Geotechnical Engineering Report

368 & 402 PETALUMA BOULEVARD NORTH

Petaluma, California

WKA No. 10410.02

March 27, 2015

(Revised July 8, 2015)

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INTRODUCTION

We have completed a revised geotechnical engineering study for the proposed mixed-used complex (parking and apartments) to be constructed at 368 and 402 Petaluma Boulevard North in Petaluma, California (see Figure 1). Our office previously prepared a Geotechnical Engineering Report (WKA No. 10410.02, dated March 27, 2015) for the referenced project.

Since the report referenced above was completed, the layout of the mixed-use development planned for the western portion of the site has been revised. At the time our report was prepared, it was planned for the western portion of the site to be developed with two, four-story, mixed-use buildings. Review of a *Unit Composite* drawing, dated April 20, 2015 and prepared by Kephart, indicates development in the western portion of the site will now consist of one, four-story, mixed-use building. This building will be located in the southwestern portion of the site and will have a larger footprint than the building originally planned for the same area of the site (see Figure 2).

Farrell Design-Build (FDB) has been contacted to provide consultation services regarding design and construction of appropriate foundation systems to support the proposed structures at the site. Supplemental subsurface information, including additional liquefaction analysis, was recommended by FDB to further assist in the design of the foundation systems planned for this project. As a result of the revised layout in the western portion of the site and the requested supplemental subsurface information, additional exploration was performed within the revised footprint of the building located in the western portion of the site and the original footprint of the building located in the eastern portion of the site.

The purposes of our study have been to explore the existing site, geologic, soil and groundwater conditions across the site, and to provide geotechnical engineering conclusions and recommendations for use in design and construction of the proposed mixed-use complex. This report presents the results of our study.

Scope of Services

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

1. Perform a site reconnaissance;
2. Review of previous geotechnical and environmental studies prepared for the project site;
3. Review of United States Geological Survey (USGS) topographic maps, geologic maps, historical aerial photographs and available groundwater information;
4. Perform subsurface explorations, including the drilling and sampling of six borings to depths ranging from about 25 to 31 feet below the existing site grades. We also advanced six cone penetrometer test (CPT) soundings to depths ranging from about 41 to 92 feet below existing site grades;
5. Collect representative bulk samples of near-surface soils;
6. Perform laboratory testing of selected soil samples;
7. Perform engineering analyses; and,
8. Prepare this report.

Our office also prepared *Phase I Environmental Site Assessment* (WKA No. 10410.01, dated March 4, 2015) and *Summary of Initial Hydrogeologic Modeling* (WKA No. 10410.03, dated May 28, 2015) reports for the mixed-use complex site.

Figures and Attachments

The following figures are included with this report:

TABLE 1 - FIGURES

Figure	Title	Figure	Title
1	Vicinity Map	4	Geologic Map
2	Site Plan	5 through 10	Logs of Soil Borings D1 through D6
3	Historical Building Location Map	11	Unified Soil Classification System

Appended to this report are:

- General information regarding project concepts, exploratory methods used during our field investigation and laboratory test results not included on the Logs of Soil Borings are included as Appendix A.
- Copies of the CPT reports provided by Gregg Drilling & Testing, Inc are included as Appendix B.

- Copies of the output files for the liquefaction analysis and associated data are included as Appendix C.
- *Guide Earthwork Specifications* and *Guide Specifications for Auger Cast-In-Place (ACIP) Piles*, both of which may be used in the preparation of contract documents, are included as Appendix D and E, respectively.

Proposed Development

Based on review of the *Unit Composite* drawing (see Figure 2), dated April 20, 2015 and prepared by Kephart, we understand the project will include demolition and complete removal of former and existing structures at the site, and construction of:

- A cast-in-place (CIP), reinforced concrete, podium-slab structure to be used as a two-level parking garage; and,
- Two four-story, mixed-use buildings to be used for parking and apartments. The ground floor of these buildings will consist of parking areas, while the remaining levels will include apartments. We assume the upper floors (apartments) of the buildings will be constructed of wood-framing and the wood-framed structures will be supported on CIP, reinforced concrete, podium-slab structures.

The buildings will include an elevator system; however, below-grade floors are not planned for this project. The *Unit Composite* drawing indicates the proposed mixed-use complex will consist of 322 parking spaces and 210 apartment units. Structural loads for the building are anticipated to be relatively light to moderate based on this type of construction. Based on the anticipated structural loads, encountered subsurface soils conditions, and conversations with members of the design team, we understand the podium-slab structures may be supported on a shallow foundation system over ground improved using Geopier® rammed aggregate piers [RAPs], vibratory Impact® piers, or a similar system. While currently not being considered as a foundation system for this project, as an alternative, we have also provided recommendations for a deep foundation system consisting of auger cast-in-place (ACIP) piles.

Associated development will include construction of landscaped areas, underground utilities, exterior flatwork, an elevated deck with a spa or swimming pool, parking areas and drive aisles associated with the lower floors of the mixed-use building, and the extensions Oak Street and Water Street. We understand the grade for the extension of Oak Street will be raised to match the existing grade of Petaluma Boulevard North, which is estimated to be about five (5) feet higher than the elevation of existing site grades.

A grading plan was not available at the time this report was prepared; however, based on the existing site topography and our understanding of the project, we anticipate cuts and fills may be on the order of one (1) to five (5) feet, depending on the extent of disturbance caused by removal of former and existing structures.

Supplemental Information

Supplemental information used in the preparation of this report included review of the following reports prepared for the project site:

- PJC & Associates, Inc., *Preliminary Geotechnical Investigation* (Job No. 3632.01, dated March 26, 2008) prepared for The Fifth Resource Inc.;
- MACTEC Engineering and Consulting, Inc., *Phase I Environmental Site Assessment* (MACTEC Project No. 4088087520 01, dated May, 9 2008) prepared for North River Landing, L.P.;
- ECON, *Phase II Environmental Site Assessment* (dated March 31, 2009) prepared for North River Landing, L.P.; and,
- ECON, *Phase I Environmental Site Assessment* (dated July 9, 2014) prepared for Pacifica Companies LLC.

We also reviewed available information on the State Water Resource Control Board (SWRCB) GeoTracker website for the site and its vicinity.

FINDINGS

Site Description

The project site is located at 368 and 402 Petaluma Boulevard North in Petaluma, California (see Figure 1). The irregular-shaped site covers an area of about 3.8 acres and is comprised of Sonoma County Assessor's Parcel Number's (APNs) 006-163-040 and -041 (see Figure 2). The site is bounded to the north by a car wash and animal feed factory; to the east by the Petaluma River; to the south by commercial development, a pump station and vacant land; and, to the west by Petaluma Boulevard North.

The topography of the site appears to be gently sloping from west to east. Based on review of the topographic map of the *Petaluma, California Quadrangle*, published by the USGS and dated 1981, the elevations at the site range from approximately +10 to +20 feet relative to mean sea level (msl).

At the time of our field explorations, February 17 and 18, and June 23, 2015, railroad tracks bisected the central portion of the site, generally extending from northwest to southeast. The northwest and southwest portions of the site are developed with one-story buildings. The building located in the northwestern portion of the site is actively used as office space and is raised up to about five (5) feet above existing site grades to match the street elevation of Petaluma Boulevard North. An asphalt paved driveway is located adjacent to and south of this building. The building located in the southwestern portion of the site is vacant, appeared to be previously used as a restaurant, and is raised up to about three (3) feet above existing site grades to match the street elevation of Petaluma Boulevard North. A concrete retaining wall is located adjacent to Petaluma Boulevard North, between the asphalt paved driveway and the building located in the southwestern portion of the site.

An interior floor slab supported on a shallow foundation system associated with previous development is located in the south-central portion of the site. Two rectangular-shaped openings were observed within the interior floor slab. Miscellaneous debris was observed within the openings; therefore, the depth to the bottom of the openings was undetermined.

Square-shaped, chain-link fencing is located in the central portion of site, west of the railroad tracks. We understand the enclosed area is used by a roofing company. Two metal storage containers, miscellaneous equipment, roofing materials and parked vehicles were observed within the fenced area. A rectangular-shaped concrete structure, measuring about two (2) feet in height, is located north of the fenced area. The use of the concrete structure is unclear.

Remnants of concrete foundations associated with previous structures, miscellaneous debris, small stockpiles consisting of aggregate base, overhead utilities, surface gravels, a moderately dense growth of volunteer grasses and weeds, and a dense growth of what appeared to be berry bushes were all observed in the remaining portions of the western portion of the site. The eastern portion of the site was largely covered with a moderately dense growth of volunteer grasses and supported a gravel-paved road that provides access to the animal feed factory.

Historical Document Review

Topographic Maps

We reviewed available historical topographic maps of the *Petaluma, California Quadrangle*, published by the USGS. Available maps were from the years 1914, 1942, 1953, 1954, 1968, 1981 and 2012. Review of the map from 1914 shows the Petaluma River meandering within the southern portion of the project site. Review of the remaining maps show the Petaluma River in its existing alignment. Based on this information, the river was constructed to its current alignment sometime between 1914 and 1942.

Aerial Photographs

We reviewed historical aerial photographs of the site available from our files and the Google Earth website. Available photographs were taken in the years 1942, 1952, 1965, 1968, 1973, 1982, 1993, 1998, 2005, 2006, 2009, 2010, and 2012 through 2014.

Review of the photograph taken in 1942 shows the western portion of the site developed with several buildings, including the two buildings currently present at the site. The eastern portion of the site is shown as vacant land apparently covered with vegetation. The railroad tracks currently present at the site are not shown on this photograph.

Review of the photograph taken in 1952 shows the railroad tracks. The remaining portions of the site show the site has generally remained the same since 1942.

Review of the photographs taken from 1965 to 2012 shows the site has generally remained the same since 1952. Review of the photograph taken in 2013 shows all of the buildings previously observed in the western portion of the site have been removed, with the exception of the two buildings currently present at the site. Remnants of concrete foundations observed in the western portion of the site during the completion of our field explorations are likely associated with demolition of the former buildings. This photograph also shows the interior floor slab, area currently used by a roofing company, and the gravel-paved road observed at the site during the completion of our field explorations. Review of the photograph taken in 2014 shows the site has generally remained unchanged since 2013.

Sanborn® Maps

One of the tasks included in our *Phase I Environmental Site Assessment* report prepared for the project site included review of available historical Sanborn® Maps. Sanborn® Maps are detailed drawings of site development, and were typically used by fire insurance companies to determine site fire insurability. Available Sanborn® Maps were from the years 1885, 1888, 1894, 1906, 1910, 1923, 1949, 1959, and 1965. Review of these maps revealed that prior to the building development observed in the historical aerial photograph taken in 1942, the western portion of the site was previously developed and re-developed with several different buildings. The general layout of the historical building development, along with a summary of the time of existence of the buildings, is shown on the Historical Building Location Map, Figure 3. Specific information regarding the buildings currently and previously located in the western portion is available in our *Phase I Environmental Site Assessment* report.

General Site Geology

Based on review of the *Geologic Map of the Petaluma 7.5' Quadrangle, Sonoma and Marin Counties, California*, published by the California Geological Survey (CGS) and dated 2002, the site is mapped as underlain by the Quaternary fan (Qhf) and terrace (Qhty) deposits. A portion of the referenced geologic map is shown in Figure 4.

The western portion of the site is mapped as underlain by Holocene-aged (less than 11,700 years old) fan deposits. Fan deposits are defined as alluvial sediments deposited by streams emanating from the mountains as debris flows, hyperconcentrated mudflows and braided stream flows. Sediments include sand, gravel, silt and clay that are moderately to poorly sorted, and are moderately to poorly bedded. The geologic deposits mapped on the western portion of the site are consistent with the soils data obtained from Borings D1 through D3, and CPT1 and CPT2.

The eastern portion of the site is mapped as underlain by terrace deposits. Stream-terrace deposits are judged to be latest-Holocene in age (less than 1,000 years old) based on records of historical inundation, the presence of youthful meander scars and braid bars, or geomorphic position very close the stream channel. Stream-terraces are deposited as point bar and overbank deposits along the Petaluma River. Terrace sediments include sand, gravel, silt and clay that are moderately to well sorted, and moderately to well bedded. The geologic deposits mapped on the eastern portion of the site are consistent with the soils data obtained from Borings D4 through D6, and CPT3.

Subsurface Soil Conditions

Six borings (D1 through D6) and six cone penetration test soundings (CPT1 through CPT6) were performed at the site on February 17 and 18, and June 23, 2015. The approximate locations of the explorations are shown on the Site Plan, Figure 2. The surface and near-surface soil conditions encountered in the explorations are variable and are summarized below.

Based on Borings D1 through D3 and CPT2 through CPT4, the surface and near-surface soil conditions within the western portion of the site generally consist of interbedded layers of soft to stiff, silty and sandy clays and loose to dense, silty and clayey sands to depths ranging from about 13½ to 24 feet below existing site grades. Subsurface soil conditions beneath these soils generally consist of very dense, silty and poorly-graded sands to the explored depth of about 50 feet below existing site grades. Variably cemented soils were encountered in Borings D1 through D3 at depths between 8½ to 31 feet below existing site grades. Undocumented fill soils consisting of medium dense, silty sands and medium stiff, sandy clay were encountered in

Borings D1 and D3 to depths ranging from 2½ to five (5) feet below existing site grades. The fill soils are likely associated with previous development activities within the western portion of the site.

Based on Borings D4 through D6 and CPT1, CPT5 and CPT6, the surface and near-surface soil conditions within the eastern portion of the site generally consist of interbedded layers of very soft to medium stiff, highly expansive, compressible, silty clays and very loose to loose silty and clayey sands to depths ranging from about 25 to 38 feet below existing site grades. Various concentrations of organics were observed within these soils. Based on review on historical topographic and geologic maps, these soils are likely fill deposits associated with the re-alignment of the Petaluma River that occurred sometime between 1914 and 1942. Subsurface soil conditions beneath these soils generally consist of a very dense mixture of silty and poorly graded sands and silty gravels to the maximum explored depth of 87 feet below existing site grades. Variable degrees of cementation were observed in samples collected from Borings D4 through D6 within the very dense, granular soils.

The soil conditions encountered in our explorations are consistent with those encountered in previous studies performed at the site, and also with the mapped geology. For specific information regarding the soil conditions at a specific exploration location, please refer to the Logs of Soil Borings, Figures 5 through 10, and/or the CPT reports included in Appendix B.

Groundwater

Groundwater was observed in all of our borings performed on February 17 and 18, 2015, at depths ranging from five (5) to 16 feet below existing site grades. Please note these borings may not have been left open long enough for groundwater to reach static equilibrium.

To supplement our groundwater data, we reviewed groundwater information available on the SWRCB GeoTracker website for the site and its vicinity. Our review indicates a monitoring well was previously constructed adjacent to the northeast corner of the existing building located in the northwestern portion of the site (see Figure 2). The well was identified as MW-11 and was installed by EDD Clark & Associates, Inc. as part of a groundwater monitoring program for the property located adjacent to northwest of the project site. Records indicate the well was constructed on April 15, 2003 and was abandoned on June 28, 2010. The well was drilled to a depth of 21½ feet below site grades. The surface elevation near the well is indicated to be about +12 feet msl, which is consistent with topography information available for the project site. Groundwater was recorded at a depth of about five (5) feet below site grades during the construction of the well (approximate elevation of +7 feet msl). Records indicate that groundwater depths at MW-11 fluctuated form about one (1) to four (4) feet below existing site grades (approximate elevation of about +11 to +8 feet msl, respectively) from April 15, 2003 to at least March 17, 2010. These groundwater elevations are consistent with the groundwater levels observed during our field explorations and previous explorations performed by others at the site and its general vicinity.

Environmental Concerns

Review of our *Phase I report* (WKA No. 10410.01) revealed no environmental liens associated with the project site; however, separate groundwater contamination plumes have been delineated for the properties adjacent to the northwest and north of the site. Review of available documents did not reveal evidence of the groundwater contamination plumes extending to the site. Other on-site environmental concerns included the previous existence of an underground gasoline storage tank in the northwestern portion of the site, former buildings in the western portion of the site labeled as “oil storage” and “garages”; and, the potential for asbestos-containing building materials and residues of historically applied lead from lead-based paint and persistent pesticides termiticides due to the age of the existing and historical development of the site.

Recommendations in our *Phase I* report included annual file review for the adjacent properties where groundwater contamination plumes have been delineated, and the collection of soil samples and laboratory testing to evaluate potential impacts associated with the former underground storage tank, existing and former structures, fill soils associated with realignment of the Petaluma River and the existing railroad tracks. For more specific information regarding environmental concerns at the site and our recommendations, please review our *Phase 1* report.

During our field explorations, we used a hand-held, photoionization detector (PID) to field screen soil samples for volatile organic compounds (VOCs). No detectable or elevated concentrations of VOCs were detected with the PID. Results of the PID readings are noted on the Logs of Soil Borings, Figures 5 through 10, at each sample depth.

We understand that based on the preliminary project design, dewatering portions of the site to a depth of 12 feet below existing site grades may be required during construction of the planned improvements. The center of the nearest groundwater plume has been estimated by others to be about 170 feet north of the site boundary. Review of our *Initial Hydrogeologic Modeling* report (WKA No. 10410.03) revealed it would take approximately 188 pumping (dewatering) days at the site to draw water associated with the center of the nearest groundwater contamination plume. The actual time may take longer because the flow vectors would not be perpendicular to the site boundaries. Recommendations in our *Initial Hydrogeologic Modeling* report included weekly monitoring of water produced during dewatering activities to be analyzed for VOCs, including methyl-tert-butyl ether (MTBE). Please note that the analysis included in the referenced modeling report is considered preliminary. For more specific information regarding environmental concerns associated with dewatering at the site and our recommendations, please review our *Initial Hydrogeologic Modeling report*.

CONCLUSIONS

Seismicity and Faults

The site is located in relatively close proximity to the seismically active San Francisco Bay area. The Alquist-Priolo Earthquake Fault Zoning Act (AP Act) has defined an active fault as one that has ruptured in the last 11,700 years. Review of the *Fault Activity Map of California*, dated 2010 and prepared by the CGS, shows there are several active faults located within a 20 mile radius of the site. The most notable active faults in the vicinity of the site are those associated with the: Rodgers Creek fault zone, located about 10 miles east of the site; San Andreas fault zone, located about 15 miles west of the site; West Napa fault zone, located about 20 miles east of the site; and, the Hayward fault zone, located about 20 miles southeast of the site. The epicenter for the August 14, 2014 earthquake on the West Napa Fault has been located about 18 miles east of the site. Based on this information, the potential for the site to experience significant ground shaking from future earthquakes is relatively high.

2013 CBC/ASCE 7-10 Seismic Design Criteria

Shear Wave Seismic Velocity and Seismic Site Class

Shear wave velocities obtained at location CPT1 varied from about 401 to 1942 feet per second (fps) within the upper 87 feet of the soil profile. The average shear wave velocity within the upper 87 feet at CPT1 was determined in accordance with Section 1613.3.2 of the 2013 California Building Code (CBC) and Chapter 20 of American Society of Civil Engineers, *Minimum Design Loads for Building and Other Structures (ASCE 7-10)* and was determined to be about 1341 fps. Based on Table 20.3-1 of ASCE 7-10, a seismic Site Class C applies to sites with average shear wave velocities between 1200 and 2500 fps. However, due to the presence of relatively young and relatively loose/soft soil conditions within the upper 13½ to 38 feet of the soil profile (depending on location of the site), we recommend the soils at this site be designated as Site Class D in determining seismic design forces for this project in accordance with Section 1613A.3 of the 2013 CBC. A summary of the shear wave velocity data collected from CPT1 is included in Appendix B.

Seismic Design Parameters

Section 1613A of the 2013 edition of the CBC references ASCE 7-10 for seismic design. The seismic design parameters provided below are based on the site latitude and longitude using the United States Seismic Design Maps public domain computer program developed by the USGS (Version 3.1.0, July 11, 2013). The 2013 CBC parameters provided in Table 2 should be used for seismic design of the proposed mixed-use structures.

TABLE 2 - 2013 CBC/ASCE 7-10 SEISMIC DESIGN PARAMETERS

Latitude: 38.2388° N Longitude: 122.6418° W	ASCE 7-10 Table/Figure	2013 CBC Table/Figure	Factor/ Coefficient	Value
Short-Period MCE at 0.2-seconds	Figure 22-1	Figure 1613.3.1(1)	S_S	1.527 g
1.0-second Period MCE	Figure 22-2	Figure 1613.3.1(2)	S_1	0.600 g
Soil Class	Table 20.3-1	Section 1613.3.2	Site Class	D
Site Coefficient	Table 11.4-1	Table 1613.3.3(1)	F_a	1.000
Site Coefficient	Table 11.4-2	Table 1613.3.3(2)	F_v	1.500
Adjusted MCE Spectral Response Parameters	Equation 11.4-1	Equation 16-37	S_{MS}	1.527 g
	Equation 11.4-2	Equation 16-38	S_{M1}	0.900 g
Design Spectral Acceleration Parameters	Equation 11.4-3	Equation 16-39	S_{DS}	1.018 g
	Equation 11.4-4	Equation 16-40	S_{D1}	0.600 g
Seismic Design Category	Table 11.6-1	Section 1613.3.5(1)	Risk Category I to IV	D
	Table 11.6-2	Section 1613.3.5(2)	Risk Category I to IV	D

Notes:
MCE = Maximum Considered Earthquake
g = Gravity

Liquefaction Potential

Liquefaction is a soil strength and stiffness loss phenomenon that typically occurs in loose, saturated cohesionless soils as a result of strong ground shaking during earthquakes. The potential for liquefaction at a site is usually determined based on the results of a subsurface geotechnical investigation and the groundwater conditions beneath the site. Hazards to buildings associated with liquefaction include bearing capacity failure, lateral spreading, and differential settlement of soils below foundations, which can contribute to structural damage or collapse.

The findings of the borings and cone penetration tests performed at the site revealed the underlying soils generally consists of relatively loose/soft interbedded sand and clay layers overlying relatively dense sands and gravels extending to the explored depths of 25 to 31 feet at the borings and 41 to 92 feet at the CPTs. Historical high groundwater is indicated to be about two (2) feet below the existing ground surface. Based on the soil conditions encountered at the boring and CPT locations and the anticipated high groundwater level at the site, an evaluation of the liquefaction potential is required at the site in accordance with the 2013 CBC.

A liquefaction analysis to determine factors of safety against liquefaction was performed for the soil and groundwater conditions encountered at CPT1 through CPT6.

Liquefaction Analysis and Results

In performing our liquefaction analysis we used the soil liquefaction assessment software LiqIT (Version 4.7) developed by GeoLogismiki that utilizes data collected from CPT soundings to determine factors of safety against liquefaction for varying earthquake input energies. The program uses the results of the National Center for Earthquake Engineering Research (NCEER) liquefaction evaluation methods summarized by Youd, et al (2001). Input values were obtained using the results of CPT1 through CPT6. A design groundwater level of two (2) feet below the existing ground surface during a design earthquake was used in our analysis based on our review of historic groundwater levels at the site. A peak ground acceleration (PGA_M) of 0.57 g was used in the liquefaction analysis based on Equation 11.8-1 of ASCE Standard 7-10. A mode magnitude earthquake of 7.05 was used for this analysis using the 2008 USGS National Seismic Hazard Mapping Project (NSHMP) Probabilistic Seismic Hazard Analysis (PSHA) Interactive Deaggregation web site.

Our analysis of the CPT data indicates that most of the soils encountered in the CPTs are soils with safety factors of 1.3 or greater against liquefaction. However, the analysis reveals that relatively thin discrete soil layers within the CPTs possess safety factors between about 0.16 and 1.3. A factor of safety of 1.3 or greater against liquefaction potential is generally considered acceptable (liquefaction-induced settlement unlikely).

Copies of the output files for the liquefaction analysis, including the results of the 2008 USGS NSHMP PSHA Interactive Deaggregation, are provided in Appendix C.

Seismically Induced Settlement

Post-liquefaction settlement calculations within LiqIT are performed using the methodology of Ishihara and Yoshimine (1992). Given the results of our analysis performed for this investigation, the worst-case estimate of total post-liquefaction settlement is calculated to be less than four (4) inches of total settlement and less than about one (1) inch of differential settlement across 50 feet, or the least dimension of the structure, whichever is less. These estimates of post-liquefaction seismic settlements represent free-field ground settlement, not settlement of the proposed structures. In general, calculated liquefaction settlements increase in magnitude closer to the Petaluma River, located adjacent to the east.

Liquefaction potential at the site was also evaluated based on the Liquefaction Potential Index (LPI). The LPI is a measure of the liquefaction potential based on an analysis of the entire vertical soil profile not just discrete layers (Iwasaki, 1986; Toprak and Holzer, 2003). Factors taken into consideration for the LPI calculations include: thickness of the liquefied layer; proximity of the liquefied layer to the surface; and, the factor of safety. The LPI ranges from 0 to 100 with the value zero representing no liquefaction potential. Surface manifestations of liquefaction occur at LPI ≥ 5. The LPI for the CPT soundings are presented in the table below:

TABLE 3 – SUMMARY OF LIQUEFACTION POTENTIAL INDEX

CPT Sounding	LPI
CPT1	6.90
CPT2	1.73
CPT3	2.73
CPT4	3.38
CPT5	1.80
CPT6	13.36

Based on the soil conditions encountered at the site and our liquefaction analysis, including LPI evaluations, it is our professional opinion that the potential for liquefaction of the soils beneath the site is low to moderate if the site experiences significant ground shaking during an earthquake.

Effect of Previous Development on New Construction

The western portion of the site currently supports two structures and an interior floor slab supported on a shallow foundation system associated with previous development. Review of historical documents indicates the western portion of the site previously supported various generations of structures. We anticipate that the current and previous structures are and were supported on shallow foundation systems. With the exception of the foundations observed at the site, we assume that foundations associated with previous structures have been completely removed from the site; however, based on remnants of previous concrete foundations observed at the site, there is a possibility that additional structural concrete and other deleterious debris still remain beneath the surface soils in the western portion of the site. Recommendations for the complete removal and demolition of all existing and former surface and subgrade structures, as well as underground utilities, are provided in the Site Clearing section of this report.

Bearing Capacity and Structural Support

We anticipate the surface and near-surface soils in the western portion of the site will become significantly disturbed during the demolition and removal of surface and subsurface structures. Regardless, the upper 13½ to 24 feet of soil within the western portion of the site and the upper 25 to 38 feet of soil within the eastern portion of the site generally possess relatively low shear strengths. In our opinion, these soils are not considered capable of providing adequate or uniform support of the proposed structures without experiencing significant total and/or differential settlements, which can potentially result in structural damage. The deeper dense sand and gravel mixtures underlying the soils referenced above are capable of supporting the anticipated structural loads associated with the proposed structures. Therefore, shallow foundations supported on an improved subgrade or a deep foundation system extending into competent soils will be necessary to support the proposed structures.

The loose, soft and disturbed near-surface soils across the site are not capable of direct support of concrete slabs and pavements. Site clearing activities will help identify remnants of former structures in the western portion of the site and facilitate their removal. Complete removal of exposed remnants and proper backfilling of the excavations with engineered fill within the western portion of the site will be important to provide uniform support for concrete slabs and pavements. Other areas of the site will require scarification, moisture conditioning and compaction of the subgrade soils as engineered fill after site clearing activities are completed to provide uniform support for concrete slabs and pavements.

Foundation Alternatives

Based on the proposed construction and the subsurface soil and groundwater conditions revealed by the subsurface exploration, in our opinion the two most feasible alternatives to support the proposed structures are: a shallow foundation system (e.g. continuous and/or isolated spread footings or a mat foundation) supported on an improved subgrade consisting of Geopier® rammed aggregate piers [RAPs], vibratory Impact® piers (or similar system) or a deep foundation system consisting of auger cast-in-place (ACIP) piles. We have provided recommendations for these foundation systems in this report. These foundation systems will increase support capacity of the near-surface soils and reduce total and/or differential settlement that are considered critical to the performance of the structures, including seismic induced settlement associated with the effect of potential liquefaction.

Driven piles and cast-in-place piers (drilled piers) were considered as foundation systems to support the proposed structures. However, due to the noise, vibrations, close proximity to existing development, and cost the driven piles were not considered practical. Due to existing subsurface soil and groundwater conditions at the site, in our opinion, drilled piers are also not considered practical for this project as the piers would be required to be fully cased for construction and would have a much lower allowable bearing capacity than ACIP foundation elements. Upon request, we can provide recommendations for these alternative foundation systems if desired.

Expansive Soils

Laboratory testing of clay soils collected from the eastern portion of the site revealed the near-surface soils possess high to very high plasticity when tested in accordance with the American Society of Testing and Materials (ASTM) D 4318 (see Figure A2). Clay soils with high to very high plasticity typically also possess a significant degree of expansion potential. Laboratory test results of near-surface soils collected from the eastern portion of the site revealed the near-surface clay soils possess a “medium” to “high” expansion potential when tested in accordance with D 4829 test method (see Figures A4 and A5). Based on these test results, the near-surface soils in the eastern portion of the site are considered capable of exerting significant expansion pressures on foundations, slabs and flatwork.

Specific recommendations for subgrade preparation, foundations, concrete slabs and flatwork construction are presented in this report to mitigate the effect of expansive clay on the planned improvements.

Groundwater Effect on Development

Groundwater was observed in all of our borings performed on February 17 and 18, 2015, at depths ranging from five (5) to 16 feet below existing site grades. However, the borings may not have been left open long enough to allow groundwater to reach full equilibrium. Review of available groundwater data revealed groundwater depths at portions of the site likely fluctuated from about one (1) to four (4) feet below existing site grades (approximate elevations of +11 to +8 feet msl) from June 28, 2010 to at least March 17, 2010. Groundwater levels at the site should be expected to fluctuate throughout the year based on variations in seasonal precipitation, time of year, water levels of the adjacent Petaluma River, and other factors.

Based on explorations performed at the site and available groundwater data, we anticipate excavations as shallow as one (1) foot below existing site grades may encounter groundwater and require dewatering (depending on the time of year). For design purposes, groundwater should be anticipated at an elevation of +11 feet msl. If groundwater is encountered, the use of sumps, submersible pumps, deep wells or a well point system could be used as methods to lower the groundwater level. The dewatering method used will depend on the soil conditions, depth of the excavation and amount of groundwater present within the excavation. Dewatering, if required, should be the contractor's responsibility. The dewatering system should be designed and constructed by a dewatering contractor with local experience. We recommend the selected dewatering system lower the groundwater level to at least two (2) feet below the bottom of the proposed excavations.

Geopier® RAPs, vibratory Impact® piers or ACIP piles used for foundation support will extend into groundwater. Therefore, the RAP, vibratory pier or ACIP pile contractor should provide proper equipment and materials to handle the anticipated groundwater depths.

Seasonal Moisture

During the wet season, infiltrating surface runoff water will create a saturated surface condition due to the relatively low permeability of the near-surface soils. It is probable that grading operations attempted following the onset of winter rains and prior to prolonged drying periods will be hampered by high soil moisture contents. Also, soils exposed beneath existing slabs and pavements designated for removal may be at elevated moisture contents regardless of the time year constructed. Such soil, intended for use as engineered fill, will require a prolonged period of dry weather and/or considerable aeration to reach a moisture content that allows achieving the required compaction. This should be considered in the construction schedule for the project.

Excavation Conditions

The surface and near-surface soils at the site should be readily excavatable with conventional earthmoving and trenching equipment. Subsurface remnants associated with previous development of the site maybe encountered and can be slow to excavate with a standard, rubber-tired backhoe; however, experience has shown that excavators can remove these materials with moderate effort.

Based on explorations performed at the site and available groundwater data, we anticipate excavations as shallow as one (1) foot below existing site grades may encounter groundwater (depending on the time of year). Therefore, excavations associated with shallow building foundations, shallow trenches for utilities, and other excavations greater than one (1) foot deep associated with the proposed construction likely will require dewatering. Please refer to the Groundwater Effect on Development section in this report for our dewatering conclusions and recommendations.

After excavations have been properly dewatered, the soils within the excavation sidewalls will remain in a saturated condition and potentially create unstable conditions that can result in caving or sloughing. The presence of cohesionless or disturbed soils may also create unstable conditions that can also result in caving or sloughing. If any of these conditions exist, the contractor should be prepared to brace or shore these shallow excavations as needed. Bracing or shoring of excavations, if necessary, should conform to current Occupational Safety and Health Administration (OSHA) requirements.

Excavations or trenches exceeding five (5) feet in depth that will be entered by workers should be sloped, braced or shored to conform to current OSHA requirements. The contractor must provide an adequately constructed and braced shoring system in accordance with federal, state and local safety regulations for individuals working in an excavation that may expose them to the danger of moving ground.

Temporarily sloped excavations less than 20 feet in height, if any, should be constructed no steeper than a one-and-a-half horizontal to one vertical (1½H:1V) inclination. Temporary slopes likely will stand at this inclination for the short-term duration of construction, provided significant pockets of loose and/or saturated granular soils are not encountered. Flatter slopes would be required if these conditions are encountered.

Excavated materials should not be stockpiled directly adjacent to an open excavation to prevent surcharge loading of the excavation sidewalls. Excessive truck and equipment traffic should be avoided near excavations. If material is stored or heavy equipment is stationed and/or operated near an excavation, a shoring system must be designed to resist the additional pressure due to the superimposed loads.

On-site Soil Suitability for Fill

The existing on-site soils, including the fill soils, are considered suitable for use as engineered fill provided that they do not contain significant quantities of organics, rubble and deleterious debris, and are at a proper moisture content to achieve the desired degree of compaction.

The clay soils present beneath the site are not suitable for direct support of concrete slabs or exterior slab-on-grade concrete. Specific recommendations for subgrade preparation have been presented in this report to mitigate the effect of expansive clay on the planned structures and concrete slabs.

Soils beneath existing concrete slabs and pavements will likely be at an elevated moisture content regardless of the time of year of construction and may require drying before compaction or use as fill.

Concrete slabs, other concrete structures and pavements designated for removal are also considered suitable for use as engineered fill, provided they are broken up or pulverized to fragments less than three (3) inches in largest dimension, mixed with soil to form a compactable mixture, and are approved by the Owner.

<u>Pavement Subgrade Quality</u>

Laboratory test results performed on a representative bulk sample of near-surface clay soils from the eastern portion of the site revealed the near-surface clays are poor quality materials for support of asphalt concrete pavements, and will require thicker pavement sections to compensate for the poor quality pavement support characteristics. Laboratory test results revealed the clays possess a Resistance ("R") value of 5 when tested in accordance with California Test 301 (see Figure A6). Variable surface and near-surface soil conditions consisting of sandy and clayey materials were encountered in the western portion of the site. Some of these materials may possess a higher quality of pavement support characteristics than those described above; however, due to the variability of the soil conditions in our opinion an R-value of 5 is appropriate for design of all pavements at the site supported on untreated subgrades.

Our experience in the vicinity of the site suggests that lime treatment of the clay soils can result in a substantial improvement to the support characteristics of the clays, and reduce the thickness of the required aggregate base materials. The performance of chemically stabilized soils is dependent on uniform mixing of the quicklime into the subgrade soils, and providing a proper curing period following compaction. An experienced soil stabilization contractor, combined with a comprehensive quality control program, is essential to achieve the best results with lime stabilized soils. Near-surface clay soils from the eastern portion of the site were mixed with five percent (5%) dolomitic quicklime and subjected to an R-value test. Laboratory test results revealed the treated clays possess an R-value of 79 when tested in accordance with California Test 301 (see Figure A6). Based on Chapter 610 of the *Caltrans Highway Design Manual*, dated May 7, 2012, a maximum R-value of 40 should be used for design of pavements to be supported on a treated subgrade. Therefore, an R-value of 40 is appropriate for design pavements at the site supported on treated near-surface clay soils.

Please note surface and near-surface sandy soils encountered in the western portion of the site, mixed with lime as described above, may not react as intended to improve the pavement support characteristics of the sandy soils. Therefore, lime treatment of pavement subgrades is only applicable to subgrades consisting of clayey soils. If it is desired to improve the support quality of sandy soil to an R-value of 40, the sandy soils will require blending with clayey soils before amendment with lime will be effective. Other methods to improve the support quality of sandy soil, such as cement-treatment, can be evaluated during construction.

Soil Corrosion Potential

Three soil samples were tested to determine resistivity, pH, chloride, and sulfate concentrations to help evaluate the potential for corrosive attack upon reinforced concrete and buried metal. The results of the corrosivity testing are summarized in Table 4. Copies of corrosion potential test results performed by Sunland Analytical are presented on Figures A7 through A9.

TABLE 4 – CORROSION TEST RESULTS

Sample Location	Depth (feet)	Soil Type	pH	Chloride Content (ppm)	Sulfate Content (ppm)	Resistivity (Ω-cm)
D5	0 to 5	CH	6.93	37.1	16.5	860
D5	9 to 9½	CH	8.55	135.7	113.2	590
D6	19 to 19½	CH	8.45	482.3	406.7	510

Notes:
Ω-cm = Ohm-centimeters
ppm = parts per million

The California Department of Transportation, Corrosion and Structural Concrete Field Investigation Branch, Corrosion Guidelines, considers a site to be corrosive to foundation elements, including deep foundations, if one or more of the following conditions exists for the representative soil and/or water samples taken: has a chloride concentration greater than or equal to 500 ppm, sulfate concentration greater than or equal to 2000 ppm, or the pH is 5.5 or less. Based on this criterion, the on-site soils tested are not considered unusually corrosive to steel reinforcement properly embedded within Portland cement concrete (PCC). However, the relatively low resistivity test results of the samples tested indicates the on-site soils may be moderately to highly corrosive to unprotected metal in contact with surface and near-surface soils at the site. The chloride content of the sample tested from Boring D6 is approaching the threshold value for corrosive soil and/or water. Based on the depth of the sample from Boring D6, in our opinion, the relatively high chloride content may be related to brackish water associated with the Petaluma River.

Table 4.2.1 – Exposure Categories and Classes, American Concrete Institute (ACI) 318, Section 4.2, as referenced in Section 1904.1 of the 2013 CBC, indicates the severity of sulfate exposure for the sample tested is *Not Applicable*. Ordinary Type I-II Portland cement is considered suitable for use on this project, assuming a minimum concrete cover is maintained over the reinforcement.

Wallace-Kuhl & Associates are not corrosion engineers. Therefore, if it is desired to further define the soil corrosion potential at the site, a corrosion engineer should be consulted.

RECOMMENDATIONS

General

The recommendations presented below are appropriate for typical construction in the late spring through fall months. The on-site soils likely will be saturated by rainfall in the winter and early spring months, and will not be compactable without drying by aeration or chemical treatment. Should the construction schedule require work to begin during the wet months, additional recommendations can be provided, as conditions dictate.

Based on explorations performed at the site and available groundwater data, we anticipate excavations as shallow as one (1) foot below existing site grades may encounter groundwater and require dewatering (depending on the time of year). For design purposes, groundwater should be anticipated at an elevation of +11 feet msl.

Site preparation should be accomplished in accordance with the provisions of this report and the appended guide specifications. A representative of the Geotechnical Engineer should be present during all earthwork operations to evaluate compliance with our recommendations and the guide specifications included in this report. The Geotechnical Engineer of Record referenced herein should be considered the Geotechnical Engineer that is retained to provide geotechnical engineering observation and testing services during construction.

Building Demolition and Site Clearing

Prior to grading, existing buildings at the site designated for removal should be demolished and construction areas should be cleared of other existing surface and sub-subsurface structures associated with previous site development to expose firm and stable soils, as determined by the Geotechnical Engineer's representative. The area of removal should extend at least five (5) feet beyond all exterior foundations, where practical. Demolition debris should be removed from the site, or used as engineered fill, provided it is processed per the recommendations in this section. Any existing underground utilities designated to be removed or relocated should include all trench backfill and be replaced with engineered fill. On-site wells or septic systems/tanks associated with previous development, if any, should be properly abandoned in accordance with Sonoma County Department Health Services requirements.

Existing surface vegetation and organically laden soil within construction areas should be removed by stripping. Strippings maybe stockpiled for later use in landscaped areas or disposed of offsite. Debris from the strippings should not be used in general fill construction areas supporting structures, concrete foundation slabs, exterior flatwork or pavements. With the prior approval of our office, strippings may be used in landscape areas, provided they are kept at least five (5) feet from concrete foundation slabs and other surface improvements, moisture conditioned, and compacted. Discing of the organics into the surface soils may be a suitable alternate to stripping, depending on the condition and quantity of the organics at the time of grading. The decision to utilize discing in lieu of stripping should be made by the Geotechnical Engineer, or his representative, at the time of earthwork construction. Discing operations, if approved, should be observed by the Geotechnical Engineer, or his representative, and be continuous until the organics are adequately mixed into the surface soils to provide a compactable mixture of soil containing minor amounts of organic matter. Pockets or concentrations of organics will not be allowed.

Removal of vegetation should include rootballs and roots larger than ½-inch in diameter associated with larger weeds/brush. Adequate removal of debris and roots may require laborers and handpicking to clear the subgrade soils to the satisfaction of the Geotechnical Engineer's on-site representative.

Depressions resulting from site clearing activities should be cleaned of loose, soft, disturbed, saturated, or organically contaminated soils, as identified by the Geotechnical Engineer's representative, and properly backfilled with engineered fill in accordance with the recommendations of this report.

<u>Subgrade Preparation</u>

Ripping and Cross-Ripping

Following site clearing activities, areas that currently and previously supported structures (including the railroad tracks and the entire area west of the tracks), should be sub-excavated to a depth of at least 12 inches. The soils exposed following the sub-excavation should be ripped and cross-ripped to a depth of at least 12 inches, thoroughly moisture conditioned to at least the optimum moisture content for granular/silty soils or at least two percent (2%) above the optimum moisture content for clay soils, and uniformly compacted to no less than 90 percent relative compaction. Relative compaction should be based on the maximum dry density as determined in accordance with the ASTM D 1557 Test Method. The intent of this recommendation is to expose buried structures associated with previous development within the western portion of the site (e.g. former foundations, slabs, utilities, etc.).

Sub-Excavation

We are not aware of previous development within the eastern portion of the site (entire area east of the existing railroad tracks); however, surface clay soils, considered capable of exerting significant expansion pressures on planned improvements, were encountered in our borings performed in the eastern portion of the site from the ground surface to depths ranging from 3½ to 23½ feet below existing site grades. Following site clearing activities, surface clay soils within all structural areas (i.e. concrete foundation slabs, exterior flatwork, etc.) of the eastern portion of the site should be sub-excavated to a depth of at least 18 inches below final subgrade elevation. The soils exposed following the recommended sub-excavation operations, as well as any other surfaces to receive fill, achieved by excavation or remain at grade, should be scarified to a depth of at least eight (8) inches, thoroughly moisture conditioned to at least two percent (2%) above the optimum moisture content for clay soils, and uniformly compacted to not less than 90 percent of the ASTM D 1557 maximum dry density. Please note this sub-excavation recommendation is not necessary within pavements (i.e. areas to support roads or parking areas and drive aisles).

Lime Treatment Alternative

The on-site surface clay soils encountered within the eastern portion of the site are anticipated to react well with the addition of quicklime (high-calcium or dolomitic). As an alternative to the sub-excavation recommendations provided above for surface clay soils at the site, lime-treating the surface clay soils within structural areas of the site could mitigate the effect of expansion pressures produced by the untreated clay soils on the planned improvements. If lime-treatment of the clay soils is selected, we recommend the upper 12 inches of final subgrade elevation within structural areas are mixed with lime at a minimum spread rate of at least five (5) pounds of quicklime per cubic foot of soil treated. Lime-treatment of the subgrade soils should be performed in general conformance with Section 24 of the *Caltrans Standard Specifications*, latest edition. Lime-treated soil for support of concrete foundation slabs or exterior flatwork should be moisture conditioned to at least two percent (2%) above the optimum moisture content and compacted to not less than 90 percent of the ASTM D 1557 maximum dry density. Please note surface and near-surface sandy soils encountered in the western portion of the site, mixed with lime as described above, may not react as intended. The lime treatment recommendations provided above are only applicable to clayey soil. Clean sandy soils may require blending with clayey soils before amendment with lime will be effective.

The near-surface clays encountered at the site are also poor quality materials for support of asphalt concrete pavements, and will require relatively thick pavement sections to compensate for the poor quality pavement support characteristics. Lime treatment of the clay soils can result in a substantial improvement to the support characteristics of the clays, and reduce the thickness of the required aggregate base materials for pavements. Therefore, we have also provided a lime treatment alternative for clay subgrades to support pavements. Please refer to the Pavement Design section of this report for specific recommendations regarding pavement subgrades.

General

It is possible that soils present at the bottom of required excavations will initially be too wet to properly compact and will require a period of drying and/or considerable aeration for the soils to dry to a workable moisture content. Alternative recommendations to stabilize the bottom of excavations can be provided upon request based on actual field conditions. The use of lime stabilization or use of geotextile fabrics or geogrids is typically recommended to stabilize soils during construction.

Subgrade preparation operations should extend at least five (5) feet beyond concrete foundations slabs and adjacent flatwork, where practical. Any debris exposed by the required operations described above should be removed, and the resulting excavations should be restored to grade with engineered fill compacted in accordance with the recommendations in the Engineered Fill Construction section of this report. Compaction of all soil subgrades should be performed using a heavy, self-propelled, sheepsfoot compactor capable of achieving the required compaction and must be performed in the presence of the Geotechnical Engineer's representative who will evaluate the performance of subgrade under compactive load.

We recommend construction bid documents contain a unit price (per cubic yard) for additional excavation and replacement with engineered fill.

Engineered Fill Construction

Engineered fill consisting of on-site or import materials should be placed in lifts not exceeding six (6) inches in compacted thickness, with each lift being thoroughly moisture conditioned to at least the optimum moisture content for granular/silty soils or at least two percent (2%) above the optimum moisture content for clay soils, and uniformly compacted to at least 90 percent relative compaction. Fill operations should extend at least five (5) feet beyond buildings pads and two (2) feet beyond pavement and exterior flatwork areas, where practical.

On-site soils encountered at our boring locations are considered suitable for use as engineered fill, provided they are at a workable moisture content to achieve required compaction, and do not contain rubbish, rubble, deleterious debris, and organics. However, clay soils should not be used in fills within the upper 18 inches of final subgrade for concrete foundation slabs or exterior flatwork, unless the clay soils are lime-treated as described in the Subgrade Preparation section of this report. Concrete slab and exterior flatwork final subgrade is the surface in which aggregate base or capillary break materials are placed.

Existing concrete slabs, other concrete structures and pavements designated for removal may be broken up or pulverized and reused as engineered fill, or removed from the site. If concrete/pavement rubble is to be reused as engineered fill, it should be pulverized to fragments less than three (3) inches in largest dimension, contain sufficient intermediate sized particles to form a compactable mixture, and approved by the Owner.

Soils beneath existing concrete slabs/structures and pavements will likely be at an elevated moisture content regardless of the time of year of construction and may require drying before compaction or use as fill.

Imported fill materials should be compactable, well-graded, granular soils with a Plasticity Index of 15 or less when tested in accordance with ASTM D 4318; an Expansion Index of 20 or less when tested in accordance with ASTM D 4829, and should not contain particles greater than three (3) inches in maximum dimension. In addition, we recommend that the contractor supply a certification for any imported fill materials that designates the fill materials do not contain known contaminants per Department of Toxic Substances Control's guidelines for clean fill, and have corrosion characteristics within acceptable limits. Imported soils should be approved by the Geotechnical Engineer prior to being transported to the site.

The upper 18 inches of final subgrade for the concrete foundation slabs and exterior flatwork should consist of imported compactable, non-expansive (Expansion Index < 20) granular soils, or, 12 inches of lime-treated soils as described in the Subgrade Preparation section of this report. All soils supporting slab-on-grade concrete should be uniformly compacted to at least 90 percent of the maximum dry density as determined by ASTM D 1557.

The upper six (6) inches of untreated pavement subgrade soils and upper 12 inches of lime-treated pavement subgrade soils should be compacted to at least 95 percent relative compaction at no less than two percent (2%) above the optimum moisture content. Subgrades for support of concrete foundation slabs, exterior flatwork, and pavements should be protected from disturbance or desiccation until covered by capillary break material or aggregate base. Disturbed subgrade soils may require additional moisture conditioning, scarification and recompaction, depending on the level of disturbance.

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

Excavations near existing improvements should not encroach on the zone within a one horizontal to one vertical (1:1) plane extending down and away from foundations, slabs or pavements. Shoring or underpinning existing improvements may be required where excavations may undermine the improvements or structures.

All earthwork operations should be accomplished in accordance with the recommendations contained within this report and the *Guide Earthwork Specifications* provided in Appendix D. We recommend the Geotechnical Engineer's representative be present on a regular basis during all earthwork operations to observe and test the engineered fill and to verify compliance with the recommendations of this report and the project plans and specifications.

Utility Trench Backfill

Utility trench backfill should be mechanically compacted as engineered fill in accordance with the following recommendations. Bedding and initial backfill around and over the pipe should conform to the pipe manufacturers recommendations and applicable sections of the governing agency standards.

We recommend that on-site soil be used as trench backfill, especially below the non-expansive or lime-treated material within the footprint of concrete foundation slabs. Utility trench backfill should be placed in maximum eight-inch thick lifts (compacted thickness), thoroughly moisture conditioned to at least the optimum moisture content for granular/silty soils or at least two percent (2%) above the optimum moisture content for clay soils, and compacted to at least 90 percent of the maximum dry density as determined by ASTM D 1557. Within the upper 18 inches of final subgrade for the concrete foundation slabs and exterior flatwork, trench backfill should consist of select granular material as described in the Engineered Fill Construction section of this report; or, 12 inches of lime-treated soils as described in the Subgrade Preparation section of this report. Within the upper six (6) inches of untreated pavement subgrade soils and upper 12 inches of lime-treated pavement subgrade soils, compaction should be increased to at least 95 percent relative compaction at no less than two percent (2%) above the optimum moisture content.

It is likely that materials excavated from trenches will be at elevated moisture contents and will require significant aeration or a period of drying to reach a compactable moisture content. We recommend bid documents contain a unit price for the removal and drying of saturated soils, or replacement with approved import soils.

We recommend that all underground utility trenches aligned nearly parallel with existing or new foundations be at least three (3) feet from the outer edge of foundations, wherever possible. As a general rule, trenches should not encroach into the zone extending outward at a one horizontal to one vertical (1H:1V) inclination below the bottom of foundations. Additionally, trenches parallel to existing foundations should not remain open longer than 72 hours. The intent of these recommendations is to prevent loss of both lateral and vertical support of foundations, resulting in possible settlement.

Foundation Design

The proposed structures may be supported a shallow foundation system (e.g. continuous and/or isolated spread footings or a mat foundation) supported on an improved subgrade consisting of Geopier® rammed aggregate piers [RAPs], vibratory Impact® piers (or similar system) or a deep foundation system consisting of auger cast-in-place (ACIP) piles. Preliminary recommendations for shallow foundations supported on an improved subgrade consisting of RAPs and/or vibratory piers and recommendations for ACIP piles are provided below. Alternative foundations may be considered at the site and can be evaluated on a case-by-case basis upon request.

Shallow Foundations on Geopier® Rammed Aggregate Piers/Vibratory Impact® Piers

We anticipate continuous and/or isolated spread foundations, or a mat foundation, supported on a Geopier® RAPs and/or vibratory Impact® piers (or similar system), extending through the upper 24 to 38 feet at the west and east sides of the site, respectively, and bearing directly on the competent, silty sand and gravel soils are considered capable of densifying the subsurface soils and provide adequate support for the proposed structures. This will result in an increase in the allowable bearing capacity, reduction of post-construction foundation settlement, and mitigation of some of the effects of liquefaction induced settlement. A qualified Geopier® RAP/vibratory Impact® pier contractor licensed in the State of California should be contacted directly to provide final recommendations for the Geopier® RAP*/vibratory Impact® pier* system, including allowable capacities and post-construction settlements.

Continuous and/or isolated spread foundations or a mat foundation bearing on a Geopier® RAP/vibratory Impact® pier improved subgrade should extend at least 18 inches below the lowest adjacent soil grade, provided the subgrade has been prepared in accordance with the Subgrade Preparation and Engineered Fill Construction sections of this report. Lowest soil grade is defined as either the adjacent exterior soil grade or the soil subgrade beneath the structure, whichever is lower. Continuous foundations should maintain a minimum width of 12 inches and isolated spread foundations should be at least 24 inches in plan dimension. We understand the lower levels of the proposed structures will be used as a parking garage; therefore, if mat foundations are used to support the proposed structures, areas to support vehicle traffic should be designed in accordance with the Pavement Design section of this report.

Preliminary design information indicates the allowable bearing capacity of conventional shallow foundations constructed over rammed aggregate piers and/or vibratory piers would be on the order of 3000 to 6000 pounds per square foot (psf) for dead plus live load condition, assuming properly installed Geopier® RAPs/ vibratory Impact® piers. The RAP/vibratory pier layout and final bearing pressures and cell capacities will depend on the actual loading conditions for each structure and should be determined by the RAP/vibratory pier designer and should include an appropriate factor of safety. The weight of foundation concrete extending below adjacent soil grade may be disregarded in sizing computations. We recommend that all foundations be adequately reinforced to provide structural continuity, mitigate cracking and permit spanning of local soil irregularities. The project structural engineer should determine final foundation reinforcement.

Preliminary resistance to lateral foundation displacement for conventional foundations supported on RAPs/vibratory piers may be computed using an allowable friction factor of 0.45, which may be multiplied by the effective vertical load on each foundation. Additional lateral resistance may be computed using an allowable passive earth pressure of 350 psf per foot of depth, acting against vertical projections of the foundations. These two modes of resistance should not be added unless the frictional value is reduced by 50 percent since full mobilization of these resistances typically occurs at different degrees of horizontal movement, effectively reducing the frictional resistance.

Auger Cast-in-Place (ACIP) Concrete Piles

The proposed structures may also be supported upon ACIP piles. ACIP elements are constructed by using a specially designed drill that displaces soil rather than returning it to the surface. The shaft formed in the soil is filled with pressurized grout as the drill is withdrawn causing further densification of the surrounding soil. Reinforcement is placed into the wet grout immediately. We anticipate total settlements on the order of one- (1) inch and differential settlements on the order of ½-inch for ACIP pile foundations.

ACIP piles for the structures should extend to a minimum of approximately five (5) feet into competent material consisting of relatively dense sands and/or gravels, which were encountered at depths ranging from about 20 to 38 feet below existing site grades. Drilled ACIP piles may be designed utilizing the following maximum allowable loads per pile with appropriate factor of safety (F.S.) as summarized in Table 5.

TABLE 5 - ALLOWABLE ACIP PILE CAPACITIES

Loading Conditions		24-inch Diameter		36-inch Diameter	
		Allowable Pile Capacity (kips)	Ultimate Pile Capacity (kips)	Allowable Pile Capacity (kips)	Ultimate Pile Capacity (kips)
Axial Compression	DL (F.S. = 3)	60	200	100	320
	DL + LL (F.S. = 2)	100	200	160	320
	Total Load (F.S. = 1.5)	120	200	200	320
Axial Uplift (Tension)	Total Load (F.S. = 1.5)	30	45	50	75

Notes:
DL = Dead Load
LL = Live Load

Reductions in pile capacity for consideration of group action are unnecessary, provided piles are spaced no closer (center-to-center) than three times the diameter of the pile.

The indicated uplift pile capacity is based upon the assumption that the piles will be properly reinforced to transfer pullout forces to the pile tip.

Lateral loading information was not available at the time this report was prepared. The lateral resistance of individual piles and the passive resistance of the pile cap against the soil can be combined to provide lateral resistance. For preliminary design purposes, 24-inch ACIP piles can be assumed to provide an allowable lateral resistance of eight (8) kips and 36-inch ACIP piles can be assumed to provide an allowable lateral resistance of 14 kips. Both lateral resistance values are based on a pile deflection of ½-inch. Resistance to lateral loads for ACIP piles can be determined and presented in a supplemental report using a lateral pile analysis program when final size design information is known and if required to further aid in the structural design.

The weight of pile cap concrete extending below grade and the weight of each pile may be disregarded in determinations of the net compressive load transmitted to the supporting soil.

Concurrent lateral resistance derived in friction between the slab and the supporting subgrade layer may be computed using an allowable friction factor of 0.30 at the interface between the slab and the subgrade.

The allowable capacities for the ACIP piles are recommended with the stipulation that a pile load-testing program be performed prior to the commencement of production pile construction. A representative of the Geotechnical Engineer must be present during all pile construction activities to record and document construction of each pile.

Pile Load Testing Program

If ACIP piles are used for support of the structures, a pile loading testing program conducted prior to installation of production piles will be necessary to determine and verify the appropriate length of pile to achieve the **ultimate capacity** of the piles. The pile load test program should include both static load tests and pile driving analyzer (PDA) tests. The purpose of the PDA testing for the pre-construction piles would be to develop a correlation between the static load test results and the PDA testing that would be used during the construction of production piles in lieu of “quick” load tests. The advantage of PDA testing over the “quick” load pile testing is the savings in time to set up the load test frame that typically takes three to five (5) days, and a “quick” load test program often takes about eight (8) hours per pile to complete. All other construction activities at the site would have to be temporarily stopped during the load testing programs.

Static “Quick” Load Testing

The pile load test frame and supply of the personnel and equipment necessary to conduct the load tests should be constructed in accordance with the latest version of ASTM Test Method D 1143 for compressive loads, ASTM Test Method D 3689 for tensile loads, and ASTM Test Method D 3966 for lateral loads as delineated in the *Guide Specifications for Auger Cast Piles* provided as Appendix E.

One test pile should be cast-in-place to reach a minimum tip elevation of at least 30 feet below the existing site grades <u>and</u> at least five (5) feet into the relatively dense sand and/or gravel stratum. Additional test piles will be required if multiple pile sizes are used in the design or if alternate pile capacities are being considered. The reaction system should be capable of resisting forces from tests on the test piles in axial compression and tension as specified in Table 5. We intend to test the test pile in compression and tension, and to perform a lateral load test between adjacent piles. The pile may be loaded to failure in any of the test configurations.

Submittals for the load testing frame, hydraulic pumps, hydraulic jacks, dial indicators, and calibration documentation must be provided by the pile contractor in accordance with the project plans and specifications.

Prior to beginning load tests, the pile concrete should achieve a minimum compressive strength of 4000 pounds per square inch (psi) when tested in accordance with ASTM C 109. Construction activities must be restricted during the load-testing program. Construction activities may proceed during the setup of the load frame and installation of the test piles. Excessive vibration of the ground near the load test can cause movement of the test frame and the sensitive pile deflection measurement devices. Using the ASTM static load testing method, the compression tests will run for about eight (8) hours for each pile; the tension testing will run for about four (4) hours per pile.

Final pile construction criteria will be determined from the results of the load-testing program. It is intended that the pile load test setup will be located outside the location of any permanent pile caps or grade beams, and that the test piles and reaction piles will be abandoned upon completion of the testing.

Pile Driving Analyzer Testing

Following the static load testing program, the test pile will be subjected to PDA testing, provided the pile is not damaged during the static load testing. PDA testing involves instrumenting piles and recording the response of the pile during dynamic loading. PDA testing consists of dropping a heavy weight from a certain height on to the pile head and monitoring the response of the pile. The capacity of the piles can be computed from the analyses of the PDA test.

Additional PDA testing will be performed during construction of production piles, in the event that as-built pile dimensions differ from the recommended dimensions, which could result from refusal to auger penetration or in random areas across the site to verify that the earth materials are supporting the piles as indicated by the load test program.

Surveillance/Protection

We recommend that photographic and written records be kept of both the pre-existing condition and new damage (if any) sustained by improvements in and around the site. The elevation of sidewalks and buildings adjacent to the construction site should be measured prior to construction activities. The elevations of selected survey points should be measured on a weekly basis during the initial stages of construction. Elevation of improvements and photographs should include basic data for determining the validity of claims lodged by nearby property owners or tenants.

Interior Slab-on-Grade Support

Interior concrete slabs-on-grade can be supported upon the soil subgrade prepared in accordance with the recommendations in this report and maintained in that condition. Slabs-on-grade that will be used for vehicle support should be designed in accordance with the recommendations provided in the Pavement Design section of this report.

Interior slab-on-grade concrete slabs that will not be used for vehicle support should be at least four (4) inches thick. We recommend that interior slabs-on-grade be adequately reinforced to provide structural continuity, mitigate cracking and permit spanning of local soil irregularities. The project structural engineer should determine final reinforcement and joint spacing. Wheel loads from forklifts, storage of palletized materials, cranes, etc., anticipated during construction should be considered in the design of the slab-on-grade floors.

Conventional floor slabs may be underlain by a layer of free-draining gravel serving as a deterrent to migration of capillary moisture. If used, the gravel layer should be at least four (4) inches thick and graded such that 100 percent passes a one-inch sieve and less than five percent (5%) passes a No. 4 sieve. Additional moisture protection may be provided by placing a water vapor retarder (at least 10-mils thick) directly over the gravel. If used, the water vapor retarder should meet or exceed that standard specification as outlined in ASTM E1745.

Floor slab construction practice over the past 30 years or more has included placement of a thin layer of sand over the vapor retarder membrane. The intent of the sand is to aid in the proper curing of the slab concrete. However, recent debate over excessive moisture vapor emissions from floor slabs includes concern of water trapped within the sand. As a consequence, we consider use of the sand layer as optional. The concrete curing benefits should be weighed against efforts to reduce slab moisture vapor transmission.

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

Floor Slab Moisture Penetration Resistance

It is likely the floor slab subgrade soils will become saturated at some time during the life of the structure, especially when slabs are constructed during the wet season and when constantly wet ground or poor drainage conditions exist adjacent to structures. For this reason, it should be assumed that all interior slabs require protection against moisture or moisture vapor penetration. Standard practice includes placing a layer of gravel and a vapor retarder membrane (and possibly a layer of sand) as discussed above. Recommendations contained in this report concerning foundation and floor slab design are presented as minimum requirements only from the geotechnical engineering standpoint.

Use of gravel and a vapor retarder membrane will not "moisture proof" the slab, nor does it assure that slab moisture vapor transmission levels will be low enough to prevent damage to floor coverings or other building components. It is emphasized that we are not slab moisture proofing or moisture protection experts. The sub-slab gravel and vapor retarder membrane simply offer a first line of defense against soil-related moisture. If increased protection against moisture vapor penetration of the slab is desired, a concrete moisture protection specialist should be consulted. It is commonly accepted that maintaining the lowest practical water-cement ratio in the slab concrete is one of the most effective ways to reduce future moisture vapor penetration of the completed slab.

Retaining Wall Design

We understand the grade for the extension of Oak Street will be raised to match the existing grade of Petaluma Boulevard North, which is estimated to be about five (5) feet higher than existing site grades. We anticipate retaining walls that are fixed at the top will be constructed to retain engineered fill associated with the construction of Oak Street. Retaining wall less than five (5) feet in height and not structurally connected to proposed structures could be supported on a continuous foundation at least 12 inches wide and extending at least 18 inches below lowest adjacent soil grade. Continuous footings for retaining wall may be designed based on an allowable bearing capacity of 1,500 pounds psf for dead plus live load conditions. The allowable bearing capacity may be increased by one-third for effects of wind or seismic forces.

Backfill material behind retaining walls associated with the extension of Oak Street will likely consist of imported soils, and will be constructed in accordance with the Engineered Fill Construction and Pavement Design sections of this report. We assume the backfill behind the retaining wall will be covered by concrete flatwork or pavements. Therefore, these retaining walls should be capable of resisting "at-rest" lateral earth pressures equal to an equivalent fluid pressure of 70 psf per foot of retained soil. Retaining walls will experience additional surcharge loading from vehicles that will use Oak Street. Surcharge loading under these circumstances should be evaluated by the structural engineer.

Backfill behind retaining walls should be fully drained to prevent the build-up of hydrostatic pressure behind the wall. Retaining walls should be provided with a drainage blanket (Class 2 permeable material, Caltrans Specification Section 68-2.02F(3)) at least one- (1) foot wide extending from the base of wall to the top of the wall. Weep holes or perforated rigid pipe should be provided near the base of the wall to allow drainage of accumulated water. Drainpipes, if used, should slope to discharge at no less than a one percent (1%) fall to suitable drainage facilities. Open-graded, ½-inch to ¾-inch, crushed rock may be used in lieu of the Class 2 permeable material, if the rock and drain pipe are completely enveloped in an approved nonwoven geotextile filter fabric.

Resistance to lateral foundation displacement may be computed using an allowable friction factor of 0.35, which may be multiplied by the effective vertical load on each foundation. Additional lateral resistance may be computed using an allowable passive earth pressure equivalent to a fluid pressure of 350 psf per foot of depth, acting against vertical projections of the foundations. These two modes of resistance should not be added unless the frictional component is reduced by 50 percent since full mobilization of the passive resistance requires some horizontal movement, effectively reducing the frictional resistance.

Exterior Flatwork Construction (Non-Pavement)

Soil subgrade areas to support exterior concrete flatwork should be prepared in accordance with the Subgrade Preparation and Engineered Fill Construction recommendations included in this report. Proper moisture conditioning of the subgrade soils is considered essential to the performance of the exterior flatwork. Exterior flatwork should be constructed independent of perimeter building foundations by the placement of a layer of felt material between the flatwork and the foundation.

Exterior flatwork concrete should be at least four (4) inches thick. Consideration should be given to thickening the slabs to at least twice the slab thickness where wheel traffic is expected over the slabs. Expansion joints should be provided to allow for minor vertical movement of the flatwork. Exterior flatwork should be constructed independent of perimeter building foundations by the placement of a layer of felt material between the flatwork and the foundation. The slab designer should determine the final thickness, strength and joint spacing of exterior slab-on-grade concrete. The slab designer should also determine if slab reinforcement for crack control is required and determine final slab reinforcing requirements.

Areas adjacent to new exterior flatwork should be landscaped to maintain more uniform soil moisture conditions adjacent to and under flatwork. We recommend final landscaping plans not allow fallow ground adjacent to exterior concrete flatwork.

Practices recommended by the Portland Cement Association (PCA) for proper placement, curing, joint depth and spacing, construction, and placement of concrete should be followed during exterior concrete flatwork construction.

Site Drainage

Final site grading should be accomplished to provide positive drainage of surface water away from buildings and prevent ponding of water adjacent to foundations, slabs or pavements. The subgrade adjacent to buildings should be sloped away from foundations at a minimum two percent (2%) gradient for at least 10 feet, where possible. We recommend connecting all roof drains to solid PVC pipes which are connected to available drainage features to convey water away from the structures, or discharging the drains onto paved, or hard surfaces that slope away from the foundations. Discharging or ponding of surface water should not be allowed adjacent to buildings, exterior flatwork or pavements. Landscape berms, if planned, should not be constructed in such a manner as to promote drainage toward buildings.

Pavement Design

Laboratory test results from near-surface clay soils encountered at the site exhibit poor support qualities for support of asphalt concrete pavements. Relatively thick pavement sections would be required for pavements unless the clays are lime-treated. Based on laboratory test results, we used a Resistance ("R") value of five (5) for untreated subgrades and an R-value of 40 for clay subgrades amended with at least five percent (5%) high calcium or dolomitic quicklime for the design of pavements. Pavement sections presented in Table 6 have been calculated using traffic indices assumed to be appropriate for on-site parking areas and drive aisles associated with the mixed-use buildings. Pavement sections presented in Table 7 have been calculated using traffic indices provided in the City of Petaluma Street Design and Construction Standards and Specifications, dated May of 1999, and are applicable to the proposed extensions of Oak Street and Water Street. The procedures used for flexible pavement design are in general conformance with Chapters 600 to 670 of the California Highway Design manual, dated May of 2012. The project civil engineer should determine the appropriate traffic index based on anticipated traffic conditions. If needed, we can provide additional pavement sections for different traffic indices.

TABLE 6 – ON-SITE PAVEMENT DESIGN ALTERNATIVES

Traffic Index (TI)	Pavement Use	Untreated Subgrades R-value = 5			Lime-Treated Subgrades Soils (a) R-value = 40		
		Type A Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)	Portland Cement Concrete (inches)	Type A Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)	Portland Cement Concrete (inches)
.5	Automobile Parking Only	2½*	10	--	2½*	4	4
		--	6	4	--	4	4
5.5	Automobile and Light to Moderate Truck Traffic	2½	13	--	2½	7	--
		3*	12	--	3*	6	--
		--	6	5	--	4	5
6.5	Moderate Truck Traffic and Fire Lanes	3	16	--	3	8	--
		4*	14	--	4*	6	--
		--	8	6	--	4	6

** = Asphalt concrete thickness contains the Caltrans safety factor.*
(a) = Lime-treated subgrade should be at least 12 inches thick and possess a minimum R-value of 40 when testing in accordance with California Test 301.

TABLE 7 – OFF-SITE CITY PAVEMENT DESIGN ALTERNATIVES

Traffic Index (TI)	Pavement Use	Untreated Subgrades R-value = 5			Lime-Treated Subgrades Soils (a) R-value = 40		
		Type A Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)	Portland Cement Concrete (inches)	Type A Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)	Portland Cement Concrete (inches)
5.0	Residential Street	2½	11	--	2½	5	--
		3*	10	--	3*	4	--
		--	6	5	--	4	5
6.0	Collector Street	3	14	--	3	7	--
		3½*	13	--	3½*	6	--
		--	8	6	--	4	6

** = Asphalt concrete thickness contains the Caltrans safety factor.*
(a) = Lime-treated subgrade should be at least 12 inches thick and possess a minimum R-value of 40 when testing in accordance with California Test 301.

We emphasize that the performance of pavement is critically dependent upon uniform and adequate compaction of the soil subgrade, as well as all engineered fill and utility trench backfill within the limits of the pavements. We recommend that pavement subgrade preparation, i.e. scarification, moisture conditioning and compaction, be performed after underground utility construction is completed and just prior to aggregate base placement.

The upper six (6) inches of untreated pavement subgrade soils and upper 12 inches of lime-treated subgrade soils should be compacted to at least 95 percent relative compaction at no less than two percent (2%) above the optimum moisture content. All aggregate base should be compacted to at least 95 percent of the maximum dry density. Refer to the Subgrade Preparation and Engineered Fill Construction sections of this report for additional recommendations regarding the construction of pavement subgrades.

Regardless of the method used to construct pavement subgrades (untreated or treated); areas to support new pavements within 10 feet of the Petaluma River embankment should include the installation of geogrid reinforcement. In these areas the geogrid reinforcement should be placed between the pavement sections and the compacted subgrade soils. The purpose of this recommendation is to reduce the potential for pavement failures caused by lateral soil creep associated with the Petaluma River embankment slope.

Pavement subgrades should be stable and unyielding under heavy wheel loads of construction equipment. To help identify unstable subgrades within the pavement limits, a proof-roll should be performed with a fully-loaded water truck on the exposed subgrades prior to placement of aggregate base. The proof-roll should be observed by the Geotechnical Engineer's representative.

In the summer heat, high axle loads coupled with shear stresses induced by sharply turning tire movements can lead to failure in asphalt concrete pavements. Therefore, we recommend that consideration be given to using Portland cement concrete (PCC) pavements in areas subjected to concentrated heavy wheel loading, such as entry driveways and in front of trash enclosures. At the time this report was prepared it was unclear if portions of the Oak Street and Water Street extensions are planned to be constructed of PCC pavements. Alternate PCC pavement sections have been provided above in Tables 6 and 7. All aggregate base should be compacted to at least 95 percent relative compaction.

We suggest the concrete slabs be constructed with thickened edges in accordance with American Concrete Institute (ACI) design standards, latest edition. Reinforcing for crack control, if desired, should be provided in accordance with ACI guidelines. Reinforcement must be located at mid-slab depth to be effective. Joint spacing and details should conform to the current PCA or ACI guidelines. PCC should achieve a minimum compressive strength of 3,500 pounds per square inch at 28 days.

Please note that regardless of the degree of compaction of the soils used to construct the ramp for the raised extension of Oak Street, differential settlement between the Oak Street ramp and the existing Petaluma Boulevard North will occur. The cause will be from inevitable settlement of new engineered fill and the supporting soil. The existing embankment supporting Petaluma Boulevard North has been in place for decades, and has come to settlement equilibrium and would be relatively stable compared to the new construction. The differential settlement will likely be on the order of one to four inches (1" to 4"). After several years, it is likely that some repair of the pavement will be necessary to maintain a smooth transition between the ramp and the existing street.

All pavement materials and construction methods of structural pavement sections should conform to the applicable provisions of the *Caltrans Standard Specifications* and applicable City of Petaluma Standards, latest edition.

Efficient drainage of all surface water to avoid infiltration and saturation of the supporting aggregate base and subgrade soils is important to pavement performance. Weep holes could be provided at drainage inlets, located at the subgrade-base interface, to allow accumulated water to drain from beneath the pavements.

Lime-treatment of Pavement Subgrade Soils

The on-site clay soils are anticipated to react well with the addition of quicklime (high-calcium or dolomitic) and could enhance the support characteristics of the subgrade and allow for a reduction in the aggregate base section. Lime treatment of pavement subgrade soils for the extensions of Oak Street and Water Street would be subject to approval by the City of Petaluma. Lime treatment of subgrade soils should be performed in general conformance with Section 24 of the *Caltrans Standard Specifications*. Additional testing should be performed during construction to verify that the pavement design parameters are achieved in the field. Samples of the field-mixed soil and lime should be collected and tested for a minimum R-value of 40, when tested in accordance with California Test 301. This additional testing will either verify the design parameters, or provide the opportunity to modify the pavement sections or spread rate based upon the test results.

For estimating purposes only, we recommend a minimum spread rate of at least 5 pounds of quicklime per square foot of treated soil, at a depth sufficient to produce a compacted lime-treated layer 12 inches thick. Lime-treated subgrades should be compacted to not less than 95 percent of the ASTM D 1557 maximum dry density at a moisture content of at least two percent (2%) above the optimum moisture content.

Geotechnical Engineering Observation and Testing During Earthwork

Site preparation should be accomplished in accordance with the recommendations of this report and the Guide Earthwork Specifications provided in Appendix D. Geotechnical testing and observation during construction is considered a continuation of our geotechnical engineering investigation. Wallace-Kuhl & Associates should be retained to provide testing and observation services during site clearing, earthwork, and foundation construction at the project to verify compliance with this geotechnical report and the project plans and specifications, and to provide consultation as required during construction. These services are beyond the scope of work authorized for this investigation; however, we would be pleased to submit a proposal to provide these services upon request.

Section 1803A.5.8 Compacted Fill Material of the 2013 CBC requires that the geotechnical engineering report provide a number and frequency of field compaction tests to determine compliance with the recommended minimum compaction. Many factors can effect the number of tests that should be performed during the course of construction, such as soil type, soil moisture, season of the year and contractor operations/performance. Therefore, it is crucial that the actual number and frequency of testing be determined by the Geotechnical Engineer during construction based on their observations, site conditions, and difficulties encountered. As a preliminary guideline we recommend the following minimum tests:

- mass grading: one test per 500 cubic yards of compacted fill or one per day of work, whichever is greater
- final subgrade preparation: one test per 5,000 square feet
- aggregate base compaction: one test per 5,000 square feet
- utility backfill: one test per foot of backfill for every 150 linear feet of trench
- wall backfill: one test per foot of backfill for every 100 linear feet of wall

In the event that Wallace-Kuhl & Associates is not retained to provide geotechnical engineering observation and testing services during construction, the Geotechnical Engineer retained to provide these services should indicate in writing that they agree with the recommendations of this report, or prepare supplemental recommendations as necessary (Form DSA-109). A final report by the "Geotechnical Engineer" should be prepared upon completion of the project.

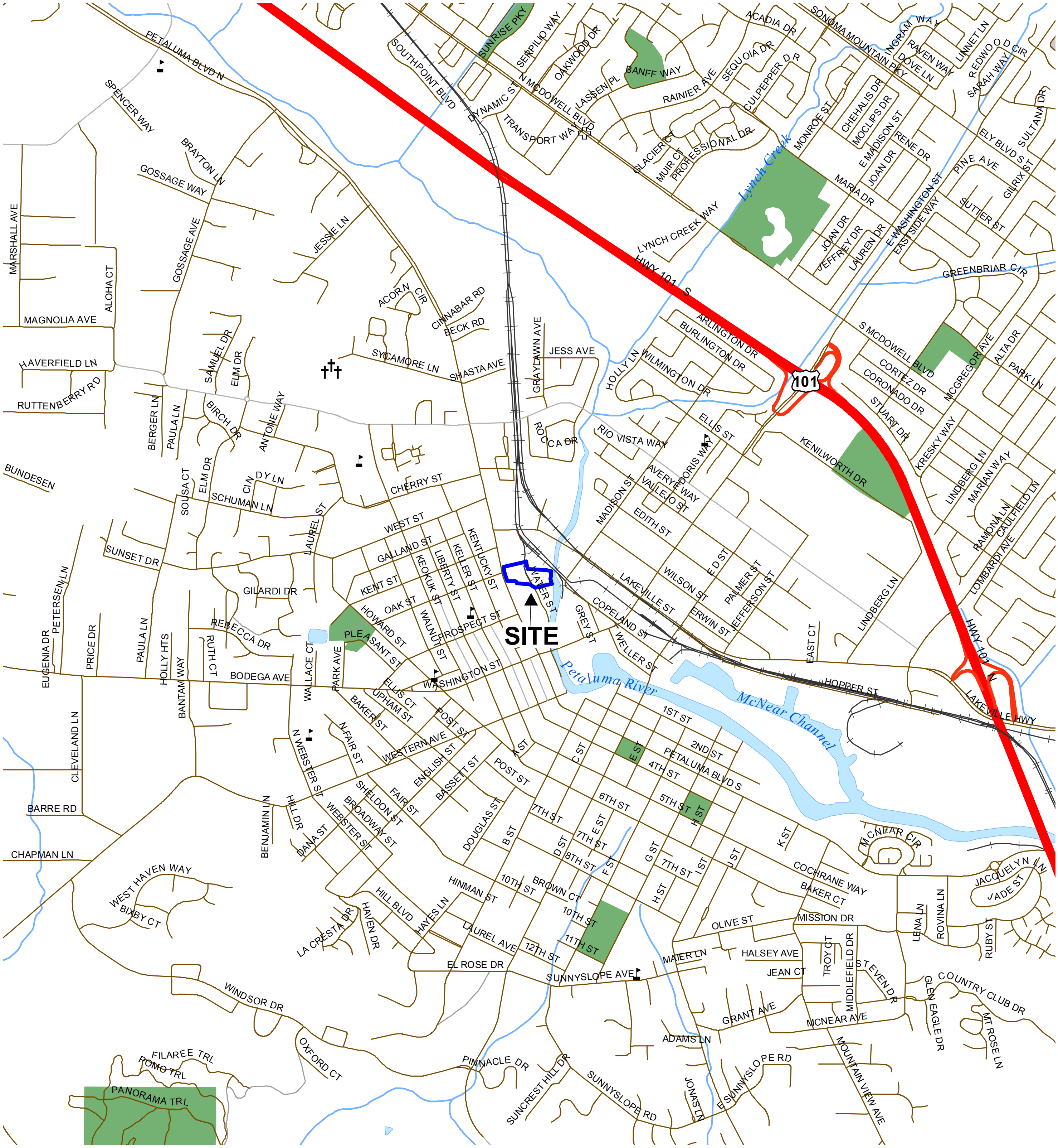
<u>Additional Services</u>

We recommend that our firm be retained to review the final plans and specifications to determine if the intent of our recommendations has been implemented in those documents. We would be pleased to submit a proposal to provide these services upon request.

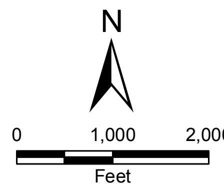
LIMITATIONS

Our recommendations are based upon the information provided regarding the proposed construction, combined with our analysis of site conditions revealed by the field exploration and laboratory testing programs. We have used prudent engineering judgment based upon the information provided and the data generated from our investigation. This report has been prepared in substantial compliance with generally accepted geotechnical engineering practices that exist in the area of the project at the time the report was prepared. No warranty, either express or implied, is provided.

If the proposed construction is modified or relocated or, if it is found during construction that subsurface conditions differ from those we encountered at our exploration locations, we should be afforded the opportunity to review the new information or changed conditions to determine if our conclusions and recommendations must be modified.



Street data courtesy of Sonoma County.
Hydrography courtesy of the U.S. Geological Survey
acquired from the GIS Data Depot, December, 2007.
Projection: NAD 83, California State Plane, Zone II



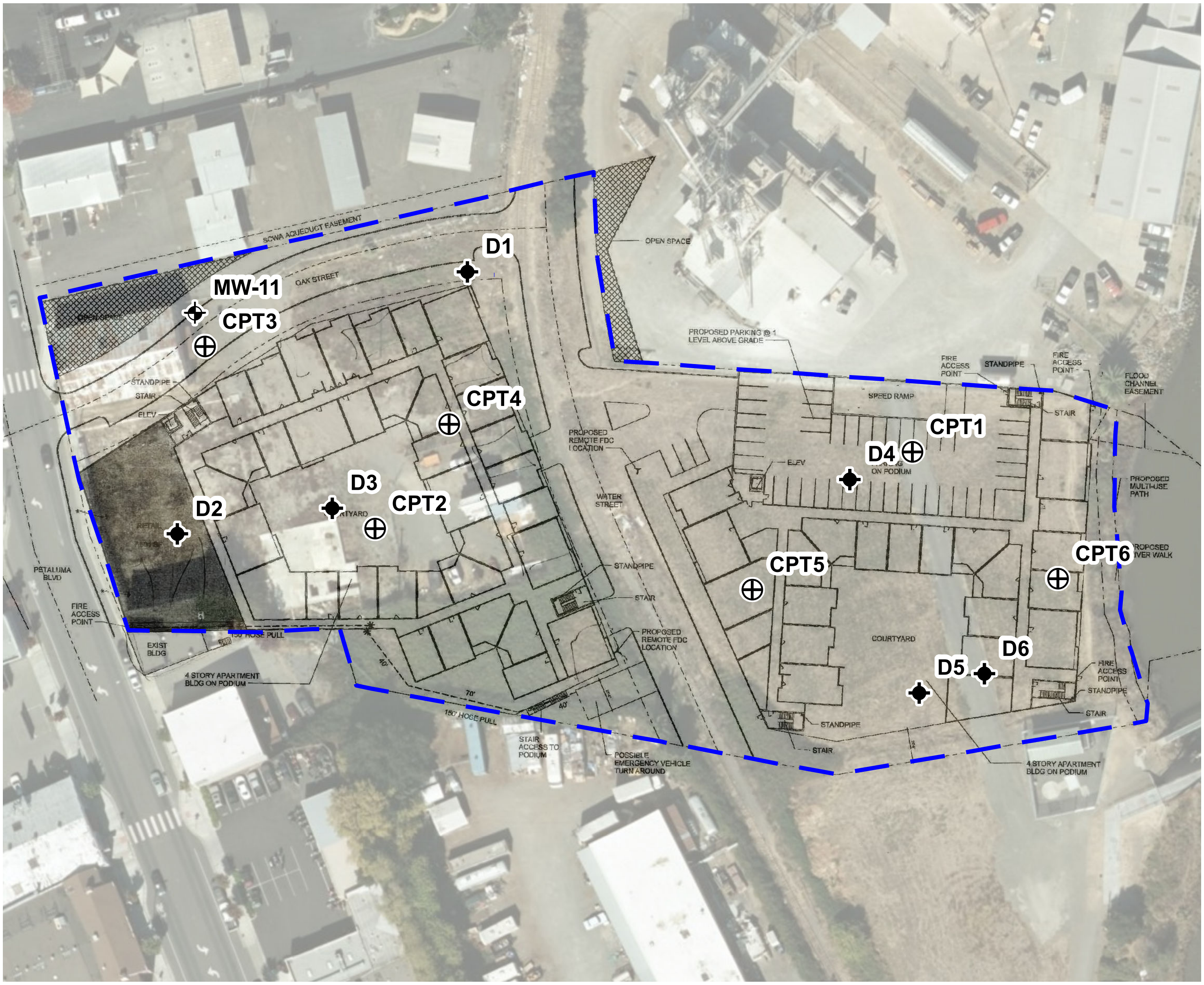
WallaceKuhl & ASSOCIATES

VICINITY MAP

368 & 402 PETALUMA BOULEVARD NORTH

Petaluma, California

FIGURE	1
DRAWN BY	RWO
CHECKED BY	ML
PROJECT MGR	DRG
DATE	06/15
WKA NO. 10410.02	



Aerial imagery courtesy of ESRI.
Site plan provided by Kephart Architects dated 4/20/15.
Projection: NAD 83, California State Plane, Zone II

Legend

- Approximate Site Boundary
- Approximate CPT Location
- Approximate Boring Location
- Approximate Location of Previous Monitoring Well (By Others)

N

0 50 100
Feet

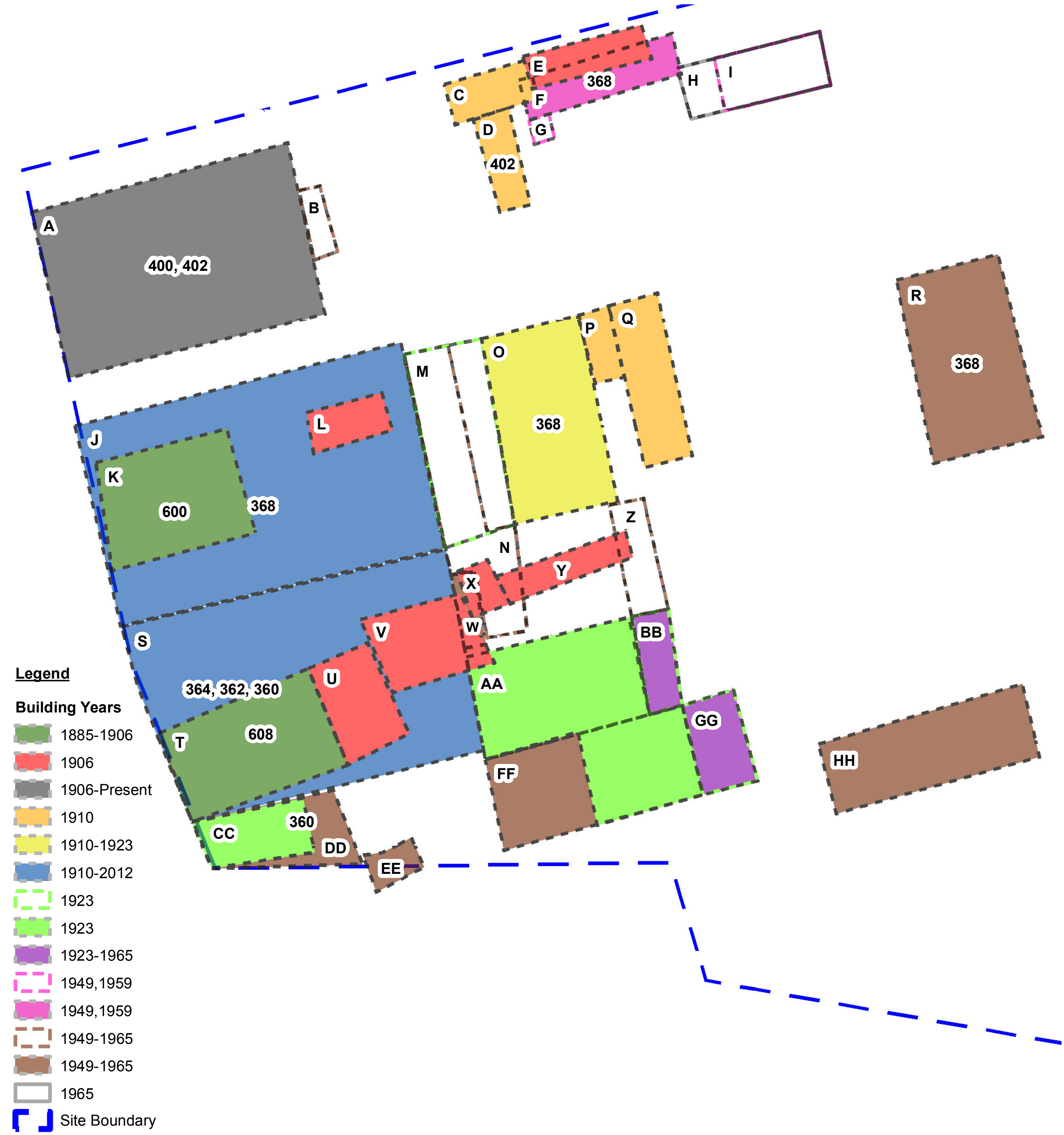
WallaceKuhl & ASSOCIATES

AERIAL SITE MAP

368 & 402 PETALUMA BOULEVARD NORTH

Petaluma, California

FIGURE	2
DRAWN BY	RWO
CHECKED BY	ML
PROJECT MGR	DRG
DATE	06/15
WKA NO. 10410.02	



Adapted from Sanborn Map Reports provided by
The Sanborn Map Library January 14, 2015.
Projection: NAD 83, California State Plane, Zone II

N

0 20 40
Feet

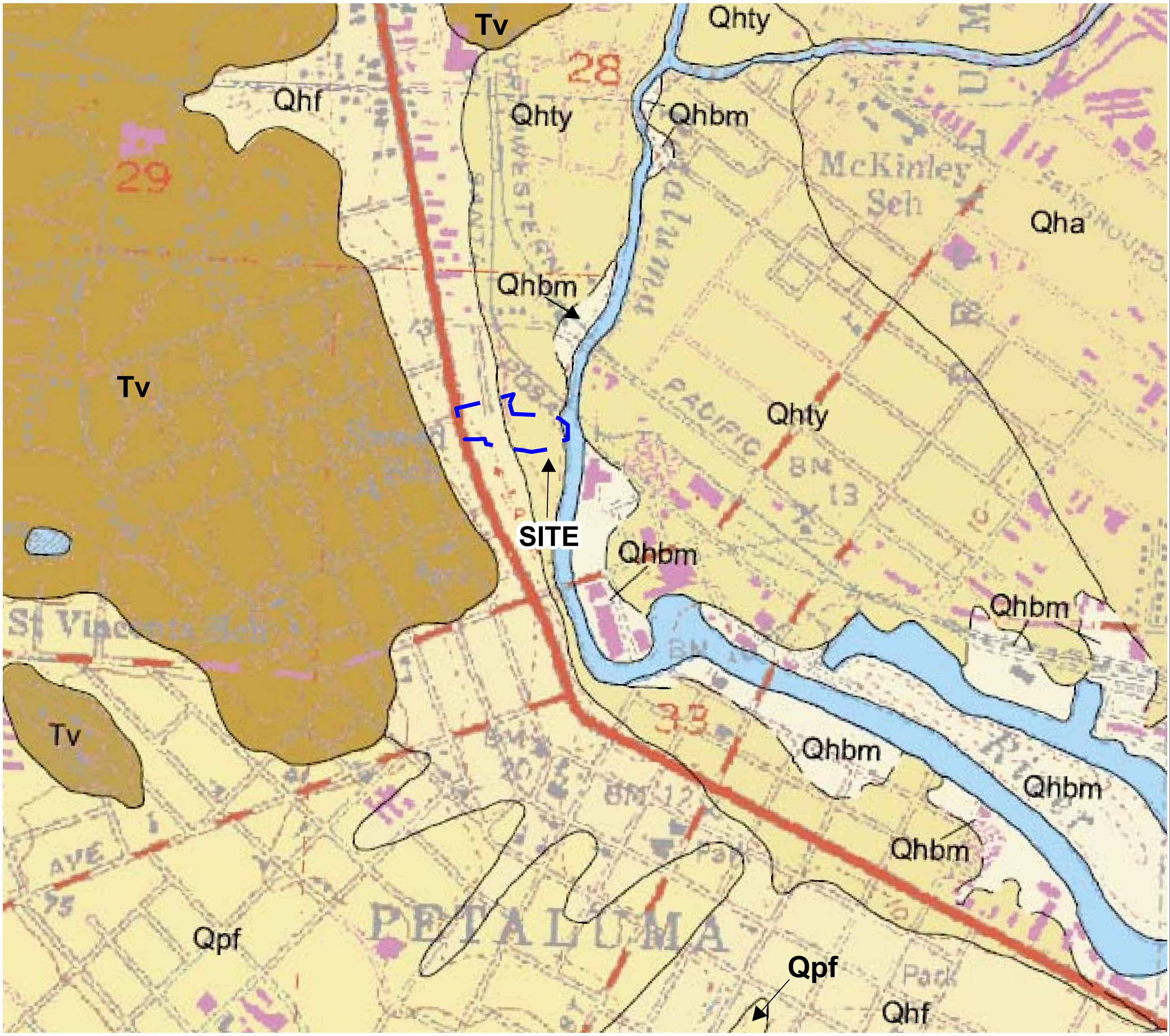
WallaceKuhl & ASSOCIATES

HISTORICAL BUILDING LOCATION MAP

368 & 402 PETALUMA BOULEVARD NORTH

Petaluma, California

FIGURE	3
DRAWN BY	RWO
CHECKED BY	ML
PROJECT MGR	DRG
DATE	06/15
WKA NO. 10410.02	



Geologic Map of the Petaluma 7.5' Quadangle
Sonoma and Marin Counties
Projection: NAD 83, California State Plane, Zone II

Legend

Site Boundary

Qhbm	Holocene estuarine deposits (bay mud)
Qhf	Holocene fan deposits
Qha	Holocene alluvium, undifferentiated
Qhty	Latest Holocene terrace deposits
Qpf	Late Pieistocene fan deposits
Tv	Volcanic rocks (Miocene)

N

0 500 1,000
Feet

WallaceKuhl & ASSOCIATES

GEOLOGIC MAP

368 & 402 PETALUMA BOULEVARD NORTH

Petaluma, California

FIGURE	4
DRAWN BY	RWO
CHECKED BY	ML
PROJECT MGR	DRG
DATE	06/15
WKA NO. 10410.02	

Project: 368 & 402 Petaluma Blvd. North **Project Location:** Petaluma, California **WKA Number:** 10410.02	**LOG OF SOIL BORING D1** Sheet 1 of 1

Date(s) Drilled	**2/18/15**	Logged By	**GJF**	Checked By	**DRG**
Drilling Method	**Hollow Stem Auger**	Drilling Contractor	**V & W Drilling, Inc.**	Total Depth of Drill Hole	**31.0 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**8"**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**7.0**	Sampling Method(s)	**Modified California**	Drill Hole Backfill	**Neat Cement**
Remarks	**Bulk Sample D1 (0' to 5')**			Driving Method and Drop	**140 lb hammer, 30 inch drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
	0		Dark brown, moist, medium dense, silty SAND (SM - Fill); fine to medium sand; with pieces of wood and asphalt.						
					D1-1I	15			PID = 0ppm
			Brown, moist, medium dense, silty SAND (SM - Fill); fine to coarse sand.						
					D1-2I	8	18.6	99	PID = 0ppm
	5		Dark brown, moist, medium stiff, sandy CLAY (CL - Fill); with pieces of brick.						
			Gray-brown, moist, medium stiff, silty CLAY (CH)		D1-3I	7			PID = 0ppm PI
	10				D1-4I	17	32.8	86	PID = 0ppm
			brown, very stiff						
	15		Brown, moist, medium stiff, silty CLAY with sand (CH).		D1-5I	8	26.1	93	PID = 0ppm UCC = 1.2 tsf
	20				D1-6I	21	24.5	99	PID = 0ppm
			Gray, wet, medium dense, silty SAND (SM); fine sand.						
	25		very dense, fine to coarse sand, trace of fine gravel, variably cemented		D1-7I	74/11"			PID = 0ppm
	30		brown		D1-8I	79/11"	13.8	119	PID = 0ppm
			Boring terminated at 31' below existing site grade.						

BORING LOG 10410.02 - 368 & 402 PETALUMA BLVD. NORTH.GPJ WKA.GDT 3/26/15 2:17 PM

Wallace Kuhl & ASSOCIATES

FIGURE 5

Project: 368 & 402 Petaluma Blvd. North
Project Location: Petaluma, California
WKA Number: 10410.02

LOG OF SOIL BORING D2

Sheet 1 of 1

Date(s) Drilled	2/18/15	Logged By	GJF	Checked By	DRG
Drilling Method	Hollow Stem Auger	Drilling Contractor	V & W Drilling, Inc.	Total Depth of Drill Hole	25.0 feet
Drill Rig Type	CME 75	Diameter(s) of Hole, inches	8"	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	16.0	Sampling Method(s)	Modified California	Drill Hole Backfill	Neat Cement
Remarks	Bulk Sample D2 (0' to 3.5')			Driving Method and Drop	140 lb hammer, 30 inch drop

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Brown, moist, dense, silty SAND (SM); fine to medium sand.						
					D2-1I	52			PID = 0ppm
			very dense		D2-2I	63	14.7	111	PID = 0ppm
	5								
			fine to coarse, variably cemented		D2-3I	89/9"			PID = 0ppm
	10								
					D2-4I	82	15.4	108	PID = 0ppm
	15								
					D2-5I	85/11"			PID = 0ppm
	20								
			wet		D2-6I	78	13.7	116	PID = 0ppm
	25								
			Boring terminated at 25' below existing site grade.						

BORING LOG 10410.02 - 368 & 402 PETALUMA BLVD. NORTH.GPJ WKA.GDT 3/26/15 2:17 PM

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FIGURE 6

Project: 368 & 402 Petaluma Blvd. North
Project Location: Petaluma, California
WKA Number: 10410.02

LOG OF SOIL BORING D3

Date(s) Drilled	2/18/15	Logged By	GJF	Checked By	DRG
Drilling Method	Hollow Stem Auger	Drilling Contractor	V & W Drilling, Inc.	Total Depth of Drill Hole	30.0 feet
Drill Rig Type	CME 75	Diameter(s) of Hole, inches	8"	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	5.0	Sampling Method(s)	Modified California	Drill Hole Backfill	Neat Cement
Remarks	Bulk Sample D3 (0' to 3.5')			Driving Method and Drop	140 lb hammer, 30 inch drop

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Light brown and brown, moist, medium dense, sandy SILT to silty SAND (SM/ML - Fill); trace of clay and fine gravel.		D3-1I	19	17.9	102	PID = 0ppm
			Brown, moist, stiff, sandy CLAY (CH); fine to medium sand.						
	5		Dark brown, moist, loose, silty SAND (SM); fine to medium sand.		D3-2I	10			PID = 0ppm
			Dark brown, wet, medium stiff, silty CLAY (CH); trace of fine to coarse sand and fine gravel.						
	10				D3-3I	7	19.7	107	PID = 0ppm
	15		Brown, wet, very dense, clayey SAND with gravel (SC); fine to coarse sand; fine gravel; variably cemented.		D3-4I	73			PID = 0ppm GR 14% fines
	20		Brown, wet, very dense, silty SAND (SM); fine to medium sand; variably cemented.		D3-5I	69	15.5	110	PID = 0ppm
	25				D3-6I	63			PID = 0ppm
	30				D3-7I	62	20.2	106	PID = 0ppm
			Boring terminated at 30' below existing site grade.						

Wallace Kuhl & ASSOCIATES

FIGURE 7

Project: 368 & 402 Petaluma Blvd. North
Project Location: Petaluma, California
WKA Number: 10410.02

LOG OF SOIL BORING D4

Date(s) Drilled	2/17/15	Logged By	GJF	Checked By	DRG
Drilling Method	Hollow Stem Auger	Drilling Contractor	V & W Drilling, Inc.	Total Depth of Drill Hole	31.0 feet
Drill Rig Type	CME 75	Diameter(s) of Hole, inches	8"	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	11.0	Sampling Method(s)	Modified California	Drill Hole Backfill	Neat Cement
Remarks	Bulk Sample D4 (0' to 5')			Driving Method and Drop	140 lb hammer, 30 inch drop

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: SAMPLE NUMBER	SAMPLE DATA: NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
			Brown, moist, medium stiff, sandy CLAY (CH - Fill); fine sand; with roots.		D4-1I	6	32.8	87	PID = 0ppm
	5		Brown, moist, loose, silty SAND (SM - Fill); fine sand; trace of clay.		D4-2I	11	31.9	88	PID = 0ppm
			Gray, moist, stiff, silty CLAY (CH - Fill).						
	10		Brown to gray, moist, very loose, silty SAND (SM - Fill); fine sand; trace of organics.		D4-3I	3			PID = 0ppm OC = 3.7%
	15		Gray, moist, soft, silty CLAY (CH - Fill).		D4-4I	4	47.3	72	PID = 0ppm PI UCC = 0.3 tsf
	20		Gray, wet, very soft, sandy CLAY (CH - Fill); trace of organics.		D4-5I	2			PID = 0ppm OC = 3.1%
			Gray, wet, very loose, silty SAND (SM - Fill); fine sand; trace of clay; trace of organics; with a piece of wood.						
	25		Gray, wet, medium dense, silty GRAVEL with sand (GM); fine to coarse sand; fine gravel.		D4-6I	15	18.2	107	PID = 0ppm
	30		Brown, wet, very dense, silty SAND (SM); fine to coarse sand; variably cemented.		D4-7I	71/11"			PID = 0ppm

Boring terminated at 31' below existing site grade.

Wallace Kuhl & ASSOCIATES

FIGURE 8

Project: 368 & 402 Petaluma Blvd. North Project Location: Petaluma, California WKA Number: 10410.02	**LOG OF SOIL BORING D5** Sheet 1 of 1

Date(s) Drilled	**2/17/15**	Logged By	**GJF**	Checked By	**DRG**
Drilling Method	**Hollow Stem Auger**	Drilling Contractor	**V & W Drilling, Inc.**	Total Depth of Drill Hole	**29.5 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**8"**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**12.0**	Sampling Method(s)	**Modified California**	Drill Hole Backfill	**Neat Cement**
Remarks	**Bulk Sample D5 (0' to 5')**			Driving Method and Drop	**140 lb hammer, 30 inch drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Brown, moist, medium stiff, silty CLAY (CH - Fill).		D5-1I	6			PID = 0ppm PI
					D5-2I	15	26.6	85	PID = 0ppm
	5		Brown, moist, stiff, silty CLAY (CH - Fill); fine sand; trace of organics; pockets of silty sand.						
	10				D5-3I	9	25.4	95	PID = 0ppm UCC = 0.5 tsf
	15		gray with black mottling, medium stiff		D5-4I	7			PID = 0ppm OC = 3.7% PI UCC =0.6 tsf
	20		wet, with pockets of silty sand, trace of organics		D5-5I	1			PID = 0ppm PI OC =7.1%
	25		Gray, wet, loose, silty SAND (SM); fine to medium sand.		D5-6I	12	24.2	103	PID = 8ppm
			Gray, wet, very dense, silty SAND with gravel (SM); fine to coarse sand; fine gravel; variably cemented.		D5-7I	71	13.5	125	PID = 0ppm TR
			Boring terminated at 30.5' below existing site grade.						

BORING LOG 10410.02 - 368 & 402 PETALUMA BLVD. NORTH.GPJ WKA.GDT 3/26/15 2:17 PM

Wallace Kuhl & ASSOCIATES

FIGURE 9

Project: 368 & 402 Petaluma Blvd. North **Project Location: Petaluma, California** **WKA Number: 10410.02**	**LOG OF SOIL BORING D6** Sheet 1 of 1

Date(s) Drilled	**2/17/15**	Logged By	**GJF**	Checked By	**DRG**
Drilling Method	**Hollow Stem Auger**	Drilling Contractor	**V & W Drilling, Inc.**	Total Depth of Drill Hole	**30.0 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**8"**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**11.0**	Sampling Method(s)	**Modified California**	Drill Hole Backfill	**Neat Cement**
Remarks	**Bulk Sample D6 (0' to 3.5')**			Driving Method and Drop	**140 lb hammer, 30 inch drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Dark gray, moist, stiff, sitly CLAY (CH - Fill); trace of organics; with a piece of fabric.		D6-1I	9	28.5		PID = 0ppm
			Brown, moist, loose, silty SAND (SM - Fill); fine sand; trace of organics; with pockets of silty clay.		D6-2I	7			PID = 0ppm
	5								
			loose						
			Brown, moist, very soft, sandy CLAY (CH - Fill); fine sand.		D6-3I	3	33.6	87	PID = 0ppm
	10								
			Olive-green to light brown, wet, loose, clayey SAND (SC - Fill); fine to medium sand.		D6-4I	8	24.2	98	PID = 0ppm UCC =0.6 tsf
	15		Dark gray, wet, medium stiff, silty CLAY (CH - Fill).						
			trace of organics		D6-5I	1	58.5	65	OC =3.2% PID = 0ppm
	20		Gray, wet, very loose, clayey SAND (SC); fine sand.						
			loose, fine to coarse sand, trace of fine gravel		D6-6I	5			PID = 0ppm GR 32%fines
	25								
			Gray, wet, very dense, poorly-graded SAND with silt and gravel (SP-SM); fine to coars sand; fine gravel; varaibly cemented.		D6-7I	67			PID = 0ppm GR 7%fines
	30								
			Boring terminated at 30' below existing site grade.						

BORING LOG 10410.02 - 368 & 402 PETALUMA BLVD. NORTH.GPJ WKA.GDT 3/26/15 2:17 PM

Wallace Kuhl & ASSOCIATES

FIGURE 10

UNIFIED SOIL CLASSIFICATION SYSTEM

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

OTHER SYMBOLS

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

= Drive Sampler: no recovery

= SPT Sampler

= Initial Water Level

= Final Water Level

= Estimated or gradational material change line

= Observed material change line

Laboratory Tests

PI = Plasticity Index

EI = Expansion Index

UCC = Unconfined Compression Test

TR = Triaxial Compression Test

GR = Gradational Analysis (Sieve)

OC = Organic Content Test

GRAIN SIZE CLASSIFICATION

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

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UNIFIED SOIL CLASSIFICATION SYSTEM

368 & 402 PETALUMA BOULEVARD NORTH

Petaluma, California

FIGURE	11
DRAWN BY	RWO
CHECKED BY	ML
PROJECT MGR	DRG
DATE	06/15
WKA NO.	10410.02

APPENDICES

APPENDIX A

General Project Information, Laboratory Testing and Results

APPENDIX A

A. GENERAL INFORMATION

The performance of a geotechnical engineering study for the proposed mixed-use complex to be constructed at 368 and 402 Petaluma Boulevard North in Petaluma, California, was authorized by Mr. Jeff Morgan of A.G. Spanos Companies on January 22, 2015. Authorization was for an investigation as described in our proposal letter dated January 15, 2015, sent to our client A.G. Spanos Companies whose address is 10100 Trinity Parkway, 5th Floor in Stockton, California; telephone (209) 955-2503.

The project architect is Kephart, whose mailing address is 2555 Walnut Street in Denver, Colorado 80205; telephone (303) 832-4474.

In performing this study, we made reference to *Unit Composite* drawing, dated April 20, 2015 and prepared by Kephart.

B. FIELD EXPLORATION

As part of our investigation for the mixed-use complex, our field exploration included the advancement of six cone penetrometer test soundings (CPT1 through CPT6) and the drilling and sampling of six borings (D1 through D6) at the approximate locations shown on Figure 2.

Cone penetrometer test soundings CPT1 through CPT3 and CPT4 through CPT6 were advanced at the site on February 17 and June 23, 2015, respectively, utilizing a 25-ton, truck-mounted rig provided by Gregg Drilling & Testing, Inc. of Martinez, California. The CPT's consisted of advancing a 10-square-centimeter cone penetrometer at a rate of about one (1) inch per second to depths ranging from about 41 to 92 feet below existing site grades. Data was collected from the cone penetrometer at an approximate depth interval of 10 centimeters (or 3.9 inches). Shear wave velocity data was collected from CPT1 at an approximate depth interval of 10 feet. Pore pressure dissipation tests were performed at the CPT's at depths ranging from about 22 to 36 feet below existing grades.

Borings D1 through D6 were drilled across the site on February 17 and 18, 2015, utilizing a CME-75 truck-mounted drill rig equipped with eight-inch (8") diameter, hollow stem augers, to depths ranging from 25 to 31 feet below existing site grades. At various intervals relatively undisturbed soil samples were recovered with a 2½-inch O.D., 2-inch I.D., modified California split-spoon sampler driven by a 140-pound hammer freely falling 30 inches. The number of blows of the hammer required to drive the 18-inch long sampler each six-inch (6") interval was recorded. The sum of the blows required to drive the sampler the lower 12-inch interval, or portion thereof, is designated as the penetration resistance or "blow count" for that particular drive. The samples were retained in two-inch (2") diameter by six-inch (6") long thin-walled brass tubes contained within the sampler. After recovery, the soils in the tubes were visually classified by the field representative and the ends of the tubes were sealed to preserve the natural moisture contents.

In addition to the driven sample from the borings, representative bulk samples of near-surface soils were also collected and retained in plastic bags. Driven and bulk samples were taken to our laboratory for additional soil classification and selection of samples for testing.

The Logs of Soil Borings, Figures 5 through 10, contain descriptions of the soils encountered at each boring location. A boring legend explaining the Unified Soil Classification System and the symbols used on the logs is contained on Figure 11.

Copies of the reports for CPT1 through CPT6, provided by Gregg Drilling & Testing, Inc. are included in Appendix B.

C. LABORATORY TESTING

Selected undisturbed samples of the soils were tested to determine dry unit weight (ASTM D2937), natural moisture content (ASTM D2216), shear strength by triaxial compression testing (ASTM D4767), and organic content (ASTM D2974). The results of these tests are included in the Logs of Borings at the depth each sample was obtained. The results of the triaxial shear strength testing are presented on Figure A1.

Five representative samples of near-surface cohesive soil were subjected to Atterberg Limits tests (ASTM D4318). The results of these tests are presented in Figure A2.

Three soil samples were tested for particle-size distribution (ASTM C136/D422) and percent passing the No. 200 sieve (ASTM D1140). The results of the particle-size distribution tests are contained in Figure No. A3. The percent passing the No. 200 sieve are included on the boring logs at the depth the samples were obtained.

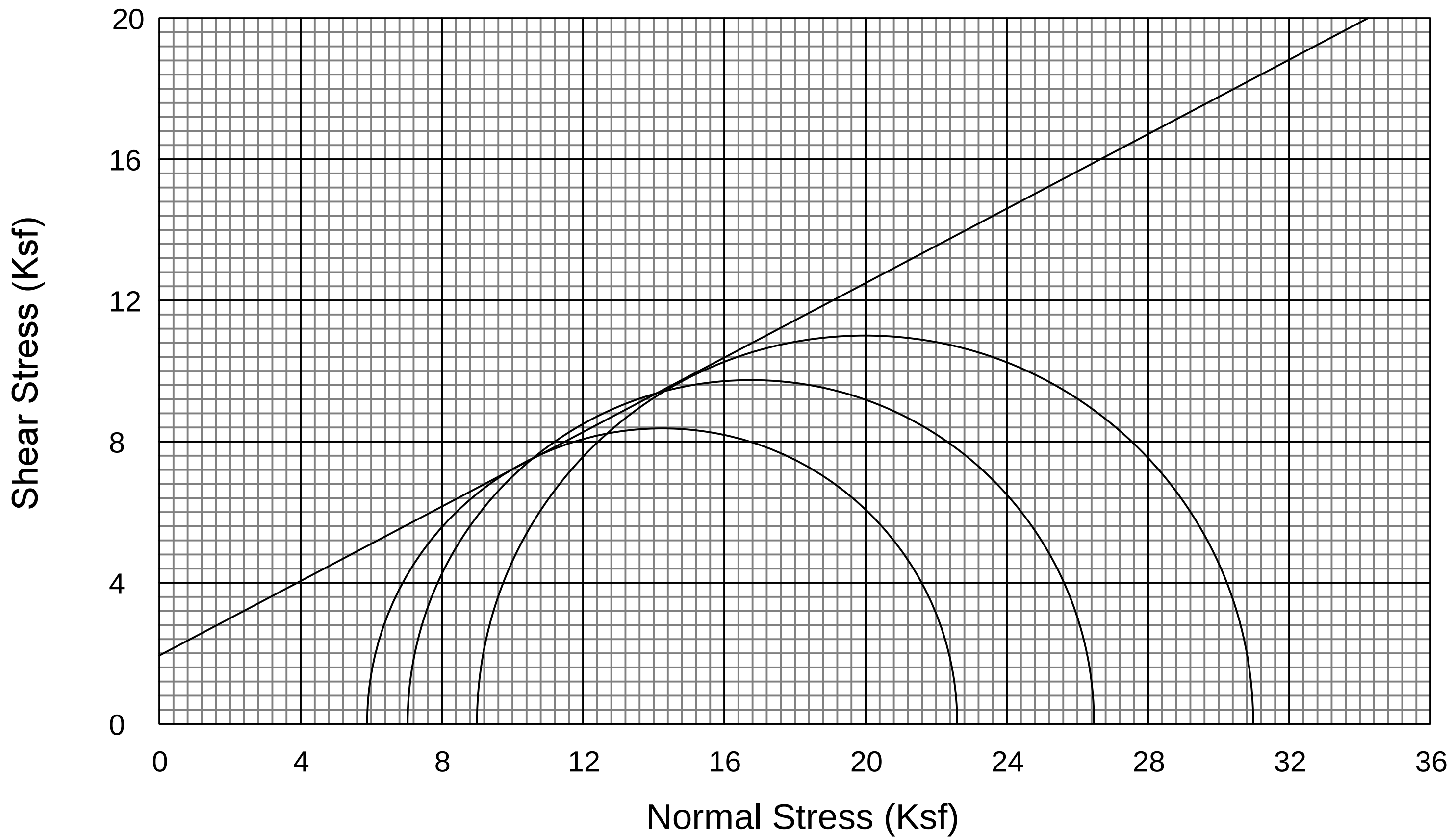
Two representative samples of near-surface soil from different areas of the site were subjected to Expansion Index testing (ASTM D4829); the results of the tests are presented in Figures A4 and A5.

One bulk sample of anticipated pavement subgrade soil was subjected to Resistance ("R") value testing in accordance with California Test 301. In addition, one sample was mixed with five percent (5%) dolomitic quicklime and subjected to an R-value test. The results of the R-value tests, which were used in the pavement design, are presented in Figure A6.

Three near-surface soil samples were submitted to Sunland Analytical to determine the soil pH and minimum resistivity (California Test 643), Sulfate concentration (California Test 417) and Chloride concentration (California Test 422). The results of these tests are presented in Figures A7 through A9.

TRIAXIAL COMPRESSION TEST

ASTM D4767



SAMPLE NO. : D5-7I

SAMPLE CONDITION : Undisturbed

SAMPLE DESCRIPTION : Gray, silty sand with gravel

DRY DENSITY (PCF) : 125
INITIAL MOISTURE (%) : 13.5
FINAL MOISTURE (%) : 14.6

ANGLE OF INTERNAL FRICTION (ø) : 28°
COHESION (PSF) : 2000

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TRIAXIAL COMPRESSION TEST RESULTS

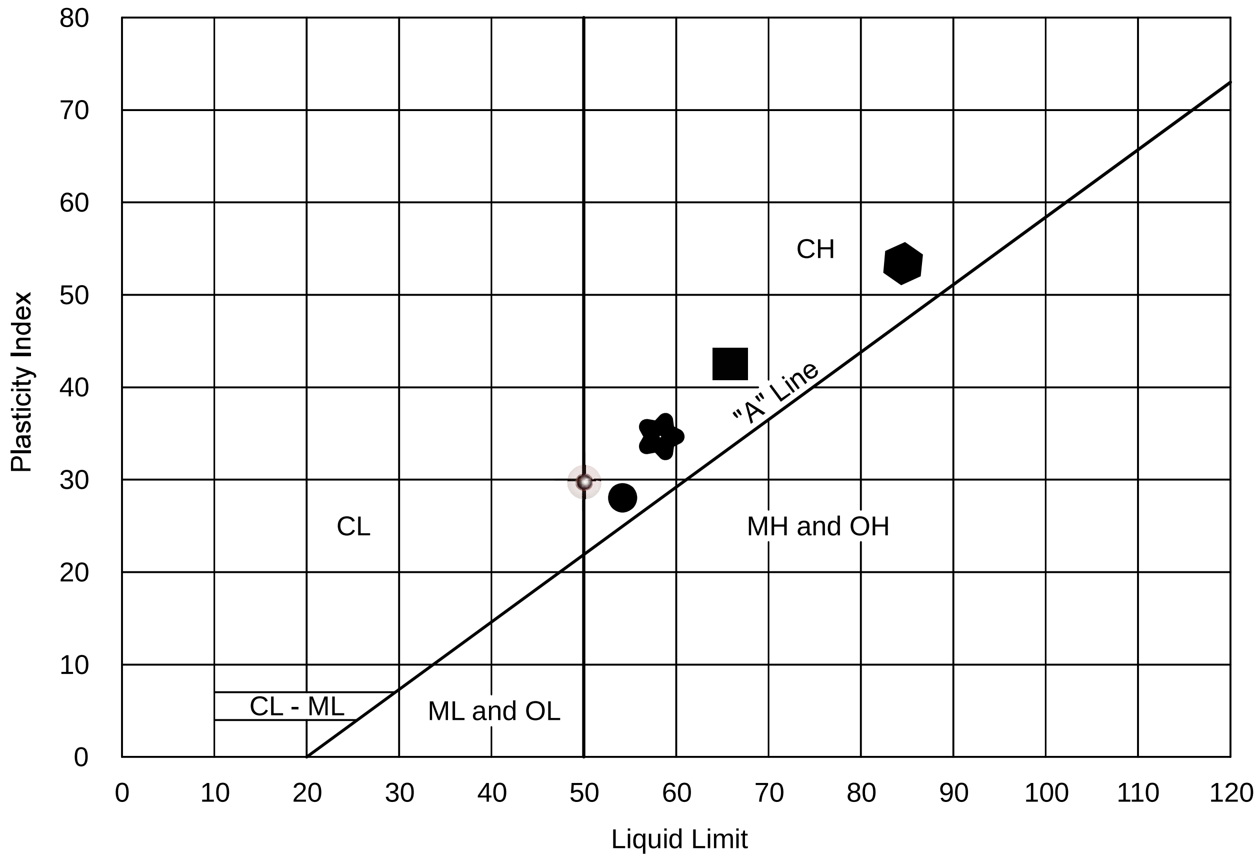
368 & 402 PETALUMA BOULEVARD NORTH

Petaluma, California

FIGURE	A1
DRAWN BY	RWO
CHECKED BY	ML
PROJECT MGR	DRG
DATE	06/15
WKA NO.	10410.02

ATTERBERG LIMITS

ASTM D4318



KEY SYMBOL	LOCATION	SAMPLE DEPTH	NATURAL WATER CONTENT (%)	ATTERBERG LIMITS		PASSING No. 200 SIEVE (%)	UNIFIED SOIL CLASSIFI-CATION SYMBOL
				LIQUID LIMIT (%)	PLASTICITY INDEX (%)		
●	D1-3I	6'-6.5'	---	54	29	---	CH
■	D4-4II	15.5'-16'	47.3	64	42	---	CH
✱	D5-1I	2'-2.5'	---	59	35	---	CH
◉	D5-4II	14'-14.5'	---	50	30	---	CH
⬢	D5-5II	19'-19.5'	---	84	52	---	CH

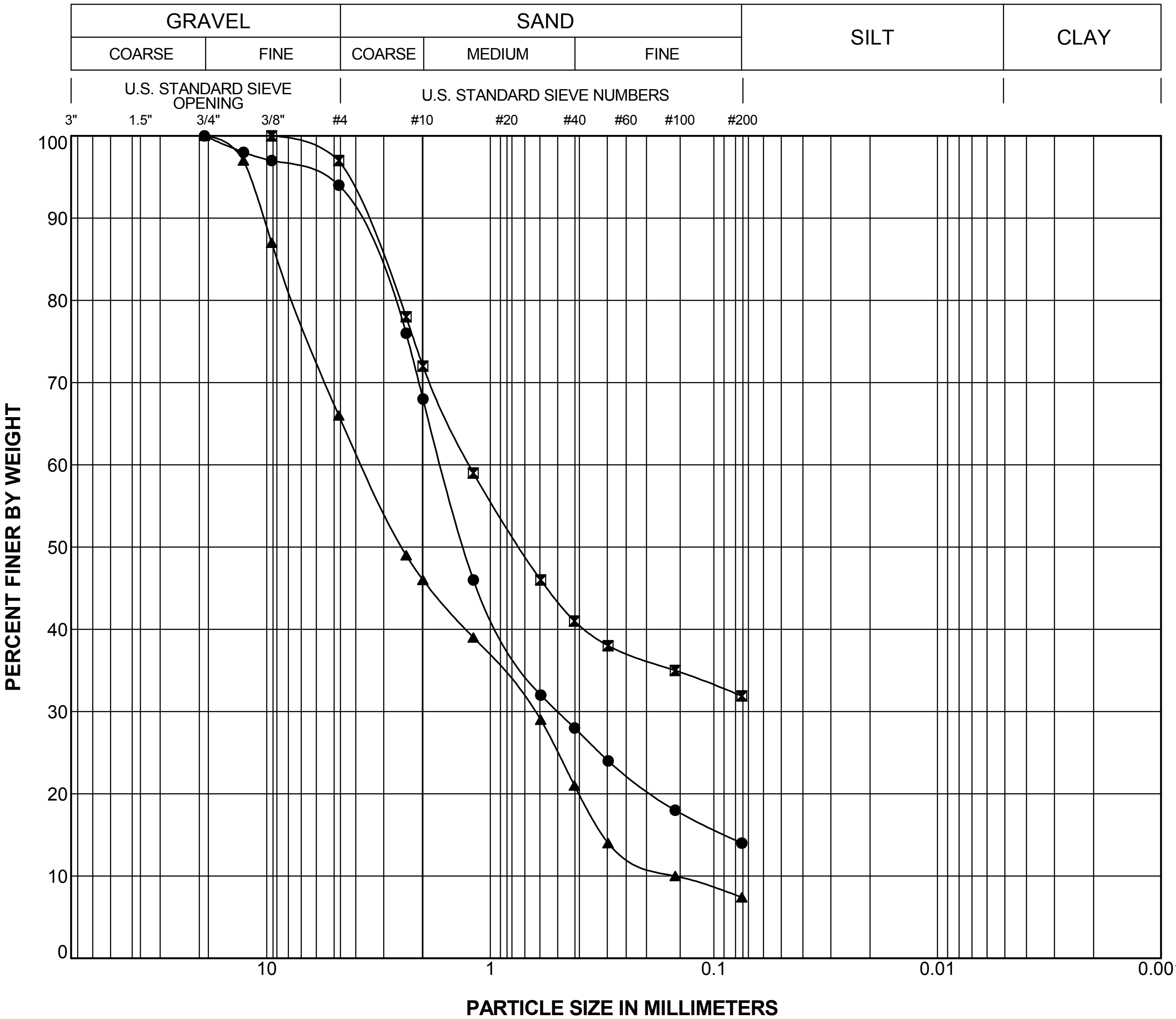
ATTERBERG LIMITS

368 & 402 PETALUMA BOULEVARD NORTH

Petaluma, California

FIGURE	A2
DRAWN BY	RWO
CHECKED BY	ML
PROJECT MGR	DRG
DATE	06/15

WKA NO. 10410.02



Boring Number	Sample Number	USCS	Depth (feet)	Symbol	LL	PI	Classification
D3	D3-4I	SC	14.5 to 15	●			clayey SAND with gravel
D6	D6-6I	SC	24.5 to 25	⧗			clayey SAND, trace of gravel
D6	D6-7I	SP-SM	29.5 to 30	▲			poorly-graded SAND with silt and gravel

PARTICLE SIZE DISTRIBUTION

Project: 368 & 402 Petaluma Blvd. North
WKA No. 10410.02

FIGURE A3

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EXPANSION INDEX TEST RESULTS

ASTM D4829

MATERIAL DESCRIPTION: Brown, silty clay

LOCATION: D5

Sample Depth	Pre-Test Moisture (%)	Post-Test Moisture (%)	Dry Density (pcf)	Expansion Index
0' - 5'	14.3	32.9	92.6	**94**

CLASSIFICATION OF EXPANSIVE SOIL *

EXPANSION INDEX	POTENTIAL EXPANSION
0 - 20	Very Low
21 - 50	Low
51 - 90	Medium
91 - 130	**High**
Above 130	Very High

* From ASTM D4829, Table 1

WallaceKuhl & ASSOCIATES

EXPANSION INDEX

368 & 402 PETALUMA BOULEVARD NORTH

Petaluma, California

FIGURE	A4
DRAWN BY	RWO
CHECKED BY	ML
PROJECT MGR	DRG
DATE	06/15
WKA NO. 10410.02	

EXPANSION INDEX TEST RESULTS

ASTM D4829

MATERIAL DESCRIPTION: Dark gray, silty clay

LOCATION: D6

Sample Depth	Pre-Test Moisture (%)	Post-Test Moisture (%)	Dry Density (pcf)	Expansion Index
0' - 3.5'	12.8	27.5	97.5	**57**

CLASSIFICATION OF EXPANSIVE SOIL *

EXPANSION INDEX	POTENTIAL EXPANSION
0 - 20	Very Low
21 - 50	Low
51 - 90	**Medium**
91 - 130	High
Above 130	Very High

* From ASTM D4829, Table 1

WallaceKuhl & ASSOCIATES

EXPANSION INDEX

368 & 402 PETALUMA BOULEVARD NORTH

Petaluma, California

FIGURE	A5
DRAWN BY	RWO
CHECKED BY	ML
PROJECT MGR	DRG
DATE	06/15

WKA NO. 10410.02

RESISTANCE VALUE TEST RESULTS

(California Test 301)

MATERIAL DESCRIPTION: Brown, silty clay with 5% lime

LOCATION: D5 (0' - 5')

Specimen No.	Dry Unit Weight (pcf)	Moisture @ Compaction (%)	Exudation Pressure (psi)	Expansion Pressure (dial, inches x 1000)	Expansion Pressure (psf)	R Value
1	106	18	540	13	56	88
2	107	19	336	12	52	83
3	104	19	259	11	48	74

R-Value at 300 psi exudation pressure = **79**

MATERIAL DESCRIPTION: Dark gray, silty clay

LOCATION: D6 (0' - 3.5')

Specimen No.	Dry Unit Weight (pcf)	Moisture @ Compaction (%)	Exudation Pressure (psi)	Expansion Pressure (dial, inches x 1000)	Expansion Pressure (psf)	R Value
1	117	14	796	66	286	0

Sample extruded, therefore R-Value = **5**

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RESISTANCE VALUE TEST RESULTS

368 &402 PETALUMA BOULEVARD NORTH

Petaluma, California

FIGURE	A6
DRAWN BY	RWO
CHECKED BY	ML
PROJECT MGR	DRG
DATE	06/15
WKA NO.	10410.02

Sunland Analytical
11419 Sunrise Gold Cir.#10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 02/27/15
Date Submitted 02/23/15

To: Mauricio Luna
Wallace-Kuhl & Assoc.
3050 Industrial Blvd
West Sacramento, CA, 95691

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following:
Location : 10410.02-PETALUMA N. Site ID: D5 @ 0-5 FT
Your purchase order number is 1522.
Thank you for your business.

* For future reference to this analysis please use SUN # 68829 - 142999

EVALUATION FOR SOIL CORROSION

Soil pH	6.93			
Minimum Resistivity	0.86	ohm-cm (x1000)		
Chloride	37.1	ppm	0.0037	%
Sulfate-S	16.5	ppm	0.0017	%

METHODS:
pH and Min.Resistivity CA DOT Test #643 Mod.(Sm.Cell)
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

WallaceKuhl & ASSOCIATES

CORROSION TEST RESULTS

368 & 402 PETALUMA BOULEVARD NORTH

Petaluma, California

FIGURE	A7
DRAWN BY	RWO
CHECKED BY	ML
PROJECT MGR	DRG
DATE	06/15
WKA NO.	10410.02

Sunland Analytical
11419 Sunrise Gold Cir.#10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 02/27/15
Date Submitted 02/23/15

To: Mauricio Luna
Wallace-Kuhl & Assoc.
3050 Industrial Blvd
West Sacramento, CA, 95691

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following:
Location : 10410.02-PETALUMA N. Site ID: D5-3II
Your purchase order number is 1522.
Thank you for your business.

* For future reference to this analysis please use SUN # 68829 - 143000

EVALUATION FOR SOIL CORROSION

Soil pH	8.55		
Minimum Resistivity	0.59	ohm-cm (x1000)	
Chloride	135.7 ppm	0.0136	%
Sulfate-S	113.2 ppm	0.0113	%

METHODS:
pH and Min.Resistivity CA DOT Test #643 Mod.(Sm.Cell)
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

WallaceKuhl & ASSOCIATES

CORROSION TEST RESULTS

368 & 402 PETALUMA BOULEVARD NORTH

Petaluma, California

FIGURE	A8
DRAWN BY	RWO
CHECKED BY	ML
PROJECT MGR	DRG
DATE	06/15
WKA NO.	10410.02

Sunland Analytical

11419 Sunrise Gold Cir.#10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 02/27/15
Date Submitted 02/23/15

To: Mauricio Luna
Wallace-Kuhl & Assoc.
3050 Industrial Blvd
West Sacramento, CA, 95691

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following:
Location : 10410.02-PETALUMA N. Site ID: D6-5II
Your purchase order number is 1522.
Thank you for your business.

* For future reference to this analysis please use SUN # 68829 - 143001

EVALUATION FOR SOIL CORROSION

Soil pH	8.45		
Minimum Resistivity	0.51	ohm-cm (x1000)	
Chloride	482.3 ppm	0.0482	%
Sulfate-S	406.7 ppm	0.0407	%

METHODS:
pH and Min.Resistivity CA DOT Test #643 Mod.(Sm.Cell)
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

WallaceKuhl & ASSOCIATES

CORROSION TEST RESULTS

368 & 402 PETALUMA BOULEVARD NORTH

Petaluma, California

FIGURE	A9
DRAWN BY	RWO
CHECKED BY	ML
PROJECT MGR	DRG
DATE	06/15
WKA NO.	10410.02

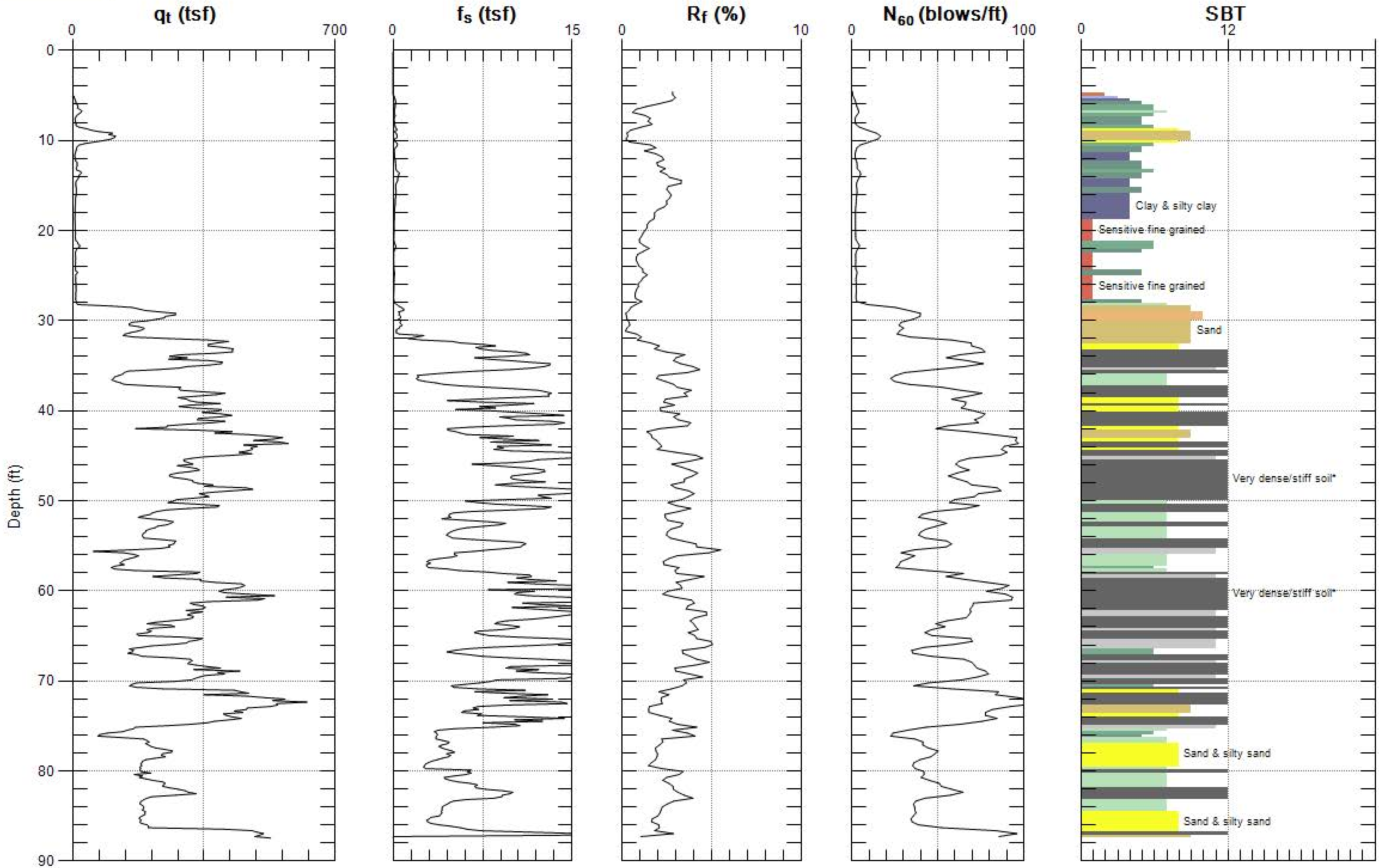
APPENDIX B

CPT Reports

GREGG **WALLACE-KUHL**

Site: 368-402 PETALUMA BLVD Engineer: D.DICKEY

Sounding: CPT-01 Date: 2/18/2015 08:35



Max. Depth: 87.434 (ft)
Avg. Interval: 0.328 (ft)

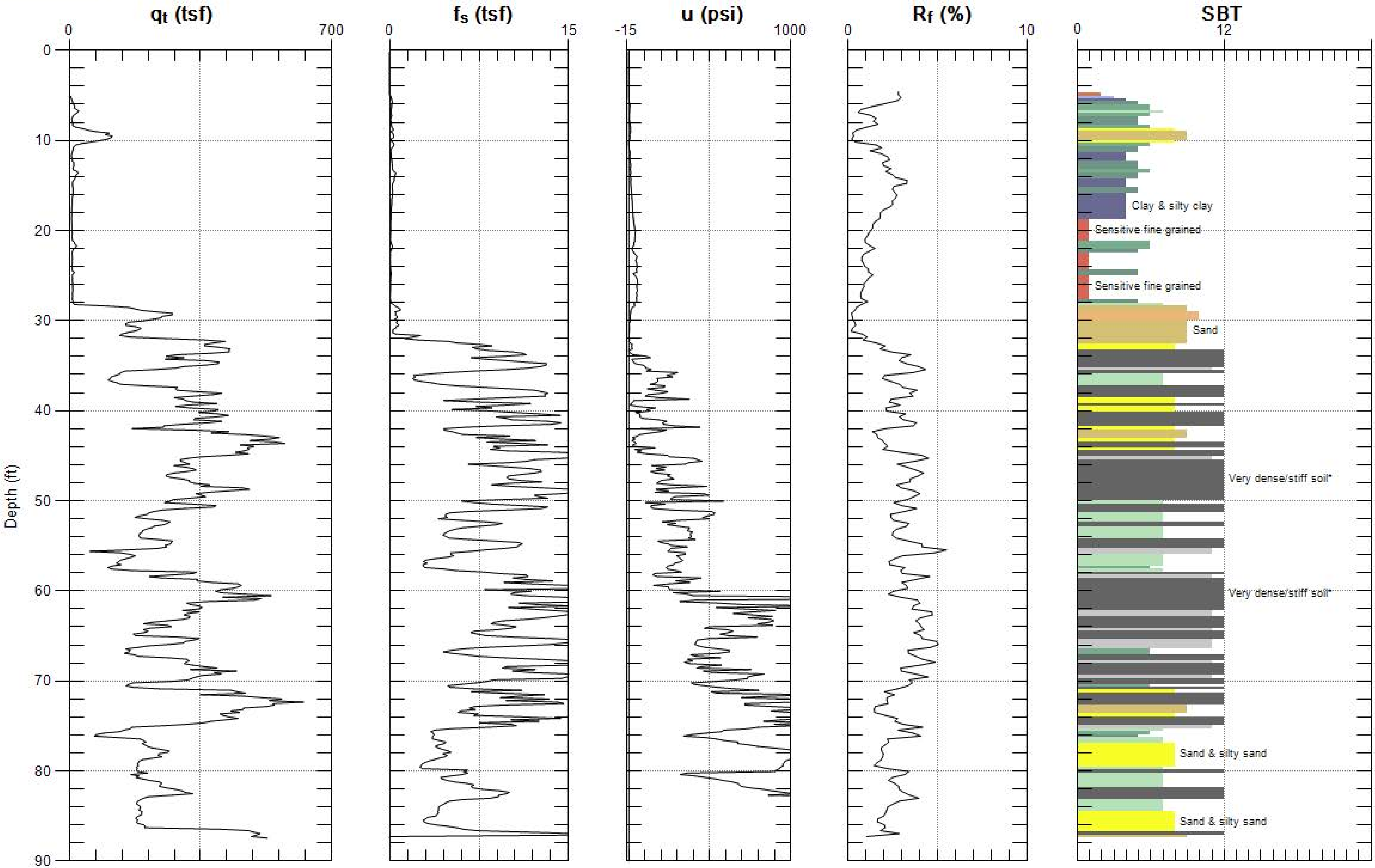
SBT: Soil Behavior Type (Robertson 1990)

GREGG

WALLACE-KUHL

Site: 368-402 PETALUMA BLVD **Engineer: D.DICKEY**

Sounding: CPT-01 **Date: 2/18/2015 08:35**



Max. Depth: 87.434 (ft)
Avg. Interval: 0.328 (ft)

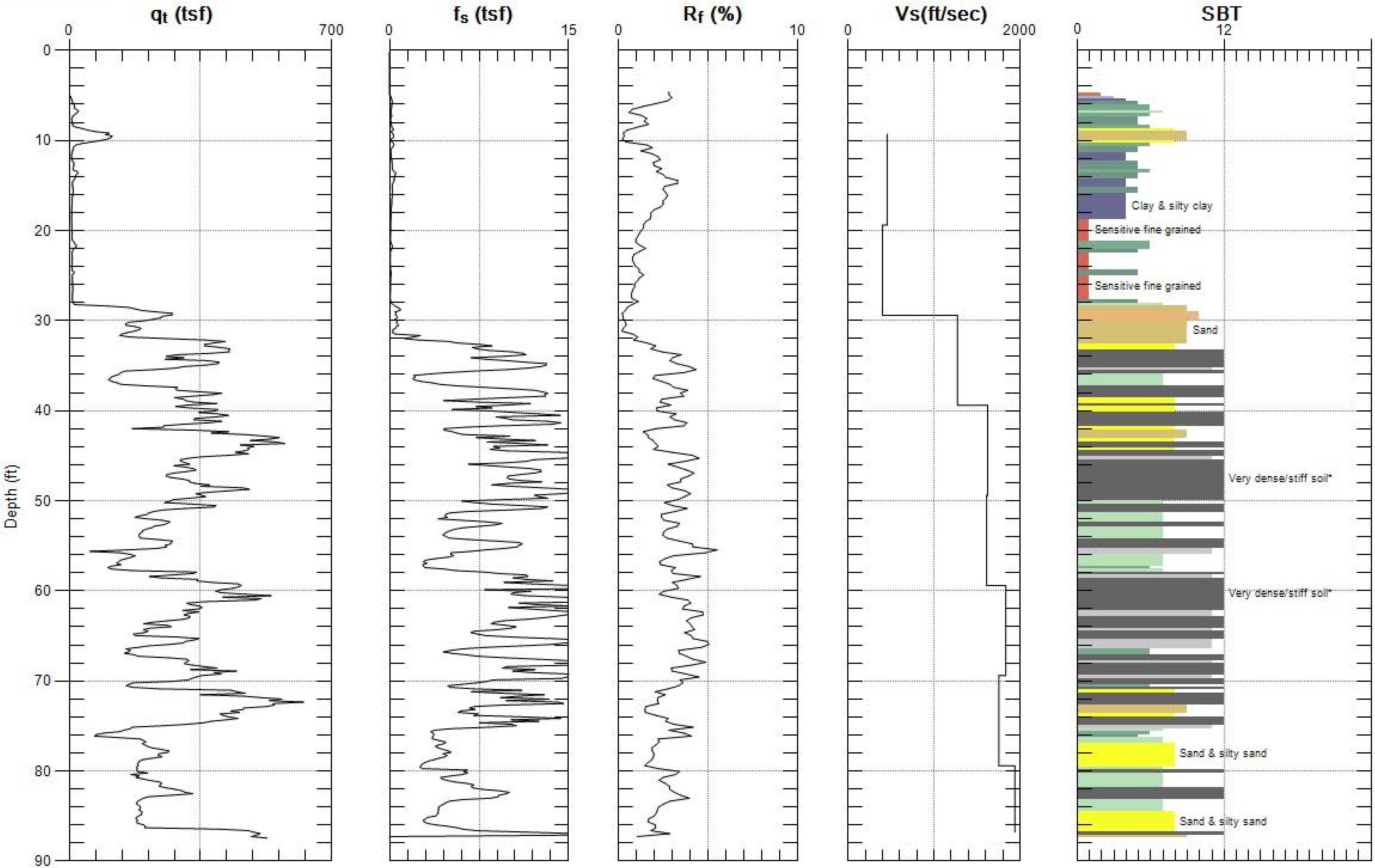
SBT: Soil Behavior Type (Robertson 1990)

GREGG

WALLACE-KUHL

Site: 368-402 PETALUMA BLVDEngineer: D.DICKEY

Sounding: CPT-01 Date: 2/18/2015 08:35



Max. Depth: 87.434 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

Shear Wave Velocity Calculations

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]			
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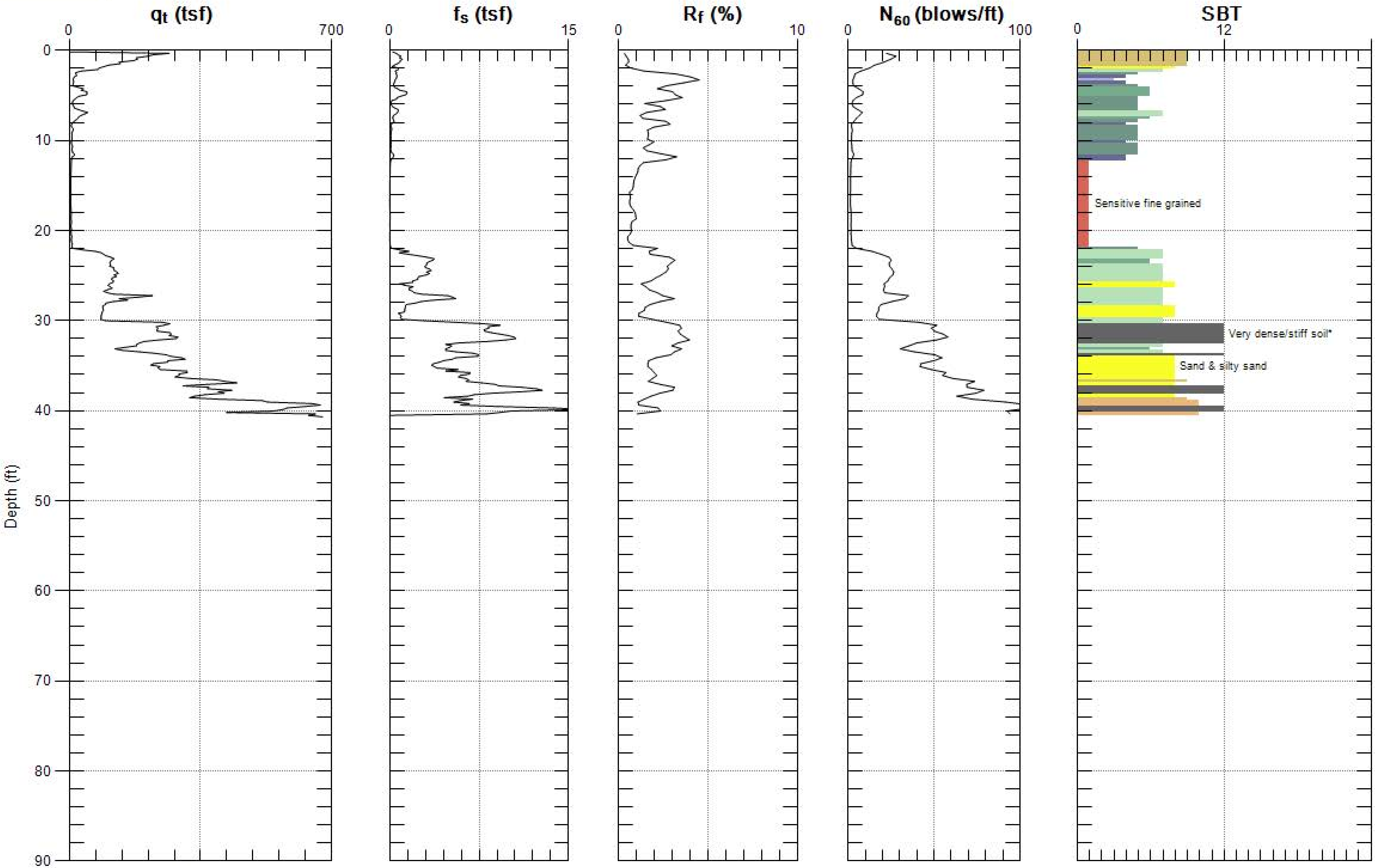
GREGG

WALLACE-KUHL

Site: 368-402 PETALUMA BLVD Engineer: D.DICKEY

Sounding: CPT-02 Date: 2/18/2015 01:07



Max. Depth: 40.682 (ft)
Avg. Interval: 0.328 (ft)

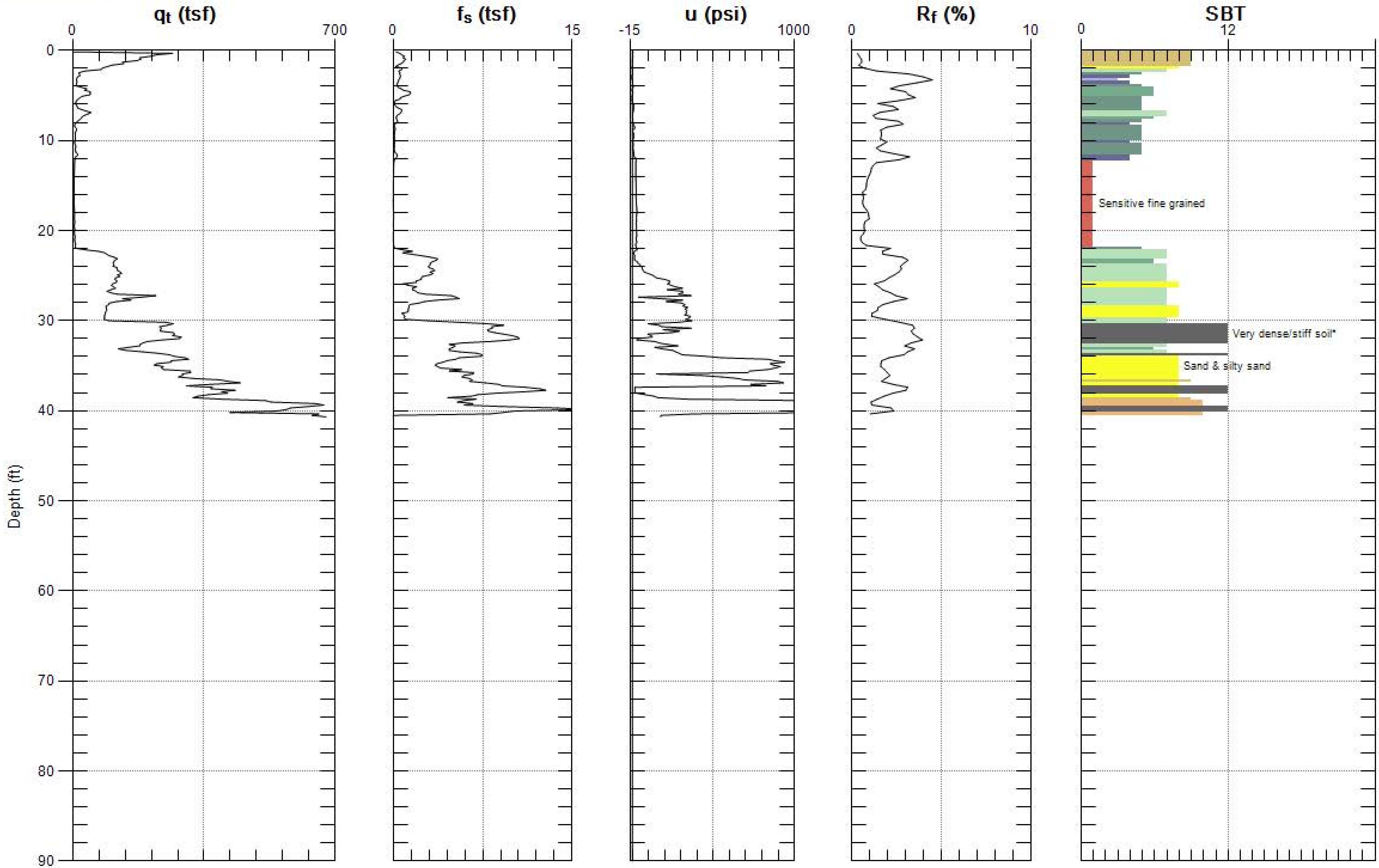
SBT: Soil Behavior Type (Robertson 1990)

GREGG

WALLACE-KUHL

Site: 368-402 PETALUMA BLVDEngineer: D.DICKEY

Sounding: CPT-02 Date: 2/18/2015 01:07



Max. Depth: 40.682 (ft)
Avg. Interval: 0.328 (ft)

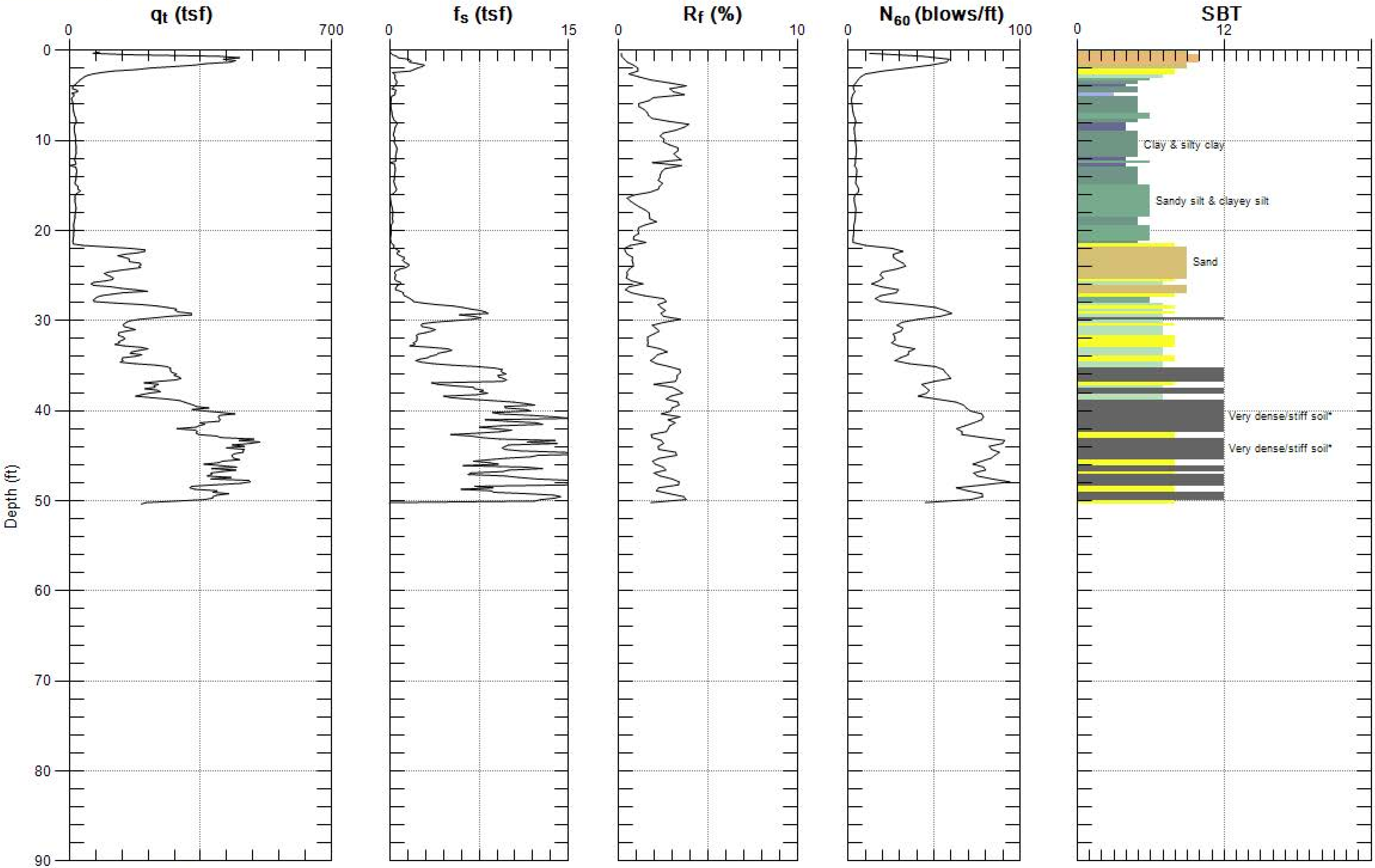
SBT: Soil Behavior Type (Robertson 1990)

GREGG

WALLACE-KUHL

Site: 368-402 PETALUMA BLVDEngineer: D.DICKEY

Sounding: CPT-03 Date: 2/18/2015 02:48



Max. Depth: 50.361 (ft)
Avg. Interval: 0.328 (ft)

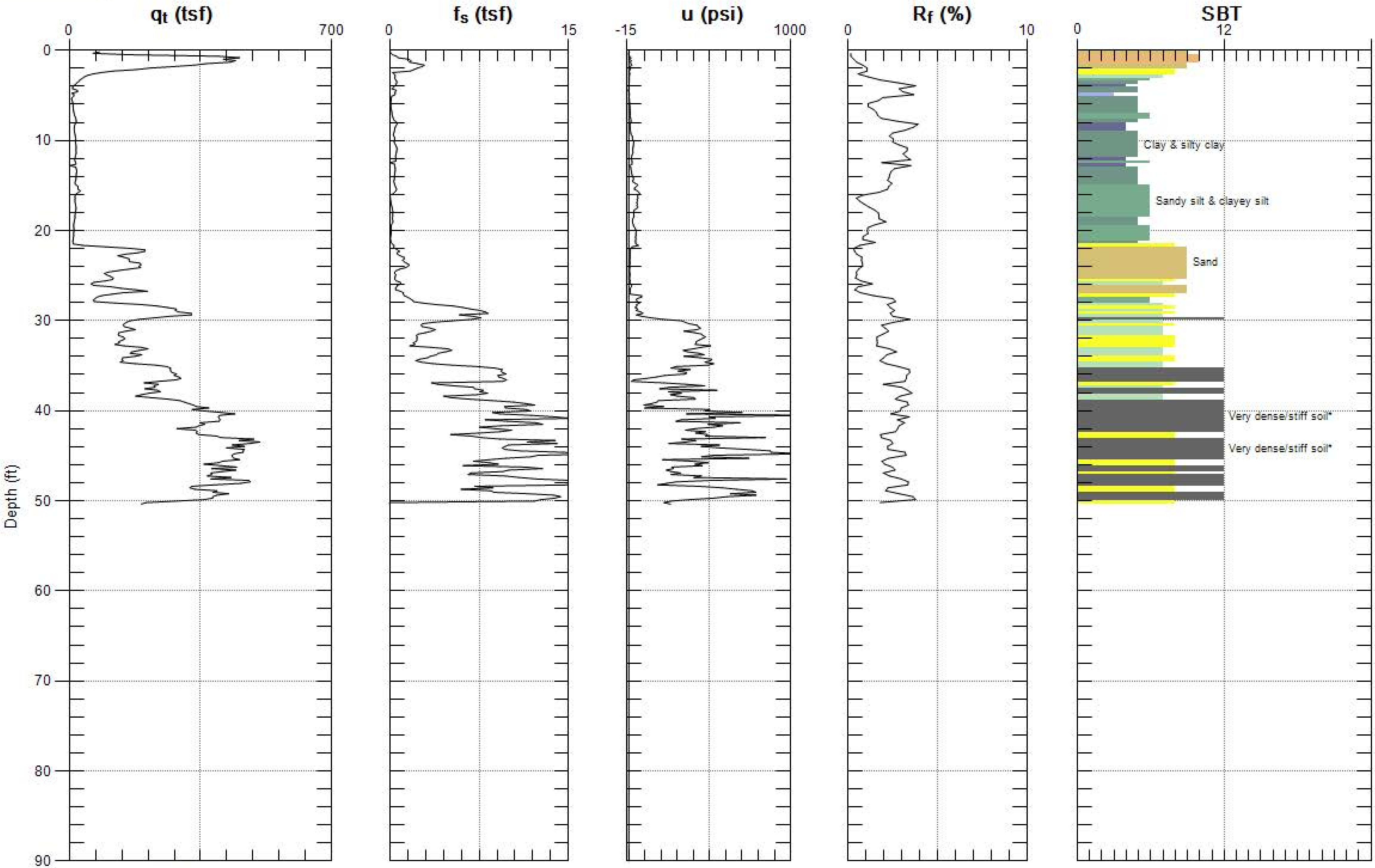
SBT: Soil Behavior Type (Robertson 1990)

GREGG

WALLACE-KUHL

Site: 368-402 PETALUMA BLVDEngineer: D.DICKEY

Sounding: CPT-03 Date: 2/18/2015 02:48



Max. Depth: 50.361 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

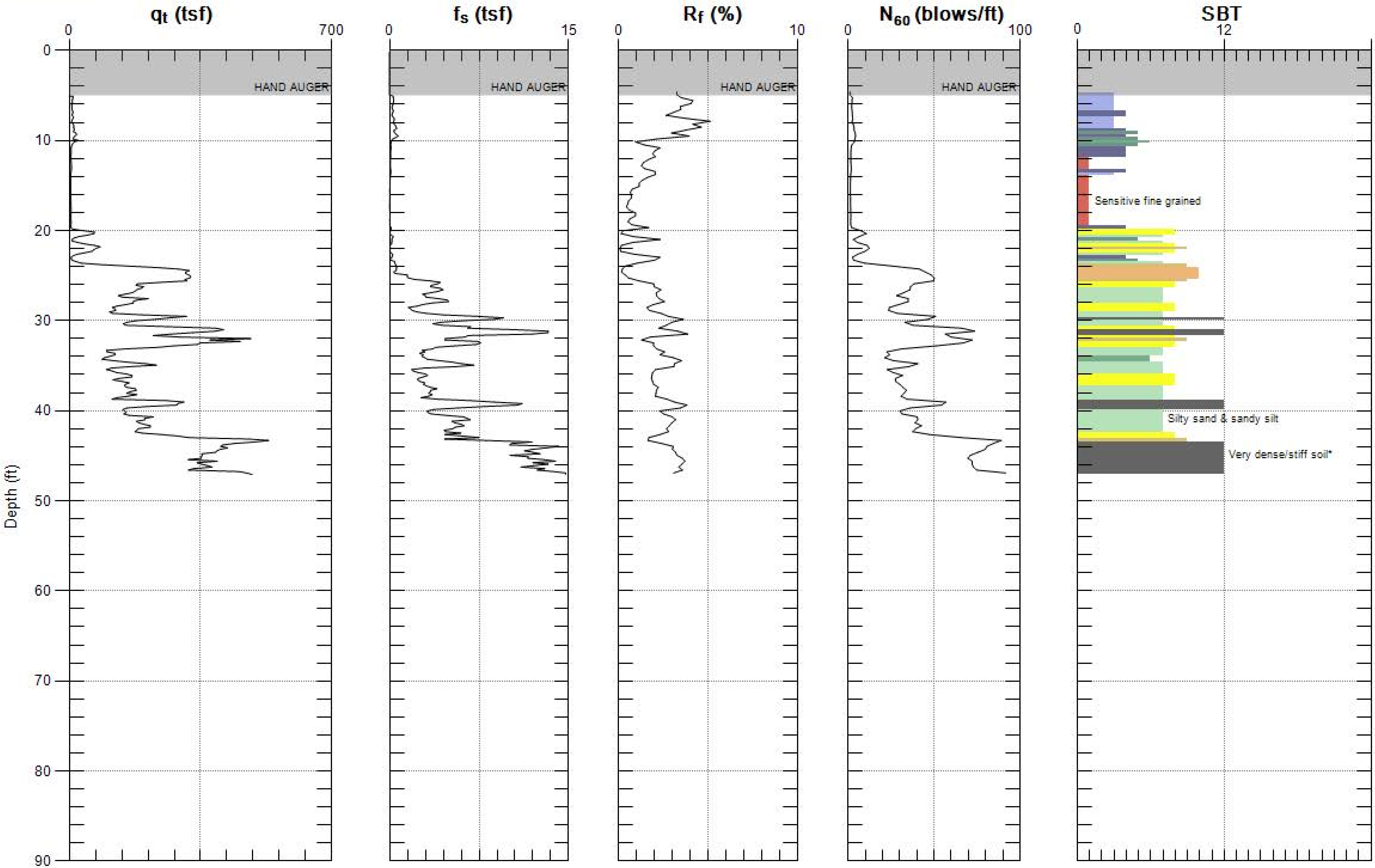
WALLACE-KUHL & ASSOC.

Site: 368&402 PETALUMA

Sounding: CPT-4

Engineer: M.LUNA

Date: 6/23/2015 07:29



Max. Depth: 47.080 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

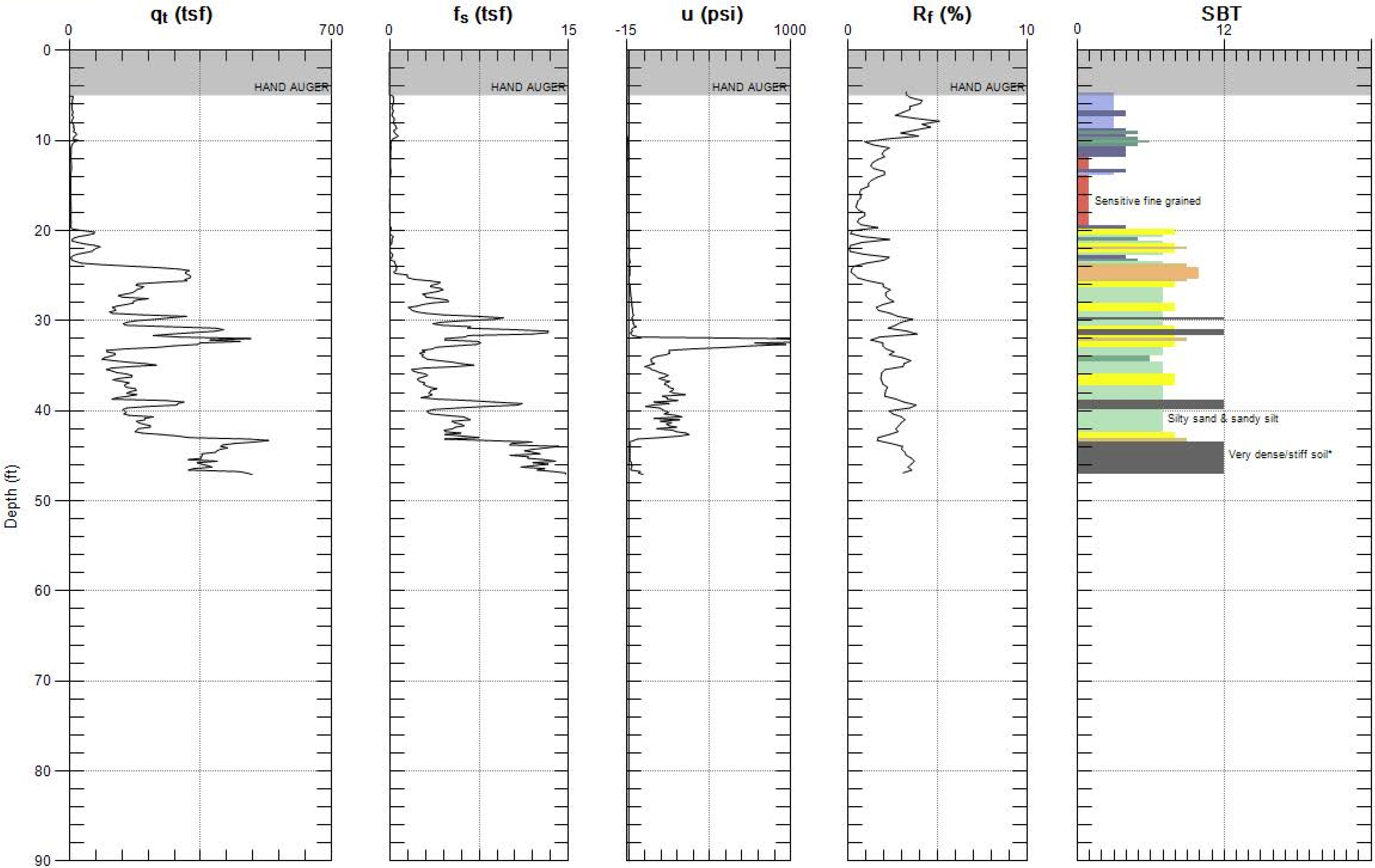
WALLACE-KUHL & ASSOC.

Site: 368&402 PETALUMA

Sounding: CPT-4

Engineer: M.LUNA

Date: 6/23/2015 07:29



Max. Depth: 47.080 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

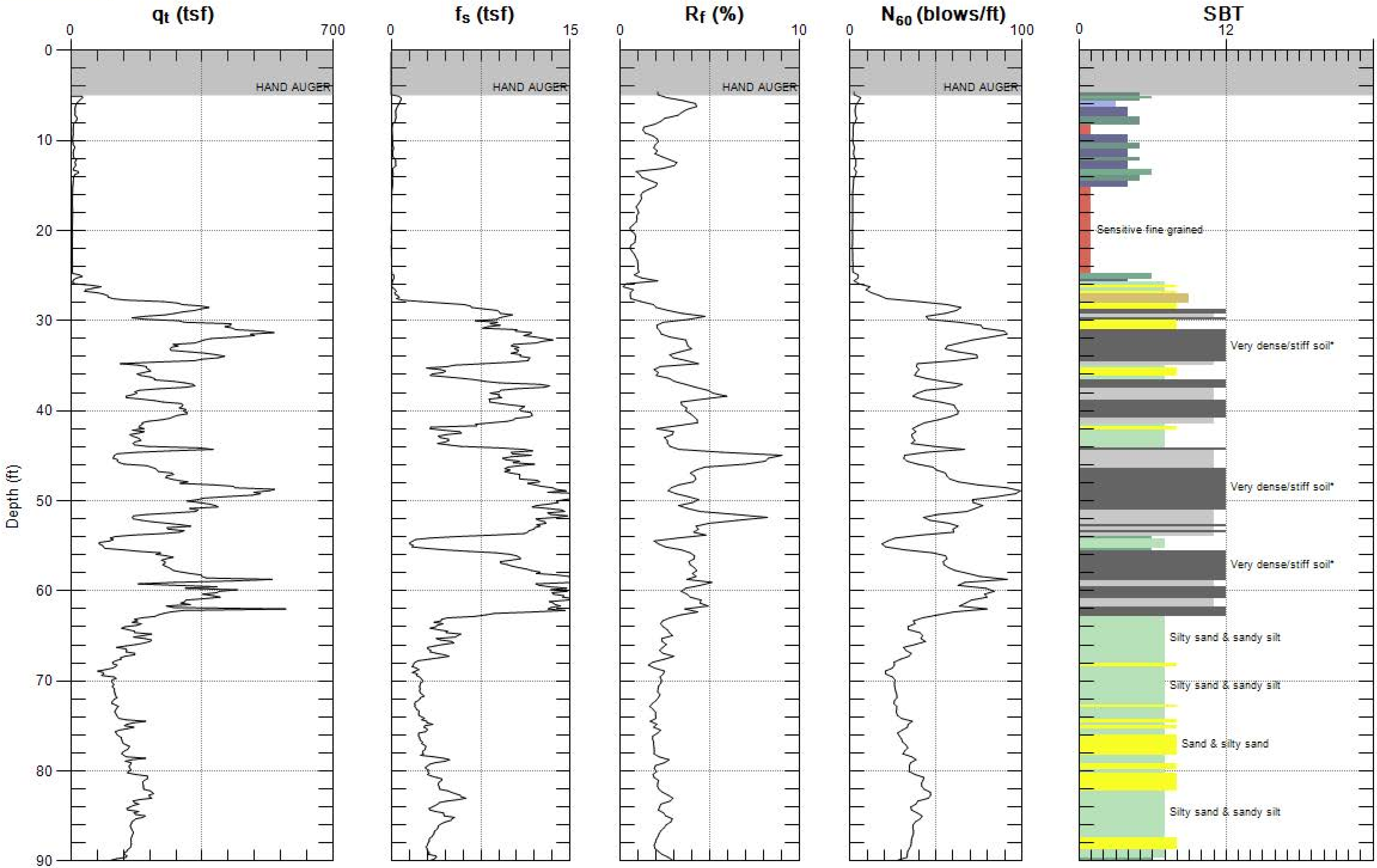
WALLACE-KUHL & ASSOC.

Site: 368&402 PETALUMA

Sounding: CPT-5

Engineer: M.LUNA

Date: 6/23/2015 10:10



Max. Depth: 91.699 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

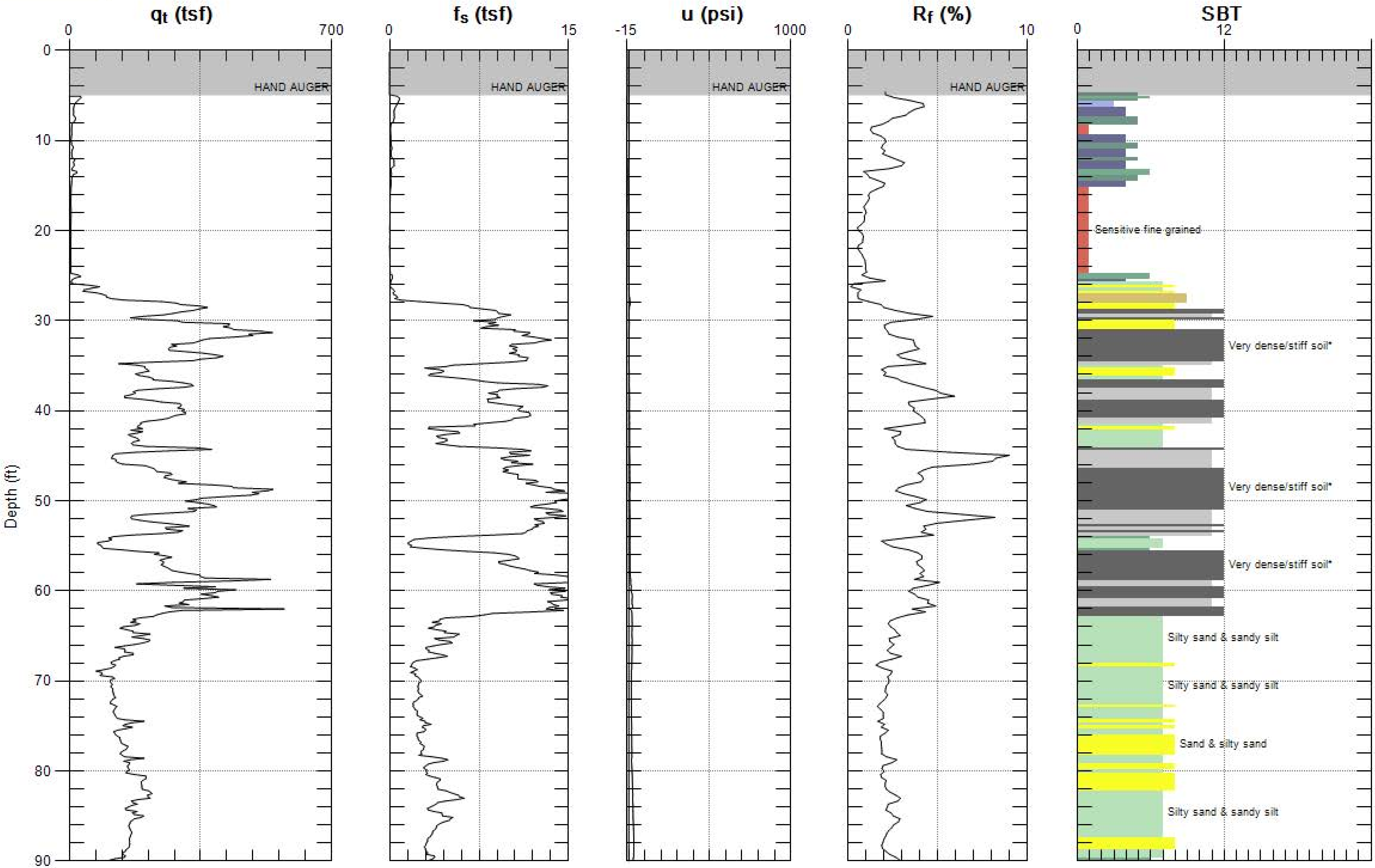
WALLACE-KUHL & ASSOC.

Site: 368&402 PETALUMA

Sounding: CPT-5

Engineer: M.LUNA

Date: 6/23/2015 10:10



Max. Depth: 91.699 (ft)
Avg. Interval: 0.328 (ft)

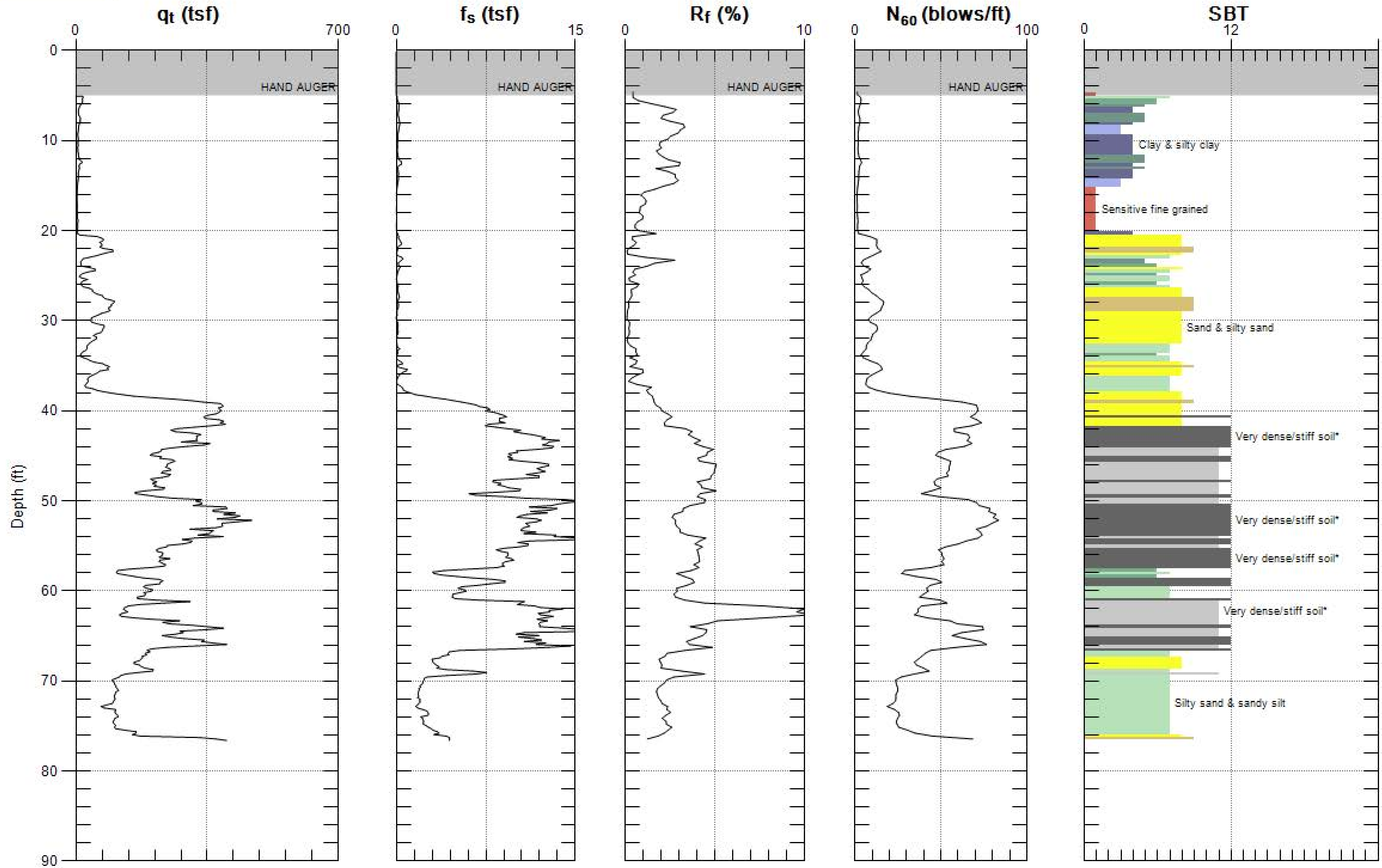
SBT: Soil Behavior Type (Robertson 1990)

GREGG

WALLACE-KUHL & ASSOC.

Site: 368&402 PETALUMA
Sounding: CPT-6

Engineer: M.LUNA
Date: 6/23/2015 01:01



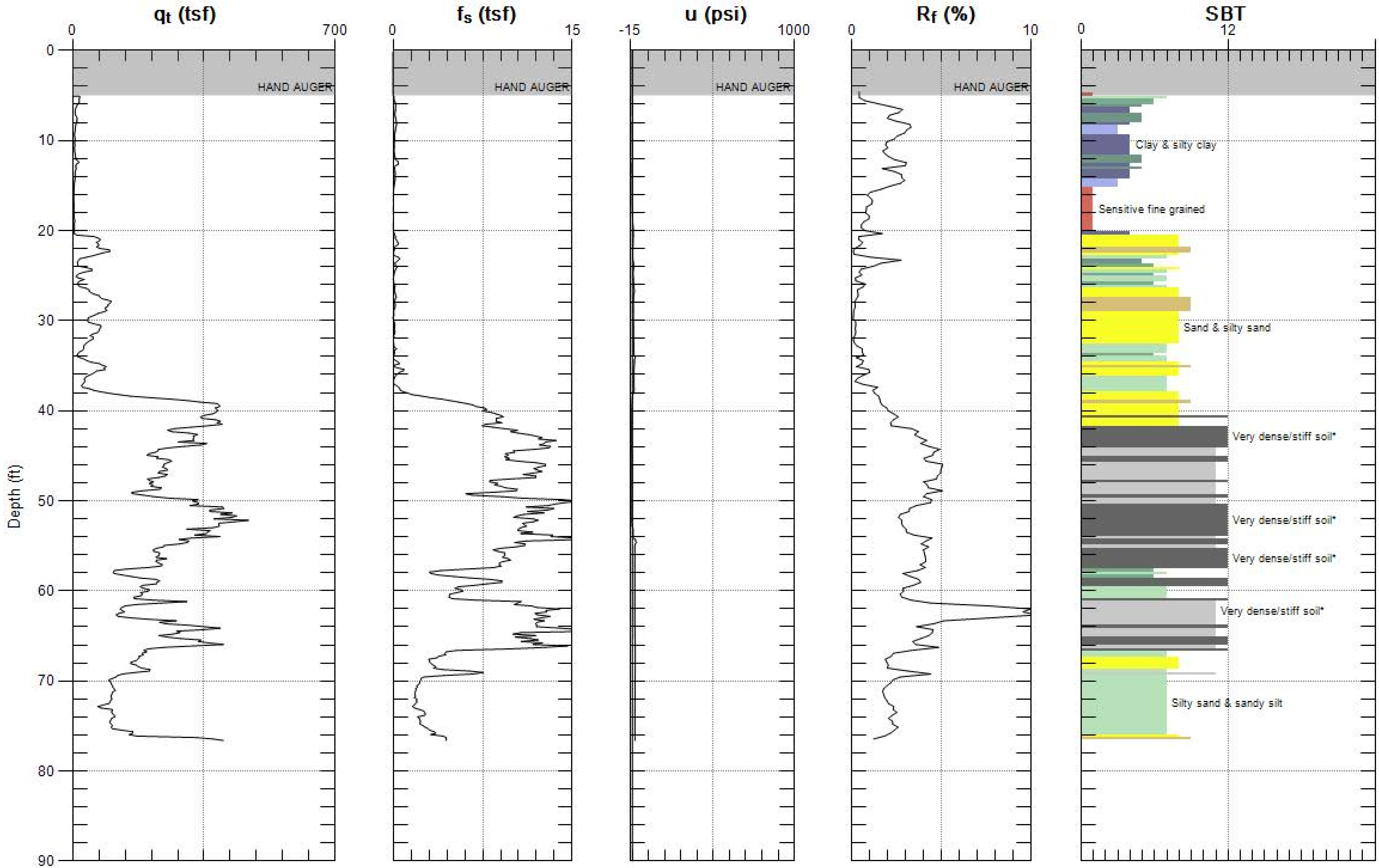
Max. Depth: 76.608 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

WALLACE-KUHL & ASSOC.

Site: 368&402 PETALUMA
Engineer: M.LUNA
Sounding: CPT-6
Date: 6/23/2015 01:01



Max. Depth: 76.608 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

APPENDIX C

Liquefaction Analysis Results

GeoLogismiki
Geotechnical Engineering Software
Merarhias 56, 621 25 - Serrai, Greece
url: http://www.geologismiki.gr - email: info@geologismiki.gr

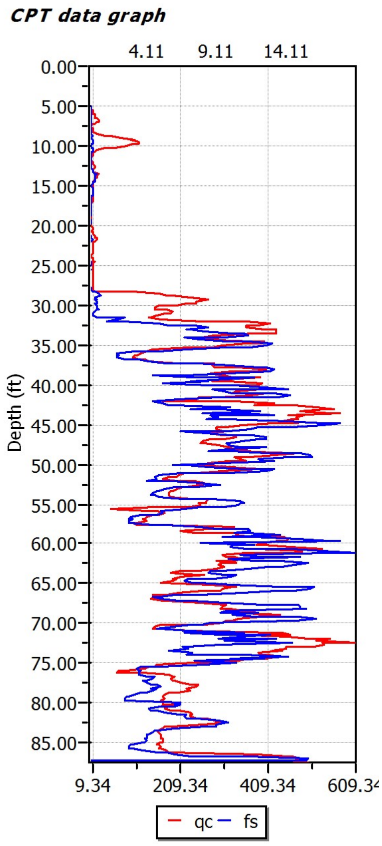
LIQUEFACTION ANALYSIS REPORT

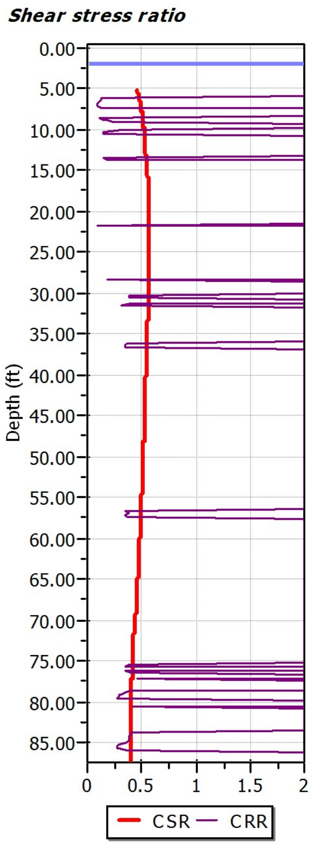
Project title : 10410.01 - Petaluma Blvd. North

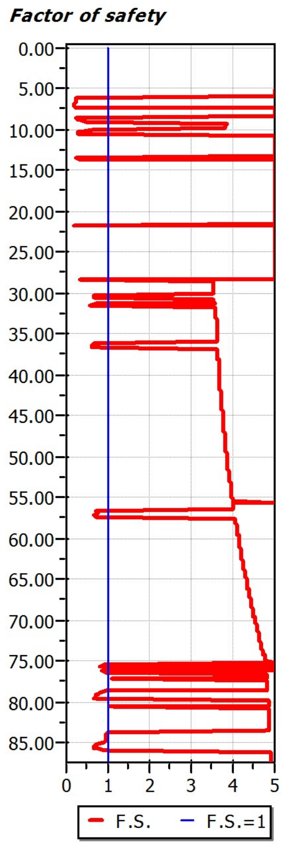
Project subtitle : CPT-1

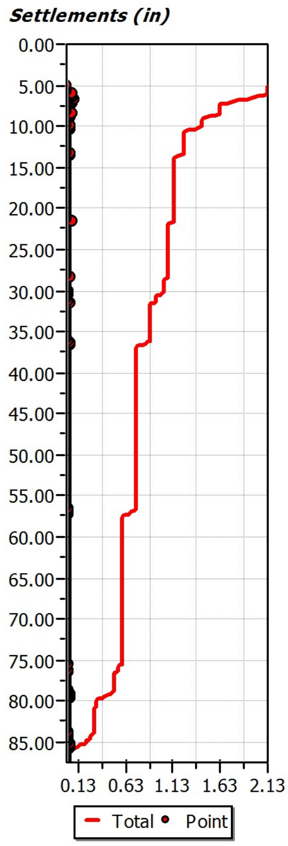
Input parameters and analysis data

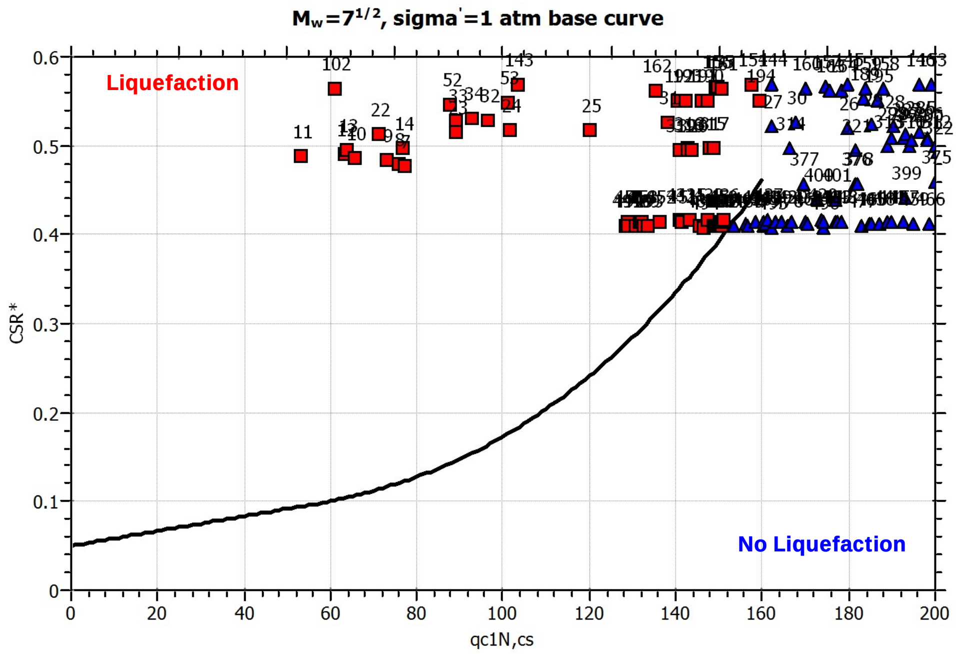
In-situ data type:	Cone Penetration Test	Depth to water table:	2.00 ft
Analysis type:	Deterministic	Earthquake magnitude M_w:	7.05
Analysis method:	Robertson (1998)	Peak ground accelaration:	0.57 g
Fines correction method:	Robertson (1998)	User defined F.S.:	1.00

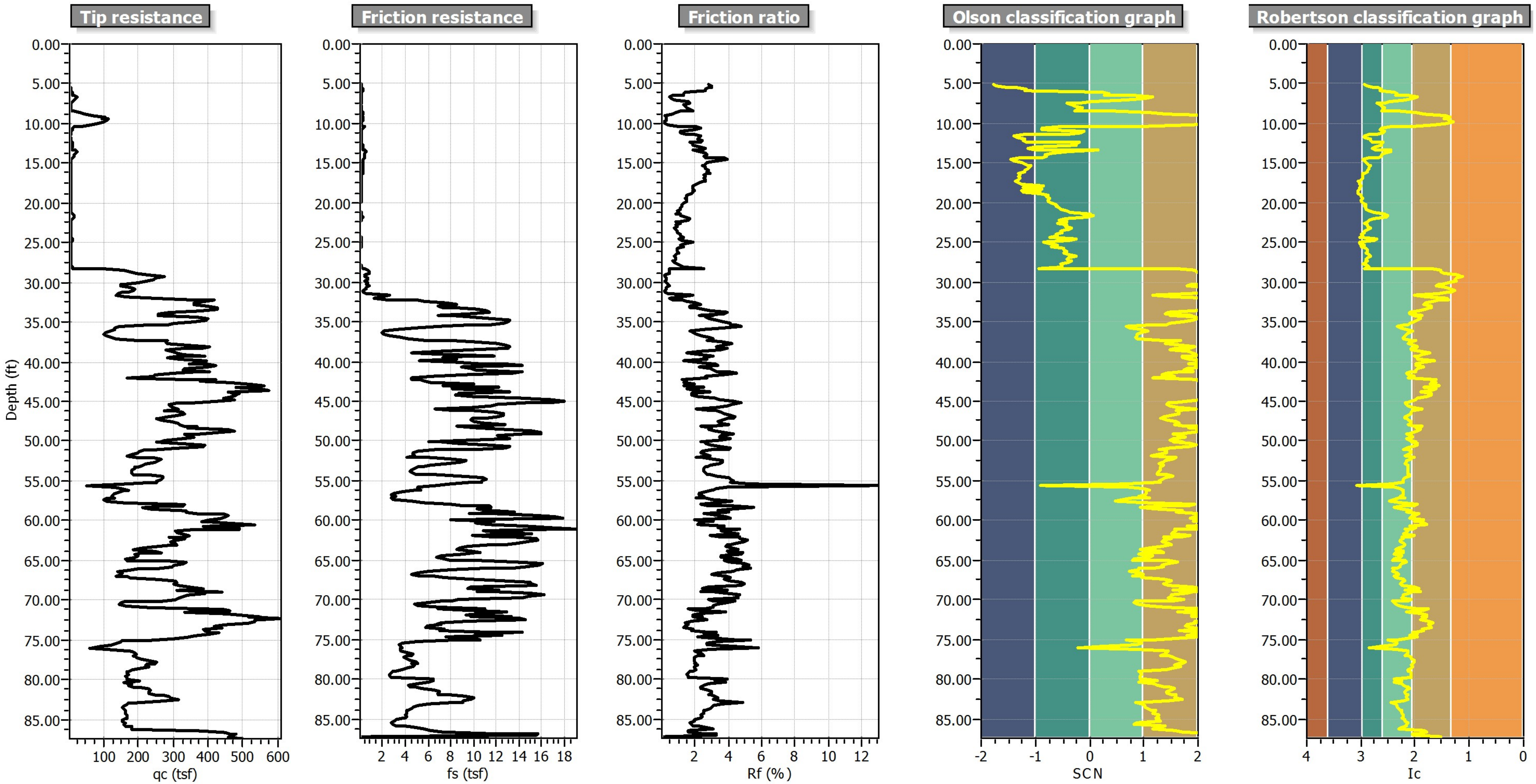












Soil type index legend

- Gravelly sand to dense sand
- Sands: clean sand to silty sand
- Sand mixtures: silty sand to sandy silt
- Silt mixtures: clayey silt to silty clay
- Clays: silty clay to clay
- Organic soils: peats

GeoLogismiki
Geotechnical Engineering Software
Merarhias 56, 621 25 - Serrai, Greece
url: http://www.geologismiki.gr - email: info@geologismiki.gr

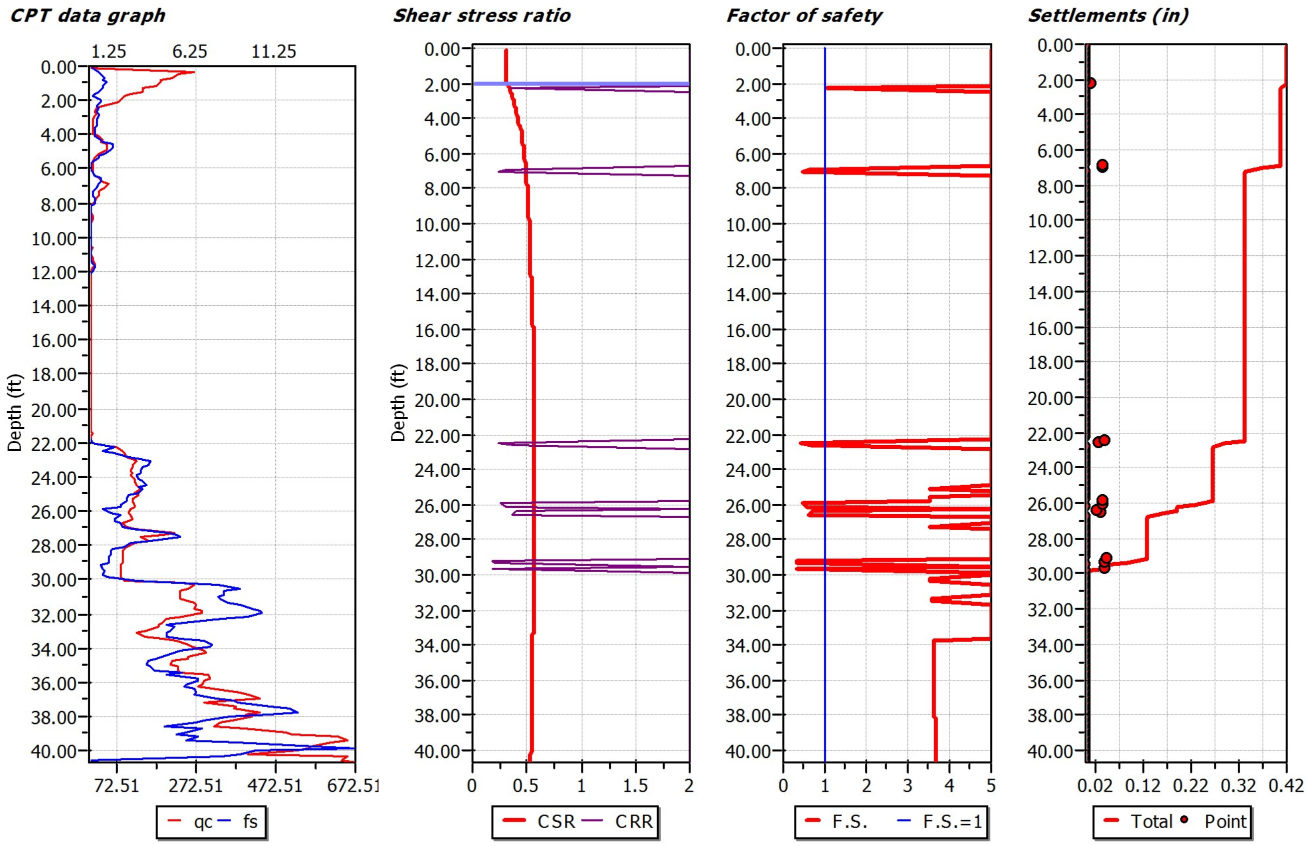
LIQUEFACTION ANALYSIS REPORT

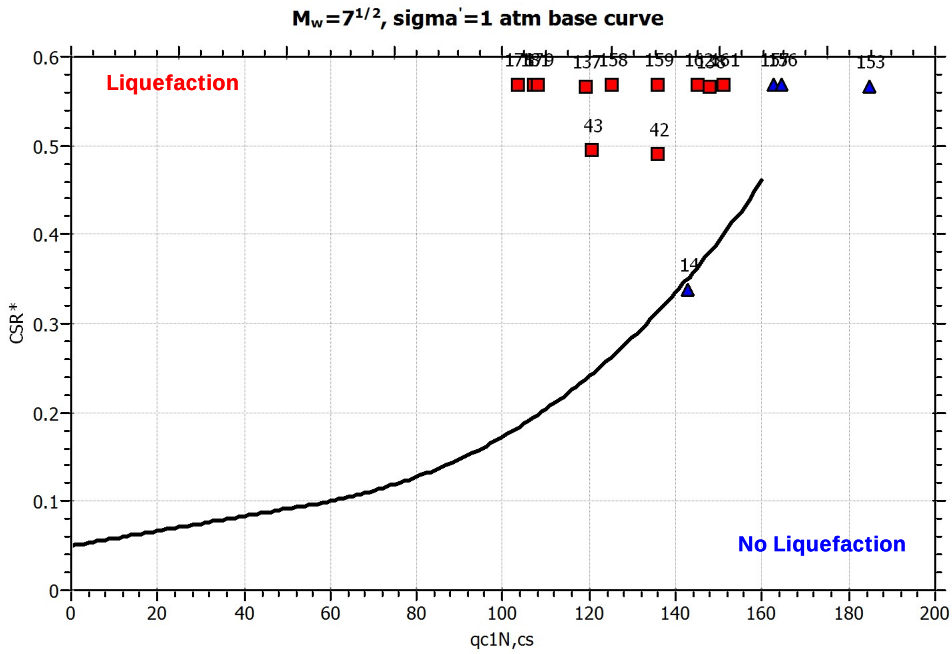
Project title : 10410.01 - Petaluma Blvd. North

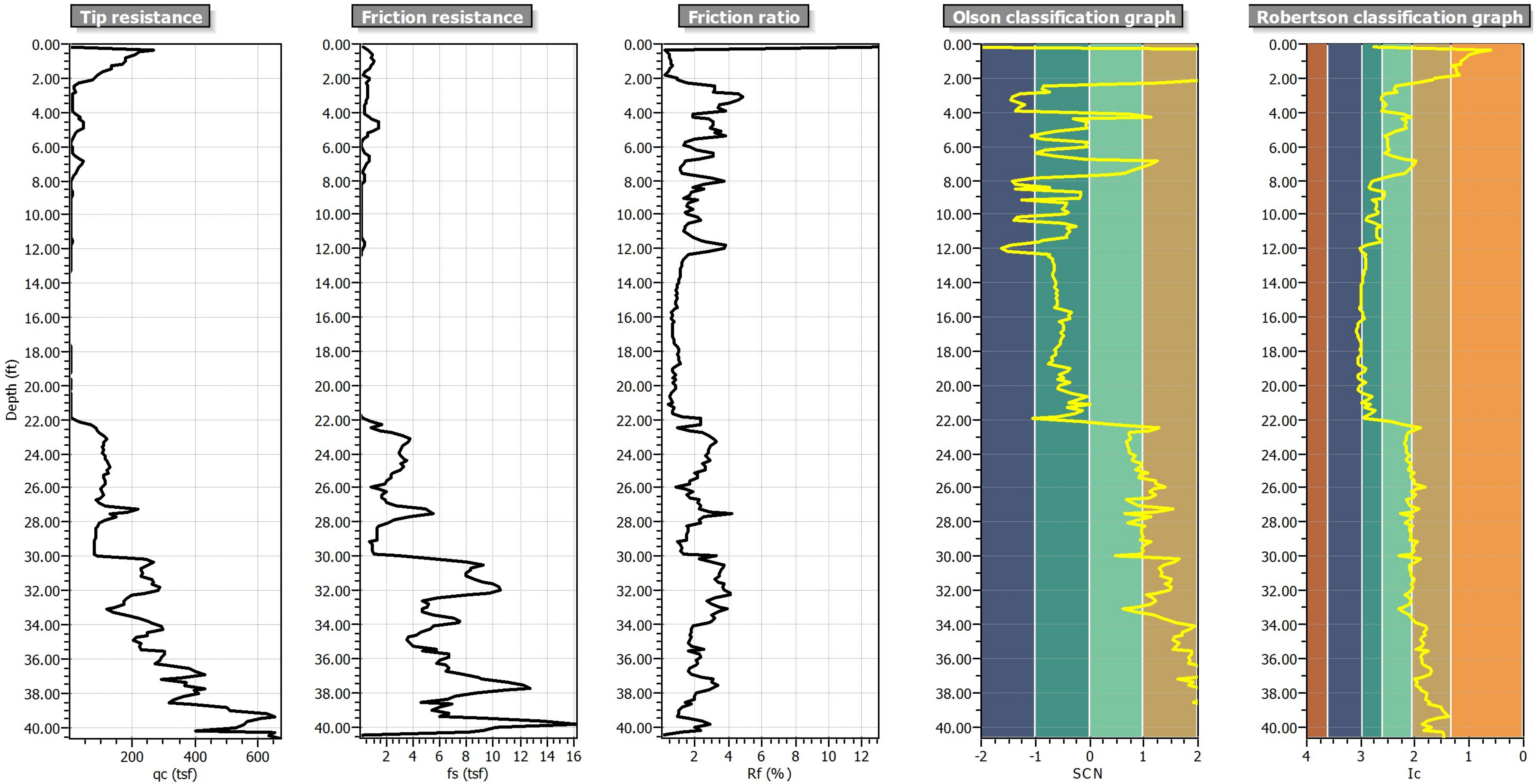
Project subtitle : CPT-2

Input parameters and analysis data

In-situ data type:	Cone Penetration Test	Depth to water table:	2.00 ft
Analysis type:	Deterministic	Earthquake magnitude M_w:	7.05
Analysis method:	Robertson (1998)	Peak ground accelaration:	0.57 g
Fines correction method:	Robertson (1998)	User defined F.S.:	1.00







Soil type index legend

- Gravelly sand to dense sand
- Sands: clean sand to silty sand
- Sand mixtures: silty sand to sandy silt
- Silt mixtures: clayey silt to silty clay
- Clays: silty clay to clay
- Organic soils: peats

GeoLogismiki
Geotechnical Engineering Software
Merarhias 56, 621 25 - Serrai, Greece
url: http://www.geologismiki.gr - email: info@geologismiki.gr

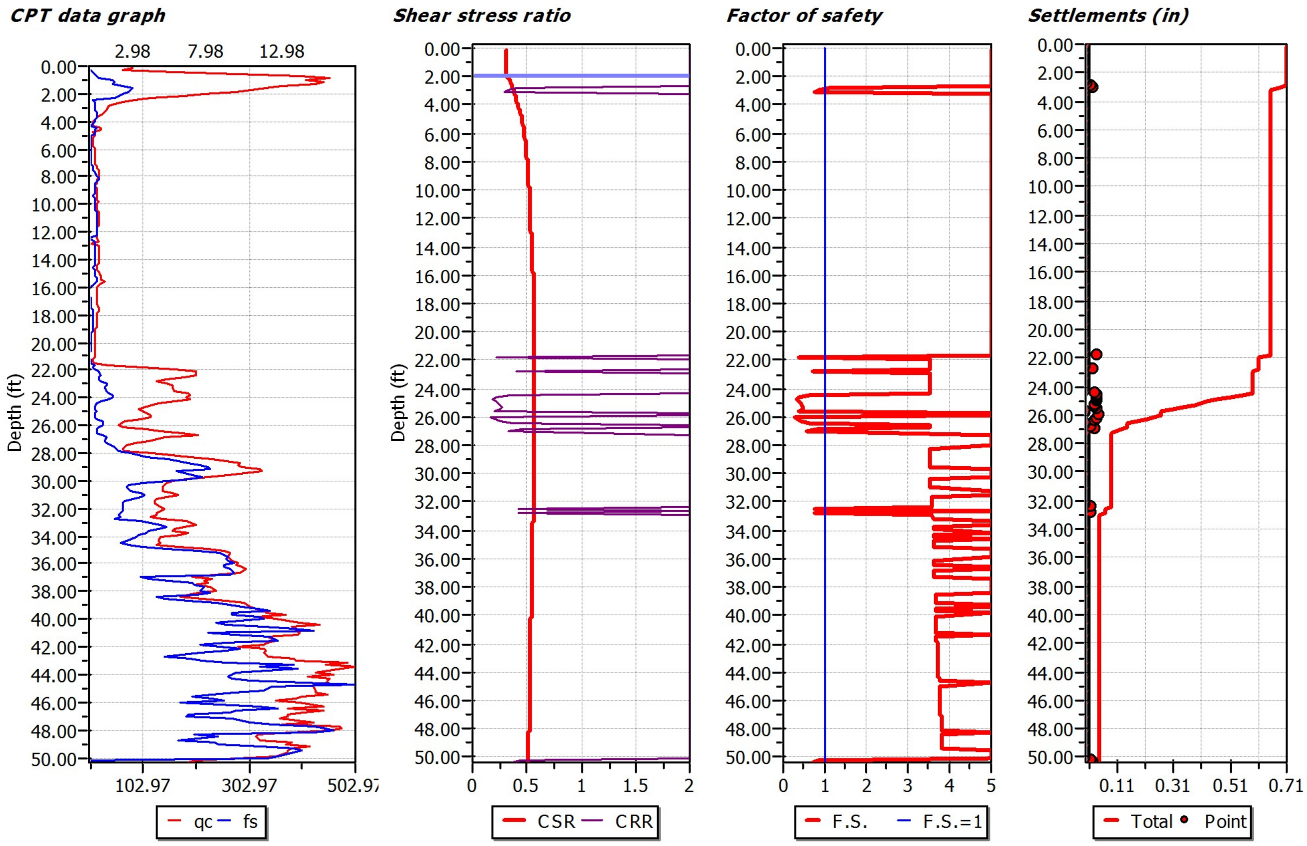
LIQUEFACTION ANALYSIS REPORT

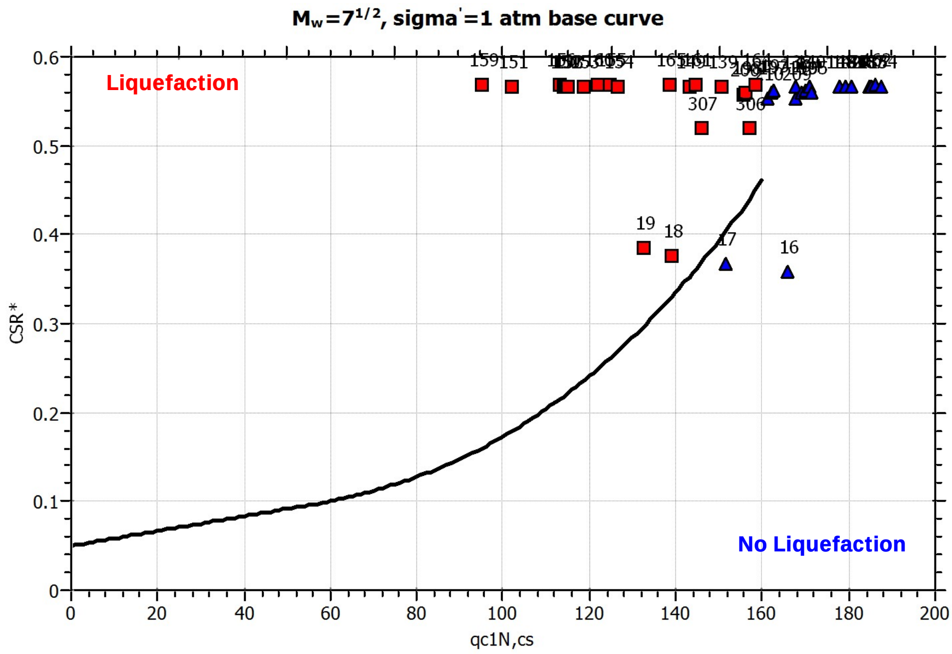
Project title : 10410.01 - Petaluma Blvd. North

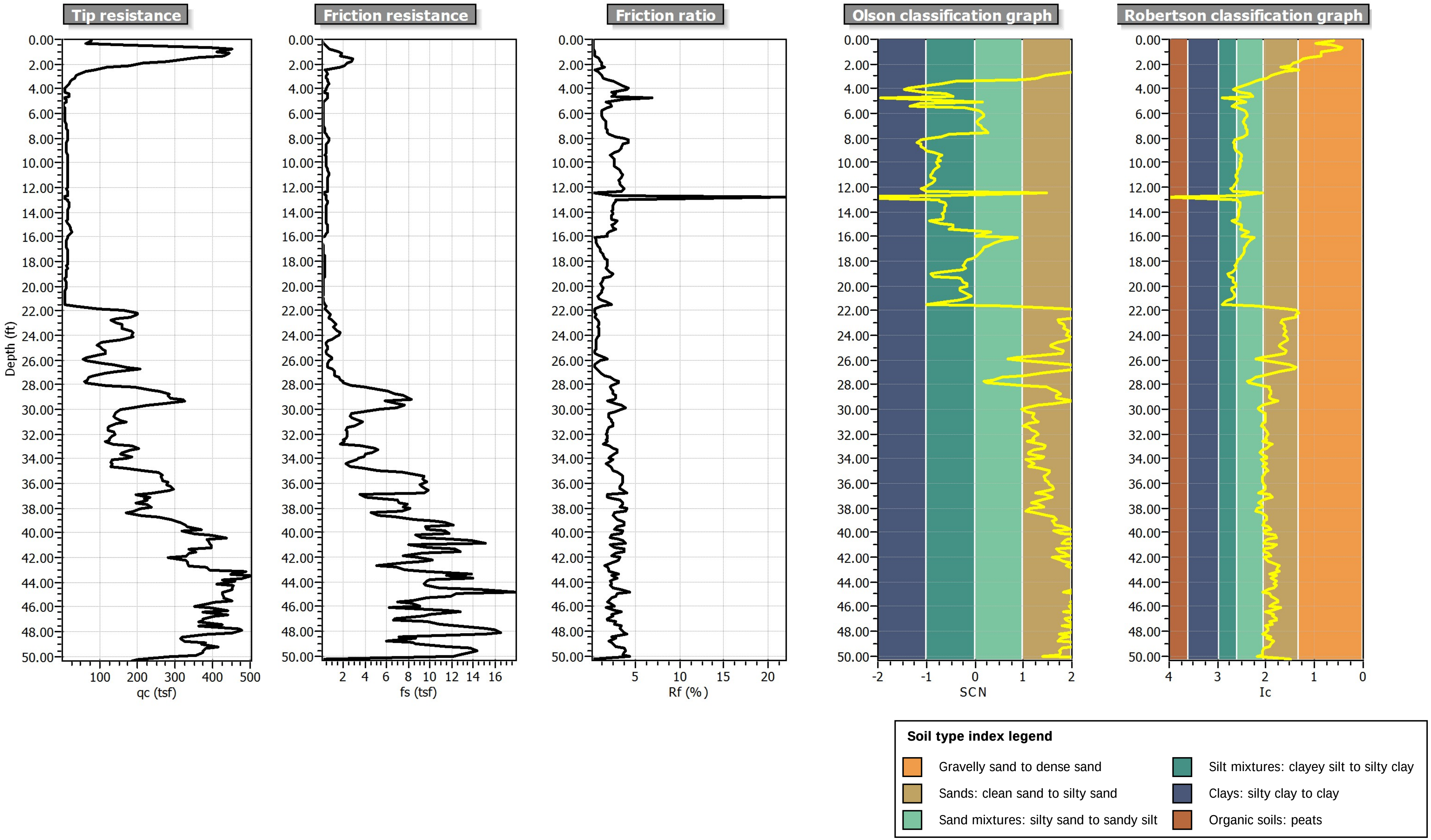
Project subtitle : CPT-3

Input parameters and analysis data

In-situ data type:	Cone Penetration Test	Depth to water table:	2.00 ft
Analysis type:	Deterministic	Earthquake magnitude M_w:	7.05
Analysis method:	Robertson (1998)	Peak ground accelaration:	0.57 g
Fines correction method:	Robertson (1998)	User defined F.S.:	1.00






Tip resistance
Friction resistance
Friction ratio
Olson classification graph
Robertson classification graph
Depth (ft)
qc (tsf)
fs (tsf)
Rf (%)
SCN
Ic
Soil type index legend
Gravelly sand to dense sand
Sands: clean sand to silty sand
Sand mixtures: silty sand to sandy silt
Silt mixtures: clayey silt to silty clay
Clays: silty clay to clay
Organic soils: peats

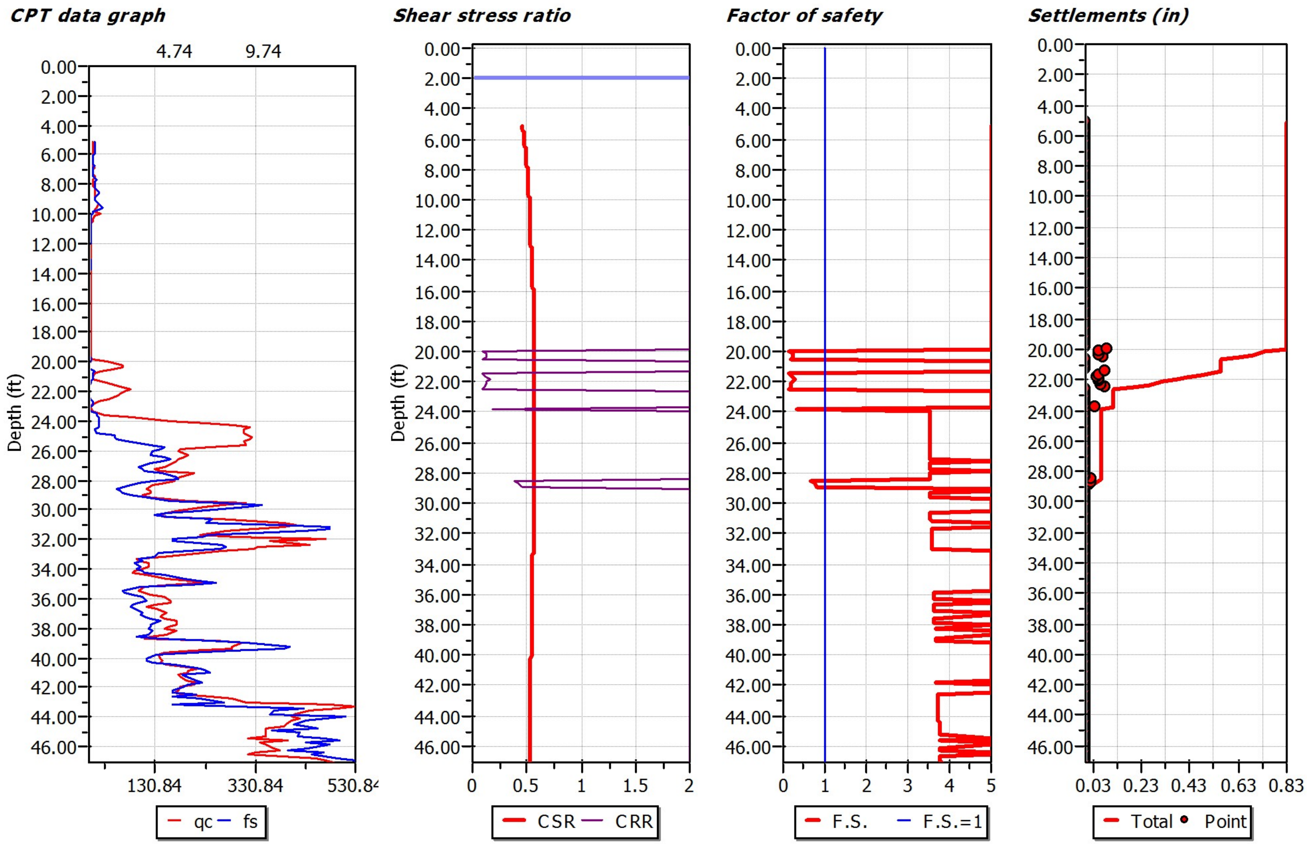
LIQUEFACTION ANALYSIS REPORT

Project title : 10410.01 - Petaluma Blvd. North

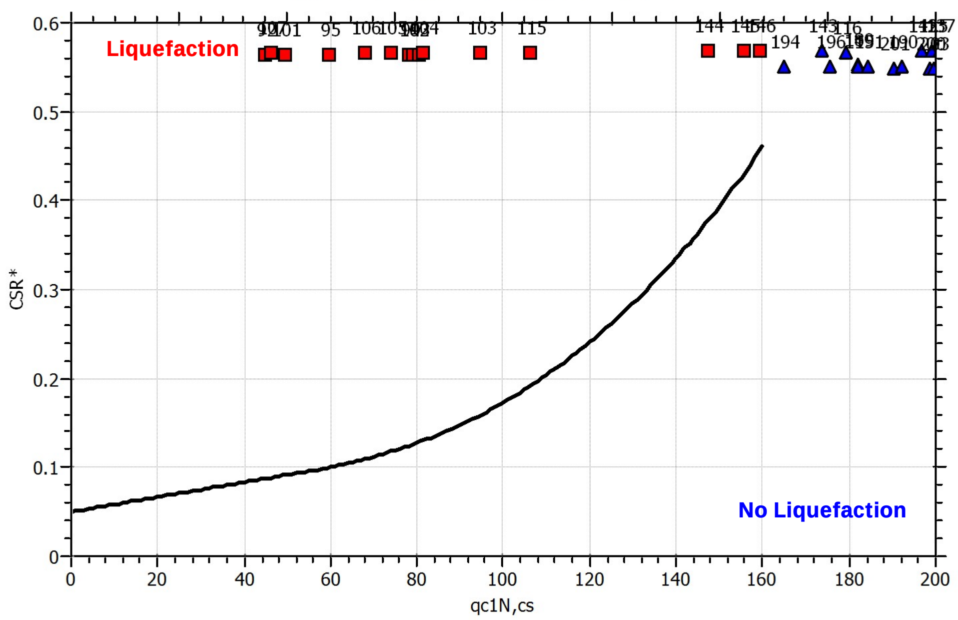
Project subtitle : CPT-4

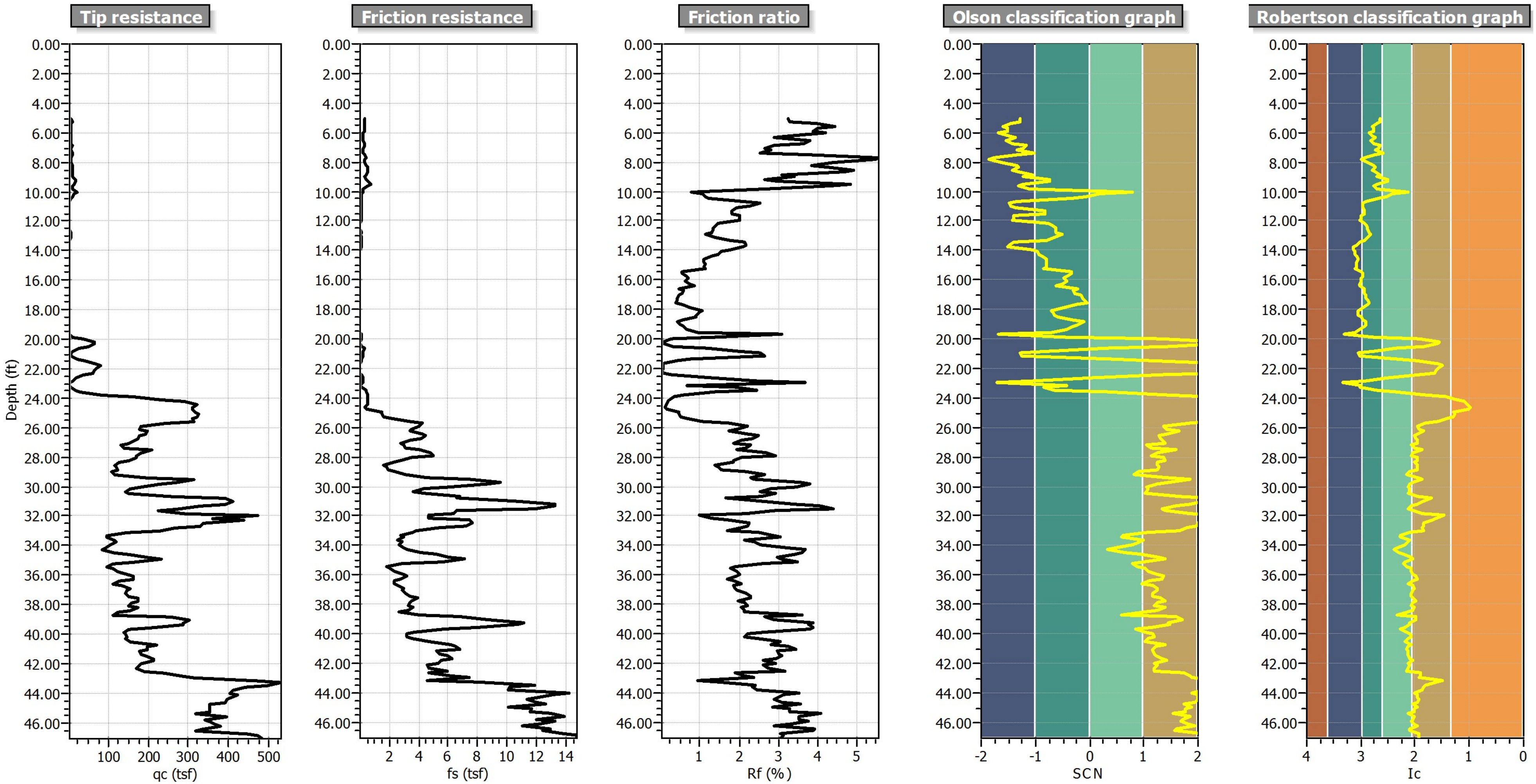
Input parameters and analysis data

In-situ data type:	Cone Penetration Test	Depth to water table:	2.00 ft
Analysis type:	Deterministic	Earthquake magnitude M_w:	7.05
Analysis method:	Robertson (1998)	Peak ground accelaration:	0.57 g
Fines correction method:	Robertson (1998)	User defined F.S.:	1.00



$M_w=7^{1/2}$, sigma'=1 atm base curve





Soil type index legend

- Gravelly sand to dense sand
- Sands: clean sand to silty sand
- Sand mixtures: silty sand to sandy silt
- Silt mixtures: clayey silt to silty clay
- Clays: silty clay to clay
- Organic soils: peats

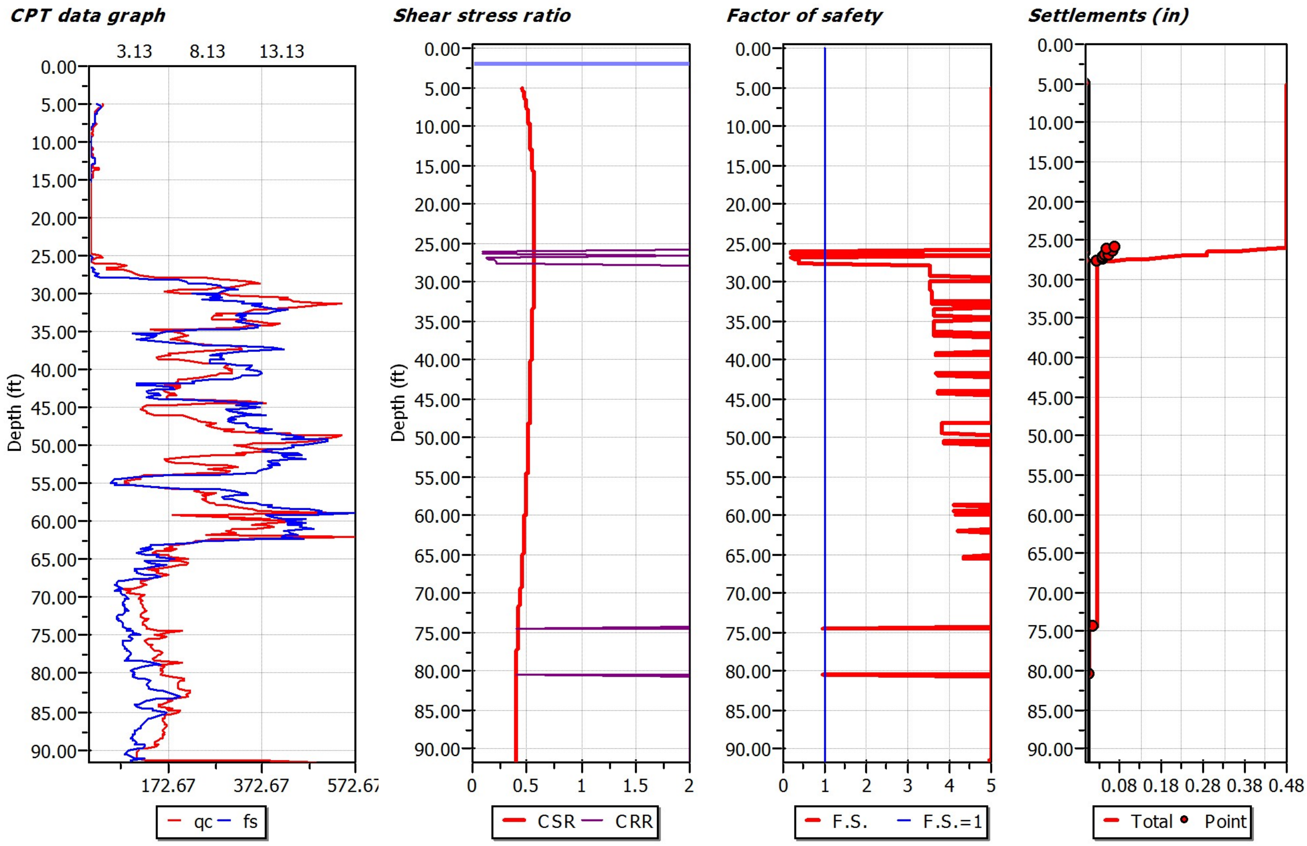
LIQUEFACTION ANALYSIS REPORT

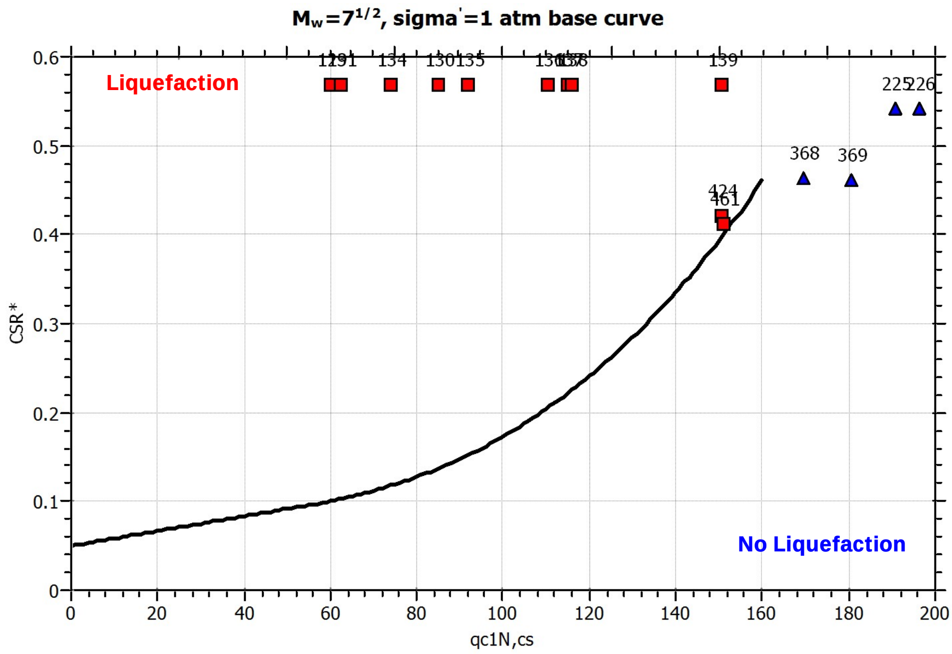
Project title : 10410.01 - Petaluma Blvd. North

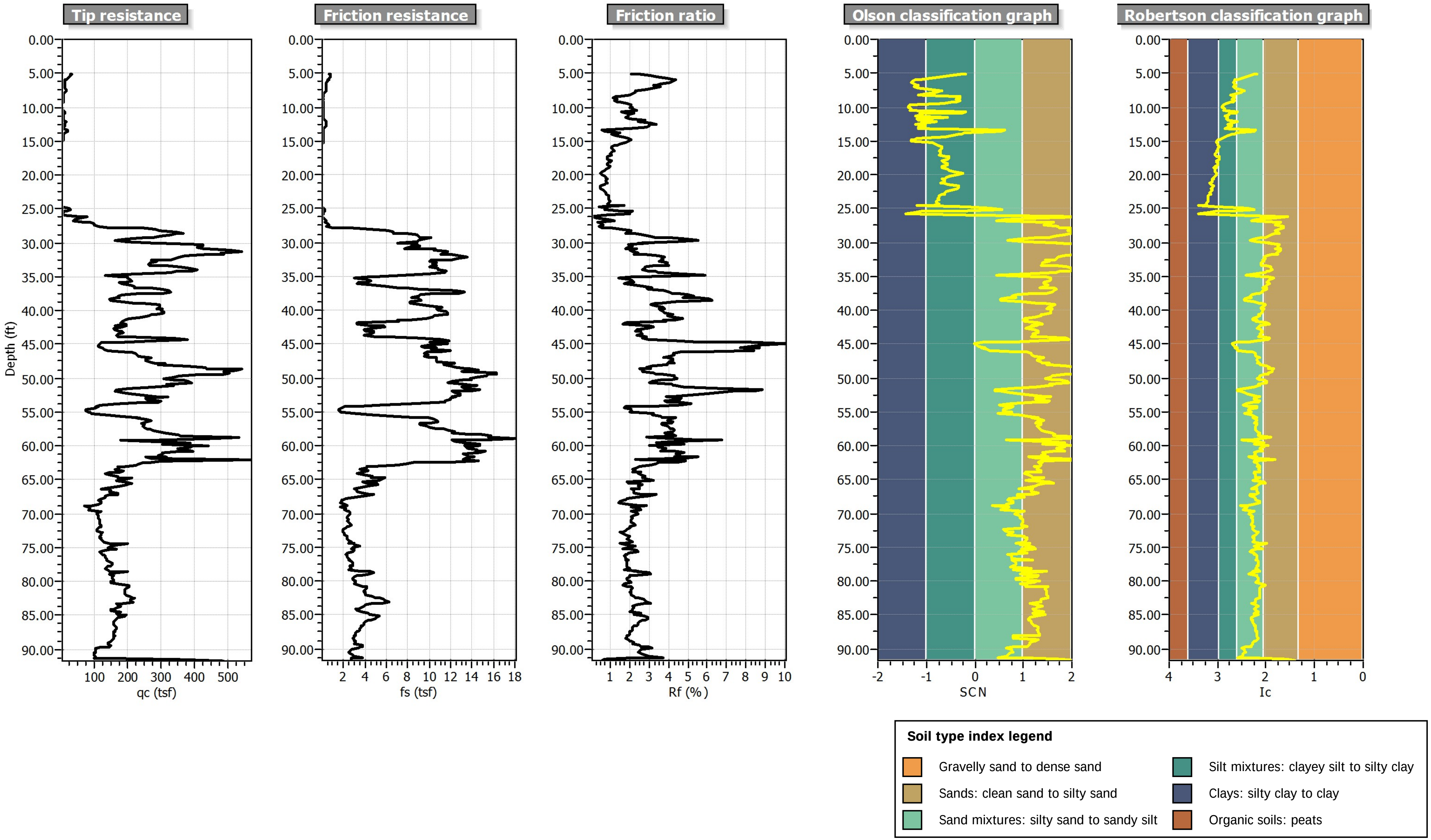
Project subtitle : CPT-5

Input parameters and analysis data

In-situ data type:	Cone Penetration Test	Depth to water table:	2.00 ft
Analysis type:	Deterministic	Earthquake magnitude M_w:	7.05
Analysis method:	Robertson (1998)	Peak ground accelaration:	0.57 g
Fines correction method:	Robertson (1998)	User defined F.S.:	1.00







Tip resistance
Friction resistance
Friction ratio
Olson classification graph
Robertson classification graph
Depth (ft)
qc (tsf)
fs (tsf)
Rf (%)
SCN
Ic
Soil type index legend
Gravelly sand to dense sand
Sands: clean sand to silty sand
Sand mixtures: silty sand to sandy silt
Silt mixtures: clayey silt to silty clay
Clays: silty clay to clay
Organic soils: peats

GeoLogismiki
Geotechnical Engineering Software
Merarhias 56, 621 25 - Serrai, Greece
url: http://www.geologismiki.gr - email: info@geologismiki.gr

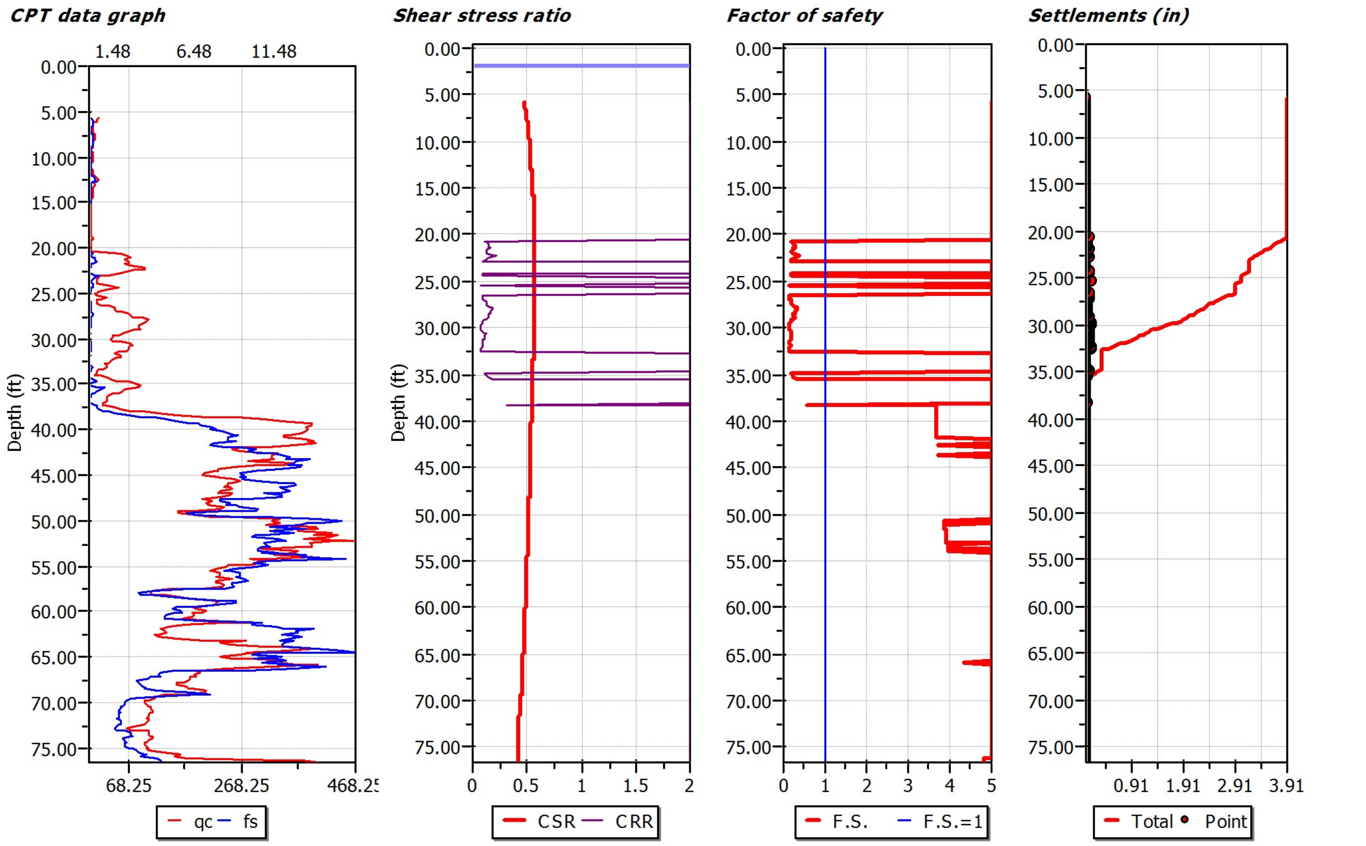
LIQUEFACTION ANALYSIS REPORT

Project title : 10410.01 - Petaluma Blvd. North

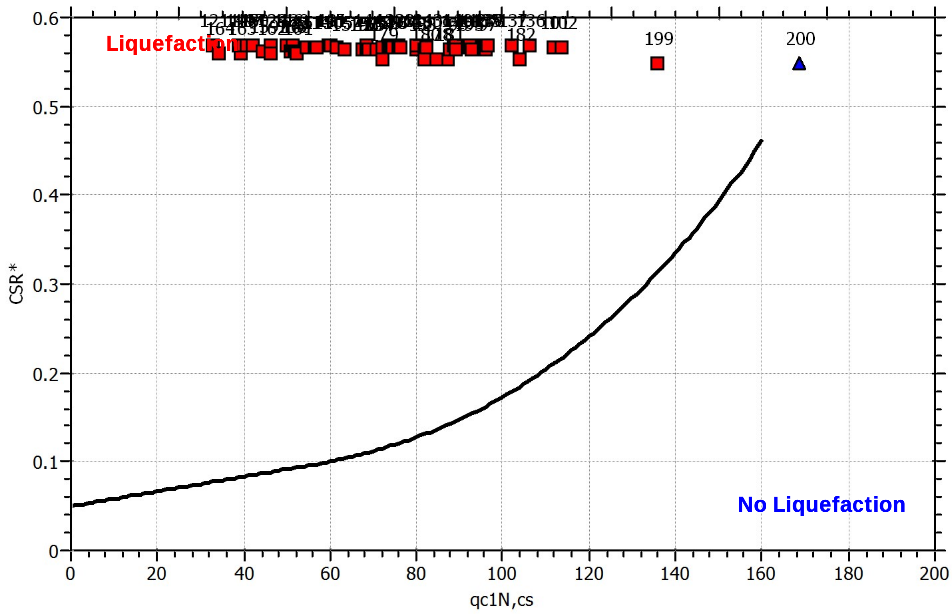
Project subtitle : CPT-6

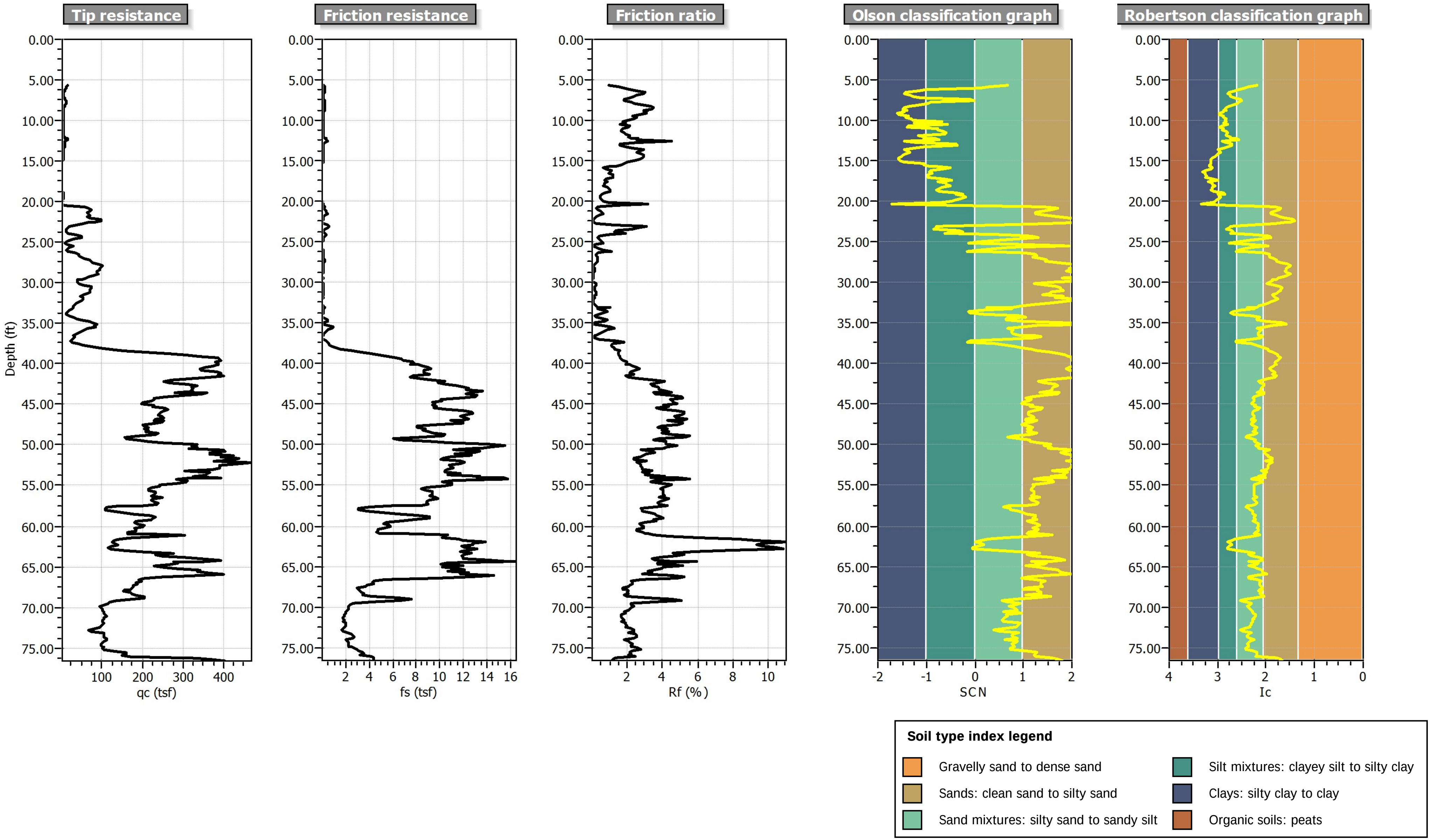
Input parameters and analysis data

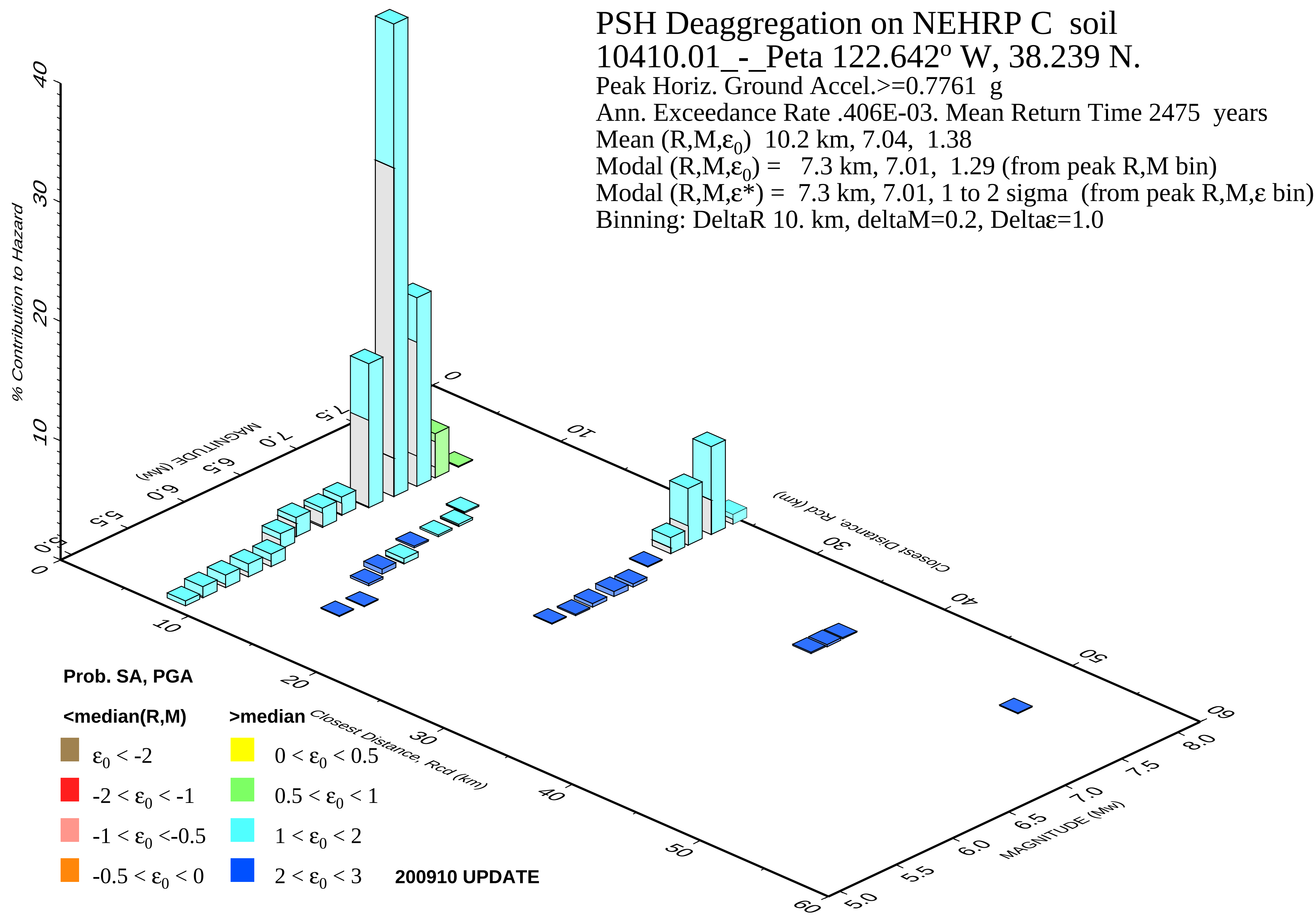
In-situ data type:	Cone Penetration Test	Depth to water table:	2.00 ft
Analysis type:	Deterministic	Earthquake magnitude M_w:	7.05
Analysis method:	Robertson (1998)	Peak ground accelaration:	0.57 g
Fines correction method:	Robertson (1998)	User defined F.S.:	1.00



$M_w=7^{1/2}$, sigma'=1 atm base curve




Tip resistance
Friction resistance
Friction ratio
Olson classification graph
Robertson classification graph
Depth (ft)
0.00
5.00
10.00
15.00
20.00
25.00
30.00
35.00
40.00
45.00
50.00
55.00
60.00
65.00
70.00
75.00
100 200 300 400
qc (tsf)
2 4 6 8 10 12 14 16
fs (tsf)
2 4 6 8 10
Rf (%)
-2 -1 0 1 2
SCN
4 3 2 1 0
Ic
Soil type index legend
Gravelly sand to dense sand
Sands: clean sand to silty sand
Sand mixtures: silty sand to sandy silt
Silt mixtures: clayey silt to silty clay
Clays: silty clay to clay
Organic soils: peats



GMT 2015 Mar 23 19:59:02 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 395. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

APPENDIX D
Guide Earthwork Specifications

APPENDIX D

GUIDE EARTHWORK SPECIFICATIONS

368 & 402 PETALUMA BOULEVARD NORTH

Petaluma, California

WKA No. 10410.02

PART 1: GENERAL

1.1 SCOPE

a. General Description

This item shall include all clearing of existing surface and subsurface structures, utilities, vegetation, rubbish, rubble, stockpiles and associated items; preparation of surfaces to be filled, filling, spreading, compaction, observation and testing of the fill; and all subsidiary work necessary to complete the grading of the site to conform with the lines, grades and slopes as shown on the accepted Drawings.

b. Related Work Specified Elsewhere

(1) Trenching and backfilling for sanitary sewer system: Section ___.

(2) Trenching and backfilling for storm drain system: Section ___.

(3) Trenching and backfilling for underground water, natural gas, and electric supplies: Section ___.

c. Geotechnical Engineer

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

1.2 PROTECTION

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

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

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

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

e. Measures shall be taken to protect storm drains in adjacent depressed areas such that minimum siltation occurs in the drainage system.

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

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

1.3 GEOTECHNICAL REPORT

a. A Geotechnical Engineering Report (WKA No. 10410.02, dated March 27, 2015 and revised July 8, 2015) has been prepared for this site by Wallace - Kuhl & Associates, Geotechnical Engineers of West Sacramento, California [(916) 372-1434]. A copy is available for review at the office of Wallace - Kuhl & Associates.

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

1.4 EXISTING SITE CONDITIONS

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

1.5 SEASONAL LIMITS

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

PART 2: PRODUCTS

2.1 MATERIALS

a. All fill shall be of approved local materials from required excavations, supplemented by imported fill, if necessary. Approved local materials are defined as local soils that do not contain significant quantities of rubble, rubbish and vegetation, and having been tested and approved by the Geotechnical Engineer prior to use.

b. Imported fill materials shall be approved by the Geotechnical Engineer; they shall meet the above requirements; shall have a Plasticity Index not exceeding fifteen (15) when tested in accordance with ASTM D4318, an expansion index not exceeding twenty (20) when tested in accordance with ASTM D4829; and, shall be of three-inch (3") maximum particle size. Import materials also shall not contain known contaminants and be within acceptable corrosion limits, with appropriate documentation provided by the contractor.

c. Materials to be lime-treated shall be on-site clayey soils free from significant quantities of rubble, rubbish and vegetation and shall have been tested and approved by the Geotechnical Engineer.

d. Capillary barrier material under floor slabs shall be provided to the thickness shown on the Drawings. This material shall be clean gravel or crushed rock of one-inch (1") maximum size, with less than five percent (5%) material passing a Number Four (#4) sieve.

e. Lime used for stabilization shall be high-calcium or dolomitic quicklime conforming to the definitions in ASTM Designation C977.

1) When sampled by the Geotechnical Engineer from the lime spreader or during the spreading operations, the sample of lime shall conform to the following requirements:

Lime Quality

Property	ASTM Designation	Requirements
Available calcium and magnesium oxide [minimum percent (%)]	C25 or C1301 & C1271	High calcium quicklime: $CaO > 90\%$ Dolomitic quicklime: $CaO > 55\%$ & $CaO + MgO > 90\%$
Loss on ignition [maximum percent (%)]	C25	7% (total loss) 5% (carbon dioxide) 2% (free moisture)
Slaking Rate [degrees Celsius (°C)]	C110	30°C rise in 8 minutes

2) When dry sieved in a mechanical sieve shaker for 10 minutes $\pm$30 seconds, a 0.5 pound (lb) test sample of quicklime shall conform to the following grading requirements:

Lime Grading

Sieve Sizes	Percentage Passing
3/8-inch	98 - 100

f. The burden of proof as to quality and suitability of alternatives shall be upon the Contractor and/or Supplier and he shall furnish test data and all information necessary, as required by the Geotechnical Engineer. Written request for alternatives, accompanied by complete data as to the quality and suitability of the material shall be made in ample time to permit testing and approval without delaying the work. The Geotechnical Engineer shall be the sole judge as to the quality and suitability of alternatives and his decision shall be final. Documentation shall be provided to the Geotechnical Engineer no later than two weeks before the alternative material is imported to the site.

g. Lime from more than one source or of more than one type may be used on the same project but the different limes shall not be mixed.

h. The lime shall be protected from moisture until used and shall be sufficiently dry to flow freely when handled.

i. Water for use in subgrade stabilization shall be clean and potable and shall be added during mixing, remixing and compaction operations, and during the curing period to keep the cured material moist until covered.

j. Other products, such as aggregate base, asphalt concrete and related asphaltic seal coats, tack coat, etc., shall comply with the appropriate provision of the State of California (Caltrans) Standard Specifications, latest edition.

PART 3: EXECUTION

3.1 LAYOUT AND PREPARATION

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

3.2 CLEARING, STRIPPING, AND PREPARING BUILDING PAD AND PAVEMENT AREAS

a. All surface and other sub-surface items associated with current site activities (including utilities) and associated backfill, vegetation, debris, and other items encountered during site work and deemed unacceptable by the Geotechnical Engineer, shall be removed and disposed of so as to leave the disturbed areas with a neat and finished appearance, free from unsightly debris. Vegetation designated for removal shall include the rootball and all surface roots larger than one-half inch (½") in diameter. Adequate removal of debris and roots may require laborers and handpicking to clean the subgrade soils to the satisfaction of the Geotechnical Engineer's on-site representative, prior to further site preparation. All demolition debris shall be hauled off site, or used as engineered fill, provided it is processed per the recommendations in Geotechnical Report.

b. On-site wells or septic systems/tanks associated with previous development, if any, should be properly abandoned in accordance with Sonoma County Department Health Services requirements.

c. Excavations and depressions resulting from the removal of such items, as determined by the Geotechnical Engineer, shall be cleaned out to firm, undisturbed soils and backfilled with suitable materials in accordance with these specifications.

d. All structural areas (building pads, pavements, exterior flatwork, etc.) shall be stripped of vegetation and organically laden topsoil. With prior approval of our office, stripping may be used in landscaped areas, provided they are kept at least five (5) from buildings pads and other surface improvements, moisture conditioned and compacted.

e. Existing concrete slabs, other concrete structures, and pavements designated for removal may be broken up, pulverized and reused as engineered fill, or removed from the site. If existing pavement rubble is reused as engineered fill, they shall be pulverized to fragments less than three inches (3”) in largest dimension and mixed with soil to for a compactable mixture.

f. Areas that currently and previously supported structures (including the railroad tracks and the entire area west of the tracks) should be sub-excavated to a depth of at least twelve inches (12”). The bottom of these excavations should be ripped and cross-ripped to a depth of at least twelve inches (12”), uniformly moisture conditioned to at least the optimum moisture content for granular/silty soils or at least two percent (2%) above the optimum moisture content for clay soils, and uniformly compacted to at least ninety percent (90%) of the maximum dry density as determined by ASTM D1557 Test Method.

g. Sub-excavation to remove clay soils from structural and slab-on-grade areas within the eastern portion of the site shall be performed as recommended in the Geotechnical Engineering Report, unless on-site clay subgrade soils are lime-treated as recommended in the Geotechnical Engineering Report.

h. The bottom of the required excavations within the eastern portion of the site, as well as areas to receive fill, achieved by excavation or remain at grade, should be scarified eight inches (8”), uniformly moisture conditioned to at least two percent (2%) above the optimum moisture content for clay soils, and uniformly compacted to at least ninety percent (90%) of the maximum dry density as determined by ASTM D1557 Test Method.

i. Compaction operations for all soil subgrades shall be undertaken with a heavy, self-propelled, sheepsfoot compactor capable of achieving the compaction requirements included in the Geotechnical Engineering Report.

j. When the moisture content of the fill material is less than the optimum moisture content for granular/silty soils or at least two percent (2%) above the optimum moisture content for clay soils as defined by the ASTM D1557 Compaction Test, water shall be added until the proper moisture content is achieved.

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

l. Compaction operations shall be performed in the presence of the Geotechnical Engineer who will evaluate the performance of the materials under compactive load. Loose, soft and saturated soils and unstable soil deposits, as determined by the Geotechnical Engineer, shall be excavated to expose a firm base and grades restored with engineered fill in accordance with these specifications.

3.3 CONSTRUCTION OF UNTREATED SUBGRADES

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

b. When the moisture content of the fill material is less than the optimum moisture content for granular/silty soils or at least two percent (2%) above the optimum moisture content for clay soils, as defined by the ASTM D1557 Compaction Test, water shall be added until the proper moisture content is achieved.

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

d. After each layer has been placed, mixed and spread evenly, it shall be thoroughly compacted to at least ninety percent (90%) as determined by the ASTM D1557 Compaction Test. Compaction shall be undertaken with equipment capable of achieving the specified density and shall be accomplished while the fill material is at the required moisture content. Each layer shall be compacted over its entire area until the desired density has been obtained.

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

3.4 <u>LIME-STABLIZED SUBGRADE CONSTRUCITON</u>

a. On-site clay material to be treated shall be placed at a moisture content at least two percent (2%) over optimum moisture as defined by the ASTM D1557 Compaction Test.

b. Material to be treated shall be scarified and thoroughly broken up to the full depth and width to be stabilized. The material to be treated shall contain no rocks or solids larger than one and one-half inches (1½") in maximum dimension.

c. Mixing lime-treated material shall consist of the following:

1) Lime shall be added to the material to be treated at a rate of no less than five pounds (5 lbs.) of lime per square foot of soil treated, to a depth sufficient to result in a twelve-inch (12”) layer of compacted lime treated soil.

2) Lime shall be spread by equipment that will uniformly distribute the required amount of lime for the full width of the prepared material. The rate of spread per linear foot of blanket shall not vary more than five percent (5%) from the designated rate.

3) The spread lime shall be prevented from blowing by suitable means selected by the Contractor. Quicklime shall not be used to make lime slurry. The spreading operations shall be conducted in such a manner that a hazard is not present to construction personnel or the public. All lime spread shall be thoroughly ripped in, or mixed into, the soil the same day lime spreading operations are performed.

4) The distance which lime may be spread upon the prepared material ahead of the mixing operation will be determined by the Geotechnical Engineer.

5) No traffic other than the mixing equipment will be allowed to pass over the spread lime until after the completion of mixing.
6) Mixing equipment shall be equipped with a visual depth indicator showing mixing depth, an odometer or foot meter to indicate travel speed and a controllable water additive system for regulating water added to the mixture.
7) Mixing equipment shall be of the type that can mix the full depth of the treatment specified and leave a relatively smooth bottom of the treated section. Mixing and re-mixing, regardless of equipment used, will continue until the material is uniformly mixed (free of streaks or pockets of lime), moisture is at approximately two percent (2%) over optimum and the mixture complies with the following requirements:

Sieve Size	Minimum Percent Passing
1-1/2"	100
1"	95
No. 4	60

8) Non-uniformity of color reaction when the treated material, exclusive of one inch or larger clods, as tested with the standard phenolphthalein alcohol indicator, will be considered evidence of inadequate mixing.
9) Lime-treated material shall not be mixed or spread while the atmospheric temperature is below 35 degrees Fahrenheit (35°F).
10) Remixing of the treated soils shall be performed no sooner than twelve (12) hours after the initial mixing, and no later than seventy-two (72) hours after the initial mixing. The entire mixing operation shall be completed within seventy-two (72) hours of the initial spreading of lime, unless otherwise permitted by the Geotechnical Engineer.

d. Spreading and compacting of lime-treated material shall consist of the following:
1) The treated mixture shall be spread to the required width, grade and cross-section. The maximum compacted thickness of a single layer may be determined by the Contractor provided he can demonstrate to the Geotechnical Engineer that his equipment and method of operation will provide uniform distribution of the lime and the required compacted density throughout the layer. If the Contractor is unable to achieve uniformity and density throughout the thickness selected, he shall rework the affected area using thinner lifts until a satisfactory treated subgrade meeting the distribution and density requirements is attained, as determined by the Geotechnical Engineer, at no additional cost to the Owner.
2) The finished thickness of the lime-treated material shall not vary more than one-tenth foot (0.1') from the planned thickness at any point.

3) The lime-treated soils shall be compacted to a relative compaction of not less than ninety percent (90%) as determined by the ASTM D1557 Compaction Test.
4) Initial compaction shall be performed by means of a sheepsfoot or segmented wheel roller. Final rolling shall be by means of steel-tired or pneumatic-tired rollers.
5) Areas inaccessible to rollers shall be compacted to meet the minimum compaction requirement by other means satisfactory to the Geotechnical Engineer.
6) Final compaction shall be completed within thirty-six (36) hours of final mixing, and within four (4) hours of the final mixing. The surface of the finished lime-treated material shall be the grading plane and at any point shall not vary more than eight one hundredths of a foot (0.08') foot above or below the grade established by the Civil Engineer except that when the lime-treated material is to be covered by material which is paid for by the cubic yard the surface of the finished lime-treated material shall not extend above the grade established by the Civil Engineer.
7) Before final compaction, if the treated material is above the grade tolerance specified in this section, uncompacted excess material may be removed and used in areas inaccessible to mixing equipment. After final compaction and trimming, excess material shall be removed and disposed of off site. The trimmed and completed surface shall be rolled with steel or pneumatic-tired rollers. Minor indentations may remain in the surface of the finished material so long as no loose material remains in the indentations.
8) At the end of each day's work, a construction joint shall be made in thoroughly compacted material and with a vertical face. After a partial-width section has been completed, the longitudinal joint against which additional material is to be placed shall be trimmed approximately three inches (3") into treated material, to the neat line of the section, with a vertical edge. The material so trimmed shall be incorporated into the adjacent material to be treated.
9) An acceptable alternate to the above construction joints, if the treatment is performed with cross shaft rotary mixers, is to actually mix three inches (3") into the previous day's work to assure a good bond to the adjacent work.

3.5 FINAL SUBGRADE PREPARATION USING UNTREATED SOILS

a. Final subgrade for building pads and exterior flatwork shall be constructed in accordance with Section 3.2 and Section 3.3 of these specifications. Clay soils should not be used in fills within the upper eighteen inches (18”) of the final concrete foundation slabs or exterior flatwork subgrade, unless the lime-treatment alternative include in the Geotechnical Engineering Report is selected. The upper eighteen inches (18") of final subgrade for the concrete foundation slabs, exterior flatwork, and Portland concrete cement pavements shall consist of granular sandy soils, be brought to a uniform moisture content not less than the optimum moisture content, and shall be uniformly compacted to not less than ninety percent (90%) as determined by ASTM D1557 Compaction Test, unless the lime-treatment alternative include in the Geotechnical Engineering Report is selected.

b. The upper six inches (6”) of any untreated final pavement subgrades shall be brought to at least the optimum moisture content for granular/silty soils or at least two percent (2%) above the optimum moisture content for clay soils, and uniformly compacted to at least ninety-five percent (95%) as determined by ASTM D1557 Compaction Test, regardless of whether final subgrade elevations are attained by filling, excavation or are left at existing grades.

3.6 FINAL SUBGRADE PREPARATION USING TREATED SOILS

a. Final subgrade for building pads and exterior flatwork using treated soils shall be constructed in accordance with Section 3.2 and Section 3.4 of these specifications. The upper twelve inches (12”) of treated final subgrades for building pads and exterior flatwork shall be brought to a uniform moisture content of at least the optimum moisture content for granular/silty soils or at least two percent (2%) above the optimum moisture content for clay soils, and shall be uniformly compacted to not less than ninety percent (90%) as determined by ASTM D1557 Compaction Test, regardless of whether final subgrade elevations are attained by filling, excavation or are left at existing grades.

b. Final subgrade for pavements using treated soils shall be constructed in accordance with Section 3.2 and Section 3.4 of these specifications. The upper twelve inches (12”) of treated final pavement subgrades shall be brought to a uniform moisture content of at least two percent (2%) above the optimum moisture content, and shall be uniformly compacted to not less than ninety-five percent (95%) as determined by ASTM D1557 Compaction Test, regardless of whether final subgrade elevations are attained by filling, excavation or are left at existing grades.

3.7 TESTING AND OBSERVATION

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

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

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

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

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APPENDIX E

Guide Specifications for Auger Cast-in-Place Piles

APPENDIX E

GUIDE SPECIFICATIONS FOR AUGER CAST-IN-PLACE PILES

368 & 402 PETALUMA BOULEVARD NORTH

Petaluma, California

WKA No. 10410.02

PART 1: GENERAL

1.1 SUMMARY

A. This Section includes construction of compression and tension auger cast piles, where shown on contract drawings and specified herein.

B. The Contractor shall furnish all labor, materials, tools, and equipment necessary for designing, furnishing, installing, inspecting and testing augered cast-in-place piles, and shall remove and dispose spoils generated by pile construction.

1.2 WORK NOT INCLUDED UNDER THIS SECTION

A. Concrete pile caps: Section _______.

B. Excavations: Section ______.

C. Shoring and bracing of earth banks: Section ______.

D. Dewatering: Section ______.

1.3 REFERENCE STANDARDS

A. Requirements, abbreviations and acronyms for reference standards are defined in Section _____.

B. American Concrete Institute (ACI)

1. ACI 305 - Hot Weather Concreting.
2. ACI 306 - Cold Weather Concreting.
3. ACI 315 - Details and Detailing of Concrete Reinforcement.

C. American Society for Testing and Materials (ASTM) latest editions

1. ASTM A 615 - Deformed and Plain Billet-Steel Bars for Concrete Reinforcement.
2. ASTM C 33 - Concrete Aggregates.
3. ASTM C 31 - Standard Practice for Making and Curing Concrete Test Specimens in the Field
4. ASTM C 109 - Test Method for Compressive Strength of Hydraulic Cement Mortars.
5. ASTM C 150 - Portland Cement.
6. ASTM C 618 - Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete.
7. ASTM C 939 - Test Method for Flow of Grout for Preplaced - Aggregate Concrete (Flow Cone Method)

8. ASTM C 942 - Test Method for Compressive Strength of Grouts for Preplaced-Aggregate Concrete in the Laboratory.
9. ASTM D 1143 - Test Method for Piles Under Static Axial Compressive Load.
10. ASTM D 3689 - Test Method for Individual Piles Under Static Axial Tensile Load.
11. ASTM D 3966 - Test Method for Piles Under Lateral Loads.

1.4 PROTECTION

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

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

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

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

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

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

1.5 EXISTING SITE CONDITIONS

Piling Contractor shall inspect the site and related conditions prior to commencing their portion of the work. If unshown active utilities are encountered during the work, the Architect shall be promptly notified for instructions. Failure to notify will make the Contractor liable for damage to these utilities arising from Contractor’s operations subsequent to the discovery of such unshown utilities.

1.6 GEOTECHNICAL ENGINEERING REPORT

A. A *Geotechnical Engineering Report* (WKA No. 10410.02, dated March 27, 2015 and revised July 8, 2015), has been prepared by Wallace - Kuhl & Associates, Geotechnical Engineers of West Sacramento, California; telephone (916) 372-1434; facsimile (916) 372-2565. That report is available for review at the office of Wallace - Kuhl & Associates.

B. The Piling Contractor shall submit in writing to the Architect and/or Structural Engineer, all applicable information as listed in Subsection 1.7 - Submittals for review and approval, in addition to the above experience record.

C. The Owner does not guarantee that the information contained in the Geotechnical Engineering Report is correct nor that the conditions revealed at the actual exploration locations will be continuous over the entire site. This report was prepared for purposes of design only. Making the report available to contractors shall not be construed in any way as a waiver of this position. The Piling Contractor shall be responsible for any conclusions the Contractor may draw from this report. Should the Contractor prefer not to assume such risk, the Contractor is under obligation to employ their own experts to analyze available information and/or to make their own tests upon which to base their conclusions.

1.7 <u>SUBMITTALS</u>

Submit the following according to Conditions of the Construction Contract and Division 1 Specifications, for Owner's approval.

A. Shop Drawings: Shall clearly indicate but not be limited to:
   1. Description of the pile drilling and grouting equipment and procedures to be utilized in installations.
   2. Proposed pile grout design mix and description of materials to be used in sufficient detail to indicate their compliance with the specifications and either;
      a. Laboratory tests of trial mixes made with the proposed mix, or
      b. Laboratory tests of the proposed mix used on previous projects.
   3. A pile layout plan referenced to the structural plans including a numbering system capable of identifying each individual pile, and indicating pile cutoff elevations.
   4. A dimensioned sketch of the pile load test arrangements, including sizes of primary members, data on testing and measuring equipment including required jack and gauge calibrations, load cell and professional engineer seal certifying the adequacy of the reaction frames.
   5. Fabrication and installation schedule covering test pile installation, pile testing, and production pile installation, with excavation schedule for pile cap and finished subgrades by area.
   6. Qualifications of pile installation construction personnel, supervisor, and technician.

B. Records
   1. The Contractor shall submit a pile design report indicating construction methods and materials which will be utilized to install piles of the specified compression and tension capacity, meeting the criteria of this specification

and the Contract Drawings. The report shall be prepared and sealed by a Professional Engineer licensed in the state of California.

2. The Contractor shall provide a Technician for each pile rig responsible for observing the auger construction, grout batching, and grouting operations and preparing installation records. The Contractor's inspector shall submit an installation record for each pile not later than two (2) days after installation is completed. The report shall include but not be limited to:
   a. Project name and number
   b. Name of contractor
   c. Pile number
   d. Pile location, date and time of installation
   e. Design pile capacity, compression or tension
   f. Pile diameter
   g. Tip elevation
   h. Cut off elevation
   i. Elevation of butt
   j. Drilling elevation
   k. Rate of advancement of auger and rotation speed
   l. Quantity of grout placed as compared to the theoretical volume for each pile, in five-foot (5') depth increments, and total for pile
   m. Grout pressures
   n. Pile reinforcing steel
   o. Grout flow cone test report
   p. Any unusual occurrences observed during pile installation, and pile deviation from vertical
3. The grout quantity shall be determined by recording grout pump displacement or by other acceptable means; the pile installation record shall reveal the observed measure and quantity.
4. Load test reports shall be in accordance with the applicable ASTM Standards.
5. Grout compression test reports.

C. Hazardous Materials Notification: In the event no alternative product or material is available that does not contain asbestos, polychlorinated biphenyls (PCBs) or other hazardous materials as determined by the Owners' Authorized Representative, a "Material Safety Data Sheet" (MSDS) equivalent to OSHA Form 20 shall be submitted for that proposed product or material prior to installation.

D. Asbestos and PCB Certification: After completion of installation, but prior to Substantial Completion, Contractor shall certify in writing that products and materials installed, and processes used, do not contain asbestos or polychlorinated biphenyls (PCB), using format in Section ____/Closeout Procedures.

1.8 DELIVERY, HANDLING, STORAGE

Comply with General Conditions and Section 01600/Product Requirements.

1.9 WARRANTY

Comply with General Conditions and Section ____/Product Requirements.

PART 2: PRODUCTS

2.1 QUALITY ASSURANCE

A. The work of this section shall be performed by a company specialized in auger cast pile work with a minimum of five (5) years of documented successful experience, and shall be performed by skilled workers thoroughly experienced in the necessary crafts. Contractor shall submit evidence of successful installation of augered cast-in-place piles under similar job and subsurface conditions, including a job supervisor who shall have a minimum of three (3) years of method specific experience.

B. Work shall comply with all Municipal, State and Federal regulations regarding safety, including the requirements of the Williams-Steiger Occupational Safety and Health Act of 1970.

2.2 MATERIALS

A. Portland Cement: conforming to ASTM C 150.

B. Mineral Admixture: Mineral admixture, if used, shall be fly ash or natural pozzolan which possesses the property of combining with the lime liberated during the process of hydration of Portland cement to form compounds containing cementitious properties, conforming to ASTM C 618, Class C or Class F.

C. Fluidifier conforming to ASTM C 937, except that expansion shall not exceed 4%.

D. Water: Potable, fresh, clean and free of sewage, oil, acid, alkali, salts or organic matter.

E. Fine Aggregate: Conforming to ASTM C 33.

F. Grout Mixes:

1. The grout shall consist of Portland cement, sand and water, and may also contain a mineral admixture and approved fluidifier.

   a. The components shall be proportioned and mixed to produce a grout capable of maintaining the solids in suspension, which may be pumped without difficulty and which will penetrate and fill open voids in the adjacent soils.

   b. These materials shall be proportioned to produce a hardened grout which will achieve the design strength within twenty-eight (28) days.

   c. The design grout strength at twenty-eight (28) days for this project shall be a minimum four thousand pounds per square inch (4000 psi).
2. All materials shall be accurately measured by volume or weight as they are fed to the mixer.
   a. Time of mixing shall be not less than one minute at the site.
   b. If agitated continuously, the grout may be held in the mixer or agitator for a period not exceeding two and one-half (2½) hours at grout temperatures below seventy degrees Fahrenheit (70°F) and for a period not exceeding one hundred degrees Fahrenheit (100°F).
   c. Grout shall not be placed when its temperature exceeds one hundred degrees Fahrenheit (100°F).
3. Protect grout from physical damage or reduced strength, which could be caused by frost, freezing actions or low temperatures or from damage during high temperatures in accordance with ACI 305/306.
4. The grout shall be tested by making a minimum of six, two-inch (2") diameter by four-inch (4") tall cylinders for each day during which piles are placed.
   a. A set of six (6) cylinders shall consist of two (2) cylinders tested at seven (7) days, and two (2) cylinders tested at twenty-eight (28) days. Two (2) cylinders shall be held in reserve.
   b. Test cylinders shall be cured and tested in accordance with ASTM C 109.
   c. Cylinder specimens shall be cast and cured in accordance with ASTM C 31.
   d. Cylinder specimens may be restrained from expansion as described in ASTM C 942.
5. Test the flow of grout for each pile and batch of grout. Maintain grout fluidity between fifteen (15) and twenty-five (25) seconds through a three-quarters inch (¾") diameter grout cone.

G. Steel Reinforcing:
1. Minimum reinforcing steel assemblies are shown on the Contract Drawings. Assemblies shall be detailed and fabricated in accordance with the manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315).
2. Reinforcing shall conform to the requirements of ASTM A 615, Grade 60.
3. All reinforcing bar shall be epoxy coated, including bars installed for contractor convenience. Wire ties do not require epoxy coating.
4. Contractor shall provide labor, materials, and method for coating cut ends and repairing holidays in epoxy coating.
5. Acceptable materials and methods shall be provided to facilitate proper centering of all steel reinforcing installed.

6. Bars may be bent in place, provided epoxy coating at all bends is inspected, flaked coating is removed by wire brush, and holidays in coating are repaired.
7. A corrugated metal pipe sleeve shall be provided for each pile equal to the diameter of the auger, to define the pile butt and permit cut-off to specified elevations.

2.3 EQUIPMENT

A. Augering Equipment:
   1. The auger flighting shall be continuous from the auger head to the top of auger without gaps or other breaks.
   2. The auger flighting shall be uniform in diameter throughout its length and shall be the diameter specified for the piles less a maximum of three percent (3%). The hole through which the grout is pumped during the placement of the pile shall be located at the bottom of the auger head below the bar containing the cutting teeth.
   3. Augers over forty feet (40') in length shall contain a middle support guide.
   4. The piling leads shall be prevented from rotating by a stabilizing arm or by firmly placing the bottom of the leads into the ground or by some other acceptable means.
   5. Leads shall be marked at one-foot (1') intervals to facilitate measurement of auger penetration.
   6. Auger hoisting equipment shall be provided that will enable the auger to be rotated while being withdrawn.

B. Mixing and Pumping Equipment:
   1. Only approved pumping and mixing equipment shall be used in the preparation and handling of the grout.
      a. Provide a screen to remove over-size particles at the pump inlet.
      b. All oil or other rust inhibitor shall be removed from mixing drums and grout pumps before each use.
      c. All materials shall be such as to produce a homogeneous grout of the desired consistency and strength.
   2. The grout pump shall be a positive displacement pump capable of developing displacement pressures at the pump of three hundred fifty pounds per square inch (350 psi) or higher.
      a. The grout pump shall be provided with a pressure gauge in clear view of the equipment operator.
      b. The grout pump shall be calibrated at the beginning of the work and periodically during the work to determine the volume of grout pumped per stroke, under operating pressure.

c. A positive method for automatic counting of grout pump strokes shall be provided. Such methods may include digital or mechanical stroke counters or other acceptable methods.

d. A second pressure gauge, if required, shall be provided close to the auger rig where it can be readily observed by the inspector, if required.

PART 3: EXECUTION

3.1 EXAMINATION

A. The Contractor is responsible for supporting pile drilling equipment and concrete grout batching and delivery equipment. Equipment shall be supported on timber mats or gravel fill work platforms, if necessary for safety and stability, and to prevent damage.

B. The Contractor shall examine the areas and evaluate conditions under which piles are to be installed and shall include measures for the proper and timely completion of the work in the construction methods and pile design.

3.2 AUGER CAST PILE SYSTEM DESCRIPTION

A. Augered Pressure Grouted Piles

1. Pressure grouted piles shall be made by drilling a continuous-flight, hollow-shaft auger into the ground to the design pile depth, or until refusal criteria is satisfied. The volume of soil extracted shall not be greater than the volume of the steel auger stem inserted.

2. Grout shall be injected through the auger shaft as the auger is being withdrawn. First develop a five-foot (5') plug at the bottom of the auger flights, then inject sufficient grout volume to fill the augered hole one hundred fifteen percent of the theoretical volume (1.15 percent) or more. Grout volumes shall be logged by depth during withdrawal.

3. Post-grouting through a special grout tube for capacity increase is permitted, given these methods are used in the test piles, and consistently throughout the entire work for this project. Post-grouting may be used for compression and tension capacity. Post-grout pressures must be sufficient to open grout portals and cause fracture and flow. Grout volumes and pressures shall be recorded and used as a measure to demonstrate pile compliance with the design and pile load test criteria.

B. Augered Displacement Pressure Grouted Piles

1. Augered Displacement Pressure Grouted piles shall be made by rotating a specialized auger capable of displacing soil surrounding the auger, with

minimal soils returned to the ground surface to reach the design pile depth, or until specified refusal criteria is satisfied.

2. Grout shall be injected through the auger shaft as the auger is being withdrawn in such a way as to exert a positive upward grout pressure on the auger, as well as a positive lateral pressure on the soil surrounding the pile.

C. Alternatives

1. Alternative pile types which meet the compression and tension pile criteria given on the drawings may be substituted for augered pressure-grouted pile systems described in this Section.

2. Alternative pile installation systems must be capable of achieving the specified compression and tension, and shall provide a working lateral capacity of twenty kips (20).

3.3 PILE DESIGN

A. The ultimate capacity of twenty four inch (24”) diameter compression piles shall be greater than two hundred kips (200) in axial compression and greater than forty five kips (45) in axial tension, or the ultimate capacity of ultimate capacity of thirty six inch (36”) diameter compression piles shall be greater than three hundred twenty kips (320) in axial compression and greater than seventy five kips (75) in axial tension. Both tension and compression piles shall achieve an ultimate lateral capacity of eight kips (8) for twenty for inch (24”) diameter piles or fourteen kips (14) for thirty six inch (36”) diameter piles. The allowable design capacities of all piles shall be determined by dividing the ultimate capacity by the appropriate factor of safety as provided in the Geotechnical Engineering Report. Load Testing performed under Part 3.4 of this section shall confirm the ultimate capacity of the piles.

B. Pile design shall be performed by the Contractor and demonstrated by load test before installation of production piles. All piles shall meet the criteria specified on the Contract Drawings.

C. The design shall be described in a pile design report. This report shall indicate variances, if any, from the reinforcing steel specified or the requirements of this section, and shall demonstrate that the design meets or exceeds the specified performance in tension, compression, and bending. The Contractor shall submit design calculations for the proposed piles demonstrating compression and tensile capacity.

3.4 LOAD TESTING

A. Pre-construction Pile Load Tests:

1. Install and test one (1) compression pile, one (1) tension pile, and one (1) lateral load test pile, at the locations shown on the plans or approved

alternate location to verify the construction methods and pile capacity. Test piles and reaction piles shall be installed outside of pile cap locations.

2. The Contractor shall provide complete testing materials and equipment as required, install test and reaction piles and perform the load tests only in the presence of the Owner.
3. The pile test reaction frame shall be capable of safely sustaining two hundred kips (200) in axial compression and forty five kips (45) in axial tension (uplift) for twenty four inch (24”) diameter piles and three hundred twenty kips (320) in axial compression and seventy five kips (75) in axial tension (uplift) for thirty six inch (36”) diameter piles.
4. Preconstruction Pile Load tests shall be performed using ASTM’s Quick Test Methods.
5. One successful compression pile load test shall be performed in accordance with ASTM D 1143.
6. One successful tension pile load test shall be performed in accordance with ASTM D 3689.
7. One lateral pile load test to eight kips (8) ultimate load for twenty four inch (24”) diameter piles or fourteen kips (14) ultimate load for thirty six inch (36”) diameter piles shall be performed in accordance with ASTM D 3966.

3.5 <u>INSTALLATION</u>

A. Tolerance
   1. Piles shall be located where shown on drawings or where otherwise directed by the Engineer.
      a. Pile centers shall be located to an accuracy of three inches (±3").
      b. Vertical piles shall be plumb within two percent (2%).
      c. Battered piles shall be installed to within four percent (4%) of the specified batter as determined by the angle from horizontal.

B. Adjacent Piles
   1. Adjacent piles within ten feet (10'), center-to-center, shall not be installed within twenty-four (24) hours of each other.
   2. Within pile caps, piles adjacent within four (4) pile diameters center-to-center, shall not be installed within twenty-four (24) hours of each other.

C. Installation Procedure
   1. The length and drilling criteria of production piles will be as defined in the Contractor’s design and as demonstrated by the successful pile load tests. Advance and rotate the auger at a continuous rate that prevents removal of excess soil.
   2. Stop advancement after reaching the required depth or refusal criteria.

3. The hole in the bottom of the auger shall be closed with a suitable plug while advancing into the ground. The plug shall be removed by grout pressure or mechanically with the reinforcing bar.
4. At the start of pumping grout, raise the auger from six inches (6") to twelve inches (12") and after the grout pressure builds up sufficiently, re-drill the auger to the previously established tip elevation.
5. Maintain a head of at least fifteen feet (15') of grout on the auger flighting above the injection point during auger withdrawal.
   a. Positive rotation of the auger shall be maintained at least until placement of the grout.
   b. Rate of grout injection and rate of auger withdrawal from the soil shall be coordinated so as to maintain at all times the minimum grout head.
   c. The total volume of grout shall be at least one hundred fifteen percent (115%) of the theoretical volume for each pile.
   d. After grout is flowing at the ground surface from the auger flighting, the rate of grout injection and auger withdrawal shall be coordinated so that there is a constant grout flow at the surface.
   e. If pumping grout is interrupted for any reason, the contractor shall reinsert the auger by drilling at least five feet (5') below the depth of the auger where the interruption occurred, and re-grout while withdrawing the auger from that depth.
6. If less than one hundred fifteen percent (115%) of the theoretical volume of grout is placed in any five foot (5') increment (until the grout head on the auger flighting reaches the ground surface), the pile increment shall be reinstalled by advancing the auger ten feet (10') or to the bottom of the pile if that is less, followed by controlled removal and grout injection.
7. Spoil material that accumulates around the auger during injection of the grout shall be promptly cleared away.
8. A steel corrugated metal pipe (CMP) sleeve shall be placed at the top of each pile to a depth of one and one half feet (1½') below the pile cutoff elevation.

D. Obstructions and Damaged Piles

1. If non-augerable material is encountered above the desired tip elevation, the pile shall be completed to the depth of the non-augerable material in accordance with these Specifications. Such short piles shall be included for payment, if completed and included within the foundation. If required by the Engineer, additional adjacent piles shall be placed. Additional piles shall also be included in the total number of piles for payment.

2. Damaged piles, and piles installed outside the required installation tolerances, will not be accepted.
3. Cut off and abandon rejected piles after installation, and replace with new piles. Cutoff shall be at a sufficient depth to avoid transfer of load from the structure to the abandoned pile.
4. Piles located within ten feet (10') of existing structures shall be installed in one continuous operation. Re-stroking piles during construction due to auger obstructions or difficulty in installation of reinforcement cages will not be allowed. The structural engineer shall be consulted in the event that replacement piles are required.

E. Cutting-Off
1. Adjust the tops of pile to the cut-off elevations where piles are constructed from a work platform above final subgrade, by removing fresh grout from the top of the pile after the CMP sleeve is in place.
2. Cut off hardened grout and the CMP shell down to final cutoff point after initial set has occurred for all piles in a single cap, or within fifteen feet (15') of any pile in a spaced pattern.

F. Disposal
1. The Contractor shall remove and dispose all spoils and grout off site.
2. The Contractor shall determine if any excavated material is contaminated, and if any contaminated material is encountered it shall be disposed of in a method acceptable to all governmental authorities having jurisdiction.

PART 4: MEASUREMENT AND PAYMENT

4.1 MEASUREMENT

A. Each compression pile and each tension pile successfully installed in accordance with the Contractor’s design and using the methods and practices of the approved test piles, cut off at the proper elevation, including steel reinforcing, and all records and grout testing specified, shall be considered a single unit price item. Pile design, materials testing, and the Contractor’s inspection are considered incidental to construction and shall not be separately measured for payment. Damaged piles and piles installed outside the required installation tolerances will not be measured for payment. Short piles caused by obstructions and meeting the requirements of Part 3.5D shall be measured for payment.

B. Each successful compression, tension and lateral pre-construction load test performed, including load frame and/or reaction piles, test pile, testing, and load test report, shall be considered a single unit price item.

C. Each successful compression, tension and lateral construction quick load test performed, including load frame and/or reaction piles, test pile, testing, and load test report, shall be considered a single unit price item.

4.2 PAYMENT

A. Each compression pile and each tension pile, approved and accepted by the Owner, shall be paid at the unit price indicated on the bid form.

B. Each successful pile load test, approved and accepted by the Owner, shall be paid at the unit prices indicated on the bid form.

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