2014. We used the relationships by Abrahamson et al. (2014), Boore et al. (2014), Campbell and Bozorgnia (2014) and Chiou and Youngs (2014). These attenuation relationships include the average shear wave velocity in the upper 100 feet. Furthermore, these relationships were developed using the same earthquake database, therefore, the mean of the relationships (using equal weights for each attenuation relationship) is appropriate and was used to develop the recommended spectra.

The NGA relationships database includes the most up-to-date recorded and processed data. They were developed for the “mean” (Rot_{D50}) horizontal components of spectral acceleration.

E1.4 Maximum Direction

ASCE 7-16 specifies the development of MCE_R site-specific response spectra in the maximum direction. Shahi and Baker (2014) provide scaling factors that modify the geometric mean spectra to provide spectral values for the maximum response (maximum direction). Therefore, we used the scaling factors presented on Table 1 of Shahi and Baker (2014) for ratios of $Sa_{RotD100}/Sa_{GMRotD50}$ to modify the mean PSHA results.

E1.5 Near-Source Effects

The site is in the near-field region (i.e. distances less than about 15 kilometers from a fault) and therefore may experience near-field directivity effects during an earthquake on a nearby fault. It has been recognized that ground motions recorded in the near-field regions show rupture directivity and near-source effects such as velocity and displacements pulses (sometimes referred to as "fling"). In general, such effects tend to increase the long period portion of the acceleration response spectrum when compared to the mean spectrum. These effects have been demonstrated by Golesorkhi and Gouchon (2002), Somerville et al. (1995 and 1997), and Singh (1985). Somerville et al. (1997) and Abrahamson (2000) quantified near-source directivity effects and provided scaling factors for modifying the mean spectra to capture these effects. Bayless and Somerville (2013) and Watson-Lamprey (2018) provides a more recent and updated methodology to incorporate these effects in the development of response spectra with consideration of near-source effects. Directivity effects were quantified for the Hayward-Rodgers Creek fault by randomizing the hypocenter using a uniform distribution for each rupture location and magnitude using the Bayless and Somerville (2013) and Watson-Lamprey (2018) approaches. The average directivity spectrum was developed using the average quantification of both approaches.

E2.0 PSHA RESULTS

Figure E-1 presents the Rot_{D50} results of the PSHA for the 2 percent probability of exceedance in 50 years hazard level (2,475-year return period) using the four relationships discussed above as well as the mean of these relationships and the mean in the maximum direction including average directivity.

As previously discussed, we used the average of the Bayless and Somerville (2013) and Watson-Lamprey (2018) relationships to account for the average directivity at the site. Figure E-2 presents average directivity quantifications using the two relationships as well as the average of both relationships.

Figure E-3 presents the deaggregation plots of the PSHA results for the 2 percent probability of exceedance in 50 years hazard level. From the examination of these results, it can be seen that the Hayward-Rodgers Creek fault dominates the hazard at the project site at shorter periods. At longer periods, the San Andreas fault dominates the hazard at the project site.

E3.0 DETERMINISTIC ANALYSIS

We performed a deterministic analysis to develop the MCE_R spectrum at the site. In a deterministic analysis, a given magnitude earthquake occurring at a certain distance from the source is considered as input into an appropriate ground motion attenuation relationship. The same attenuation relationships, weighting factors, maximum direction factors and near-source effects as discussed in Section E1.3, E1.4, and E1.5 were used in our deterministic analysis.

On the basis of the deaggregation results we developed deterministic spectra for both scenario earthquakes:

- a Moment Magnitude of 7.3 on the Hayward-Rodgers Creek fault at a distance of 9 kilometers from the site, and;
- a Moment Magnitude of 8.1 on the San Andreas fault at a distance of 20 kilometers from the site.

Figures E-4 and E-5 present the 84th percentile deterministic results for the Hayward-Rodgers Creek and San Andreas scenarios, respectively. The mean of the four attenuation relationships for the Rot_{D50} and the mean in the maximum direction are also presented on those figures. Average directivity was included for the Hayward-Rodgers Creek scenario.

We conclude the envelope of the two scenarios be used as the deterministic basis for the development of the MCE_R. Figure E-6 presents the mean of the 84th percentile deterministic results in the maximum direction for both scenarios as well as the recommended envelope of both scenarios.

E4.0 RECOMMENDED SPECTRA

The MCE_R as defined in ASCE 7-16 is the lesser of the maximum direction PSHA spectrum having a two percent probability of exceedance in 50 years (2,475-year return period) or the maximum direction 84th percentile deterministic spectrum of the governing earthquake scenario and the DE spectrum is defined as 2/3 times the MCE_R spectrum. Furthermore, the MCE_R spectrum is defined as a risk targeted response spectrum, which corresponds to a targeted collapse probability of one percent in 50 years. The USGS Risk-Targeted Ground Motion calculator was used to determine the risk coefficients for each period of interest for the probabilistic spectrum. We used these risk coefficients to develop the risk-targeted PSHA spectrum.

Furthermore, we followed the procedures outlined in Chapter 21 of ASCE 7-16 and Supplement No. 1 to develop the site-specific spectra for MCE_R and DE. Chapter 21 of ASCE 7-16 requires the following checks:

- the largest spectral response acceleration of the resulting 84th percentile deterministic ground motion response spectra shall not be less than $1.5 \times F_a$ where F_a is equal to 1.0.

- the DE spectrum shall not fall below 80 percent of S_a determined in accordance with Section 11.4.6, where F_a is determined using Table 11.4-1 and F_v is taken as 2.5 for $S_1 \geq 0.2$ (Section 21.3 of Chapter 21 ASCE 7-16).

- The site-specific MCE_R spectral response acceleration at any period shall not be taken as less than 150 percent of the site-specific design response spectrum determined in accordance with Section 21.3.

Table E-2 presents digitized values of the site-specific spectra for the risk targeted PSHA 2,475 year return period in the maximum direction and the envelope of the 84th percentile deterministic in the maximum direction, including average directivity. The largest spectral response acceleration of the 84th percentile deterministic response spectrum in the maximum direction including average directivity is 1.805g and is greater than $1.5 \times F_a$ (where $F_a = 1.0$ for Site Class D); therefore, no further scaling of the 84th percentile deterministic spectra was needed.

Figure E-7 and Table E-2 present a comparison of the site-specific spectra for the risk-targeted 2,475-year return period PSHA and the 84th percentile deterministic spectra, both in the maximum direction including average directivity. In this case, the 84th percentile deterministic spectrum is less than the risk-targeted PSHA spectrum for a 2 percent probability of exceedance in 50 years (2,475 year return period) for periods less than or equal to 5 seconds, therefore, the basis for the development of the MCE_R spectrum should be the deterministic spectrum for periods up to 5 seconds. The DE spectrum is defined as 2/3 times the MCE_R; however the DE spectrum should not be less than 80 percent of the DE code spectrum as determined using F_a equal to 1.0 and F_v equal to 2.5 (per Section 21.3 of ASCE 7-16). As shown on Figure E-7 and Table E-2, the DE spectrum is greater than or equal to 80 percent of the of the DE code spectrum for periods up to approximately 5 seconds.

TABLE E-2

Comparison of Site-specific and Code Spectra for Development of MCE_R Spectrum per ASCE 7-16
Sa (g) for 5 percent damping

Period (sec.)	Risk-Targeted PSHA – 2,475-Year Return Period Max. Dir. – Average Directivity	Deter-ministic 84th Percentile Max. Dir. Envelope – Average Directivity	Lesser of PSHA and Deter-ministic (Initial MCE_R)	2/3 of Initial MCE_R (Initial DE)	ASCE 7-16 - 80% DE per Section 21.3 Site Class D; F_v = 2.50	Recommended Spectra	
						DE	MCE_R
0.01	1.085	0.689	0.689	0.459	0.344	0.459	0.689
0.10	1.857	1.050	1.050	0.700	0.560	0.700	1.050
0.20	2.482	1.484	1.484	0.989	0.800	0.989	1.484

Period (sec.)	Risk-Targeted PSHA – 2,475-Year Return Period Max. Dir. – Average Directivity	Deter-ministic 84th Percentile Max. Dir. Envelope – Average Directivity	Lesser of PSHA and Deter-ministic (Initial MCE$_R$)	2/3 of Initial MCE$_R$ (Initial DE)	ASCE 7-16 - 80% DE per Section 21.3 Site Class D; F$_v$ = 2.50	Recommended Spectra	
						DE	MCE$_R$
0.30	2.768	1.733	1.733	1.155	0.800	1.155	1.733
0.40	2.800	1.805	1.805	1.203	0.800	1.203	1.805
0.50	2.734	1.789	1.789	1.193	0.800	1.193	1.789
0.75	2.322	1.556	1.556	1.037	0.800	1.037	1.556
1.00	2.077	1.431	1.431	0.954	0.800	0.954	1.431
1.50	1.520	1.076	1.076	0.717	0.533	0.717	1.076
2.00	1.187	0.843	0.843	0.562	0.400	0.562	0.843
3.00	0.797	0.575	0.575	0.383	0.267	0.383	0.575
4.00	0.576	0.407	0.407	0.271	0.200	0.271	0.407
5.00	0.438	0.299	0.299	0.200	0.160	0.200	0.299

The recommended MCE$_R$ and DE spectra are presented in Table E-3 and on Figure E-8.

TABLE E-3

Recommended MCE$_R$ and DE Spectra Sa (g) for 5 percent damping

Period (seconds)	MCE$_R$	DE
0.01	0.689	0.459
0.10	1.050	0.700
0.20	1.484	0.989
0.30	1.733	1.155
0.40	1.805	1.203
0.50	1.789	1.193
0.75	1.556	1.037
1.00	1.431	0.954
1.50	1.076	0.717
2.00	0.843	0.562
3.00	0.575	0.383
4.00	0.407	0.271
5.00	0.299	0.200

Because site-specific procedure was used to determine the recommended response spectra, the corresponding values of S_{MS}, S_{M1}, S_{DS} and S_{D1} per Section 21.4 of ASCE 7-16 should be used as shown in Table E-4.

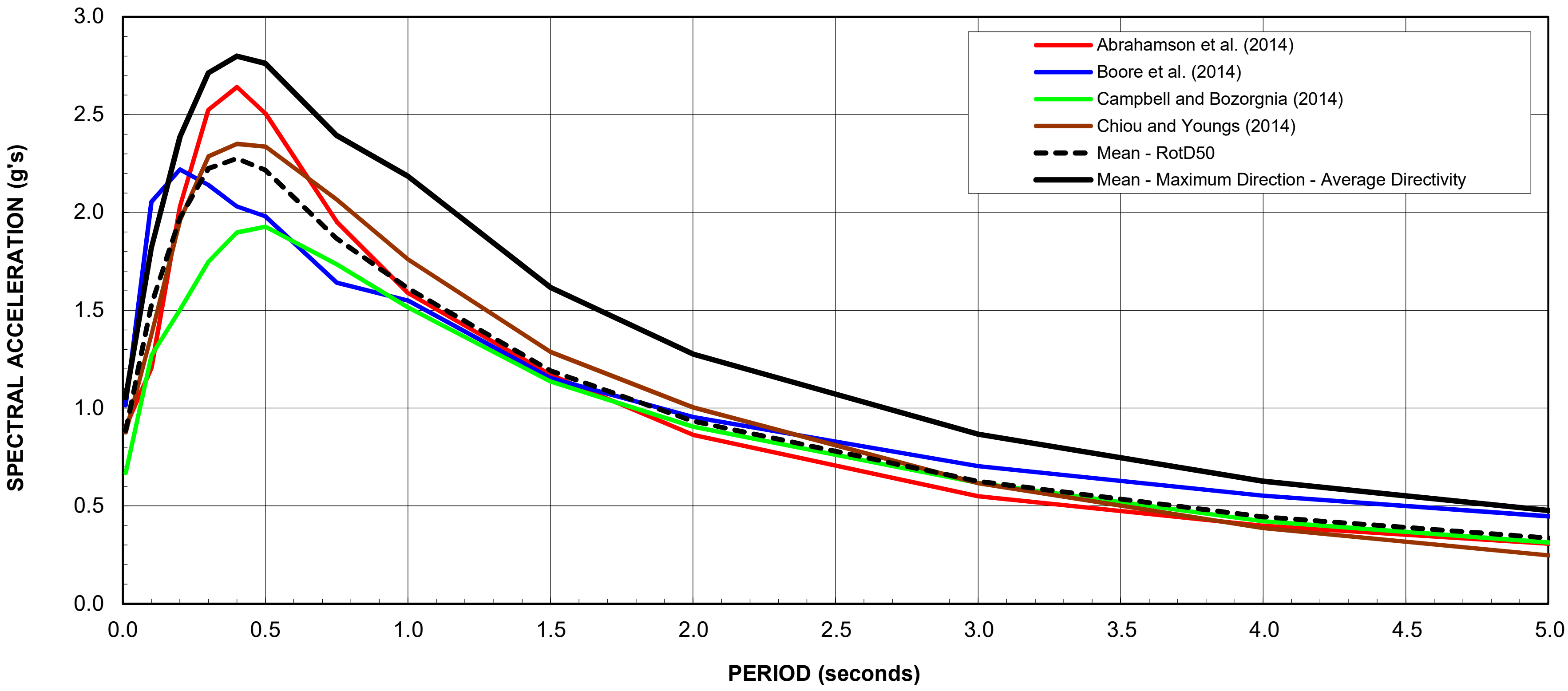
TABLE E-4
Design Spectral Acceleration Value

Parameter	Spectral Acceleration Value (g's)
S_{MS}[2]	1.624
S_{M1}[3]	1.724
S_{DS}[2]	1.083
S_{D1}[3]	1.149

---

[2] S_{DS} is based on the site-specific response spectra and is based on 90 percent of the maximum spectral acceleration within the period range of 0.2 to 5 seconds; it is governed by 90 percent of the spectral acceleration at a period of 0.4 seconds.

[3] S_{D1} is based on the site-specific response spectra and is the maximum of the product of period, T, and spectral acceleration, Sa, for periods from 1.0 to 5.0 seconds; it is governed by the product of the period and spectral acceleration at a period of 3.0 seconds.



Notes

1. Damping Ratio = 5%
2. Estimated V_{S30} = 250 m/s
3. Maximum direction factors from Shahi and Baker (2014), Directivity uses average of Bayless and Sommerville (2013) and Watson-Lamprey (2018).
4. Z_1 = 600 m, $Z_{2.5}$ = 1 km

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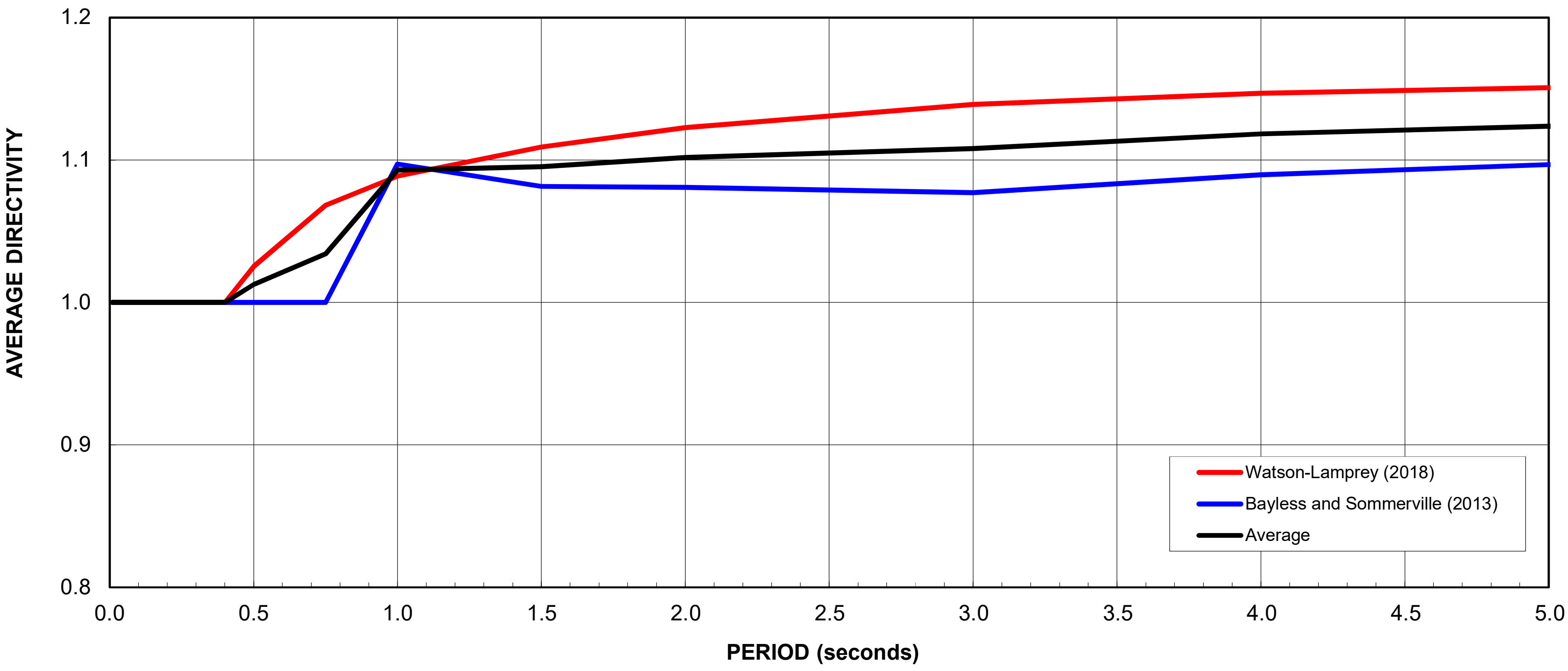
Figure Title

RESULTS OF PSHA,
2 PERCENT PROBABILITY OF
EXCEEDANCE IN 50 YEARS

Project No.
770651906
Date
1/3/2023
Scale
AS SHOWN
Prepared By:
KM

Figure No.
E-1



Notes 1. Damping Ratio = 5%

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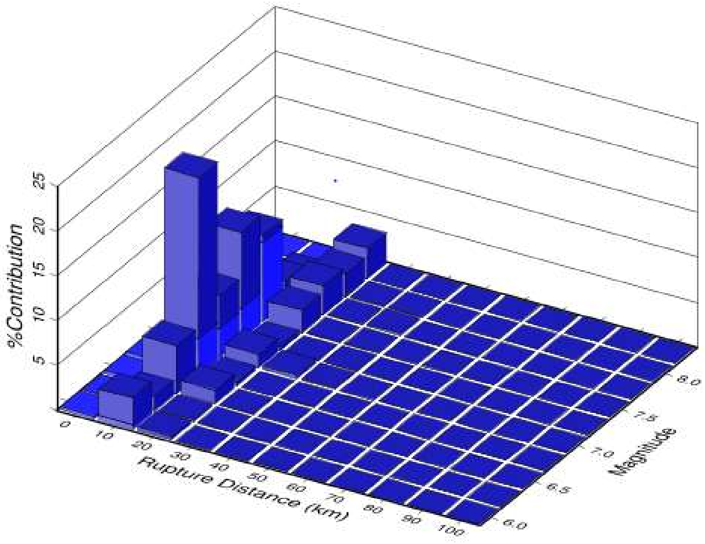
Figure Title

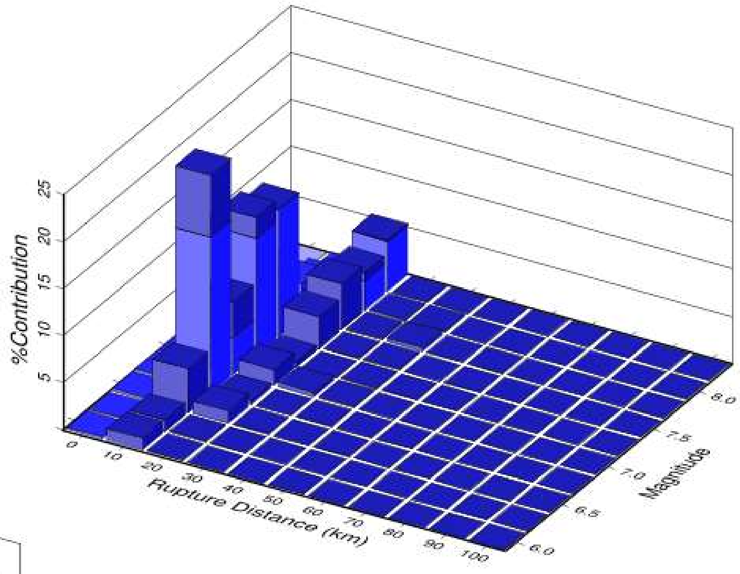
MCE$_R$ AVERAGE DIRECTIVITY
FACTORS FOR THE HAYWARD-
RODGERS CREEK FAULT

Project No. 770651906
Date 1/3/2023
Scale AS SHOWN
Prepared By: KM

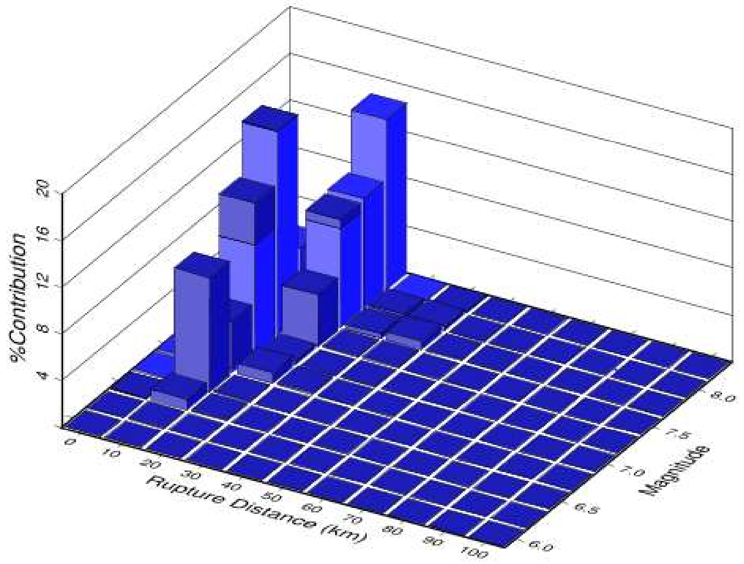
Figure No.
E-2



(a) PGA



(b) S_a, T = 1.0 second



(c) S_a, T = 4.0 seconds

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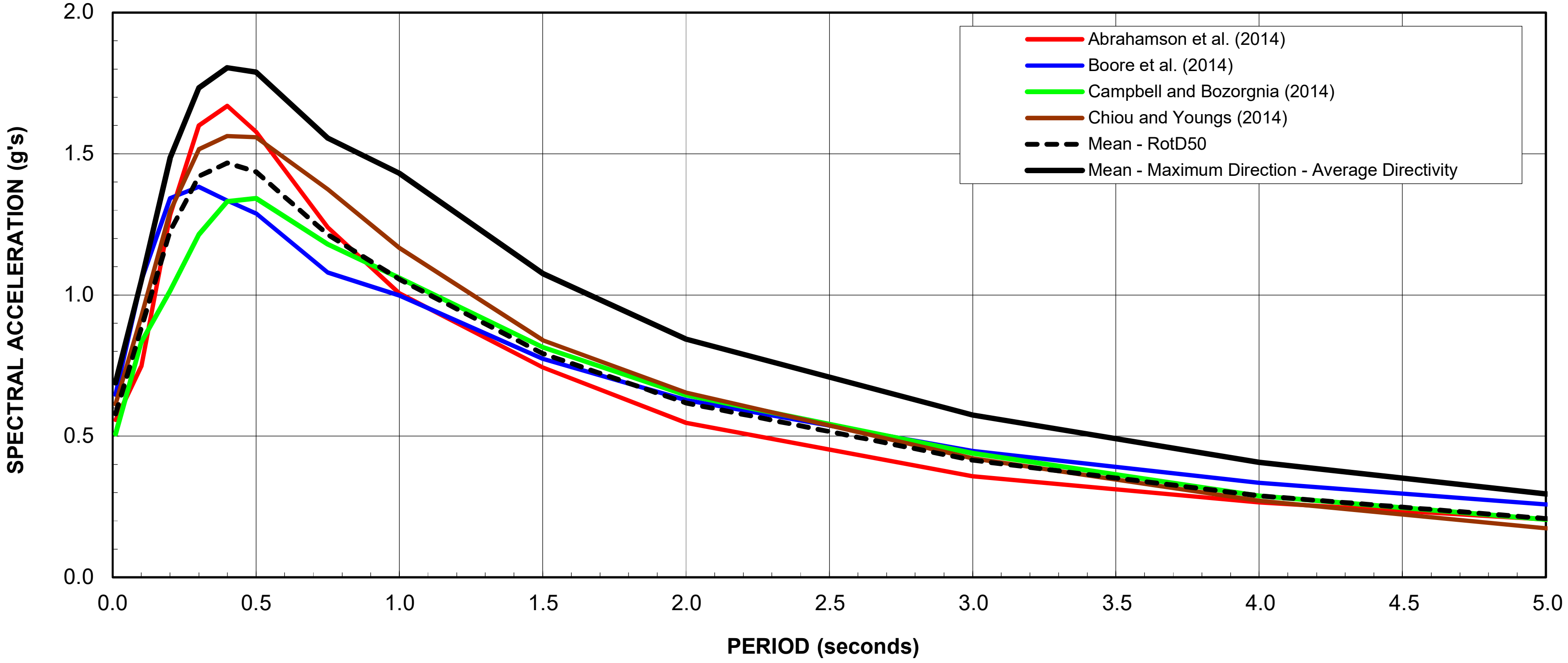
Figure Title

2% PROBABILITY OF EXCEEDANCE IN 50 YEARS - MAGNITUDE DISTANCE DEAGGREGATION PLOTS

Project No. 770651906
Date 1/3/2023
Scale AS SHOWN
Prepared By: KM

Figure No. **E-3**



Notes

1. Damping Ratio = 5%
2. Estimated V_{S30} = 250 m/s
3. Maximum direction factors from Shahi and Baker (2014), Directivity uses average of Bayless and Sommerville (2013) and Watson-Lamprey (2018).
4. Deterministic results are for a Moment Magnitude 7.3 occuring on the Hayward-Rodgers Creek fault about 9 km from the site.

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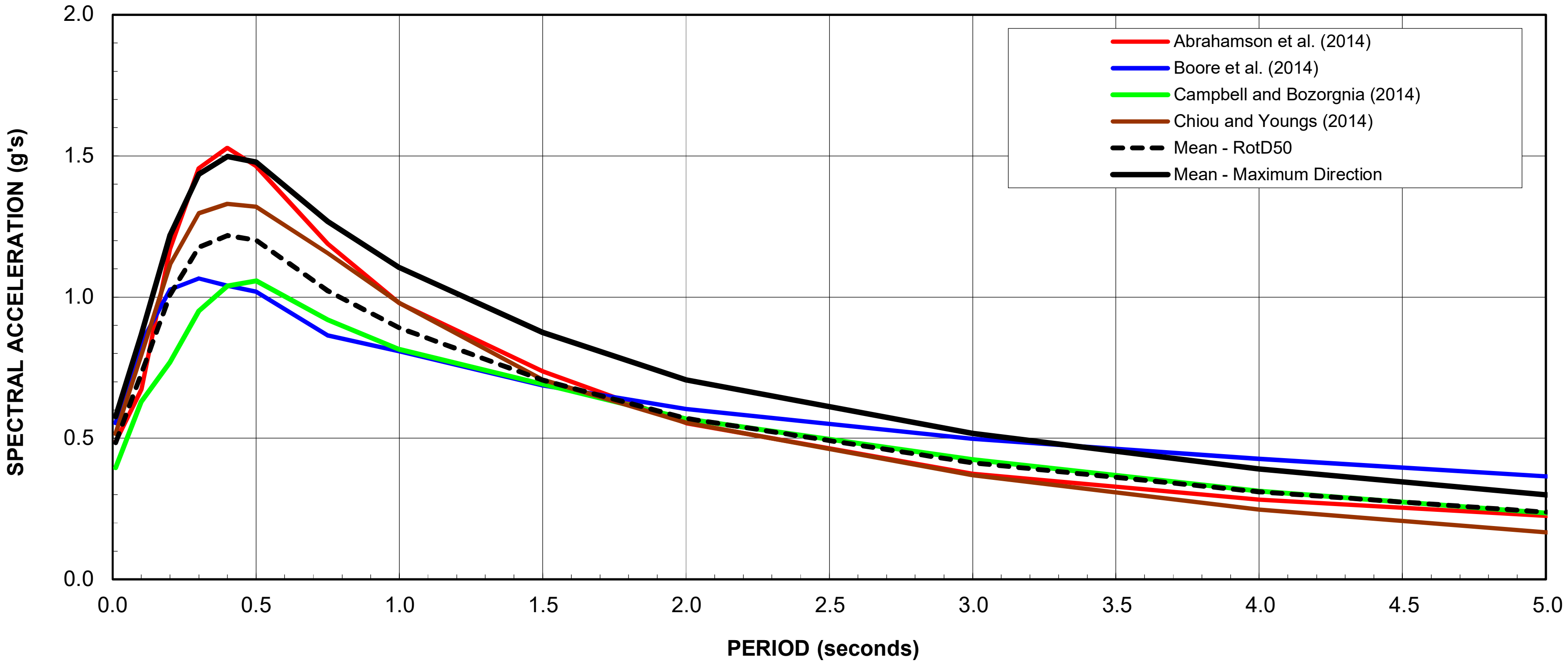
Figure Title

RESULTS OF DETERMINISTIC ANALYSIS - 84th PERCENTILE SPECTRA - HAYWARD-RODGERS CREEK

Project No. 770651906
Date 1/3/2023
Scale AS SHOWN
Prepared By: KM

Figure No.

E-4



Notes

1. Damping Ratio = 5%
2. Estimated V_{S30} = 250 m/s
3. Maximum direction factors from Shahi and Baker (2014).
4. Deterministic results are for a Moment Magnitude 8.1 occuring on the San Andreas fault about 20 km from the site

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Figure Title

RESULTS OF DETERMINISTIC ANALYSIS - 84th PERCENTILE SPECTRA - SAN ANDREAS

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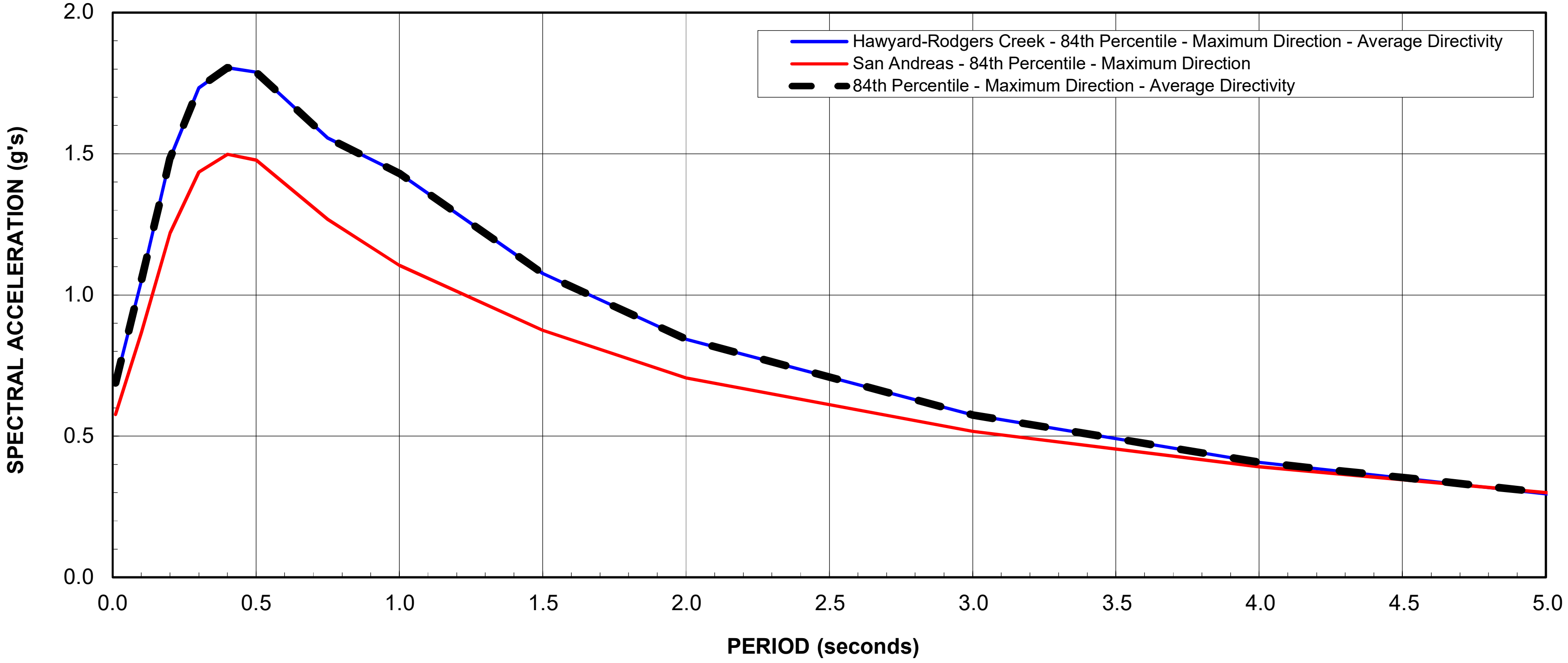
Figure No.
E-5



Notes

1. Damping Ratio = 5%
2. Estimated V_{S30} = 250 m/s
3. Maximum direction factors from Shahi and Baker (2014), Directivity uses average of Bayless and Sommerville (2013) and Watson-Lamprey (2018).

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Figure Title

COMPARISON OF 84th
PERCENTILE DETERMINISTIC
ANALYSIS

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1/3/2023
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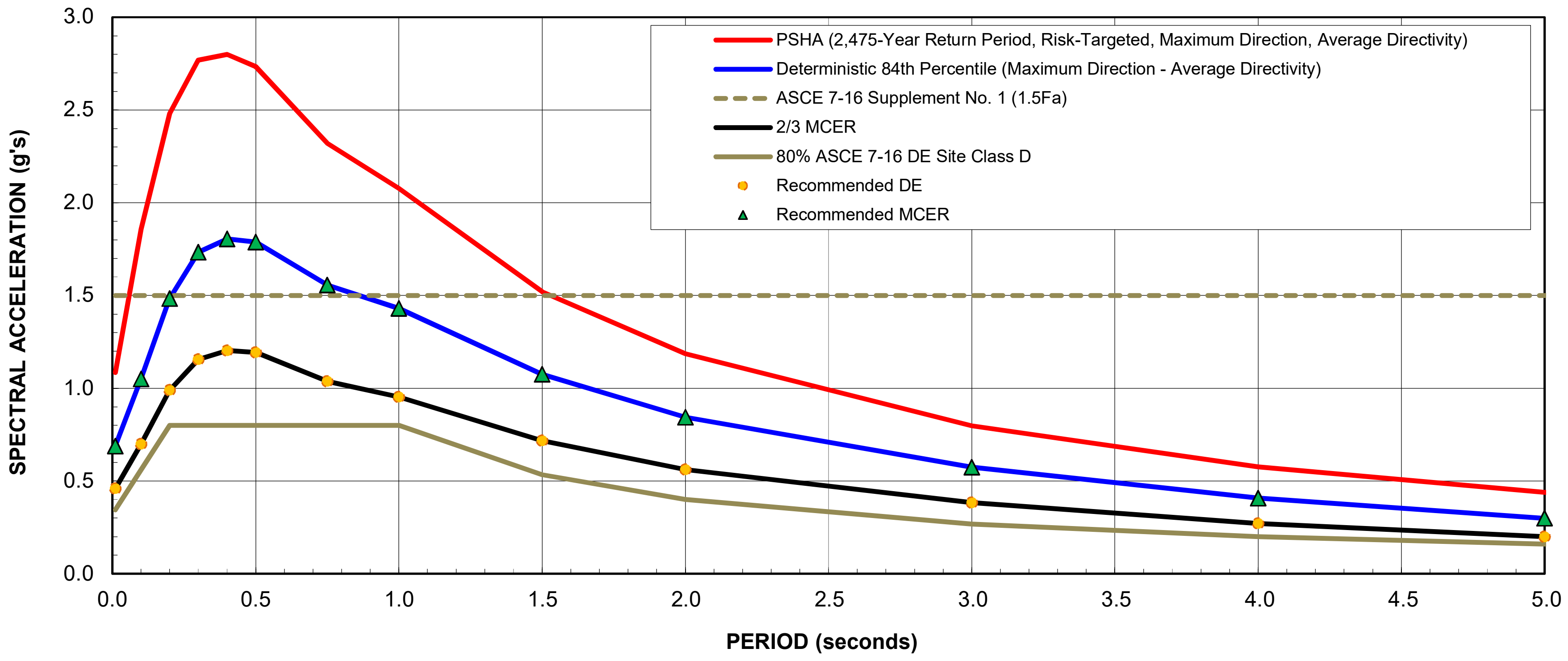
Prepared By:
KM

Figure No.
E-6



Notes

1. Damping Ratio = 5%
2. Estimated V_{S30} = 250 m/s
3. Maximum direction factors from Shahi and Baker (2014), Directivity uses average of Bayless and Sommerville (2013) and Watson-Lamprey (2018).
4. Deterministic results are the envelope for a Moment Magnitude 7.3 occuring on the Hayward-Rodgers Creek fault about 9 km from the site and a Moment Magnitude of 8.1 occuring on the San Andreas fault about 20 km from the site.

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Figure Title

COMPARISON OF PROBABILISTIC, DETERMINISTIC AND DETERMINISTIC LOWER LIMIT SPECTRA

Project No.
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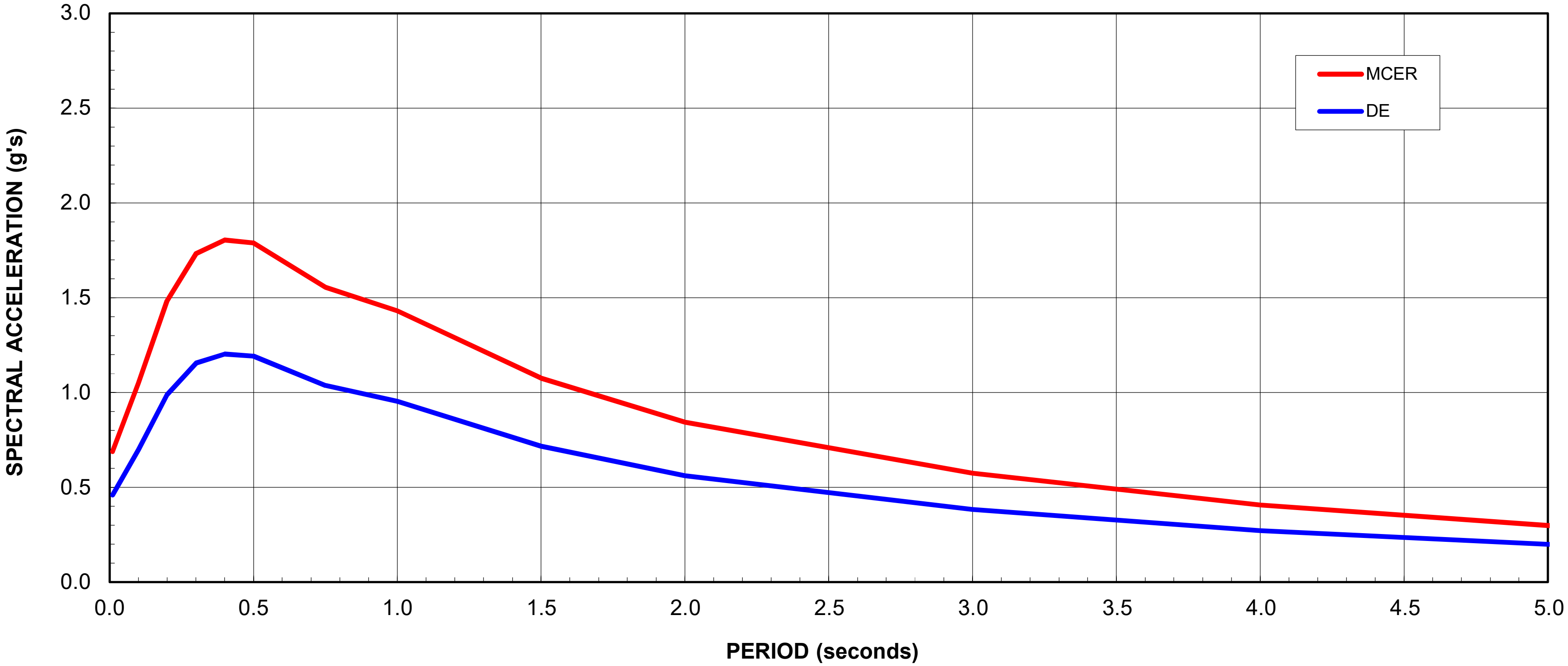
Figure No.
E-7



Notes

1. Damping Ratio = 5%
2. Estimated V_{S30} = 250 m/s

LANGAN 1 Almaden Boulevard, Suite 590 San Jose, California 95113 T: 408.283.3600 F: 408.283.3601 www.langan.com LANGAN ENGINEERING AND ENVIRONMENTAL SERVICES, INC.	Project Advanced Manufacturing Building 330 W. TRIMBLE ROAD, SAN JOSE SANTA CLARA COUNTY CALIFORNIA	Figure Title RECOMMENDED HORIZONTAL SPECTRA	Project No. 770651906 Date 1/3/2023 Scale AS SHOWN Prepared By: KM	Figure No. E-8

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APPENDIX I

San Jose Greenhouse Gas Reduction Strategy

Compliance Checklist

CITY OF
SAN JOSE
CAPITAL OF SILICON VALLEY

DEPARTMENT OF PLANNING, BUILDING AND CODE ENFORCEMENT

Purpose of the Compliance Checklist

In 2020, the City adopted a Greenhouse Gas Reduction Strategy (GHGRS) that outlines the actions the City will undertake to achieve its proportional share of State greenhouse gas (GHG) emission reductions for the interim target year 2030. The purpose of the Greenhouse Gas Reduction Strategy Compliance Checklist (Checklist) is to:

- Implement GHG reduction strategies from the 2030 GHGRS to new development projects.
- Provide a streamlined review process for proposed new development projects that are subject to discretionary review and trigger environmental review pursuant to the California Environmental Quality Act (CEQA).

The 2030 GHGRS presents the City's comprehensive path to reduce GHG emissions to achieve the 2030 reduction target, based on SB 32, BAAQMD, and OPR. Additionally, the 2030 GHGRS leverages other important City plans and policies; including the General Plan, Climate Smart San José, and the City Municipal Code in identifying reductions strategies that achieve the City's target. CEQA Guidelines Section 15183.5 allows for public agencies to analyze and mitigate GHG emissions as part of a larger plan for the reduction of greenhouse gases. Accordingly, the City of San José's 2030 GHGRS represents San José's qualified climate action plan in compliance with CEQA.

As described in the 2030 GHGRS, these GHG reductions will occur through a combination of City initiatives in various plans and policies and will provide reductions from both existing and new developments. This Compliance Checklist specifically applies to proposed discretionary projects that require environmental review pursuant to CEQA. Therefore, the Checklist is a critical implementation tool in the City's overall strategy to reduce GHG emissions. Implementation of applicable reduction actions in new development projects will help the City achieve incremental reductions toward its target. Per the 2030 GHGRS, the City will monitor strategy implementation and make updates, as necessary, to maintain an appropriate trajectory to the 2030 GHG target.

Pursuant to CEQA Guidelines Sections 15064(h)(3), 15130(d), and 15183(b), a project's incremental contribution to a cumulative GHG emissions effect may be determined not to be cumulatively considerable if it complies with the requirements of the GHGRS.

Instructions for Compliance Checklist

Applicants shall complete the following sections to demonstrate conformance with the City of San José 2030 Greenhouse Gas Reduction Strategy for the proposed project. All projects must complete Section A. General Plan Policy Conformance and Section B. Greenhouse Gas Reduction Strategies. Projects that propose alternative GHG mitigation measures must also complete Section C. Alternative Project Measures and Additional GHG Reductions.

A. General Plan Policy Compliance

Projects need to demonstrate consistency with the Envision San José 2040 General Plan's relevant policies for Land Use & Design, Transportation, Green Building, and Water Conservation, enumerated in Table A. All applicants shall complete the following steps.

1. Complete Table A, Item #1 to demonstrate the project's consistency with the General Plan Land Use and Circulation Diagram.
2. Complete Table A, Items #2 through #4 to demonstrate the project's consistency with General Plan policies[1] related to green building; pedestrian, bicycle & transit site design; and water conservation and urban forestry, as applicable. For each policy listed, mark the relevant yes/no check boxes to indicate project consistency, and provide a qualitative description of how the policy is implemented in the proposed project or why the policy is not applicable to the proposed project. Qualitative descriptions can be included in Table A or provided as separate attachments. This explanation will provide the basis for analysis in the CEQA document.

B. Greenhouse Gas Reduction Strategies

Table B identifies the GHGRS strategies and recommended consistency options. Projects need to demonstrate consistency with the GHGRS reduction strategies listed in Table B or document why the strategies are not applicable or are infeasible. The corresponding GHGRS strategies are indicated in the table to provide additional context, with the full text of the strategies preceding Table B.

Residential projects must complete Table B, Part 1 and 2; Non-residential projects must complete Table B, Part 2 only. All applicants shall complete the following steps for Table B.

1. Review the project consistency options described in the column titled 'GHGRS Strategy and Consistency Options'.
2. Use the check boxes in the column titled "Project Conformance" to indicate if the strategy is 'Proposed', 'Not Applicable', 'Not Feasible', or if there is an 'Alternative Measure Proposed'.

---

[1] The lists in items # 2-4 do not represent all General Plan policies but allow projects to demonstrate consistency and achievement of policies that are related to quantified reduction estimates in the 2030 GHGRS.

3. Provide a qualitative analysis of the proposed project's compliance with the GHGRS strategies in the column titled "Description of Project Measure". This will be the basis for CEQA analysis to demonstrate compliance with the 2030 GHGRS and by extension, with SB 32. The qualitative analysis should provide:

   a. A description of which consistency options are included as part of the proposed project, or

   b. A description of why the strategy is not applicable to the proposed project, or

   c. A description of why the consistency options are infeasible. If applicants select 'Not Feasible' or 'Alternative Measure Proposed', they must complete Table C to document what alternative project measures will be implemented to achieve a similar level of greenhouse gas reduction and how those reduction estimates were calculated.

C. Alternative Project Measures and Additional GHG Reductions

Projects that propose alternative GHG mitigation measures to those identified in Table B or propose to include additional GHG mitigation measures beyond those described in Tables A and B, shall provide a summary explanation of the proposed measures and demonstrate efficiency or greenhouse gas reductions achievable though the proposed measures. Documentation for these alternative or additional project measures shall be documented in Table C. Any applicants who select 'Not Feasible' or 'Alternative Measure Proposed' in Table B must complete the following steps for Table C.

1. In the column titled "Description of Proposed Measure" provide a qualitative description of what measure will be implemented, why it is proposed, and how it will reduce GHG emissions.

2. In the column titled "Description of GHG Reduction Estimate" demonstrate how the alternative project measure would achieve the same or greater level of greenhouse gas reductions as the GHGRS strategy it replaces. Documentation or calculation files can be attached separately.

3. In the column titled "Proposed Measure Implementation" identify how the measure will be implemented: incorporated as part of the project design or as an additional measure that is not part of the project (e.g., purchase of carbon offsets).

Compliance Checklist

Evaluation of Project Conformance with the 2030 Greenhouse Gas Reduction Strategy

Table A: General Plan Consistency

Development Type: ☐ Commercial ☐ Residential ☐ Office ☒ Other: Specify

The NorthTown Data Center project consists of two industrial data center buildings and one substation.

1) Consistency with the Land Use/Transportation Diagram (Land Use and Density)	**Yes**	**No**
Is the proposed Project consistent with the Land Use/Transportation Diagram?	☒	☐
If not, and the proposed project includes a General Plan Amendment, does the proposed amendment decrease GHG emissions (in absolute terms or per capita, per employee, per service population) below the level assumed in the GHGRS based on the existing planned land use? (The project could have a higher density, mix of uses, or other features that would reduce GHG emissions compared to the planned land use).[2]	☐	☐
If not, would the proposed project and the General Plan Amendment increase GHG emissions (in absolute terms or per capita, per employee, per service population)? Project is not consistent with GHGRS and further modeling will be required to determine if additional mitigation measures are necessary.	☐	☐

Response documentation: *[Either here or as an attachment]*

The proposed Project will develop an approximately 28.5-acre project site with two data center buildings (DC West and DC North), a substation, and other associated infrastructure to support the two proposed data centers. Both DC West and DC North would have a maximum height of 81.4 feet and a floor area ratio of 0.22. The DC West portion of the project site has a General Plan designation of Combined Industrial Commercial (CIC) Industrial Park (IP). The DC North portion of the project site is designated IS. The proposed industrial data center development will be compatible with both land use designations.

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[2] For example, a General Plan Amendment to change use from single-family residential to multi-family residential or a General Plan Amendment to change the use from regional-serving commercial to mixed-use urban in a transit-served area might reduce travel demand, and therefore GHG emissions from mobile sources.

2) Implementation of Green Building Measures	**Yes**	**No**
***MS-2.2**: Encourage maximized use of on-site generation of renewable energy for all new and existing buildings.*	☐	☐
Not applicable	☒	☐
Describe how the project is consistent or why the measure is not applicable. [Either here or as an attachment] The project owner will either participate in the San Jose Clean Energy (SJCE) at the Total Green Level (i.e., 100% carbon-free electricity) for electricity accounts associated with the project. Alternatively, the project owner may also participate in a clean energy program that accomplishes the same goals of 100 percent carbon-free electricity as the SJCE Total Green Level. As a result, on-site renewable energy generation is not needed to offset the project's emissions.		
***MS-2.3**: Encourage consideration of solar orientation, including building placement, landscaping, design and construction techniques for new construction to minimize energy consumption.*	☐	☐
Not applicable	☒	☐
Describe how the project is consistent or why the measure is not applicable. [Either here or as an attachment] The Project will not utilize solar orientation to minimize energy consumption due to the nature of a data center. To keep the interior of the data center buildings cool, which is necessary for the servers housed in the data centers, the building design includes minimal windows to reduce exposure to the data hall areas. Energy consumption associated with cooling is reduced with this type of building design.		
***MS-2.7**: Encourage the installation of solar panels or other clean energy power generation sources over parking areas.*	☐	☐
Not applicable	☒	☐
Describe how the project is consistent or why the measure is not applicable. [Either here or as an attachment] The Project will not install solar panels over the parking areas due to site constraints, which make this design option infeasible. Instead, the project owner will either participate in the SJCE at the Total Green Level (i.e., 100 percent carbon-free electricity) for electricity accounts associated with the project or participate in a clean energy program that accomplishes the same goals of 100% carbon-free electricity as the SJCE Total Green Level. As a result, on-site renewable energy generation is not needed to offset the project's emissions.		
***MS-2.11**: Require new development to incorporate green building practices, including those required by the Green Building Ordinance. Specifically, target reduced energy use through construction techniques (e.g., design of building envelopes and systems to maximize energy performance), through architectural design (e.g., design to maximize cross ventilation and interior daylight) and through site design techniques (e.g., orienting buildings on sites to maximize the effectiveness of passive solar design).*	☒	☐
Not applicable	☐	☐
Describe how the project is consistent or why the measure is not applicable. [Either here or as an attachment] The Project will implement Leadership in Energy and Environmental Design (LEED) and other US Green Building Council (USGBC) design and construction methodologies. Furthermore, the proposed data center buildings will be constructed in accordance with the current Title 24 and California Green Building Standards (CALGreen) requirements.		

***MS-16.2**: Promote neighborhood-based distributed clean/renewable energy generation to improve local energy security and to reduce the amount of energy wasted in transmitting electricity over long distances.*	☐	☐
Not applicable	☒	☐
Describe how the project is consistent or why the measure is not applicable. [Either here or as an attachment] The proposed Project does not include the construction of transmission lines to transmit electricity across adjacent properties. As mentioned under the response for General Plan Policy MS-2.2, the Project will utilize carbon-free electricity from SJCE (or an equivalent source) to reduce GHG emissions associated with energy consumption. Therefore, additional renewable energy generation or infrastructure is not needed to offset the Project’s emissions.		

3) Pedestrian, Bicycle & Transit Site Design Measures	**Yes**	**No**
***CD-2.1**: Promote the Circulation Goals and Policies in the Envision San José 2040 General Plan. Create streets that promote pedestrian and bicycle transportation by following applicable goals and policies in the Circulation section of the Envision San José 2040 General Plan.*		
a) Design the street network for its safe shared use by pedestrians, bicyclists, and vehicles. Include elements that increase driver awareness.	☒	☐
b) Create a comfortable and safe pedestrian environment by implementing wider sidewalks, shade structures, attractive street furniture, street trees, reduced traffic speeds, pedestrian-oriented lighting, mid-block pedestrian crossings, pedestrian-activated crossing lights, bulb-outs and curb extensions at intersections, and on-street parking that buffers pedestrians from vehicles.	☒	☐
c) Consider support for reduced parking requirements, alternative parking arrangements, and Transportation Demand Management strategies to reduce area dedicated to parking and increase area dedicated to employment, housing, parks, public art, or other amenities. Encourage de-coupled parking to ensure that the value and cost of parking are considered in real estate and business transactions.	☒	☐
Not applicable	☐	☐
Describe how the project is consistent or why the measure is not applicable. [Either here or as an attachment] The Project will improve pedestrian facility with the removal of refuge (or pork-chop) islands within the right-of-way in West Trimble Road and Orchard Parkway.		
***CD-2.5**: Integrate Green Building Goals and Policies of the Envision San José 2040 General Plan into site design to create healthful environments. Consider factors such as shaded parking areas, pedestrian connections, minimization of impervious surfaces, incorporation of stormwater treatment measures, appropriate building orientations, etc.*	☒	☐
Not applicable	☐	☐
Describe how the project is consistent or why the measure is not applicable. [Either here or as an attachment] As mentioned under Response MS.-2.11, the Project will implement LEED, USGBC, Title 24, and CALGreen design and construction methodologies. Integration of these design practices would reduce energy and water consumption.		

	Yes	No
***CD-2.11**: Within the Downtown and Urban Village Overlay areas, consistent with the minimum density requirements of the pertaining Land Use/Transportation Diagram designation, avoid the construction of surface parking lots except as an interim use, so that long-term development of the site will result in a cohesive urban form. In these areas, whenever possible, use structured parking, rather than surface parking, to fulfill parking requirements. Encourage the incorporation of alternative uses, such as parks, above parking structures.*	☐	☐
Not applicable	☒	☐
Describe how the project is consistent or why the measure is not applicable. [Either here or as an attachment] This measure is not applicable since the project site is not located within the Downtown or an Urban Village Overlay area.		
***CD-3.2**: Prioritize pedestrian and bicycle connections to transit, community facilities (including schools), commercial areas, and other areas serving daily needs. Ensure that the design of new facilities can accommodate significant anticipated future increases in bicycle and pedestrian activity.*	☒	☐
Not applicable	☐	☐
Describe how the project is consistent or why the measure is not applicable. [Either here or as an attachment] As mentioned above, the Project will improve pedestrian facilities with the removal of refuge islands within the right-of-way in West Trimble Road and Orchard Parkway. The Project would also provide 14 bicycle parking spaces with 2 long-term space and 12 short-term spaces.		
***CD-3.4**: Encourage pedestrian cross-access connections between adjacent properties and require pedestrian and bicycle connections to streets and other public spaces, with particular attention and priority given to providing convenient access to transit facilities. Provide pedestrian and vehicular connections with cross-access easements within and between new and existing developments to encourage walking and minimize interruptions by parking areas and curb cuts.*	☐	☐
Not applicable	☐	☐
Describe how the project is consistent or why the measure is not applicable. [Either here or as an attachment] The Project Site would include adequate pedestrian connectivity on-site such as an internal network of sidewalks and crosswalks connecting the buildings, substation, storage tank area, and parking lots. The sidewalks and crosswalks would connect to the existing pedestrian facilities on West Trimble Road and Orchard Parkway, which would allow pedestrians to travel to adjacent properties.		
***LU-3.5**: Balance the need for parking to support a thriving Downtown with the need to minimize the impacts of parking upon a vibrant pedestrian and transit oriented urban environment. Provide for the needs of bicyclists and pedestrians, including adequate bicycle parking areas and design measures to promote bicyclist and pedestrian safety.*	☐	☐
Not applicable	☒	☐
Describe how the project is consistent or why the measure is not applicable. [Either here or as an attachment] This measure is not applicable since the project site is not located within the Downtown area.		

	Yes	**No**
TR-2.8: *Require new development to provide on-site facilities such as bicycle storage and showers, provide connections to existing and planned facilities, dedicate land to expand existing facilities or provide new facilities such as sidewalks and/or bicycle lanes/paths, or share in the cost of improvements.*	☒	☐
Not applicable	☐	☐
Describe how the project is consistent or why the measure is not applicable. [Either here or as an attachment] The Project would provide 14 bicycle parking spaces with 2 long-term space and 12 short-term space for future employees and visitors.		
TR-7.1: *Require large employers to develop TDM programs to reduce the vehicle trips and vehicle miles generated by their employees through the use of shuttles, provision for car-sharing, bicycle sharing, carpool, parking strategies, transit incentives and other measures.*	☒	☐
Not applicable	☐	☐
Describe how the project is consistent or why the measure is not applicable. [Either here or as an attachment] The Project will prepare a Transportation Demand Management (TDM) plan in accordance with City of San José Municipal Code Chapter 20.90 Part 9. The TDM plan will demonstrate how the project will reduce vehicle miles traveled.		
TR-8.5: *Promote participation in car share programs to minimize the need for parking spaces in new and existing development.*	☒	☐
Not applicable	☐	☐
Describe how the project is consistent or why the measure is not applicable. [Either here or as an attachment] As mentioned above, the Project will prepare a TDM plan. Measures that could be included in the TDM include Provide Commute Trip Reduction Marketing/Education and Rider Sharing Program. The final TDM Plan measures will be decided by the City of San José and the Project Applicant.		
4) Water Conservation and Urban Forestry Measures	**Yes**	**No**
MS-3.1*: Require water-efficient landscaping, which conforms to the State's Model Water Efficient Landscape Ordinance, for all new commercial, institutional, industrial and developer-installed residential development unless for recreation needs or other area functions.*	☒	☐
Not applicable	☐	☐
Describe how the project is consistent or why the measure is not applicable. [Either here or as an attachment] The project's landscaping would conform to the State's Model Water Efficient Landscape Ordinance.		

	Yes	**No**
MS-3.2*: Promote the use of green building technology or techniques that can help reduce the depletion of the City's potable water supply, as building codes permit. For example, promote the use of captured rainwater, graywater, or recycled water as the preferred source for non-potable water needs such as irrigation and building cooling, consistent with Building Codes or other regulations.*	☒	☐
Not applicable	☐	☐
Describe how the project is consistent or why the measure is not applicable. [Either here or as an attachment] The project would utilize recycled water for landscape irrigation and building cooling. Potable water would only be used for uses such as toilets, sinks, and water fountains.		
MS-19.4*: Require the use of recycled water wherever feasible and cost-effective to serve existing and new development.*	☒	☐
Not applicable	☐	☐
Describe how the project is consistent or why the measure is not applicable. [Either here or as an attachment] As mentioned in response to for General Plan Policy MS-3.2, the project would utilize recycled water for landscape irrigation and building cooling.		
MS-21.3*: Ensure that San José's Community Forest is comprised of species that have low water requirements and are well adapted to its Mediterranean climate. Select and plant diverse species to prevent monocultures that are vulnerable to pest invasions. Furthermore, consider the appropriate placement of tree species and their lifespan to ensure the perpetuation of the Community Forest.*	☒	☐
Not applicable	☐	☐
Describe how the project is consistent or why the measure is not applicable. [Either here or as an attachment] The landscaping proposed by the project would be reviewed by the City prior to receiving building permits, which would ensure that the plant species selected would be appropriate and comply with the City's Community Forest guidelines.		
MS-26.1*: As a condition of new development, require the planting and maintenance of both street trees and trees on private property to achieve a level of tree coverage in compliance with and that implements City laws, policies or guidelines.*	☒	☐
Not applicable	☐	☐
Describe how the project is consistent or why the measure is not applicable. [Either here or as an attachment] The project will comply with the City's laws, policies, and/or guidelines regarding the planting and maintenance of street trees or trees.		

	Yes	No
ER-8.7*: Encourage stormwater reuse for beneficial uses in existing infrastructure and future development through the installation of rain barrels, cisterns, or other water storage and reuse facilities.*	☐	☒
Not applicable	☐	☐
Describe how the project is consistent or why the measure is not applicable. [Either here or as an attachment] Based on the low annual stormwater runoff within the San José region, the Project will not include infrastructure for water storage or reuse of runoff water.		

GHGRS Strategies

GHGRS #1: The City will implement the San José Clean Energy program to provide residents and businesses access to cleaner energy at competitive rates.

GHGRS #2: The City will implement its building reach code ordinance (adopted September 2019) and its prohibition of natural gas infrastructure ordinance (adopted October 2019) to guide the city's new construction toward zero net carbon (ZNC) buildings.

GHGRS #3: The City will expand development of rooftop solar energy through the provision of technical assistance and supportive financial incentives to make progress toward the Climate Smart San José goal of becoming a one-gigawatt solar city.

GHGRS #4: The City will support a transition to building decarbonization through increased efficiency improvements in the existing building stock and reduced use of natural gas appliances and equipment.

GHGRS #5: As an expansion to Climate Smart San José, the City will update its Zero Waste Strategic Plan and reassess zero waste strategies. Throughout the development of the update, the City will continue to divert 90 percent of waste away from landfills through source reduction, recycling, food recovery and composting, and other strategies.

GHGRS #6: The City will continue to be a partner in the Caltrain Modernization Project to enhance local transit opportunities while simultaneously improving the city's air quality.

GHGRS #7: The City will expand its water conservation efforts to achieve and sustain long-term per capita reductions that ensure a reliable water supply with a changing climate, through regional partnerships, sustainable landscape designs, green infrastructure, and water-efficient technology and systems.

Table B: 2030 Greenhouse Gas Reduction Strategy Compliance

GHGRS Strategy and Consistency Options	Description of Project Measure	Project Conformance
PART 1: RESIDENTIAL PROJECTS ONLY		
Zero Net Carbon Residential Construction 1. Achieve/exceed the City's Reach Code, and 2. Exclude natural gas infrastructure in new construction, or 3. Install on-site renewable energy systems or participate in a community solar program to offset 100% of the project's estimated energy demand, or 4. Participate in San José Clean Energy at the Total Green level (i.e., 100% carbon-free electricity) for electricity accounts associated with the project until which time SJCE achieves 100% carbon-free electricity for all accounts. **Supports Strategies**: GHGRS #1, GHGRS #2, GHGRS #3	*Describe which, if any, project consistency options from the leftmost column you are implementing.* *OR,* *Describe why this strategy is not applicable to your project.* *OR,* *Describe why such measures are infeasible.* This measure is not applicable to the Project since it applies to residential projects only. The data center buildings and substation are industrial developments.	☐ Proposed ☒ Not Applicable ☐ Not Feasible* ☐ Alternative Measure Proposed ** The 2030 GHGRS assumed this strategy would be feasible for 50% of residential units constructed between 2020 and 2030.*
PART 2: RESIDENTIAL AND NON-RESIDENTIAL PROJECTS		
Renewable Energy Development 1. Install solar panels, solar hot water, or other clean energy power generation sources on development sites, or 2. Participate in community solar programs to support development of renewable energy in the community, or 3. Participate in San José Clean Energy at the Total Green level (i.e., 100% carbon-free electricity) for electricity accounts associated with the project. **Supports Strategies**: GHGRS #1, GHGRS #3	*Describe which, if any, project consistency options from the leftmost column you are implementing.* *OR,* *Describe why this strategy is not applicable to your project.* *OR,* *Describe why such measures are infeasible.* The Project Owner will either participate in the SJCE at the Total Green Level (i.e., 100% carbon-free electricity) for electricity accounts associated with the project or participate in a clean energy program that accomplishes the same goals of 100% carbon-free electricity as the SJCE Total Green Level.	☐ See Part 1 (Residential projects only) ☒ Proposed ☐ Not Applicable ☐ Not Feasible ☐ Alternative Measure Proposed

GHGRS Strategy and Consistency Options	Description of Project Measure	Project Conformance
Building Retrofits – Natural Gas[3] This strategy only applies to projects that include a retrofit of an existing building. If the proposed project does not include a retrofit, select "Not Applicable" in the Project Conformance column. 1. Replace an existing natural gas appliance with an electric alternative (e.g., space heater, water heater, clothes dryer), or 2. Replace an existing natural gas appliance with a high-efficiency model **Supports Strategies:** GHGRS #4	*Describe which, if any, project consistency options from the leftmost column you are implementing.* *OR,* *Describe why this strategy is not applicable to your project.* *OR,* *Describe why such measures are infeasible.* This measure is not applicable to the Project because it does not include the retrofitting of an existing building.	☐ Proposed ☒ Not Applicable ☐ Not Feasible ☐ Alternative Measure Proposed
Zero Waste Goal 1. Provide space for organic waste (e.g., food scraps, yard waste) collection containers, and/or 2. Exceed the City's construction & demolition waste diversion requirement. **Supports Strategies:** GHGRS #5	*Describe which, if any, project consistency options from the leftmost column you are implementing.* *OR,* *Describe why this strategy is not applicable to your project.* *OR,* *Describe why such measures are infeasible.* The Project would exceed the City's construction and demolition waste diversion requirement.	☒ Proposed ☐ Not Applicable ☐ Not Feasible ☐ Alternative Measure Proposed
Caltrain Modernization 1. For projects located within ½ mile of a Caltrain station, establish a program through which to provide project tenants and/or residents with free or reduced Caltrain passes or	*Describe which, if any, project consistency options from the leftmost column you are implementing.* *OR,* *Describe why this strategy is not applicable to your project.* *OR,* *Describe why such measures are infeasible.*	☒ Proposed ☐ Not Applicable ☐ Not Feasible ☐ Alternative Measure Proposed

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[3] GHGRS Strategy #4 applies to existing building retrofits and not to new construction; Strategy #2 applies to new construction to reduce natural gas related GHG emissions

GHGRS Strategy and Consistency Options	Description of Project Measure	Project Conformance
2. Develop a program that provides project tenants and/or residents with options to reduce their vehicle miles traveled (e.g., a TDM program), which could include transit passes, bike lockers and showers, or other strategies to reduce project related VMT. **Supports Strategies:** GHGRS #6	The project would implement a TDM plan, as required for all new development in San José, to reduce vehicle miles traveled.	
Water Conservation 1. Install high-efficiency appliances/fixtures to reduce water use, and/or include water-sensitive landscape design, and/or 2. Provide access to reclaimed water for outdoor water use on the project site. **Supports Strategies:** GHGRS #7	*Describe which, if any, project consistency options from the leftmost column you are implementing.* *OR,* *Describe why this strategy is not applicable to your project.* *OR,* *Describe why such measures are infeasible.* The Project will install high-efficiency appliances and fixtures pursuant with Title 24 and CalGreen requirements and include a water-sensitive landscape design. The project would utilize recycled water for landscape irrigation and building cooling.	☐ Proposed ☐ Not Applicable ☐ Not Feasible ☐ Alternative Measure Proposed

Table C: Applicant Proposed Greenhouse Gas Reduction Measures

Description of Proposed Measure	**Description of GHG Reduction Estimate**	**Proposed Measure Implementation**
[Describe the proposed project measure and why it is proposed] **Supports Strategies/Sectors:** GHGRS #	*[Demonstrate the effectiveness of the proposed measure to reduce the project's GHG emissions.* *Include a description of how your measure will reduce emissions and provide supporting quantification documentation/assumptions.]*	☐ Part of Design ☐ Additional Measure
[Describe the proposed project measure and why it is proposed] **Supports Strategies/Sectors:** GHGRS #	*[Demonstrate the effectiveness of the proposed measure to reduce the project's GHG emissions.* *Include a description of how your measure will reduce emissions and provide supporting quantification documentation/assumptions.]*	☐ Part of Design ☐ Additional Measure
[Describe the proposed project measure and why it is proposed] **Supports Strategies/Sectors:** GHGRS #	*[Demonstrate the effectiveness of the proposed measure to reduce the project's GHG emissions.* *Include a description of how your measure will reduce emissions and provide supporting quantification documentation/assumptions.]*	☐ Part of Design ☐ Additional Measure
[Describe the proposed project measure and why it is proposed] **Supports Strategies/Sectors:** GHGRS #	*[Demonstrate the effectiveness of the proposed measure to reduce the project's GHG emissions.* *Include a description of how your measure will reduce emissions and provide supporting quantification documentation/assumptions.]*	☐ Part of Design ☐ Additional Measure