GEOTECHNICAL INVESTIGATION
NORTH TOWN
San Jose, California

Prepared For:

LBA Realty
1 Embarcadero Center, Suite 710
San Francisco, California

Prepared By:

Langan Engineering and Environmental Services, Inc.
1 Almaden Boulevard, Suite 590
San Jose, California 95113

Serena T. Jang, GE #2702
Senior Associate/Vice President

John Gouchon, GE #2282
Principal/Vice President

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770651903

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1.0 INTRODUCTION

This report presents the results of the geotechnical investigation by Langan Engineering for the proposed North Town development in San Jose, California. The approximate location of the site is shown on Figure 1.

The site is at the southwest corner of the intersection W. Trimble Road and Orchard Parkway, as shown on Figure 2. It is currently occupied by paved parking and landscaped areas associated with the adjacent commercial buildings and is relatively flat with ground surface elevations ranging from approximately Elevation 25.5 to 30 feet[1].

Based on schematic development plans for the site (Kay Victor, 2019), we understand the development will consist of five new buildings, as shown on Figure 2. The proposed development, including structural loads provided by PK Associates, the structural engineer for Buildings B through D, includes:

- Parking Structure: a four-story structure, at-grade; building loads for the parking structure are currently not available.
- Building A: a make-ready pad for a future 6- to 7-story Hotel development;
- Building B: one-story building with dead plus live column loads of 100 kips with average column spacing of 40 feet;
- Building C: three-story office building with dead plus live load of 375 kips with average column spacing of 30 feet;
- Building D: one-story building with dead plus live column loads of 75 kips with average column spacing of 40 feet; and
- associated parking, hardscaped, and landscaped areas.

Proposed grading plans are currently not available.

[1] All elevations reference North American Vertical Datum of 1988 (NAVD 88). Existing elevations taken from topographic map titled "20190522 515402SIEX Survey Exhibit.pdf" provided by HMH on 22 May 2019.

2.0 SCOPE OF SERVICES

Our geotechnical investigation was performed in general accordance with the scope of services outlined in our proposal dated 22 January 2019. Our services included evaluating the findings from field exploration at the project site and performing engineering analyses to develop conclusions and recommendations regarding:

- anticipated subsurface conditions including groundwater levels
- 2016 California Building Code (CBC) site classification, mapped values S_S and S_1, modification factors F_a and F_v and S_{MS} and S_{M1}
- site seismicity and potential for seismic hazards including liquefaction, lateral spreading, fault rupture
- appropriate foundation type(s) including shallow foundations and alternatives for deep foundations, as necessary
- design parameters for the recommended foundation type(s), including vertical and lateral capacities and associated estimated settlements
- subgrade preparation for slab-on-grade floors and exterior slabs and flatwork, including sidewalks
- site preparation, grading, and excavation, including criteria for fill quality and compaction
- soil corrosivity with a brief evaluation
- construction considerations

3.0 FIELD EXPLORATION AND LABORATORY TESTING

As part of our field exploration, we drilled six borings and performed seven cone penetration tests (CPTs) at the site. The approximate locations of the borings and CPTs are presented on Figure 2. Prior to 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 boring locations for underground utilities using a private utility locator. Details of each aspect of the field exploration and laboratory testing are discussed in the remainder of this section.

3.1 Borings

Six borings, designated B-1 through B-6, were drilled on 4 to 7 March 2019. Borings B-1, B-4 through B-6 were drilled using a truck-mounted, drill rig operated by Exploration Geoservices, Inc.; the 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 Drilling Company; the borings were drilled to depths of approximately 81½ feet bgs.

Our engineer 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 on the basis of 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 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, B-4 through B-7) falling 30 inches. The samplers were driven up to 18 inches and the hammer blows required to drive the samplers every six inches of penetration were recorded and are presented on the boring logs. A "blow count" is defined as the number of hammer blows per six inches of penetration or 50 blows for six inches or less of penetration. The driving of samplers was discontinued if the observed (recorded) blow count was 50 for six 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 for Borings B-1, B-4 through B-7, respectively, and factors of 0.7 and 1.2 for Borings B-2 and B-3, respectively, to account for sampler type and hammer energy, and are shown on the boring logs. The blow counts used for this conversion were the last two blow counts.

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

Upon completion, the boreholes were backfilled with cement grout in accordance with the requirements of the SCVWD.

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.

3.2 Cone Penetration Test

Seven CPTs (designated as CPT-1 through CPT-7) were performed on 4 and 5 March 2019 by ConeTec 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 the 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.

Pore-pressure dissipation tests (PPDTs) were performed during the advancement of all the CPTs at various depths. PPDTs are 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 this investigation, the duration of the tests range from approximately 215 to 635 seconds. The results of the seven PPDTs are presented in Appendix B. Soil types were estimated using the classification chart shown at the end of 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 testing 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-9.

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 three 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% 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 is currently occupied by paved parking and landscaped areas. It is relatively flat, with ground surface elevations ranging from approximately Elevation 25.5 to 30 feet (HMH, 2019).

The surface material encountered consists of approximately 2½ inches of asphalt concrete (AC). Beneath the pavement section, the borings and CPTs indicate the site is underlain by alluvial deposits. The near surface clay (within 7½ to 10 feet of the existing ground surface) consists 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) ranging from 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 range from 360 to 1,680 pounds per square foot (psf). Laboratory test results indicate the clay has a compression ratios of 0.11 to 0.15, is overconsolidated[3] with overconsolidated rations (OCRs) of 1.6 to 3.8. In addition, laboratory test results indicate the clay layers below a depth of about 7 feet have low to moderate expansion potential with a plasticity index (PI) ranging from 8 to 10. The sand and gravel layers contain about 9 to 22½ percent fines, where tested.

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

Groundwater was encountered in the borings at depths ranging from 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 at depths between approximately 20.8 and 68.1 feet bgs, in the sand layers. The potentiometric surface of the groundwater 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 the recent rainfalls 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.

[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.

5.0 SEISMIC AND GEOLOGIC HAZARDS

5.1 Regional Seismicity

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 55 kilometers (km) of the site, the distance from the site and estimated mean characteristic Moment magnitude[4] [2007 Working Group on California Earthquake Probabilities (WGCEP) (2008) and Cao et al. (2003)] are summarized in Table 1.

TABLE 1
Regional Faults and Seismicity

Fault Segment	Approx. Distance from fault (km)	Direction from Site	Mean Characteristic Moment Magnitude
Total Hayward	11	Northeast	7.00
Total Hayward-Rodgers Creek	11	Northeast	7.33
Total Calaveras	13	East	7.03
Monte Vista-Shannon	14	Southwest	6.50
N. San Andreas - Peninsula	20	Southwest	7.23
N. San Andreas (1906 event)	20	Southwest	8.05
N. San Andreas - Santa Cruz	24	Southwest	7.12
Zayante-Vergeles	33	South	7.00
Greenville Connected	37	East	7.00
Mount Diablo Thrust	40	North	6.70
San Gregorio Connected	42	West	7.50
Great Valley 7	53	Northeast	6.90

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

[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.

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 ($M_w = 6.2$).

The most recent earthquake to be felt in the Bay Area occurred on 24 August 2014 and was located on the West Napa fault, approximately 99 kilometers north of the site, with a M_w of 6.0.

The 2014 Working Group for California Earthquake Probabilities (2014 WGCEP) at 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 (2014 WGCEP, 2015). More specific estimates of the probabilities for different faults in the Bay Area are presented in Table 2.

TABLE 2
2014 WGCEP (2015) Estimates of 30-Year Probability (2014 to 2043) of a Magnitude 6.7 or Greater Earthquake

Fault	Probability (percent)
Hayward-Rodgers Creek	32
N. San Andreas	33
Calaveras	25

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 cyclic densification[7]. Each of these conditions has been preliminarily evaluated based on our literature review, field investigation and analyses, and is discussed in this section.

5.2.1 Liquefaction and Associated Hazards

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 presented in the 1996 NCEER and the 1998 NCEER/NSF workshops on the Evaluation of Liquefaction Resistance of Soils (Youd and Idriss, 2001). The NCEER methods are updates of the simplified procedures developed by Seed et al. (1971). To estimate volumetric strain and associated liquefaction-induced settlement, we used the procedure developed by Tokimatsu and Seed (1987) for the borings and CPTs.

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:

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

- 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)*	0.50g
Predominant Earthquake Moment Magnitude (Mw)	8.0
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

* Values obtained from USGS website for liquefaction analysis per ASCE 7-10 and 2016 California Building Code

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 8.0, the cyclic resistance ratio (CRR) has been scaled to a moment magnitude of 7.5 using magnitude scaling factors developed by Idriss (Youd and Idriss, 2001).

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 the medium dense layers could potentially liquefy during a major earthquake and may experience liquefaction-induced settlement. A summary of the data from borings for 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 and 5 for the borings and CPTs, respectively. The potential for sand boils and lateral spreading is discussed in the following sections.

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(%)	Estimated Vertical Settlement (inches)
B-2	12	14.3	1	28	0.5	0.35	0.49	1.1	0.8	0.09
	30	-3.7	4	27	0.5	0.44	0.32	0.6	1.1	0.51
	35	-8.7	4.5	24	0.5	0.44	0.27	0.5	1.3	0.70
									Total Settlement at B-1	**1.30**
B-3	14	13.1	4.5	21	0.5	0.37	0.23	0.6	1.5	0.81
	25	2.1	2	27	0.5	0.43	0.31	0.6	1	0.24
	35.5	-8.4	1	10	0.5	0.44	0.16	0.3	2.6	0.31
									Total Settlement at B-2	**1.36**

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(%)	Estimated Vertical Settlement (inches)
CPT-1	14.2	0.5	2.22	119	24	0.57	0.27	0.48	1.50	0.11
	23.1	0.2	2.51	129	26	0.61	0.28	0.47	1.16	0.04
	42.2	0.4	2.27	118	24	0.60	0.24	0.38	0.57	0.06
Total Settlement at CPT-1										**0.21**
CPT-2	15.8	1.3	1.95	144	29	0.58	0.36	0.62	1.19	0.17
	27.3	0.1	1.51	143	29	0.61	0.36	0.59	0.86	0.02
	31.0	0.6	2.15	107	21	0.61	0.21	0.34	1.05	0.08
	33.1	4.1	1.66	123	25	0.61	0.26	0.42	0.80	0.51
	37.5	0.7	1.44	141	28	0.60	0.34	0.56	0.63	0.09
	39.3	0.3	1.54	127	25	0.59	0.28	0.45	0.64	0.05
	41.8	0.1	2.46	100	20	0.58	0.18	0.29	0.68	0.02
	42.8	0.1	2.11	108	22	0.58	0.20	0.32	0.60	0.01
Total Settlement at CPT-2										**0.95**
CPT-3	19.6	0.3	2.34	75	15	0.62	0.12	0.19	1.97	0.10
	25.5	2.1	2.15	121	24	0.64	0.25	0.39	1.11	0.26
	30.6	0.3	2.16	83	17	0.64	0.13	0.21	1.30	0.05
	34.2	0.1	2.53	79	16	0.64	0.13	0.20	1.20	0.02
	37.8	3.6	1.95	137	27	0.62	0.32	0.51	0.60	0.45
	41.8	0.1	1.64	127	25	0.60	0.28	0.44	0.56	0.02
Total Settlement at CPT-3										**0.90**
CPT-4	20.9	0.4	2.22	82	16	0.63	0.13	0.21	1.78	0.12
Total Settlement at CPT-4										**0.12**
CPT-5	13.6	0.6	2.53	121	24	0.57	0.25	0.43	1.53	0.11
	38.6	0.3	1.57	140	28	0.60	0.33	0.54	0.61	0.05
	40.9	0.1	1.63	135	27	0.59	0.31	0.50	0.56	0.01
Total Settlement at CPT-5										**0.17**
CPT-6	20.2	0.6	2.25	100	20	0.60	0.17	0.29	1.54	0.11
	32.5	0.1	2.36	97	19	0.63	0.16	0.26	1.08	0.01
	36.4	0.2	2.54	85	17	0.62	0.14	0.22	1.02	0.03
	37.1	0.2	1.97	140	28	0.62	0.34	0.54	0.66	0.03
	40.4	0.1	1.92	146	29	0.60	0.37	0.60	0.54	0.02
	40.9	0.8	1.83	135	27	0.60	0.31	0.50	0.55	0.11
Total Settlement at CPT-6										**0.31**

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(%)	Estimated Vertical Settlement (inches)
CPT-7	16.0	0.2	2.36	104	21	0.59	0.19	0.32	1.64	0.06
	18.0	0.5	1.97	123	25	0.60	0.26	0.44	1.35	0.09
	25.4	0.1	1.51	138	28	0.62	0.33	0.54	0.95	0.02
	30.8	0.1	1.57	142	28	0.62	0.35	0.56	0.83	0.01
	31.3	0.1	2.53	98	20	0.62	0.17	0.27	1.12	0.02
Total Settlement at CPT-7										**0.20**

We conclude several layers are potentially liquefiable during a major earthquake.

We estimate that up to 1½ inch of liquefaction-induced settlements may occur at the Building A (Hotel Building) and Parking Structure and up to ½ inch of liquefaction-induced settlements may occur at the Buildings B through D. Because the layers appear discontinuous, differential settlement may be up to one inch over 30 feet.

5.2.2 Seismic Densification

Cyclic 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 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 they appear to be discontinuous. In addition, the Guadalupe River 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 buildings. 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 existing near-surface soil has very high expansion potential. Moisture fluctuations in 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 proposed in 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. New foundations can be embedded below the zone of severe moisture change to reduce the effects of expansive soil.

An alternative to importing select fill includes lime treatment of the near surface soil. Lime stabilization of the at-grade building pads 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.

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. Lime can also reduce the swell potential and increase the shear strength of the soil.

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 to 7 percent by weight and should be determined by laboratory testing. If lime 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. We should be provided with the results to evaluate the effectiveness of the lime.

6.2 Settlement and Foundations

Currently, a site grading plan is not available. If new fill is placed to grade the site, we estimate approximately ¼-inch and ½-inch of consolidation settlement for one and two feet of new fill, respectively.

A summary of the preliminary average column spacing and dead plus live column load for the proposed buildings are presented on Table 6. The column spacing and columns loads for Buildings B through D were provided by PK Associates (PK Associates, 2019). However, information regarding structural loads for the parking structure and Building A (Hotel Building) are currently not available; therefore, we have assumed the following column spacing and loads for the proposed buildings based on their proposed heights.

TABLE 6
Preliminary Building Loads

Building		Building Stories/ Levels	Preliminary Average Column Spacing (feet)	Preliminary Dead Plus Live Column Load[1] (kips)
Buildings B through D (Office Buildings)	At-Grade	1 to 3	30 to 40	75 to 375
Parking Structure	At-Grade	4	30	450 to 720
Building A (Hotel Building)	At-Grade	6 to 7	20	300 to 560

Note:
1. For parking structure and Building A, assume dead plus live load per level for concrete structures is 200 pounds per square foot (psf) and steel structures is 125 psf.

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

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

Settlements and potential foundation types for the proposed structures, including shallow and deep foundations, are discussed in the following subsections. Once building design, loads and grading plans are available, the settlement estimates can be refined.

All the buildings are proposed to be constructed at grade. Due to the presence of the very highly expansive near-surface clay, we conclude at-grade structures be supported on mat foundations bearing on stiff to very stiff clay, provided the static and seismically-induced settlement discussed below and in Section 5.2.1 are tolerable. If the settlements are not tolerable, then deep foundations should be considered.

The proposed building sites are susceptible to the following potential sources of settlement:

- consolidation of the underlying alluvial deposits under the weight of new building loads or new fill
- liquefaction-induced settlement.

To evaluate the settlement of the site due to consolidation of the alluvial deposits under the weight of the new building loads, we reviewed the laboratory consolidation tests on relatively undisturbed samples of the clay, as presented in Appendix A. The test results indicate the alluvial clay generally have OCRs ranging from 1.6 to 3.8.

Assuming the building loads presented on Table 6, there will be an increase in overburden pressure from the building loads, which we conclude will cause the underlying soils to settle. The estimated mat pressures with the resulting estimated total and differential static settlement for the structures on a mat foundation are presented in Table 7. The rigidity of the mat is not included in the settlement analysis. The mat should reduce the estimated differential settlement.

TABLE 7
Summary of Estimated Static Settlements for Mat Foundation

Buildings[1]	Approximate Total Settlement (inch)	Approximate Differential Settlement[2] (inch)	Allowable Dead Plus Live Load Mat Pressure (psf)
Buildings B through D (Office Buildings)	¼ to ¾	¼ to ½	50 to 420
Parking Structures (4-stories)	½ to 1½	¾ to 1	800
Building A (Hotel Building, 6-stories)	½ to 2½	¾ to 1½	750 to 1,400

Notes:
1. We have assumed the building finished floor elevations will be founded near the existing ground surface elevation.
2. Differential settlement between columns, typically 20 feet apart.

As discussed previously, we estimate that up to 1½ inch of liquefaction-induced settlements may occur at the Parking Structure and Building A (Hotel Building) and up to ½ inch of liquefaction-induced settlements may occur at the Buildings B through D; differential settlement between columns may be on the order of ½ to 1-inch during a major earthquake. These settlements are in addition to the predicted consolidation static settlement.

The structural engineer should evaluate the impact of the static and liquefaction-induced settlement to structures supported on a mat foundation. If the total and differential settlements are not tolerable, a deep foundation should be considered. We judge the anticipated static and liquefaction-induced settlement of the Building A (Hotel Building) on a shallow foundation will not be tolerable; therefore, we conclude the Building A (Hotel Building) should be supported on a deep foundation system.

Where deep foundations are considered, we conclude a drilled pile system, such as auger cast piles (ACP) or similar pile types, which are low-vibration, low-noise, deep foundation options, are most practical the site based on our past experience of sites in the vicinity. These pile types are designed and installed by specialty contractors. If these pile types are used, they will need to be tested to confirm the design values.

ACPs are installed by drilling to the required depth with a hollow-stem auger. When the auger reaches the required depth, cement grout or concrete is injected through the bottom of the hollow-stem auger. Grout or concrete is injected continuously as the auger, still rotating in a forward direction, is slowly withdrawn. While the grout is still fluid, a steel reinforcing cage is inserted into the shaft.

We estimate properly constructed ACPs with approximately lengths of 65 feet should have a total settlement less than one inch, with less than ½ inch of differential settlement between adjacent columns supported on new piles. Most of these static settlements are expected to occur during construction.

Although a pile system will reduce total and differential settlements of the Building A (Hotel Building), liquefaction induced settlements might still occur, which could impact the floor slabs. If a slab-on-grade is used, then total and differential liquefaction-induced settlements of up to 1½ inches may occur. This could cause cracking and off-sets in the slab. If this performance is not tolerable, a structural slab spanning between the pile caps and grade beams should be used. In addition, the near-surface soil consists of very high expansion potential clay. The floor slabs should be designed to resist uplift forces due to the expansive soil.

6.3 Groundwater and Dewatering Considerations

Historic high groundwater 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 to evaluate corrosion potential to buried metals and concrete. The results of the tests are presented in Appendix D and summarized in Table 8.

TABLE 8
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 and liquefaction induced settlements discussed in Section 6.2 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, fill placement 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 excavation 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 portions 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. Where highly expansive clay is encountered the near surface soil should be moisture conditioned to 3 to 5 percent above the optimum moisture content and compacted to between 88 and 93 percent relative compaction[8].

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.

To reduce the effects of expansive soil, we recommend at-grade buildings supported on a mat foundation have mat thicknesses of 24-inches or greater; recommendations for at-grade mat foundations is discussed in Section 7.2. If the mat foundation thickness is less than 24-inches, we recommend at least 24 inches of imported (select) material or lime treated soil be placed beneath the mat slab; the select fill should extend at least five feet beyond building footprint. Prior to placement of select fill in building areas, the onsite soil exposed by stripping should be

[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.

scarified to a depth of at least 12 inches, moisture-conditioned to between 3 to 5 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 (the approximate plastic limit of the soil) should be excavated, moisture-conditioned to at least three to five percent above optimum moisture content, and recompacted to between 88 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 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 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.

[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 pads, 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.

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.

7.2 Mat Foundation

Due to the presence of the very highly expansive near surface clay soil, the at-grade buildings should be supported on a mat foundation, provided the settlement discussed in Section 6.2 are tolerable to the structural design. To reduce the potential of the mat foundation due to shrink and swell of the expansive clay, we recommend that the mat foundation have a minimum thickness of 24-inches. In addition, we recommend that perimeter of the mat edge be thickened at least 36 inches below the lowest adjacent soil subgrade. If a 24-inch thick mat is used, select fill is not required beneath the mat foundation.

To design the mat using the modulus of subgrade reaction method, we recommend a modulus of subgrade reaction of 7 kips per cubic foot (kcf). The modulus value is representative of the anticipated settlement under the preliminary building loads in Table 6. After the mat analysis is completed, we should review the computed settlement and bearing pressure plots to check that the modulus value is appropriate and provide an updated modulus of subgrade reaction, if needed. The modulus is applicable for localized dead plus live loads up to 3,000 psf.

Resistance to lateral loads can be mobilized by a combination of passive pressure acting against the vertical faces of the mat and friction along the base of the mat. Passive resistance may be calculated using lateral pressures corresponding to a uniform pressure of 1,200 psf. The upper foot of passive resistance should be neglected unless it is confined by a slab. Frictional resistance should be computed using a base friction coefficient of 0.3. These values include a factor of safety of about 1.5 and may be used in combination without reduction.

If weak or non-engineered fill is encountered in the mat excavation bottom, it should be over-excavated to firm, competent material and replaced with engineered fill or lean concrete. We should check the mat subgrade after cleaning, but prior to placement of waterproofing, mud slab, crushed rock or reinforcing steel to confirm bearing and that loose and disturbed material has been removed. The bottom and sides of mat excavation should be wetted following excavation and maintained in a moist condition until concrete is placed. If the foundation soil dries during construction, the foundation will heave when exposed to moisture, which may result in cracking and distress.

7.3 Auger Cast Piles

Up to two inches of settlement is anticipated for the settlement of the Building A (Hotel Building). Therefore, we recommend the Building A (Hotel Building) be supported on a deep foundation system consisting of ACPs. The ACP piles will gain capacity from skin friction in the medium stiff to very stiff clays and the medium dense to dense sands.

ACP are installed by design-build or specialty contractors. The vertical and lateral capacities presented in the following subsections for ACP are preliminary and may be used in pricing and estimating. Final design capacities should be determined by the selected specialty/design-build contractor and verified by a test program.

7.3.1 Axial Capacity

Table 9 provides preliminary design axial capacities for 16-inch diameter ACPs. The preliminary allowable compressive capacities for the ACPs are based on the results of our analyses and our discussions with contractors with experience installing these pile types in the Bay Area. Typically the lengths of ACPs are about 60 feet. Greater lengths may be used, but it may slow down production and limit the number of capable contractors. In our analysis, we have provided capacities for ACPs with lengths of 60 to 65 feet due to the presence of sand layers with the potential for end bearing capacity at those depths. Final design axial pile capacities for ACPs should be determined by the design/build contractors after they have been selected.

TABLE 9
Preliminary Axial Pile Capacities for 16-inch-diameter ACDPs

Length (feet)	Pile Tip Elevation (feet)	Allowable Axial Capacity Dead Plus Live Loads[1] (kips)
65	-38	300

Note:
1. The allowable dead plus live load axial capacities include a factor of safety (FS) of at least 2. The allowable dead plus live load capacities may be increased by one-third for total loads, including wind or seismic forces.

ACP design capacities should be verified by a test program. We recommend at least one compression and one tension pile load test be performed per 2016 CBC Section 1810.3.3.1.2.

Piles should be spaced at least three pile diameters center-to-center, to prevent vertical capacity reductions due to pile interaction effects; the outer auger-tip diameter should be used when determining the pile spacing for ACP piles. The piles should also be designed to account for the presence of corrosive soil; a corrosion consultant should be retained to provide specific recommendations regarding the long term corrosion protection of pile elements.

We preliminarily estimate static settlement of pile foundations will be less than ½ inch, with differential settlement between columns on the order of ½ inch. These estimates are preliminary and may vary depending on the actual pile design and the results of the pile load testing.

7.3.2 Lateral Load Resistance

The piles should develop lateral resistance from the passive pressure acting on the upper portion of the piles and their structural rigidity. The allowable lateral capacity of the piles depends on:

- the pile stiffness
- the strength of the surrounding soil
- axial load on the pile
- the allowable deflection at the pile top and the ground surface
- the allowable moment capacity of the pile.

We have evaluated the lateral capacity of 16-inch diameter ACPs for ½-inch deflection at the pile head; however limited the maximum movement to 1,000 kip-inch for a 16-inch diameter ACDP. For a free-head condition, the pile top is free to move laterally and rotate. For a fixed-head condition, the pile top is restrained from rotating but free to move laterally. Preliminary deflection and moment profiles for a single 16-inch diameter ACP are presented on Figures 5 and 6. Final design lateral pile capacities for ACPs should be determined by the design/build contractors.

The lateral capacities are for single piles only. To account for group effects, the lateral load capacity of a single pile should be multiplied by the appropriate reduction factors shown in Table 10. However, the maximum moment for a single pile with an unfactored load should be used to check the design of individual piles in a group. The reduction factors are based on a minimum center-to-center spacing of three pile diameters. Where piles are spaced at least eight pile diameters in all directions, no group reduction factors need to be applied. Reduction for other pile group spacing can be provided once the number and arrangement of piles are known.

TABLE 10
Lateral Group Reduction Factors

Number of Piles within Pile Cap	Lateral Group Reduction Factor
2	0.9
3 to 5	0.8
$\geq$6	0.7

Additional lateral load resistance can be developed by passive resistance acting against the faces of the pile caps and grade beams. To calculate the passive resistance against the vertical faces of pile caps and grade beams, we recommend a uniform pressure 1,200 psf. This value has a factor of safety of about 1.5. The upper foot should be ignored unless it is confined by a slab.

7.3.3 ACP Construction Considerations

We recommend that before production ACP pile lengths are selected, indicator piles be installed to: 1) evaluate predrilling requirements, if any, and 2) estimate production pile lengths. We recommend a minimum of 10 indicator piles be installed. We expect the indicator piles can be used as production piles if installed in the proper location and are not damaged during installation or testing. If indicator piles are to be abandoned following the indicator program, then the indicator piles should be located at least seven pile diameters (center-to-center) from production pile locations. Indicator piles should be installed with the same equipment and using the same procedure, including predrilling depth and predrill auger diameter, that will be used for production piles.

7.3.4 Pile Load Test Program

We recommend load tests of the ACP piles be performed to confirm the axial compression and tensile pile capacities. We recommend a minimum of one compression and one uplift load tests be performed for each proposed production pile installation methodology (i.e. rig type, predrilling depth and diameter, pile length, etc.) The test pile locations should be selected by the geotechnical engineer and approved by the structural engineer. The compression load tests should be performed in accordance with ASTM D1143-07, Standard Test Method for Piles under Static Axial Compressive Load, and the tension tests should be performed in accordance with ASTM D3689-07. 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 and at least 1.5 times the total load. The Davisson Method or other accepted criteria per the 2016 California Building Code should be used to interpret the ultimate capacities of the piles.

7.3.5 Pile Installation Work Plan

A work plan describing the proposed ACP installation equipment and methodology, including, but not limited to, predrilling depth, diameter of auger used for predrilling, pile diameter and pile length, as well as the proposed indicator pile location, pile load test set-up and procedure should be submitted to Langan for review and approval at least five working days prior to the indicator pile and pile load test programs. The work plan should include a site plan showing the locations of indicator test and reaction piles relative to permanent foundation elements and a drawing showing the layout of the load test set up. Following the completion of pile load tests, the Geotechnical Engineer will require at least three working days to review and evaluate the load test results and propose recommendations for production pile installation.

Additional pile load tests will be required if, during production pile installation, the equipment or installation procedure deviates from the approved work plan and indicator pile load test program.

7.4 Retaining Wall Design

We recommend 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 pressure presented in Table 11. 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 six 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 11.

TABLE 11
Retaining Wall Design Earth Pressures
(Drained Conditions)

Condition	Static Conditions		Seismic Conditions[2]
	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.

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.

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 1.5:1 (horizontal to vertical) plane projected upward from the bottom edge of the utility trench (or adjacent footings).

10 California Building Code (2016) Section 1803.5.12.

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 we recommend 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 four 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.

7.5 Floor Slabs

Because expansive soil is present near the existing ground surface, we recommend the thickness of the mat foundation be a minimum of 24-inches; no select fill is required beneath the 24-inch thick mat slab. However, if the mat foundation is less than 24-inches thick, we recommend a 24-inch thick layer of select fill (or lime treated soil) should be placed beneath the floor slab for at-grade buildings. 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 moisture barrier is generally not required beneath parking garage slabs, except for areas beneath mechanical, electrical, and storage rooms. 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 12.

TABLE 12
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.

7.6 2016 CBC Mapped Values

For seismic design in accordance with the provisions of 2016 California Building Code (CBC) we recommend the following:

- Risk-Targeted Maximum Considered Earthquake (MCE_R) S_s and S_1 of 1.500g and 0.600g, respectively.
- Site Class D
- Site Coefficients F_a and F_v of 1.0 and 1.5

- MCE_R spectral response acceleration parameters at short periods, S_{MS}, and at one-second period, S_{M1}, of 1.500g and 0.900g, respectively.
- Design Earthquake (DE) spectral response acceleration parameters at short period, S_{DS}, and at one-second period, S_{D1}, of 1.000g and 0.600g, respectively.
- PGA_M is 0.5g

7.7 Utilities and Utility Backfill

Utility trenches should be excavated a minimum of four inches below the bottom of pipes or conduits and have clearances of at least four 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 four 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 six inches with sand or fine gravel, which should then be mechanically tamped 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.

As discussed in Section 7.1.1, where utility trenches backfilled with sand or gravel enter the building pads, 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.

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.

7.8 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 13 presents our recommendations for asphalt pavement sections.

TABLE 13
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 six 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.

7.9 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 HMH, the project civil engineer, concrete pavements will be designed for a TI of 13, the recommended rigid pavement section for these axle loads is six inches of Portland cement concrete over six inches of Class 2 AB. The concrete pavement section should rest on at least 12 inches of select fill.

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 moderately to 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.

7.10 Concrete Flatwork (Non-Vehicular)

We recommend new sidewalks and concrete flatwork (in non-vehicular traffic area) be underlain by at least four inches of Class 2 aggregate base 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 four inches of select fill can consist of the AB. The select fill should extend at least two 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.

7.11 Pavers

Interlocking pavers (assumed to have minimum thickness of 2.375 inch) should be placed on two 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 14.

TABLE 14
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 is highly expansive, we recommend 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 six 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.

7.12 Grasspave2 and Gravelpave2

To accommodate the anticipated tire load from fire trucks in landscape areas, we understand a porous pavement products called Grasspave2 and Gravelpave2 are being considered. The Grasspave2 and Gravelpave2 products are manufactured by Invisible Structures, Inc. and consists of high density polyethylene (HDPE) rings connected on a flexible grid system. The rings are filled with sand or gravel and turf can be laid over the rings.

We understand the City of San Jose require emergency vehicle access (EVA) roads to be designed and maintained to support the imposed load of fire apparatus with a gross vehicle weight of 75,000 pounds. Therefore, we have assumed EVA loads would come from an Aerial Platform Rear Mount (Tandem Rear Axle) fire truck. According to the "Emergency Vehicle Size and Weight Regulation Guide" (International Fire Chief Association, 2011), the Aerial Platform

Rear Mount fire truck has a maximum gross axle weight rating (GAWR) of 24,000 pounds (lbs) and 62,000 lbs, for the front and rear axles, respectively. We have assumed the fire truck will use the EVA area twice a year over a 20 year design life.

Using an R-value of 12, we recommend the Class 2 Aggregate Base (AB) that underlies the Grasspave2 or Gravelpave2 product have a minimum thickness of 15 inches. The upper six inches of the soil subgrade in should be moisture-conditioned to above optimum and compacted to at least 95 percent relative compaction. The AB should conform to current Caltrans Standard Specifications. All AB should be compacted to at least 95 percent relative compaction.

7.13 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 buildings, we recommend the ground surface within a horizontal distance of five feet from the buildings be designed to slope down and away from the buildings with a surface gradient of at least two percent in unpaved areas and one percent in paved areas. In addition, roof downspouts should be discharged into controlled drainage facilities to keep the water away from the foundations.

7.14 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 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 six 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.

7.15 Bioretention Systems

Bioretention areas are landscaping features used to treat stormwater runoff. 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 directed to the storm drain system. For larger storms, runoff is generally diverted past the bioretention areas 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. Based on the "Bioretention Manual" prepared by The Prince George's County (2007), the infiltration rate of the bioretention soil is recommended to exceed ½ inch per hour; cohesionless soils like sand meet this criterion. Cohesive soils like clay and silts do not meet the infiltration rate requirement and are considered unsuitable in a bioretention system, particularly the soils are expansive. For areas where there are unsuitable in-situ soils, the bioretention system can be created by importing a suitable soil mix and providing an underdrain. Based on our observation of the soil at the site, the in-situ clays are impervious and do not meet the infiltration rate requirements. The bioretention system will need to be constructed with suitable imported soil and include an underdrain system.

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 in a gravel blanket. The gravel should be virgin rock, double washed, uniformly graded and should be ½ inch to 1½ inches in diameter. It should also be wrapped in a filter fabric (Mirafi 140N or equivalent). The perforated PVC pipe cross-section area should be determined based on the desired hydraulic conductivity of the underdrain. The PVC pipe should be bedded on two to three inches of gravel and covered with gravel and a filter fabric (Mirafi 140NC or equivalent).

Because of the presence of near surface expansive soil, bioretention systems should be set back a minimum of five feet from building foundations, slabs, concrete flatwork or pavements. 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 two 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 recommendations. During construction, we should observe the installation of shallow and deep foundations and 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 sidewalks 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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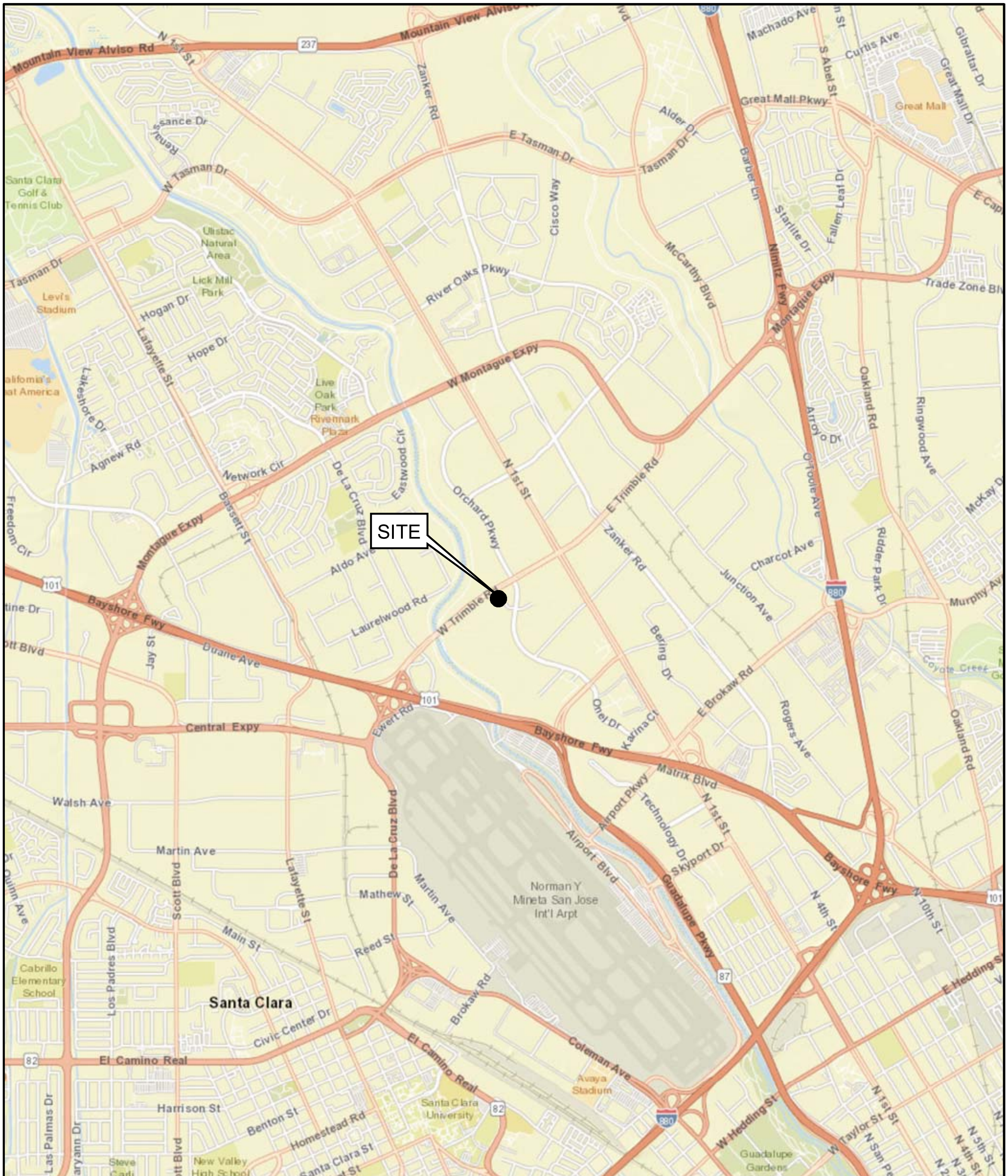
FIGURES



NOTES:

World street basemap is provided through Langan's Esri ArcGIS software licensing and ArcGIS online.
Credits: Sources: Esri, DeLorme, NAVTEQ, USGS, Intermap, iPC, NRCAN.

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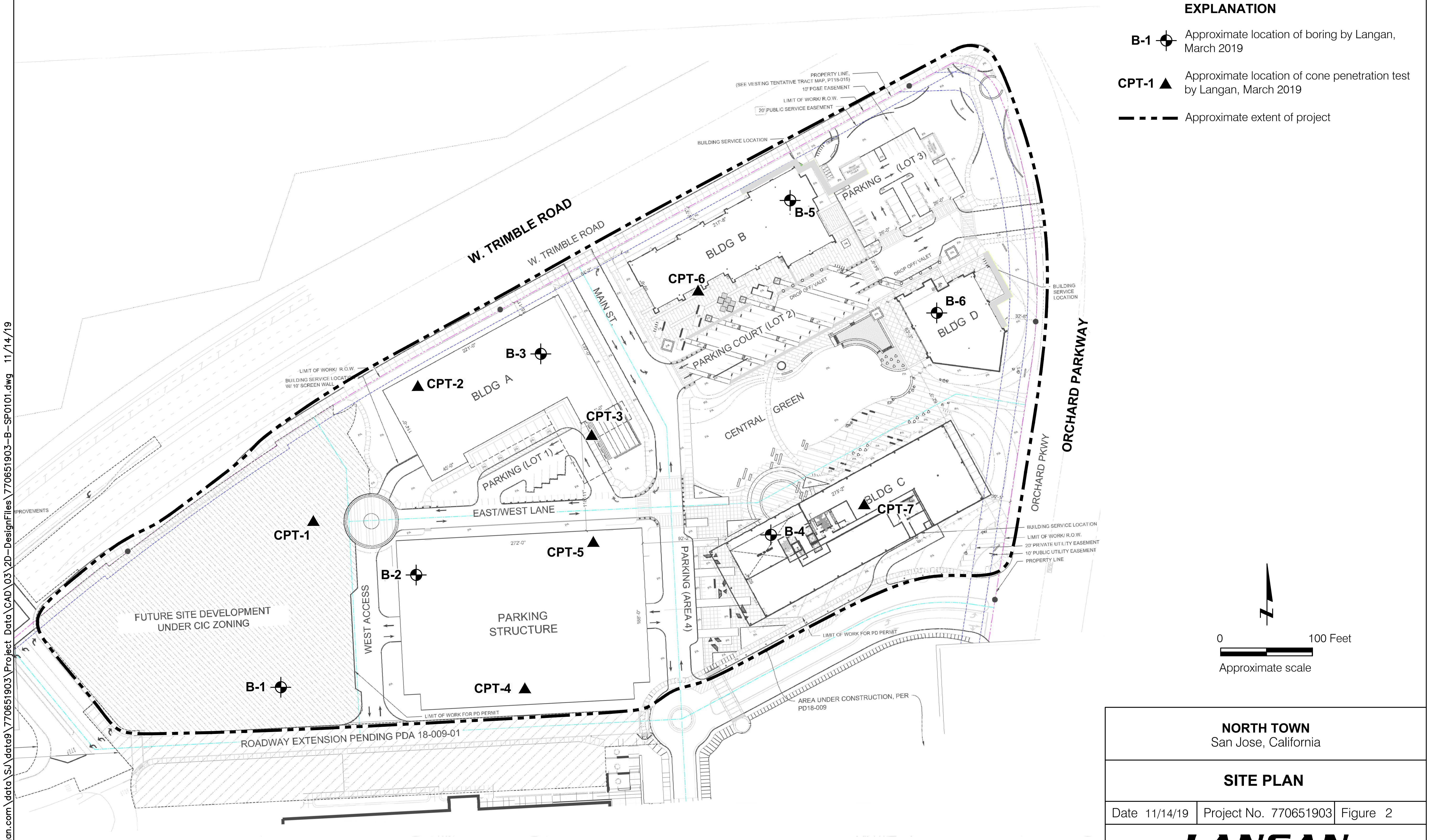
NORTH TOWN
San Jose, California

LANGAN

SITE LOCATION MAP

Date 05/20/19	Project No. 770651903	Figure 1



Reference: "Site Plan," by Kay Victor, dated October 2019.

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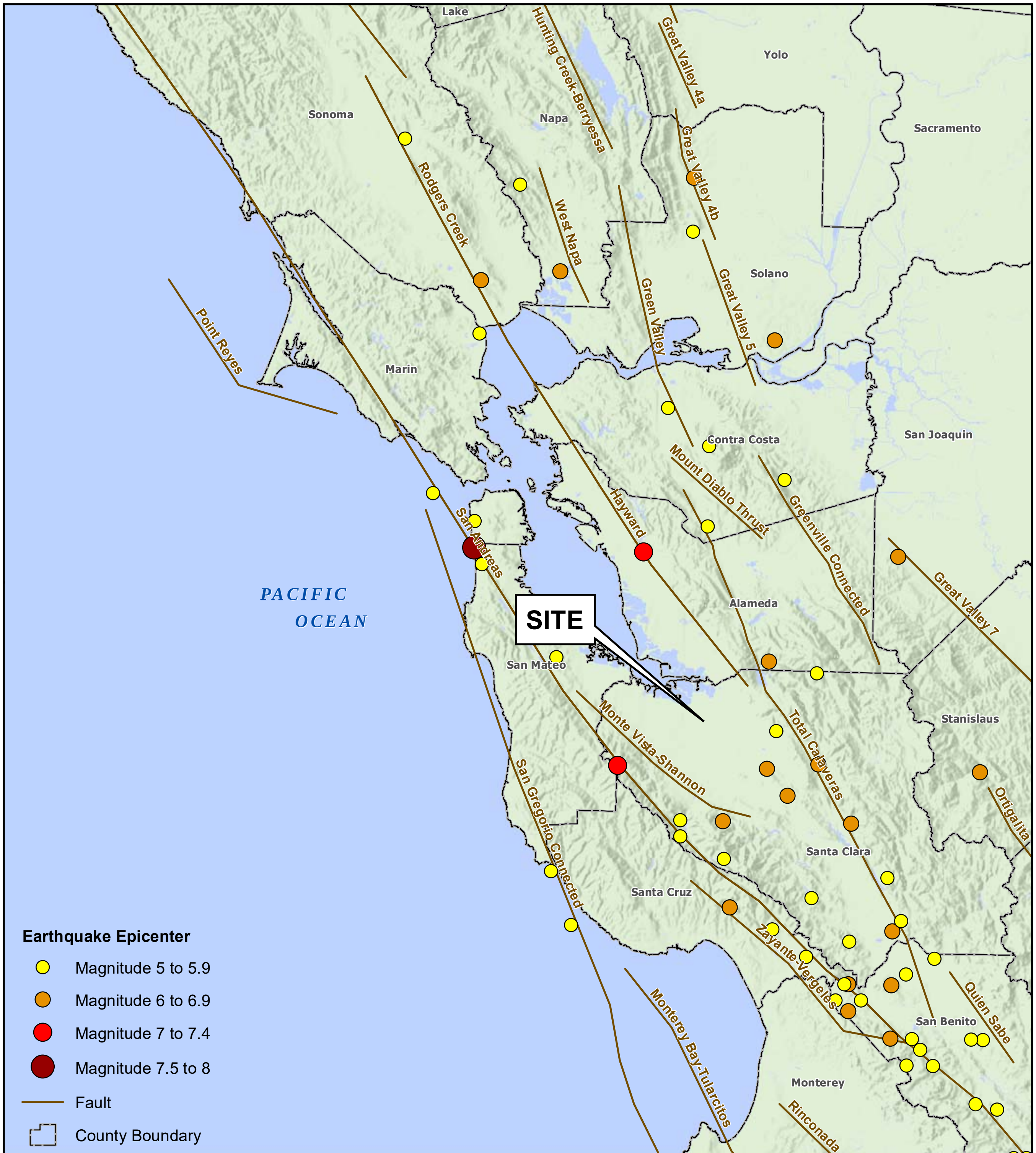
NORTH TOWN
San Jose, California

SITE PLAN

Date 11/14/19	Project No. 770651903	Figure 2

LANGAN



Notes:

1. Quaternary fault data displayed are based on a generalized version of U.S Geological Survey (USGS) Quaternary Fault and fold database, 2010. For cartographic purposes only.
2. The Earthquake Epicenter (Magnitude) data is provided by the USGS and is current through 08/26/2014.
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.

NORTH TOWN San Jose, California	**MAP OF MAJOR FAULTS AND EARTHQUAKE EPICENTERS IN THE SAN FRANCISCO BAY AREA**
LANGAN	Date 05/20/19 · Project No. 770651903 · 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.

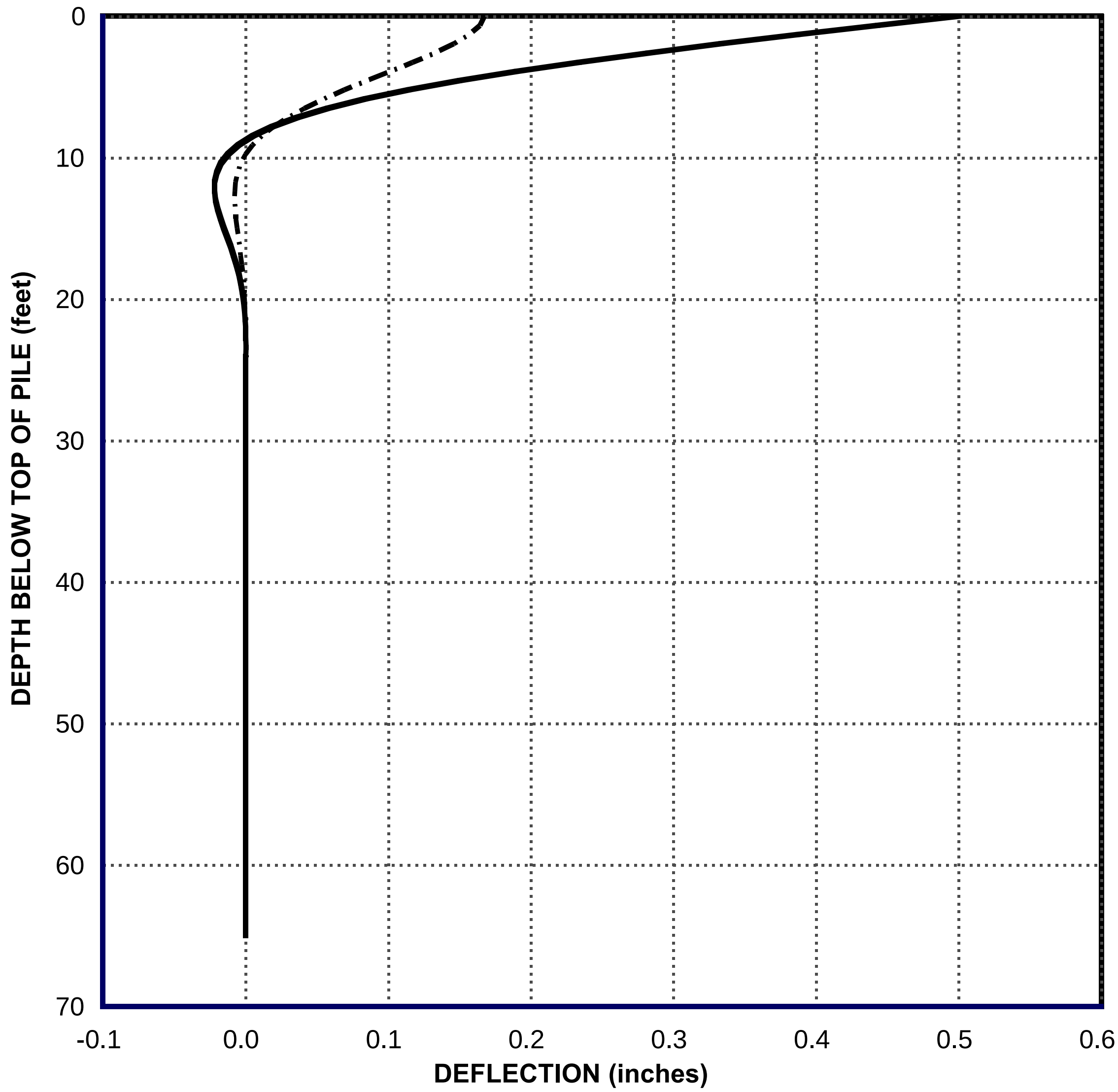
IX **Panic is general.**
Ground cracks conspicuously. Damage is considerable in masonry structures built especially to withstand earthquakes; great in other masonry buildings - some collapse in large part. Some wood frame houses built especially to withstand earthquakes are thrown out of plumb, others 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.

NORTH TOWN San Jose, California	**MODIFIED MERCALLI INTENSITY SCALE**
LANGAN	Date 05/20/19 | Project No. 770651903 | Figure 4

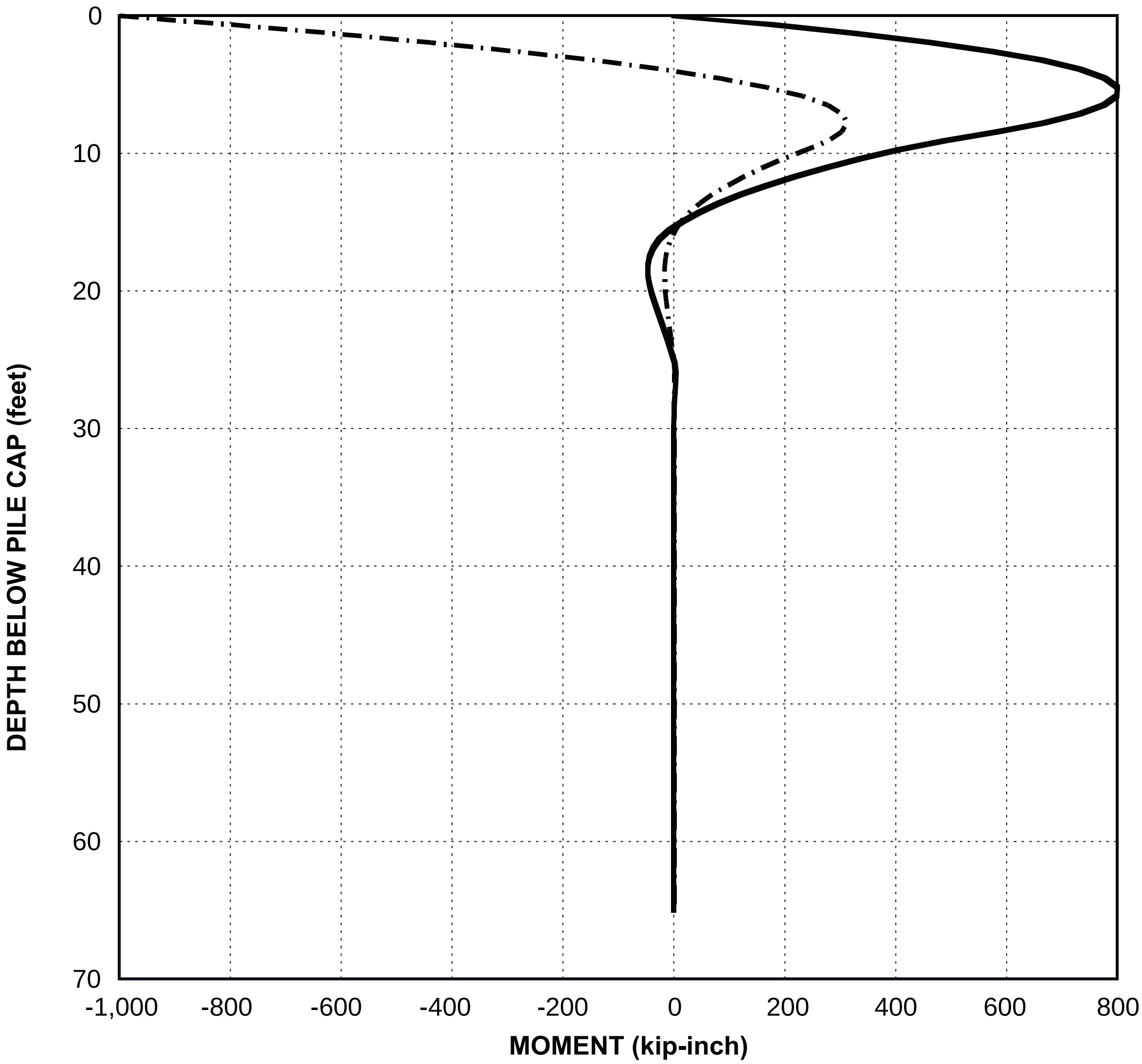


	Pile Diameter (inches)	Pile Head Connection	Top of Pile Deflection (inches)	Lateral Load (kips)
——— (solid line)	16	Free	0.5	21.5
—·—·— (dash-dot line)	16	Fixed	0.5	26.5

Notes:

1. Assumes there are no additionally applied moments at the pile head.
2. Passive resistance of pile caps has not been included.
3. The profiles shown are for single 16-inch diameter Auger Cast Pile (ACP) with axial compression loads of 250 kips.
4. To account for group effect, the lateral load capacity of the pile group should be multiplied by the factors shown on Table 10 of the geotechnical investigation report; however, moment profile used to check individual piles in a group should be for the unfactored load.
5. Lateral deflection at top of pile for fixed head conditions is limited by an assumed maximum moment capacity of approximately 1,000 kip-inch.

NORTH TOWN San Jose, California	**PRELIMINARY DEFLECTION PROFILES FOR AUGER CAST PILES**
LANGAN	Date 05/30/19 | Project No. 770651903 | Figure 5



Symbol	Pile Diameter (inches)	Pile Head Connection	Top of Pile Deflection (inch)	Lateral Load (kips)
———	16	Free	0.5	21.5
— · — · — ·	16	Fixed	0.5	26.5

Notes:

1. Assumes there are no additionally applied moments at the pile head.
2. Passive resistance of pile caps has not been included.
3. The profiles shown are for single 16-inch diameter Auger Cast Pile (ACP) with axial compression loads of 250 kips.
4. To account for group effect, the lateral load capacity of the pile group should be multiplied by the factors shown on Table 10 of the geotechnical investigation report; however, moment profile used to check individual piles in a group should be for the unfactored load.
5. Lateral deflection at top of pile for fixed head conditions is limited by an assumed maximum moment capacity of approximately 1,000 kip-inch.

NORTH TOWN
San Jose, California

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PRELIMINARY BENDING MOMENT PROFILES FOR AUGER CAST PILES

Date 05/30/19 | Project No. 770651903 | Figure 6

APPENDIX A

LOGS OF TEST BORING

PROJECT:	**NORTH TOWN** **San Jose, California**	**Log of Boring B-1**	PAGE 1 OF 2

Boring location:	See Site Plan, Figure 2	Logged by:	T. Toledo
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)		

MATERIAL DESCRIPTION — Ground Surface Elevation: 26.1 feet[2]

DEPTH (feet)	Sampler Type	Blows/6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION
0					2.5 inches asphalt concrete (AC)
1				CH	CLAY (CH) dark brown, very stiff, moist
3	BULK			CH	LL = 77, PL = 23, PI = 54, see Figure C-7
6	S&H	6 17 26	22	CH	brown, trace fine sand
9	S&H	8 11 15	13	CH	light brown
10				CL	SANDY CLAY (CL) olive with gray mottling, stiff, moist, fine sand
13				CL	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	CL	
19	S&H	13 26 18	22	CL	
21	SPT	13 18 20	34	CL	hard
21				SM	SILTY SAND (SM) gray-brown, dense, wet, fine-grained
24	S&H	18 30 42	36	SM	gray
28				SP	SAND (SP) gray-brown, very dense, wet, fine-grained
29	SPT	18 27 38	59	SP	

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

LANGAN

Project No.: 770651903 | Figure: A-1a

PROJECT: **NORTH TOWN**
San Jose, California

Log of Boring B-1

DEPTH (feet)	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
						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												
51	SPT		18 12 14	23		medium dense, fine- to medium-grained						
						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 based on -- datum.

LANGAN

Project No.: 770651903

Figure: A-1b

PROJECT:	NORTH TOWN San Jose, California	Log of Boring B-2
		PAGE 1 OF 3

Boring location: See Site Plan, Figure 2
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)

Logged by: T. Toledo

LABORATORY TEST DATA

Ground Surface Elevation: 26.3 feet[2]

DEPTH (feet)	Sampler Type	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
0					2.5 inches asphalt concrete (AC)						
0–8				CH	CLAY (CH) dark brown, stiff, moist						
5–6.5	S&H	14, 9, 11	14	CH	dark brown with gray						
8.5–12				CL	SANDY CLAY (CL) brown, very soft to soft, wet, fine sand, trace fine gravel						
10–11.5	S&H	1, 1, 2	2	CL							
12–13	SPT	3, 6, 9	18	SC	CLAYEY SAND (SC) olive-gray, medium dense, wet, trace fine gravel						
13–16	SPT	10, 8, 12	24	SP-SC	SAND with CLAY (SP-SC) brown, medium dense, wet, fine- to coarse-grained, trace subrounded gravel						
16–23				SP	SAND with GRAVEL (SP) yellow-brown, dense, wet, fine- to coarse-grained, fine to coarse subrounded gravel						
20–21.5	SPT	10, 16, 15	37	SP							
23–30				SP-SC	SAND with CLAY and GRAVEL (SP-SC) brown, medium dense, wet						
25–26.5	SPT	16, 13, 11	29	SP-SC					5.5	9.1	

LANGAN

Project No.: 770651903
Figure: A-2a

TEST GEOTECH LOG 770651903_NORTH TOWN.GPJ TR.GDT 6/26/19

DEPTH (feet)	Sampler Type	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
31	SPT	4 9 11	24	SP	SAND with CLAY and GRAVEL (SP-SC) (continued) SAND (SP) brown, medium dense, wet, trace fine gravel						
35–36	SPT	9 7 8	18	SM	SILTY SAND (SM) gray, medium dense, wet, fine-grained, trace wood fragments				20.1	21.2	
39–41	ST	300 psi		CL	CLAY (CL) gray, wet						
42–43	SPT	19 35 50/ 5"	102/ 11"	SP	CLAYEY SAND with GRAVEL (SC) yellow-brown, very dense, fine- to coarse-grained, fine gravel						
47–48				CL	CLAY (CL) brown, wet						
49–51	SPT	16 20 19	47	SP	SAND (SP) brown, dense, fine- to coarse-grained, trace fine subrounded to subangular gravel						
52–60	S&H	6 9 14	16	CL	CLAY with SAND (CL) yellow-brown, very stiff, wet, fine sand Consolidation Test, see Figure C-1					23.4	102

PROJECT:	NORTH TOWN San Jose, California	Log of Boring B-2 PAGE 3 OF 3

DEPTH (feet)	Sampler Type	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
61	S&H	25 30 29	41	SP	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											
66	SPT	22 25 31	67								
67											
68											
69											
70				SM	SILTY SAND (SM) brown, dense, wet, fine-grained						
71	SPT	10 12 24	43								
72											
73											
74				SP	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											
78											
79											
80				SP	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								
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 based on -- datum.

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Project No.: 770651903 | Figure: A-2c

PROJECT:	NORTH TOWN San Jose, California	**Log of Boring B-3** PAGE 1 OF 3

Boring location:	See Site Plan, Figure 2
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)
Logged by:	C. Leege

Ground Surface Elevation: 27.1 feet[2]

DEPTH (feet)	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
0						2.5 inches asphalt concrete (AC)						
1–2					CH	CLAY (CH) dark brown, moist						
2–3.5					SM	SILTY SAND with GRAVEL (SM) olive-gray, moist, fine-grained, coarse subrounded to subangular gravel						
3.5–10	S&H (5–6.5)		3 6 8	10	CH	CLAY (CH) dark brown, stiff, moist						
10–14	S&H (10–11.5)		2 2 5	5	CL	SANDY CLAY (CL) olive, medium stiff, wet, fine sand						
14–18.5	SPT (15–16.5)		5 4 5	11	SC	CLAYEY SAND (SC) olive-gray, medium dense, wet, fine-grained, trace fine gravel LL = 28, PL = 18, PI = 10, see Figure C-7 grades with increase gravel content				21.3	14.1	
18.5–25	SPT (20–21.5)		8 10 11	25	SP	SAND (SP) yellow-brown, medium dense, wet, fine-grained, trace fine to coarse subrounded gravel, trace clay						
25–27	SPT (25–26.5)		12 8 8	19	SM	SILTY SAND (SM) yellow-brown, medium dense, wet, fine-grained						
27–30					CL	CLAY (CL) gray, stiff, wet						

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Project No.:	Figure:
770651903	A-3a

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
						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												

PROJECT: **NORTH TOWN**
San Jose, California

Log of Boring B-3

DEPTH (feet)	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
61	S&H		13 25 35	42	SP	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								
72												
73												
74												
75												
76	SPT		17 20 20	48		dense, trace clay						
77												
78												
79												
80												
81	SPT		19 20 23	52		very dense						
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 based on -- datum.

LANGAN

Project No.: 770651903

Figure: A-3c

PROJECT: **NORTH TOWN**
San Jose, California

Log of Boring B-4

Boring location: See Site Plan, Figure 2

Logged by: T. Toledo

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)

Ground Surface Elevation: 29 feet[2]

DEPTH (feet)	Sampler Type	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
0					2.5 inches asphalt concrete (AC)						
0–7				CH	CLAY (CH) dark brown, very stiff, moist						
5–6.5	S&H	8, 17, 24	21	CH							
7–13				CL	CLAY (CL) yellow-brown with orange, stiff, moist, trace fine to coarse sand						
8.5–10	S&H	12, 13, 13	13	CL	Consolidation Test, see Figure C-3					26.7	95
13–17				SC	CLAYEY SAND (SC) gray-brown with orange, medium dense, wet, fine-grained, trace fine gravel						
13.5–15	S&H	11, 15, 13	14	SC	▽ (03/07/19, 10:00 a.m.) at 14.5 feet: LL: = 26, PL = 18, PI = 8				40.5	21.5	
17–19				SM	SILTY SAND (SM) light gray, very dense, fine-grained, wet, trace fine to coarse gravel						
19–24				SP	SAND (SP) gray-brown, very dense, wet, fine- to coarse-grained, trace fine to coarse subangular gravel						
18.5–20	SPT	26, 40, 41	73	SP							
23.5–24.5	SPT	31, 50/ 6"	45/ 6"	SP							
24–28				GP	GRAVEL with SAND (GP) gray-brown, very dense, wet, fine- to coarse-grained, subrounded to subangular, fine to coarse sand						
28–31				SM	SILTY SAND (SM) light gray, dense, wet, fine-grained, trace fine subrounded gravel, some clay						
28.5–30	SPT	20, 18, 23	37	SM							

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Project No.: 770651903 | Figure: A-4a

PROJECT:	NORTH TOWN San Jose, California	Log of Boring B-4

DEPTH (feet)	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
32–34					SM	SILTY SAND (SM) (continued)						
34–35	SPT		7, 8, 10	16	SM							
35–38					CL	CLAY (CL) brown, very stiff, wet						
38–45					CL	CLAY (CL) yellow-brown with gray, very stiff, wet, trace coarse sand						
39–40	S&H		11, 15, 22	19	CL							
44–45	S&H		12, 20, 27	24	CL							
45–53					SM	SILTY SAND (SM) gray, dense, wet, fine-grained						
49–50	SPT		12, 20, 25	41	SM	red-yellow to light brown						
53–60					SP	SAND (SP) yellow-brown, very dense, wet, fine-grained						
53–54	SPT		17, 31, 42	66	SP							
59–60	SPT		12, 18, 22	36	SP	dense						

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 based on -- datum.

LANGAN

Project No.: 770651903

Figure: A-4b

PROJECT:	NORTH TOWN San Jose, California	Log of Boring B-5 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)
Logged by:	T. Toledo

LABORATORY TEST DATA

Ground Surface Elevation: 27.1 feet[2]

DEPTH (feet)	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
0						2.5 inches asphalt concrete (AC)						
1					CH	CLAY (CH) dark brown, very stiff, moist						
2–4	BULK				CH	LL = 78, PL = 21, PI = 57, see Figure C-7 R-value Test, see Figure C-8						
5–6	S&H		13 22 32	27	CH							
8–9	S&H		11 15 17	16	CL	SANDY CLAY (CL) yellow-brown, very stiff, moist, fine to coarse sand						
12–14	SPT		7 11 13	22	SC	CLAYEY SAND (SC) yellow-brown to gray-brown, medium dense, wet, fine-grained (03/07/19)				31.2	22.4	
18–19	SPT		9 11 20	28	CL	CLAY (CL) yellow-brown, very stiff, wet						
24	SPT		7 12 19	28	CL	gray, trace wood fragments						
28–29	S&H		12 15 23	19	SM	SILTY SAND (SM) gray-brown, medium dense, wet, fine-grained						
31												

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Project No.:	Figure:
770651903	A-5a

DEPTH (feet)	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
					SM	SILTY SAND (SM) (continued)						
32–43					CL	CLAY (CL) yellow-brown, hard, wet						
34	SPT		12 17 22	35								
39	S&H		11 13 26	20		yellow-brown with gray-brown, very stiff						
43.5–48.5					SM	SILTY SAND (SM) yellow-brown, medium dense, wet, fine-grained						
44	S&H		13 21 26	24								
48.5–52					SP	SAND (SP) brown, very dense, wet, fine-grained						
49	SPT		17 50/ 5"	45/ 5"								
52–58					CL	SANDY CLAY (CL) yellow-brown, hard, wet, fine- to medium-grained sand						
54	SPT		14 26 37	57								
58–60					CL	CLAY (CL) gray, very stiff, wet						
59	S&H		14 20 27	24								

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 based on -- datum.

PROJECT:	NORTH TOWN San Jose, California

Log of Boring B-6

Boring location: See Site Plan, Figure 2

Logged by: T. Toledo

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)

Ground Surface Elevation: 26.9 feet[2]

DEPTH (feet)	Sampler Type	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
0					2.5 inches asphalt concrete (AC)						
0.5				CH	CLAY (CH) dark brown, very stiff, moist						
1–4	BULK				R-value Test, see Figure C-8						
5–6.5	S&H	11 20 34	27		brown						
8.5				CL	SANDY CLAY (CL) yellow-brown, soft, moist, fine- to coarse-grained sand						
8.5–10	S&H	6 6 6	6		▽ (03/06/19, 3:30 p.m.)	TxUU	1,100	360		19.3	108
13				CL	CLAY with SAND (CL) gray-brown, stiff to very stiff, wet, with fine sand						
13.5–15	S&H	11 13 17	15								
18				SP	SAND (SP) yellow-brown, medium dense, wet, fine-grained						
18.5–20	S&H	11 10 27	19								
20–21.5	SPT	16 17 19	32		brown, dense						
21				SC	CLAYEY SAND (SC) gray, dense, wet, fine-grained						
22				CL	CLAY (CL) gray, hard, wet, trace fine sand						
23.5–25	SPT	13 18 32	36								
28				SM	SILTY SAND (SM) gray-brown, medium dense, wet, trace fine subrounded gravel						
28.5–30	S&H	25 22 26	24								

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Project No.: 770651903 | Figure: A-6a

DEPTH (feet)	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
31					CL	CLAY (CL) gray with yellow-brown mottling, hard, wet, trace fine sand						
34	S&H		16 29 32	31								
39	S&H		9 13 14	14		yellow-brown, stiff						
41	ST			200 psi								
44	S&H		14 12 30	21								
45					SM	SILTY SAND (SM) yellow-brown, dense, wet, fine-grained						

Depth scale: 31, 32, 33, 34, 35, 36, 37, 38, 39, 40, 41, 42, 43, 44, 45, 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 based on -- datum.

LANGAN

Project No.: 770651903
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

NORTH TOWN
San Jose, California

LANGAN

CLASSIFICATION CHART

APPENDIX B

CONE PENETRATION TESTS

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

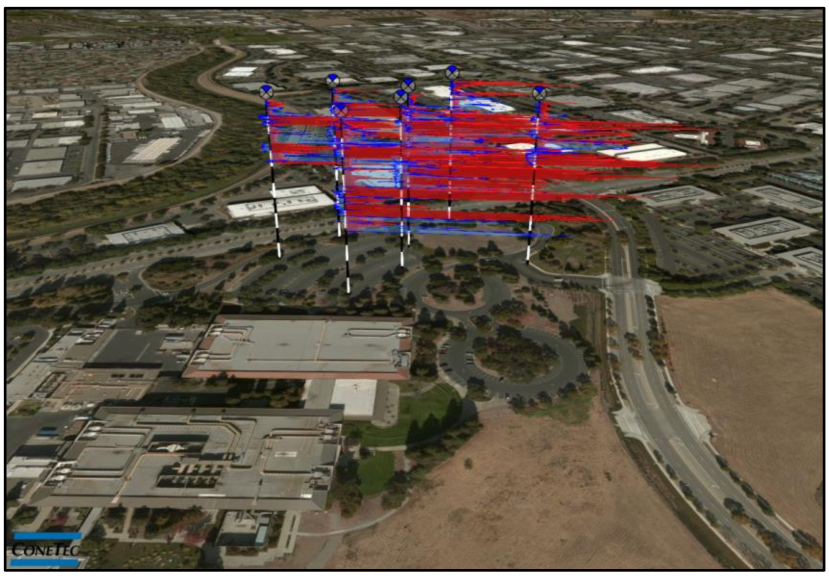
North Town

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:

ConeTec Inc.
820 Aladdin Avenue
San Leandro, CA 94577

Tel: (510) 357-3677

Email: ConeTecCA@conetec.com
www.conetec.com
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.

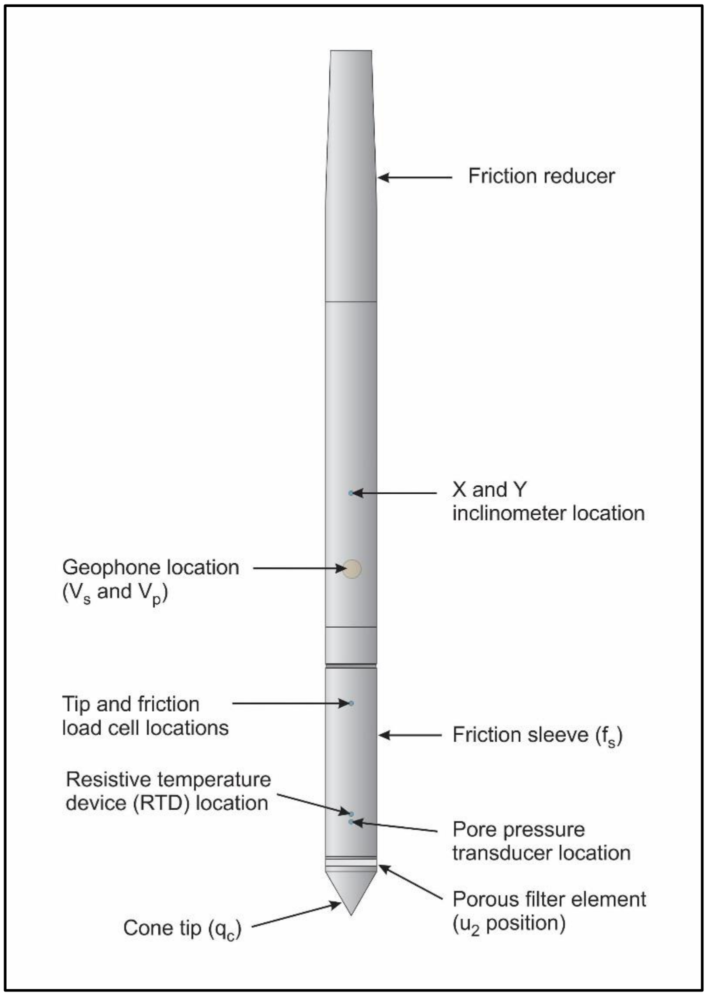


Figure CPTu. Piezocone Penetrometer (15 cm^2)

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).

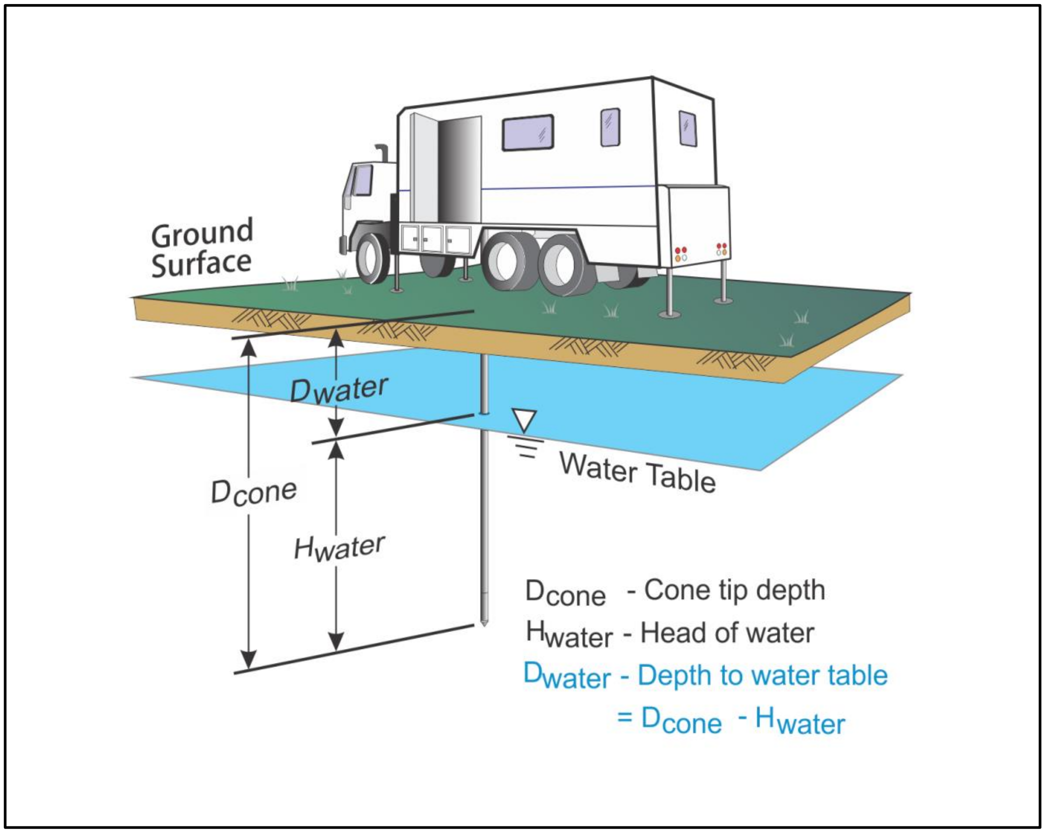


Figure PPD-1. Pore pressure dissipation test setup

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.

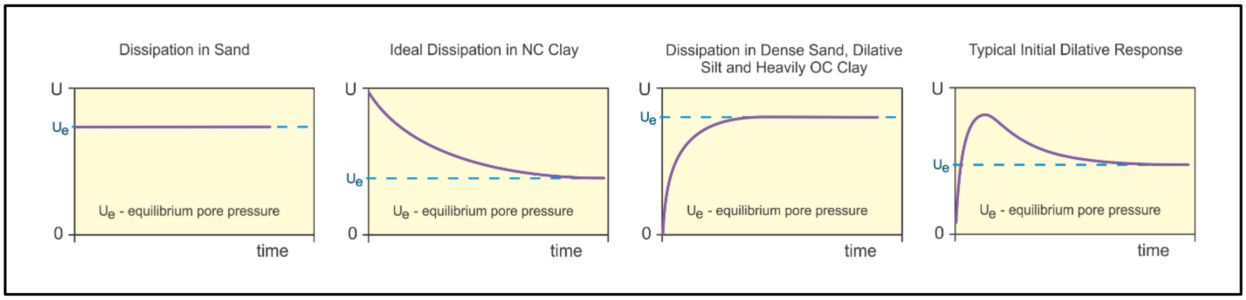


Figure PPD-2. Pore pressure dissipation curve examples

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.

Burns, S.E. and Mayne, P.W., 2002, “Analytical cavity expansion-critical state model cone dissipation in fine-grained soils”, Soils & Foundations, Vol. 42(2): 131-137.

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	

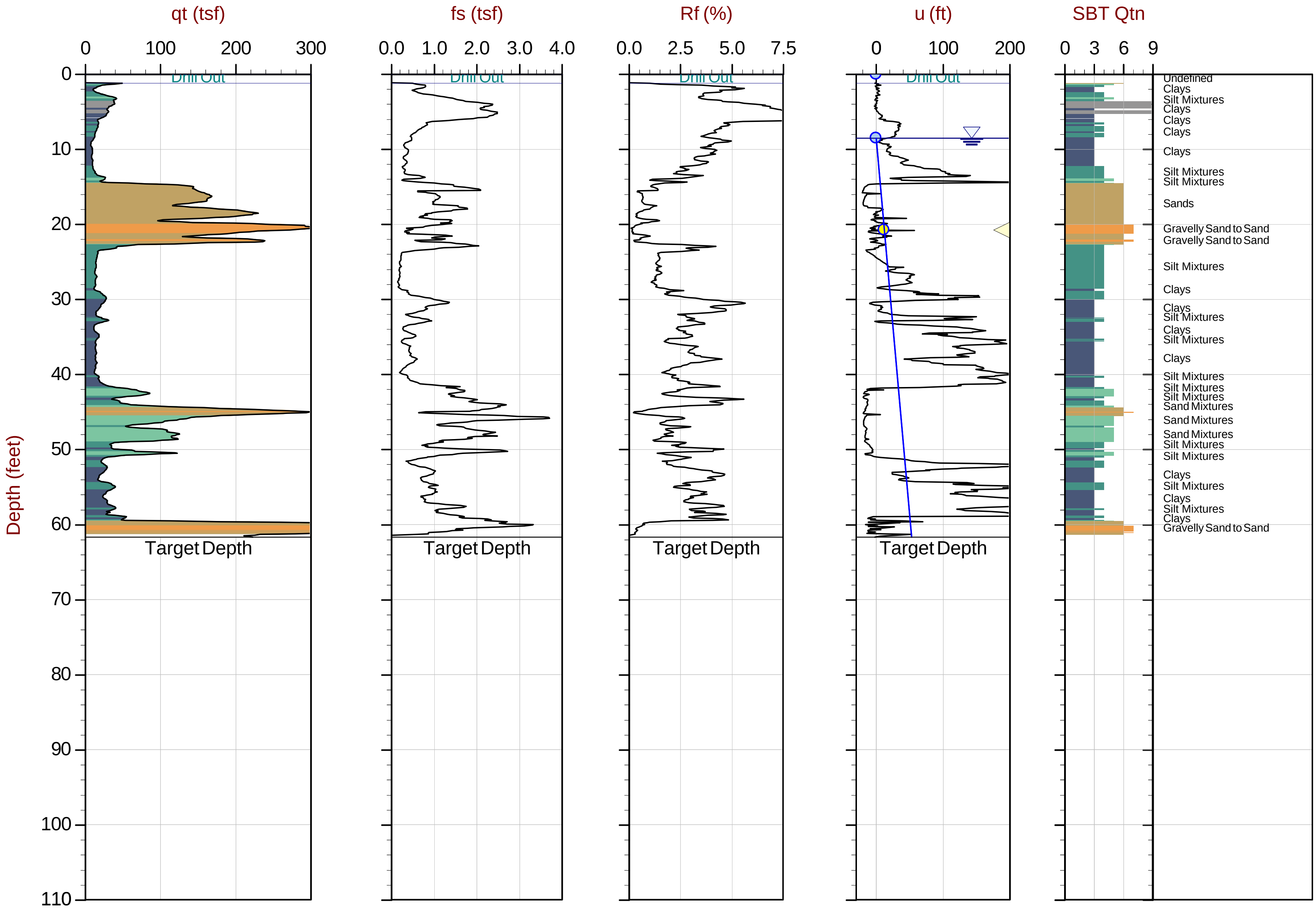
1. The assumed phreatic surface was based on pore pressure dissipation tests, unless otherwise noted. Hydrostatic conditions were assumed for the calculated parameters.
2. The coordinates were acquired using consumer grade GPS equipment in datum: WGS84 / UTM Zone 10 North.

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

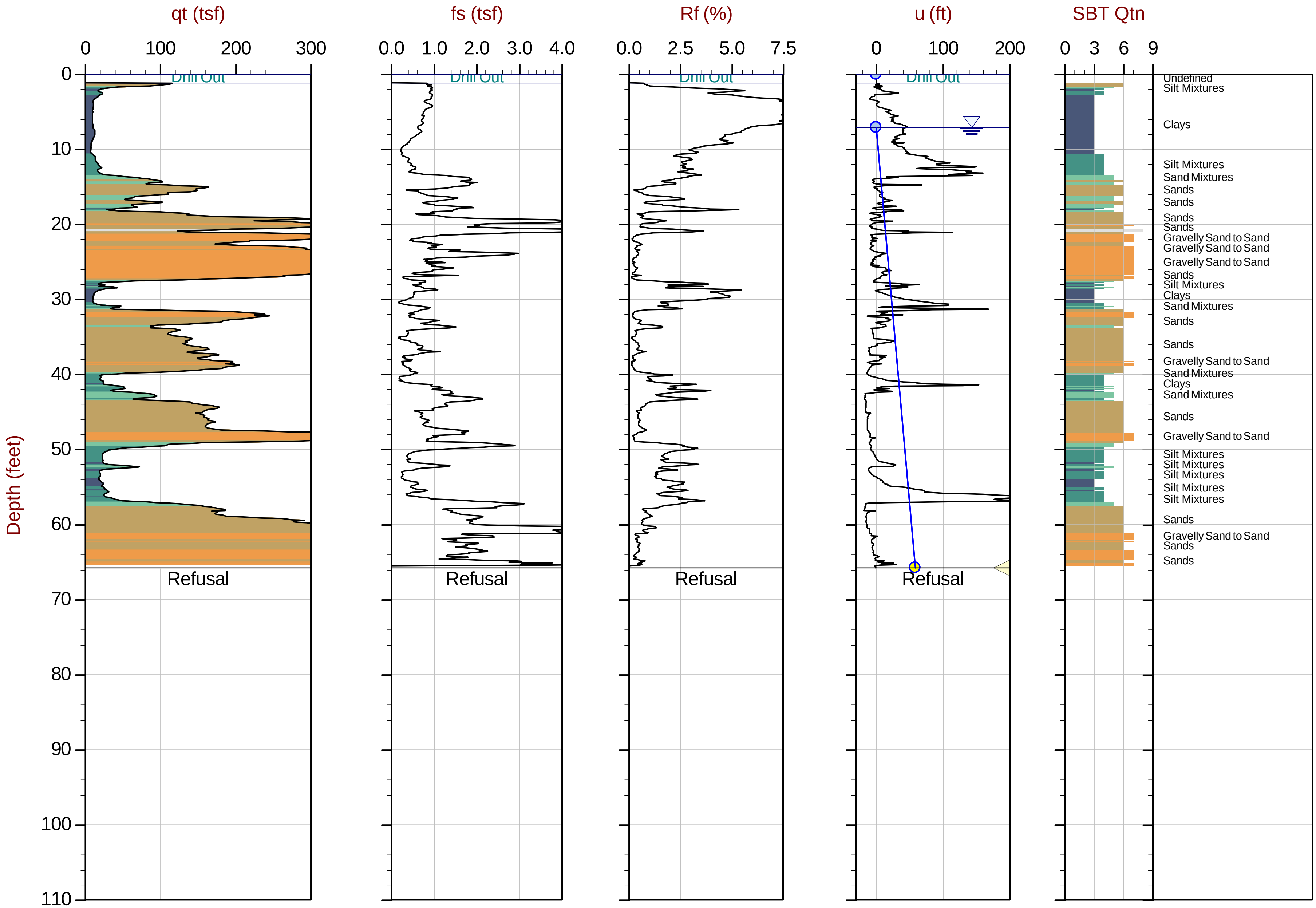
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

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)

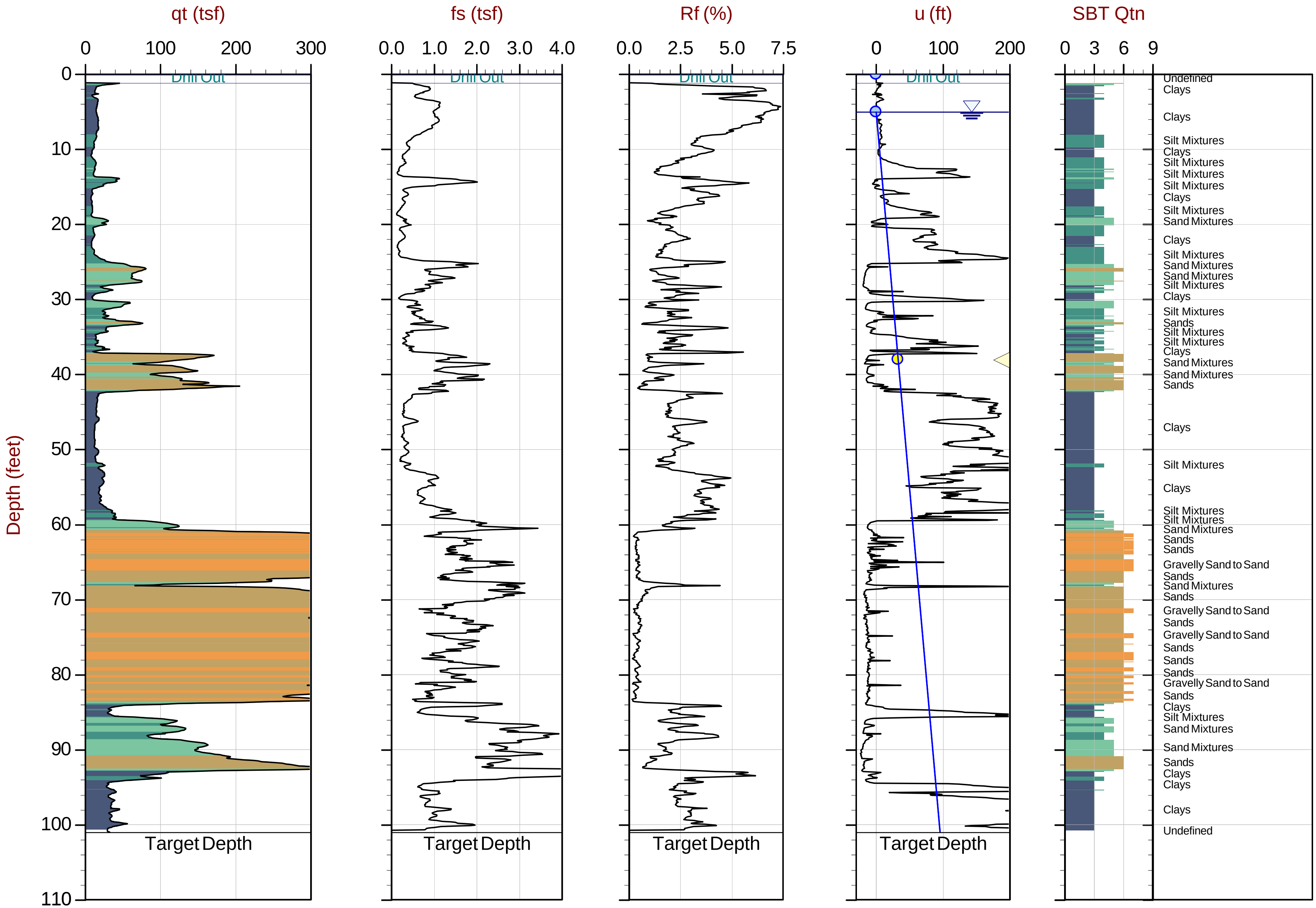
SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4137883m E: 594393m
Sheet No: 1 of 1

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.

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

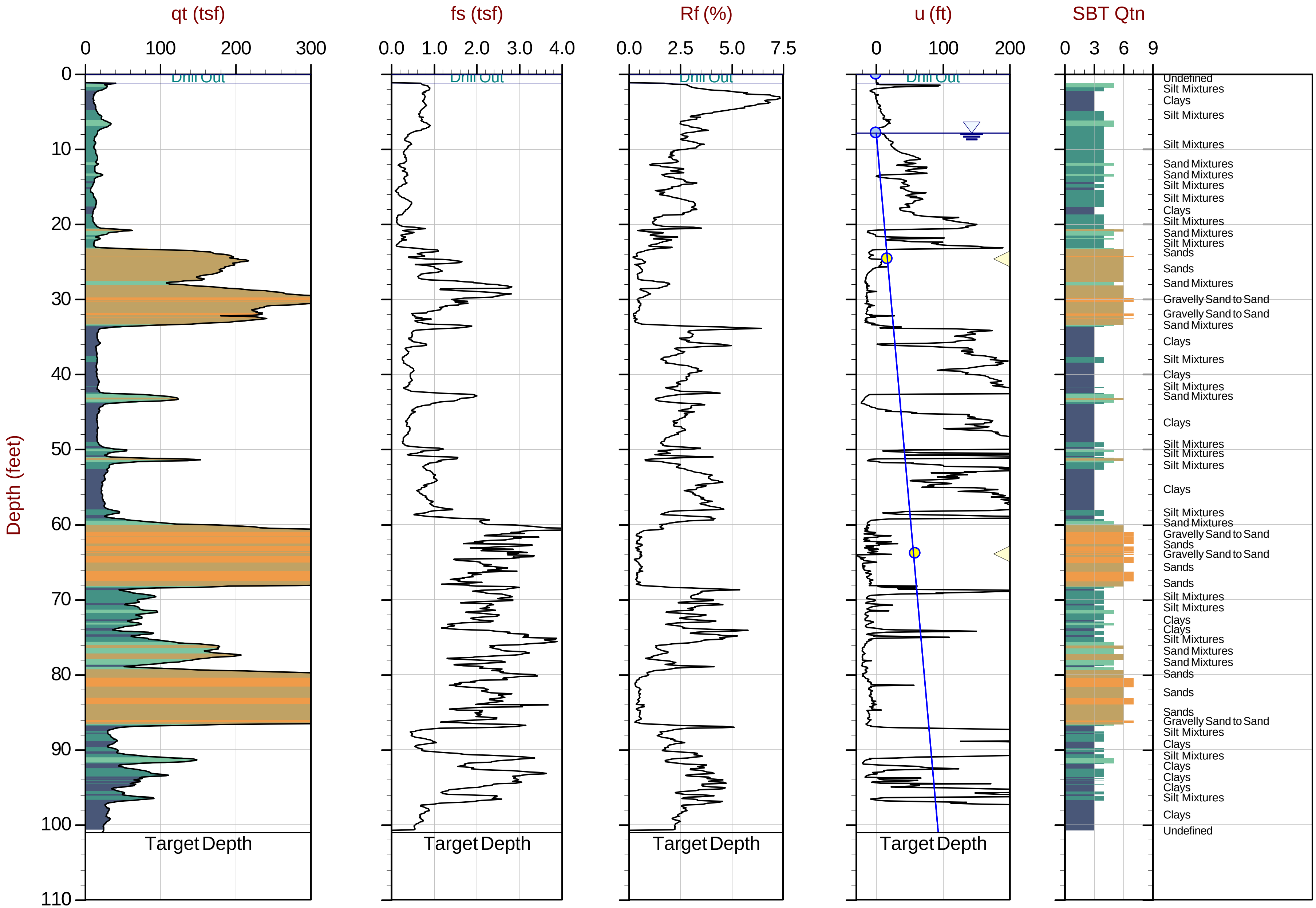
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

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

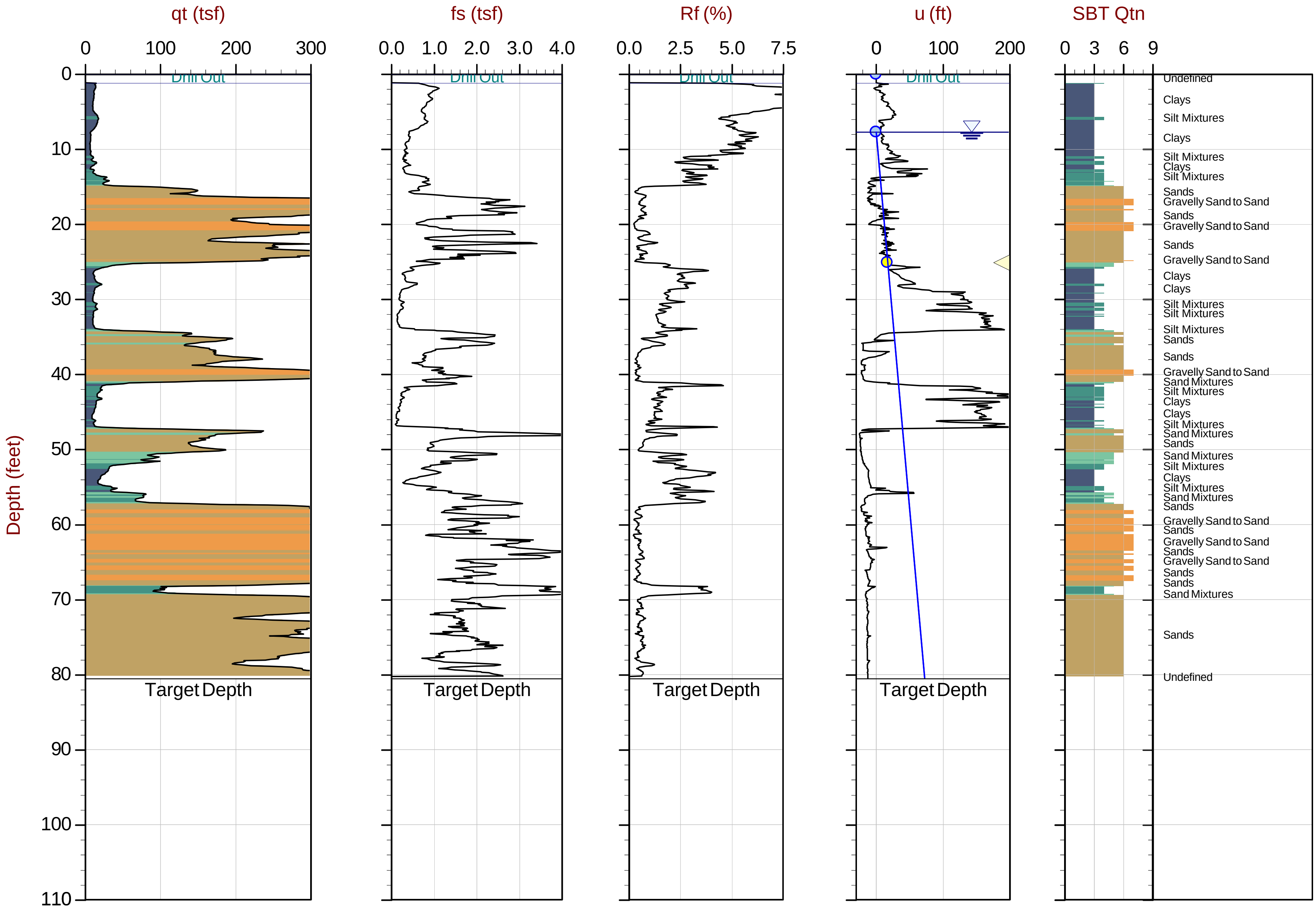
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

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

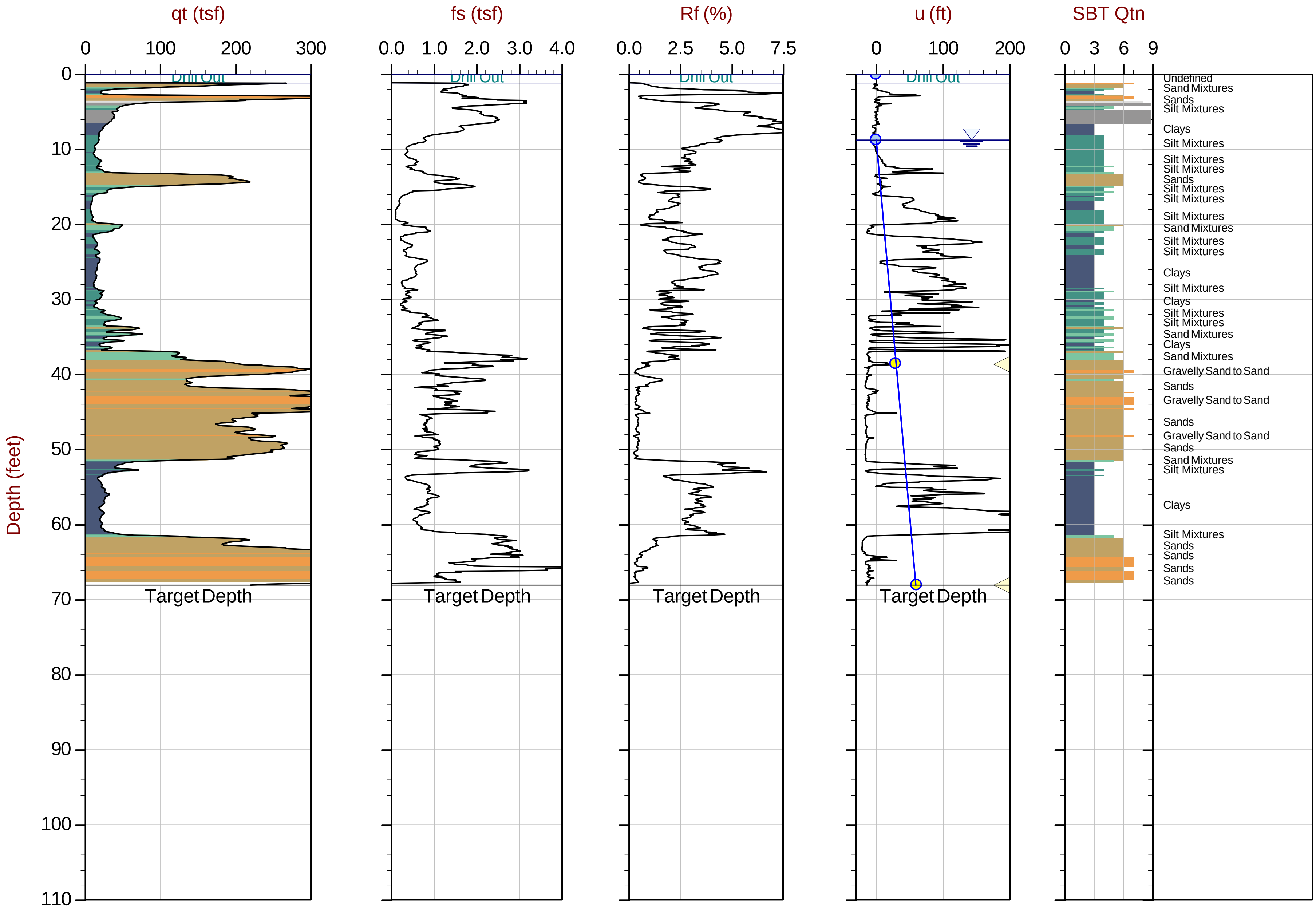
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

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

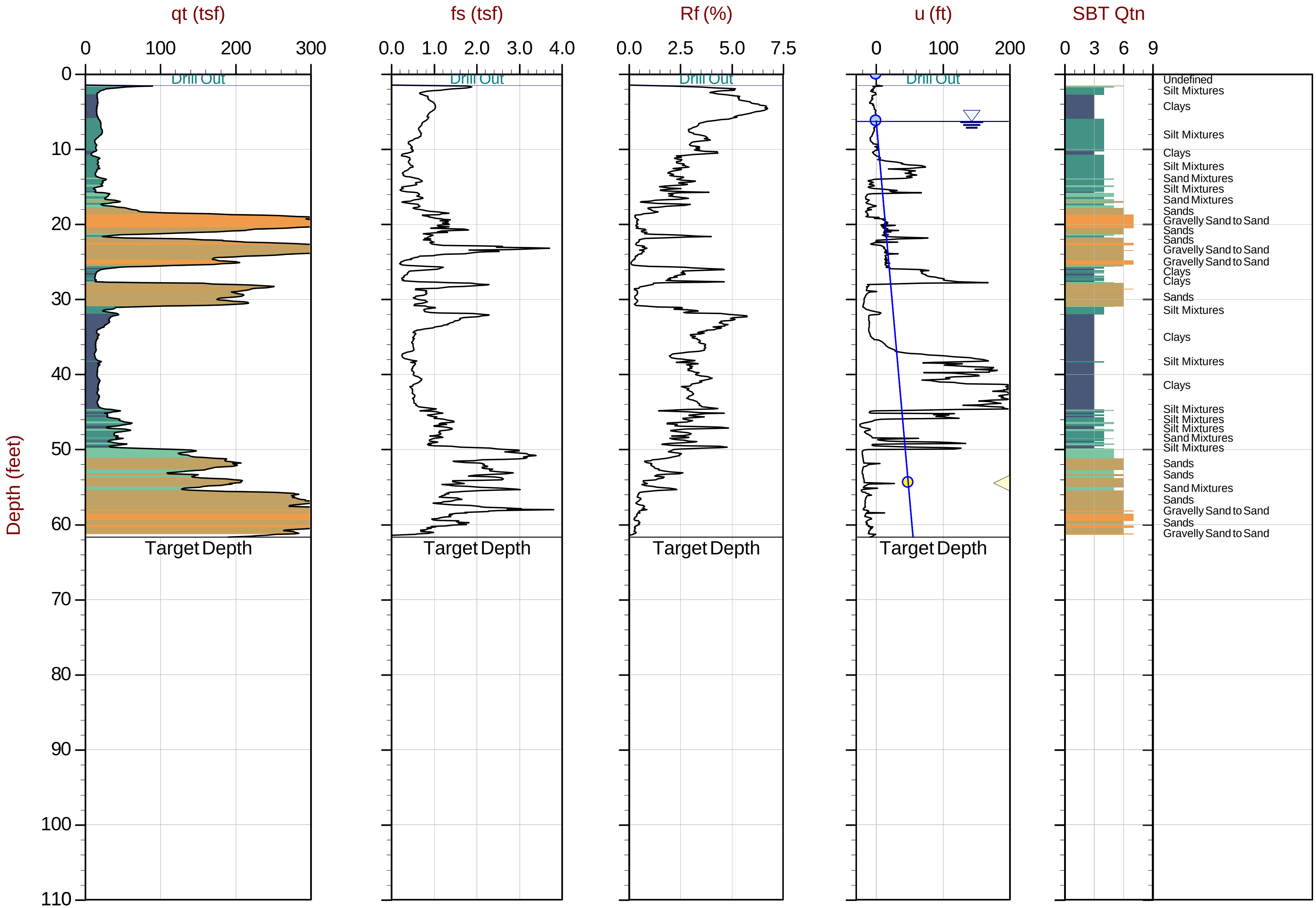
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

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.

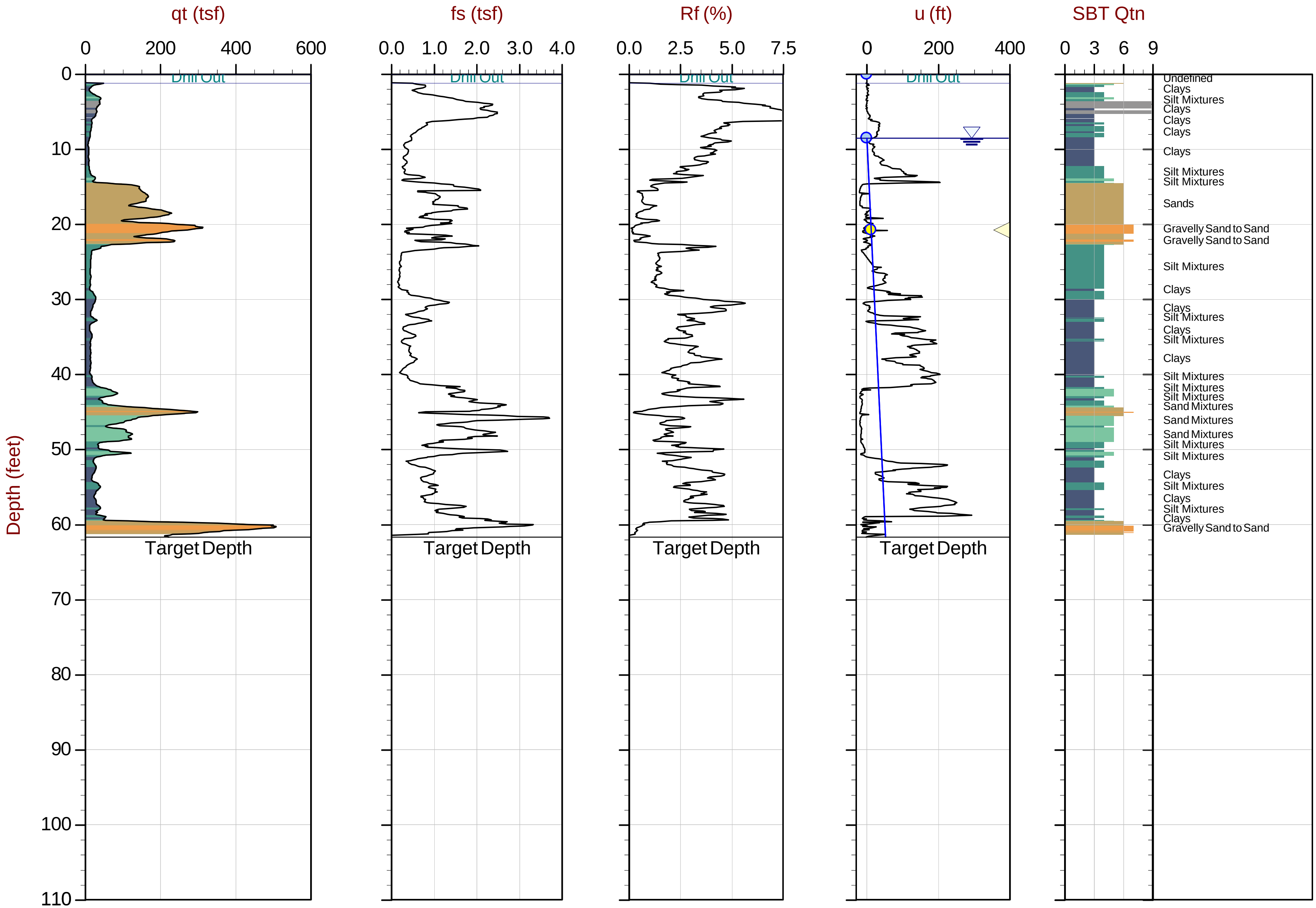
Standard Cone Penetration Test Plots with Expanded Scales

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

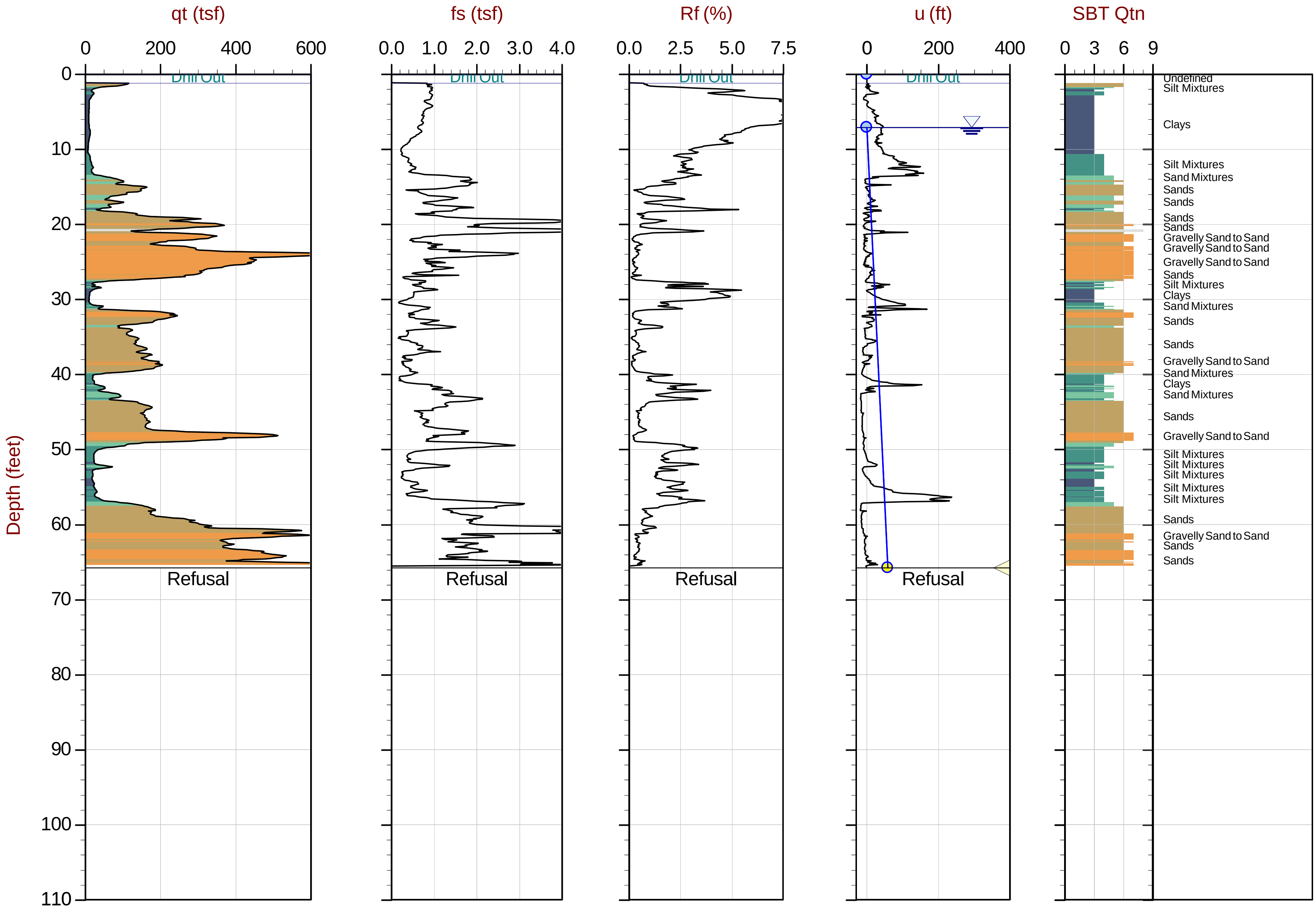
File: 19-56026_CP01.COR
Unit Wt: SBTQtn (PKR2009)

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	Sounding: CPT-02
	Date: 2019-03-04 09:00	Cone: 483:T1500F15U500
	Site: North Town	



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

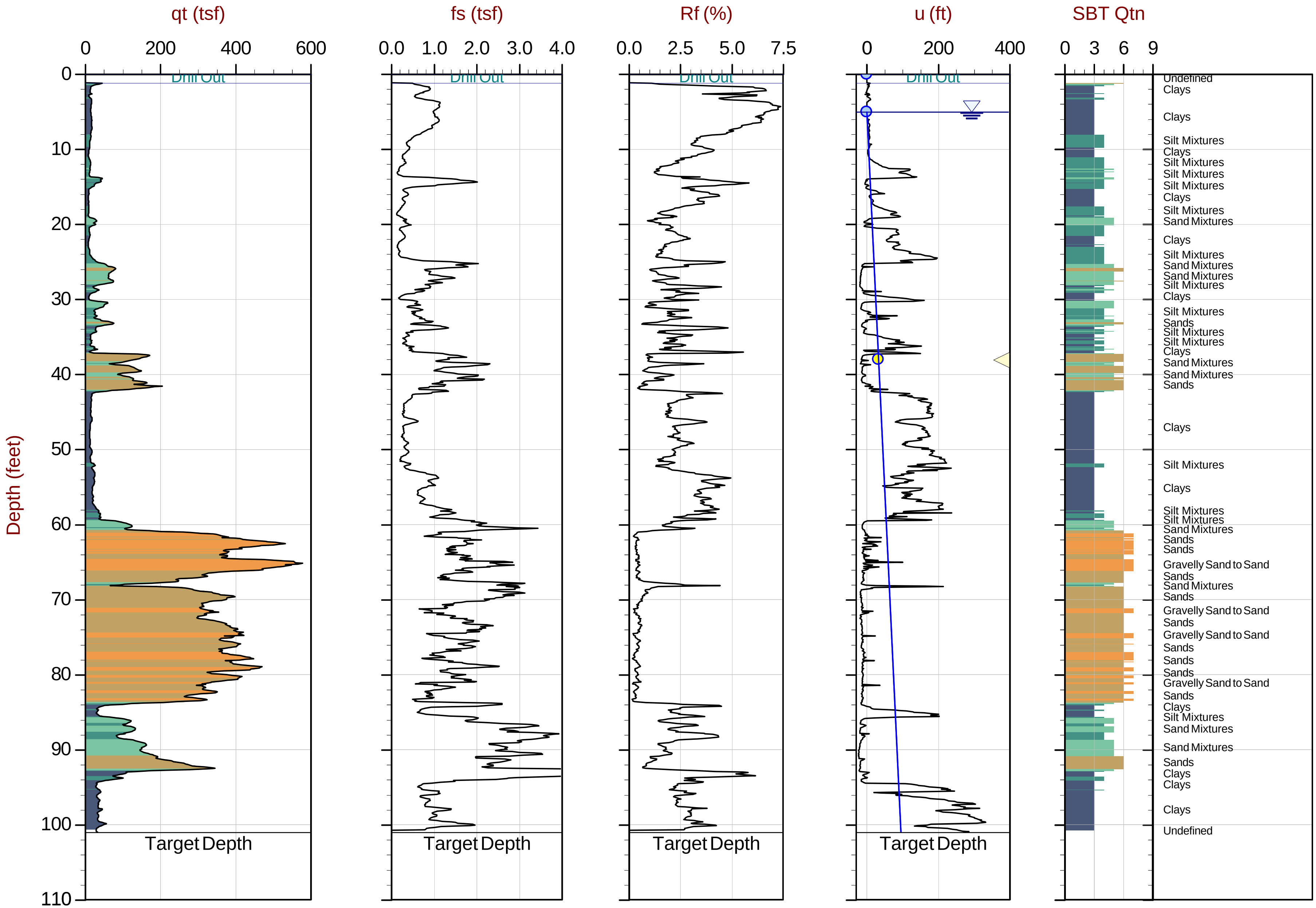
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

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

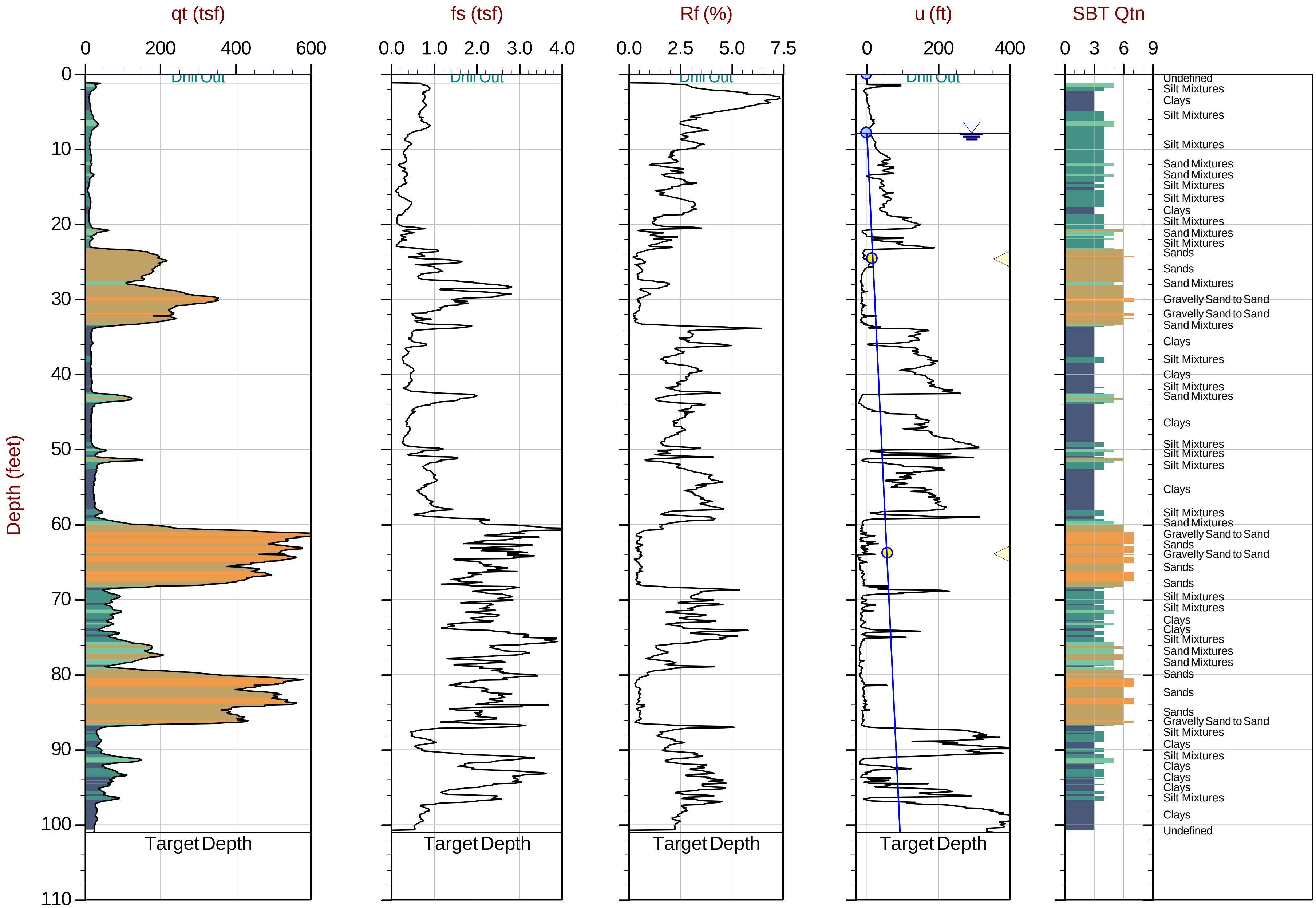
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

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

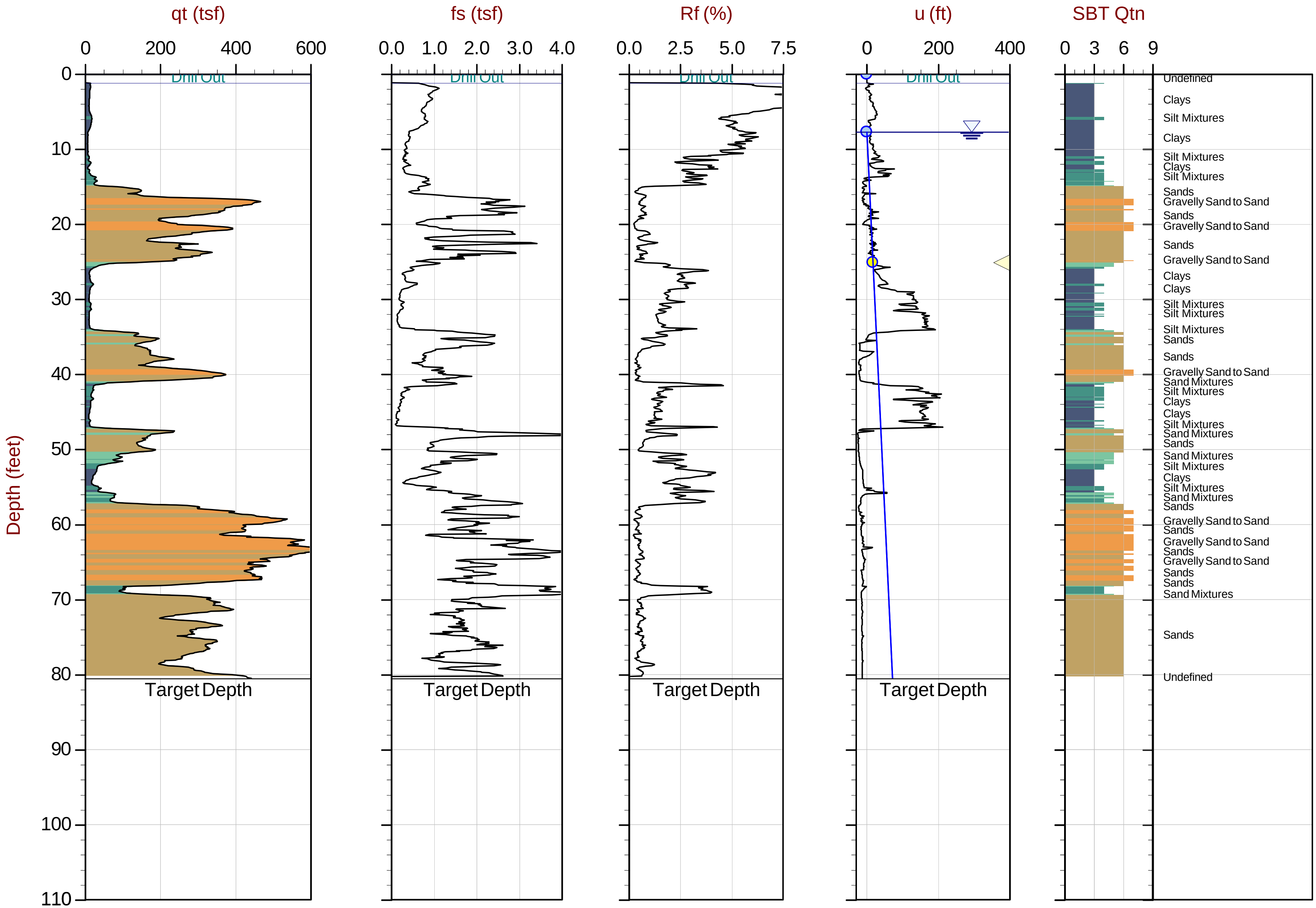
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

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

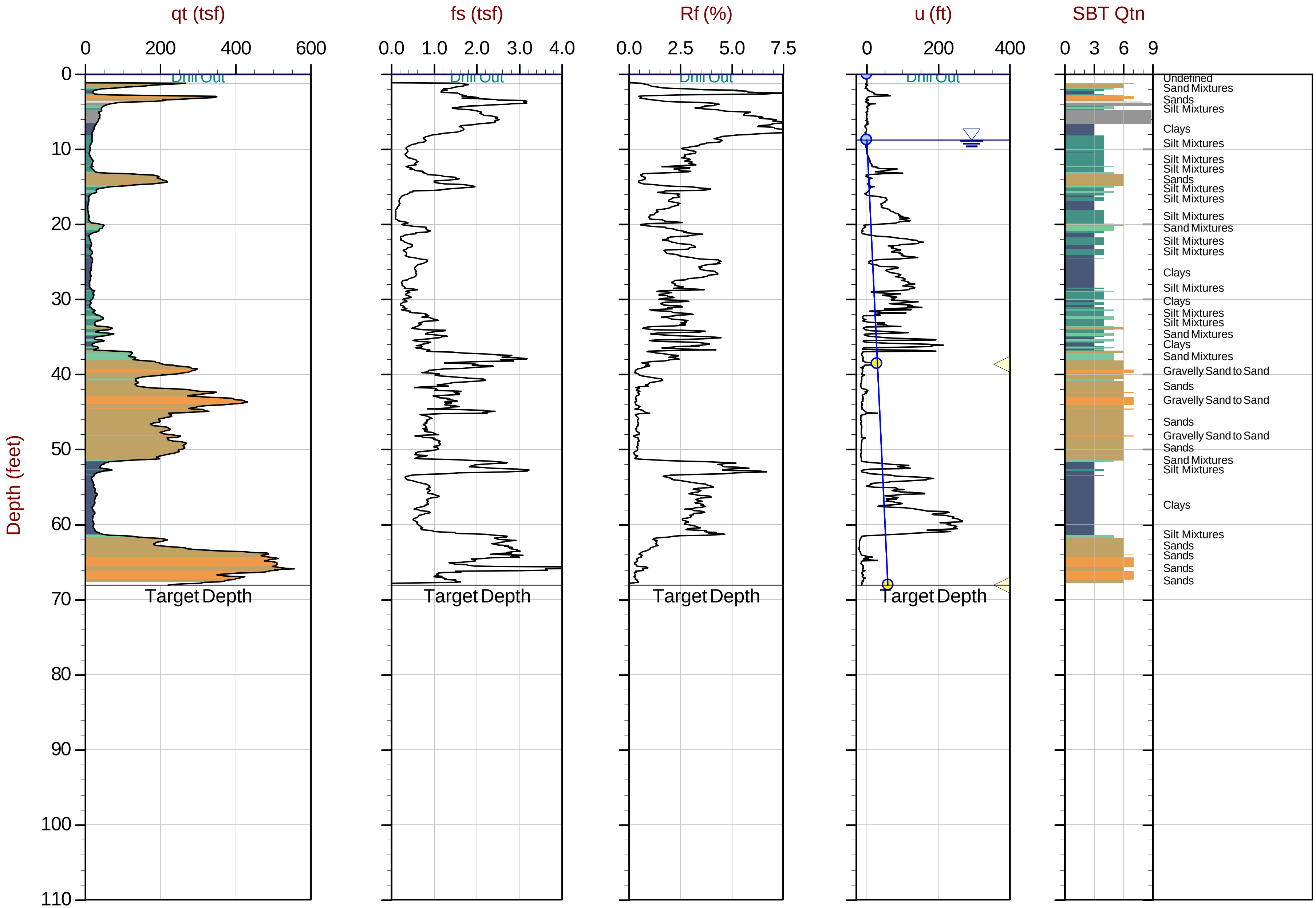
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

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

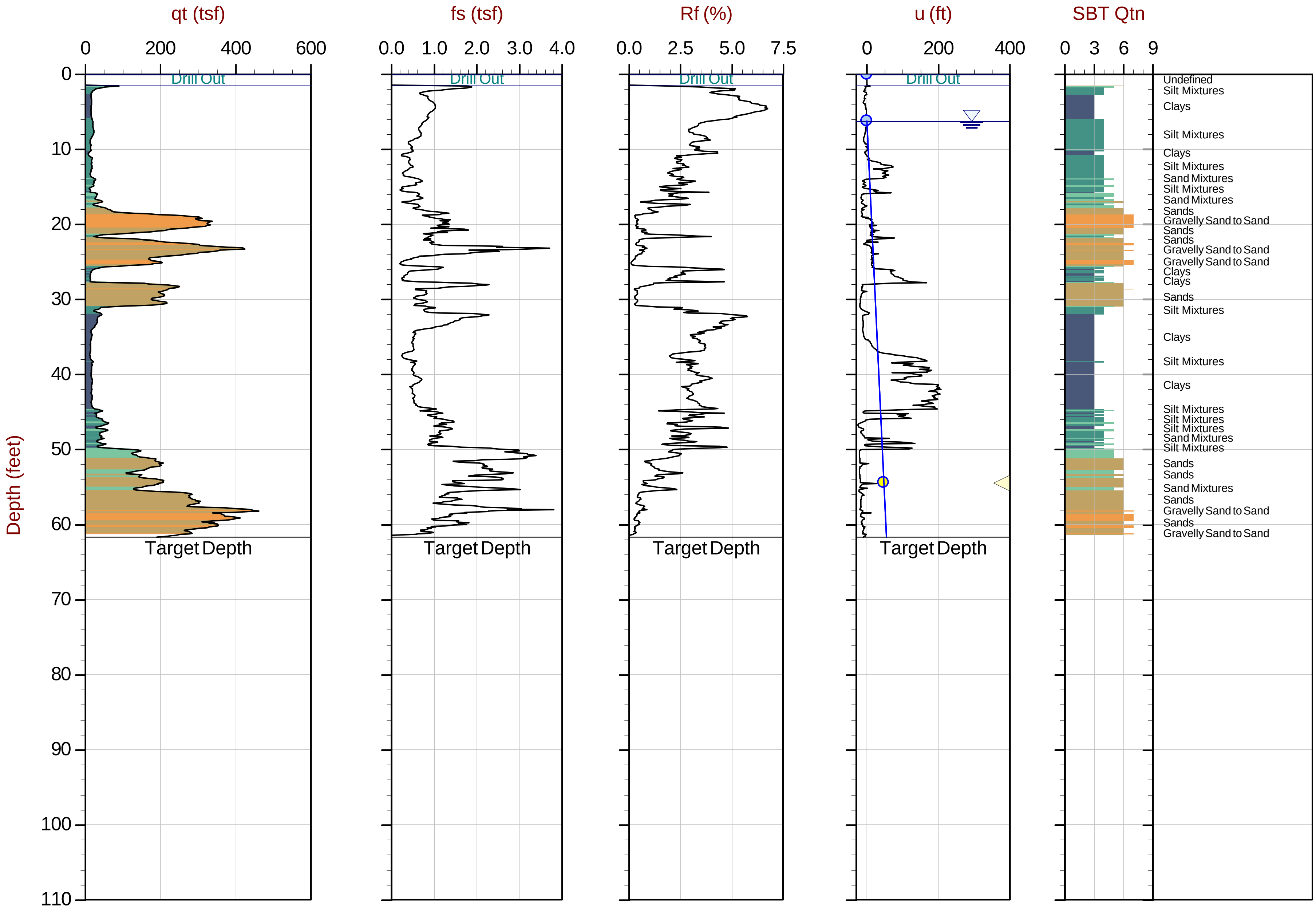
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

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.

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

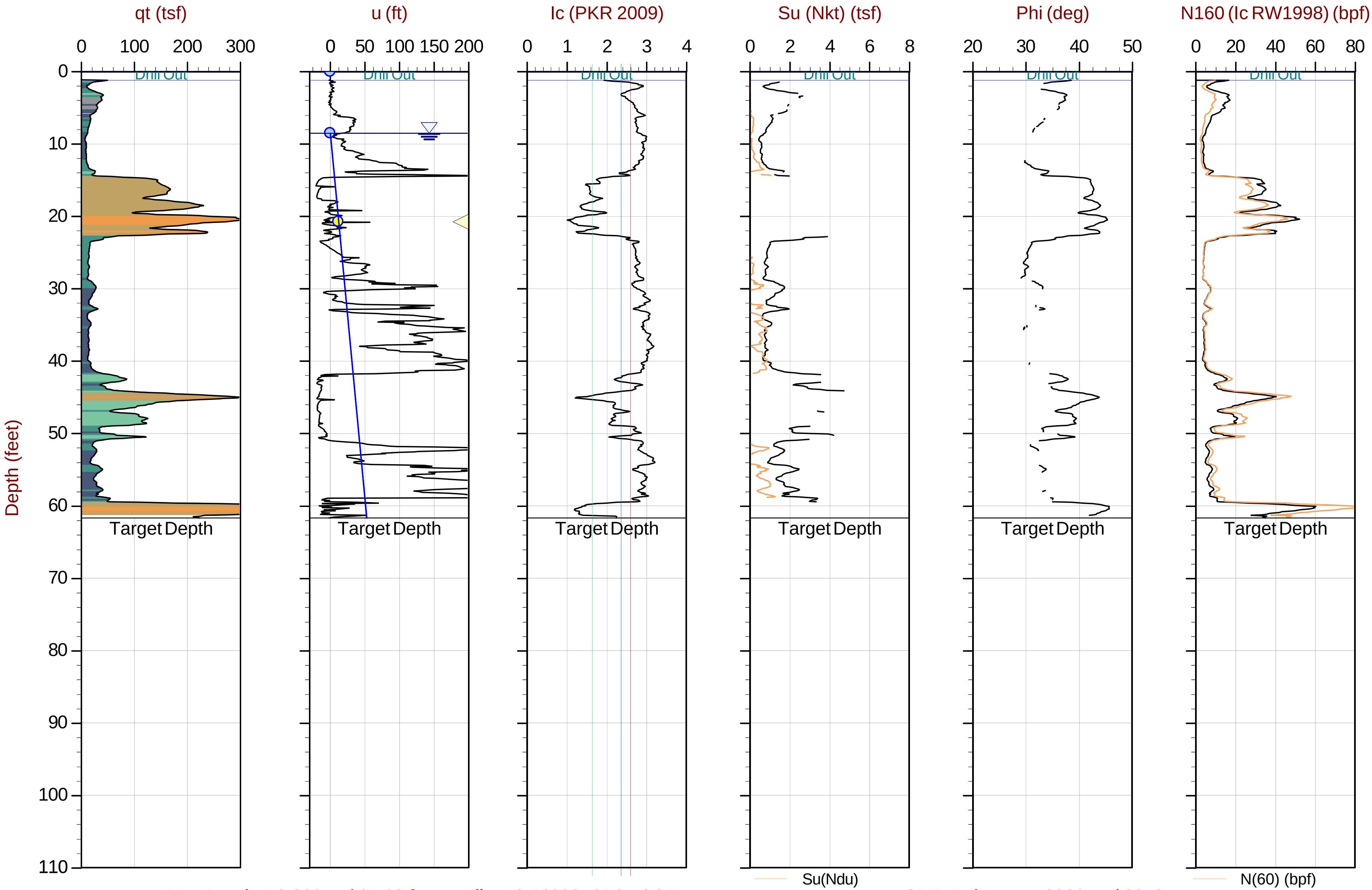
ConeTec

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

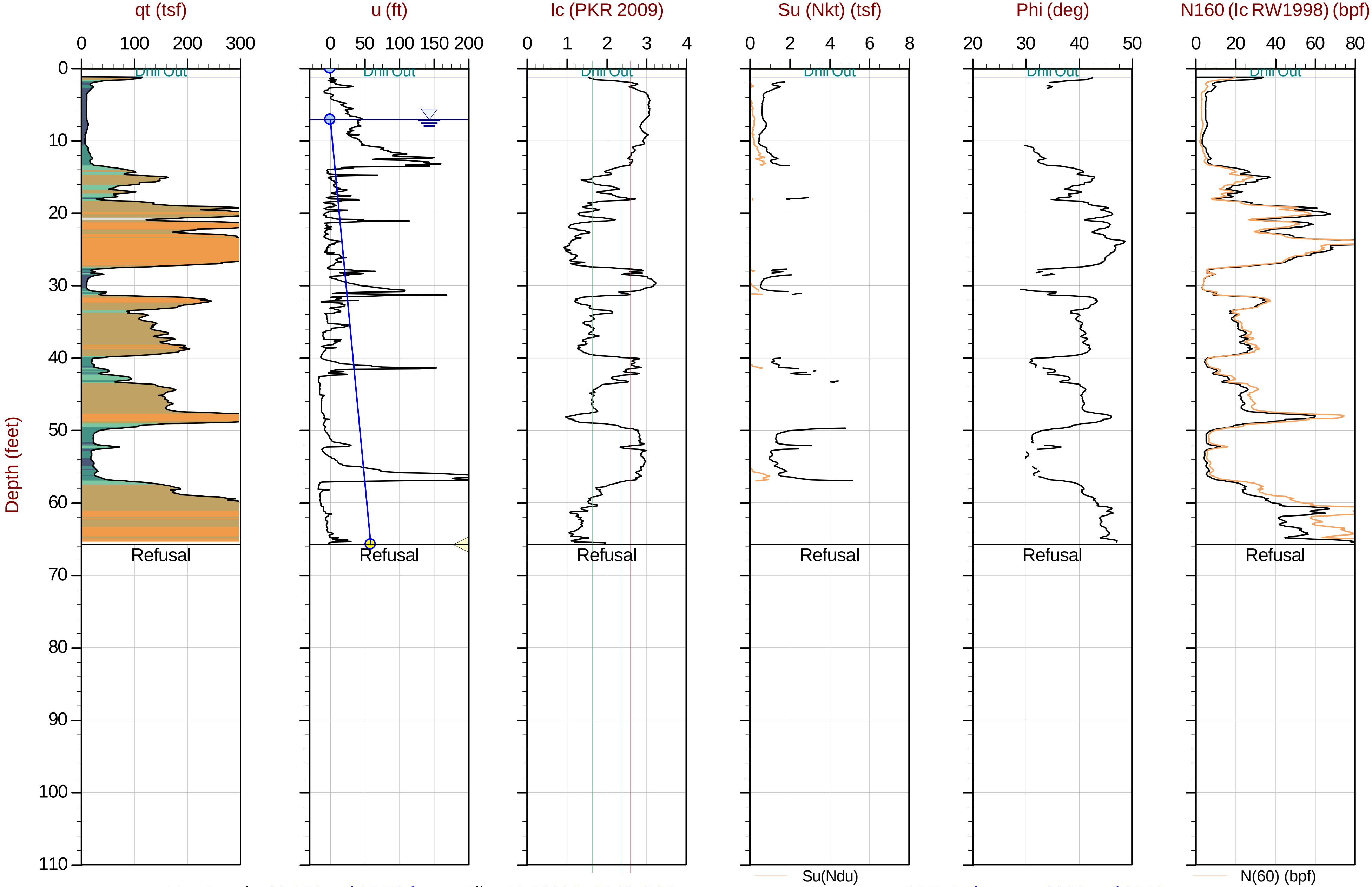
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

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

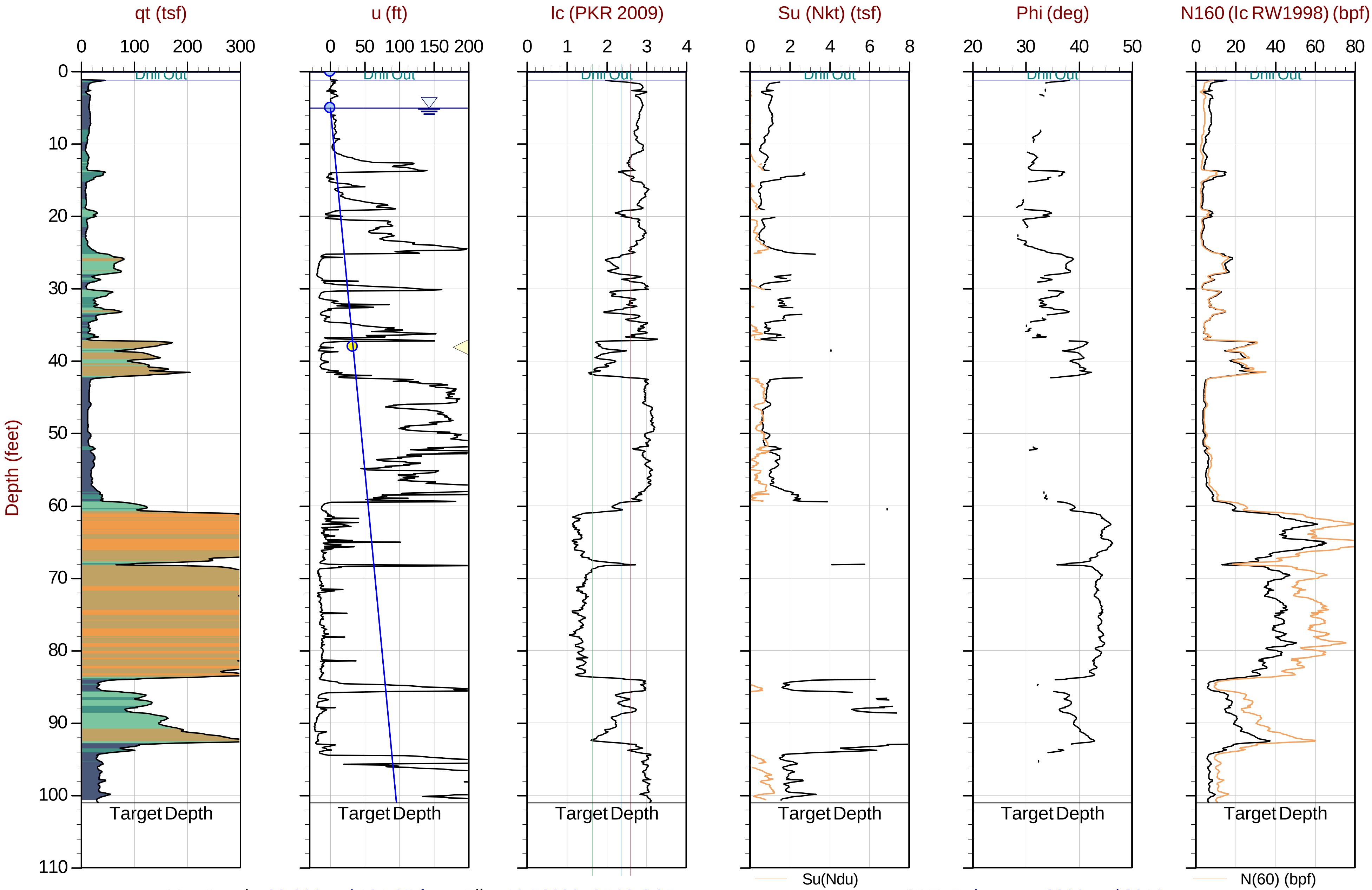
Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Hydrostatic Line

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

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

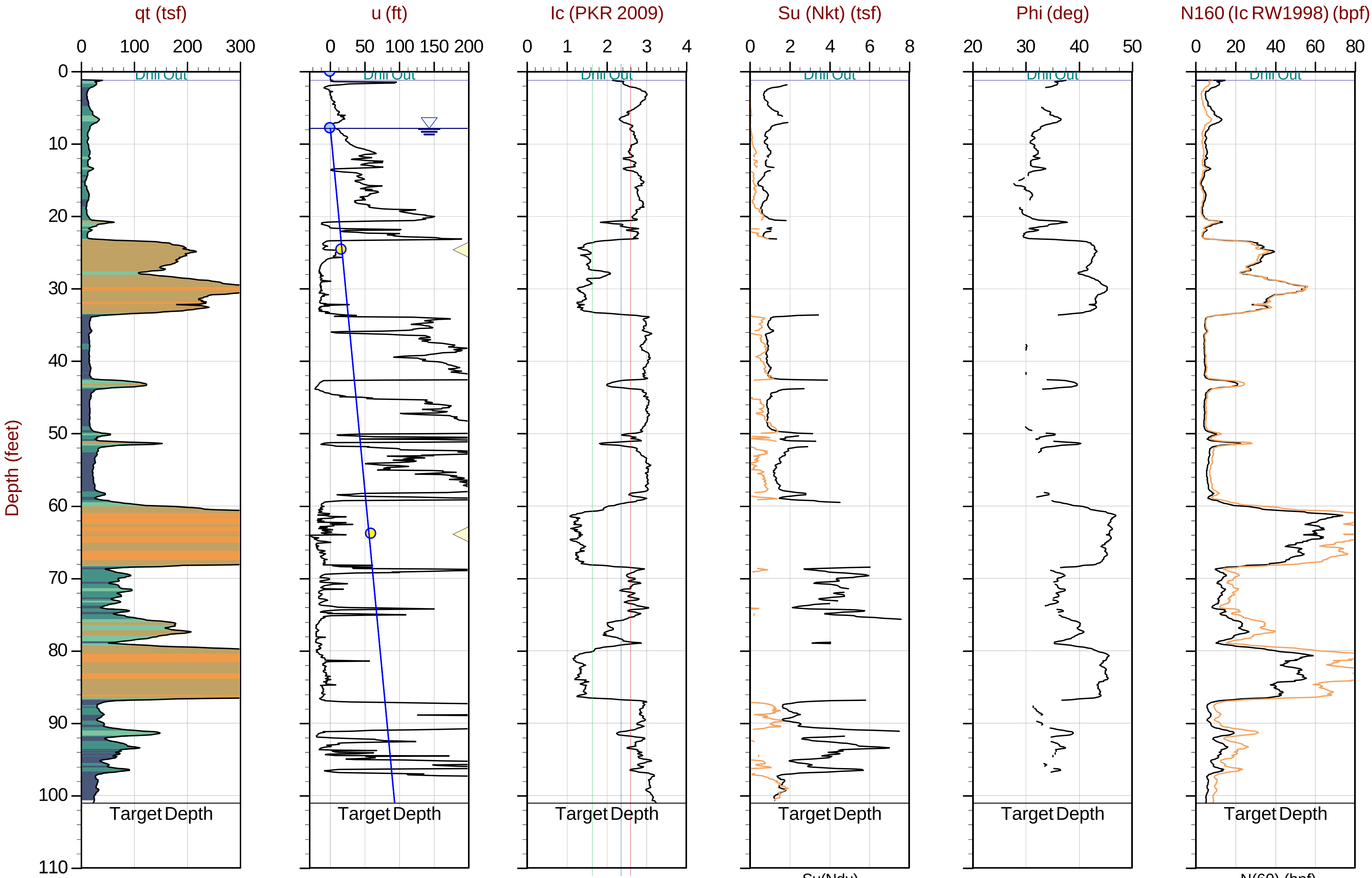
SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4137871m E: 594449m
Sheet No: 1 of 1

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

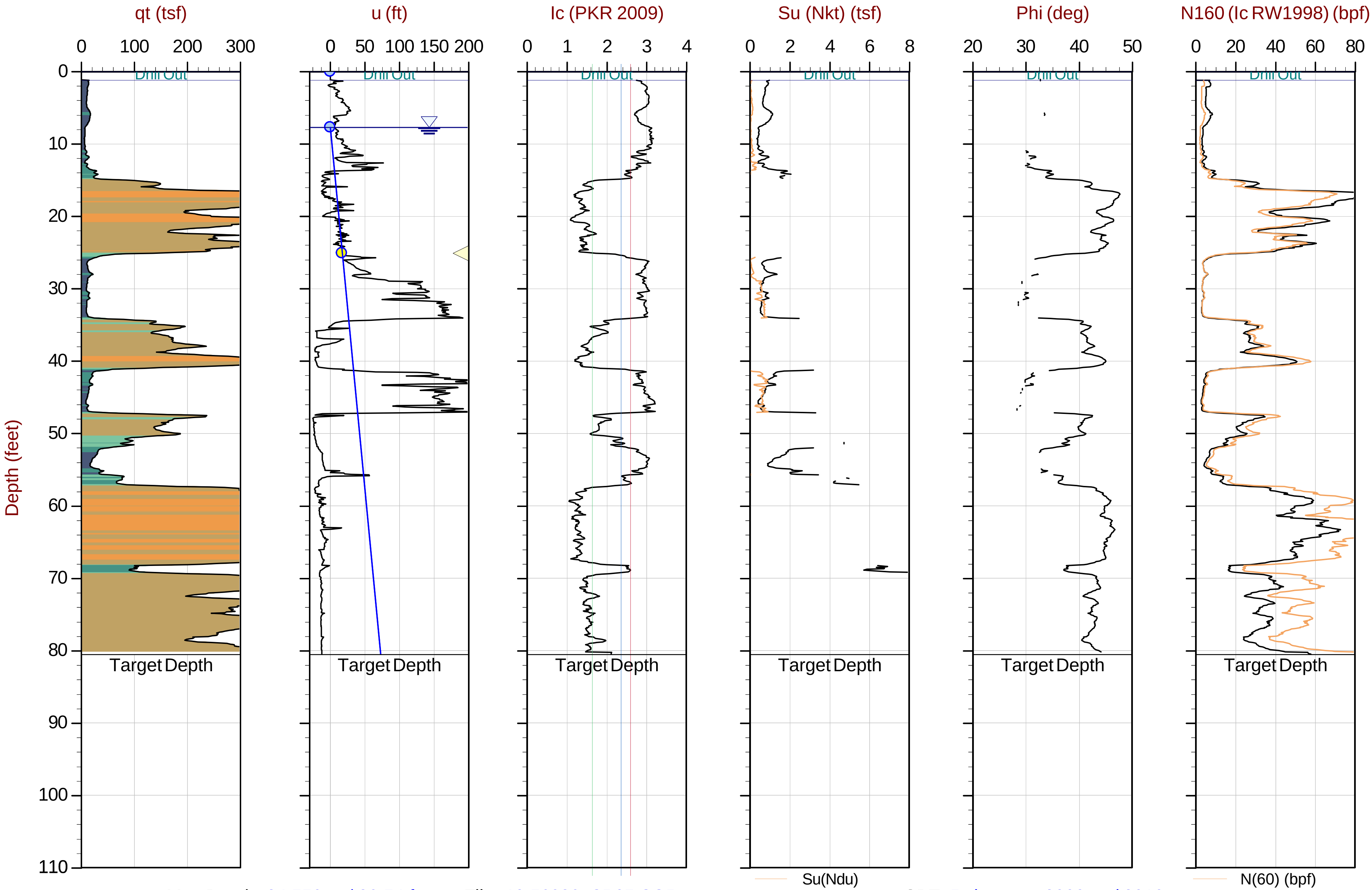
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

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

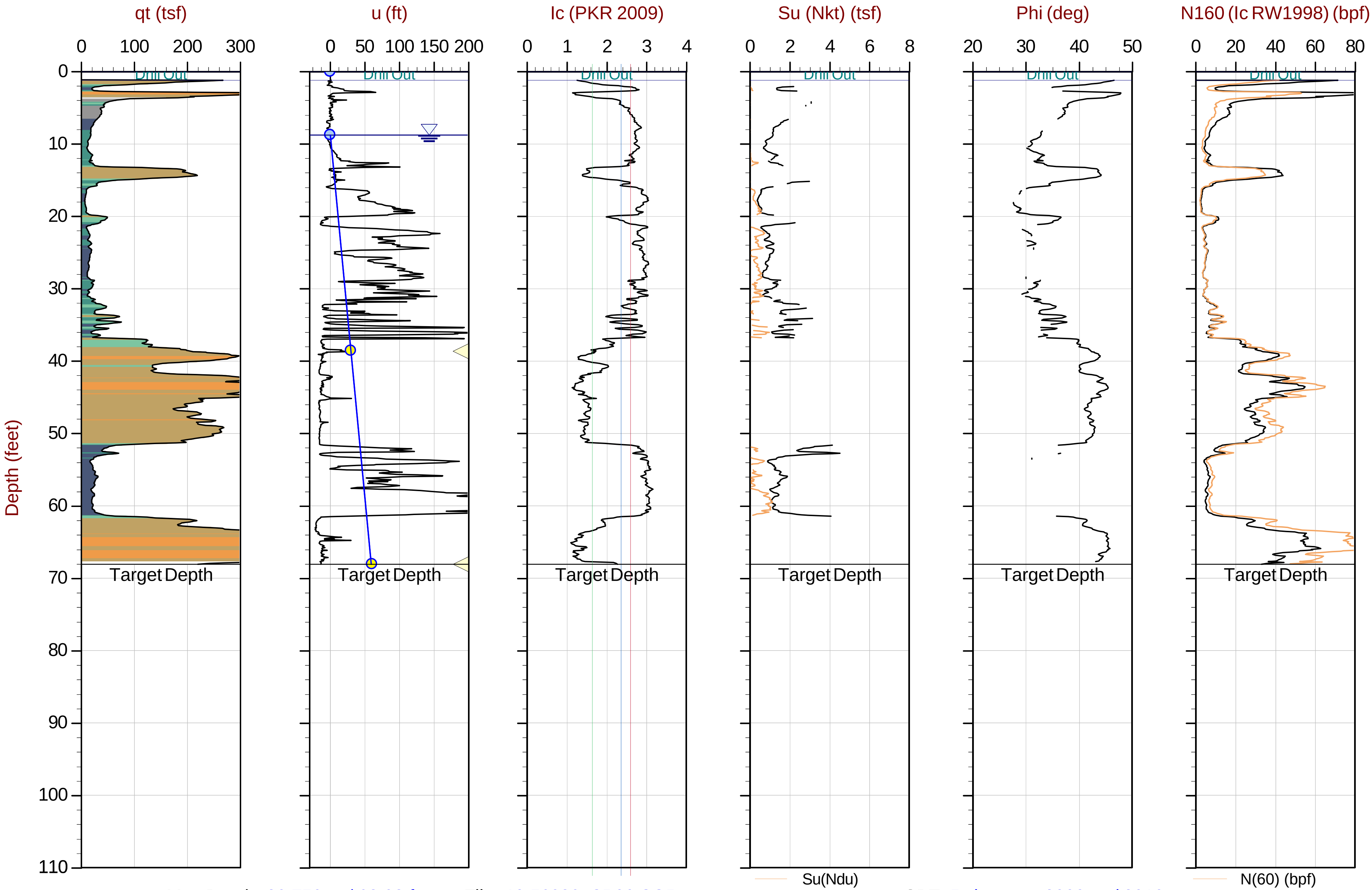
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

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)
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

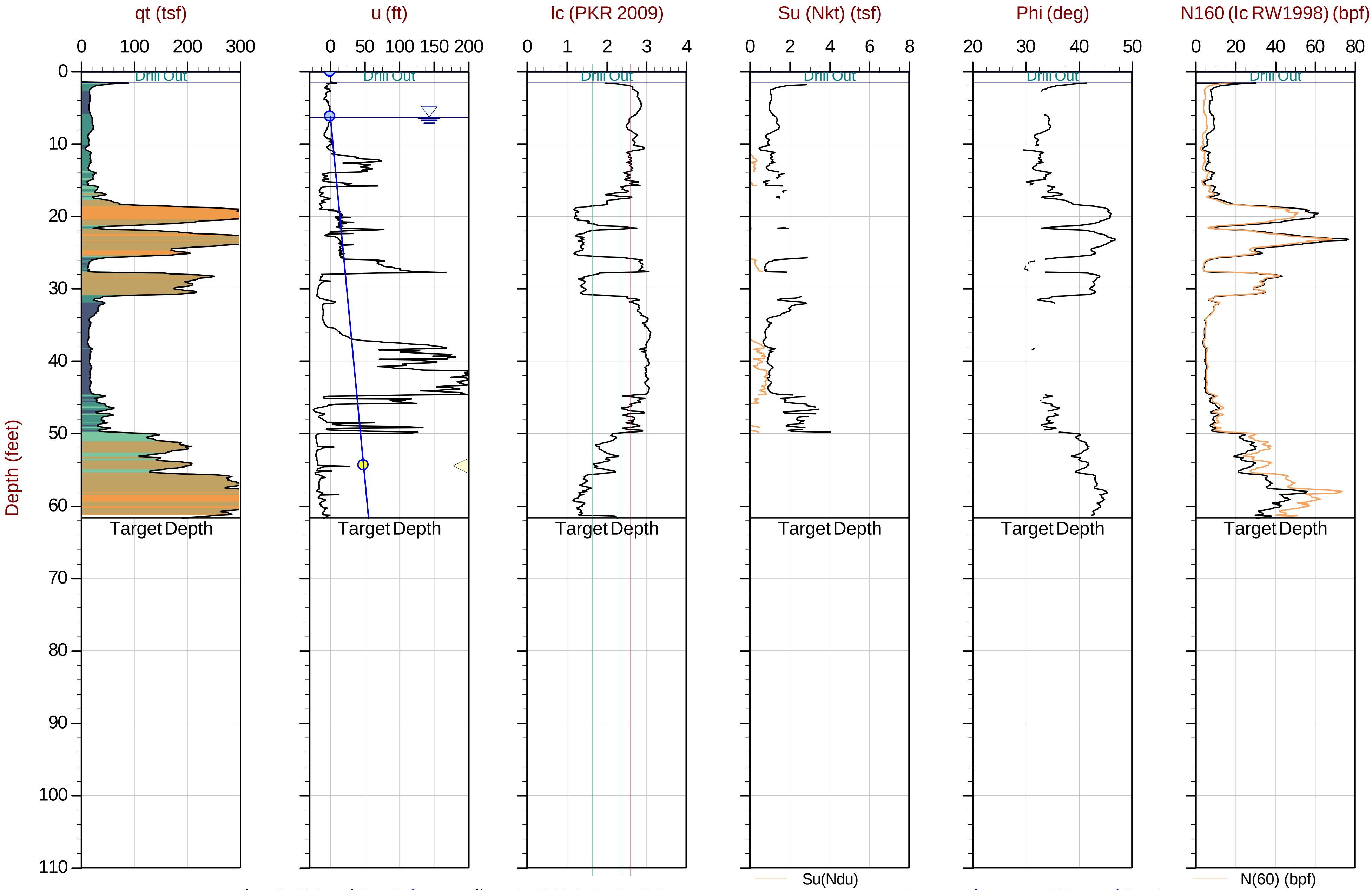
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

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)
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

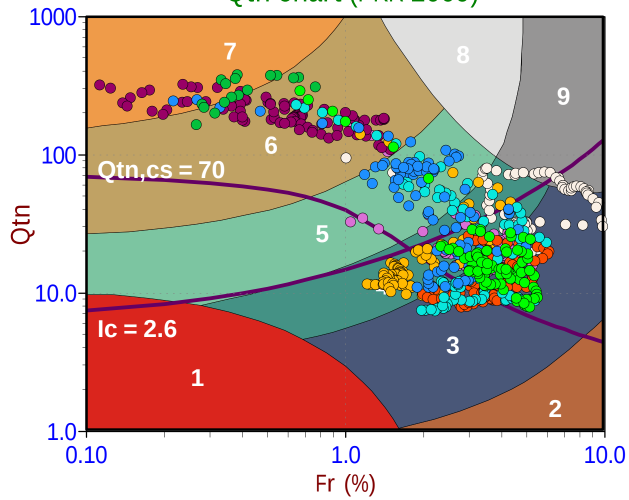
CONETEC

Langan Engineering

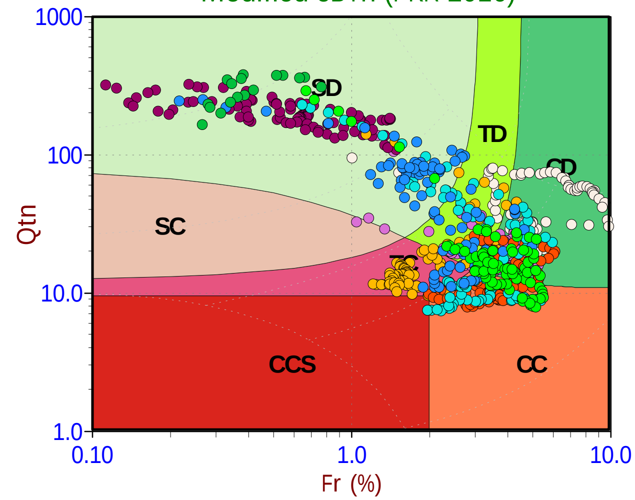
Job No: 19-56026
Date: 2019-03-04 07:46
Site: North Town

Sounding: CPT-01
Cone: 483:T1500F15U500

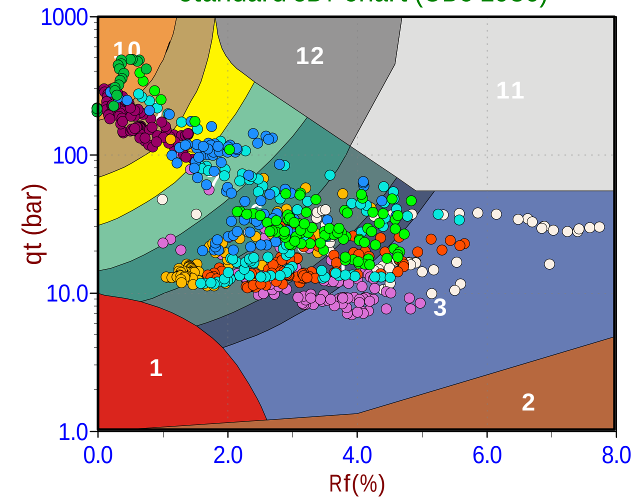
Qtn Chart (PKR 2009)



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

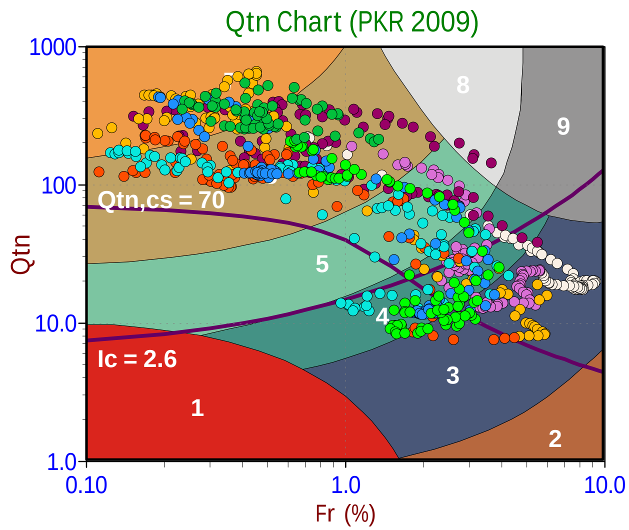
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

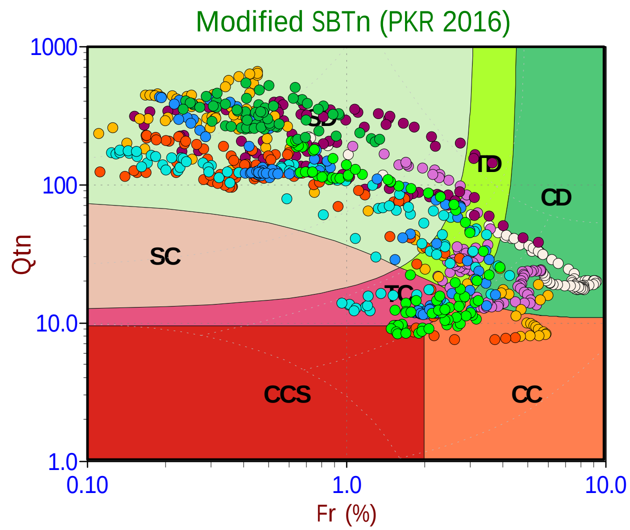
CONETEC

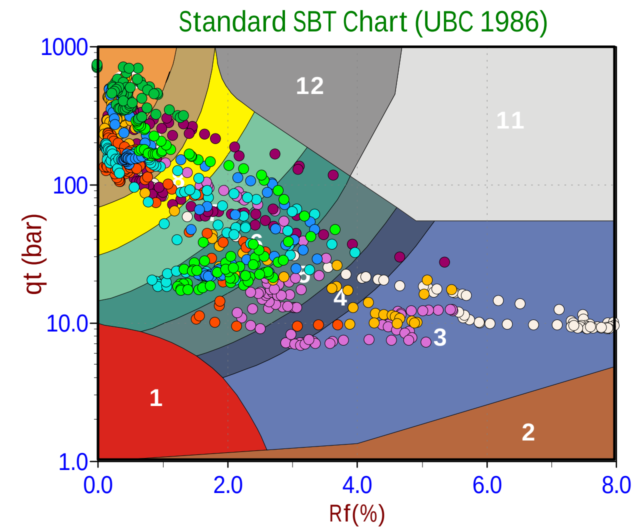
Langan Engineering

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

Sounding: CPT-02
Cone: 483:T1500F15U500







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

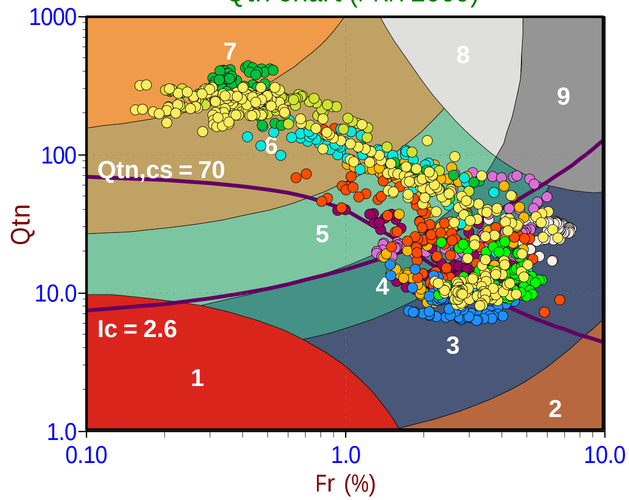
CONETEC

Langan Engineering

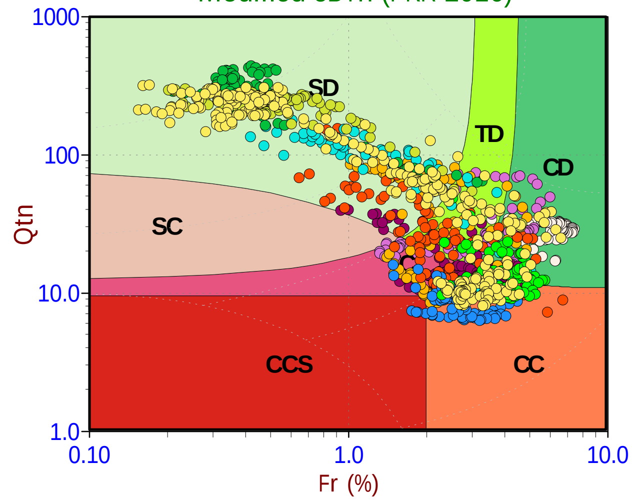
Job No: 19-56026
Date: 2019-03-05 07:37
Site: North Town

Sounding: CPT-03
Cone: 483:T1500F15U500

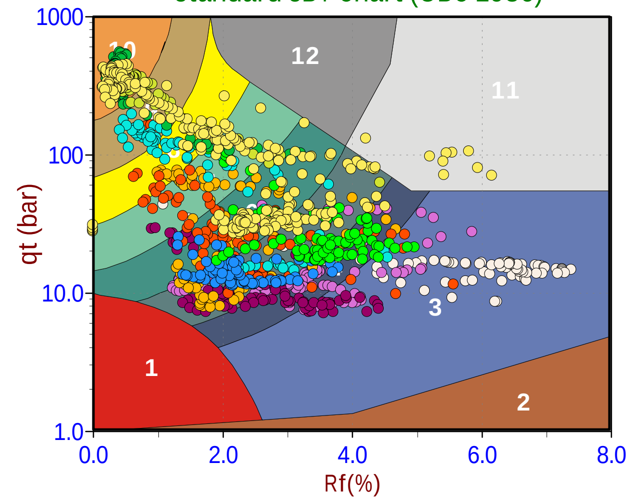
Qtn Chart (PKR 2009)



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
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- >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

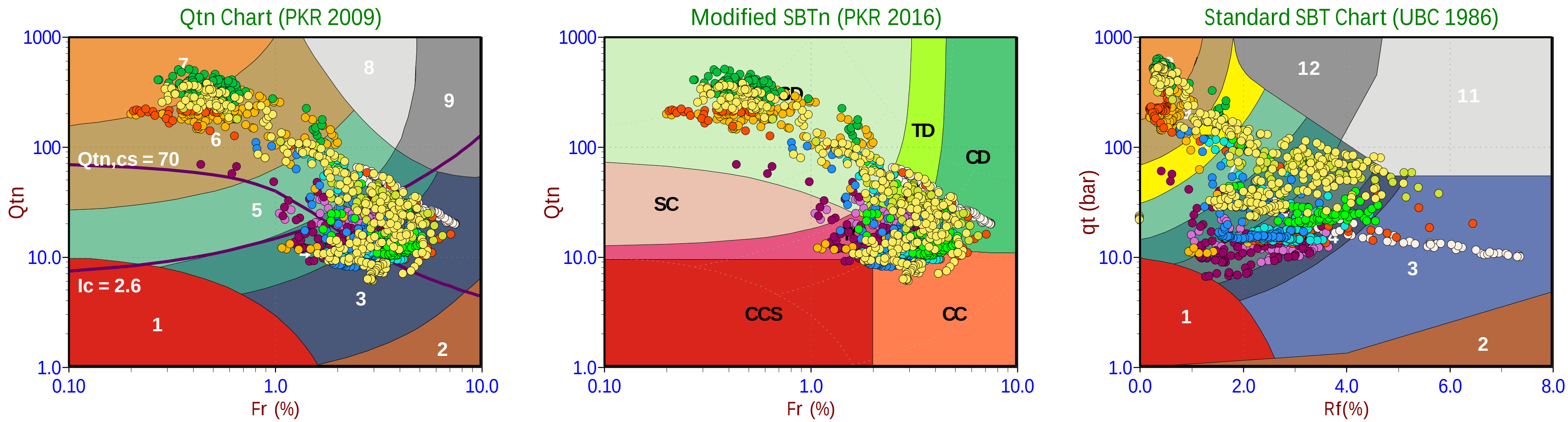
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

CONETEC

Langan Engineering

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

Sounding: CPT-04
Cone: 483:T1500F15U500



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

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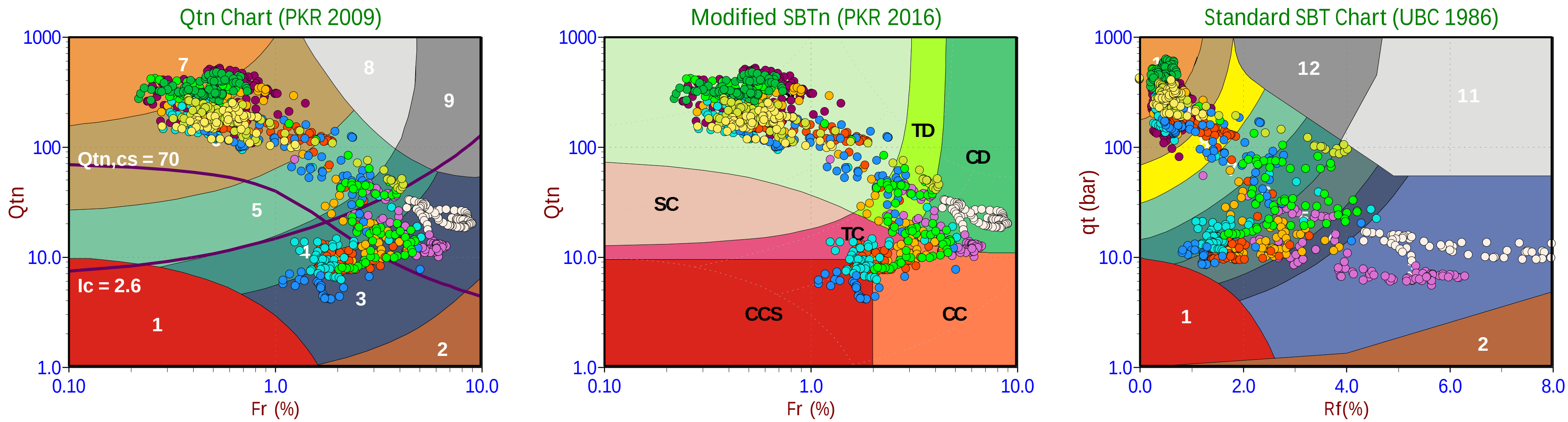
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

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

Langan Engineering

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

Sounding: CPT-05
Cone: 483:T1500F15U500



Depth Ranges
- >0.0 to 7.5 ft
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- >15.0 to 22.5 ft
- >22.5 to 30.0 ft
- >30.0 to 37.5 ft
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- >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
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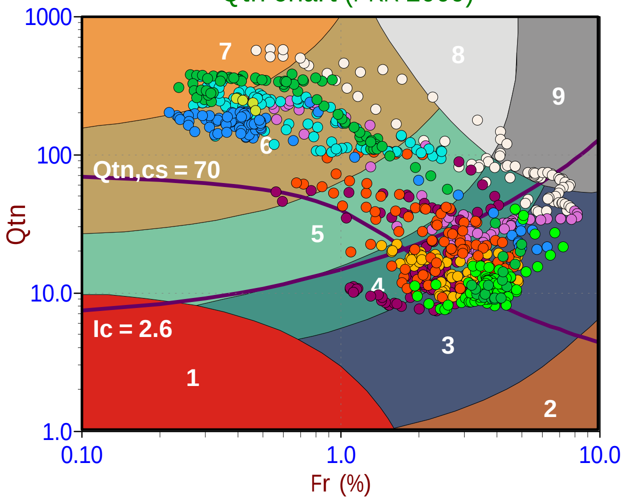
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

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

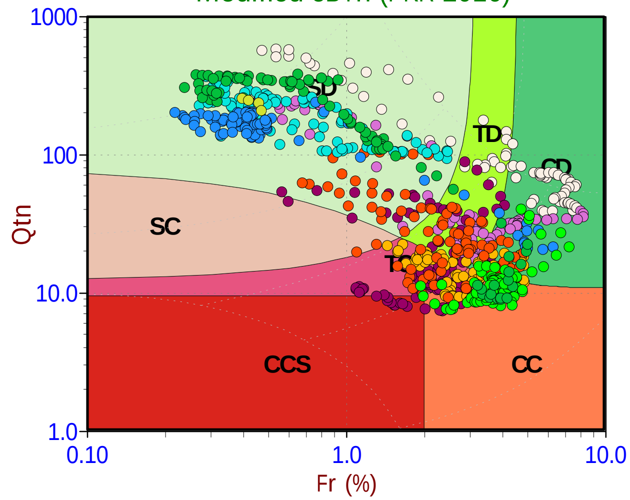
ConeTec

Langan Engineering	Job No: 19-56026	Sounding: CPT-06
	Date: 2019-03-04 10:12	Cone: 483:T1500F15U500
	Site: North Town	

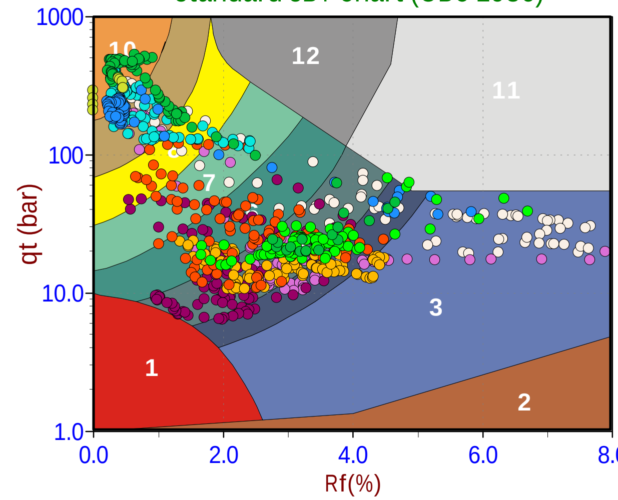
Qtn Chart (PKR 2009)



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

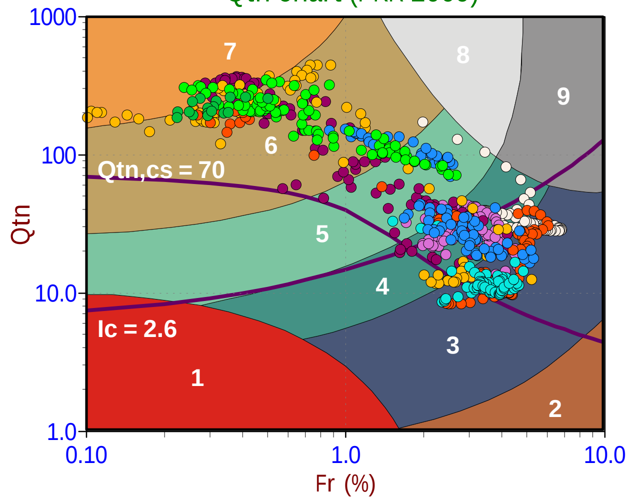
CONETEC

Langan Engineering

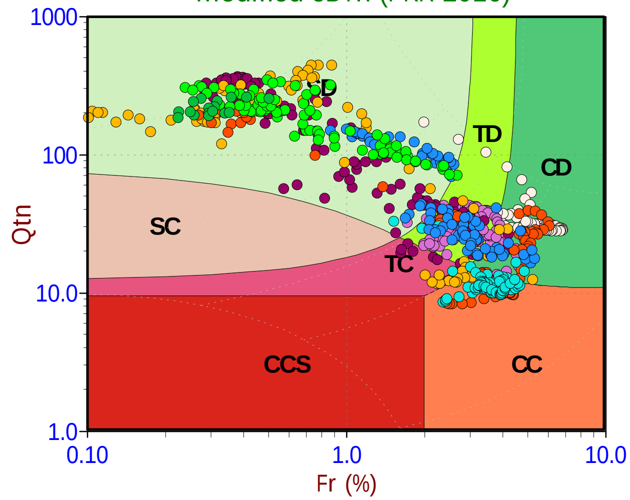
Job No: 19-56026
Date: 2019-03-04 11:10
Site: North Town

Sounding: CPT-07
Cone: 483:T1500F15U500

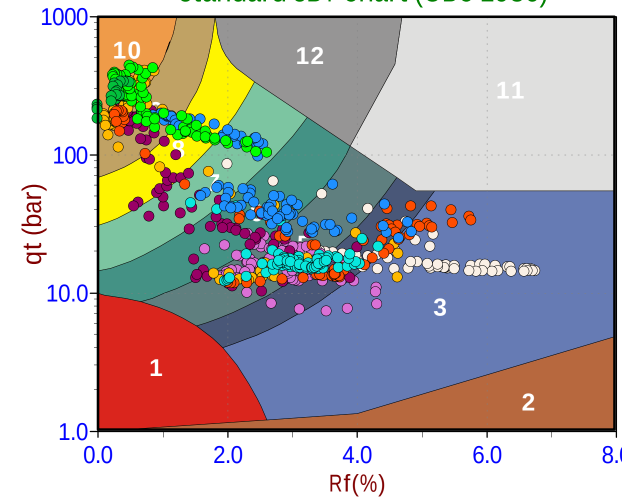
Qtn Chart (PKR 2009)



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

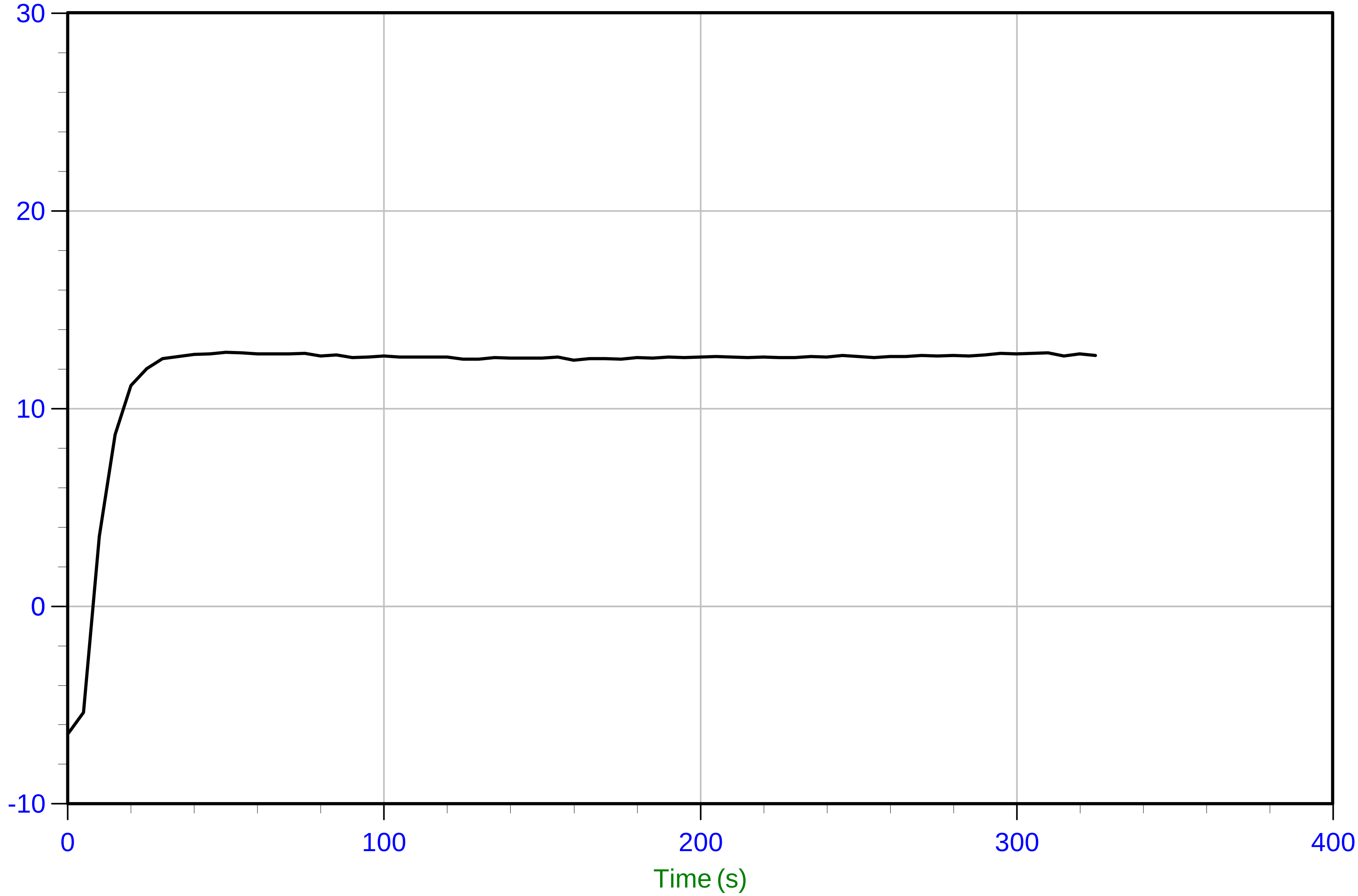
Pore Pressure Dissipation Summary and
Pore Pressure Dissipation Plots

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

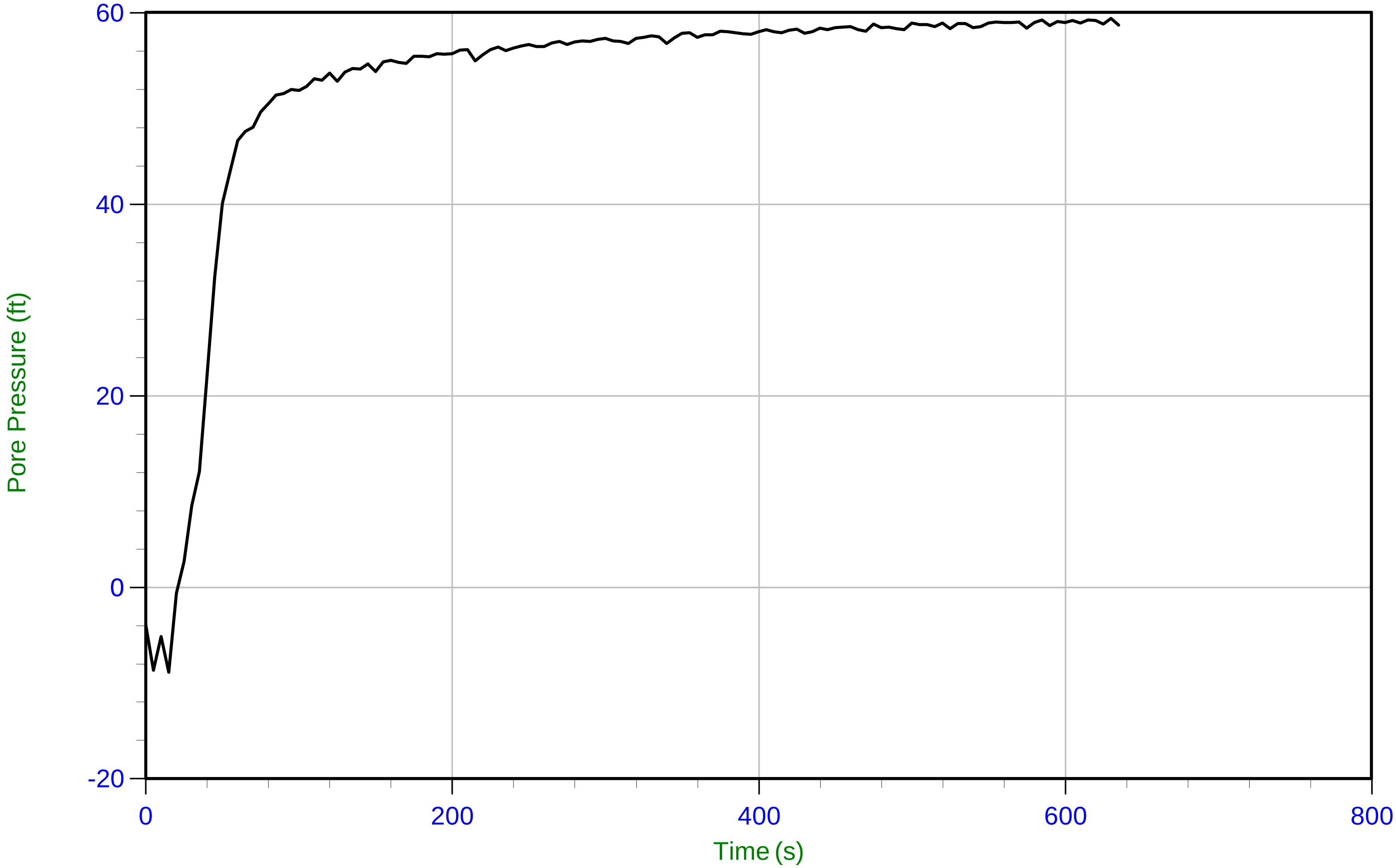
u Min: -6.5 ft
u Max: 12.8 ft
u Final: 12.7 ft

WT: 2.600 m / 8.530 ft
Ueq: 12.2 ft

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

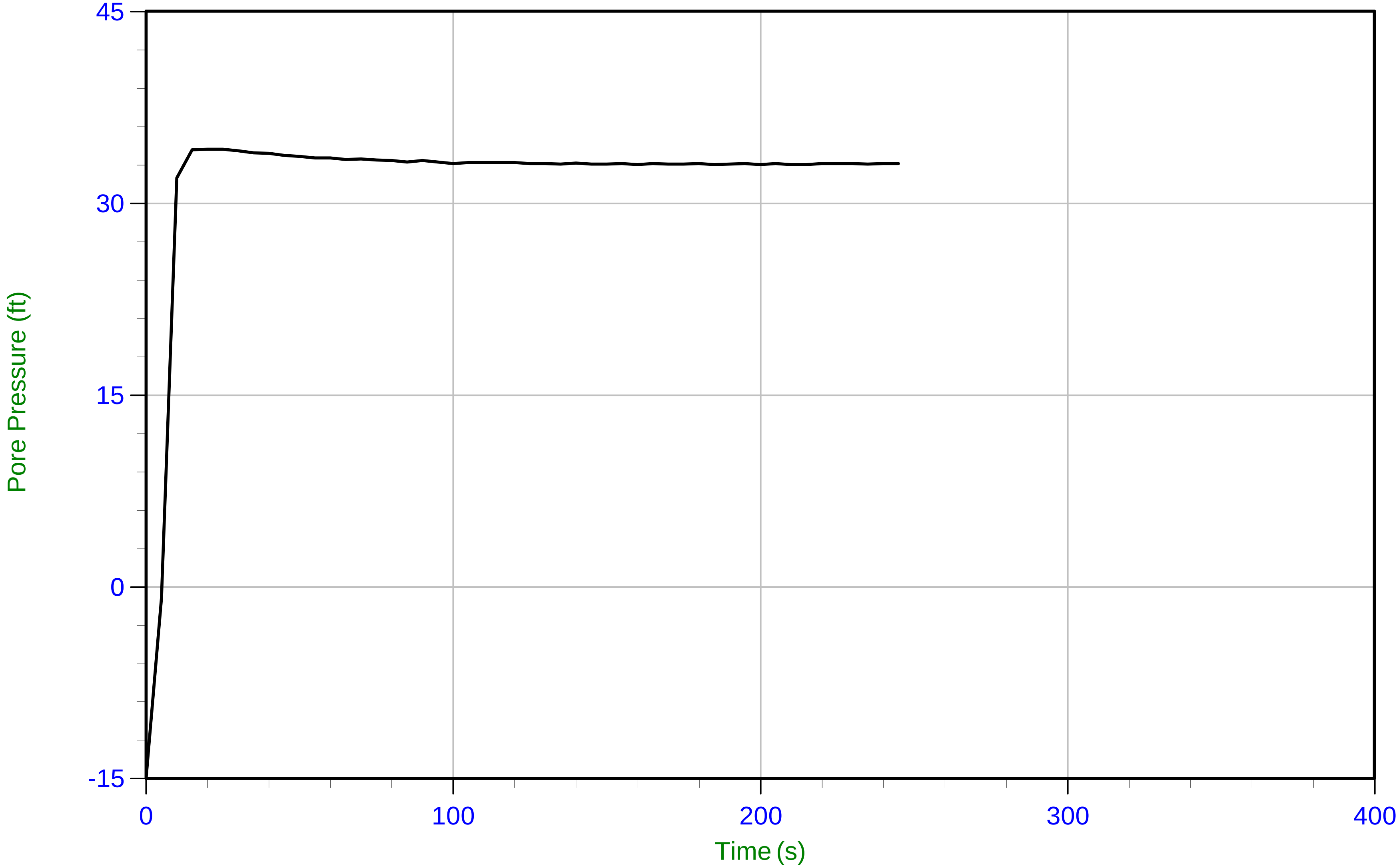
WT: 2.162 m / 7.093 ft
Ueq: 58.7 ft

CONETEC

Langan Engineering

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

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



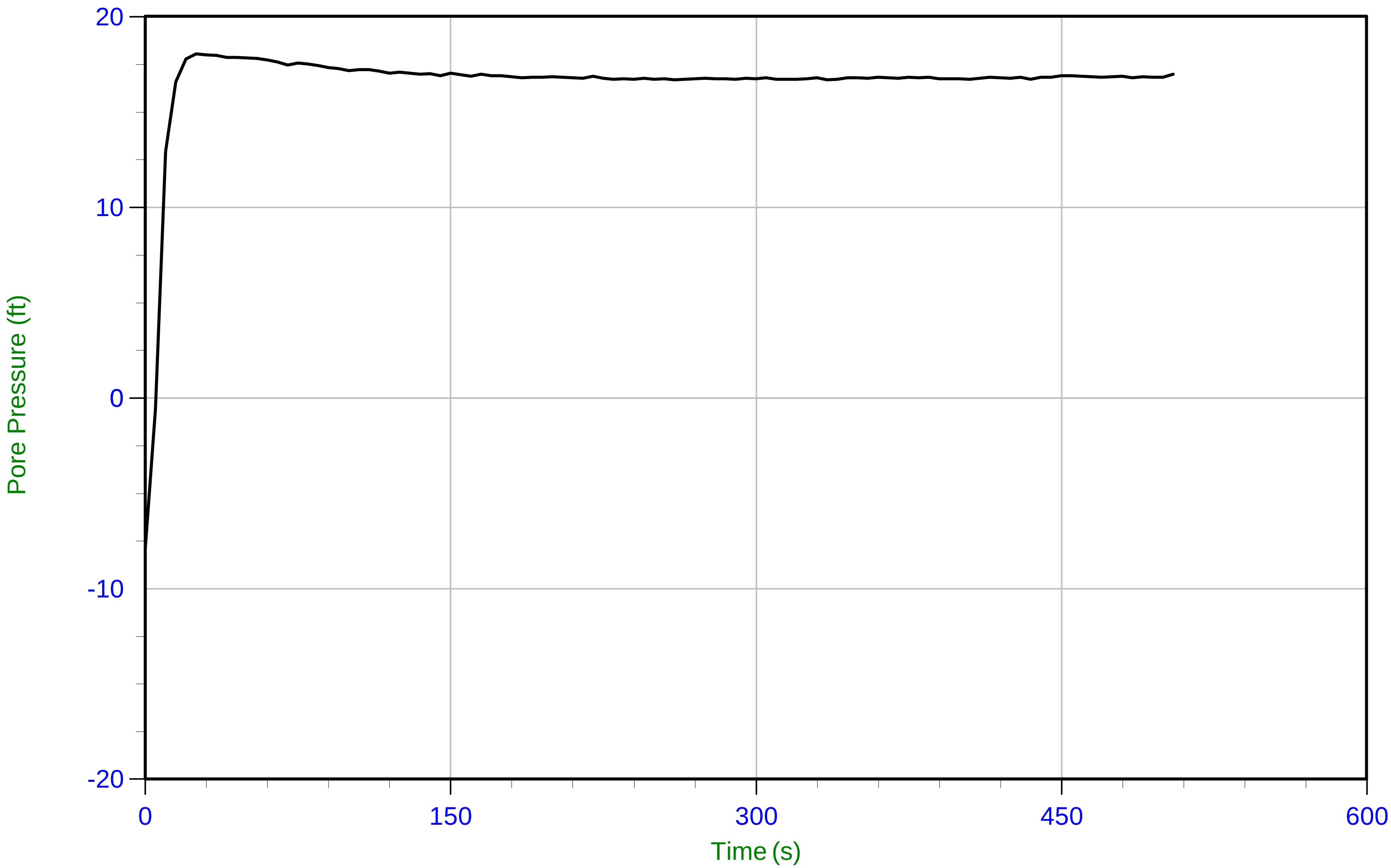
Trace Summary:

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

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:

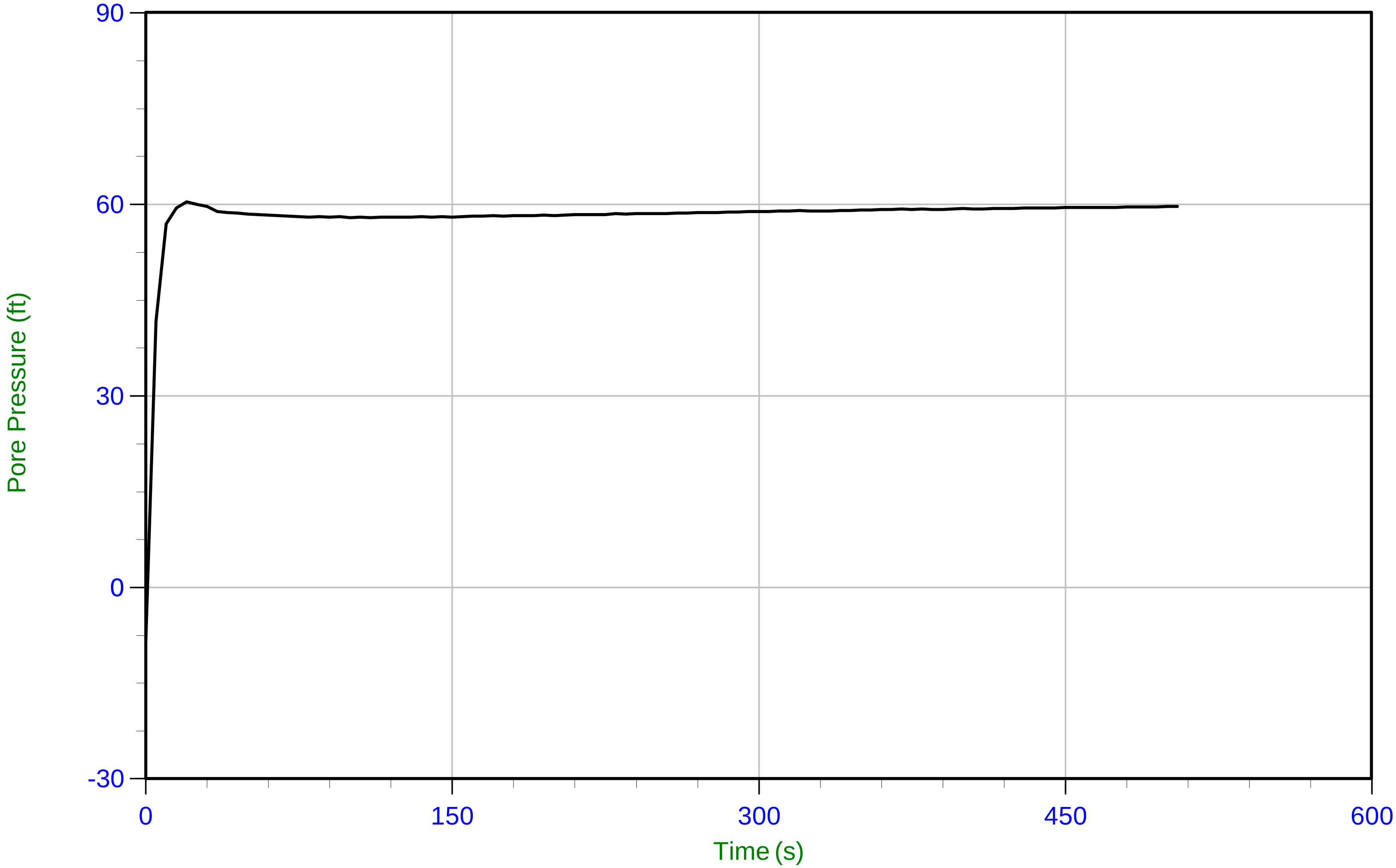
Filename: 19-56026_CP04.PPF	u Min: -7.9 ft	WT: 2.389 m / 7.838 ft
Depth: 7.500 m / 24.606 ft	u Max: 18.0 ft	Ueq: 16.8 ft
Duration: 505.0 s	u Final: 17.0 ft	

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: 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

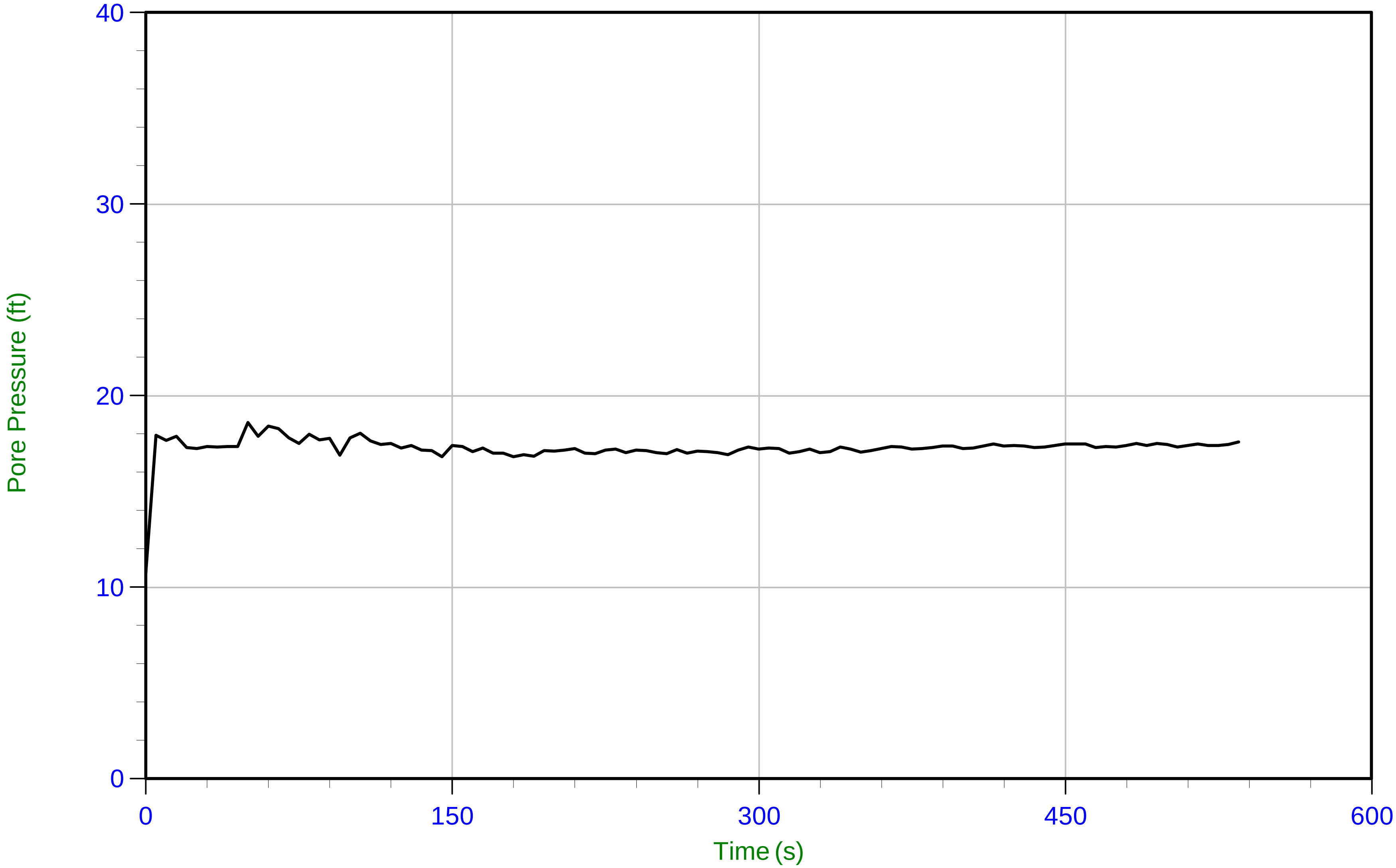
WT: 1.427 m / 4.682 ft
Ueq: 59.2 ft

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



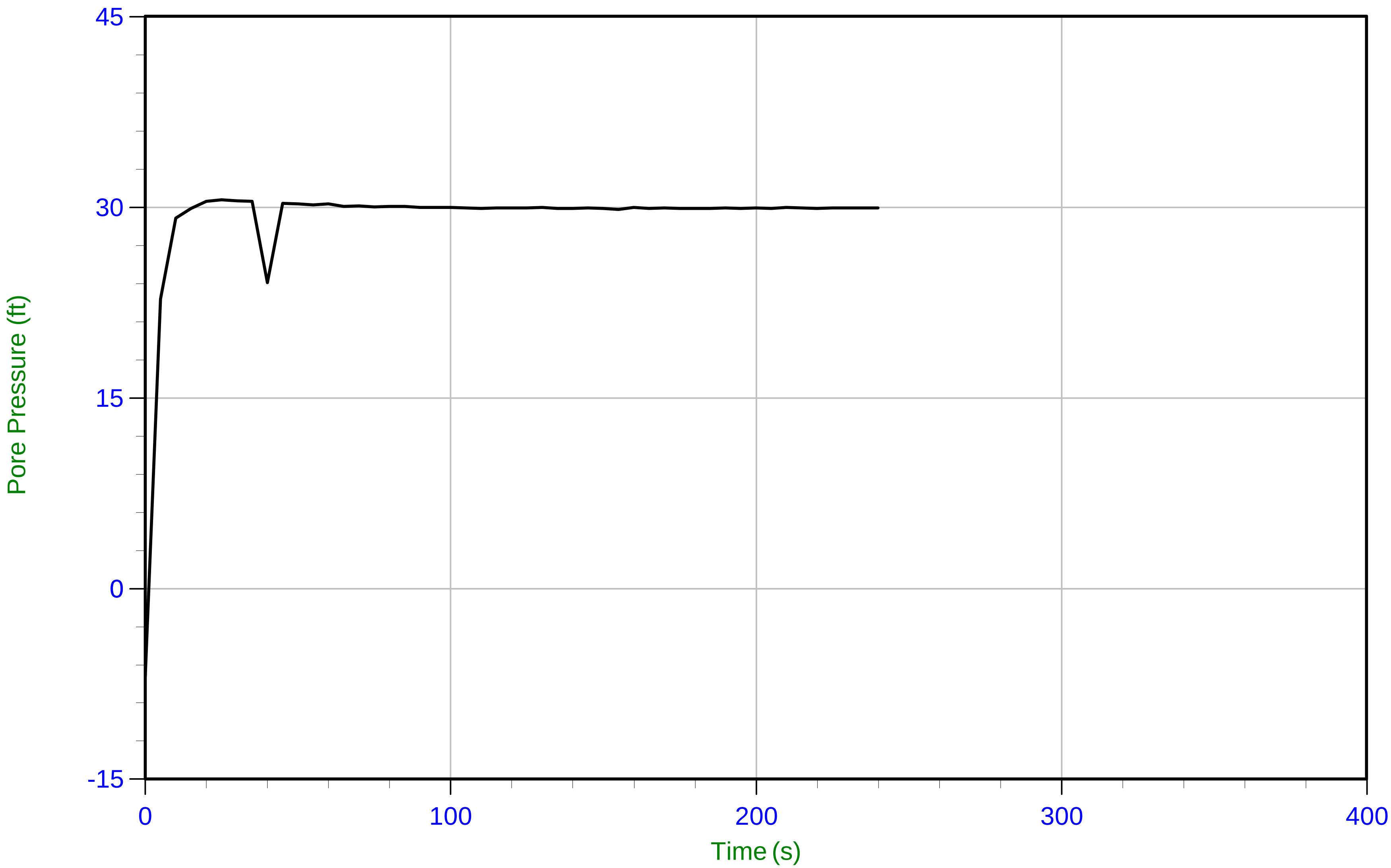
Trace Summary:

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

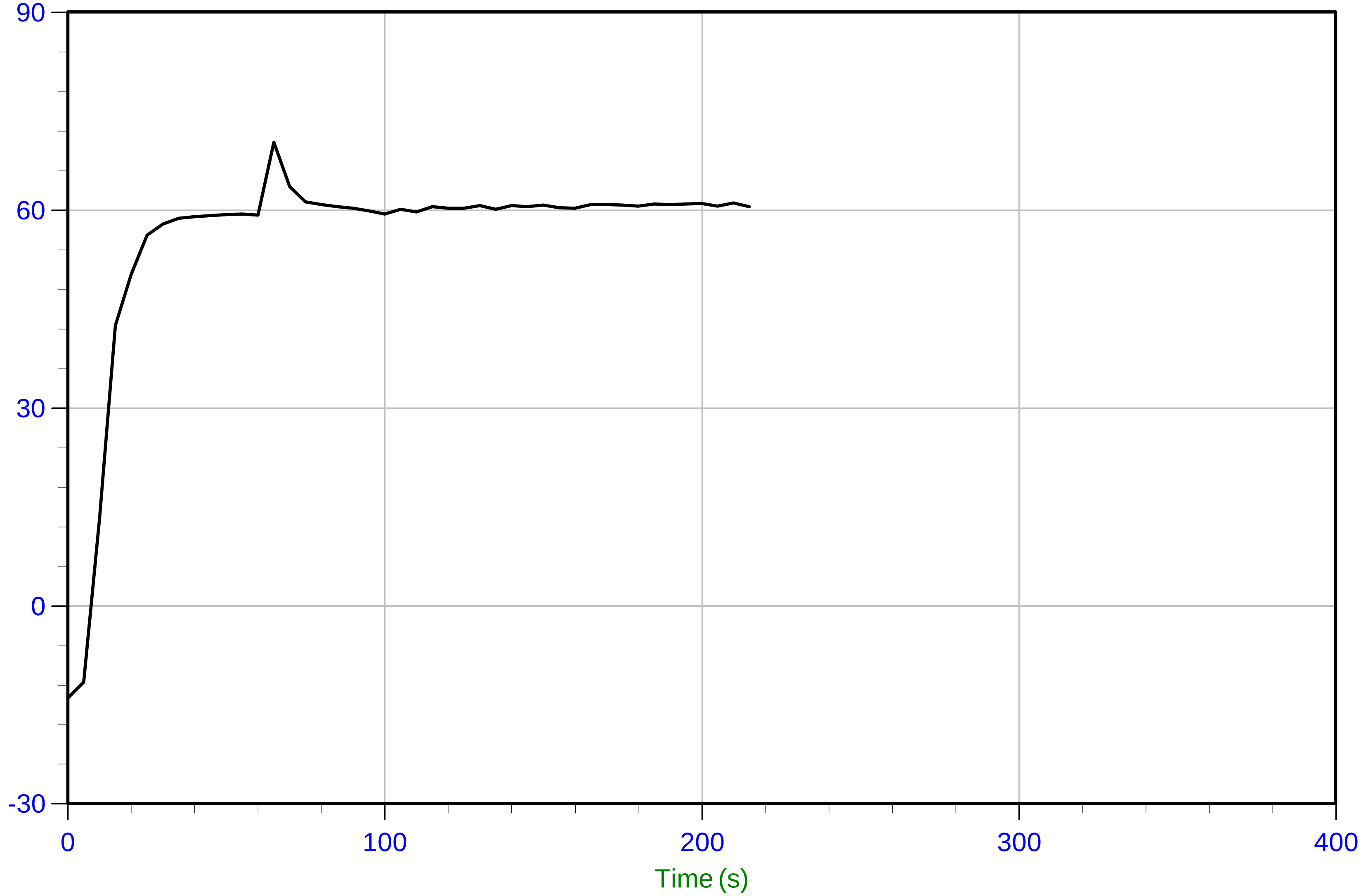
Langan Engineering	Job No: 19-56026 Date: 03/04/2019 10:12 Site: North Town	Sounding: CPT-06 Cone: 483:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 19-56026_CP06.PPF	u Min: -6.9 ft	WT: 2.677 m / 8.783 ft
Depth: 11.775 m / 38.631 ft	u Max: 30.6 ft	Ueq: 29.8 ft
Duration: 240.0 s	u Final: 29.9 ft	

CONETEC Langan Engineering	Job No: 19-56026 Date: 03/04/2019 10:12 Site: North Town	Sounding: CPT-06 Cone: 483:T1500F15U500 Area=15 cm²



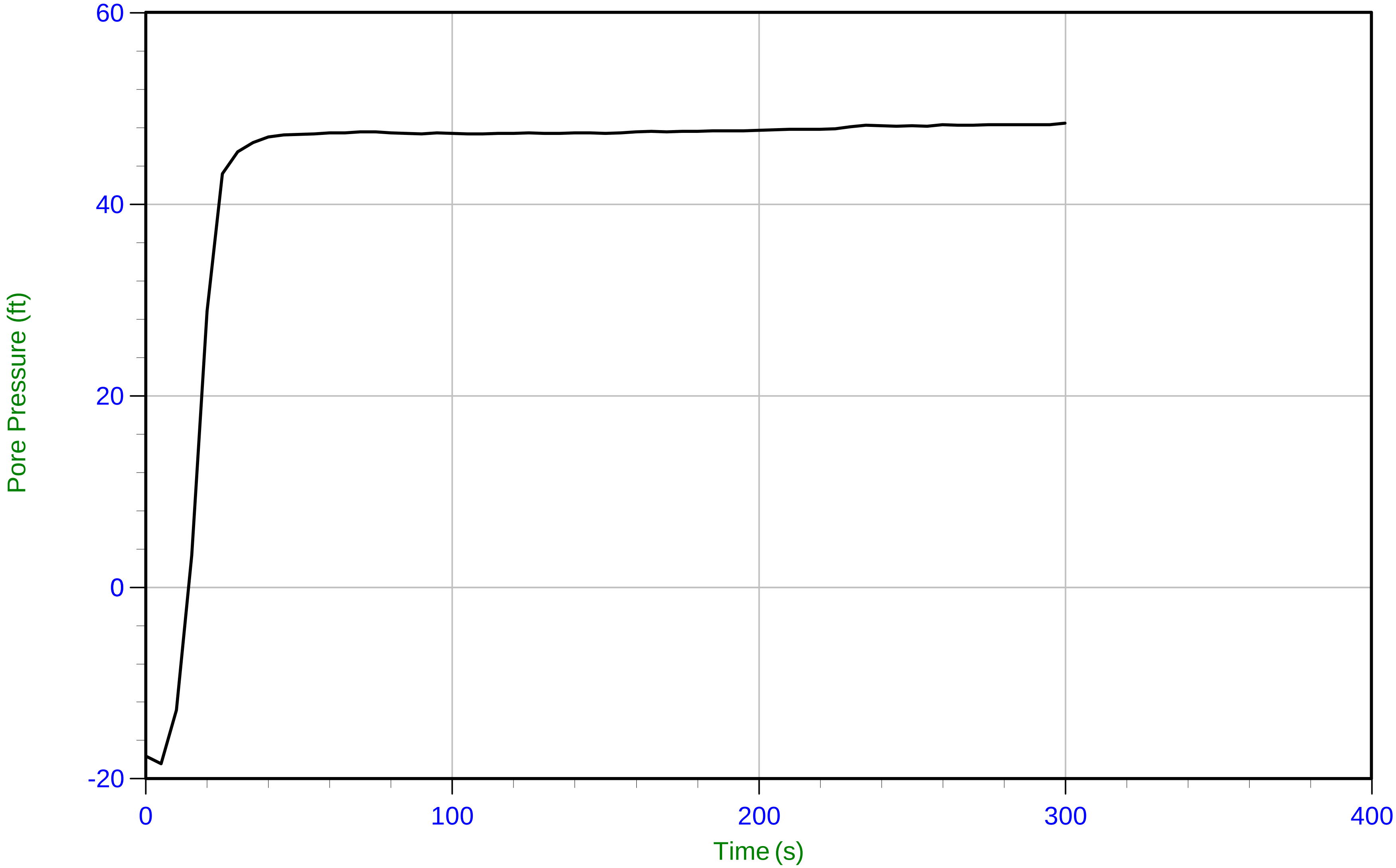
Trace Summary:

Filename: 19-56026_CP06.PPF	u Min: -14.0 ft	WT: 2.185 m / 7.169 ft
Depth: 20.750 m / 68.077 ft	u Max: 70.3 ft	Ueq: 60.9 ft
Duration: 215.0 s	u Final: 60.5 ft	

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:

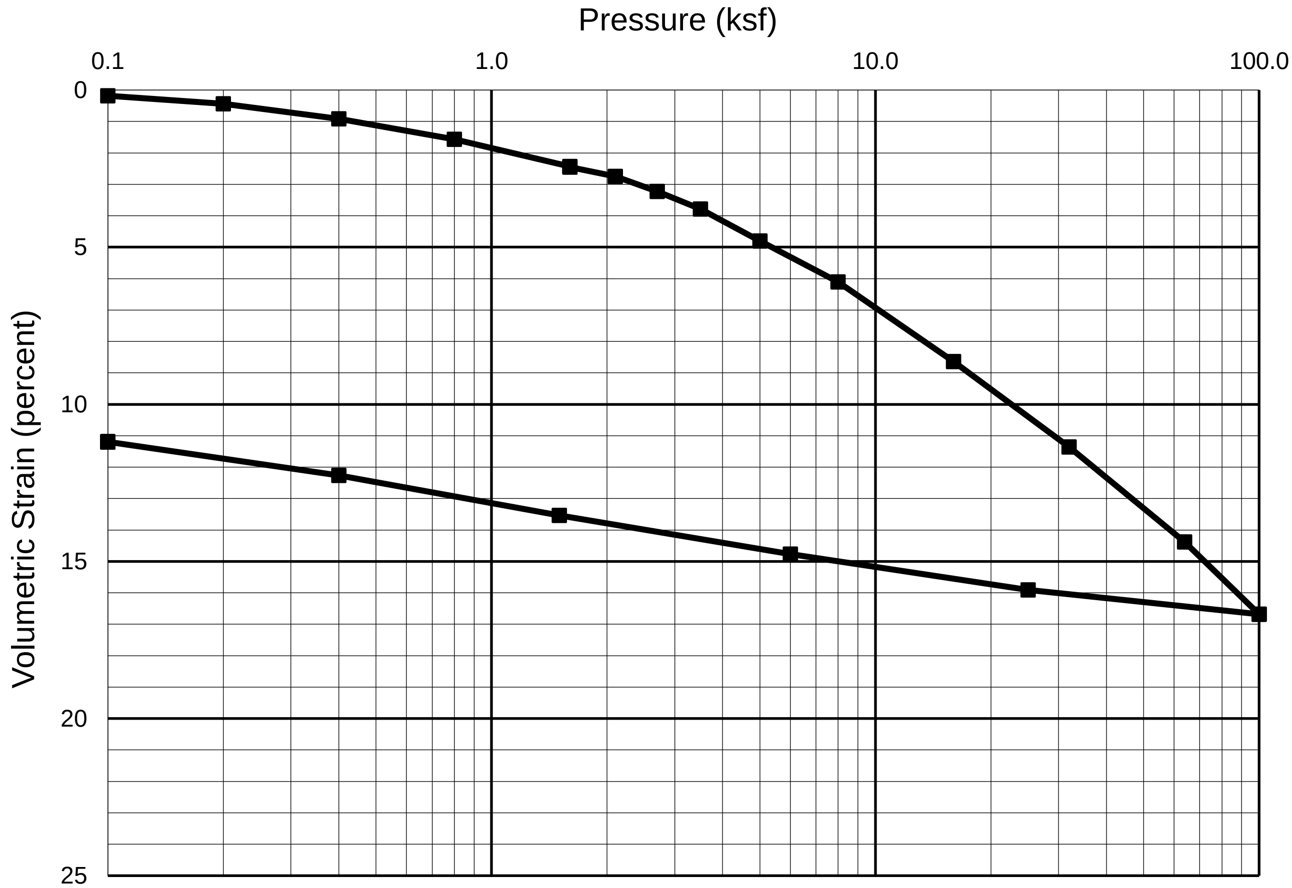
Filename: 19-56026_CP07.PPF
Depth: 16.600 m / 54.461 ft
Duration: 300.0 s

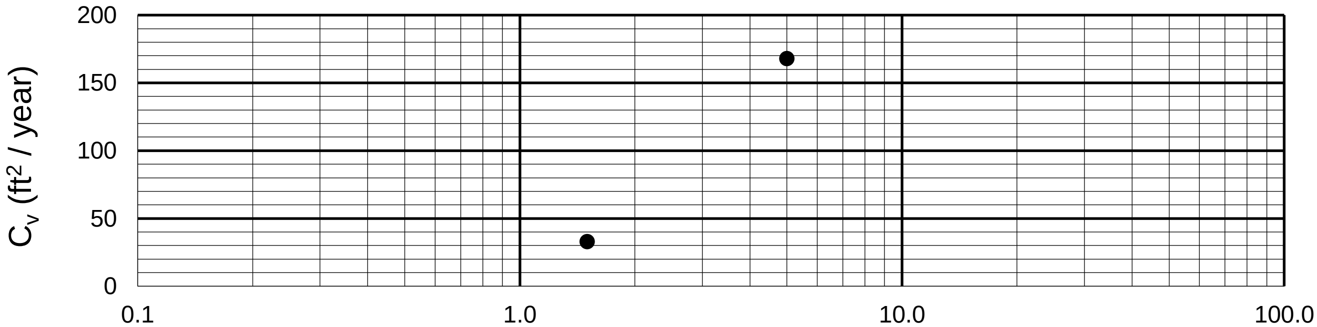
u Min: -18.4 ft
u Max: 48.5 ft
u Final: 48.5 ft

WT: 1.908 m / 6.260 ft
Ueq: 48.2 ft

APPENDIX C

LABORATORY DATA





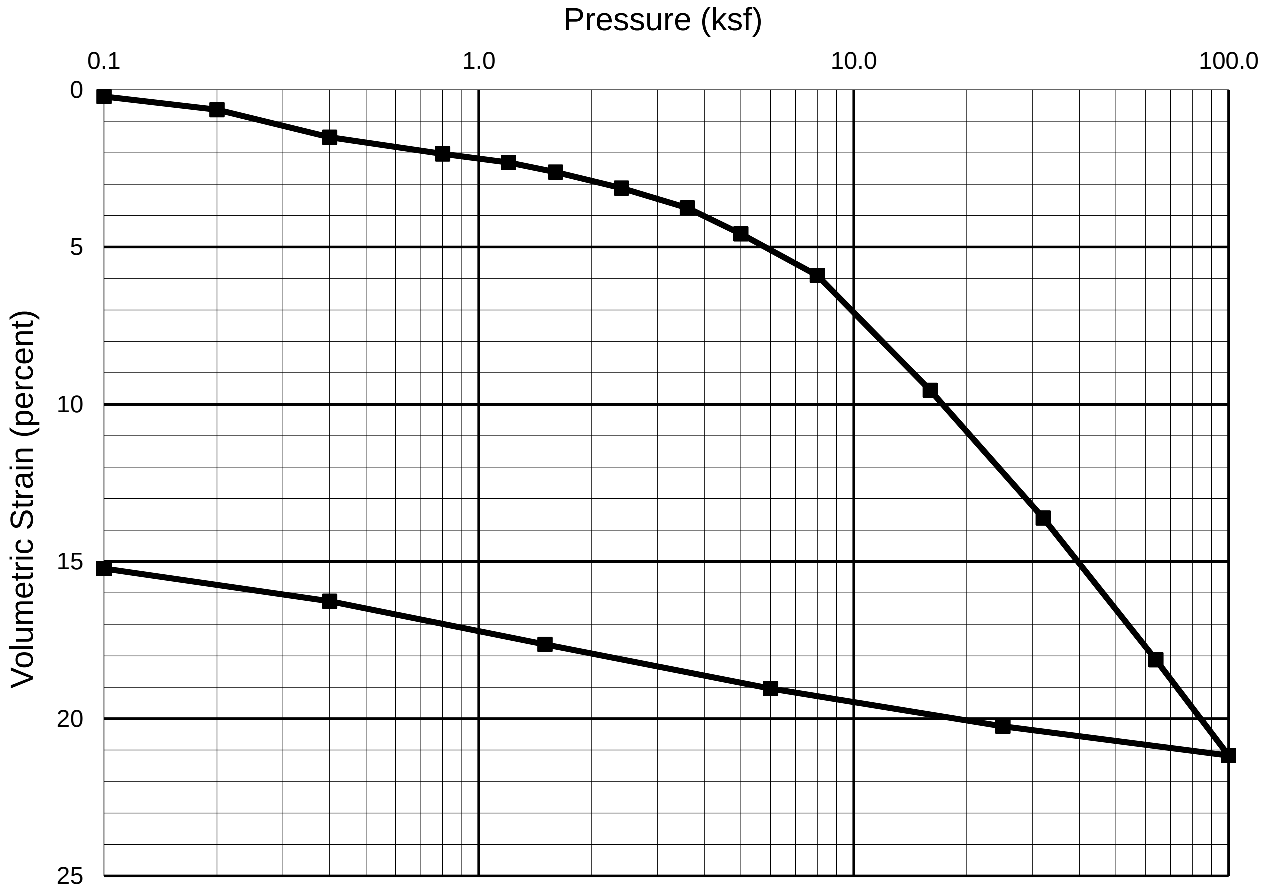
Sampler Type: Sprague & Henwood			Condition		Before Test			After Test	
Diameter (in) 2.42	Height (in)	1.00	Water Content	w_o	23.4	%	w_f	17.4	%
Overburden Pressure, p_o	3,900	psf	Void Ratio	e_o	0.66		e_f	0.47	
Preconsol. Pressure, p_c	8,100	psf	Saturation	S_o	97	%	S_f	100	%
Compression Ratio, $C_{\varepsilon c}$	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				

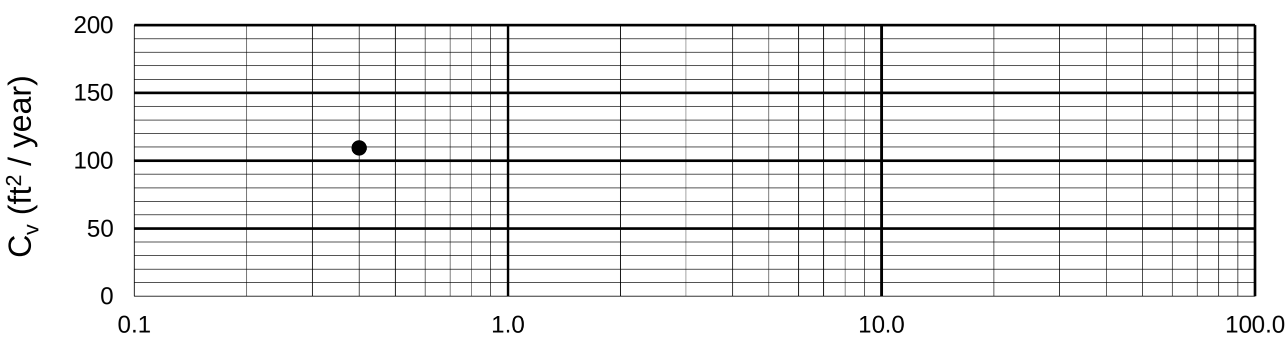
NORTH TOWN
San Jose, California

CONSOLIDATION TEST REPORT

LANGAN

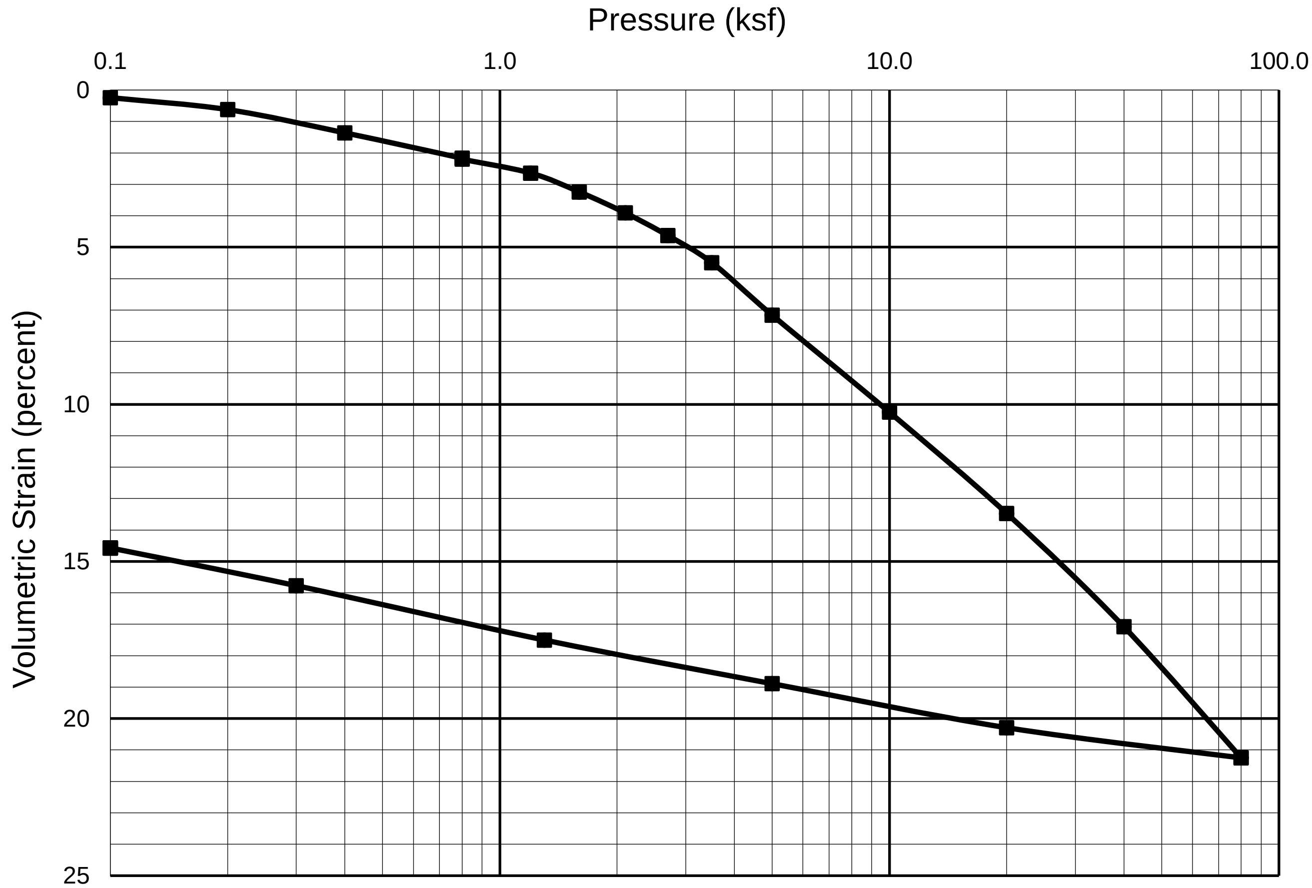
Date	03/26/19	Project No.	770651903	Figure	C-1

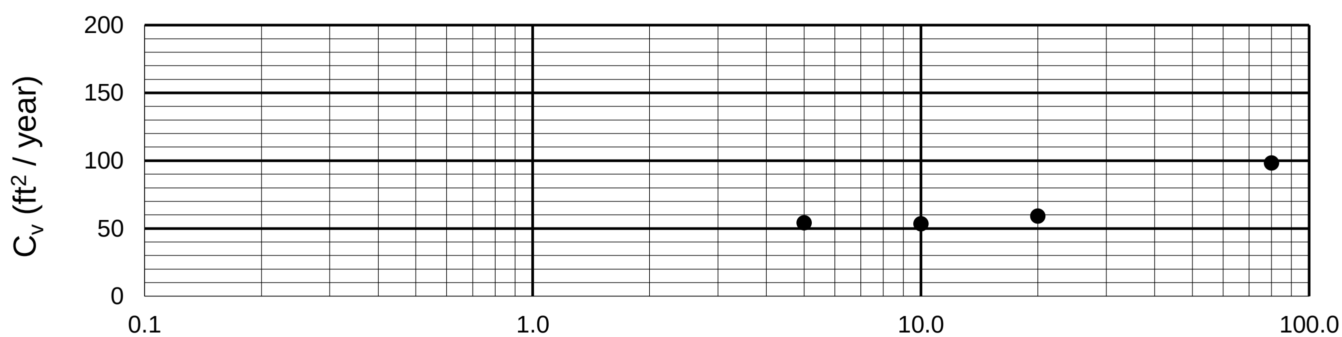




Sampler Type: Shelby Tube				Condition		Before Test			After Test	
Diameter (in)	2.42	Height (in)	1.00	Water Content	w_o	29.6	%	w_f	20.4	%
Overburden Pressure, p_o	2,350	psf		Void Ratio	e_o	0.83		e_f	0.55	
Preconsol. Pressure, p_c	9,000	psf		Saturation	S_o	96	%	S_f	100	%
Compression Ratio, $C_{\varepsilon c}$	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					

NORTH TOWN San Jose, California	**CONSOLIDATION TEST REPORT**
LANGAN	Date 04/03/19 · Project No. 770651903 · Figure C-2





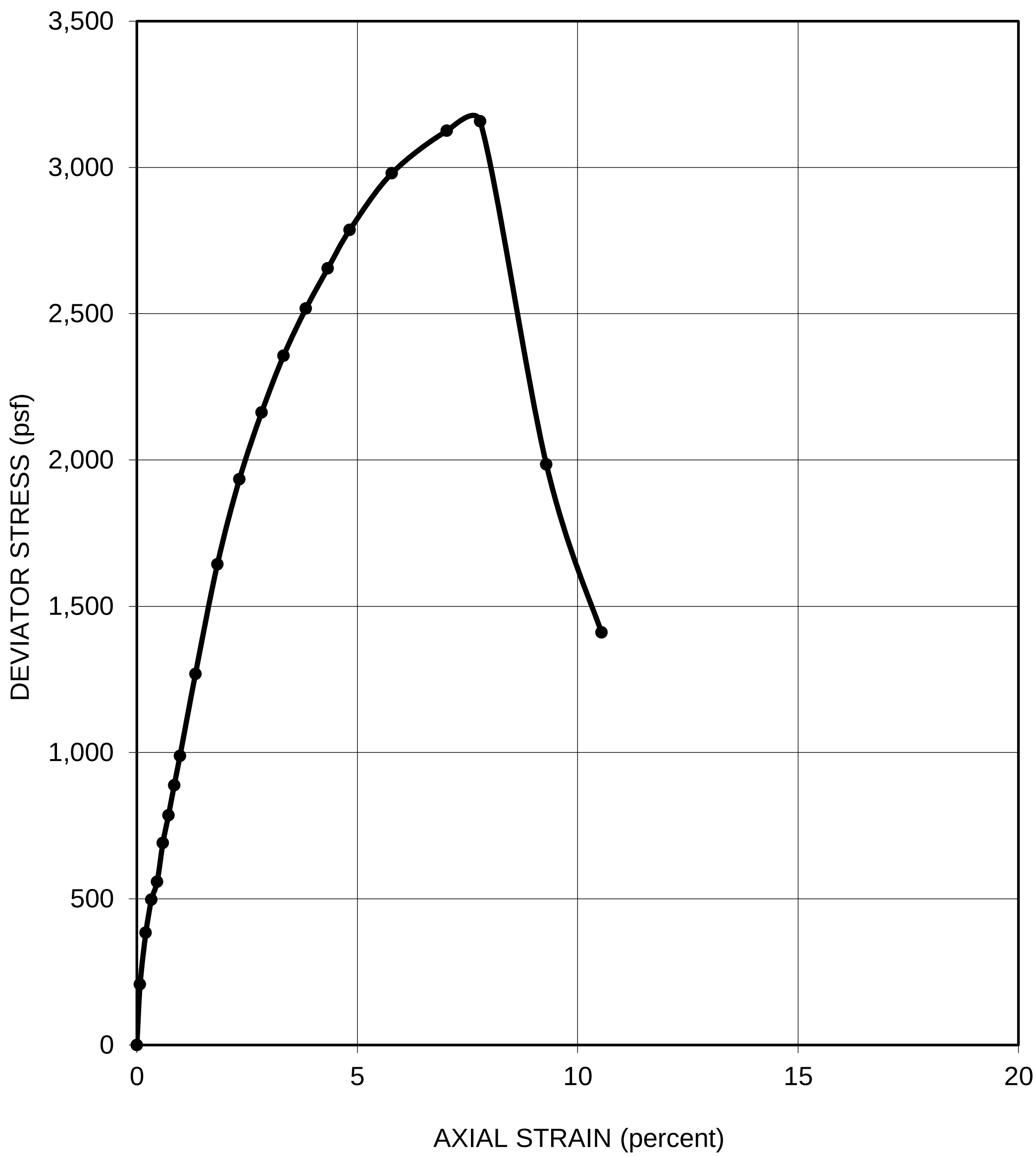
Sampler Type: Sprague & Henwood				Condition		Before Test		After Test	
Diameter (in)	2.42	Height (in)	1.00	Water Content	w_o	26.7	%	w_f	18.9 %
Overburden Pressure, p_o	1,140	psf		Void Ratio	e_o	0.77		e_f	0.51
Preconsol. Pressure, p_c	3,300	psf		Saturation	S_o	94	%	S_f	100 %
Compression Ratio, $C_{\varepsilon c}$	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			

NORTH TOWN
San Jose, California

CONSOLIDATION TEST REPORT

LANGAN

Date	03/26/19	Project No.	770651903	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	

NORTH TOWN
San Jose, CA

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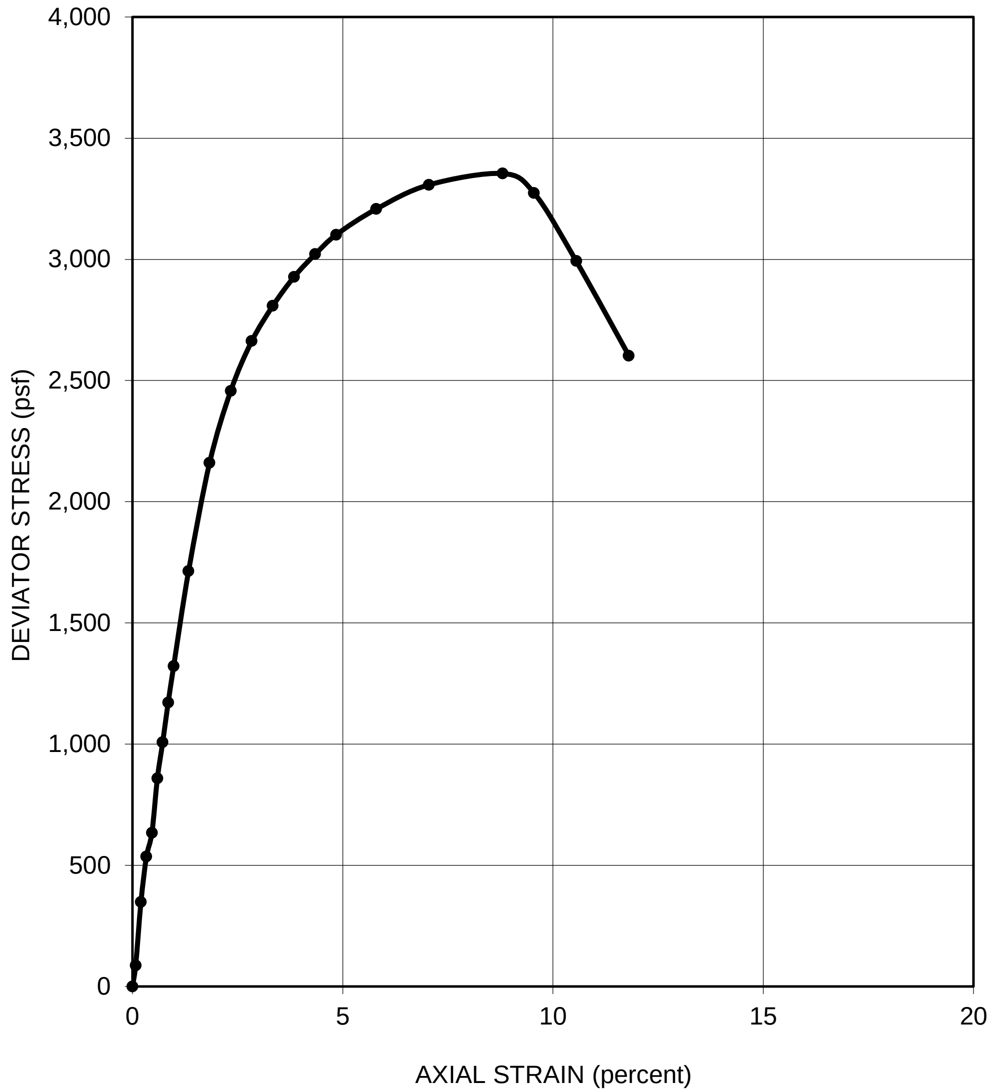
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 05/29/19 | Project No. 770651903 | 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

NORTH TOWN
San Jose, CA

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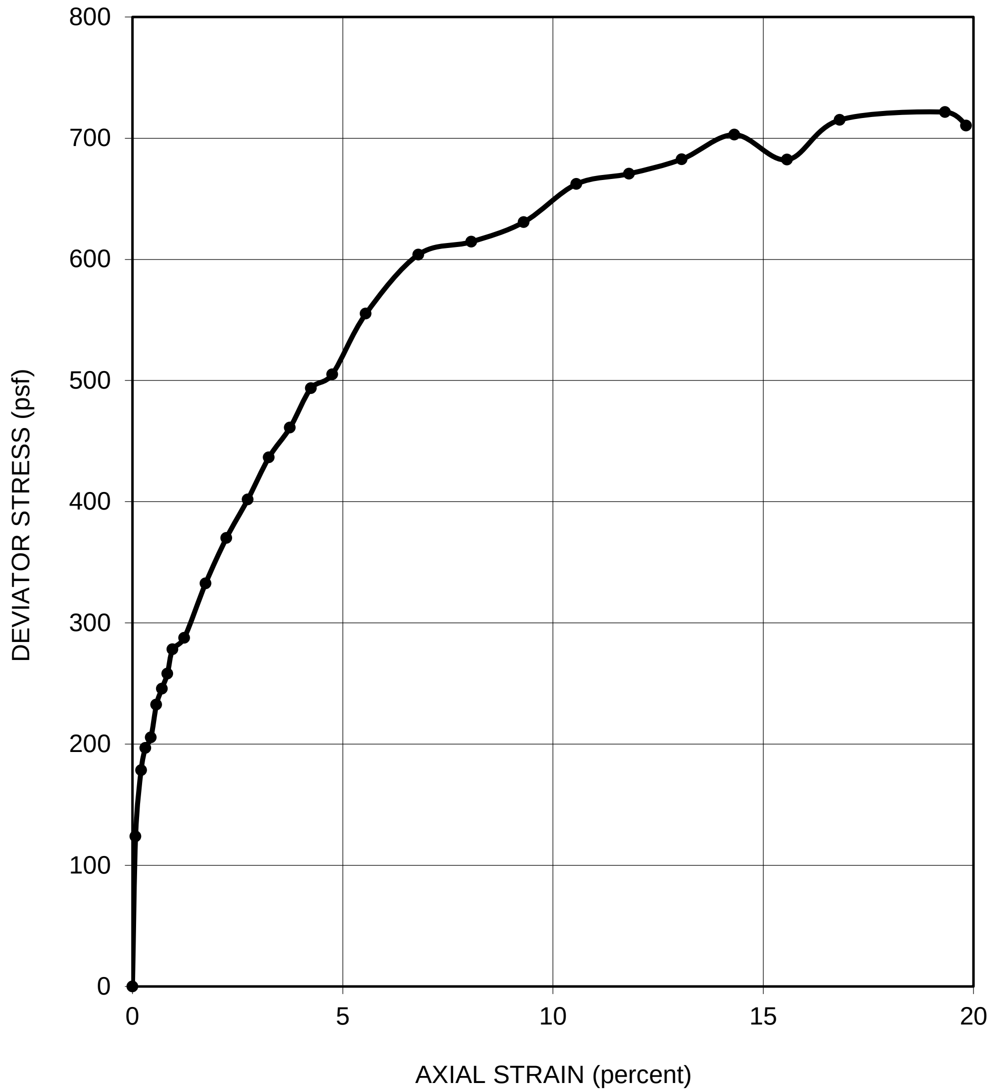
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 05/29/19 | Project No. 770651903 | 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

NORTH TOWN
San Jose, CA

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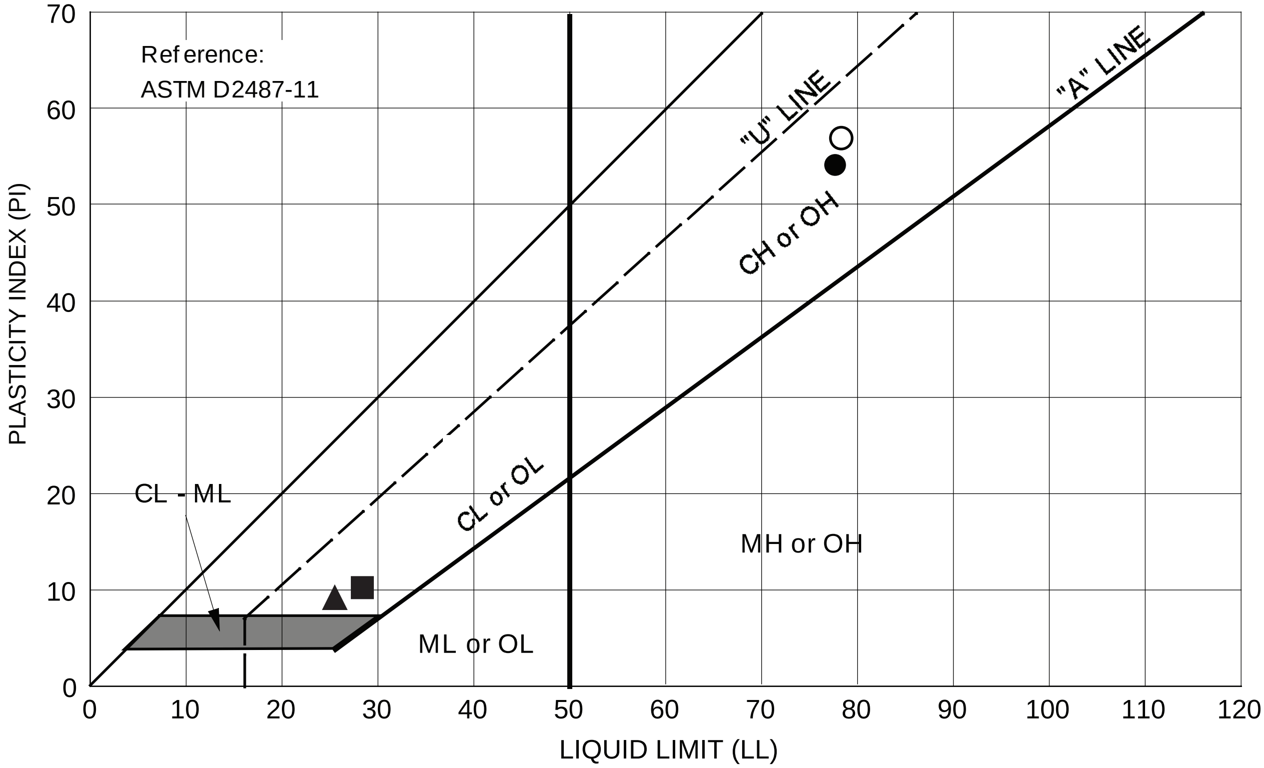
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 05/29/19 | Project No. 770651903 | Figure C-6



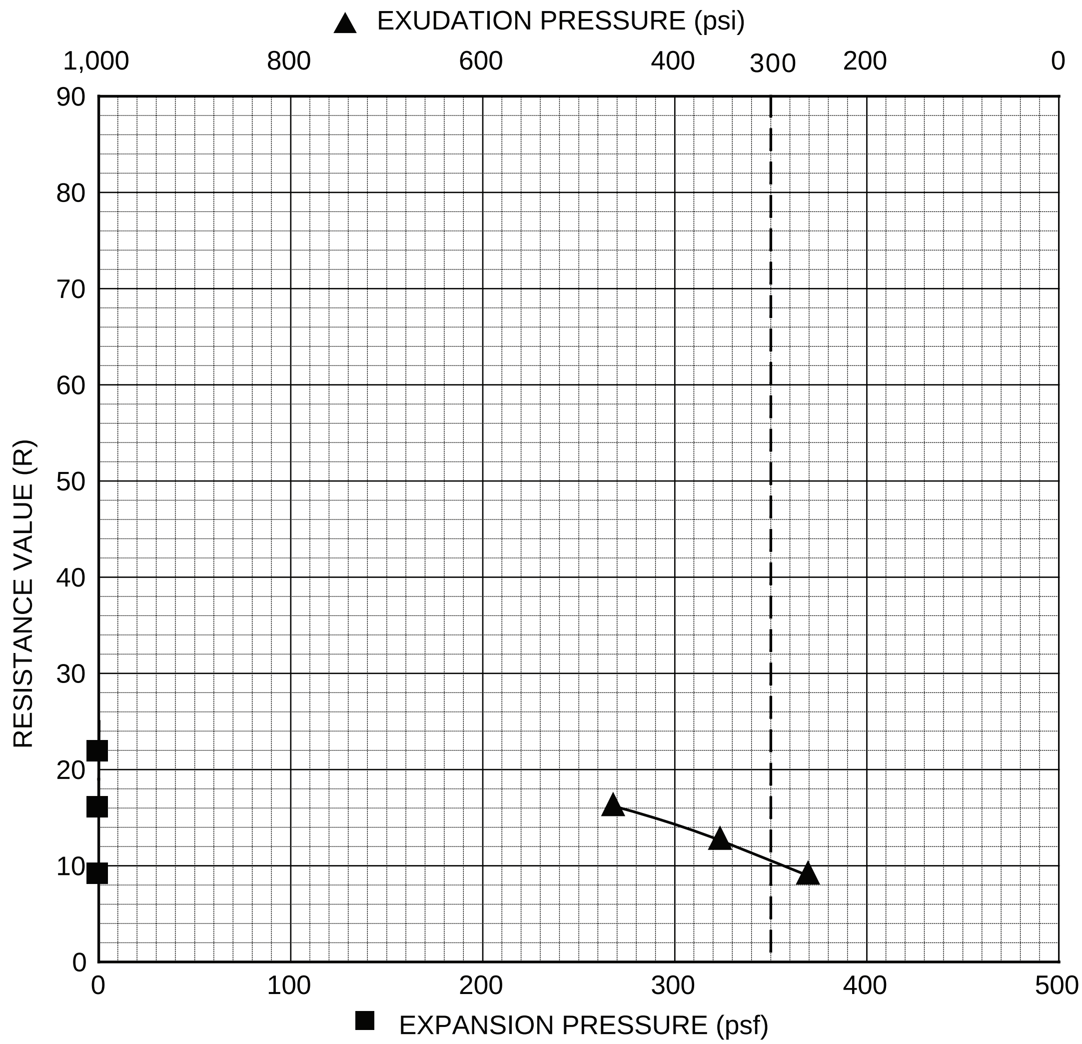
Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	B-1 at 1 - 5 feet	CLAY (CH), dark brown	--	77	54	--
■	B-3 at 15 feet	CLAYEY SAND (SC), olive-gray	14.1	28	10	21.3
▲	B-4 at 14.5 feet	CLAYEY SAND (SC), gray-brown with orange	21.5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

NORTH TOWN
San Jose, California

LANGAN

PLASTICITY CHART

Date 05/20/19 | Project No. 770651903 | 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

NORTH TOWN
San Jose, California

LANGAN

RESISTANCE VALUE TEST DATA

Date 05/20/19 | Project No. 770651903 | Figure C-8

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

Cheryl McMillen
Laboratory Director

* Results Reported on "As Received" Basis
N.D. - None Detected

<u>Quality Control Summary</u> - All laboratory quality control parameters were found to be within established limits

12601

LANGAN

CHAIN OF CUSTODY RECORD

Page 1 of 1

CA1903076

- [ ] 555 Montgomery Street, Suite 1300, San Francisco, CA 94111
- [ ] 501 14th Street, Third Floor, Oakland, CA 94612
- [ ] 3320 Data Drive, Suite 350, Rancho Cordova, CA 95670-7982
- [x] 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 Lecas

Turnaround Time: Standard

	Field Sample Identification No.	Date	Time	Lab Sample No.	Matrix: Soil	Water	Air	Other	No. Containers & Preservative: HCL	H_2SO_4	HNO_3	Ice			Analysis Requested: Corrosion Ev	Brief Fuel											Silica gel clean-up	Hold	Remarks
001	B-1 SA 4'	3/6/19		1	X										X	X													
002	E5 S1 6'	3/7/19		2	X										X	X													TUBE

Relinquished by: (Signature)	Date:	Time	Received by: (Signature)	Date	Time
Chad Lecas	3/9/19		[signature]	3/12/19	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:

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1 copy:	Mr. Scott Landsittel LBA Realty 1 Embarcadero Center, Suite 710 San Francisco, California 94111