PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED CoOp RESIDENTIAL/COMMERCIAL PROJECT
890 PETALUMA BOULEVARD NORTH
PETALUMA, CALIFORNIA

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Job No. 3032.001

Prepared for:
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CERTIFICATION

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1.0 INTRODUCTION

This report presents the results of our Preliminary Geotechnical Investigation for the proposed CoOp residential/commercial development at 890 Petaluma Boulevard North in Petaluma, California. As shown on Figure 1, the project site is located west of Highway 101 at 890 Petaluma Boulevard North. The site is located in the southeasterly quadrant of the intersection of Petaluma Boulevard North and West Payran Street in Petaluma.

Our work was performed in accordance with our Agreement for Professional Services dated June 30, 2020. The purpose of our investigation was to explore subsurface conditions and to develop preliminary geotechnical criteria for design and construction of the proposed improvements. The scope of our services includes:

- Review of available, published geologic mapping and geotechnical background information from our files, the City of Petaluma files, and any geologic/geotechnical background information supplied by you.
- Coordinate with Underground Service Alert (USA) to mark underground utilities in areas where we plan to conduct subsurface exploration.
- Subsurface exploration consisting of one day of Cone Penetration Tests (CPTs). We completed four CPTs that extended through the near-surface soils to a depth of about 50 feet below the ground surface or to refusal in firm soil or bedrock.
- Evaluation of relevant geologic hazards including seismic shaking, liquefaction, settlement, and other hazards.
- Preparing preliminary geotechnical recommendations and design criteria related to building foundations, site grading, temporary shoring, seismic design, and other geotechnical-related items.
- Preparing a Preliminary Geotechnical Investigation report which summarizes the referenced subsurface exploration, evaluation of relevant geologic hazards, and preliminary geotechnical recommendations and design criteria.

This report completes our Phase 1 scope of services for the project. Subsequent phases of work include supplemental subsurface exploration and laboratory testing as part of a design level investigation, design consultation/geotechnical plan review, and observation and testing of geotechnical-related work items during construction.

2.0 PROJECT DESCRIPTION

We understand that project details are not yet fully developed. However, the project generally would consist of developing the property as a mixed-use CoOp residential/commercial development. We anticipate that the new buildings would be three-story, wood frame structures with concrete slab-on-grade floors at the ground floor levels. No significant below-grade structures are currently anticipated except for underground utilities and stormwater infiltration or detention structures. Ancillary improvements are expected to include exterior hardscape/flatwork and asphalt paving, new underground utilities, new site drainage, landscaping, lighting, and other improvements "typical" of such developments. No detailed structural information is available at this time. However, we understand the structures will likely impose moderate foundation loads. A site plan showing the existing site conditions is presented on Figure 2A, and a site plan showing the proposed improvements and the approximate CPT locations is presented on Figure 2B.

3.0 SITE CONDITIONS

3.1 Regional Geology

The project site is located within the Coast Ranges geomorphic province of California. It is typified by generally northwest-trending ridges and intervening valleys that formed as a result of movement along a group of northwest-trending fault systems, including the San Andreas Fault. Bedrock geology within the San Francisco bay area is dominated by sedimentary, igneous, and metamorphic rocks of the Jurassic-Cretaceous age Franciscan Complex. Most of Franciscan rock types are composed of sandstone and pervasively sheared shale. It also includes less common rocks such as chert, serpentinite, basalt, greenstone, and exotic low- to high-grade metamorphic rocks, including phyllite, schist, and eclogite.

Regional geologic mapping (Bezore, et al, 2002) indicates that the project site is underlain by Fan Deposits (Holocene). This unit generally consists of Holocene alluvial fan sediments deposited by streams emanating from the mountains as debris flows. Sediments include moderately to poorly sorted sand, gravel, silt, and clay. A regional geologic map is presented on Figure 3.

3.2 Seismicity

The project site is located within the seismically active San Francisco Bay Area and will therefore experience the effects of future earthquakes. Earthquakes are the product of the build-up and sudden release of strain along a "fault" or zone of weakness in the earth's crust. Stored energy may be released as soon as it is generated, or it may be accumulated and stored for long periods of time. Individual releases may be so small that they are detected only by sensitive instruments, or they may be violent enough to cause destruction over vast areas.

Faults are seldom single cracks in the earth's crust but are typically composed of localized shear zones which link together to form larger fault zones. Within the Bay Area, faults are concentrated along the San Andreas Fault zone. The movement between rock formations along either side of a fault may be horizontal, vertical, or a combination, and is radiated outward in the form of energy waves. The amplitude and frequency of earthquake ground motions partially depends on the material through which it is moving. The earthquake force is transmitted through hard rock in short, rapid vibrations, while this energy becomes a long, high-amplitude motion when moving through soft ground materials, such as Bay Mud.

3.2.1 Regional Active Faults

An "active" fault is one that shows displacement within the last 11,000 years (i.e., Holocene) and has a reported average slip rate greater than 0.1 mm per year. The California Division of Mines and Geology has mapped various active and inactive faults in the region. These faults are shown in relation to the project site on the attached Active Fault Map, Figure 4. The nearest known active faults are the Rodgers Creek, San Andreas, and Maacama Faults which are located roughly 8.1 kilometers northeast, 24.2 kilometers southwest, and 31.9 kilometers north of the site, respectively.

3.2.2 Historic Fault Activity

Numerous earthquakes have occurred in the region within historic times. Earthquakes (magnitude 2.0 and greater) that have occurred in the San Francisco Bay Area since 1985 have been plotted on a map shown on Figure 5.

3.2.3 Probability of Future Earthquakes

The site will likely experience moderate to strong ground shaking from future earthquakes originating on any of several active faults in the San Francisco Bay region. The historical records do not directly indicate either the maximum credible earthquake or the probability of such a future event. To evaluate earthquake probabilities in California, the USGS has assembled a group of researchers into the "Working Group on California Earthquake Probabilities" (USGS 2003, 2008, 2013) to estimate the probabilities of earthquakes on active faults. These studies have been published cooperatively by the USGS, CGS, and Southern California Earthquake Center (SCEC) as the Uniform California Earthquake Rupture Forecast, Versions 1, 2, and 3. In these studies, potential seismic sources were analyzed considering fault geometry, geologic slip rates, geodetic strain rates, historic activity, micro-seismicity, and other factors to arrive at estimates of earthquakes of various magnitudes on a variety of faults in California.

Conclusions from the most recent UCERF3 and USGS indicate the highest probability of an earthquake with a magnitude greater than 6.7 originating on any of the active faults in the San Francisco Bay region by 2043 is assigned to the Hayward/Rodgers Creek Fault system. The Rodgers Creek Fault is located approximately 8.1 kilometers (5.0 miles) northeast of the site and is assigned a probability of 33 percent. The San Andreas Fault, located approximately 24.2 kilometers (15.0 miles) southwest of the site, is assigned a 22 percent probability of an earthquake with a magnitude greater than 6.7 by 2043. Additional studies by the USGS regarding the probability of large earthquakes in the Bay Area are ongoing. These current evaluations include data from additional active faults and updated geological data.

3.3 Surface Conditions

The project site is bounded to the north by West Payran Street, to the west by Petaluma Boulevard North, and to the east and west by existing commercial properties. Existing site elevations within the proposed residential development area range from approximately +23 to +25 feet MSL (mean sea level) based on Google Earth elevations. The project area is relatively flat and currently developed with an existing commercial structure and asphalt parking and drive areas.

The project site was formerly occupied by a Chevron Service/Fuel station. We understand that environmental contamination from the previous site usage and from nearby off-site contamination sources has been cleaned up, and the subject site has been given environmental closure status. As a part of the environmental clean up, existing underground fuel tanks and related piping systems have been removed from the site, and the resulting excavations have been backfilled. At this time, compaction documentation for tank excavation backfill is not available. The approximate location of the tank removal excavation/backfill is shown on Figure 2A.

3.4 Field Exploration

We explored the subsurface conditions near the proposed improvements on August 10, 2020 with four cone penetration tests (CPTs) at the approximate locations shown on Figure 2A and 2B. The CPTs were excavated using an International Paystar 5000 by Middle Earth Geo Testing to depths ranging from 48 feet to 50 feet below the ground surface. The interpreted soil types, densities, strengths, and liquefaction potential are presented in Appendix A. Additional geotechnical exploration with borings and laboratory testing of soil samples should be performed at a later date as part of a Phase 2 design level geotechnical investigation.

3.5 Subsurface Soil Conditions

The subsurface exploration generally confirms the regionally mapped geologic conditions at the site. The site is overlain by approximately 5-feet of medium dense to dense clayey sand to stiff sandy clay overlying about 15-feet of soft to medium stiff silty clay (alluvial soil deposits). Loose to dense silty sands were encountered beneath the silty clay to the maximum explored depth of 50.5-feet.

3.6 Groundwater

Groundwater was encountered in the CPTs at depths between 10 and 13-feet below the ground surface. Groundwater should generally be expected in on-site excavations deeper than about 5 feet below grade and may be shallower during the winter months or following periods of heavy rain.

4.0 GEOLOGIC HAZARDS

This section summarizes our review of commonly considered geologic hazards and discusses their potential impacts on the planned improvements. The primary geologic hazards which could affect the proposed development include strong seismic ground shaking, liquefaction, and settlement. Other geologic hazards are judged less than significant regarding the proposed project. Geologic hazards, potential impacts and mitigation measures are discussed in further detail in the following sections.

4.1 Fault Surface Rupture

Under the Alquist-Priolo Earthquake Fault Zoning Act, the California Division of Mines and Geology (now known as the California Geological Survey) produced 1:24,000 scale maps showing known active and potentially active faults and defining zones within which special fault studies are required. The nearest known active fault to the site is the Rodgers Creek Fault located approximately 8.1 kilometers to the northeast. The site is not located within an Alquist-Priolo Special Studies Zone. We therefore judge the potential for fault surface rupture in the development area to be low.

Evaluation: *Less than significant.*
Recommendation: *No mitigation measures are required.*

4.2 Seismic Shaking

The site will likely experience seismic ground shaking similar to other areas in the seismically active Bay Area. The intensity of ground shaking will depend on the characteristics of the causative fault, distance from the fault, the earthquake magnitude and duration, and site-specific geologic conditions. Estimates of peak ground accelerations are based on either deterministic or probabilistic methods.

Deterministic methods use empirical attenuation relations that provide approximate estimates of median peak ground accelerations. A summary of the active faults that could most significantly affect the planning area, their maximum credible magnitude, closest distance to the center of the planning area, probable peak ground accelerations, and 84th percentile peak ground accelerations are summarized in Table 1. The calculated accelerations should only be considered as reasonable estimates. Many factors (e.g., soil conditions, orientation to the fault, etc.) can influence the actual ground surface accelerations.

Table 1 – Deterministic Peak Ground Accelerations for Active Faults

Fault	Moment Magnitude for Characteristic Earthquake	Closest Estimated Distance (km)	Median Peak Ground Acceleration (g)	84% Peak Ground Acceleration (g)
Rodgers Creek	7.3	8.1	0.36	0.61
San Andreas	8.0	24.2	0.25	0.43
Maacama	7.4	31.9	0.17	0.29
Hayward	7.3	32.4	0.16	0.28
San Gregorio	7.4	38.6	0.15	0.26

Reference: Caltrans ARS Online v2.3.09 accessed on August 20, 2020. Site Class D= 270 (ft/sec)

Probabilistic Seismic Hazard Analysis analyzes all possible earthquake scenarios while incorporating the probability of each individual event to occur. The probability is determined in the form of the recurrence interval, which is the average time for a specific earthquake acceleration to be exceeded. The design earthquake is not solely dependent on the fault with the closest distance to the site and/or the largest magnitude, but rather the probability of given seismic events occurring on both known and unknown faults.

We calculated the peak ground acceleration for two separate probabilistic conditions; the 2 percent chance of exceedance in 50 years (2,475-year statistical return period) and the 10 percent chance of exceedance in 50 years (475-year statistical return period). The peak ground acceleration values were calculated utilizing the USGS Unified Hazard Tool (USGS, 2020). The results of the probabilistic analyses are presented below in Table 2.

Table 2 – Probabilistic Peak Ground Accelerations for Active Faults

Probability of Exceedance	Statistical Return Period	Magnitude	Peak Ground Acceleration (g)
2% in 50 years	2,475 years	7.12	0.80
10% in 50 years	475 years	7.06	0.48

Reference: USGS Unified Hazard Tool accessed on July 28, 2020. Site Class D= 270 (ft/sec)

Ground shaking can result in structural failure and collapse of structures or cause non-structural building elements (such as light fixtures, shelves, cornices, etc.) to fall, presenting a hazard to building occupants and contents. Compliance with provisions of the most recent version of the California Building Code (2019 CBC) should result in structures that do not collapse in an earthquake. Damage may still occur, and hazards associated with falling objects or non-structural building elements will remain.

The potential for strong seismic shaking at the project site is high. Due to their proximity and historic rates of activity, the Rodgers Creek, San Andreas, and Hayward Faults present the highest potential for severe ground shaking. The significant adverse impact associated with strong seismic shaking is potential damage to structures and improvements.

Evaluation: *Less than significant with mitigation.*

Recommendation: *Measures include design of new structures in accordance with the provisions of the 2019 California Building Code or subsequent codes in effect when final design occurs. Preliminary seismic design coefficients are presented in Section 5.1 of this report.*

4.3 <u>Liquefaction and Related Effects</u>

Liquefaction refers to the sudden, temporary loss of soil shear strength during strong ground shaking. Liquefaction-related phenomena include liquefaction-induced settlement, flow failure, and lateral spreading. These phenomena can occur where there are saturated, loose, granular deposits. Recent advances in liquefaction studies indicate that liquefaction can occur in granular materials with a high, 35 to 50%, fines content (soil particles that pass the #200 sieve), provided the fines exhibit a plasticity less than 7. Saturated granular layers were observed during our subsurface exploration below the ground surface. Additionally, the site is mapped by the Association of Bay Area Governments (ABAG) as being moderately susceptible to liquefaction as shown on Figure 6.

To evaluate soil liquefaction, the seismic energy from an earthquake is compared with the ability of the soil to resist pore pressure generation. The earthquake energy is termed the cyclic stress ratio (CSR) and is a function of the maximum considered earthquake peak ground acceleration (PGA) and depth. Soil resistance to liquefaction is based on its relative density, and the amount and plasticity of the fines (silts and clays). The relative density of cohesionless soil is correlated with CPT data measured in the field and corrected for overburden and percent fines.

We analyzed the potential for liquefaction utilizing the procedures outlined by Idriss and Boulanger (2014) within the liquefaction interpretation program, CLiq (Geologismiki, CLiq). The seismic event input into the model consisted of a magnitude 7.3 earthquake producing a PGA of 0.74 g, which corresponds to the PGA_M defined in ASCE 7-16. The results of our analyses indicate several liquefiable layers of various thicknesses underlie the site at various depths between 5 and 50-feet below the ground surface. The results of our liquefaction analyses are presented on Figures 7 through 10.

4.3.1 Post Liquefaction Settlement

Based on current post liquefaction settlement analyses procedures, settlement can occur in soils that exhibit a factor of safety against liquefaction of 2.0 or less. Utilizing the procedures outlined by Idriss and Boulanger (2014) approximately 1.5 to 5.0-inches of post liquefaction settlement may occur. It is noted that the estimated liquefaction induced settlement based on an analysis of CPT 2 through CPT 4 is about 1.5 to 2.0 inches. Approximately 5.0 inches of liquefaction induced settlement is predicted based on an analysis of CPT 1. We recommend that an exploratory boring should be positioned near CPT 1 during the Phase 2 design level geotechnical study to confirm or modify the predicted liquefaction induced settlement at this location. Differential settlement is estimated to be approximately one half of the estimated total settlement.

Additionally, we utilized the procedures outlined by Ozocak and Sert (2010) to calculate the Liquefaction Potential Index (LPI), which is a gauge to determine if liquefiable layers will impact the ground surface. LPI is a function of the thickness, depth, and factor of safety against liquefaction in the individual layers within a soil column. The resulting LPI value corresponds to a relative potential for surface deformation impacting the ground surface. Typically, an LPI value of zero indicates the liquefiable layer will not impact the ground surface; while a value less than 5 has a low probability, value between 5 and 15 have a moderate probability and an LPI value greater than 15 have a high probability of surface impact. The results of our liquefaction analyses indicate LPI between 5.0 and 20.0, suggesting a moderate to high probability of liquefaction effects impacting the ground surface.

4.4 Settlement

Significant settlement can occur when new loads are placed at sites that are located over soft compressible clays, such as Bay Mud. The amount and rate of settlement is dependent on the magnitude of additional new loads (i.e. new structures and/or new fill), the thickness of compressible material, and the inherent compressibility properties of the soft clay. The project site is underlain by soft to medium stiff clay between a depth of roughly five to twenty feet below the ground surface. Therefore, the risk of total and differential settlements due to static loading of the soft to medium stiff clay is moderate.

Evaluation: *Less than significant with mitigation.*
Recommendation: *Measures include settlement analysis based on further exploration, laboratory testing, and more detailed information regarding building loads as part of the design level report. Foundations should be designed to accommodate the predicted settlements.*

4.5 Seismic Densification

Seismic ground shaking can induce settlement in unsaturated, loose, granular soils. Settlement occurs as the loose soil particles rearrange into a denser configuration when subjected to seismic ground shaking. Varying degrees of settlement can occur throughout a deposit, resulting in differential settlement of structures founded on such deposits. Based on our subsurface exploration, the soil above the groundwater level is generally classified as medium dense to dense sands or soft to medium stiff clay alluvial soils. Therefore, the risk of seismic densification impacting the new structures is low.

Evaluation: *Less than significant.*
Recommendation: *Measures include compaction of any loose sandy surficial soil as part of the site grading, and proper design of building foundations.*

4.6 Expansive Soils

Soil expansion occurs when clay particles interact with water causing seasonal volume changes in the soil matrix. The clay soil swells when saturated and then contracts when dried. This phenomenon generally decreases in magnitude with increasing confinement pressures at increasing depths. These volume changes may damage lightly loaded foundations, concrete slabs, pavements, retaining walls and other improvements. Expansive soils also cause soil creep on sloping ground.

Near surface soils are generally clayey in nature and based on experience with projects in the immediate vicinity, expansive clay soils may be present at or near the ground surface. Therefore, the risk of expansive soil affecting the proposed improvements appears moderate to high. Additional exploration and laboratory testing should be performed to determine the expansive potential of surficial soils as part of a Phase 2 design-level report.

Evaluation: *Less than significant with mitigation.*

Recommendation:	*Evaluate slab subgrades for expansive soils as part of a design level report. Foundations and slabs should be designed to account for expansive soil conditions, and grading can be undertaken to replace the expansive surface soil with a layer of nonexpansive fill beneath building footprints.*

4.7 Lurching and Ground Cracking

Lurching and associated ground cracking can occur during strong ground shaking. The ground cracking generally occurs along the tops of slopes where stiff soils are underlain by soft deposits or along steep slopes or channel banks. These conditions do not exist at the project site. Therefore, the risk of lurching and ground cracking impacting the new structures is low.

Evaluation:	*Less than significant.*
Recommendation:	*No mitigation measures are required.*

4.8 Erosion

Sandy soils on moderately steep slopes or clayey soils on steep slopes are susceptible to erosion when exposed to concentrated surface water flow. The potential for erosion is increased when established vegetation is disturbed or removed during normal construction activity.

Evaluation:	*Less than significant with mitigation.*
Recommendation:	*For new improvements at the site, careful attention should be paid to finished grades and the project Civil Engineer should design the site drainage system to collect surface water into a storm drain system that discharges water at appropriate locations. Re-establishment of vegetation on disturbed areas will also minimize erosion. Erosion control measures during and after construction should be in accordance with a prepared Storm Water Pollution Prevention Plan and should conform to the most recent version of the California Stormwater Quality Association, Stormwater Best Management Practice Handbook.*

4.9 Slope Instability/Landsliding

Slope instability generally occurs on relatively steep slopes and/or on slopes underlain by weak materials. The project site lies on level terrain, therefore, slope instability/landsliding is not considered a significant geologic hazard at the project site.

Evaluation:	*No significant impact.*
Mitigation:	No mitigation measures are required.

4.10 Flooding

The project site is located at about elevation +23 feet and is not mapped as being within a 100- or 500-year flood zone (ArcGIS, 2020). Therefore, large scale flooding is considered a low hazard at the project site.

Evaluation: *Less than significant with mitigation.*
Recommendation: *The project Civil Engineer or Architect is responsible for site drainage and should evaluate localized flooding potential and provide appropriate mitigation.*

4.11 Tsunami and Seiche

Seiche and tsunamis are short duration, earthquake-generated water waves in large enclosed bodies of water and the open ocean, respectively. The extent and severity of a seiche or tsunami would be dependent upon ground motions and fault offset from nearby active faults. Tsunami hazard mapping of the project area (ArcGIS, 2020) indicates the site is not located within an area that is susceptible to tsunami inundation. Therefore, the likelihood of inundation by seiche or tsunami is low.

Evaluation: *Less than significant.*
Recommendation: *No mitigation measures are required.*

5.0 PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our preliminary investigation, we conclude the site conditions are suitable for the proposed improvements. The primary geotechnical issues to address in design of the project are strong seismic shaking due to the close proximity of the Rodgers Creek Fault, liquefaction and liquefaction induced settlement, static settlement, and expansive soil. In addition, existing fills in former tank excavations should be investigated in more detail to evaluate potential settlement related to these fill areas.

5.1 Seismic Design

The project site is located in a seismically active area. Therefore, structures should be designed in conformance to the seismic provisions of the California Building Code (CBC). However, since the goal of the building code is protection of life safety, some structural damage may still occur during strong ground shaking. The 2019 CBC/ASCE 7-16 was adopted in January 2020. We recommend minimum mitigation of ground shaking include seismic design per the 2019 California Building Code/ASCE 7-16.

The magnitude and character of these ground motions will depend on the particular earthquake and the site response characteristics. Based on the interpreted subsurface conditions and close proximity to the Rodgers Creek, San Andreas, and Hayward Faults, we recommend the CBC coefficients and site values shown in Table A below for use to calculate the design base shear of the new construction.

Based on the subsurface conditions, the project site is classified as a “Site Class E”. Additionally, because the S_1 value is greater than 0.20 g a site-specific ground motion analysis should be performed per the procedures outlined in ASCE 7-16. However, per ASCE 7-16 Section 11.4.8, a site-specific analysis is not required for structures located on sites classified as “Site Class E” if the Short Period Site Coefficient, F_a, is taken as equal to that of “Site Class C”. This exception applies to structures with fundamental periods within the "short-period" range. We should perform a site-specific ground motion analysis if it is determined by the design team that "long-period" accelerations are needed.

Table 3 – Preliminary 2019 California Building Code Seismic Design Criteria

Parameter	Design Value	
Site Latitude	38.2447°N	
Site Longitude	-122.6443°W	
Site Class	C	E
Spectral Response (short), S_S	1.50 g	1.50 g
Spectral Response (1-sec), S_1	0.60 g	0.60 g
Spectral Response (Short), S_{MS}	1.80 g	n/a
Design Spectral Response (short), S_{DS}	1.20 g	n/a
Short Period Site Coefficient, F_a	1.2	n/a
MCE_G PGA Adjusted, PGA_M	0.74 g	0.68 g

The effects of earthquake shaking (i.e., protection of life safety) can be mitigated by close adherence to the seismic provisions of the current edition of the CBC. However, some building damage may still occur during strong ground shaking.

5.2 Site Grading

Site grading and earthwork should be performed in accordance with the recommendations and criteria outlined in the following sections.

5.2.1 Site Preparation

Clear all grass, brush, roots, and other organic matter from areas where improvements are planned. Any construction debris or abandoned utilities encountered should be removed from the site. Any old foundations should be completely removed. Both loose sandy soil and expansive clay soil may be locally present near the existing ground surface. Within building areas, loose sands and expansive soil should be over-excavated 3 feet below grade and backfilled with non-expansive compacted structural fill in accordance with subsequent sections in this report. Vegetation scrapings should be stockpiled for re-use in landscape areas or removed from the site.

5.2.2 Excavations

Subsurface conditions at the site generally consist of 5-feet of medium stiff sandy clay and medium dense silty sand underlain by about 15-feet of soft to medium stiff silty clay. We anticipate excavations can be reasonably completed with "traditional" equipment such as backhoes and dozers. Excavations having a depth of 5-feet or more must be excavated and shored in accordance with OSHA regulations. We recommend that the project Contractor be responsible for site safety, including trench shoring and de-watering. Pursuant to OSHA classifications, the onsite fill soils are "Type C" and may be prone to "squeezing" and raveling in open excavations. Additionally, groundwater should be anticipated in excavations deeper than 2-feet, and the Contractor should anticipate the need for adequate de-watering and shoring in all excavations deeper than 5 feet. Many shoring systems are available, and the Contractor should select an appropriate system that allows for efficient installation to prevent collapse. We recommend that de-watering be accomplished by use of submersible pumps.

5.2.3 Fill Materials, Placement and Compaction

Selected soil and rock mixtures generated from on-site excavations are likely to be suitable for re-use as fill, provided they can be processed to meet the specifications presented below. Whether imported or derived of onsite materials, all fill material should consist of soil and rock mixtures that: (1) are free of organic material, (2) have a Liquid Limit less than 40 and a Plasticity Index of less than 15 with very low to low expansion index, and (3) have a maximum particle size of four inches. Any imported fill material needs to be tested to verify its suitability for use as fill material prior to placement.

Prior to fill placement, all subgrades should be scarified a minimum of 8-inches deep, moisture-conditioned slightly above the optimum moisture content, and compacted to a minimum of 90 percent relative compaction. New fill shall be placed in layers not exceeding 8-inches and compacted to at least 90 percent relative compaction. The compacted subgrade and new fill should be firm and unyielding when proof rolled with heavy compaction equipment.

Within pavement areas, relative compaction of the upper 12 inches of subgrade soil should be increased to 95 percent minimum. These areas should also produce a smooth, firm, and unyielding surface when proof-rolled with heavy construction equipment such as loaded water trucks or scrapers. Relative compaction, maximum dry density, and optimum moisture content of fill materials should be determined in accordance with ASTM Test Method D 1557, "Moisture-Density Relations of soils and Soil-Aggregate Mixtures Using a 10-lb. Rammer and 18-in. Drop."

5.3 Preliminary Foundation Design

As previously discussed, the site is underlain by potentially liquefiable soils that may settle up to about 5.0-inches during the design seismic event. Provided that the grading is undertaken as described above, a rigid, interconnected shallow foundation system may be utilized to support the proposed structures. However, the foundation system should be designed to resist up to 5.0-inches of total and 2.5-inches of differential settlement over 30-feet. The rigid foundation system may consist of interconnected shallow foundations or a concrete mat slab-on-grade. Foundation design criteria are shown in Table 4.

We recommend that the perimeter footing or thickened slab edge at the perimeter of the building should extend to a minimum depth of 12-inches below the rough compacted pad grade to reduce the potential for surface water to seep into the underslab area.

TABLE 4
SHALLOW FOUNDATION PRELIMINARY DESIGN CRITERIA
890 Petaluma Boulevard North
Petaluma, California

Concrete Mat Slab Foundation	
Minimum thickness[1]:	12 inches
Modulus of subgrade reaction, k_s:	100 pci
Maximum unsupported interior span[2]:	23 feet
Maximum unsupported edge/corner span[2]:	10 feet
Base friction:	0.30
Shallow Stiff Grid Foundations	
Minimum embedment below existing grade:	30 inches
Minimum width[3]:	
One-story:	12 inches
Two-story:	15-inches
Three-story:	18-inches
Allowable bearing pressure:	
Dead plus live loads:	1,800 psf
Total including wind and seismic	2,400 psf
Base friction coefficient:	0.30
Lateral passive resistance[4,5]:	250 pcf
Maximum unsupported interior span[2]:	23 feet
Maximum unsupported edge/corner span[2]:	10 feet

Notes:
1.) Thickened slab at perimeter of building should extend at least 12 inches below the compacted rough pad grade.
2.) Assumes rigid slab behavior with idealized fixed conditions.
3.) Design shallow foundations to similar bearing pressures, i.e., size footing widths to maintain uniform bearing loads. Maintain above optimum moisture contents until concrete slabs are completed.
4.) May increase design values by 1/3 for total design loads, including wind and seismic.
5.) Neglect upper 12-inches unless confined by concrete. Equivalent Fluid Pressure, not to exceed 2,500 psf.

If the predicted post-liquefaction settlement is unacceptable, the structures may be supported on a deep foundation system. However, the deep foundation will need to extend on the order of 50-feet or more below the ground surface. Suitable deep foundation options include helical piles, drilled piers, driven piles, and torque down piles. We can provide additional deep foundation design criteria for the chosen system upon request.

The foundation design for new structures will primarily depend on building loads and layouts, and the results of supplemental exploratory boring and laboratory testing conducted during the design level geotechnical study.

5.4 Interior Concrete Slabs-On-Grade

We recommend that interior concrete slabs should be placed on a minimum 36-inch-thick layer of compacted select, nonexpansive import soil fill over a moist compacted subgrade as previously described above.

To improve interior (conditioned space) moisture conditions, a 6-inch layer of clean, free draining, 3/4-inch angular gravel should be placed beneath the interior concrete slabs to form a capillary moisture break. The rock must be placed on a properly moisture conditioned and compacted subgrade that has been approved by the Geotechnical Engineer.

A plastic membrane vapor barrier, 15 mils or thicker and meeting the requirements of ASTM E-1745 Class A, should be placed over the rock layer and be installed per ASTM 1643. A 2-inch dry sand layer can be placed over the vapor barrier to reduce puncture of the plastic membrane and aid in slab curing. However, the 2-inch sand layer may be omitted if approved by structural engineer and moisture sensitive floor coverings will be used. Eliminating the capillary moisture break and/or plastic vapor barrier may result in excess moisture intrusion through the floor slabs resulting in poor performance of floor coverings, mold growth, or other adverse conditions.

It should be pointed out that where the gravel capillary break layer is placed beneath floor slabs, there is a possibility that water will tend to collect in the gravel layer and become trapped. If this condition occurs, the potential for moisture problems at the surface of the slab will be increased. One method of minimizing the potential for this to occur would be to construct a subdrain trench through and just below the gravel layer so that water collected in this area can escape. The subdrain should extend at least 12 inches below the base of the slab and 6 inches below the bottom of the gravel layer, and would consist of a four-inch diameter, perforated pipe (Schedule 40 PVC) surrounded by gravel. The subdrain would connect to the gravel layer beneath the slab, and the pipe should lead (at a minimum one percent slope) to a storm drain or another suitable outlet point. The outlet pipe should transition to nonperforated pipe at a point two feet inside the perimeter footing of the structure. A compacted clayey soil plug or other type of moisture barrier should be used at the point where the outlet pipe penetrates the perimeter footing to prevent seepage from back-flowing into the under-slab area. We recommend that the under-slab drains should be spaced no more than 25 feet on center.

This industry standard approach to floor slab moisture control, as discussed above, does not assure that floor slab moisture transmission rates will meet floor covering manufacturer's requirements or that indoor humidity levels will be low enough to inhibit mold growth. Building design, construction, and intended use have a significant role in moisture problems and should be carefully evaluated by the owner, designer, and builder in order to meet the project requirements.

5.5 Exterior Concrete Slabs

Exterior concrete walkway slabs and other concrete slabs that are not subjected to vehicle loads should be a minimum of 5 inches thick and underlain with 4 inches or more of Class 2 aggregate baserock. The aggregate baserock should be moisture conditioned to near optimum and compacted to at least 95 percent relative compaction. The upper 8 inches of subgrade on which aggregate baserock is placed should be prepared as previously discussed under Section 5.2.

Where improved performance is desired (i.e., reduced risks of cracking or small movements), exterior slabs can be thickened to 6 inches and reinforced with steel reinforcing bars (not welded wire mesh). We recommend crack control joints no farther than 6 feet apart in both directions and that the reinforcing bars extend through the control joints. Some movement or offset at sidewalk joints should be expected as the underlying soils expand and shrink from seasonal moisture changes or experience differential settlement due to static or seismic loading.

5.6 Site and Foundation Drainage

Careful consideration should be given to design of finished grades at the site. We recommend that the building areas be raised slightly and that the adjoining landscaped areas be sloped downward at least 0.25 feet for 5 feet (5 percent) from the perimeter of building foundations. Where hard surfaces, such as concrete or asphalt adjoin foundations, slope these surfaces at least 0.10 feet in the first 5 feet (2 percent).

Roof gutter downspouts may discharge onto the pavements but should not discharge onto landscaped areas immediately adjacent to the buildings. Provide area drains for landscape planters adjacent to buildings and collect downspout discharges into a tight pipe collection system that discharges well away from the building foundations. Site drainage should be discharged away from the building areas and outlets should be designed to reduce erosion. Site drainage improvements should be connected into an established storm drainage system.

5.7 Underground Utilities

Excavations for utilities will generally encounter a combination of loose to dense clayey sand and soft to stiff clayey soils containing variable amounts of sand and gravel. Groundwater may be encountered at shallow depths. Trench excavations having a depth of 5 feet or more must be excavated and shored in accordance with OSHA regulations, as discussed in Section 5.2.2.

Unless otherwise recommended by the pipe manufacturer, pipe bedding and embedment materials should consist of well-graded sand with 90 to 100 percent of particles passing the No. 4 sieve and no more than 5 percent finer than the No. 200 sieve. Crushed rock or pea gravel may also be considered for pipe bedding. Provide the minimum bedding thickness beneath the pipe in accordance with the manufacturer's recommendations (typically 3 to 6 inches). Trench backfill may consist of on-site soils, provided that the soil meets the fill criteria outlined in Section 5.2.3 or imported aggregate baserock. Trench backfill should be moisture conditioned and placed in thin lifts and compacted to at least 90 percent. Use equipment and methods that are suitable for work in confined areas without damaging utility conduits.

5.8 Pavements

We have calculated thicknesses for asphalt pavements in accordance with Caltrans procedures for flexible pavement design. Our calculations assume an R-value of 10 and a range of Traffic Indices from 4.0 to 7.0 depending on the expected traffic loads for a twenty-year design life. The R-value should be confirmed with future laboratory testing. In general, areas expected to experience loading from heavy vehicles should be designed using the higher Traffic Index, while parking areas and other lightly loaded areas can utilize a thinner pavement section based on the lower Traffic Index. The recommended pavement sections are presented in Table 5.

Table 5 – Preliminary Asphalt-Concrete Pavement Sections

Traffic Index[1]	Asphalt Concrete (inches)	Aggregate Base (inches)
4.0	2.5	8.0
5.0	3.0	9.0
6.0	3.5	12.0
7.0	4.0	15.0

(1) Traffic Index for final pavement design to be determined by the project Civil Engineer.

In pavement areas, the upper 12 inches of subgrade should be compacted to at least 95 percent relative compaction. The aggregate base and asphalt-concrete should conform to the most recent version of Caltrans Standard Specifications and should be compacted to at least 95 percent relative compaction. Additionally, the subgrade and aggregate base should be firm and unyielding under heavy, rubber-tired construction equipment. If heavier truck traffic or “superior” performance is desired, the thickness of the aggregate base and asphalt may be increased.

6.0 SUPPLEMENTAL GEOTECHNICAL SERVICES

Following review and consideration of this report, we should consult with the project team regarding the “preferred” foundation type for the new structures. Supplemental exploration and laboratory testing will be required once building details are better defined (e.g., building layouts and structural loads, extent of excavation, etc.) to prepare design level geotechnical recommendations. We will also be available to provide consultation throughout the design process on other geotechnical-related items.

As project plans near completion, we should review them to ensure that the intent of our recommendations has been sufficiently incorporated in the design. During construction, we should be present intermittently to observe and test the geotechnical portions of the work. The purpose of our observation and testing is to confirm that site conditions are as anticipated, to adjust our recommendations and design criteria if needed, and to confirm that the Contractor’s work is performed in accordance with the project plans and specifications.

7.0 LIMITATIONS

We believe this report has been prepared in accordance with generally accepted geotechnical engineering practices in the San Francisco Bay Area at the time the report was prepared. This report has been prepared for the exclusive use of Matthew Ridgway and/or his assignees specifically for this project. No other warranty, expressed or implied, is made. Our evaluations and recommendations are based on the data obtained during our subsurface exploration program and our experience with soil conditions in this geographic area.

8.0 LIST OF REFERENCES

American Society of Civil Engineers (ASCE) (2010), "Minimum Design Loads for Buildings and Other Structures" (2010 ASCE-7), Structural Engineering Institute of the American Society of Civil Engineers.

American Concrete Institute, "318-14: Building Code Requirements for Structural Concrete and Commentary".

American Society for Testing and Materials, (2009) "2009 Annual Book of ASTM Standards, Section 4, Construction, Volume 4.08, Soil and Rock; Dimension Stone; Geosynthetics," ASTM, Philadelphia.

California Building Code, 2019 Edition, California Building Standards Commission/International Conference of Building Officials, Whittier, California.

California Department of Transportation (Caltrans) (2015), 2015 Standard Specifications.

California Department of Transportation (Caltrans) (2015), Highway Design Manual, 6th Edition.

Occupational Safety and Health Administration (OSHA)(2005), Title 29 Code of Federal Regulations, Part 1926 (www.OSHA.gov).

United States Geological Survey, "Database of Potential Sources for Earthquakes Larger than Magnitude 6 in Northern California," The Working Group on Northern California Earthquake Potential, Open File Report 96-705, 1996.

United States Geological Survey (2003), "Summary of Earthquake Probabilities in the San Francisco Bay Region, 2002 to 2032," The 2003 Working Group on California Earthquake Probabilities, 2003.

United States Geological Survey (2008), "The Uniform California Earthquake Rupture Forecast, Version 2," The 2007 Working Group on California Earthquake Probabilities, Open File Report 2007-1437, 2008.

United States Geological Survey (2013), "The Uniform California Earthquake Rupture Forecast, Version 3 (UCERF3) – The Time Independent Model", USGS Open-File Report 2013-1165, CGS Special Report 228, Southern California Earthquake Center Publication 1792.

Youd, T.L. "Liquefaction-Induced Lateral Ground Displacement" (April 2, 1995). *International Conferences on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics.* Paper 3.

SITE: LATITUDE, 38.2447°
LONGITUDE, -122.6443°

SITE LOCATION

N.T.S.

NORTH

REFERENCE: Google Earth, 2020

MPEG MILLER PACIFIC ENGINEERING GROUP	504 Redwood Blvd. Suite 220 Novato, CA 94947 T 415 / 382-3444 F 415 / 382-3450 www.millerpac.com	SITE LOCATION MAP		
		Mixed Use Development 890 Petaluma Boulevard North Petaluma, California	Drawn MMT Checked	1 FIGURE
FILENAME: 3032.001 Figures.dwg		Project No. 3032.001 Date: 9/17/2020		

SITE PLAN

SCALE

0 20 40 80 FEET

NORTH

Approximate location of CPT completed by MPEG, 2020

REFERENCE: Google Earth, 2020

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EXISTING CONDITIONS SITE PLAN

Mixed Use Development
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Project No. 3032.001 Date: 9/17/2020

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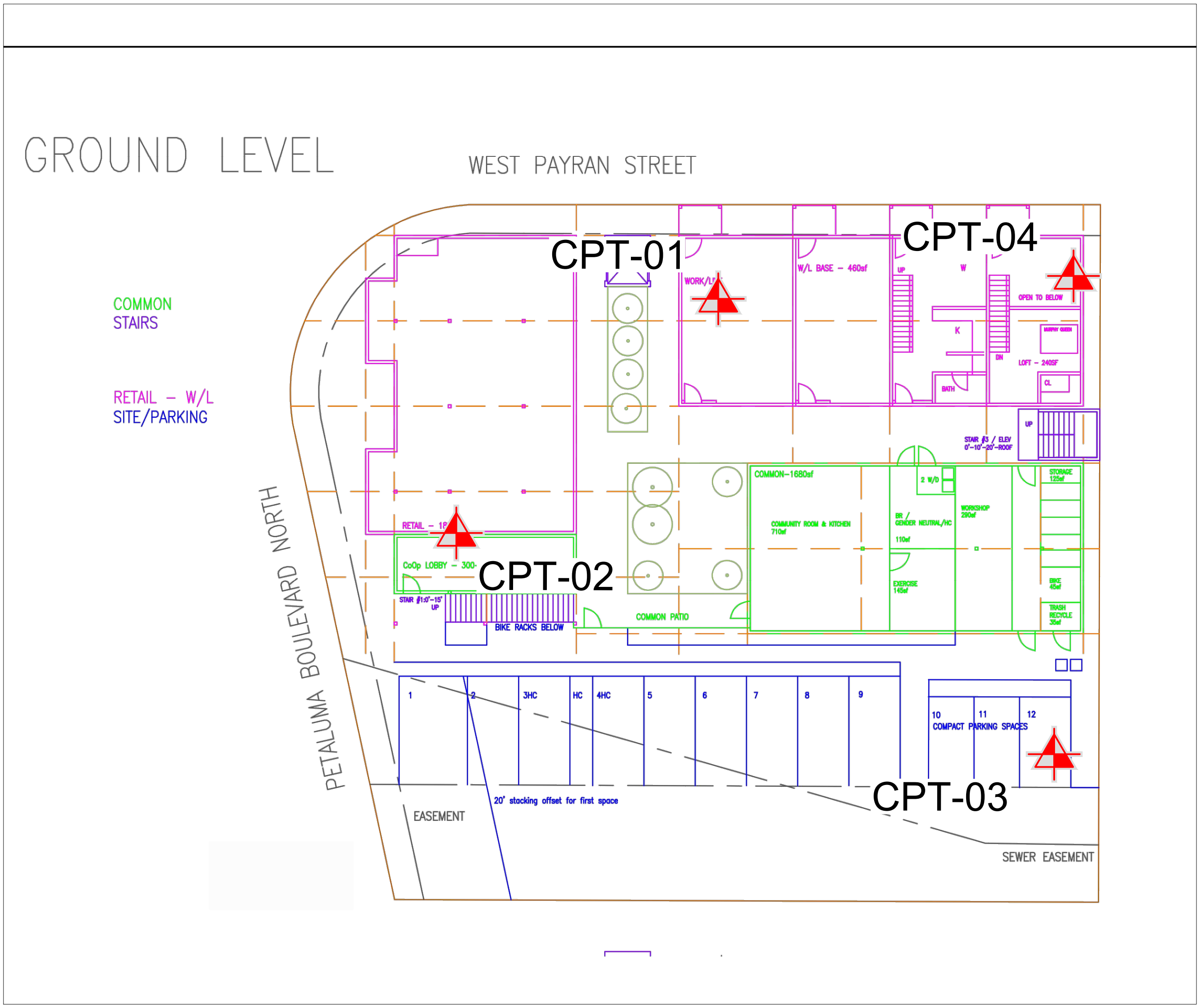
Checked

2A
FIGURE



SITE PLAN

SCALE

0 15 30 60 FEET

Approximate location of CPT completed by MPEG, 2020

REFERENCE: Site plan provided by client, "Ground Level Concept".

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PROPOSED CONDITIONS SITE PLAN

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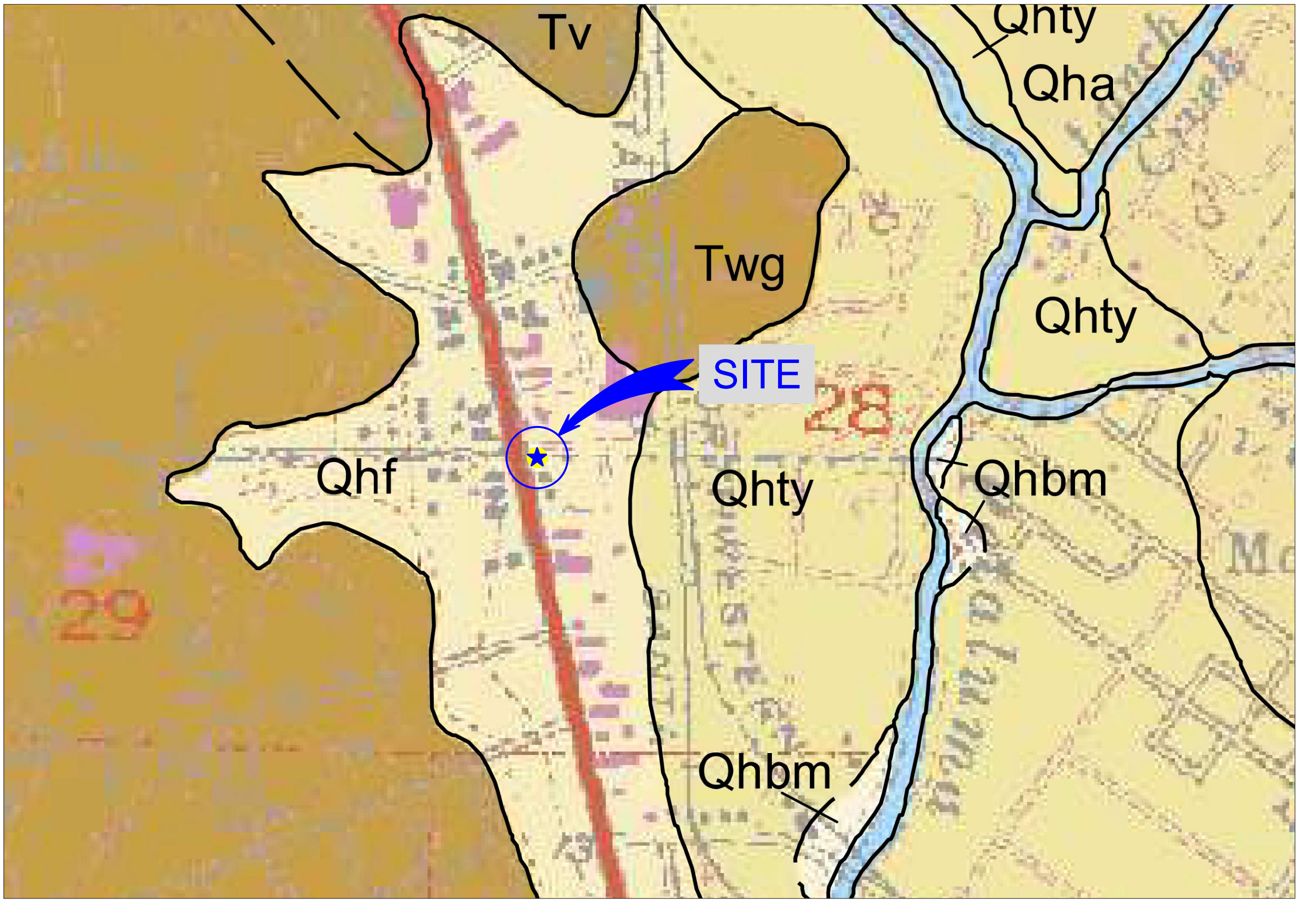
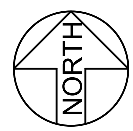
2B
FIGURE



REGIONAL GEOLOGIC MAP
(NOT TO SCALE)



LEGEND:

Qhbm	Bay Mud (Holocene) - Composed of silts, clays, fine sands and peat..
Qhf	Fan Deposits (Holocene) - Moderately to poorly sorted sand, gravel, silt and clay.
Qhty	Terrace Deposits (Latest Holocene) - Moderately to well sorted sand, gravel, silt and minor clay
Twg	Wilson Grove Formation (Miocene) - Fine grained, well sorted marine sandstone.

REFERENCE: Bezore, Stephen, et. al. (2002), "Geologic Map of the Petaluma 7.5' Quadrangle, Sonoma and Marin Counties, California: A Digital Database." California Department of Conservation, CGS, Scale 1:24,000.

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REGIONAL GEOLOGIC MAP

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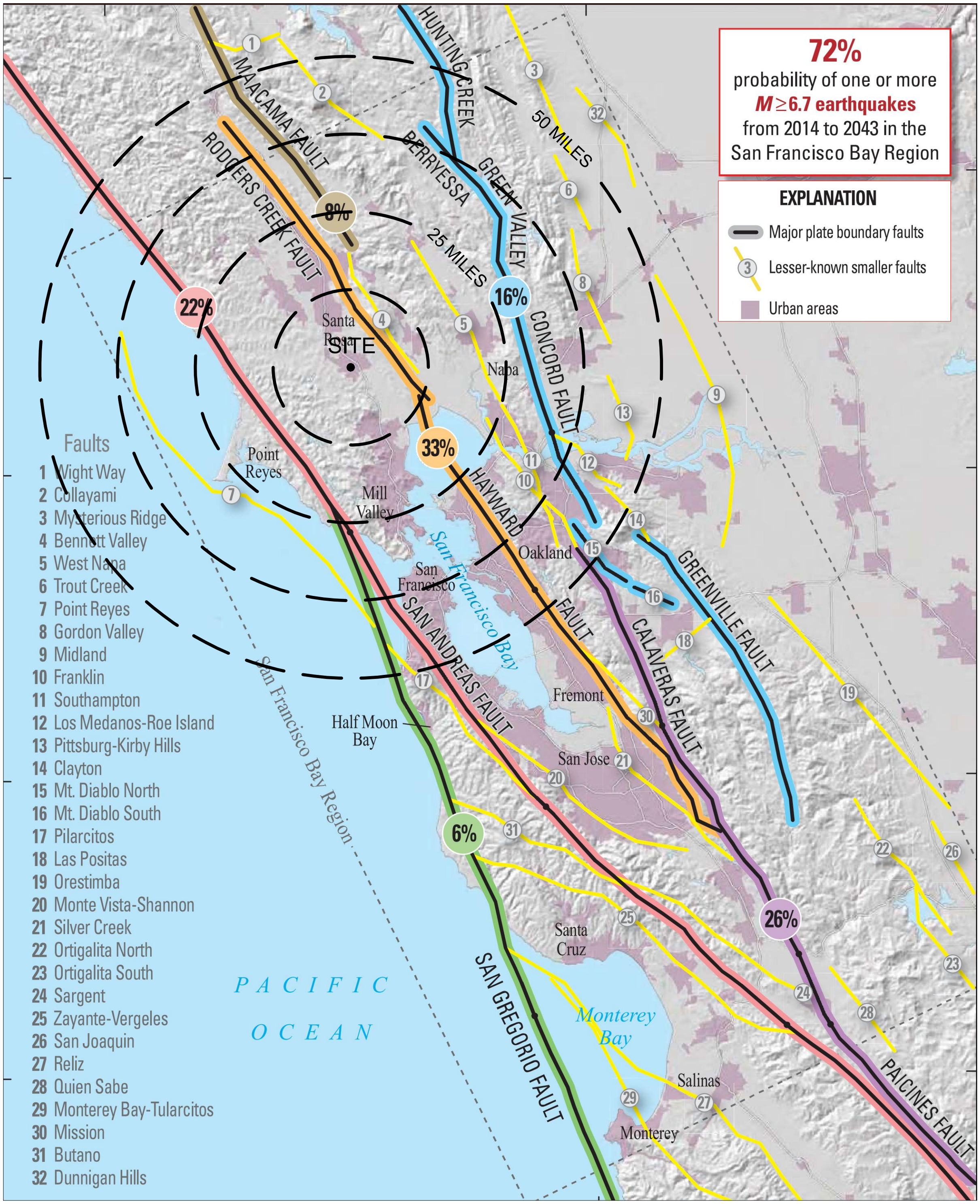
3
FIGURE



SITE COORDINATES
LAT. 38.2447°
LON. -122.6443°

SCALE
0 12.5 25 50 MILES

NORTH

DATA SOURCE:
1) U.S. Geological Survey, U.S. Department of the Interior, "Earthquake Outlook for the San Francisco Bay Region 2014-2043", Map of Known Active Faults in the San Francisco Bay Region, Fact Sheet 2016-3020, Revised August 2016 (ver. 1.1).

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ACTIVE FAULT MAP

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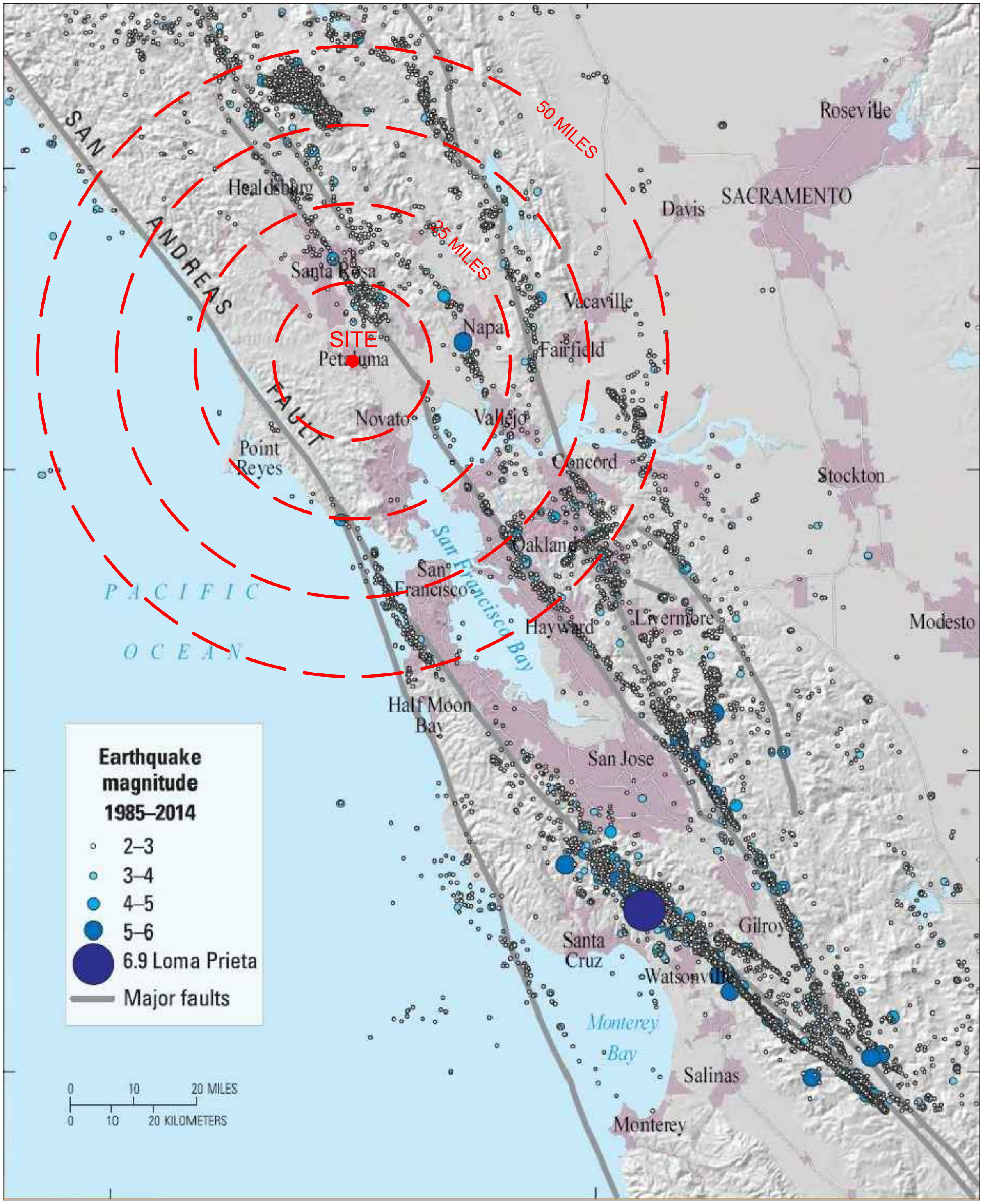
4
FIGURE



SITE COORDINATES
LAT. 38.2447°
LON. -122.6443°

SCALE
0 12.5 25 50 MILES

NORTH

DATA SOURCE:
1) U.S. Geological Survey, U.S. Department of the Interior, "Earthquake Outlook for the San Francisco Bay Region 2014-2043", Map of Earthquakes Greater Than Magnitude 2.0 in the San Francisco Bay Region from 1985-2014, Fact Sheet 2016-3020, Revised August 2016 (ver. 1.1).

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HISTORIC EARTHQUAKE ACTIVITY

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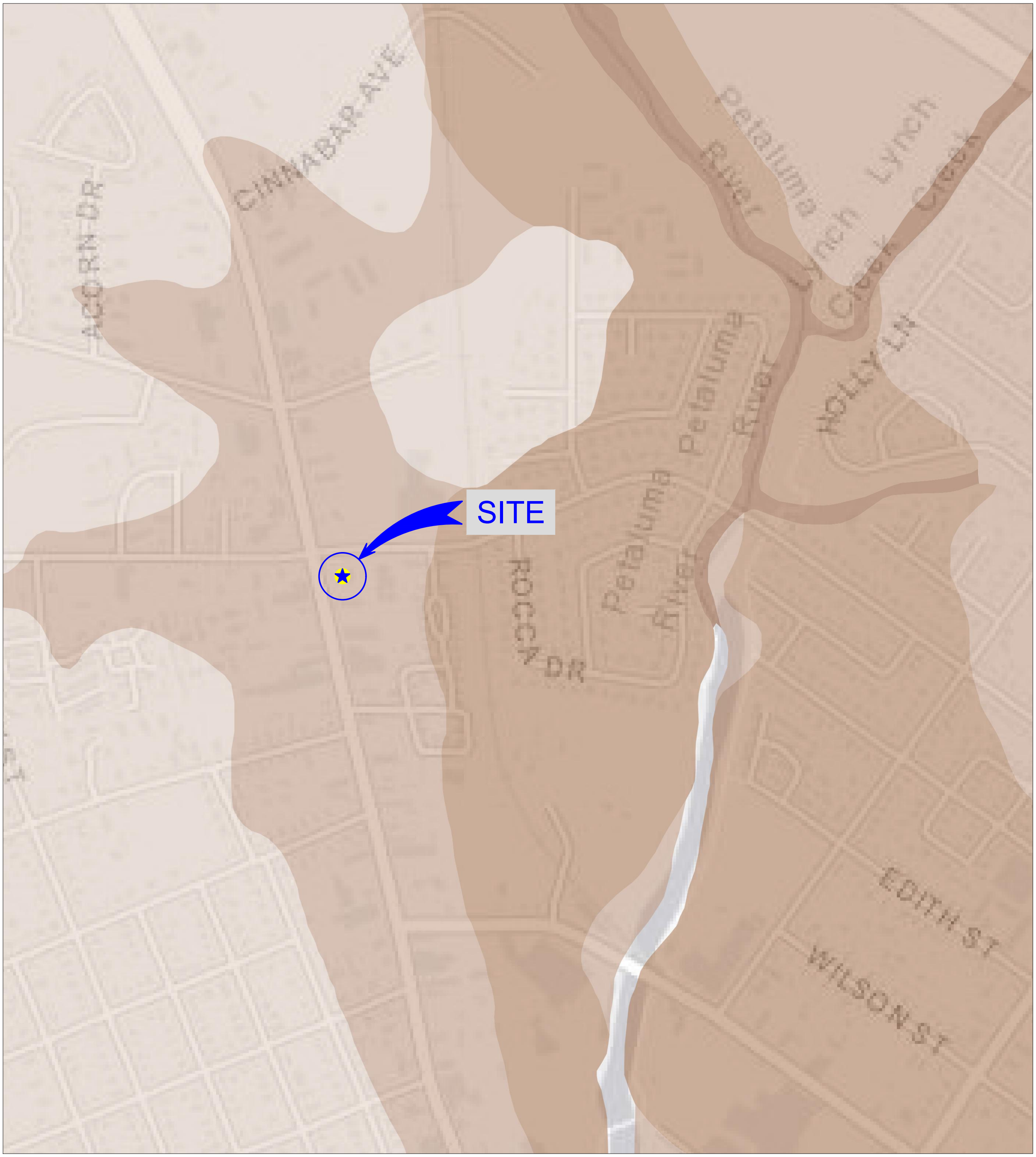
5
FIGURE



Susceptibility Level:

- Very High
- High
- Moderate
- Low
- Very Low

No Scale

NORTH

Reference: ABAG Hazard Map Viewer.

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LIQUEFACTION SUSCEPTIBILITY MAP

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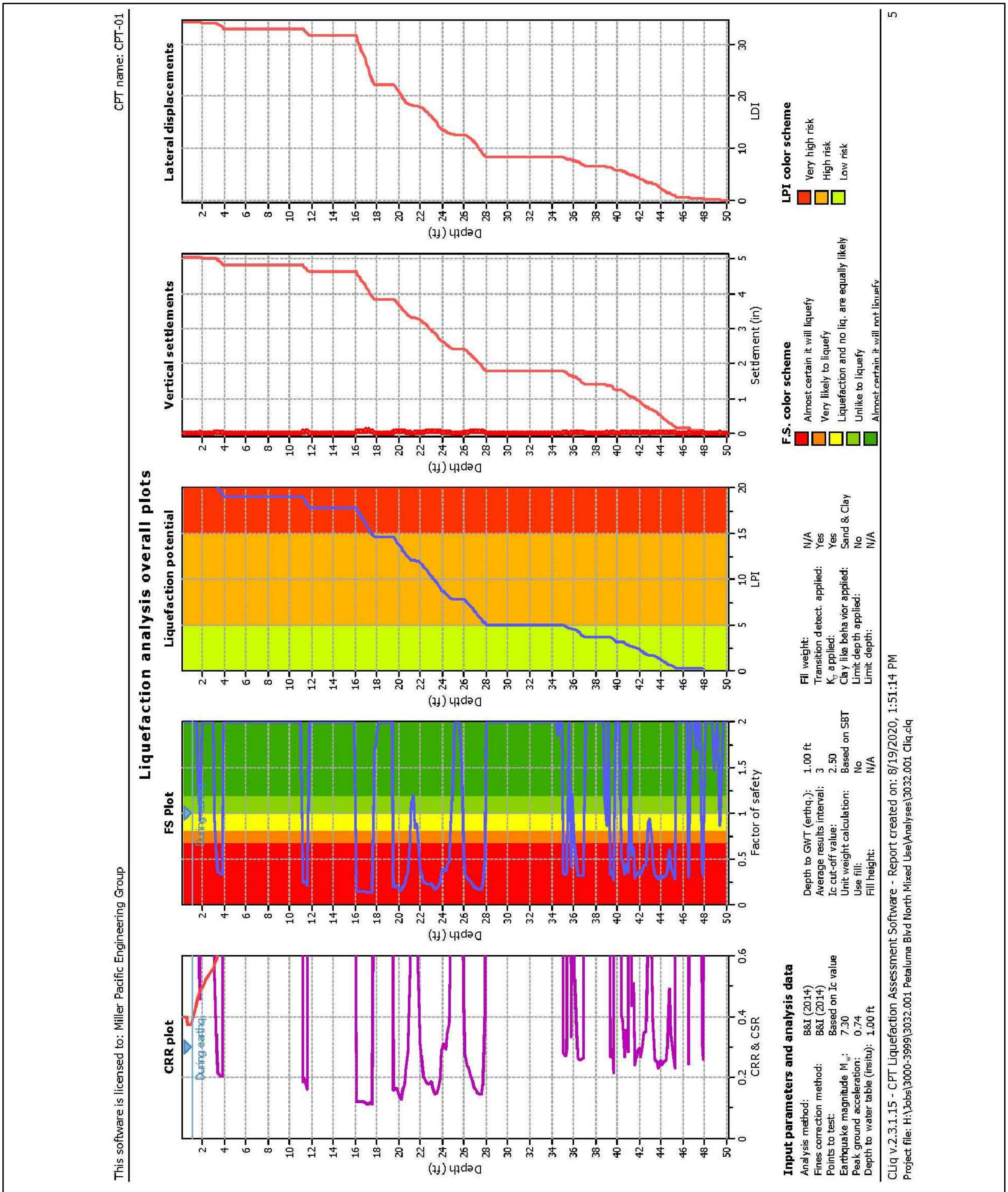
6
FIGURE



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CPT-01 LIQUEFACTION ANALYSIS

Mixed Use Development
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7
FIGURE

CPT name: CPT-02

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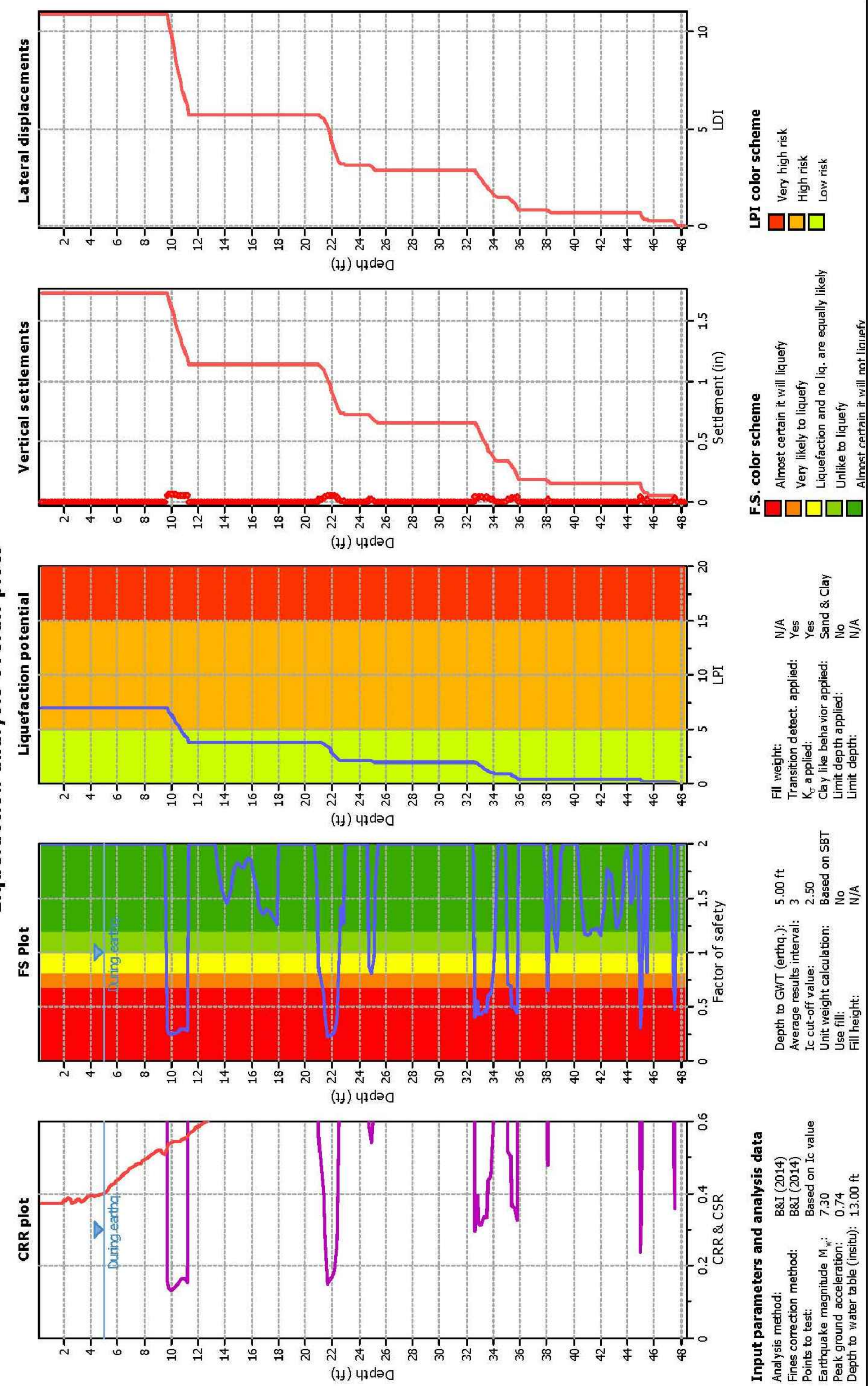
Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.50	K_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.74	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	13.00 ft	Fill height:	N/A	Limit depth:	N/A

CLiq v.2.3.1.15 - CPT Liquefaction Assessment Software - Report created on: 8/19/2020, 1:51:15 PM
Project file: H:\Jobs\3000-3999\3032.001 Petaluma Blvd North Mixed Use\Analyses\3032.001 Cliq.clq

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CPT-02 LIQUEFACTION ANALYSIS

Mixed Use Development
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Drawn MMT
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8
FIGURE

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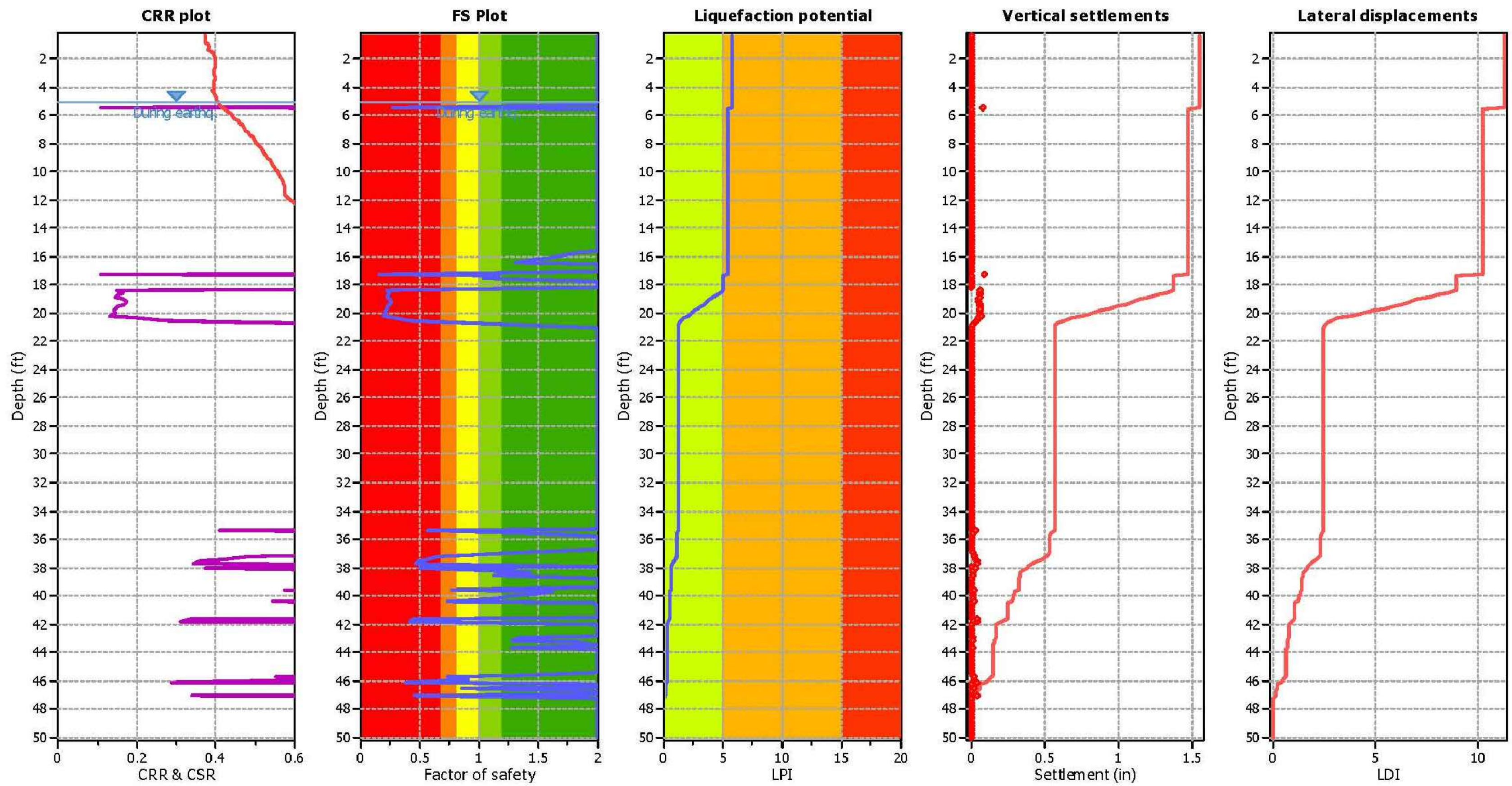
CPT name: CPT-03

Liquefaction analysis overall plots



Input parameters and analysis data

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Points to test:	Based on Ic value	Ic cut-off value:	2.50	K_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.74	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	12.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

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CPT-03 LIQUEFACTION ANALYSIS

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9
FIGURE

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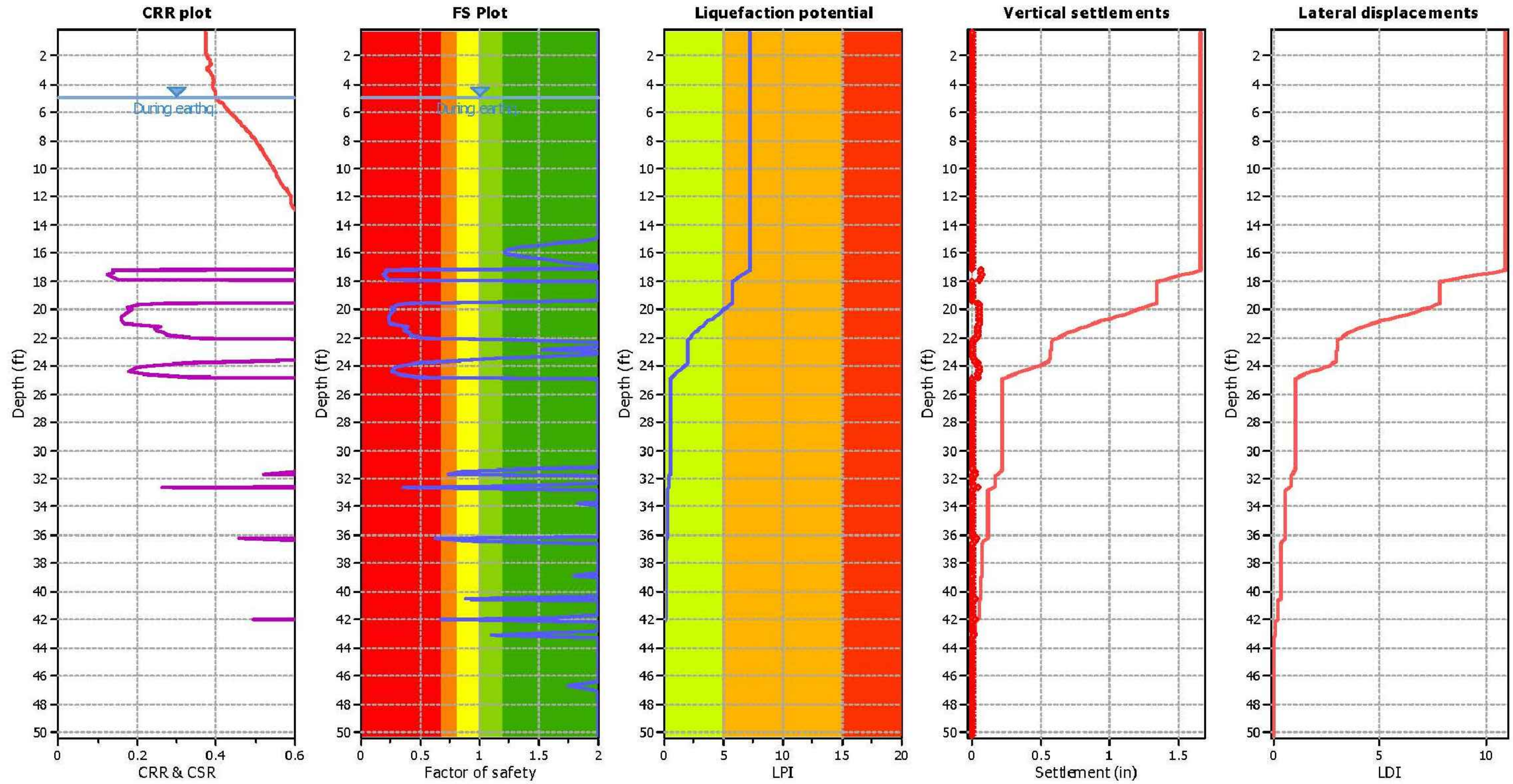
CPT name: CPT-04

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.50	K_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.74	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

CLiq v.2.3.1.15 - CPT Liquefaction Assessment Software - Report created on: 8/19/2020, 1:51:17 PM

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CPT-04 LIQUEFACTION ANALYSIS

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10
FIGURE

APPENDIX A
SUBSURFACE EXPLORATION AND LABORATORY TESTING

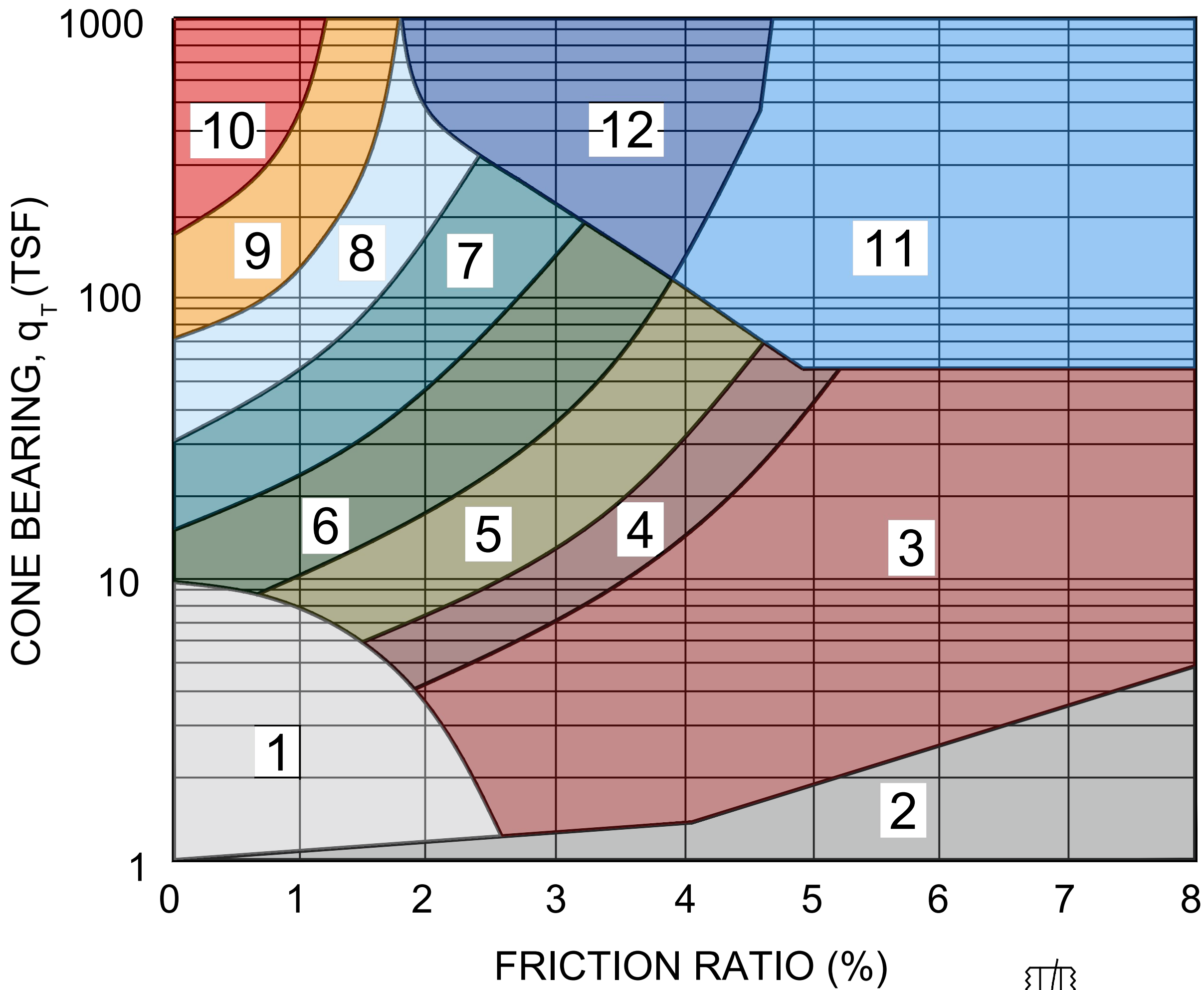
A. SUBSURFACE EXPLORATION

We performed four Cone Penetration Tests (CPT) on August 10, 2020 at the locations shown on the Site Plan, Figure 2A and 2B. The CPT is a special exploration technique that provides a continuous profile of data throughout the depth of exploration. It is particularly useful in defining stratigraphy, relative soil strength and in assessing liquefaction potential.

The CPT is a cylindrical probe, 35 mm in diameter, which is pushed into the ground at a constant rate of 2 cm/sec. The device is illustrated on Figure A-1. It is instrumented to obtain continuous measurements of cone bearing (tip resistance), sleeve friction and pore water pressure. The data is sensed by strain gages and load cells inside the instrument. Electronic signals from the instrument are continuously recorded by an on-board computer at the surface, which permits an initial evaluation of subsurface conditions during the exploration.

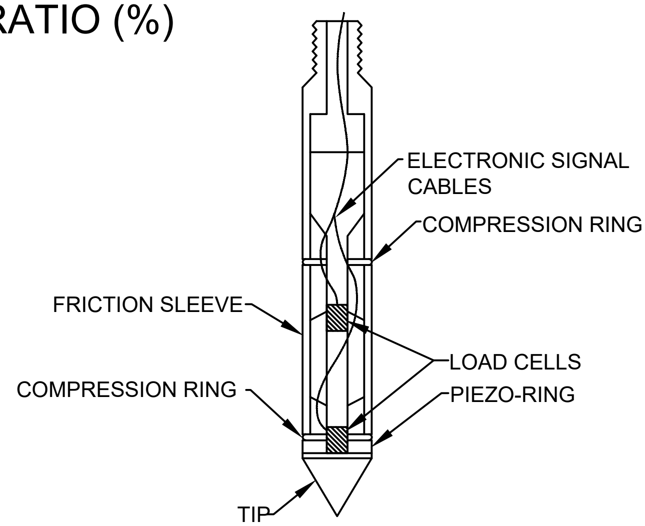
The recorded data is transferred to an in-office computer for reduction and analysis. The analysis of cone bearing and sleeve friction (i.e. friction ratio) indicates the soil type, the cone bearing alone indicates soil density or strength, and the pore pressure indicates the presence of clay. Variations in the data profile indicate changes in stratigraphy. This test method has been standardized and is described in detail by the ASTM Standard Test Method D3441 "Deep, Quasi-Static Cone and Friction Cone Penetration Tests of Soil." The interpretation of CPT data is illustrated on Figure A-1, and the CPT data logs are presented on Figures A-2 through A-5.

The exploratory CPT logs, description of soils encountered, and the laboratory test data reflect conditions only at the location of the CPTs at the time they were excavated. Conditions may differ at other locations and may change with the passage of time due to a variety of causes including natural weathering, climate, and changes in surface and subsurface drainage.



Zone:	Qc/N	Soil Behavior Type:
1)	2	Sensitive Fine Grained
2)	1	Organic Material
3)	1	Clay
4)	1.5	Silty Clay to Clay
5)	2	Clayey Silt to Silty Clay
6)	2.5	Sandy Silt to Clayey Silt
7)	3	Silty Sand to Sandy Silt
8)	4	Sand to Silty Sand
9)	5	Sand
10)	6	Gravelly Sand to Sand
11)	1	Very Stiff Fine Grained (*)
12)	2	Sand to Clayey Sand (*)

(*) Overconsolidated or Cemented



CONE PENETROMETER
(NO SCALE)

Reference: Robertson, P.K. (1986), "In-Situ Testing and Its Application to Geotechnical Engineering," Canadian Geotechnical Journal, Vol. 23; No. 23; No. 4, pp. 573-594

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FILENAME: 3032.001 CPTs.dwg

CPT SOIL INTERPRETATION CHART

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Project No. 3032.001 Date: 9/17/2020

Drawn MMT
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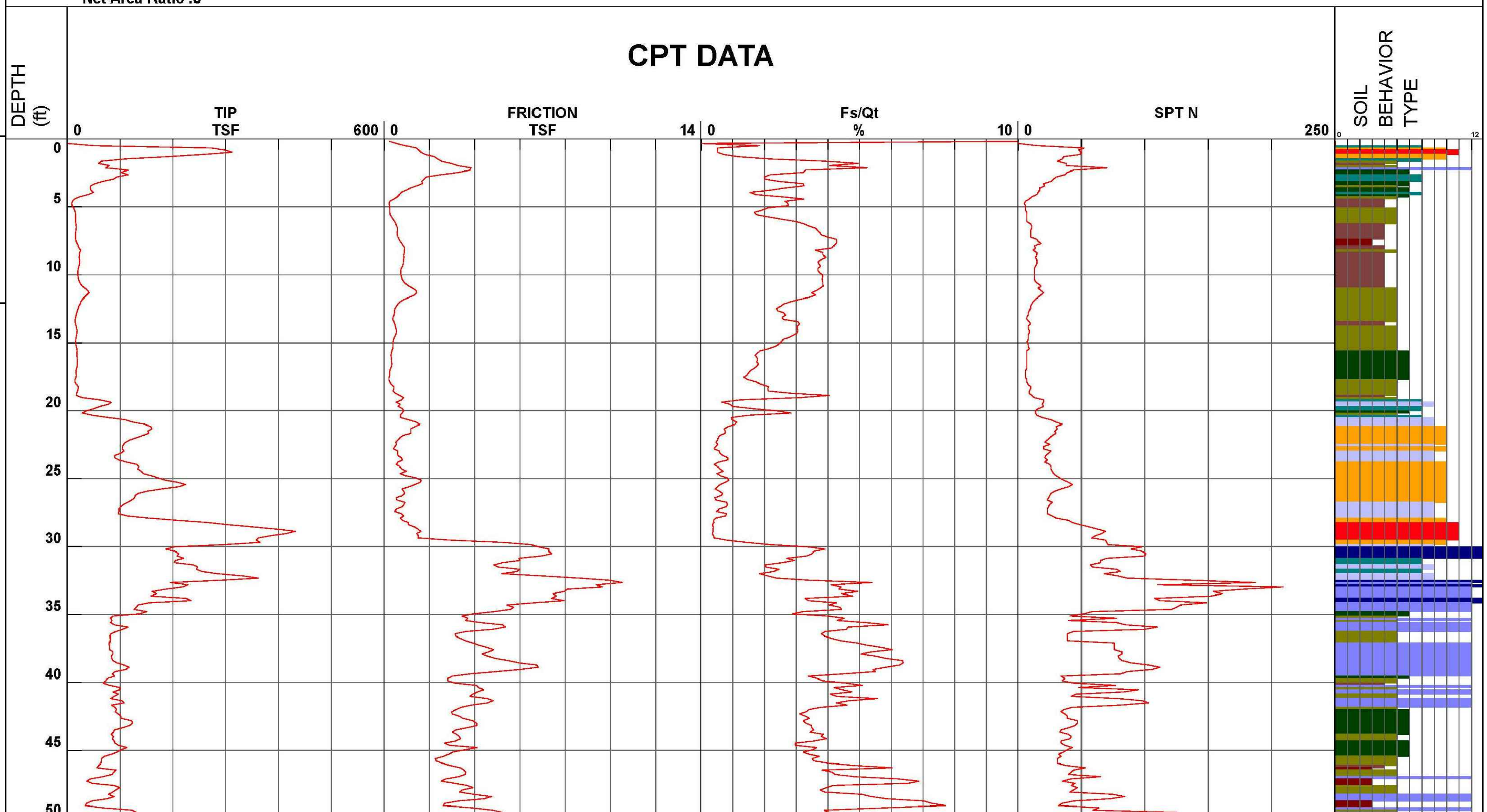
A-1
FIGURE

Miller Pacific Engineering

Project	890 Petaluma Boulevard N	Operator	JM-AJ	Filename	SDF(967).cpt
Job Number	3032.001	Cone Number	DDG1496	GPS	
Hole Number	CPT-01	Date and Time	8/10/2020 8:38:00 AM	Maximum Depth	50.52 ft
EST GW Depth During Test		12.00 ft			

Net Area Ratio .8



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

Cone Size 15cm squared

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

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FILENAME: 3032.001 CPTs.dwg

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CPT-01 DATA

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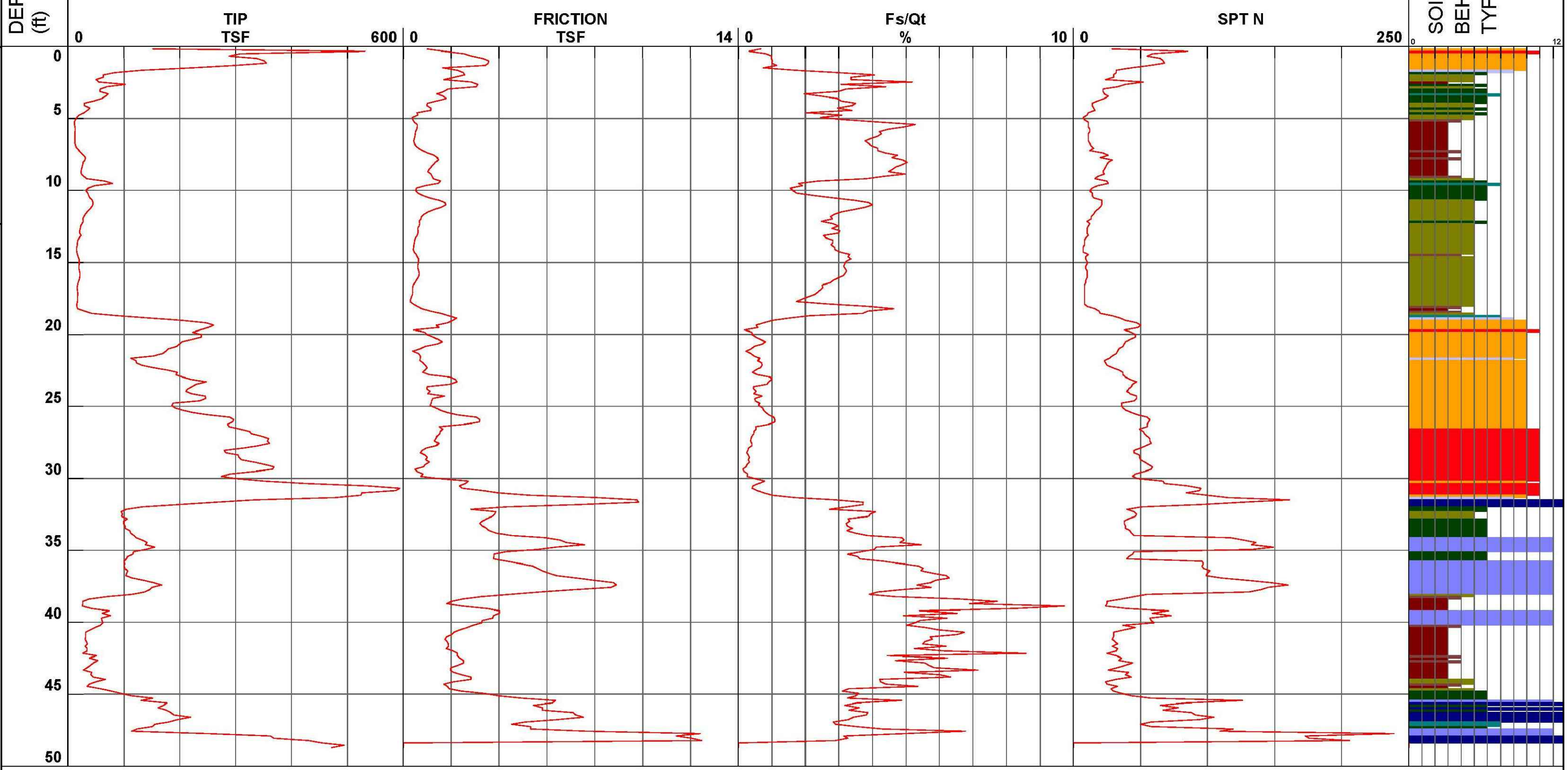
A-2
FIGURE

Miller Pacific Engineering

Project	890 Petaluma Boulevard N	Operator	JM-AJ	Filename	SDF(970).cpt
Job Number	3032.001	Cone Number	DDG1496	GPS	
Hole Number	CPT-02	Date and Time	8/10/2020 11:10:43 AM	Maximum Depth	48.72 ft
EST GW Depth During Test		13.00 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 15cm squared

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

MPEG MILLER PACIFIC ENGINEERING GROUP

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILENAME: 3032.001 CPTs.dwg

CPT-02 DATA

Mixed Use Development
890 Petaluma Boulevard North
Petaluma, California

Project No. 3032.001 Date: 9/17/2020

Drawn MMT
Checked

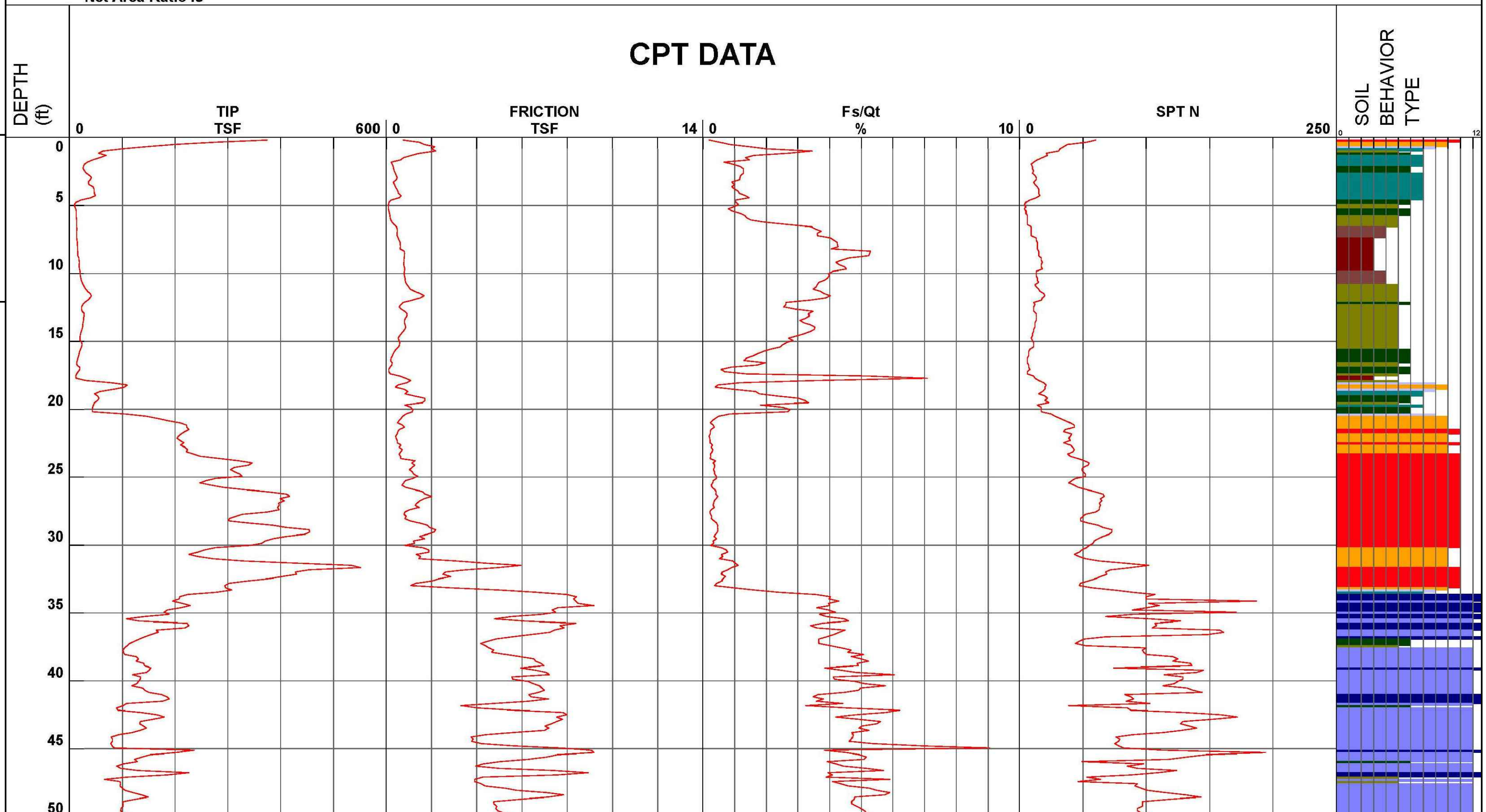
A-3
FIGURE

Miller Pacific Engineering

Project	890 Petaluma Boulevard N	Operator	JM-AJ	Filename	SDF(969).cpt
Job Number	3032.001	Cone Number	DDG1496	GPS	
Hole Number	CPT-03	Date and Time	8/10/2020 10:20:41 AM	Maximum Depth	50.52 ft
EST GW Depth During Test	12.00 ft				

Net Area Ratio .8

CPT DATA



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

Cone Size 15cm squared

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

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FILENAME: 3032.001 CPTs.dwg

CPT-03 DATA

Mixed Use Development
890 Petaluma Boulevard North
Petaluma, California

Project No. 3032.001 Date: 9/17/2020

Drawn MMT
Checked

A-4
FIGURE

Miller Pacific Engineering

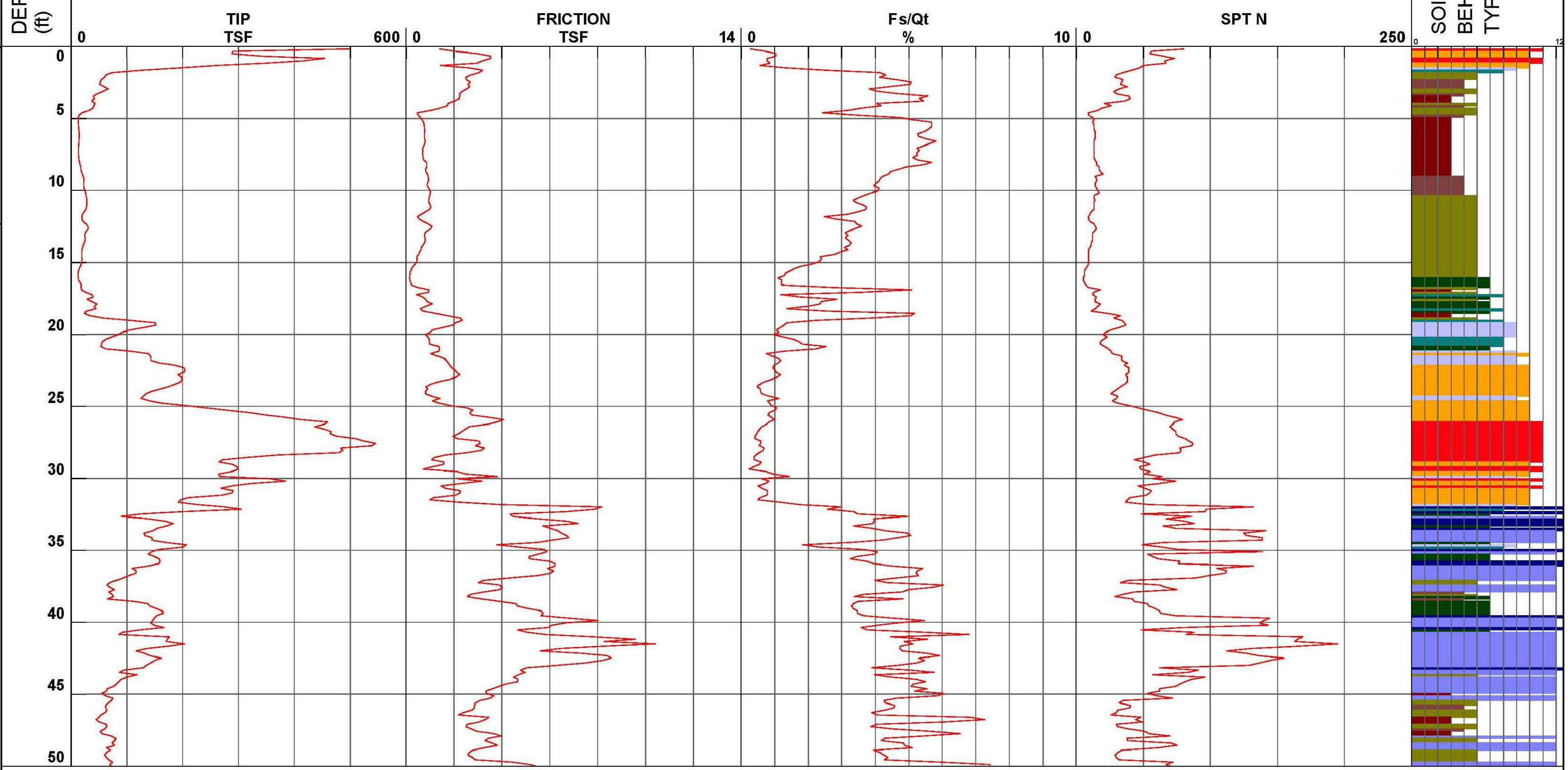
Project: 890 Petaluma Boulevard N
Job Number: 3032.001
Hole Number: CPT-04
EST GW Depth During Test: 10.00 ft

Operator: JM-AJ
Cone Number: DDG1496
Date and Time: 8/10/2020 9:20:17 AM

Filename: SDF(968).cpt
GPS:
Maximum Depth: 50.69 ft

Net Area Ratio .8

CPT DATA



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

Cone Size 15cm squared

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

MPEG MILLER PACIFIC ENGINEERING GROUP

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FILENAME: 3032.001 CPTs.dwg

CPT-04 DATA

Mixed Use Development
890 Petaluma Boulevard North
Petaluma, California

Project No. 3032.001
Date: 9/17/2020

Drawn MMT
Checked

A-5
FIGURE