APPENDIX D

Preliminary Geotechnical Investigation

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Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

April 16, 2018
Project No. 403246001

Ms. Emily Mandrup
LBA Realty Fund VI, L.P.
3347 Michelson Drive, Suite 200
Irvine, California 92612

Subject: Preliminary Geotechnical Evaluation
1605 Industrial Avenue
San Jose, California

Dear Ms. Mandrup:

In accordance with your authorization, we have performed a preliminary geotechnical evaluation for the property located at 1605 Industrial Avenue in San Jose, California (Figure 1). The subject property is relatively flat, irregular in shape and covers an area of about 10.2 acres, and is located along the eastern side of Interstate 880 just north of the Interstate 880 and Highway 101 interchange. It is our understanding that future development of the property may include construction of concrete tilt-up warehouses with office space with structure footprints varying from about 30,000 to 100,000 square feet.

This report presents the findings and conclusions from our preliminary geotechnical evaluation for the subject property. The results of our geologic hazards assessment are provided in our report for the property dated March 26, 2018.

SCOPE OF SERVICES

Our scope of services included the following:

- Review of readily available background materials, including geologic maps, aerial photographs, topographic data, and hazard maps.
- Site reconnaissance to observe the general site conditions, and to mark the locations for our subsurface exploration.
- Coordination with Underground Service Alert (USA) to locate underground utilities in the vicinity of our subsurface exploration.
- Subsurface exploration consisting of four (4) cone penetration test (CPT) soundings. The upper 5 feet of each CPT was excavated with hand-auger equipment. A representative of Ninyo & Moore logged the subsurface conditions exposed in the upper 5 feet of each CPT and collected bulk soil samples for laboratory testing.
- Performed a percolation test at a depth of 2 feet below the existing grade to assess infiltration

rates.

- Laboratory testing on selected samples to evaluate in-situ soil moisture content, Atterberg limits, expansion index, particle size distribution, and soil corrosivity.
- Compilation and engineering analysis of the field and laboratory data, and the findings from our background review.
- Preparation of this report presenting our findings and conclusions from our preliminary evaluation, and our preliminary geotechnical recommendations for design of the proposed improvements.

FIELD EXPLORATION AND LABORATORY TESTING

Our field exploration for this study included a preliminary subsurface exploration of the proposed site, which was conducted on April 3, 2018. The subsurface exploration consisted of four CPT soundings and one shallow excavation for percolation testing. The approximate locations of the CPT soundings and percolation test are shown on Figure 2. Prior to commencing the subsurface exploration, USA was notified for field marking of the existing utilities.

The CPT soundings were performed on April 3, 2018 using a truck-mounted rig with a 20-ton reaction capacity. After hand excavation to a depth of about 5-feet, the soundings were advanced to depths of approximately 45 feet below the existing grade. Cone tip resistance, sleeve friction, and pore pressure were electronically measured and recorded at vertical intervals of approximately 2 inches while the cone was advanced. The soil behavior type index (I_c) and corresponding soil behavior for the subsurface materials encountered was assessed using correlations (Robertson et al., 1986) based on the cone penetration data and sleeve friction. The CPT soundings were backfilled with grout, and the CPT sounding logs are presented in Appendix A.

Laboratory testing of soil samples recovered from the borings included tests to evaluate in-situ soil moisture content, Atterberg limits, expansion index, particle size distribution, and soil corrosivity. The results of the laboratory tests are presented in Appendix B.

GENERAL GEOLOGIC AND SUBSURFACE CONDITIONS

According to regional geologic maps covering the subject property, the site is underlain by Holocene age alluvial soils deposited by nearby Guadalupe and Coyote Creeks (Helley et al., 1994; Knudsen et al., 2000; Wesling and Helley, 1989; and Witter et al., 2006). These deposits typically consist of silt and clay interspersed with layers of sand and gravel. The silt and clay deposits can compress under heavy loads and are also expansive.

The surface of the site is covered by asphalt concrete pavement, concrete pavement, and aggregate base. The asphalt concrete pavement encountered in CPT-1 consisted of approximately

7 inches of asphalt concrete over 2 inches of aggregate base. The other CPT soundings encountered aggregate road base at the surface that varied from about 6 to 12 inches in thickness.

Our CPT soundings encountered alluvial deposits that consisted of layers of silt and clay in the upper to 40 to 45 feet with occasional layers of sand and gravelly sand below depths of 40 feet (Appendix A). Sand and gravelly sand layers were encountered in CPT-2 and CPT-3 from depths of about 41 to 45 feet (the total depth of exploration). A thin layer of sand was encountered in CPT-4 at a depth of about 44 feet.

Groundwater was measured in the CPT soundings at depths ranging from 6 to 9 feet below the ground surface. Regional records indicate that the historically high groundwater level is less than 10 feet below the ground surface (CGS, 2002).

Fluctuations in the groundwater level across the site and over time may occur due to seasonal precipitation, variations in topography or subsurface hydrogeologic conditions, or as a result of changes to nearby irrigation practices or groundwater pumping. In addition, seeps may be encountered at elevations above the observed groundwater levels due to perched groundwater conditions, leaking pipes, preferential drainage, or other factors not evident at the time of our exploration.

PRELIMINARY GEOTECHNICAL CONSIDERATIONS

Our geologic hazards assessment for the subject property (Ninyo & Moore, 2018) identified strong ground motion, liquefaction, dynamic settlement, and expansive soils as the main geologic and seismic related hazards that could impact site development. As part of this study, we evaluated settlement caused by liquefaction, expansive soil characteristics and the infiltration characteristics of the near surface soils. These issues are summarized in the following sections.

Liquefaction and Strain Softening

The strong vibratory motions generated by earthquakes can trigger a rapid loss of shear strength in saturated, loose, granular soils of low plasticity (liquefaction) or in wet, sensitive, cohesive soils (strain softening). Liquefaction and strain softening can result in a loss of foundation bearing capacity, or lateral spreading of sloping or unconfined ground. Liquefaction can also generate sand boils leading to subsidence at the ground surface. Liquefaction (or strain softening) is generally not a concern at depths more than 50 feet below ground surface. The site is located within a State of California (CGS, 2007) liquefaction hazard zone as identified in our geologic hazards assessment.

We encountered deposits of sand and fine-grained soil of low plasticity below the groundwater level during our subsurface exploration. We evaluated the potential for liquefaction using in-house developed spreadsheets developed in accordance with the methods presented by Idriss and Boulanger (2008) using the CPT data collected during our subsurface exploration, a design groundwater level of 5 feet below the ground surface, and considering a seismic event producing a PGA of 0.505g resulting from a Magnitude 7.3 earthquake. The results of our analysis, presented in Appendix C, indicate that thin layers of sandy and silty soil below the assumed design groundwater level will liquefy under the considered ground motion based on a factor of safety against liquefaction of less than one. The analysis assumed that borderline soils (with Soil Behavior Type index, Ic, between 2.4 and 2.6) are susceptible to liquefaction. However, the results of our laboratory testing on soils from depths of 1 to 5 feet indicate that the soils tested are generally not susceptible to liquefaction, based on the ratio of the in-situ water content to the Atterberg limits liquid limit (Idriss and Boulanger, 2008). We anticipate that testing of soils from depths of 5 to 8 feet would have similar results and thereby provide justification for not including these borderline soils in the liquefaction analysis. Due to the depth and relative thickness of the other liquefiable layers, we do not regard the potential for liquefaction-induced reduction in the bearing capacity of shallow foundations as a design consideration for the project. Other consequences of liquefaction, including dynamic settlement, sand-boil-induced ground subsidence, and lateral spreading, are addressed in the following sections.

Estimates of undrained and remolded shear strength based on CPT tip resistance and sleeve friction, respectively, indicate that the cohesive soils during our subsurface exploration are not particularly sensitive. As such, we do not regard seismically induced strain-softening behavior as a design consideration.

Dynamic Settlement

The strong vibratory motion associated with earthquakes can also dynamically compact loose granular soil, leading to surficial settlements. Dynamic settlement is not limited to the near surface environment and may occur in both dry and saturated sand and silt. Cohesive soil is not typically susceptible to dynamic settlement.

We evaluated the potential for dynamic settlement for layers with factor of safety against liquefaction of 1.3 or less using the CPT data collected during our subsurface exploration and an in-house developed spreadsheet program based on the method presented by Zhang et al. (2002) for saturated soil and by Robertson and Shao (2010) for dry soil. Our analysis considered a Magnitude 7.3 earthquake producing a PGA of 0.505g and groundwater level at 5 feet below the ground surface. The results of our analyses, presented in Appendix C, indicate that the total dynamic settlement following the considered seismic event will be approximately 1 inch.

Differential dynamic settlement is estimated to be on the order of about ½ inch over a horizontal distance of approximately 30 feet.

Static Settlement

We anticipate that the proposed improvement foundation loads will be relatively low to moderate and that significant changes to the site grade are not proposed. We performed an analysis to evaluate the potential static settlement due to sustained loads. The results of our analysis indicate that the total static settlement will be up to approximately 1 inch for column loads of up to 50 kips and wall loads up to 5 kips/foot. We anticipate, therefore, that conventional shallow foundations will be suitable to support the proposed structures.

Corrosive/Deleterious Soil

An evaluation of the corrosivity of the on-site material was conducted to assess the impact to concrete and metals. The corrosion impact was evaluated using the results of limited laboratory testing on samples obtained during our subsurface study. Laboratory testing to quantify pH, resistivity, chloride, and soluble sulfate contents was performed on a sample of the fill. The results of the corrosivity tests are presented in Appendix B. California Department of Transportation (Caltrans) defines a corrosive environment as an area within 1,000 feet of brackish water or where the soil contains more than 500 parts per million (ppm) of chlorides, sulfates of 0.2 (2,000 ppm) percent or more, or pH of 5.5 or less (Caltrans, 2018). The laboratory test results indicate that the sulfate exposure to concrete is negligible and the site does not meet the definition of a corrosive environment, based on the criteria considered, with no brackish body of water within 1,000 feet of the site. Exposed ferrous metals will undergo corrosion but conventional measures to mitigate corrosion, such as galvanization or reliance on a corrosion allowance, should be effective. A corrosion engineer may be consulted to provide specific guidance.

Expansive Soils

Some clay minerals undergo volume changes upon wetting or drying. Unsaturated soils containing those minerals will shrink/swell with the removal/addition of water. The heaving pressures associated with this expansion can damage structures and flatwork. Laboratory testing was performed on a select sample of the near-surface soil to evaluate the expansion index. The tests were performed in general accordance with the American Society of Testing and Materials (ASTM) Standard D 4829 (Expansion Index). The results of our laboratory testing indicate that the expansion index of the near-surface soil ranges from 69 to 77, which is consistent with a medium expansion characteristic. To reduce the potential for differential movement and distress to the proposed improvements due to shrink/swell behavior, foundations should be designed for expansive soils. We anticipate that suitable foundation embedment depths and subgrade preparation can be used to mitigate the expansive soil conditions.

Infiltration Characteristics

Ninyo & Moore performed percolation testing in the central portion of the site to evaluate the rate of infiltration on site for design of storm water management systems. The percolation test procedures utilized are presented in Appendix D. The test results, presented in Appendix D and summarized in Table 1, indicate that the infiltration rate of the near surface soil on site is very slow. Due to the variability of subsurface materials encountered during our exploration, variability in subsurface infiltration should be anticipated.

Table 1 – Percolation Test Results

Test (Boring)	Test Depth (ft)	Subsurface Conditions	Percolation Rate (inch/hour)	Infiltration Rate[1] (inch/hour)
P-1	2	Lean Clay	0.8	0.14

[1] Infiltration rate is percolation rate adjusted by a reduction factor to exclude percolation through sides of test hole.

CONCLUSIONS

Based on the results of our preliminary geotechnical evaluation, it is our opinion that redevelopment of the existing site is geotechnically feasible. Geotechnical considerations for future phases of the project include the following:

- Our subsurface exploration encountered alluvial deposits. The alluvial deposits, as encountered, generally consisted of layers of silt and clay with occasional layers of sand and gravelly sand at depth. The sand and gravelly sand layers were encountered depths below 40 feet.
- Groundwater was encountered in the CPT soundings at depths ranging from approximately 6 to 9 feet below the existing grade during our subsurface exploration. Variation and fluctuation in groundwater levels should be anticipated.
- The site could experience a relatively large degree of ground shaking during a significant earthquake on a nearby fault.
- Historic records indicate that ground effects consisting of settlement, lateral spreading, ground cracking, and broken pipes were reported in the vicinity of the site in response to past seismic activity.
- The results of our liquefaction analysis, presented in Appendix C, indicate that relatively thin layers of granular soil between depths of approximately 5 to 8 feet and 40 to 45 feet will liquefy under the considered ground motion. We anticipate that future laboratory testing of soils from depths of 5 to 8 feet would have similar results as our laboratory testing of soils from 1 to 5 feet and thereby provide justification for not including these borderline soils in the liquefaction

analysis. The potential for bearing capacity reduction due to deeper liquefaction is not a consideration based on the depth of the liquefiable soils.

- The results of our dynamic settlement analysis, presented in Appendix C, indicate that dynamic settlement following the seismic event considered will be relatively minor with approximately 1 inch of total dynamic settlement and a differential of about ½ inch over a horizontal distance of approximately 30 feet.
- We do not regard settlement due to sustained loading as a design consideration for light to moderately loaded structures provided that any loose surficial materials or undocumented fill is mitigated by remedial grading and the improvements are constructed at or below the existing grade.
- Based on the results of our soil corrosivity tests during this study and the Caltrans corrosion guidelines (2018), the site does not meet the definition of a corrosive environment.
- Expansion Index testing indicates that the near-surface soil on site has a medium expansion characteristic. We anticipate that suitable foundation embedment depths and subgrade preparation can be used to mitigate the expansive soil conditions as part of future site development.
- Our percolation testing at a depth of 2 feet below the existing grade indicates that the infiltration rate of the near-surface soils is very slow.

PRELIMINARY RECOMMENDATIONS

The following section presents our preliminary recommendations for the project, including preliminary earthwork guidelines and conceptual foundation recommendations.

Foundation Selection

Based on the results of our liquefaction, dynamic settlement, and static settlement analysis, we anticipate that the settlements due to liquefaction and building loads from the lightly loaded warehouse buildings that are currently under consideration will be tolerable for structures founded on shallow foundations. We do not anticipate the need for deep foundations based on the proposed building type.

Preliminary Earthwork Recommendations

Based on the results of our laboratory testing and observations, it is our opinion that the near surface soil should be suitable for reuse as general fill provided that it is processed, as-needed, to exclude rocks or lumps more than 3-inches in median dimension, and trash, debris, vegetation or other deleterious material that may be encountered. The aggregate base that overlies many of the interior access roads should also be suitable for reuse as general fill as described above. Reuse of the material as aggregate base for roadway or parking lot use would require additional testing. Based on our experience, reuse of the aggregate base for such purposes is not practical; however, reuse of the material at the surface as general fill may increase the resistance (R-value) of the subgrade and possibly reduce the thickness of structural pavement sections for drive lanes and parking areas.

To reduce the potential for differential settlement due to variable conditions and support characteristics, any loose surficial materials and undocumented fill under new footings slabs should be improved by remedial grading (removal and replacement with compacted fill). The depth and lateral extent of removal and replacement should be determined during future geotechnical design level evaluations.

In order to reduce the impacts of the expansion characteristics of the on-site soils, foundations should have embedment depths of 2 feet or more and subgrades for slab-on-grades, fill, flatwork, and pavement should be compacted to 2, or more, percentage points above the laboratory optimum moisture content. To further reduce the impacts of the expansion characteristics of the on-site soils, importing non-expansive select materials should be considered below slab-on-grades, flatwork, and pavement. As an alternative, the on-site expansive soils can be treated with quicklime to reduce their expansion characteristics.

Preliminary Asphalt Concrete Pavements

Recommended alternative asphalt pavement structural sections, based on the methodology in Caltrans Highway Design Manual, are provided in Table 1 for a range of traffic levels. The designer may interpolate between the values provided for an intermediate traffic level once the design traffic level has been selected for the pavement. The design subgrade R-value was selected based on characteristic values for the materials encountered during the subsurface exploration for this project. The pavement subgrade should be observed by the geotechnical during grading to check that the exposed materials are consistent with the support characteristics assumed for pavement design. The asphalt pavements were evaluated for a service life of 20 years presuming that periodic maintenance, including crack sealing and resurfacing, will be performed during the design service life. Premature deterioration may occur without periodic maintenance.

Table 1 – Asphalt Concrete Pavement Sections				
Traffic Index	R-value	Alternative 1	Alternative 2	Alternative 3
5.0	5	3 inches AC 12 inches AB	3 inches AC 12 inches AB SEG	3 inches AC 6 inches AB 12 inches LTS
7.5	5	4½ inches AC 17 inches AB	4½ inches AC 13 inches AB SEG	4½ inches AC 10 inches AB 12 inches LTS
10.5	5	6½ inches AC 25 inches AB	6½ inches AC 19 inches AB SEG	6½ inches AC 14 inches AB 12 inches LTS

Notes:

[1] AC is Type A, Dense-Graded Hot Mix Asphalt complying with Caltrans Standard Specification 39-2 (2010).
[2] AB is Class II Aggregate Base complying with Caltrans Standard Specification 26-1.02 (2010).
[3] SEG is subgrade enhancement geotextile such as Mirafi 600X or equivalent.
[4] LTS is lime-treated subgrade.

Aggregate base should be placed in lifts of no more than 8 inches in loose thickness, moisture conditioned as-needed to approximately 2 percentage points above the optimum moisture content, and compacted to 95 percent of the reference density on a dry density basis as evaluated by American Society for Testing and Materials (ASTM) standard D1557. Asphalt concrete should be placed and compacted in accordance with Section 39 of the Caltrans Standard Specification (2010) to not less than 91 percent and not more than 97 percent of the reference density as evaluated by ASTM D2041 on a wet density basis. Pavements should be sloped so that runoff is diverted to an appropriate collector (concrete gutter, swale, or area drain) to reduce the potential for ponding of water on the pavement. Concentration of runoff over aphalt pavement should be discouraged.

LIMITATIONS

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

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

This report is intended for conceptual design purposes only. It does not provide sufficient data to prepare an accurate bid by contractors.

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

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

Ninyo & Moore appreciates the opportunity to provide services on this project.

Respectfully submitted,
NINYO & MOORE

David C. Seymour, PG, CEG
Principal Engineering Geologist

Timothy P. Sneddon, PE, GE
Principal Engineer

DCS/TPS/vmn

Attachments: References
Figure 1 – Site Location
Figure 2 – Exploration Locations
Appendix A – CPT Logs
Appendix B – Laboratory Testing
Appendix C – Calculations
Appendix D – Percolation Testing

Distribution: (1) Addressee (via e-mail)

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FIGURES

1_403246001_SL.mxd 4/4/2018 JDL

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE. | SOURCE: ESRI WORLD TOPO, 2018

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FEET

0 1,500 3,000

FIGURE 1

SITE LOCATION

1605 INDUSTRIAL AVENUE
SAN JOSE, CALIFORNIA

403246001 | 4/18

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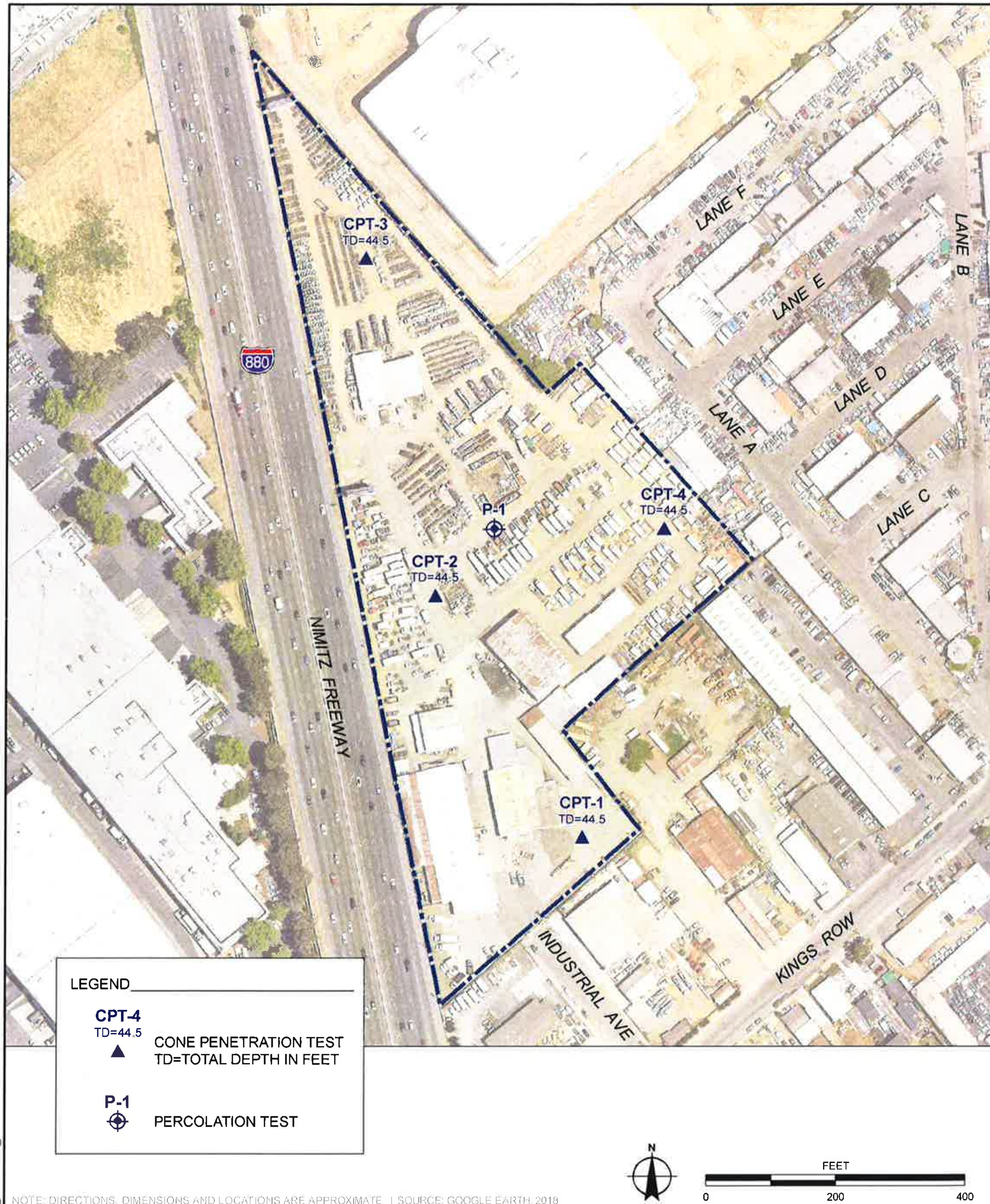

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE | SOURCE: GOOGLE EARTH, 2018

2_403246001_BL.mxd 4/11/2018 JDL

FIGURE 2

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EXPLORATION LOCATIONS

1605 INDUSTRIAL AVENUE
SAN JOSE, CALIFORNIA

403246001 | 4/18

APPENDIX A

Cone Penetration Testing Logs

APPENDIX A

CONE PENETRATION TESTING

Field Procedure for Cone Penetration Testing

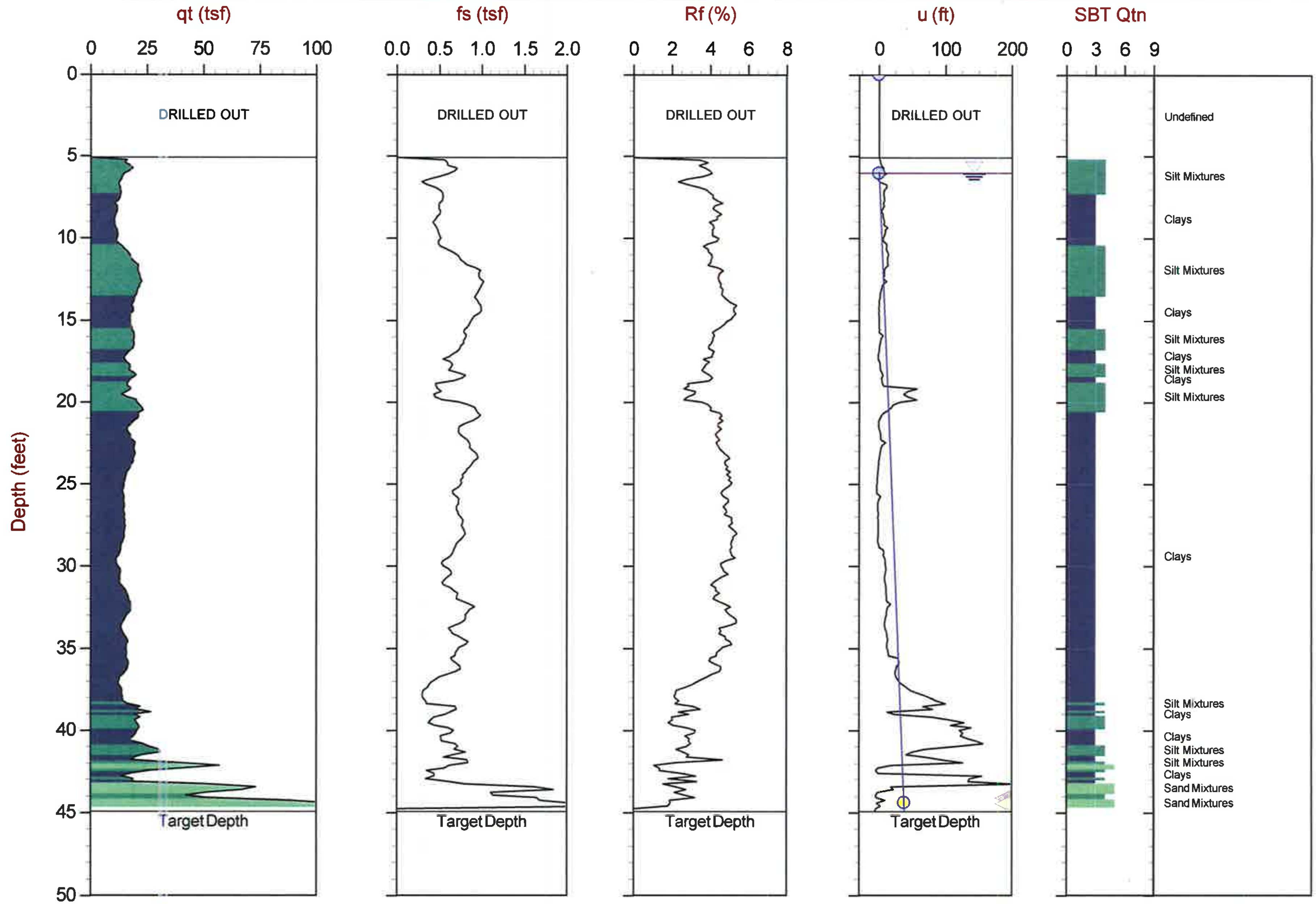
A penetrometer with a conical tip having an apex angle of 60 degrees and a cone base area of 10 square centimeters was hydraulically pushed through the soil using the reaction mass of a 20-ton rig at a constant rate of about 20 millimeter per second in accordance with ASTM D 5778. The penetrometer was instrumented to measure, by electronic methods, the force on the conical point required to penetrate the soil, the force on a friction sleeve behind the cone tip as the penetrometer was advanced, and the pore pressure (Pw) on a transducer behind the cone tip. Penetration data was collected and recorded electronically at intervals of about 2-inches. Cone resistance (Q_c) was calculated by dividing the measured force of penetration by the cone base area. Friction sleeve resistance (F_s) was calculated by dividing the measured force on the friction sleeve by the surface area of the sleeve. The friction ratio (Fs/Qc) was calculated as the ratio of the tip resistance to the sleeve friction. A graph of the computed values of cone resistance (tip) and friction ratio are presented on the logs in the following pages. The tip resistance and friction ratio were used to classify the soil type encountered using the method by Robertson & Campanella (1986). Equivalent SPT blowcounts at a 60 percent energy ratio (N_{60}-values) were calculated from the tip resistance and friction ratio using the method by Jeffries and Davies (1993). A graph of the equivalent N_{60} values (SPT N_{eq}) and the encountered soil types are also presented on the logs in the following pages.

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

Ninyo & Moore

Job No: 18-56040
Date: 2018-04-03 08:00
Site: 1605 Industrial Ave

Sounding: CPT-01
Cone: 483:T1500F15U500



Max Depth: 13.700 m / 44.95 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point
Overplot Item: Assumed Ueq ◁ Dissipation, equilibrium achieved — Hydrostatic Line

File: 18-56040_CP01.COR
Unit Wt: SBTQtn (PKR2009)

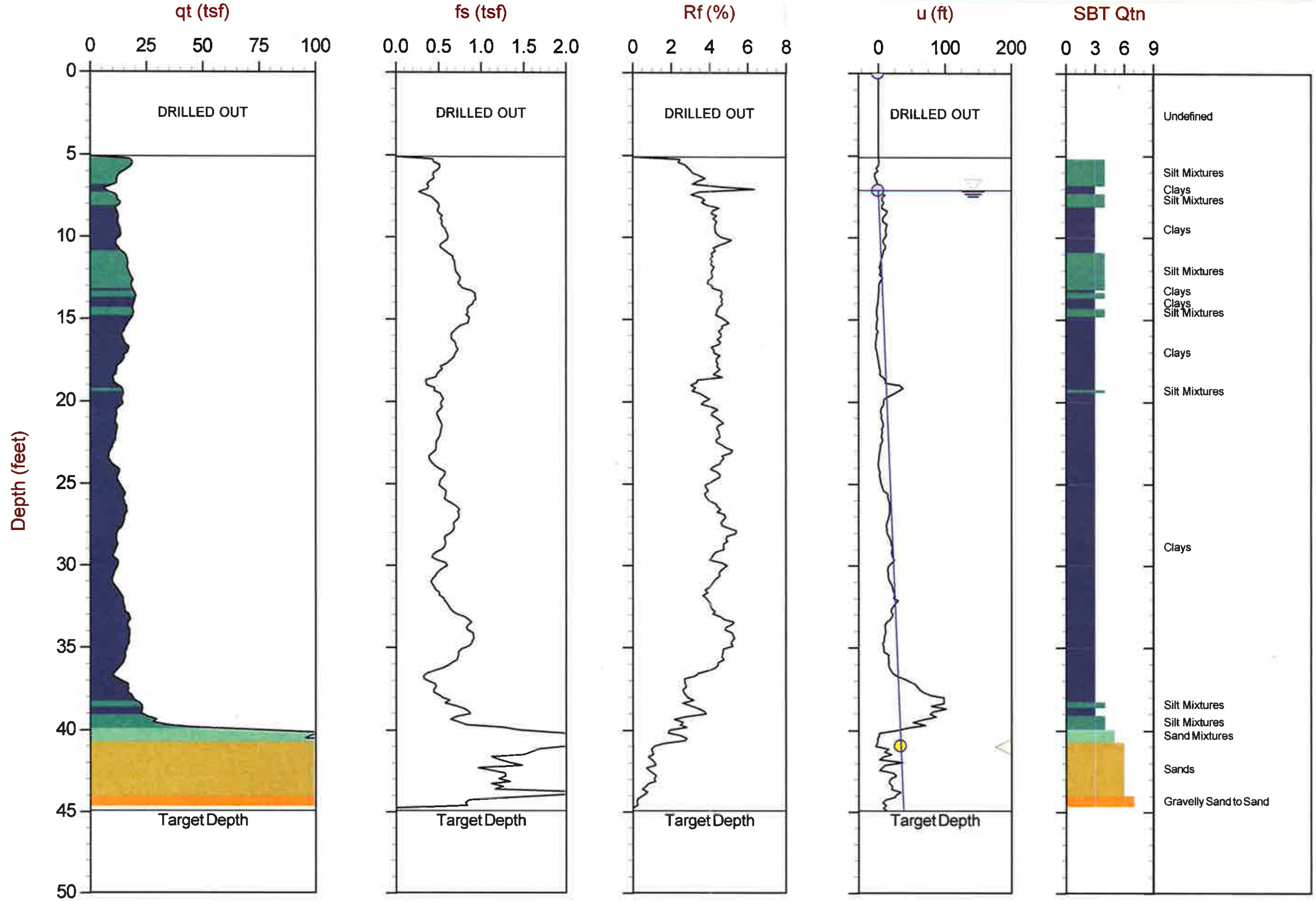
SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4136728m E: 597320m
Page No: 1 of 1

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

Ninyo & Moore

Job No: 18-56040
Date: 2018-04-03 10:46
Site: 1605 Industrial Ave

Sounding: CPT-02
Cone: 483:T1500F15U500



Max Depth: 13.700 m / 44.95 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point

File: 18-56040_CP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4136868m E: 597240m
Page No: 1 of 1

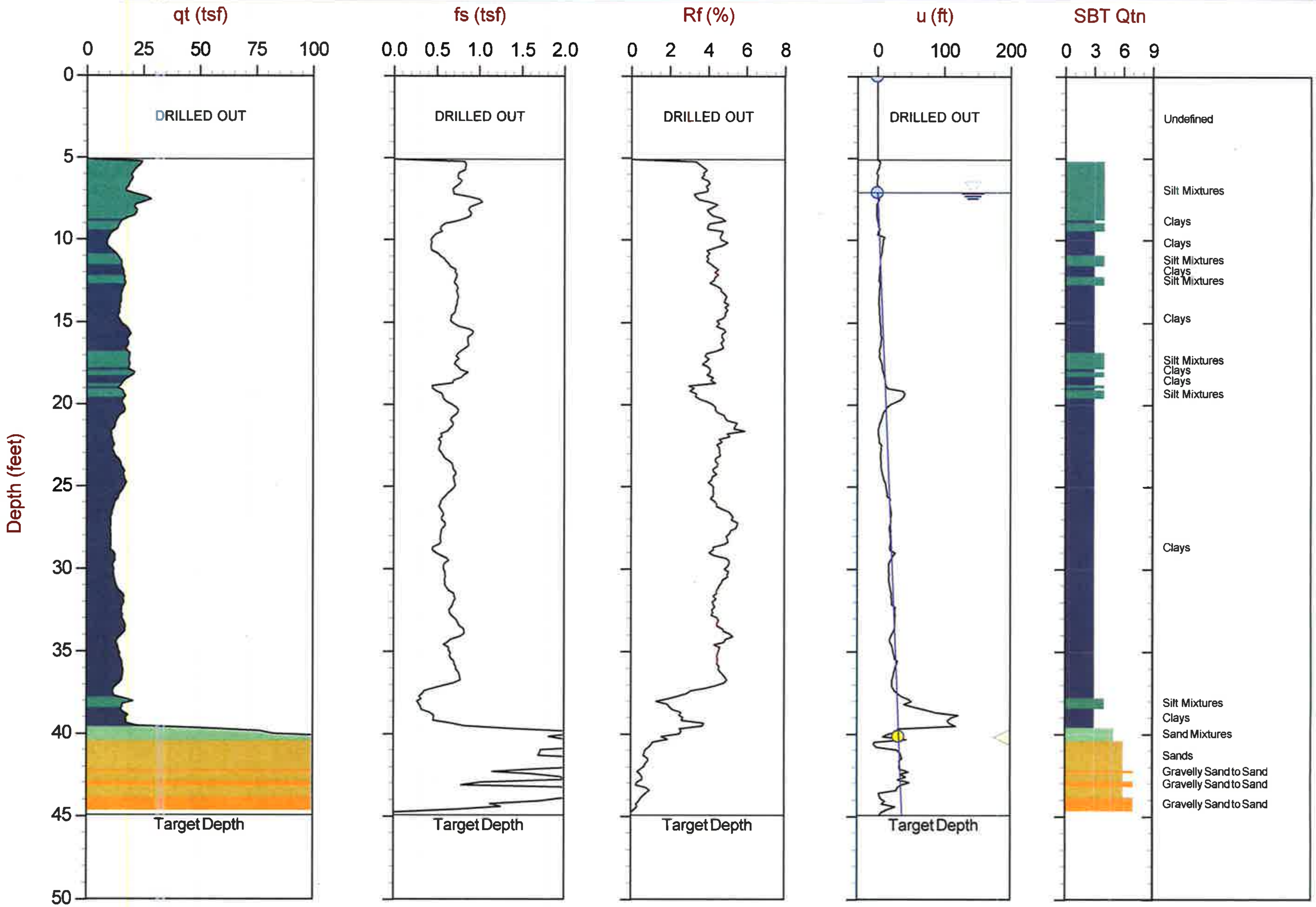
Overplot Item: Assumed Ueq ◁ Dissipation, equilibrium achieved —— Hydrostatic Line

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

Ninyo & Moore

Job No: 18-56040
Date: 2018-04-03 09:53
Site: 1605 Industrial Ave

Sounding: CPT-03
Cone: 483:T1500F15U500



Max Depth: 13.700 m / 44.95 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point
Overplot Item: ○ Assumed Ueq ◁ Dissipation, equilibrium achieved — Hydrostatic Line

File: 18-56040_CP03.COR
Unit Wt: SBTQtn (PKR2009)

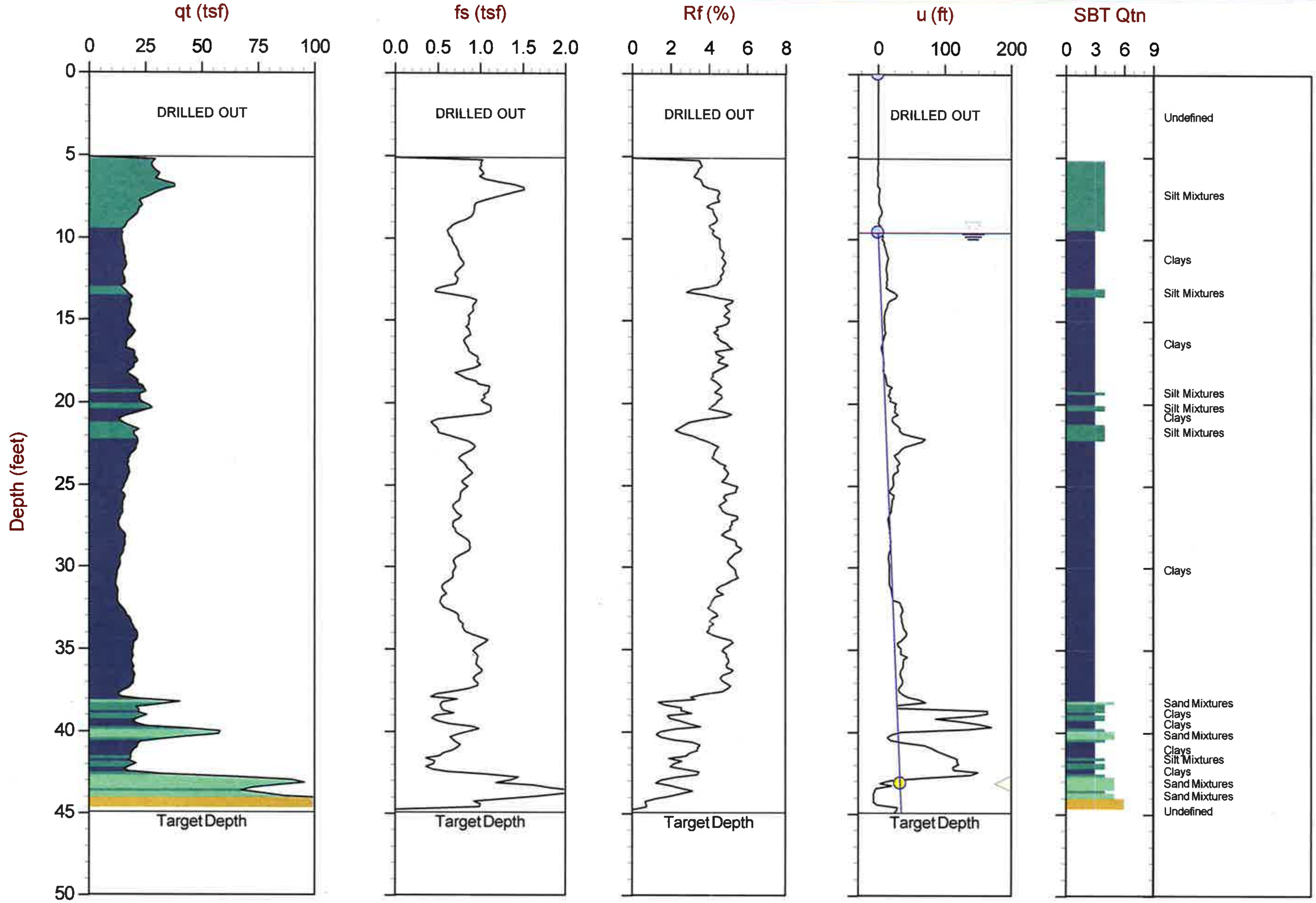
SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4136996m E: 597219m
Page No: 1 of 1

CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

Ninyo & Moore

Job No: 18-56040
Date: 2018-04-03 11:47
Site: 1605 Industrial Ave

Sounding: CPT-04
Cone: 483:T1500F15U500



Max Depth: 13.700 m / 44.95 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: Every Point
Overplot Item: Assumed Ueq ◁ Dissipation, equilibrium achieved — Hydrostatic Line

File: 18-56040_CP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 10 N: 4136870m E: 597359m
Page No: 1 of 1

APPENDIX B

Laboratory Testing

APPENDIX B

LABORATORY TESTING

Classification
Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D 2488

Moisture Content
The moisture content of samples obtained from the exploratory borings was evaluated in accordance with ASTM D 2216.

200 Wash
Evaluations of the percentage of particles finer than the No. 200 sieve in selected soil samples were performed in general accordance with ASTM D 1140. The results of the tests are presented on Figure B-1.

Atterberg Limits
Tests were performed on a selected representative fine-grained soil sample to evaluate the liquid limit, plastic limit, and plasticity index in general accordance with ASTM D 4318. These test results were utilized to evaluate the soil classification in accordance with the USCS. The test results and classifications are shown on Figure B-2.

Expansion Index Test
The expansion index of a selected material was evaluated in general accordance with ASTM D 4829. The specimen was molded under a specified compactive energy at approximately 50 percent saturation (plus or minus 1 percent). The prepared 1 inch thick by 4 inch diameter specimen was loaded with a surcharge of 144 pounds per square foot and inundated with tap water. Readings of volumetric swell were made for a period of 24 hours. The test results are presented on Figure B-3.

SAMPLE LOCATION	SAMPLE DEPTH (ft)	DESCRIPTION	PERCENT PASSING NO. 4	PERCENT PASSING NO. 200	USCS (TOTAL SAMPLE)
B-1	1--5		100	81	

PERFORMED IN ACCORDANCE WITH ASTM D 1140

FIGURE B-1

Ninyo & Moore

Geotechnical & Environmental Sciences Consultants

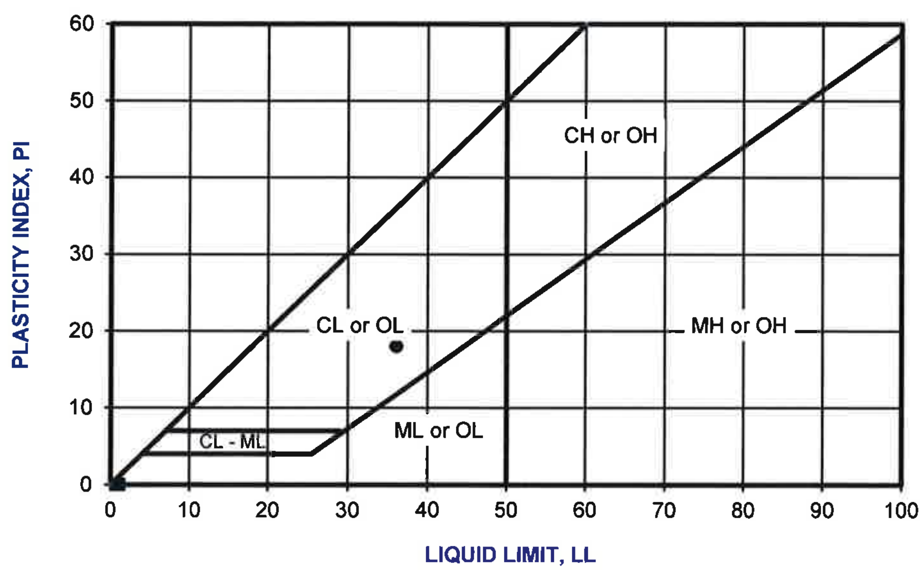
NO. 200 SIEVE ANALYSIS TEST RESULTS

1605 INDUSTRIAL AVENUE SAN JOSE CA

403246001 | APRIL 2018

SYMBOL	LOCATION	DEPTH (ft)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS
●	B-1	1--5	36	18	18	CL	
■							
◆							
○							
□							
Δ							
X							
+							

NP - INDICATES NON-PLASTIC



PERFORMED IN ACCORDANCE WITH ASTM D 4318

FIGURE B-2

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

ATTERBERG LIMITS TEST RESULTS

1605 INDUSTRIAL AVENUE SAN JOSE CA

403246001 | APRIL 2018

SAMPLE LOCATION	SAMPLE DEPTH (ft)	INITIAL MOISTURE (percent)	COMPACTED DRY DENSITY (pcf)	FINAL MOISTURE (percent)	VOLUMETRIC SWELL (in)	EXPANSION INDEX	POTENTIAL EXPANSION
B-2	1--5	11.8	104.4	25.7	0.069	69	Medium
B-4	1--5	11.0	104.9	25.7	0.077	77	Medium

PERFORMED IN ACCORDANCE WITH ☐ UBC STANDARD 18-2 ☑ ASTM D 4829

FIGURE B-3

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

EXPANSION INDEX TEST RESULTS

1605 INDUSTRIAL AVENUE SAN JOSE CA
403246001 | APRIL 2018

APPENDIX C

Calculations

LIQUEFACTION AND DYNAMIC SETTLEMENT EVALUATION BY CPT

Project Name:	LBA Realty		
Project Number:	403246001		
Calculation By:	KCC	Date:	4/6/2018
Checked By:	SPS	Date:	4/11/2018

CPT Sounding: CPT-1
Location: CPT-1

Depth to GWT during CPT evaluation (ft)	6
Design Depth to GWT (ft)	5
Atmospheric Pressure (tsf)	1.0581
Design EQ Peak Ground Acceleration, amax (g)	0.506
Design Earthquake Moment Magnitude, Mw	7.3
Magnitude Scaling Factor, MSF	1.07
At-Rest Coefficient Lateral EP, Ko	0.5
Number of Strain Cycles, Nc	13.34

Estimated dry soil dynamic settlement (in)	0.00
Estimated saturated soil dynamic settlement (in)	0.58
Total estimated dynamic settlement (in)	0.58

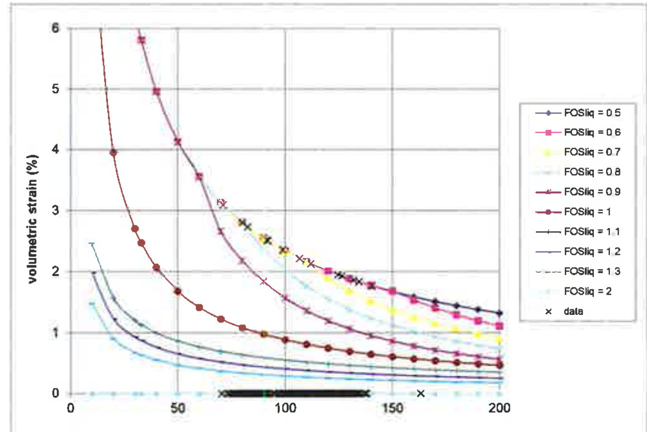
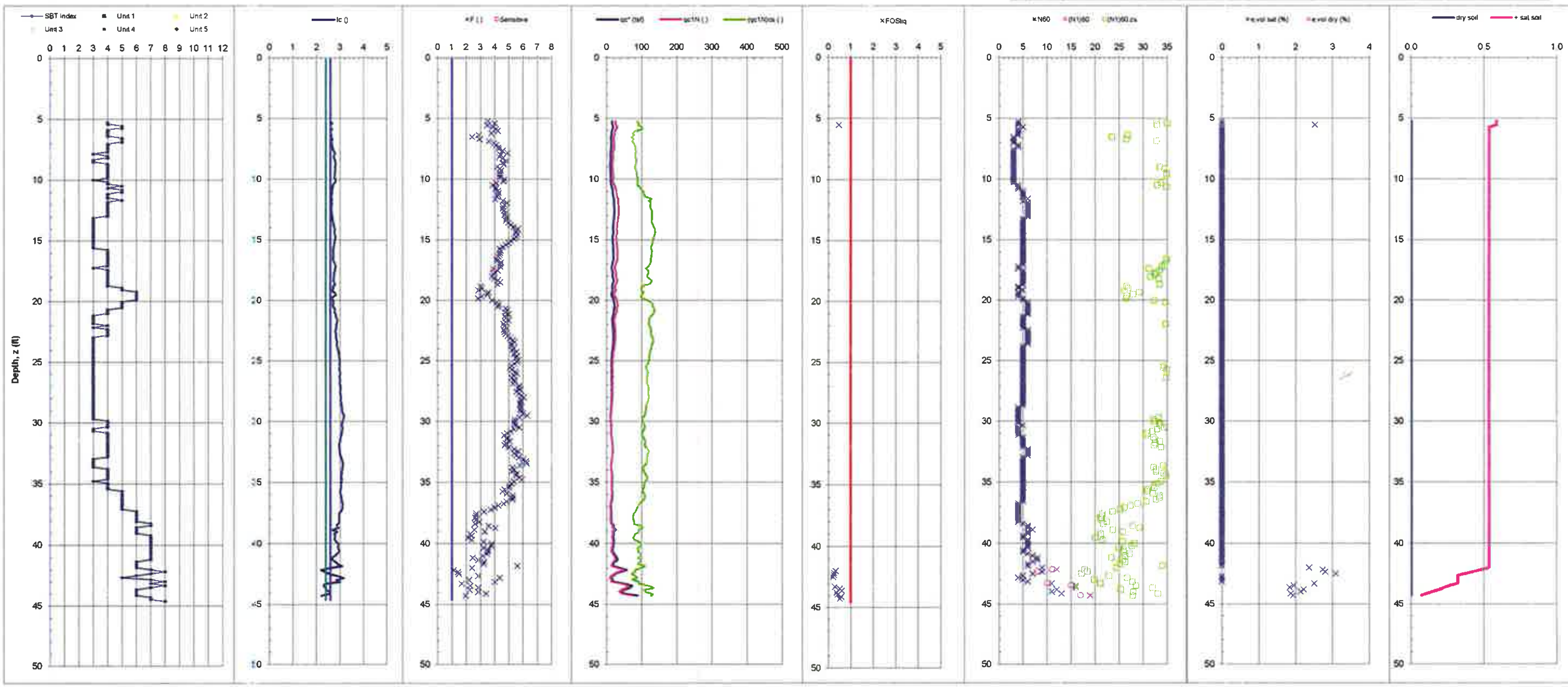


Fig 3: Zhang et al, 2002



File: cpt liqset vers10
Sheet: CPT1

4/16/2018
Page 1 of 4

LIQUEFACTION AND DYNAMIC SETTLEMENT EVALUATION BY CPT

Project Name:	LBA Realty		
Project Number:	403246001		
Calculation By:	KCC	Date:	4/6/2018
Checked By:	SPS	Date:	4/11/2018

CPT Sounding: CPT-2
Location: CPT-2

Depth to GWT during CPT evaluation (ft)	7
Design Depth to GWT (ft)	5
Atmospheric Pressure (tsf)	1.0581
Design EQ Peak Ground Acceleration, amax (g)	0.506
Design Earthquake Moment Magnitude, Mw	7.3
Magnitude Scaling Factor, MSF	1.07
At-Rest Coefficient Lateral EP, Ko	0.5
Number of Strain Cycles, Nc	13.34

Estimated dry soil dynamic settlement (in)	0.00
Estimated saturated soil dynamic settlement (in)	0.83
Total estimated dynamic settlement (in)	0.83

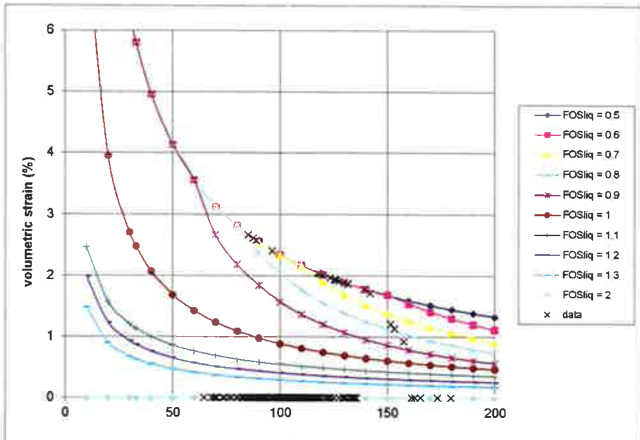
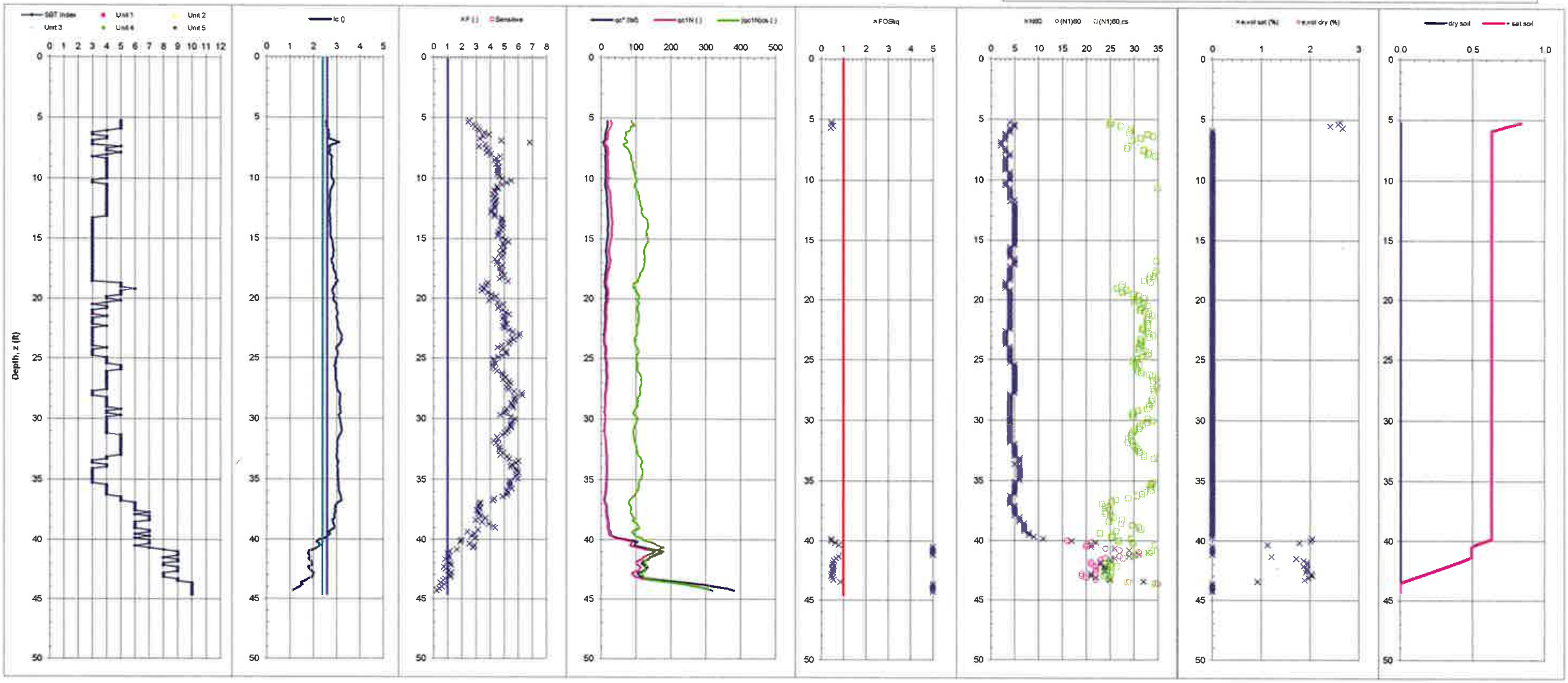


Fig 3: Zhang et al, 2002



File: cpt liqset vers10
Sheet: CPT2

4/16/2018
Page 2 of 4

LIQUEFACTION AND DYNAMIC SETTLEMENT EVALUATION BY CPT

Project Name:	LBA Realty		
Project Number:	403246001		
Calculation By:	KCC	Date:	4/6/2018
Checked By:	SPS	Date:	4/11/2018

CPT Sounding: CPT-3
Location: CPT-3

Depth to GWT during CPT evaluation (ft)	7
Design Depth to GWT (ft)	5
Atmospheric Pressure (tsf)	1.0581
Design EQ Peak Ground Acceleration, amax (g)	0.506
Design Earthquake Moment Magnitude, Mw	7.3
Magnitude Scaling Factor, MSF	1.07
At-Rest Coefficient Lateral EP, Ko	0.5
Number of Strain Cycles, Nc	13.34

Estimated dry soil dynamic settlement (in)	0.00
Estimated saturated soil dynamic settlement (in)	0.60
Total estimated dynamic settlement (in)	0.60

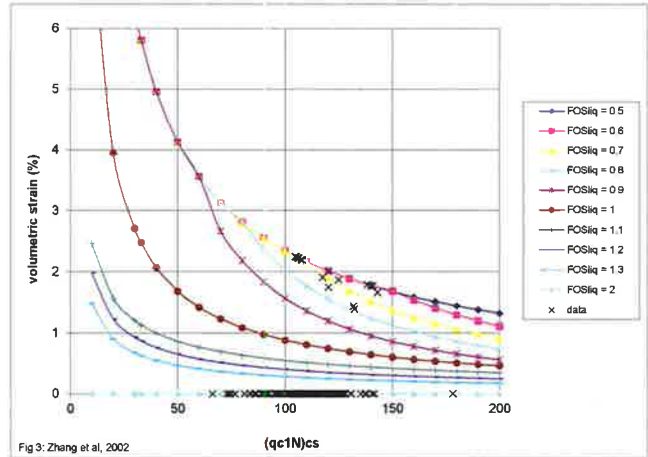
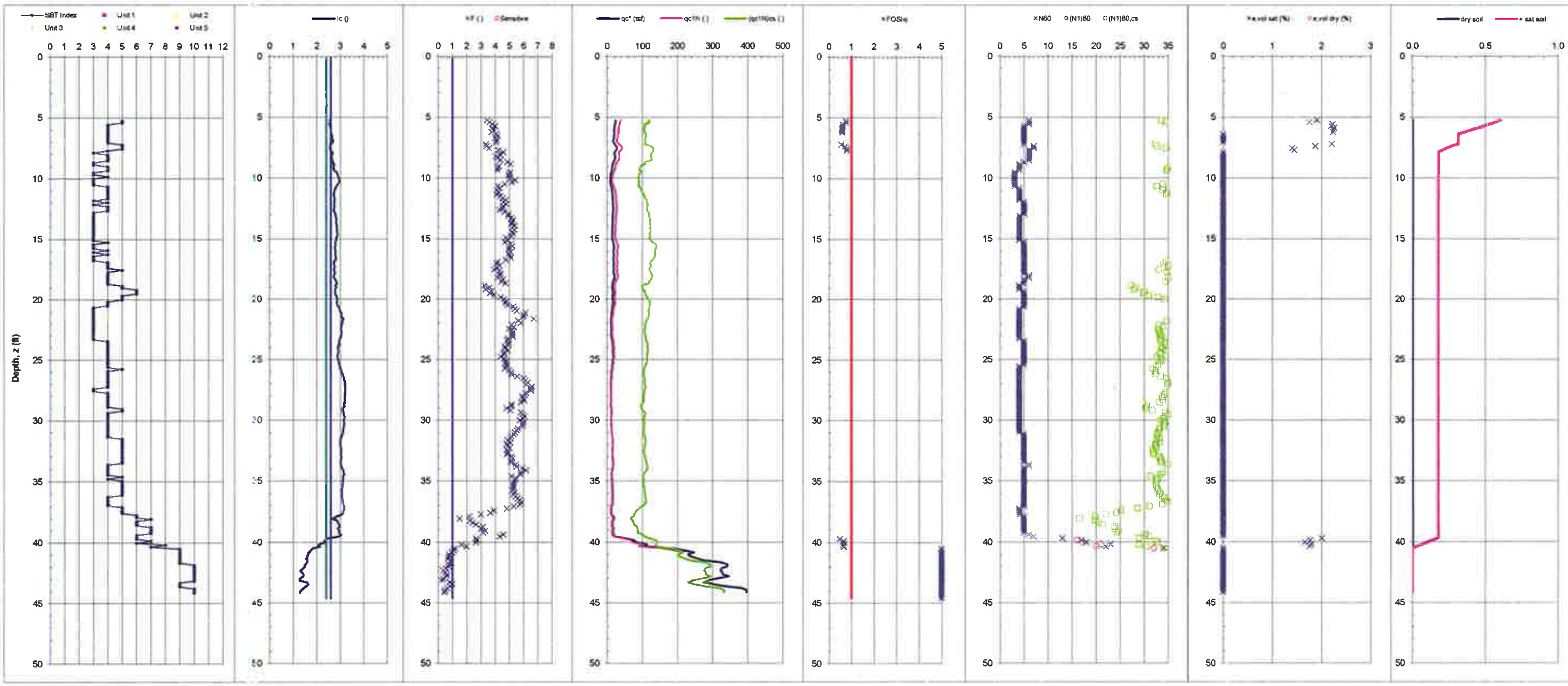


Fig 3: Zhang et al, 2002



File: cpt liqset vers10
Sheet: CPT3

4/16/2018
Page 3 of 4

LIQUEFACTION AND DYNAMIC SETTLEMENT EVALUATION BY CPT

Project Name:	LBA Realty		
Project Number:	403246001		
Calculation By:	KCC	Date:	4/6/2018
Checked By:	SPS	Date:	4/11/2018

CPT Sounding: CPT-4
Location: CPT-4

Depth to GWT during CPT evaluation (ft)	10
Design Depth to GWT (ft)	5
Atmospheric Pressure (tsf)	1.0581
Design EQ Peak Ground Acceleration, amax (g)	0.506
Design Earthquake Moment Magnitude, Mw	7.3
Magnitude Scaling Factor, MSF	1.07
At-Rest Coefficient Lateral EP, Ko	0.5
Number of Strain Cycles, Nc	13.34

Estimated dry soil dynamic settlement (in)	0.00
Estimated saturated soil dynamic settlement (in)	0.98
Total estimated dynamic settlement (in)	0.98

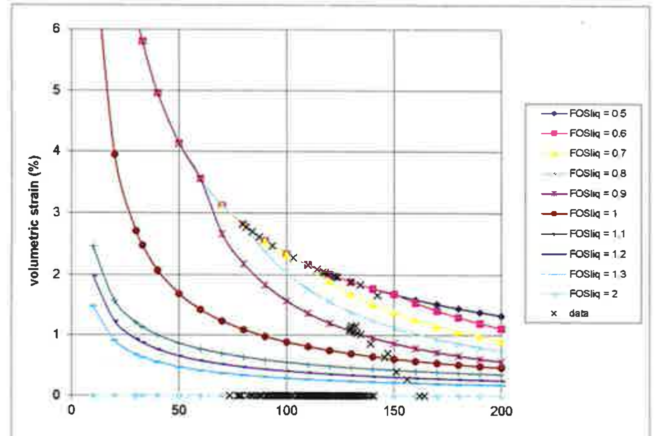
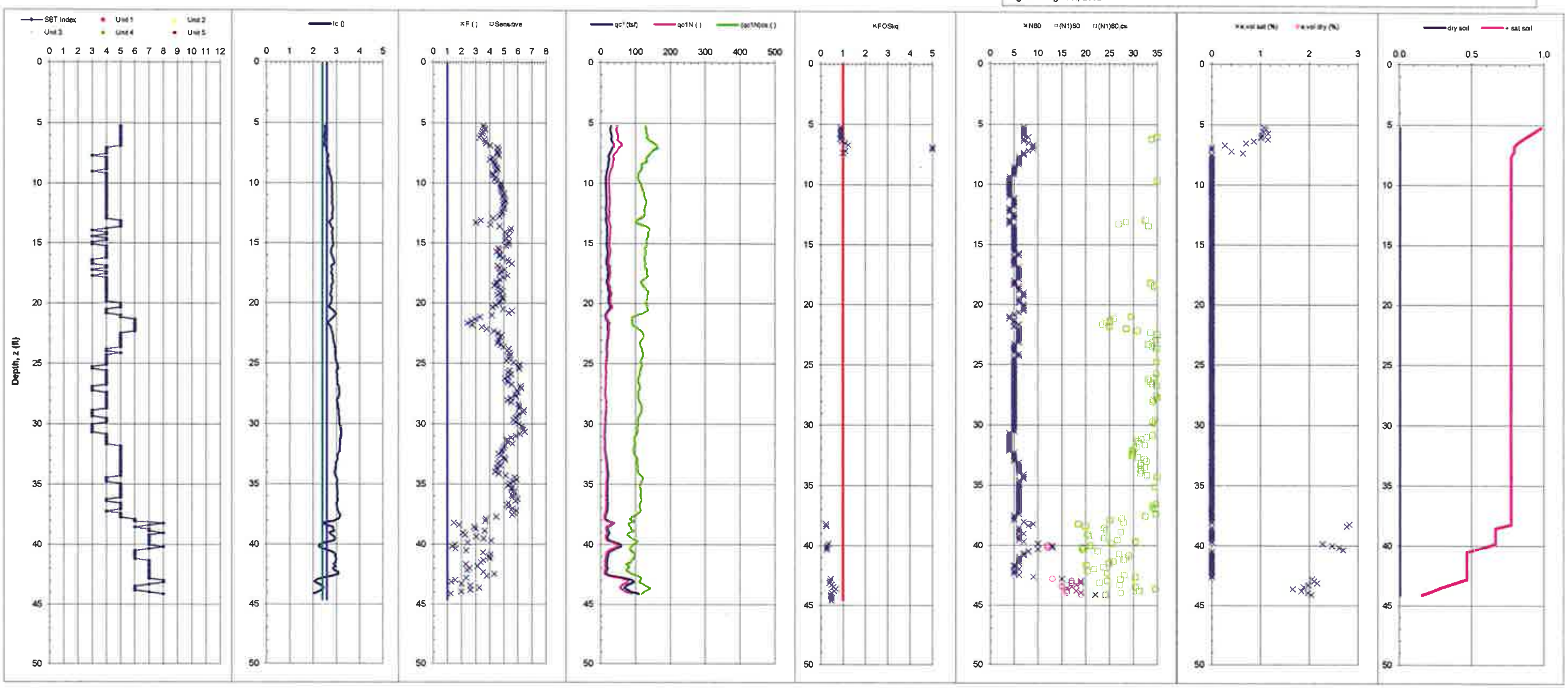


Fig 3: Zhang et al, 2002



File: cpt liqset vers10
Sheet: CPT4

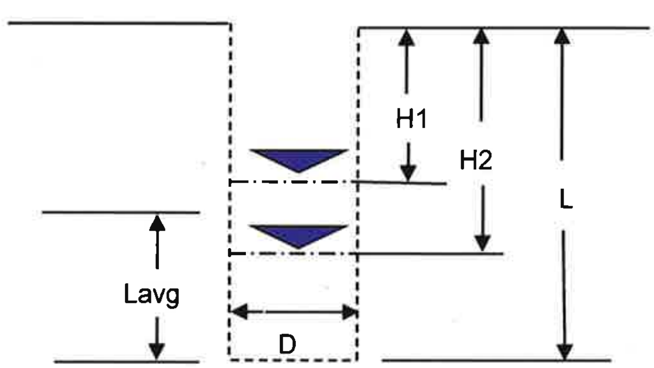
APPENDIX D

Percolation Testing

Percolation Test Data Sheet (in Borings)

Project =	LBA-1605 INDUSTRIAL AVENUE
Project No. =	403246001
Depth of Test Hole, L (ft) =	2.0
Diameter, D (in) =	8.0
Initial Water Depth, d1 (in) =	18.0
Average/Final Water Level Drop, Δd (in) =	0.3
Reduction factor, Rf =	5.5

Test Hole No.	Time (hr:min)	Elapsed Time (min)	Water Level (in)	Change in Water Level (in)	Time Interval (hour)	Pre-Adjusted Percolation Rate (inch/hour)	Adjusted Percolation Rate (inch/hour)
P-1	11:00		4.50				
	11:20	20	5.00	0.50	0.33	1.5	2.74E-01
	11:40		4.50				
	12:00	20	4.75	0.25	0.33	0.8	1.37E-01
	12:00		4.50				
	12:20	20	4.75	0.25	0.33	0.8	1.37E-01
	12:20		4.50				
	12:40	20	4.75	0.25	0.33	0.8	1.37E-01
	12:40		4.50				
	1:00	20	4.75	0.25	0.33	0.8	1.37E-01
	1:00		4.50				
	1:20	20	4.75	0.25	0.33	0.8	1.37E-01
	1:20		4.50				
	1:40	20	4.75	0.25	0.33	0.8	1.37E-01
	1:40		4.50				
	2:00	20	4.75	0.25	0.33	0.8	1.37E-01



d1 = L - H1 (in inches)