June 22, 2017
BAGG Job No: COREH-16-00

The CORE Companies
470 South Market Street
San Jose, California
Attn: Mr. Vince Cantore

Preliminary
Geotechnical Engineering Investigation
Preliminary Geotechnical Engineering Investigation
253 Race Street
San Jose, California

Dear Mr. Cantore:

As requested, BAGG Engineers performed a geotechnical engineering investigation to develop preliminary recommendations regarding construction of new senior apartments on the northern portion of the property and family apartments on the southern portion of subject 2.3 acre property in San Jose, California. This report was prepared in accordance with the scope of services outlines in our proposal Number 16-494, dated December 12, 2016.

SITE DESCRIPTION

Plate 1, Site Vicinity Map shows the general location of the captioned site and Plate 2a, Site Plan, shows the current site conditions and the locations of the subsurface exploratory points. The subject site is located at 253 Race Street in San Jose, California. The site area is bordered by Race Street to the east, Grand Avenue to the west, and various residential developments to the north and south. The 2.3 acre site is rectangular in shape, flat, and currently occupied by eight residences on the western portion of the property and several former businesses including the Race Street Fish and Poultry Market, other retail business and their associated parking spaces on the southern portion of the property.

PROJECT DESCRIPTION

We understand that the proposed project will consist of demolishing the existing buildings to accommodate new Senior Apartments on the northern portion of the property and Family Apartments on the southern portion of the property. It is likely that both the Senior Apartments and Family Apartments will consist of a six-story Type III-A wood over concrete deck slab(s) with the Family Apartments containing a partial basement. The Senior Apartments will consist of 103 units while the Family Apartments will consist of 110 units. No underground levels are planned for the project. The ground floor of both buildings will likely consist of parking garages, while the upper stories will contain the apartment units.

PURPOSE

The purpose of our investigation was to obtain limited geotechnical information regarding soil and groundwater conditions at the site in order to prepare preliminary geotechnical recommendations for design and construction of the proposed multi-family residential structures. Based on our understanding of the project, this report presents conclusions, opinions, and recommendations regarding:

- Geologic site conditions and seismicity of the project site, including distance to the active faults in the region, probability of a major earthquake on each fault,
- Potential for liquefaction beneath the site, including potential settlements and/or lateral spreading as a result of the liquefaction,
- Seismic design parameters for the site per the 2016 edition of the California Building Code,
- Specific soil conditions discovered by our CPTs, such as loose, saturated, collapsible, or soft and compressible surface and subsurface soils that may require special mitigation measures or impose restrictions on the project, including the thickness and consistency of any saturated, granular soils, and depth to groundwater as encountered,

- Preliminary criteria for site grading, earthwork, preparation of the building pads and pavement subgrade, placement of fills and backfills, and trench backfill requirements, including the suitability of the excavated soils from the site for use as fill and backfill material,

- Preliminary criteria for the support of the proposed buildings, including allowable bearing pressures and lateral resistance (passive resistance and coefficient of friction) for deep foundations, spread footings and/or mat foundations with subgrade modulus, as appropriate,

- Support requirement for concrete slabs-on-grade floors and exterior concrete patios/walkways/flatwork,

- Estimate of liquefaction-related settlement resulting from a design-level seismic event on a nearby active fault,

- General provisions for the control of surface drainage in areas surrounding the proposed structures,

SCOPE OF SERVICES

The scope of our services consisted of the following:

1. Research and review pertinent geotechnical and geological maps and reports relevant to the site and vicinity.

2. Visit the site, mark the CPT locations at least 72 hours in advance of the planned explorations, and notify Underground Service Alert to mark the known utilities entering to and/or within the site. Retain the services of an independent utility locating firm to clear each CPT and boring location with respect to underground utilities.

3. Obtain a permit from the Santa Clara Valley Water District for the subsurface exploration proposed for this investigation.

4. Perform four (4) Cone Penetration Tests (CPT) to a depth of 50 feet with a 25-ton CPT rig using piezo cones. Advance the subsurface exploration under the direction of one of our engineers/geologists. Backfill the CPT holes with cement grout per standard protocol.

5. Using the information from the CPTs, perform a preliminary engineering analyses to develop conclusions, opinions, and recommendations oriented towards the above-noted purpose of the investigation.

6. Prepare one electronic (pdf) of a preliminary report summarizing our findings and recommendations, and including a vicinity map, a site plan, a regional geologic map, a regional fault map, and CPT logs.

SUBSURFACE EXPLORATION

Subsurface conditions at the site were explored on June 05, 2017 by advancing CPTs (designated as CPT-1 through CPT-4) at four locations to approximately 50 feet below the existing ground surface at the approximate locations shown on the attached Plate 2, Site Plan. Portions of the site contained existing structures or were not accessible for multiple reasons; therefore, CPTs were located only in the areas which were readily accessible. Additional subsurface exploration will be necessary when the entire site becomes readily accessible.

All tests were performed using a truck-mounted, 25-ton, CPT rig equipped with hydraulic jacks required to push a steel cone connected on the end of a series of metal rods. The steel cone has a tip with a surface area of 10 cm^2 and friction sleeve with a surface area of 150 cm^2. The steel cone was fitted with saturated porous element behind the cone and a hydrostatic pressure element used to measure in-place pore water pressures. The steel cone is fitted with load cells and as it is pushed into the ground, the load cell measurements are recorded at 0.05 meter intervals. The load cell data is processed and correlated to various soil properties using the relationships suggested by researchers who have been working on improving the soil behavior type with load cell data. This method also allows the collects of disturbed samples from selected depths, if needed for laboratory testing; however, no samples were collected for our investigation. At the completion of the test, the CPT holes were backfilled with cement grout as per the condition of the drilling permit obtained from the Santa Clara Valley Water District.

GEOLOGY AND SEISMICITY

Regional Geology

The site is located in the central portion of the Santa Clara Valley, which is a relatively broad and level alluvial basin that is filled with Quaternary-age (1.6 million years old or younger) unconsolidated sediments derived from the nearby mountain ranges. The Santa Clara Valley is bounded by the San Francisco Bay to the north, by the Santa Cruz Mountains to the west and southwest, and by the Diablo Mountain range to the east and northeast.

The site and the San Francisco Bay Area lie within the Coast Range geomorphic province, a series of discontinuous northwest trending mountain ranges, ridges, and intervening valleys characterized by complex folding and faulting. The broad Santa Clara Valley contains alluvial fan deposits that slope down gently to the north toward San Francisco Bay. Five creeks (Calabazas, Saratoga, Los Gatos, San Tomas Aquinas, Coyote) and the Guadalupe River along with numerous intermittent streams cross the Santa Clara Valley in this area.

The San Jose West Quadrangle is within the active San Andreas Fault system, which distributes shearing across a cross system of primarily northwest-trending, right-lateral, strike-slip faults that include the San Andreas, Hayward, and Calaveras faults. Several oblique-slip and reverse-slip faults, including the Berrocal, Shannon, Monte-Vista, and Santa Clara faults are within or slightly west of the quadrangle along the base of the foothills.

Site Geology

A published geology map pertinent to the site area titled "Quaternary Geology of Santa Clara Valley, Santa Clara, Alameda, and San Mateo Counties, California," (Digital Database Open-File Report, Helley, Graymer, Phleps, Showalter, and Wentworth, 1994) indicates the site is underlain by Alluvial Fan Deposits (Qhaf, Holocene) described as "Brown or tan, medium dense to dense gravelly sand or sandy gravel that grades upward to sandy or silty clay."

Quaternary geologic map of the San Jose West Quadrangle, California (modified from Knudsen and Others, 2000) and included as Plate 1.1 in Seismic Hazards Zone Report 058 for San Jose West Quadrangle indicates that the site area is underlain by Holocene alluvium deposits which consist of 44 percent lean clay (CL), 14 percent clayey silt (ML), 13 percent sandy silt (SM) and 20 percent others.

A review of the State seismic hazard maps relevant to the site (State of California Seismic Hazard Zones, San Jose West Quadrangle, California Geological Survey, February 7, 2002) indicates the subject site is situated in an area designated as an earthquake-induced liquefaction hazard zone. Liquefaction Hazard Zones are defined as the areas where historic occurrence of liquefaction or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements. Therefore, our report addresses liquefaction related site hazards.

The State Seismic Hazard Zone Report depicts the depth to groundwater in the site area to be approximately 30 feet. SZHR 058 indicates liquefaction susceptibility to be moderate for the sites underlain by Holocene alluvium deposits with depth to groundwater between 10 to 30 feet. A portion of that map which includes the site area is presented herein as the Local Geologic Map, Plate 4.

Seismicity

The Santa Clara Valley, as is the entire San Francisco Bay Area, is considered to be an active seismic region due to the presence of several active earthquake faults. Three major, northwest-trending earthquake faults extending through the Bay Area are responsible for the majority of movement on the San Andreas fault system. They include the San Andreas fault, the Hayward fault and its southeast extension, and the Calaveras fault which are respectively located about 17 kilometers to the southwest, 10 kilometers northeast of the site, and 16 kilometers to the northeast. The Monte Vista – Shannon fault is located 11.9 km southwest of the site and the northern San Gregario fault is located 44 km west of the site. The site area is not located within the Alquist-Priolo Earthquake fault zone.

SITE CONDITIONS

Surface Conditions

The CPTs were advanced at the approximate locations shown on Plate 2, Site Plan. The majority of the site is covered with asphaltic pavement with the exception of some landscape area around the existing residences.

Subsurface Conditions

The CPTs advanced at the site indicate the presence of generally clayey soils to the maximum depth explored. The CPT results indicate the presence of sandy soils at the depth intervals listed below:

CPT ID	Depth Intervals of Sandy Soils (feet below Ground Surface)
CPT-1	8.2 -12.6 feet 17.4 to 18.5 feet 40.8 to 41.5 feet 46.6 to 50 feet
CPT-2	4.3 to 6.2 feet 7.7 to 10.2 13.6 to 14.1 feet 15.6 to 15.9 16.7 to 17.6
CPT-3	0.3 to 19.2 feet 29.9 to 30.2 feet 39.5 to 41.5 feet 47.9 to 49.7 feet
CPT-4	13.6 - 18 feet 41.5 to 43 feet

The subsurface soil conditions appeared a bit anomalous at CPT-3 because sandy soils were encountered to a depth of 19.2 feet. It is possible that CPT-3 was advanced in an old excavation that was backfilled with granular soil. These conditions will be confirmed by performing additional CPTs near the location of CPT-3 during the next phase of subsurface investigation. The majority of the cohesive soils within the load bearing zone were medium stiff to very stiff and the granular soils were loose to medium dense. For more information on the subsurface soil and groundwater conditions, we refer you to Appendix A, which includes CPT Test Results.

Groundwater

Groundwater was encountered at 26 feet bgs in CPT-01 and CPT-02, 29 feet bgs in CPT-03, and 28 feet bgs in CPT-04. It should be noted that 2016-2017 was an extremely wet year and groundwater levels typically fluctuate due to variations in rainfall, and temperature. Due to the interbedded and interfingering nature of alluvial sediments, it is also likely that fluctuations in the groundwater level and/or perched water conditions may occur across the site.

Liquefaction Potential

Soil liquefaction is a condition where saturated granular soils near the ground surface undergo a substantial loss of strength due to increased pore water pressure resulting from cyclic stress applications induced by earthquakes or other vibrations. In the process, the soil acquires mobility sufficient to permit both vertical and horizontal movements, if not confined. Soils most susceptible to liquefaction are loose, uniformly graded, fine-grained, sands, and loose silts with very low cohesion. In general, liquefaction hazards are most severe in the upper 50 feet of the soil profile. In the deeper deposits the overburden soils tend to isolate the ground surface from any liquefaction and the overburden pressures tends to limit shear strains that occur during liquefaction.

According to the State of California Seismic Hazard Zones Map for the San Jose West Quadrangle, the site is located within an area delineated as a Liquefaction Hazard Zone; therefore engineering analysis was performed to evaluate liquefaction potential at the site and to estimate seismic settlements during a design-level earthquake on one of the nearby faults. There is no known history of liquefaction induced damage at the site.

The liquefaction potential evaluation was performed using the CPT data collected from the site because a CPT provides more continuous information about the subsurface soil conditions. The liquefaction analysis was performed with the aid of an in-house spreadsheet and was confirmed with the help of the computer program CLIQ, Version 2.

Assuming the depth to groundwater to be 26 feet bgs (the shallowest water level recorded in the CPTs), the peak ground acceleration at the site to be 0.79 (2,475 return period) generated by an earthquake of moment magnitude 6.7 on the nearby Hayward fault, the average liquefaction related settlement was calculated to be slightly less than 1-inch. The liquefaction potential analysis was performed using the methodology suggested by Boulanger and Idriss, 2014. The differential settlement should be anticipated to be approximately ⅔ the total anticipated settlement.

CBC 2016 Seismic Design Parameters

Based on the soil information obtained from the exploratory borings and CPTs advanced on the site, the soil profile at the site is classified as a Class "D", defined as a "stiff soil" profile with an average shear wave velocity in the range of 600 to 1200 feet per second (180 to 360 m/s), average Standard Penetration Test (N) values in the range of 15 to 50, and/or average undrained shear strength in the range of 1,000 to 2,000 psf in the top 100 feet of the soil profile.

Using the site coordinates of 37.3252 degrees North Latitude and 121.9121 degrees West Longitude, and the USGS Java-based program, earthquake ground motion parameters were computed in accordance with 2016 California Building Code are as listed in the following table.

Table 1

Parameters for Seismic Design

2016 CBC Site Parameter	Value
Site Latitude	37.3252° N
Site Longitude	121.9121 ° W
Site Class,	Stiff Soil, Class D
Mapped Spectral Acceleration for Short Periods S_s	1.50 g
Mapped Spectral Acceleration for a 1-second Period S_1	0.60 g
Site Coefficient F_a	1.00
Site Coefficient F_v	1.50
Site-Modified Spectral Acceleration for short Periods S_{Ms}	1.50 g
Site-Modified Spectral Acceleration for a 1-second Period S_{M1}	0.90 g
Design Spectral Acceleration for short Periods S_{Ds}	1.00 g
Design Spectral Acceleration for short Periods S_{D1}	0.60 g

DISCUSSION AND RECOMMENDATIONS

General

Based on the subsurface data collected from the site, and the laboratory test results, it is our opinion that the proposed project is feasible from a geotechnical engineering standpoint, provided the recommendations presented in this report are incorporated into the project design and implemented during site grading and foundation construction. Please note that these recommendations are preliminary in nature and are subject to revision after a more thorough subsurface exploration is completed.

The main challenges from a geotechnical engineering standpoint are:

- The presence of compressible soils in the load bearing zone of the subsurface soil environment.
- The presence of liquefiable soils that would undergo settlement during a design level seismic event.

If the proposed building is supported on conventional spread footings, the static settlement under the anticipated column loads is estimate to be excessive. Furthermore, the liquefaction-related settlement could be on the order of 1-inch. These settlement estimates exceed the typical tolerance of a conventional spread footing foundation system; therefore, we recommend that the building be supported on a concrete mat slab. Alternatively, the building could be supported using conventional spread/strip footings if the load bearing capacity of the soils supporting the buildings is enhanced using one of the commonly used ground improvement system such as soil mixed columns or drilled displacement piles.

The site could experience very strong ground shaking from future earthquakes during the anticipated lifetime of the project. The intensity of the ground shaking will depend on the magnitude of the earthquake, distance to the epicenter, and the response characteristics of the on-site soils. While it is not possible to totally preclude damage to structures during major earthquakes, strict adherence to good engineering design and construction practices will help reduce the risk of damage.

Site Grading

Detailed site grading plans were not available when this report was prepared but it is our understanding that site grading will involve cuts and fills on the order of three feet or less. The demolition of existing structure may involve removal of the existing structures and associated footings. The depressions created by removal of footings should be backfilled with engineered fill. Removed asphaltic concrete may be recycled at an asphalt recycling facility. The aggregate base below the existing asphalt pavement may be re-used as select fill if carefully segregated.

Prior to placing fill, the top 6 to 8 inches of subgrade should be scarified, moisture conditioned to slightly above optimum moisture content and compacted to a minimum of 90 percent relative compaction. BAGG recommends that the fill be placed in thin lifts and compacted to a minimum of 90 percent relative compaction, as determined by ASTM Test Method D1557. Fill material placed in the top 12 inches below the concrete slab and pavement areas should be compacted to a minimum of 95 percent of the maximum dry density, as determined by ASTM Test Method D1557.

All aspects of site grading including the placement of fill and backfilling of excavations should be performed under the observation of the Geotechnical Engineer's field representatives. It must be the Contractor's responsibility to select equipment and procedures that will accomplish the grading as described above. The Contractor must also organize his work in such a manner that one of our field representatives can observe and test the grading operations, including compaction of fill and backfill, and compaction of subgrades.

Subgrade Stabilization

If the site grading work is scheduled to begin in the wintertime, the near surface soils, after the removal of asphaltic pavement may become unstable under the heavy traffic loads of construction equipment. Typical ways of stabilizing the subgrade involve: 1) removal of the wet soil and replacement with imported dry soil or aggregate baserock; 2) addition of geofabrics or geogrids to bridge minor unstable areas; 3) reduction of moisture content through aeration; and 4) addition of quick lime which reacts with the soil to change the chemical composition of the soil thus resulting in soil with lower shrinkage swelling potential and the chemical reaction between soil and lime produces heat which in turn results in the loss of moisture. In our experience, addition of 4 to 5 percent quick lime (by weight of soil) in the top 18 inches of soil subgrade generally results in a stable subgrade in areas likely to experience heavy construction traffic. The depth of chemical treatment may be reduced to 12 inches in areas which are less likely to experience heavy traffic. Treated soil should be compacted to a minimum of 90 percent relative compaction and allowed to cure for a minimum of 48 hours prior to subjecting it to heavy traffic loads.

Foundations

As previously mentioned, if the proposed building is supported on conventional spread footings, the static settlement under the column loads will be excessive. Furthermore, the liquefaction-related settlement could be on the order of 1-inch. These settlement estimates exceed the typical tolerance of a conventional spread footing foundation system. Alternatively, the building could be supported using conventional spread/strip footings if the load bearing capacity of the soils supporting the buildings is enhanced using one of the commonly used ground improvement system such as soil mixed columns or drilled displacement piles.

Concrete Mat Foundation

The proposed building loads may be supported on a concrete mat foundation system capable of spreading the loads uniformly over large areas. The concrete mat slab can be designed for a maximum localized allowable bearing pressure of 1,500 psf for dead plus live loads. This allowable bearing pressure may be increased by one-third for all loads including wind or seismic. The average areal loads spread over the mat should not exceed 1,000 psf.

The mat slab should be reinforced in both directions with top and bottom steel reinforcement, as specified by the project Structural Engineer, to help span local irregularities. A modulus of subgrade reaction of 15 kips per cubic foot (kcf) can be used for the design of the mat. This value has been reduced to account for the size of the mat; therefore, it is not K_{v1} for 1-foot square plate. We estimate that total settlement due to static loads would range from 1 to 1.5 inches across the mat and the total post construction differential settlement would not exceed ¾-inch between adjacent columns due to static loading. Accounting for both liquefaction-induced and static differential settlement, we recommend the mat should be designed to tolerate a total differential movement of approximately 1½ inches between the center and corner of the mat.

The concrete mat slab should be placed over a minimum 15-mil thick vapor retarder conforming with ASTM Standard Specification E 1745 and the latest recommendations of ACI committee 302. The vapor retarder should be installed in accordance wih ASTM Standard Practice E 1643 and per the manufacturer's recommendations. Care should be taken to properly lap and seal the vapor retarder, particularly around utilities, and to protect it from damage during construction

Lateral forces can be resisted by friction between the vapor retarder and the underlying soil and the passive resistance between the mat and the adjacent soil. For design purposes, a coefficient of friction of 0.15 may be used between the mat and the underlying soil. The passive resistance between the walls of the concrete mat and the adjacent soil may be calculated using an equivalent fluid weight of 300 pcf. The passive resistance and the friction coefficient may be used in combination without reduction.

The mat slab should be cast on a compacted and stable soil subgrade. Since the water table is relatively shallow, it will be important to keep the rubber tired and vibratory equipment off the pad subgrade to minimize localized instability of the pad. The subgrade should be free of standing water, debris, and disturbed materials and should be approved by the geotechnical engineer prior to placing reinforcing steel. The mat slab subgrade should be kept moist following excavation and maintained in a moist condition until concrete is placed.

Ground Improvement Systems

There are several different types of ground improvement systems that could be utilized to improve the load bearing capacity of the soils and transfer the loads to more competent soils at depth. These include compacted aggregate piers (CAP), drilled displacement columns, and soil-cement mixed columns. Due to the desire to minimize the volume of soil requiring off-haul, it is our opinion that compacted aggregate piers may not be appropriate for the site use.

Deep soil mixing is an advanced ground improvement method in which cement is mixed with in-situ soil to form in-place soil cement columns that increase the strength and reduce the compressibility of soft ground. The strength of soil-cement varies with soil type and mix design and can be tailored to the exact application and soil conditions. In the deep soil mixing admixtures/binders (typically cement) are injected and blended into the in-place soils throughout the treatment depths and mixed thoroughly using large diameter mixing tool to produce columns of improved soil. The diameter of the soil cement columns could be adjusted from 18 inches to a few feet with the larger diameter columns requiring rigs with higher torque.

Drilled displacement column (DDC) system is a deep ground improvement system used to improve any soft/loose soil for the support of heavy loads on shallow conventional spread/strip footings. The installation process involves drilling a hole into the ground at a pre-determined location using a specially designed auger that is shaped to laterally displace and compact soil at the edges into the ground. The soil displacement produces cavity expansion effets that 1) increase shear strength, 2) increase density, and 3) increase stiffness and modulus of surrounding soil. Upon reaching the final design depth, the drilling auger is withdrawn as the hole is backfilled with cement grout of specified strength. Typical diameters of drilled displacement columns range from 12 to 24 inches with depths up to 80 feet. The drilling process generates minimal cuttings, which is another reason this process is preferred in contaminated soil environments. DDC are sometimes installed with a single rebar for higher strength and to resist tension forces. Upon installation, full-scale load tests are performed to confirm DDC design bearing capacity. Cone penetration tests are performed to confirm ground improvement in soil within groups of DDC.

In order to provide relatively uniform footing support, a compacted crushed stone layer is normally placed over the tops of the drilled displacement columns. The purpose of the aggregate layer is to spread the load and provide a shear break should the footing be subjected to lateral forces.

Based on our understanding of the subsurface soil conditions, it is our opinion that the two viable ground improvement systems will include soil-cement columns or drilled displacement columns. Ground improvements are installed under design-build contracts by specialty contractors. The required size, spacing, length, and strength of the ground improvement elements should be determined by the contractor based on the proposed structural loads and the desired level of improvement. For planning purposes, we recommend the ground improvement elements extend to a depth of 30 feet below existing grade. The length and spacing of the ground improvement elements should be sufficient to limit total long-term static settlement to one-inch. In our experience, it is quite reasonable to expect allowable load bearing capacity of the properly designed conventional foundation system supported on the improved subsurface soil to be in the range of 5,000 psf.

Lateral forces can be resisted by friction between the bottom of the footing and the underlying baserock and the passive resistance between the footing and the adjacent soil. For design purposes, a coefficient of friction of 0.35 may be used between the bottom of the footing and the underlying aggregate baserock. The passive resistance between the walls of the concrete mat and the adjacent soil may be calculated using an equivalent fluid weight of 300 pcf. The passive resistance and the friction coefficient may be used in combination without reduction.

A copy of our final geotechnical engineering investigation report should be provided to potential design-build ground improvement contractors and BAGG should be retained to review the design calculations and provide technical input related to the geotechnical aspects of the design prior to construction. It is quite possible that the selected/short-listed ground improvement contractor may choose to perform additional subsurface exploration at the site to refine their design. Additional subsurface information that may be used to refine the design should be shared with BAGG Engineers. Furthermore, BAGG should be retained to provide quality assurance services during the installation of the ground improvements systems and collect independent data to confirm the load bearing capacity of the foundations supported on the selected ground improvement system.

Deep Foundation Systems

Another suitable foundation option for supporting columns loads will be drilled displacement (screw) piles. Drilled displacement (DD) piles are rotary displacement piles installed by inserting a specially designed segmented helical auger into the ground with both vertical force and torque. The soil is displaced laterally within the ground (with minimal spoils generated), and the void created is filled with grout or concrete. The radial displacement of soil during installation of DD piles contributes to the high capacity obtained for these piles. The advantages of DD piles are: the ease of construction with minimal vibration or noise, and the high load bearing capacity due to displacement of soil surrounding the pile. The drilled displacement piles are a foundation system and not a ground improvement system. These piles will be reinforced per the recommendations of the project Structural Engineer to withstand axial and shear forces. The results of our analysis indicate that 18-inch diameter drilled displacement piles extending to 60 feet (maximum depth of our exploration) may be designed for an allowable load bearing capacity of 50 tons with a factor of safety of 2 for skin friction between the pile and surrounding soil and 3 for end bearing. Full scale load bearing tests will be required to confirm the actual load bearing capacity of the piles.

Should you choose DD piles as the foundation system, please provide the axial loads, shear force and bending moment, so BAGG could perform analysis to estimate pile deflections, shear force, and bending moment corresponding to the depth of pile.

Utility Trench Backfill

The utility trenches may be backfilled with environmentally clean on-site soils, which are free of debris, roots and other organic matter, and rocks or lumps exceeding 3 inches in greatest dimension. The utility lines should be properly bedded and shaded with granular material, such as, sand or pea gravel. The bedding and shading layers should be compacted using a vibratory compactor. The contractor should use extreme caution with the vibratory compactor on the shading layer because excessive vibrations and/or imbalanced shading materials could result in loosening of the pipe joints. Alternatively, the utility trenches may be backfilled with 1 to 2-sack mix of low density fill (flowable fill). BAGG Engineers should be allowed an opportunity to observe the trench backfill operations and perform field compaction tests to evaluate the moisture content and relative compaction of the fill materials.

Vertical trenches deeper than 5 feet will require temporary shoring. Where shoring is not used, the sides should be sloped or benched, with a maximum slope of 1:1 (horizontal: vertical). The trench spoils should not be placed closer than 3 feet or one-half of the trench depth (whichever is greater) from the trench sidewalls. All work associated with trenching must conform to the State of California, Division of Industrial Safety requirements.

We trust this report provides you the required information from a planning standpoint. We thank you for the opportunity to perform these services. Please do not hesitate to contact us, should you have any questions or comments.

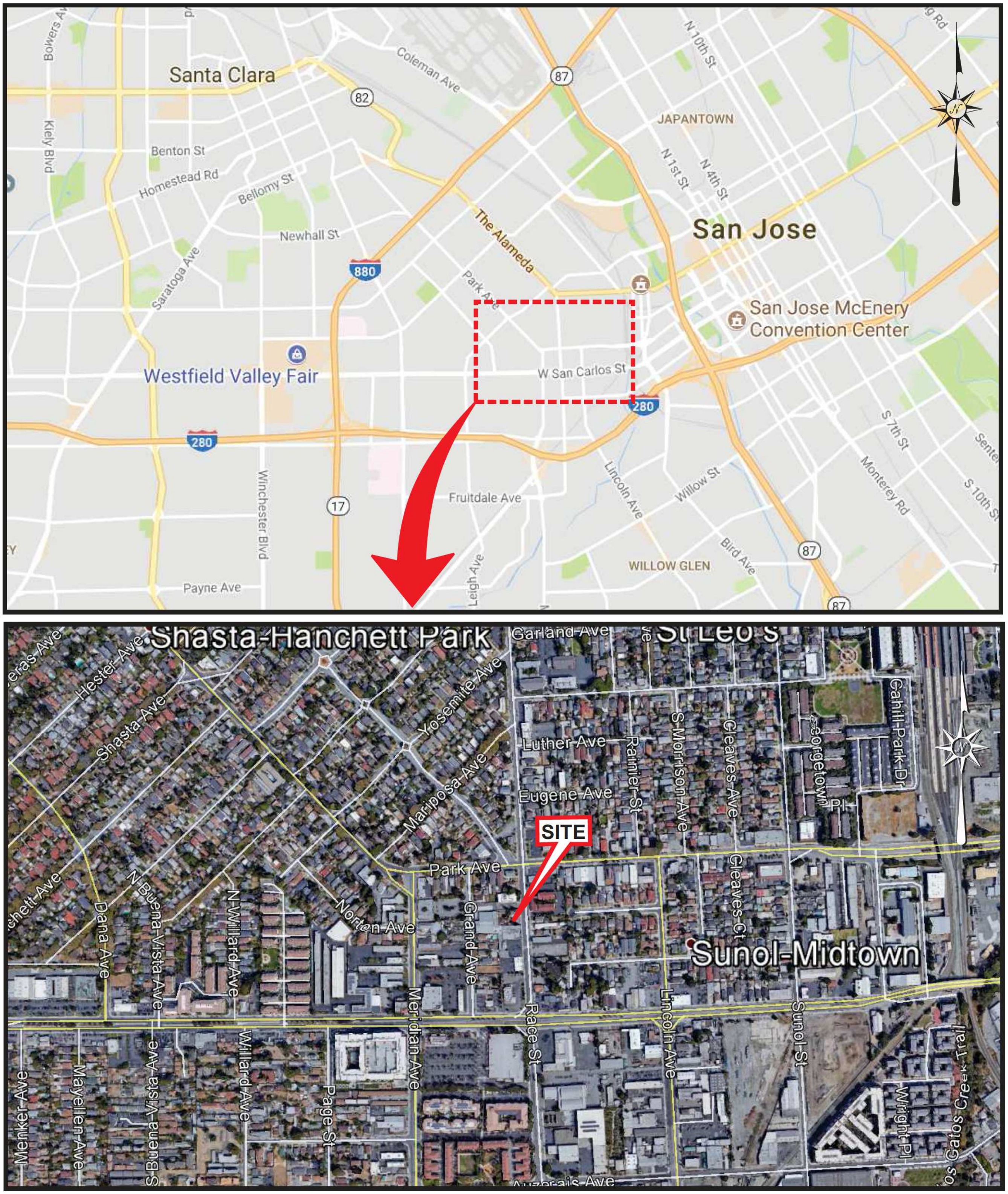
Very truly yours,
BAGG Engineers

Ajay Singh
Senior Geotechnical Engineer

Attachments:

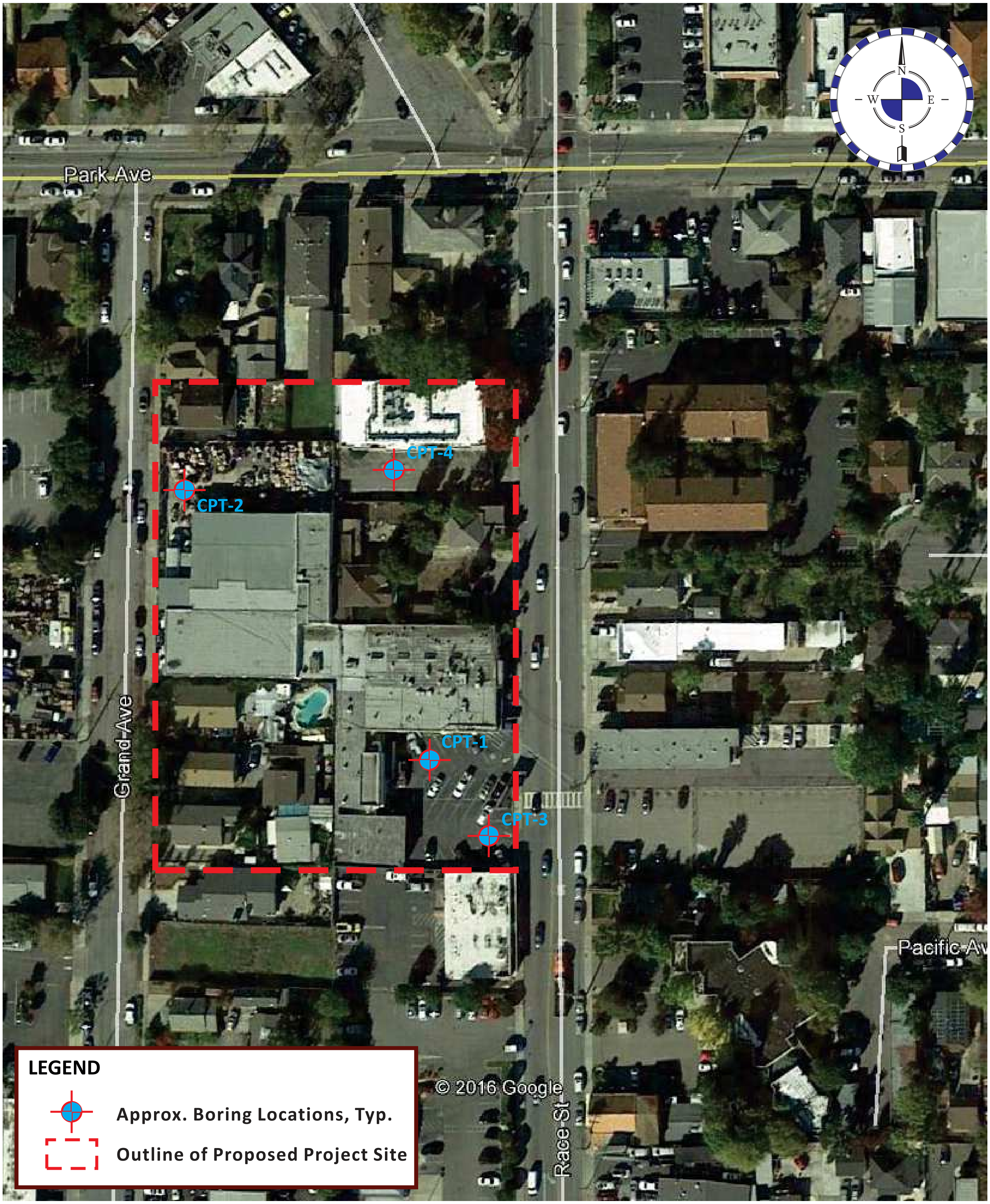
Plate 1, Site Vicinity Map
Plate 2a, Site Plan
Plate 2b, Site Development Plan
Plate 3, Regional Geology
Plate 4, Local Geology
Plate 5, Regional Fault Map

Appendix A
CPT Test Results



Source: Google Maps

GEOTECHNICAL ENGINEERING INVESTIGATION RACE STREET RESIDENTIAL SAN JOSE, CALIFORNIA	VICINITY MAP		
	DATE: JUNE 2017	JOB NO.: COREH-16-00	PLATE: 1

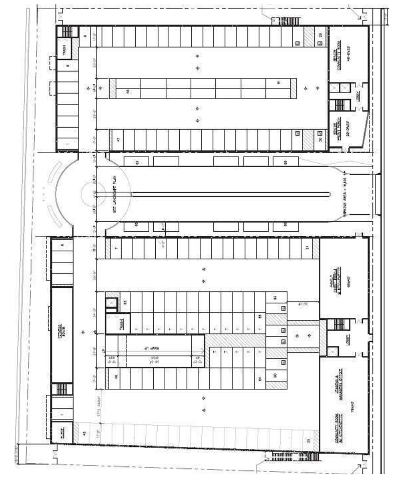


GEOTECHNICAL ENGINEERING INVESTIGATION
RACE STREET RESIDENTIAL
SAN JOSE, CALIFORNIA

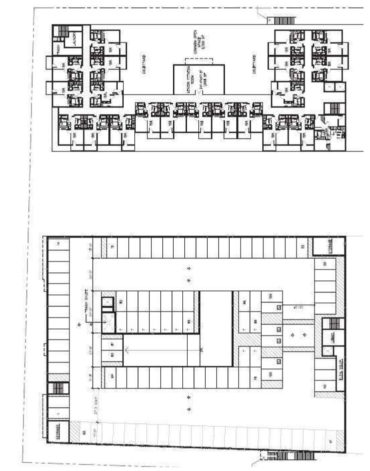
SITE PLAN

DATE:	JOB NUMBER:	PLATE:
JUNE 2017	COREH-16-00	2A

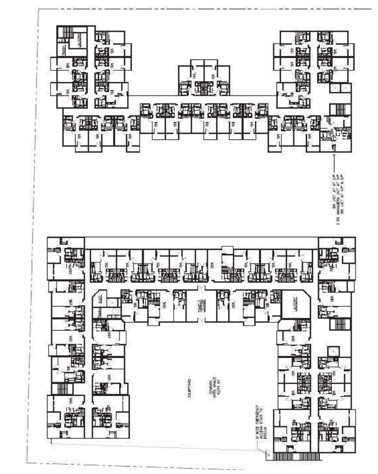
BAGG ENGINEERS



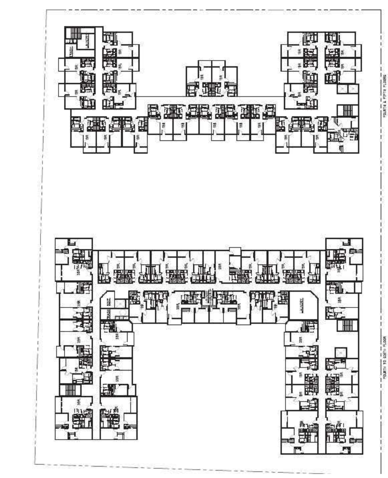
Ground/First Floor



Second Floor



Third Floor



Fourth Floor / Upper Floor

Conceptual view from Race Street

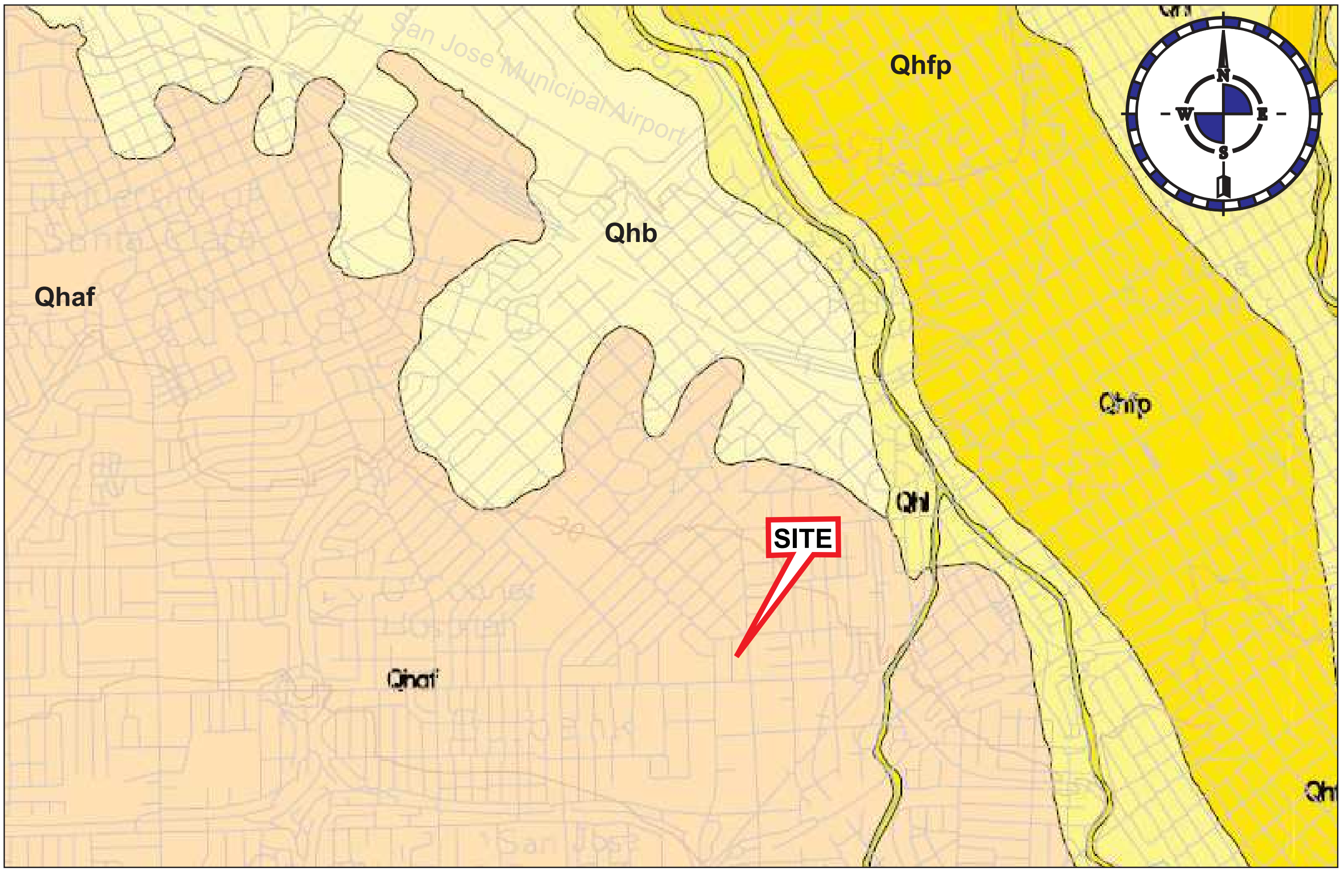
SOURCE: "Race Street Residential," Prepared by LPMD Architects, dated 2-06-2017.

GEOTECHNICAL ENGINEERING INVESTIGATION
RACE STREET RESIDENTIAL
SAN JOSE, CALIFORNIA

BAGG ENGINEERS

SITE DEVELOPMENT PLAN

JOB NO.:	SCALE:	DATE:	PLATE
COREH-16-00	NTS	JUNE 2017	2B



LEGEND

Qhl **Natural Levee Deposits (Holocene)** – Loose, moderate- to well- sorted sandy or clayey silt grading to sandy or silty clay. Levee deposits are generally well drained.

Qhfp **Floodplain Deposits (Holocene)** – Medium to dark-gray, dense, sandy to silty clay. Lenses of coarser material (silt, sand, and pebbles) may be locally present.

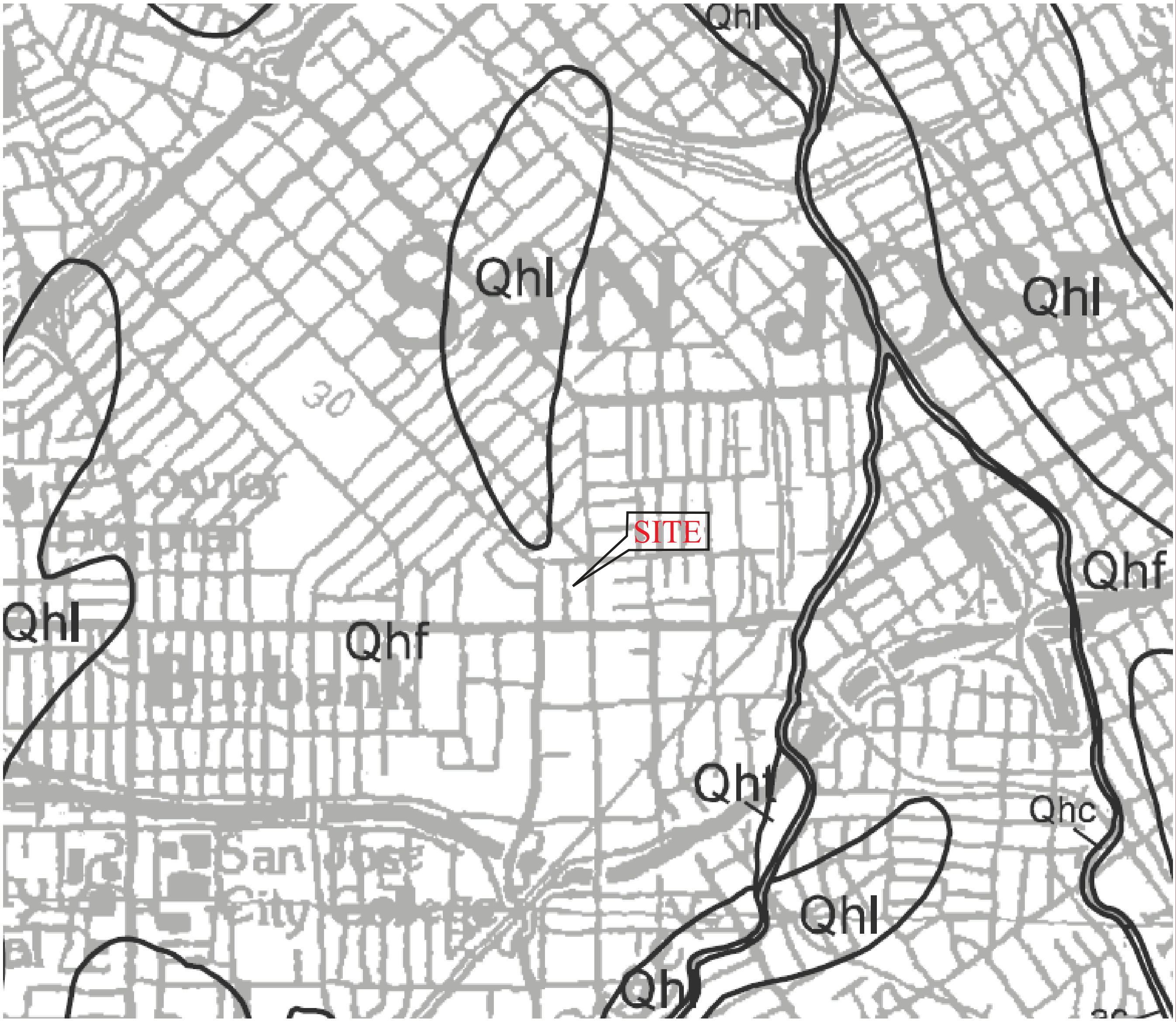
Qhb **Floodbasin Deposits (Holocene)** – Organic - rich clay to very fine silty-clay deposits occupying the lowest topographic positions between Holocene levee deposits or Holocene floodplain deposits.

Qhaf **Alluvial Fan Deposits (Holocene)** – Brown or tan, medium dense to dense gravelly sand or sandy gravel that grades upward to sandy or silty clay. Near the distal fan edges, deposits are typically brown, medium dense gravelly sand or clayey gravel that grades upward to sandy or silty clay.

Reference: *Quaternary Geology of Santa Clara Valley, Santa Clara, Alameda, and San Mateo Counties, California: A digital database*, by E.J. Helley, R.W. Graymer, G.A. Phelps, P.K. Showalter, and C.M. Wentworth, Derived from the USGS Digital Database 94-231, May 1994.

GEOTECHNICAL ENGINEERING INVESTIGATION RACE STREET RESIDENTIAL SAN JOSE, CALIFORNIA	REGIONAL GEOLOGY MAP		
	DATE: JUNE 2017	JOB NUMBER: COREH-16-00	PLATE: 3

BAGG ENGINEERS



Source: Quaternary Geologic Map of the San Jose West Quadrangle, California,
Modified from Knudsen and Others (2000), SHZR 058 for San Jose West Quadrangle

Qhf - Holocene alluvial fan deposits

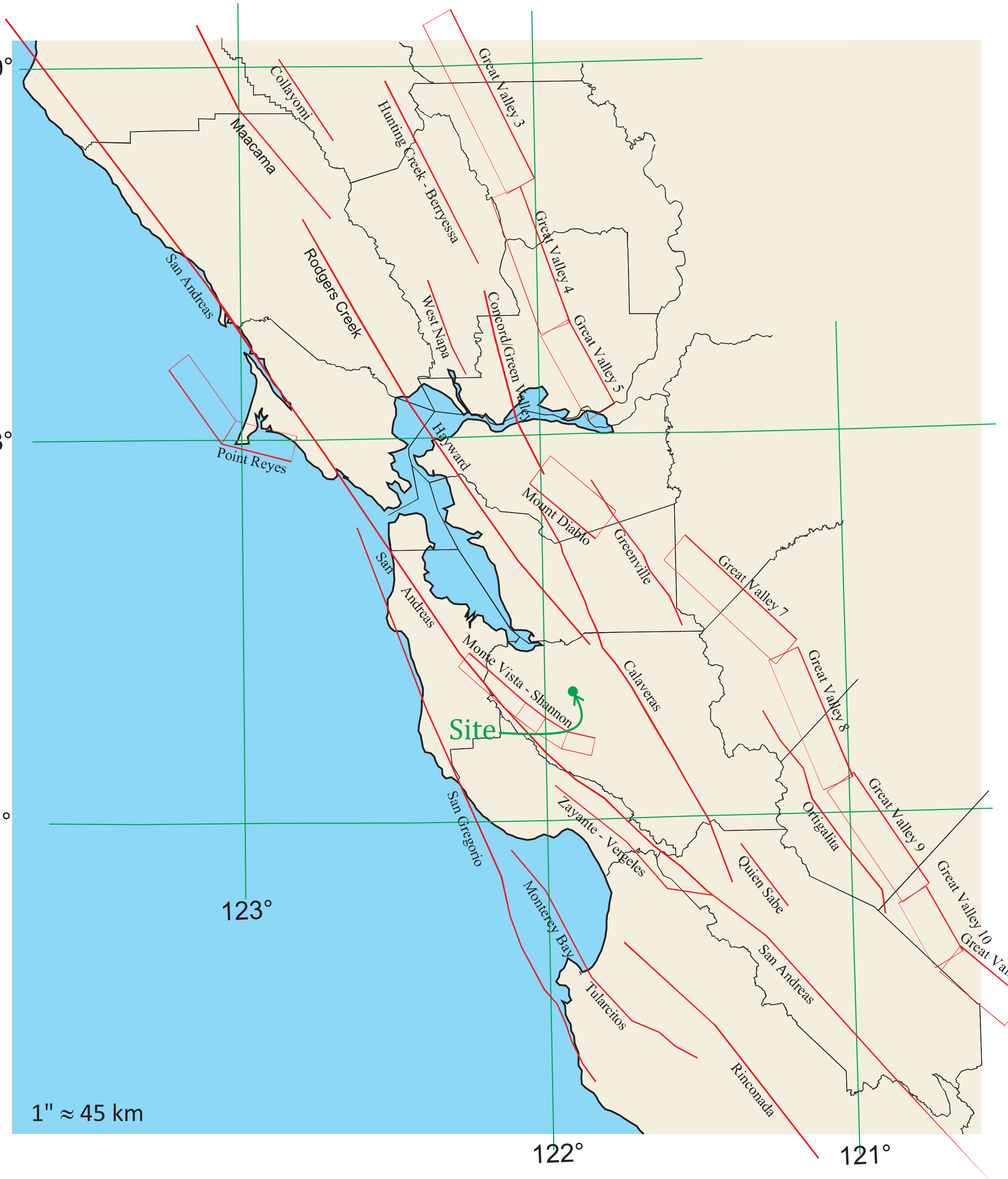
Qhl - Holocene alluvial fan levee deposits

Qhc - Modern stream channel deposits

Qht - Holocene stream terrace deposits

GEOTECHNICAL ENGINEERING INVESTIGATION RACE STREET RESIDENTIAL SAN JOSE, CALIFORNIA	LOCAL GEOLOGIC MAP		
	DATE: JUNE 2017	JOB NUMBER: COREH-16-00	PLATE 4

BAGG ENGINEERS



GEOTECHNICAL ENGINEERING INVESTIGATION
RACE STREET RESIDENTIAL
SAN JOSE, CALIFORNIA

REGIONAL FAULT MAP

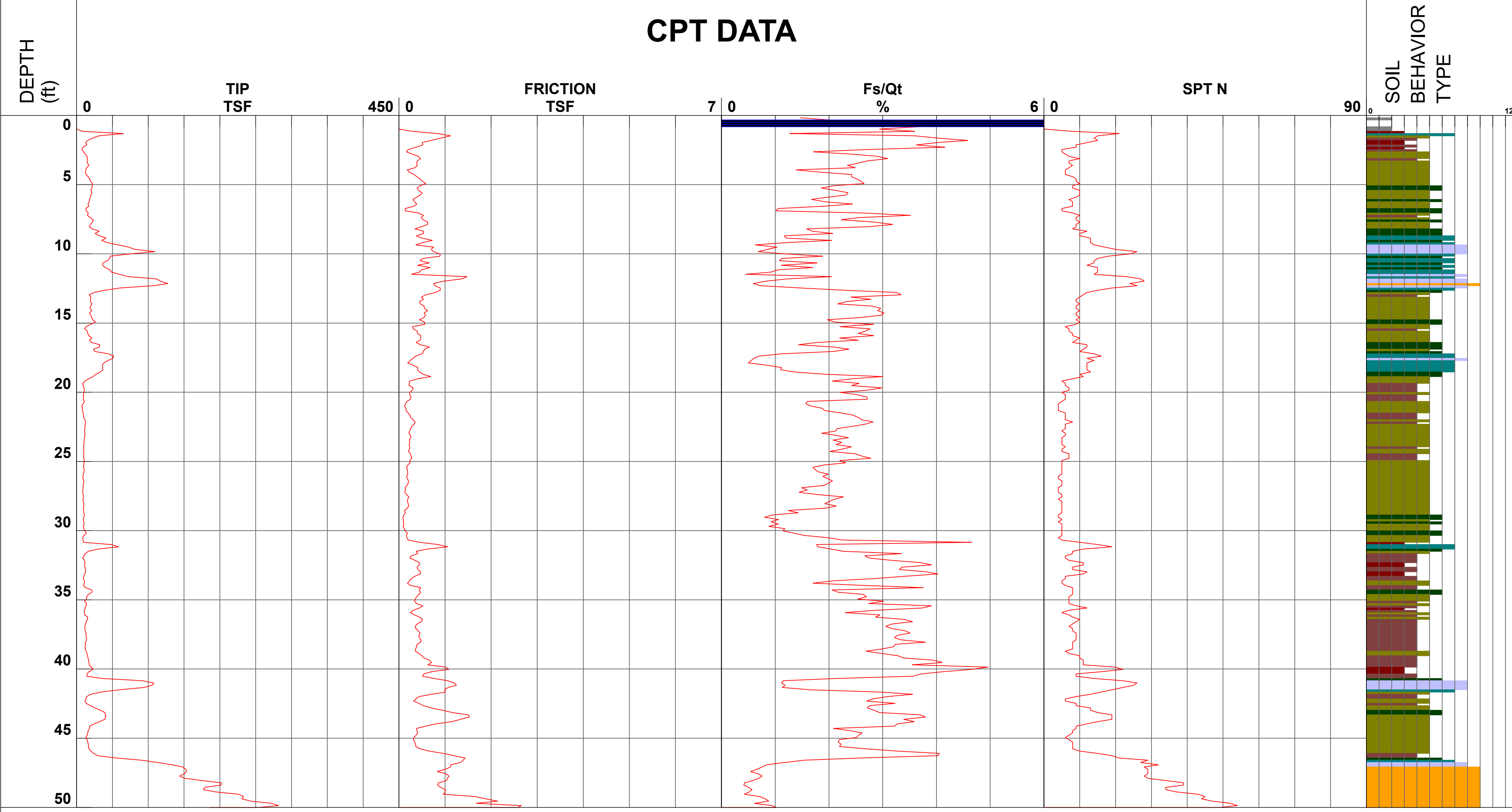
DATE:	JOB NUMBER:	PLATE
JUNE 2017	COREH-16-00	5

BAGG ENGINEERS

BAGG Engineers

Project	Race Street	Operator	RB KK	Filename	SDF(051).cpt
Job Number	COREH-16-00	Cone Number	DDG1281	GPS	
Hole Number	CPT-01	Date and Time	6/5/2017 8:50:25 AM	Maximum Depth	50.36 ft
EST GW Depth During Test		26.00 ft			

Net Area Ratio .8



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

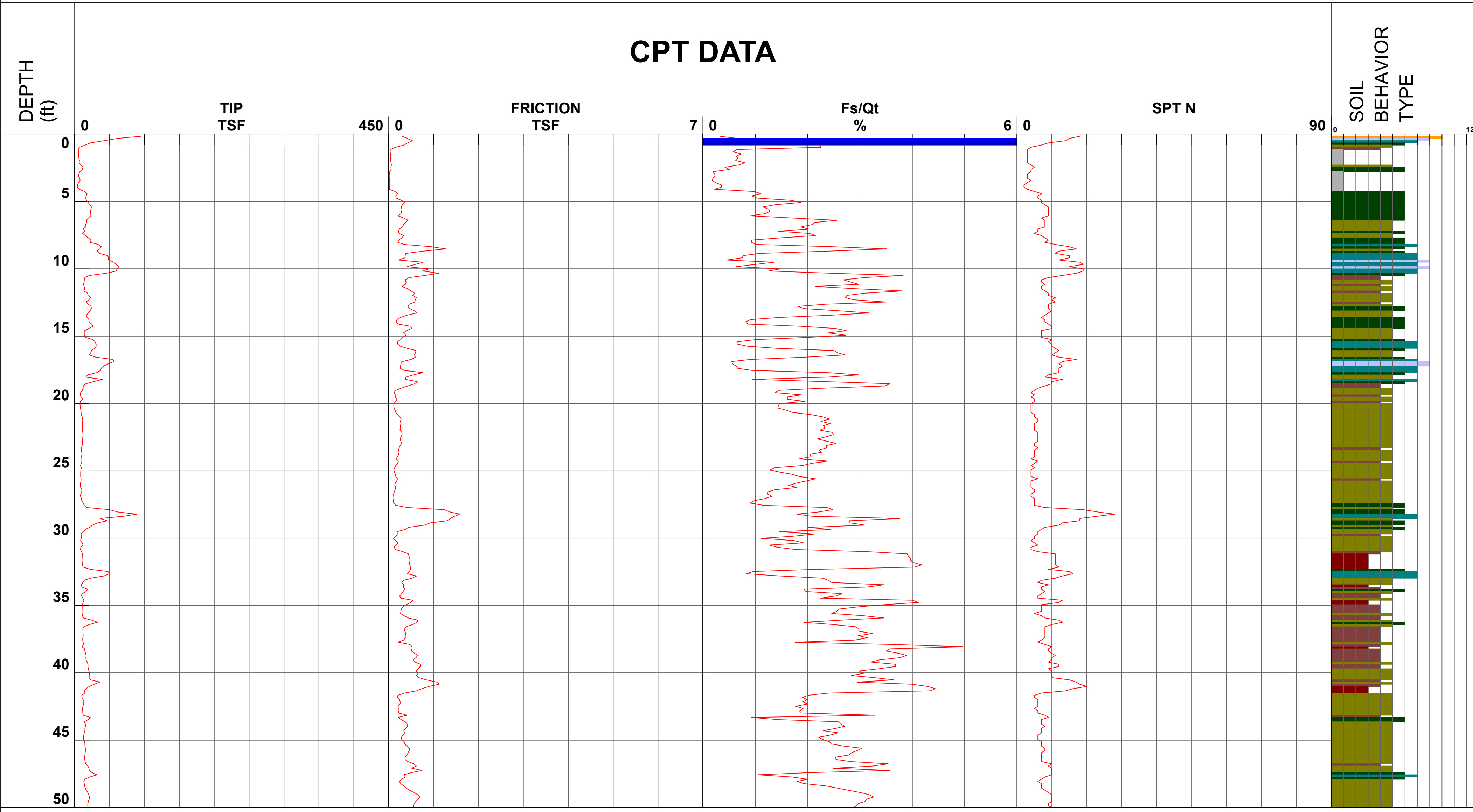
Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

BAGG Engineers

Project	Race Street	Operator	RB KK	Filename	SDF(053).cpt
Job Number	COREH-16-00	Cone Number	DDG1281	GPS	
Hole Number	CPT-02	Date and Time	6/5/2017 10:30:39 AM	Maximum Depth	50.36 ft
EST GW Depth During Test		28.2 ft			

Net Area Ratio .8



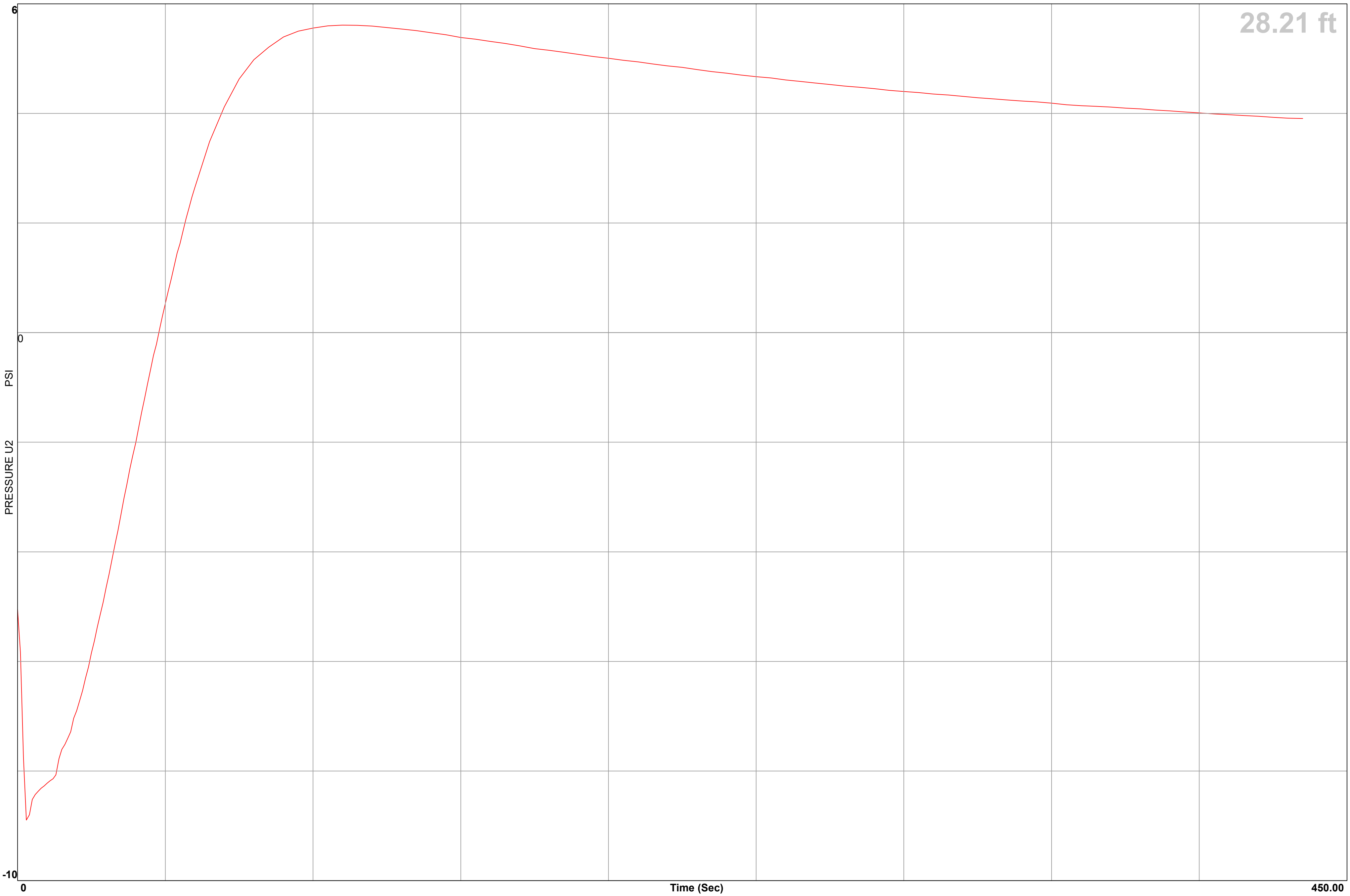
1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

BAGG Engineers

Location	Race Street	Operator	RB KK		
Job Number	COREH-16-00	Cone Number	DDG1281	GPS	
Hole Number	CPT-02	Date and Time	6/5/2017 10:30:39 AM		
Equilized Pressure	3.9	EST GW Depth During Test	28.2		

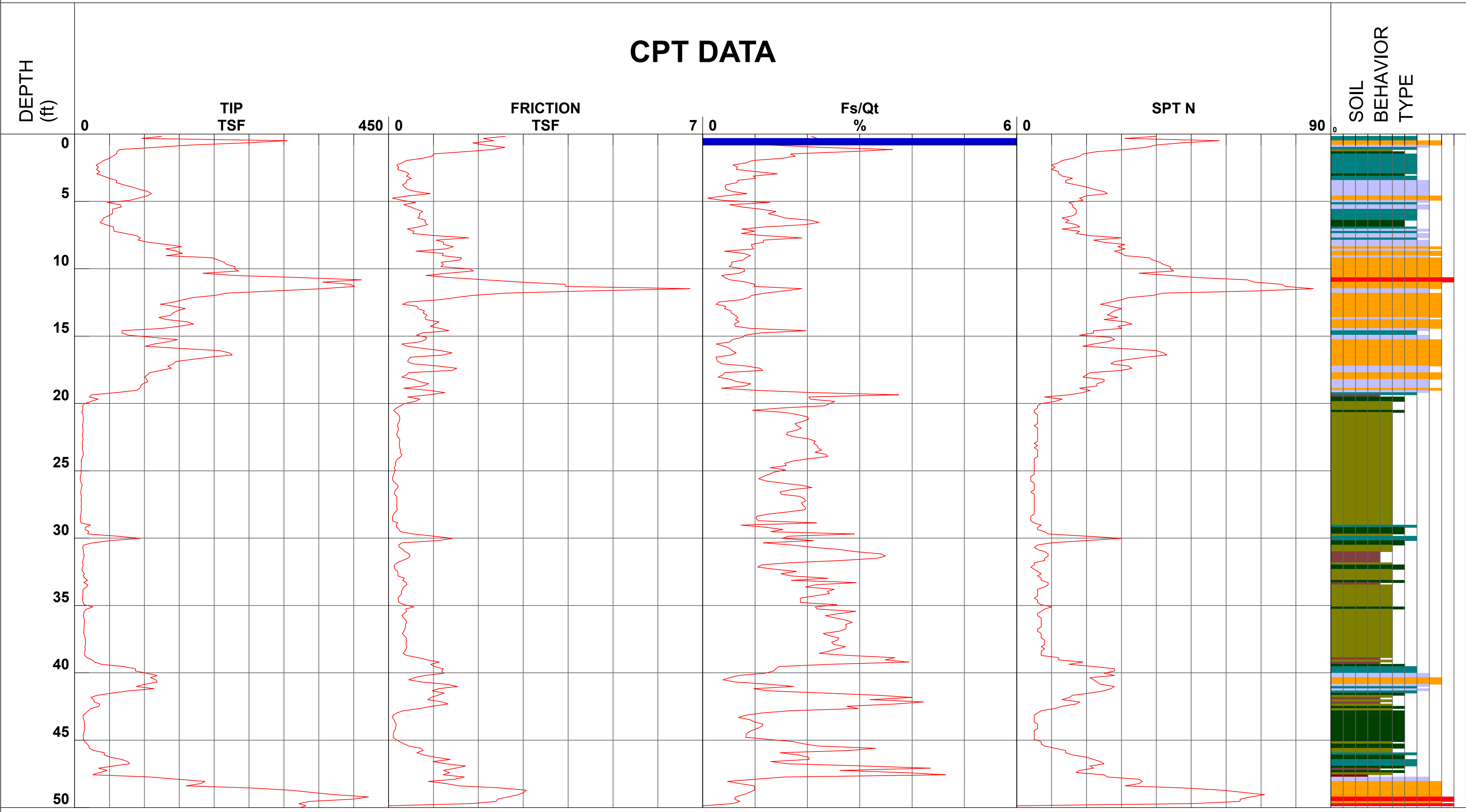


BAGG Engineers

Project	Race Street	Operator	RB KK	Filename	SDF(052).cpt
Job Number	COREH-16-00	Cone Number	DDG1281	GPS	
Hole Number	CPT-03	Date and Time	6/5/2017 9:27:05 AM	Maximum Depth	50.03 ft
EST GW Depth During Test		29.20 ft			

Net Area Ratio .8

CPT DATA



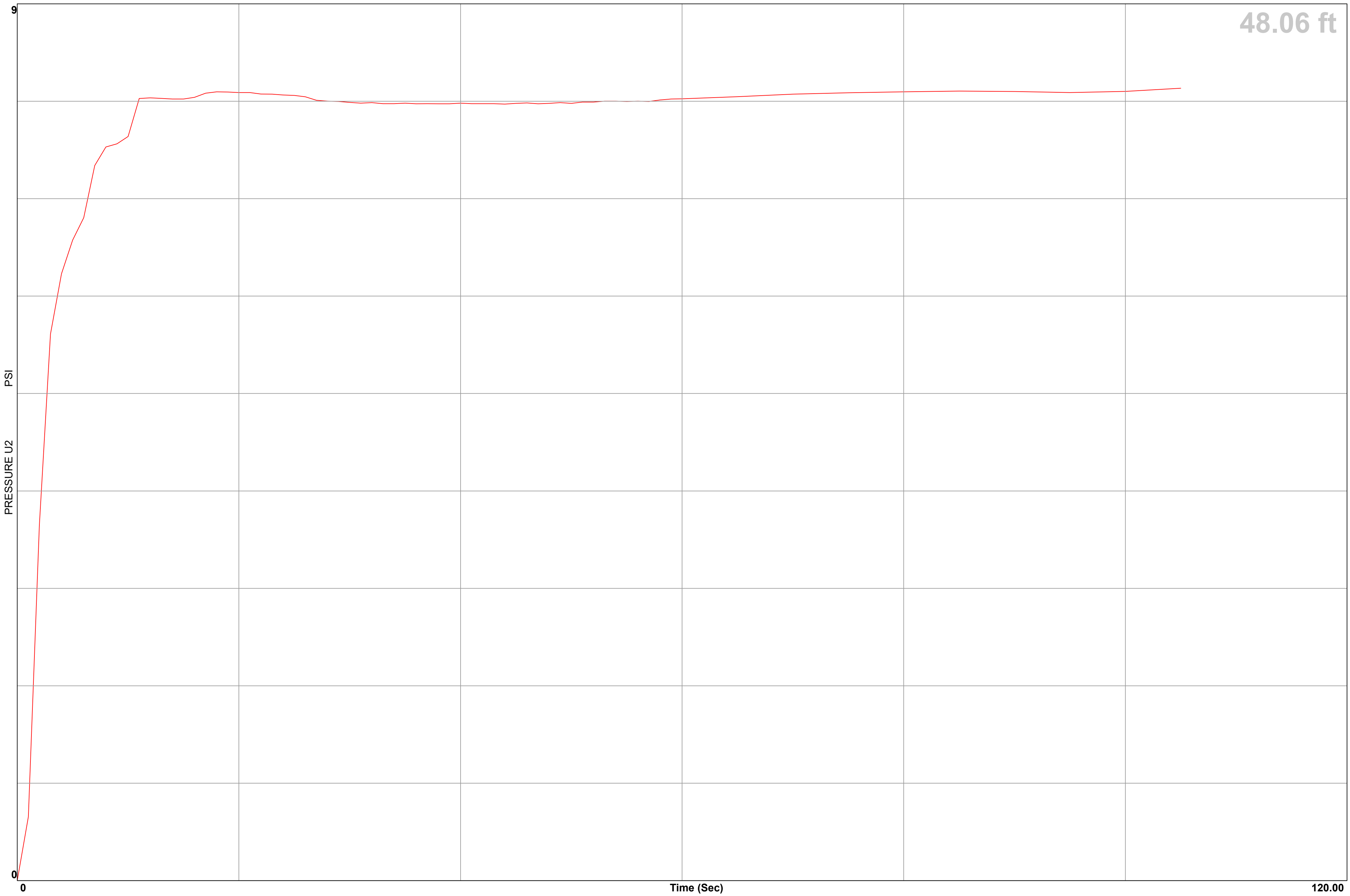
1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

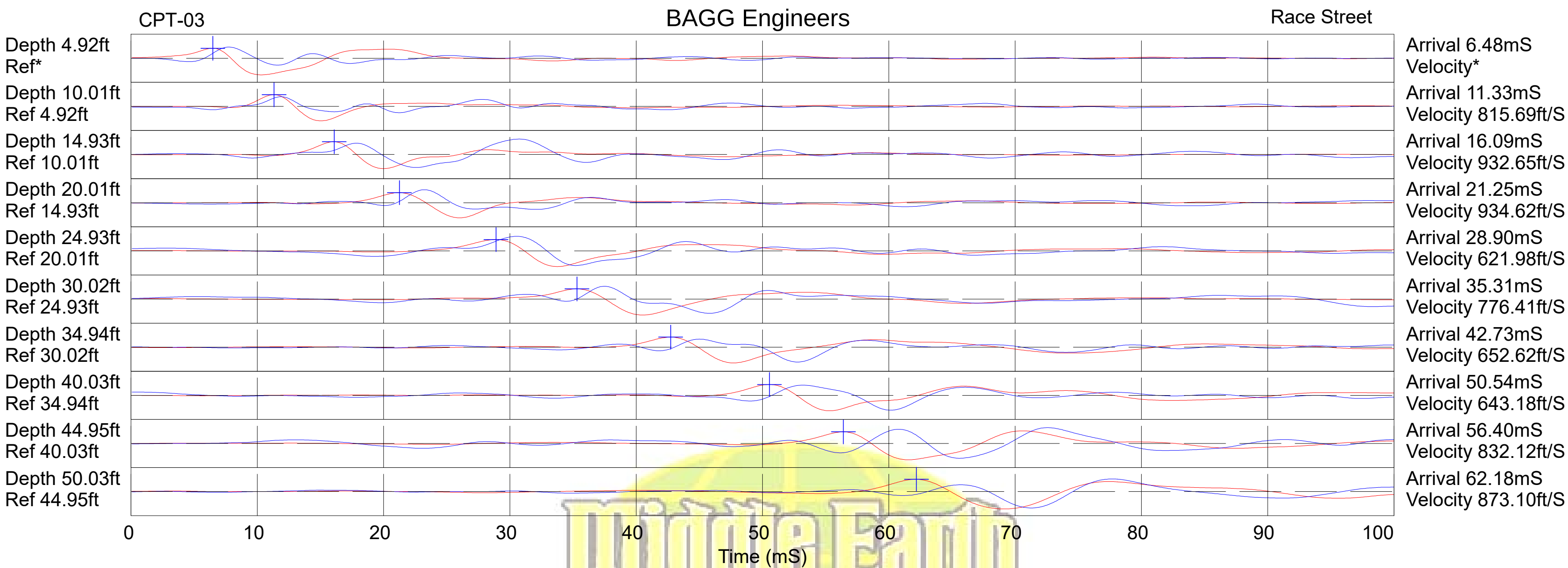
Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

BAGG Engineers

Location	Race Street	Operator	RB KK		
Job Number	COREH-16-00	Cone Number	DDG1281	GPS	
Hole Number	CPT-03	Date and Time	6/5/2017 9:27:05 AM		
Equilized Pressure	8.1	EST GW Depth During Test	29.2		





Hammer to Rod String Distance (ft): 5.83

* = Not Determined

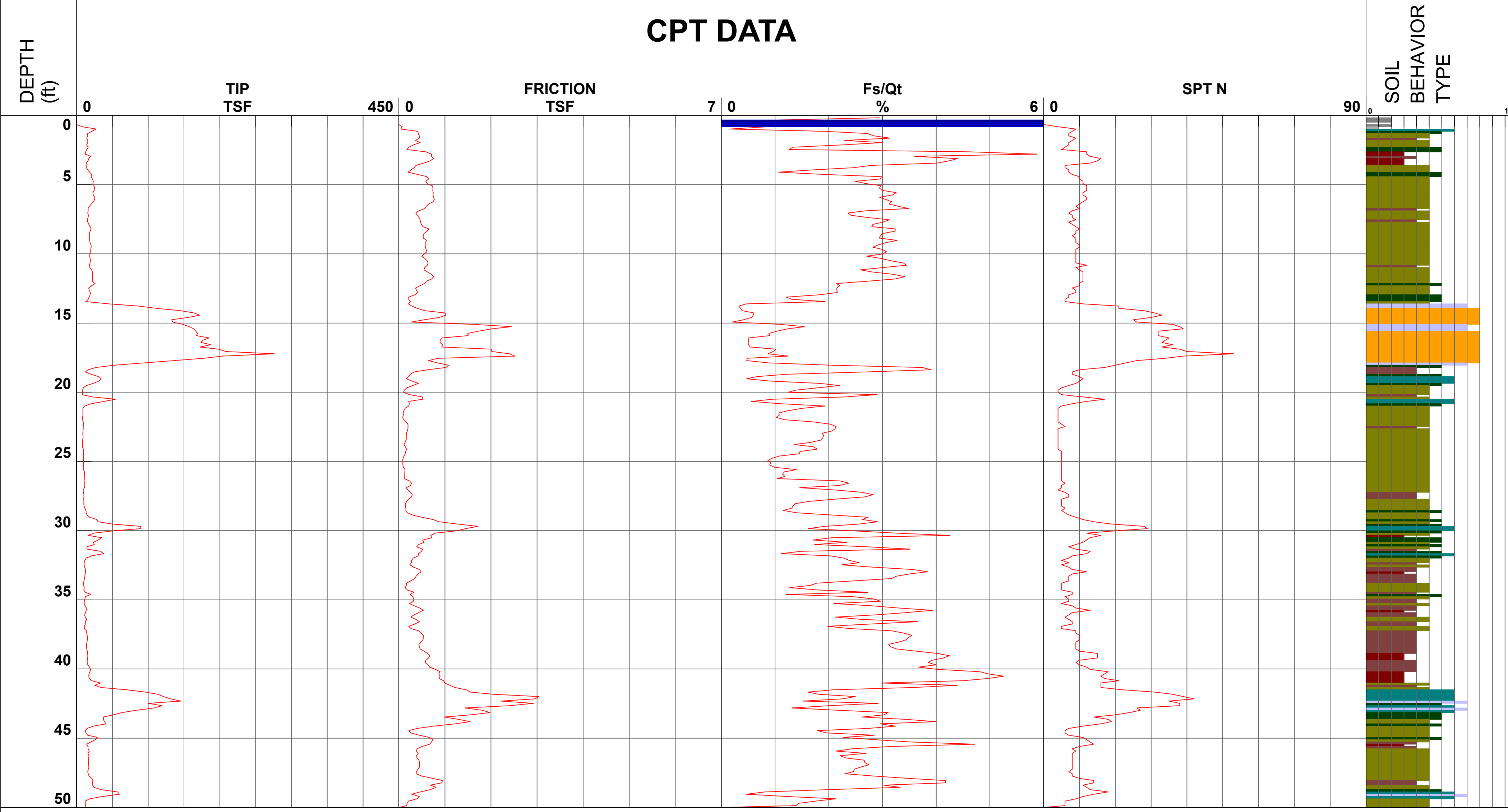
COMMENT:

BAGG Engineers

Project	Race Street	Operator	RB KK	Filename	SDF(054).cpt
Job Number	COREH-16-00	Cone Number	DDG1281	GPS	
Hole Number	CPT-04	Date and Time	6/5/2017 11:43:53 AM	Maximum Depth	50.20 ft
EST GW Depth During Test		28.00 ft			

Net Area Ratio .8

CPT DATA



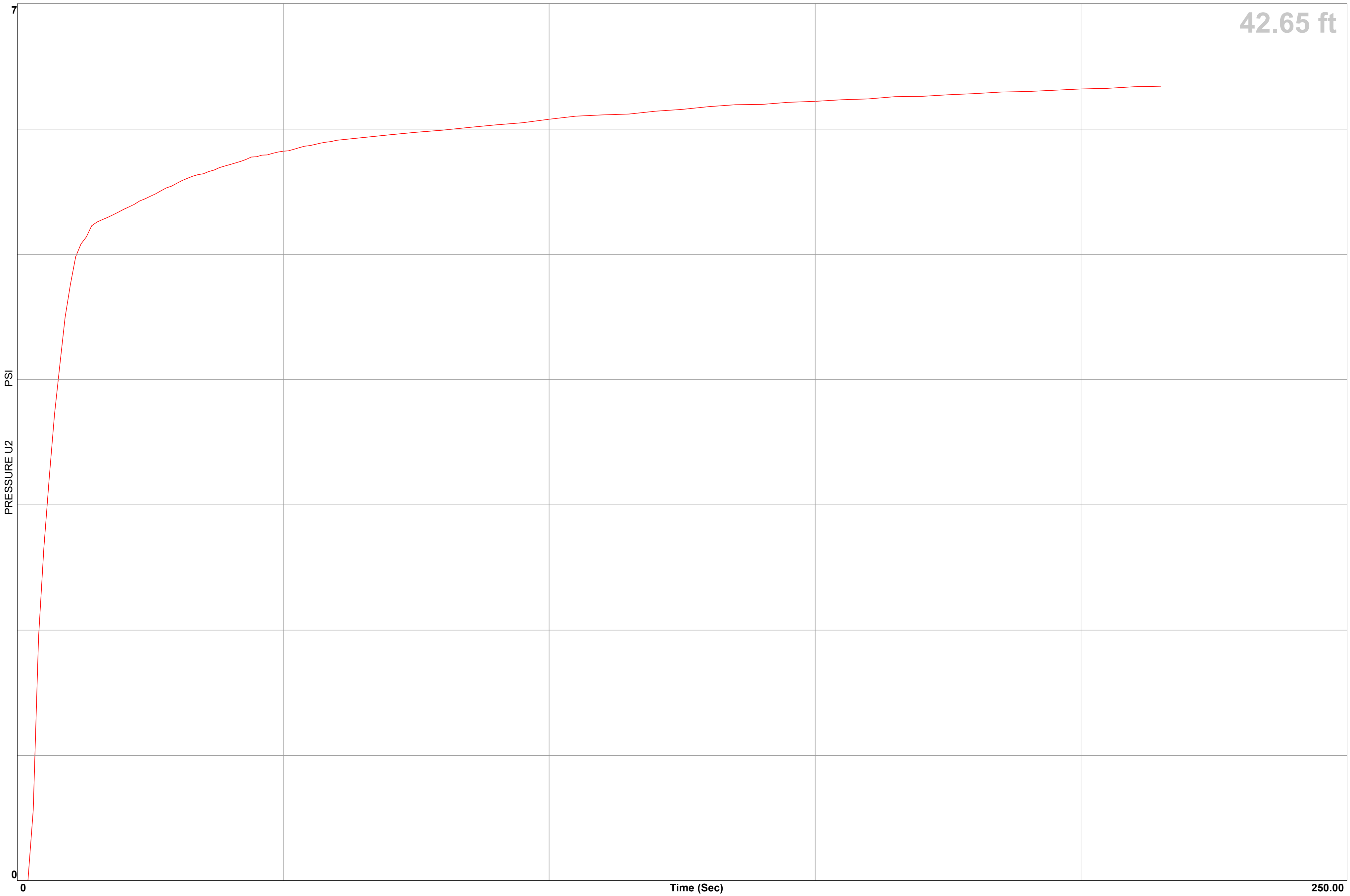
1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

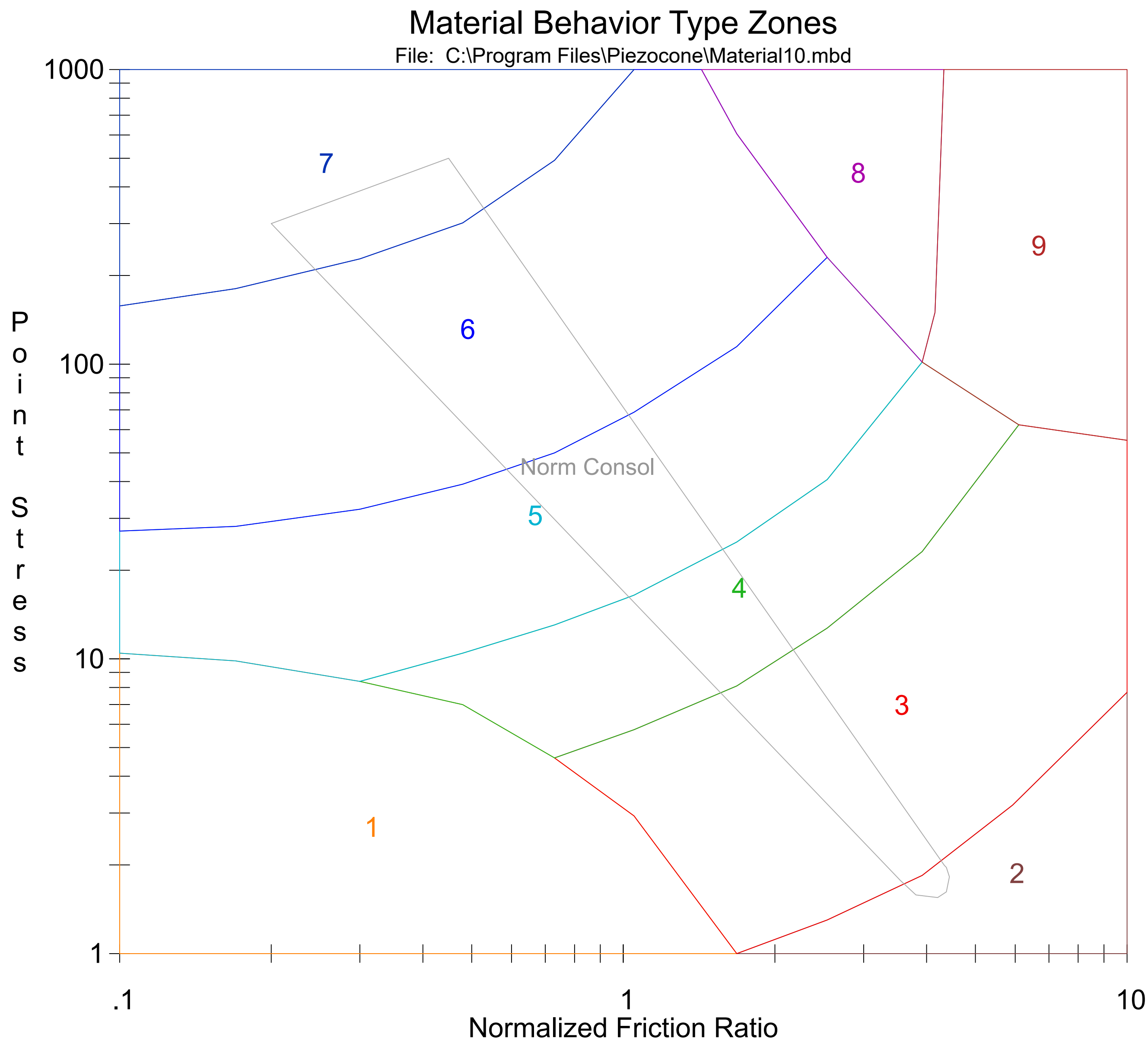
Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

BAGG Engineers

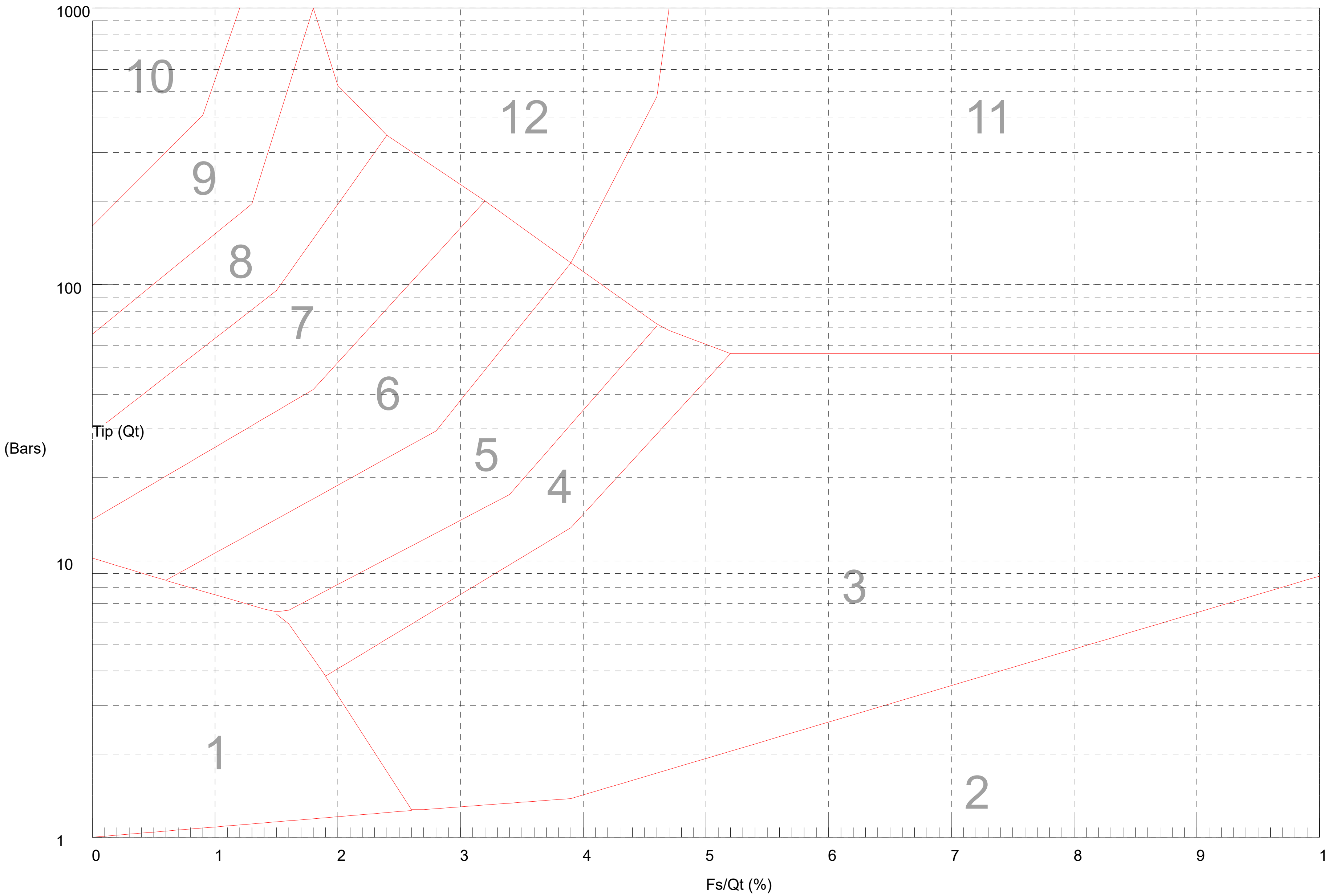
Location	Race Street	Operator	RB KK		
Job Number	COREH-16-00	Cone Number	DDG1281	GPS	
Hole Number	CPT-04	Date and Time	6/5/2017 11:43:53 AM		
Equilized Pressure	6.3	EST GW Depth During Test	28.0		





1. sensitive fine SOIL
2. Organic SOILS - Peats
3. silty CLAY to CLAY
4. clayy SILT to silty CLAY
5. silty SAND to sandy SILT
6. clean SAND to silty SAND
7. grvly SAND to dense SAND
8. stiff SAND to clayy SAND
9. very stiff fine SOIL

Classification Data:
Robertson and Campanella UBC-1983



1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)