MILLER PACIFIC ENGINEERING GROUP

PRELIMINARY GEOTECHNICAL INVESTIGATION
THE BCRE PROJECT
2801 PINOLE VALLEY ROAD
PINOLE, CALIFORNIA

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Job No. 3039.001

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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 mixed-use BCRE development located at 2801 Pinole Valley Road. As shown on Figure 1, the project site is located in Pinole Valley, southeast of Highway 80, on a parcel bounded by Pinole Valley Road to the west and other commercial and residential developments to the north, south, and east.

Our work was performed in accordance with our Agreement for Professional Services dated July 6, 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 Pinole, and any geologic/geotechnical background information supplied by you.
- Review of historic aerial photographs to evaluate the history of previous site development.
- Coordination 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 five CPTs that extended through the near-surface soils and into firm and dense bearing materials.
- 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, retaining walls, seismic design, and other geotechnical-related items.
- Preparing a Preliminary Geotechnical Investigation report which summarizes the subsurface exploration, evaluation of relevant geologic hazards, and preliminary geotechnical recommendations and design criteria.

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

2.0 PROJECT DESCRIPTION

Based on our review of preliminary plans and discussions with the project team, we understand that design details are not yet fully developed; however, the project would generally consist of construction a new 5-story apartment building and a new 4-story addition to the existing 2-story office building on the site. Existing parking lots will be reconfigured to accommodate the new structures, and ancillary improvements are expected to include exterior hardscape and asphalt paving, new underground utilities, new site drainage, landscaping, and other improvements "typical" of such developments. A Site Plan showing the approximate extents of the proposed work is presented on Figure 2.

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 by the United States Geological Survey (Dibblee, 1980) indicates the site is underlain by the Holocene (less than 11,000 years old) alluvial deposits. Alluvium is typically composed of unconsolidated gravel, sand, silt, and clay, deposited by fluvial processes. Upland areas to the east are shown as being underlain by a variety of Tertiary-age marine deposits, principally siltstone and sandstone. An inferred/concealed trace of the Pinole Fault Zone is mapped as passing through the site on a northwesterly trend, while a second trace is shown farther northeast with a higher level of confidence. A copy of the Regional Geologic Map is shown 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 Hayward, Contra Costa, and Concord Faults which are located roughly 5.2 kilometers (3.2 miles) west, 11.2 kilometers (6.9 miles) east, and 17.6 kilometers (10.9 miles) east of the site, respectively. We note a concealed trace of the Pinole Fault is also mapped as lying nearly coincident with the eastern property boundary. The seismogenic and surface fault rupture potential associated with the Pinole Fault is discussed in more detail in Section 4.1 of this report.

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; Field et al 2015) 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 Hayward Fault is located approximately 5.2 kilometers (3.2 miles) east of the site and is assigned a probability of 33 percent. The San Andreas Fault, located approximately 33.8 kilometers (21.0 miles) west of the site, is assigned a 22 percent probability of an earthquake with a magnitude greater than 6.7 by 2043. Note that the Pinole Fault, which is mapped along the east side of the site, is not included in the UCERF3 report and as such no probability of future earthquakes on the faut is provided. 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 Review of Reference Materials

In addition to the regional geologic mapping discussed above, we reviewed historic imagery and the results of a subsurface fault trench investigation performed at the nearby Pinole Valley High School campus. The materials are discussed in further detail below.

3.3.1 Historic Aerial Imagery

We reviewed black-and-white aerial photographs procured from Pacific Aerial Surveys of Novato, California. Photographs covered the time period between 1953 and 1968. We also reviewed historic aerial imagery from 1939 available through Google Earth. Brief descriptions of photographs we reviewed are summarized below:

1939 (Date Unknown, Google Earth)
This image shows Pinole Valley largely in its natural state. Pinole Valley Road can be seen in its present alignment, but surrounding lands consist largely of undeveloped, rolling, grassy hills. Notably, a west-flowing tributary of Pinole Creek crosses the northern portion of the site, and second tributary flows just south of the site, with the site situated on a low, east-facing topographic “nose” between the two channels. Areas west of Pinole Valley Road have some structures which may be agricultural barns or similar, while areas to the west contain no discernable man-made features aside from some apparent vehicle or animal tracks on parcels to the south of the site. No significant lineaments, tonal variations, or other evidence indicative of faults or related geomorphology is observed.

August 14, 1953 (AV119-09-09, Scale 1:10,000)
Some new development is apparent at the north end of the image, north of the present-day location of Interstate 80, and a new house is visible between Pinole Creek and Pinole Valley Road just west of the site. The site itself appears to have been graded, including excavation at the north end and apparent fill placement at the south end, as evidenced by realignment of the southern tributary channel observed in the 1939 image. Two small structures appear to occupy the southern part of the site.

May 3, 1957 (AV253-08-07, Scale 1:12,000)
Extensive grading for the new Interstate 80 alignment and interchange at Pinole Valley Road is underway, and continued development north of the highway is apparent. New residential subdivision construction is also underway south of the site, along the east side of Pinole Valley Road. Although a portion of the Estates Drive alignment is visible north of the site, the road does not appear improved and may be a haul road associated with ongoing subdivision construction southeast of the site. Of note, the two structures observed at the site in the 1953 image are gone, and the site appears to be largely dormant, suggesting it may have been a local quarry or borrow area, and possibly a fill source for either the new subdivision to the south or the new highway to the north.

June 30, 1959 (AV334-06-55, Scale 1:9,600)
The Estates Drive alignment has been improved and completed, and residential development is well underway to the east of Estates Drive. Grading also appears to be underway for new commercial development on the east side of Pinole Valley Road at the I-80 interchange. Grading work is also apparent north of the site, where vehicle tracks indicate continued excavation/borrow activities. The upper part of the northern tributary channel has been modified and partly filled by the ongoing development, and a new access road appears to have been graded across the site in a north-south direction, approximately at the present-day location of the rear/eastern parking lot.

April 10, 1968 (AV844-12-22, Scale 1:30,000)
In this image, Interstate 80 and the Pinole Valley Road interchange have been completed and appear in their current alignment. The shopping center at the southwest corner of the interchange has been completed, and the Pinole Valley High School campus south of that has also been developed. Subdivision construction has filled in areas east of Estates Drive, and some multi-family or light commercial development is in place on the east side

of Estates Drive south of the site. Construction also appears to have commenced on the development immediately east of (adjacent to) the site. The site itself remains undeveloped, although a new set of apparent loop-shaped vehicle tracks is visible.

3.3.2 Fault Trench Studies by Others

We also reviewed a Subsurface Fault Investigation Report (Kleinfelder, 2011) prepared for the Pinole Valley High School, located just southwest of the site. The report was prepared as part of a campus redevelopment project and was prompted by existing mapping showing a trace of the Pinole Fault crossing the site. The report describes several hundred feet of exploratory trench along with pedochronological (soil dating) evaluation. The report concludes that "it appears that the school campus is free from active fault traces and that the Pinole Fault trace is most likely situated to the east of Pinole Valley Road, as mapped by Dibblee in 1980."

3.4 Surface Conditions

The project site is composed of a flag-shaped, approximately 1.75-acre parcel located just southeast of Highway 80. The site is bounded to the east, north, and south by existing light commercial and medium-density residential developments and to the west by Pinole Valley Road. The site slopes gently down to the east, with surface elevations ranging from a maximum of about +70-feet in the northeast corner of the parcel to a minimum of about 55-feet along the Pinole Valley Road frontage.

The site is currently developed with a 2-story commercial/office structure sited in the north-central part of the property. The building is constructed with a "daylight" lower floor supported by an internal retaining wall, so as to provide direct vehicle access to the upper floor at the rear/east side of the building. Exterior areas are developed with asphalt-paved parking and vehicle access areas, exterior concrete flatwork, and minor landscaping.

3.5 Field Exploration and Laboratory Testing

We explored subsurface conditions at the site on August 17, 2020 with five cone penetration tests (CPTs) at the approximate locations shown on Figure 2. The Cone Penetration Test (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 and assessing relative soil strength and liquefaction potential for eventual foundation and structural design.

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 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." A schematic diagram of the CPT instrument and a CPT Soil Interpretation Chart are shown on Figure A-1, and CPT plots are shown on Figures A-2 through A-6.

In addition to continuous measurement of soil properties under "static" conditions, shear wave velocity measurements are facilitated at selected intervals via use of an incorporated tri-axial geophone. A steel plate is placed at the ground surface and struck with a hammer; and the shear wave velocity is determined based on the time between which the plate is struck, and the seismic wave arrives at the geophone. For this study, we performed shear-wave velocity measurements on 5-foot intervals at CPT-1 and CPT-5. The results of these tests are shown on Figures A-7 and A-8.

3.6 Subsurface Conditions

Interpreted subsurface conditions are generally consistent with regional geologic mapping discussed above, although some anomalies were observed. CPTs 1 and 2 were located on the eastern, upslope side of the parcel. CPT-1, performed in the northeast corner, penetrated about 10-feet of medium-stiff silty to clayey soils. At a depth of about 12-feet, a significant spike in cone tip resistance and interpreted lithologic change to "very stiff fine-grained" is likely indicative of completely weathered sedimentary bedrock. CPT-1 encountered effective hydraulic refusal at a depth of about 20-feet.

CPT-2 was located in the southeast corner of the property and also penetrated about 10 to 15-feet of silty to clayey soils before encountering a significantly stiffer horizon between 15- and 20-feet. Unlike CPT-1, CPT-2 was advanced to a maximum depth of about 100-feet through "very stiff fine-grained" material without reaching equipment refusal.

CPT-3 was located in the central part of the site and encountered about 35-feet of medium-stiff clayey soils underlain by stiffer fine-grained deposits and was terminated at about 50-feet due to hydraulic refusal. CPTs 4 and 5 each encountered upwards of 60-feet of interbedded medium-stiff clays and silts, and equipment refusal in stiffer materials was not reached until 50- to 65-feet.

Shear-wave velocity measurements performed at CPT-1 in the northwest part of the site indicate the upper 10-feet of the subsurface has a velocity of about 600 feet/sec (typically correlative to unconsolidated soil), while measurements below 10-feet returned values between about 1,700 and 1,800 feet/second (typically correlative to consolidated soils or weak/weathered bedrock). Conversely, measurements through the upper 50-feet of the subsurface in CPT-5 returned values between about 600 and 900 feet/second, suggesting the thickness of unconsolidated soils increases significantly from east to west across the site, ranging from about 10-feet along the eastern property line to a maximum of about 40-feet or more along Pinole Valley Road.

3.7 Groundwater

Pore pressure dissipation tests performed during the CPT testing indicate groundwater at approximately 14 to 20 feet below the ground surface. Note that our exploration and testing was performed during the summer, several months following the most recent rain. A cursory search of the State Water Resources Control Board's Geotracker website indicates that several monitoring wells are present nearby, including at the Arco fuel station just north of the site at 2747 Pinole Valley Road. Available reports associated with ongoing LNAPL and other environmental water-quality monitoring indicate that, as of the first quarter of 2020, depths to groundwater ranged from about 0- to 99-feet. Reports suggest that the Arco site is underlain primarily by fractured bedrock which locally impedes subsurface flow, and that "perched" groundwater locally exists at several depth intervals.

Based on our preliminary exploration and research, we recommend that for design purposes, highest historic groundwater be taken as a depth of 10-feet below the surface.

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 settlement, strong seismic ground shaking, liquefaction, and seismic densification. 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

As noted previously, regional geologic mapping (Dibblee, 1980) indicates that a concealed surface trace of the Pinole Fault lies nearly coincident with the eastern property boundary, as shown on Figure 3.

Previous research (Williams et al, 1995), which included very-high resolution (VHR) imaging of Holocene sediments beneath San Pablo Bay, evaluated structural characteristics and activity implications along submerged portions of the southern Rodgers Creek and northern Pinole Fault traces. Following identification of vertical fault offsets in young bay sediments, subsequent core drilling and soil/fossil sample recovery resulted in age-dating of the most recent apparent rupture on the northern portion of the Pinole Fault. The report concludes that “multiple vertical fault offsets are imaged in Holocene strata, and demonstrate that the Pinole Fault, formerly believed inactive, produced two major earthquakes during an 1800-year interval between about 800 and 2600 (radiocarbon) years before present. . . The Pinole Fault ruptures thus appear to have an average recurrence interval of ca. 900 years, and a latest detected event about 800 years ago.”

More recent work by USGS (Phelps et al, 2008) provides a 3-dimensional geologic map of the larger Hayward Fault Zone, prepared via compilation of previous surface mapping, remote-sensed imagery, and subsurface geophysical investigations. Notably, Williams’ 1995 study described above is not referenced. The USGS concludes that “the Palomares-Miller Creek-Moraga-Pinole Fault System . . . has accommodated 50km of total slip, all within the past 10 million years. . . between 6 and 10 million years ago, the Palomares-Miller Creek-Moraga-Pinole Fault system was more active than the Hayward Fault, but currently it is not considered one of the active fault systems in the San Francisco Bay Area.” The report does not provide any significant evidence indicating activity has ceased since 6 million years ago and contradicts William’s 1995 conclusion that fault rupture has occurred during Holocene time.

Also, as previously discussed, Kleinfelder’s 2011 work at Pinole Valley High School included extensive subsurface exploration and concluded that the Pinole Fault likely lies “east of Pinole Valley Road”, near the site location.

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 (zoned) fault to the site is the Hayward Fault located approximately 5.2 kilometers (3.2 miles) to the east. The site is not currently located in an Alquist-Priolo Earthquake Fault Zone.

Regardless, based on the apparently limited research focusing on this fault trace, there is convincing evidence of Holocene activity on the submerged, northern segment of the fault. Although no additional significant evidence of active faulting was observed during our site reconnaissance or review of historic documents, we judge that the combination of existing published mapping, previous academic/scientific research, and Kleinfelder’s positing of the fault’s location as mapped by Dibblee through the site suggests, barring new evidence to the contrary, that at least a moderate risk of fault surface rupture may exist at the site.

Evaluation: *Less than significant with mitigation.*
Recommendation: *A subsurface Fault Trench Investigation should be performed as part of a future design-level Investigation to determine whether active faults exist at the site and to develop appropriate setbacks for new structures as warranted, in accordance with the intent of the Alquist-Priolo Act.*

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[1]	Closest Estimated Distance (km)	Median Peak Ground Acceleration (g)[2,3]	84% Peak Ground Acceleration (g)[2,3]
Hayward	7.3	5.2	0.42	0.70
Contra Costa	6.5	11.2	0.26	0.44
San Andreas	8.0	33.8	0.21	0.36
Rodgers Creek	7.3	24.0	0.20	0.34
Concord	6.6	17.6	0.19	0.33

1.) Values determined using Google Earth KML Files showing Quaternary Faults & Folds in the US obtained from USGS website September 17, 2020.
2.) Values determined using Vs^{30} = 260 m/s for Site Class "D" ("Stiff Soil" Conditions) in accordance with the 2019 CBC and 2016 ASCE-7. See additional discussion regarding Site Class determination and preliminary recommended seismic design criteria in Section 5.1.
3.) Values determined using Pacific Earthquake Engineering Research Center (PEER) NGS-West2 Excel Spreadsheet, http://peer.berkeley.edu/ngawest2/databases/
4.) Note that, although not included in the current database and assigned a maximum credible magnitude value, the Pinole Fault, if active, may also generate very strong ground shaking.

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	6.97	0.99
10% in 50 years	475 years	6.89	0.59

Reference: USGS Unified Hazard Tool accessed on September 17, 2020.

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 Hayward and Contra Costa Faults present the highest potential for severe ground shaking. The Pinole Fault, although not currently considered an active fault by CGS/USGS, may also pose some unknown seismogenic potential and cause significant 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: *Minimum mitigation includes 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 Liquefaction and Related Effects

Liquefaction refers to the sudden, temporary loss of soil strength during strong ground shaking. The strength loss occurs as a result of the build-up of excess pore water pressures and subsequent reduction of effective stress. While liquefaction most commonly occurs in saturated, loose, granular deposits, recent studies indicate that it can also occur in materials with relatively high fines content provided the fines exhibit lower plasticity.

The effects of liquefaction can vary from cyclic softening resulting in limited strain potential to flow failure which cause large settlements and lateral ground movements. Lateral spreading refers to a specific type of liquefaction-induced ground failure characterized primarily by horizontal displacement of surficial soil layers due to liquefaction of a subsurface granular layer (Youd, 1995). Lateral spreads generally move down gentle slopes or slip toward a free face such as an incised river channel.

As shown on Figure 6, regional mapping of seismic hazard zones (ABAG, 2020) indicates the site lies within a zone of “moderate” liquefaction potential. CPTs 1 through 3 encountered isolated horizons of saturated, granular materials, each about a foot thick, at various depths, while CPTs 4 and 5 encountered thicker deposits of clayey materials with a few intervals of sandy silt. We analyzed the potential for liquefaction utilizing the software program Cliq 2.0, developed by Geologismiki (2006) in conjunction with methods prescribed by Idriss and Boulanger (2014). Our analysis is based on an estimated maximum groundwater elevation of 10-feet below the ground surface and considers an M=7.3 earthquake on the Hayward Fault producing peak ground acceleration of 0.42g per our deterministic seismic analysis.

The results of our liquefaction analyses for CPT-1 indicate sand layers at depths of 10- to 11-feet and 15- to 16-feet may liquefy under a strong seismic event. CPT-2 indicates sand layers at depths from 17- to 18-feet and 49- to 50-feet may liquefy. CPT-3 indicates sand layers at depths of 10-to 11-feet and 43- to 44-feet may liquefy. CPT-4 indicates several sandy to clayey silt layers at depths of 14-to 35-feet and 48- to 52-feet may liquefy. CPT-5 indicates several sand layers at 28- to 48-feet may liquefy. Our analysis further indicates that if liquefaction does occur, post-liquefaction settlements may range from a few 1/10ths of an inch in the eastern part of the site to 4-inches in the western portion. Note that liquefiable horizons at depths exceeding 40-feet in CPTs 2 and 3 may be erroneously interpreted and could be weathered sandstone bedrock horizons which are in fact non-liquefiable.

Additionally, we calculated 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 LPI plots generated by our modeling software indicates that the liquefaction potential index is greater than 15, suggesting a moderate to high probability that the effects of liquefaction will manifest at the surface in the form of sand boils, differential settlement, and other phenomena.

Evaluation: *Less than significant with mitigation.*

Recommendation: *Liquefaction and differential settlement estimates should be refined on the basis of soil borings, laboratory testing, and additional analysis undertaken as part of a future design-level investigation. On a preliminary basis, if liquefaction is confirmed to present a significant risk to the project, we recommend the new structures be supported on deep foundation systems which derive their capacity entirely from non-liquefiable materials. Additional discussion regarding foundation options is presented in Section 5 of this report.*

4.4 Settlement

Significant settlement can occur when new loads are placed over soft, compressible clays or loose granular soils. The site is underlain by between about 10- and 40-feet of medium-stiff clayey and silty soils, and soft, compressible clays were not indicated by CPT data. We judge the risk of significant consolidation settlement at the site is generally low, pending confirmation via future soil borings and laboratory testing.

Evaluation:	*Less than significant.*
Recommendation:	*Settlement estimates should be refined on the basis of soil borings and laboratory testing performed as part of a future design-level Investigation. The use of deep foundations, as may be required for mitigation of liquefaction and seismically induced settlement mitigation, would also suitably mitigate the risk of consolidation settlement.*

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 data, subsurface soils above groundwater level are generally classified as sandy clays in the fill material. Therefore, the risk of seismic densification impacting the new structures is low.

Evaluation:	*Less than significant.*
Recommendation:	*Dry sand settlement estimates should be refined on the basis of soil borings and laboratory testing performed as part of a future design-level Investigation. The use of deep foundations, as may required for mitigation of liquefaction and seismically induced settlement mitigation, would also suitably mitigate the risk of densification.*

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.

Our CPT exploration indicates that the near-surface soils are generally granular, suggesting a low expansion potential. During our site reconnaissance, we noted localized and relatively minor apparent differential heave and settlement of the older asphalt parking lot areas, though some of the distress appears as though it may be related to utility backfill settlement and tree root heave/intrusion. Therefore, the risk of expansive soil affecting the proposed improvements appears low to moderate.

Evaluation:	*Less than significant.*
Recommendation:	*Evaluation should be confirmed on the basis of supplemental subsurface exploration and laboratory testing undertaken as part of a future design-level Investigation.*

4.7 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.

The project will result in little exposed soil, and as such the risk of significant erosion is judged to be low. However, new grading may result in altered drainage patterns around the site.

Evaluation: *Less than significant with mitigation.*
Recommendation: *The project Civil Engineer should design a site drainage system for the maximum credible rainfall event, which is capable of collecting runoff and discharging it at a location unlikely to result in significant erosion, ideally an established storm drain system. The site storm drain system should be designed to accommodate runoff associated with the maximum credible rainfall event. An erosion control plan could be developed prior to construction per the current guidelines of the California Stormwater Quality Association's Best Management Practice Handbook.*

4.8 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 site, therefore the risk of lurching and ground cracking at the project site is low.

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

4.9 Slope Instability/Landsliding

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

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

4.10 Flooding

The project site is located at about elevation +55 to +65 feet and the site is not mapped within a designated 100- or 500-year flood zone based upon the preliminary Flood Insurance Rate Map prepared by FEMA (Federal Emergency Management Agency, 2015). Therefore, we judge the risk of large-scale flooding at the site is generally low.

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. The project site lies about 1.3-miles inland of San Pablo Bay at elevations above about +55-feet. Tsunami hazard mapping of the project area (ABAG, 2020) indicates the site is not located within a tsunami inundation zone. Therefore, the likelihood of inundation by seiche or tsunami is low.

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

4.12 Dam Failure Inundation

Based on the City of Pinole Potential Geologic Hazards Map (City of Pinole, 2010) the site is not mapped in a Dam Failure Inundation zone. Therefore, the threat of inundation of the site from dam failure is judged low.

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

4.13 Soil Corrosion

Corrosive soil can damage buried metallic structures, cause concrete spalling, and deteriorate rebar reinforcement. In general, the sedimentary bedrock which underlies most of the Pinole Creek watershed does not weather to significantly corrosive soils. However, the site is located in an area of previous commercial development, and hydrothermal or other chemical alteration of soil and rock materials at the site (such as may be associated with the Pinole Fault) could result in corrosive soils. Therefore, we judge the risk of corrosive soils at the site is low to moderate, pending future testing and verification.

Evaluation: *No significant impact.*
Recommendation: *Corrosive soil potential should be evaluated on the basis of soil borings and laboratory testing undertaken as part of a future design-level Investigation. Mitigation for corrosive soils would typically include avoiding metallic piping, providing cathodic protection as warranted, and specifying special cement and concrete mixes.*

4.14 Radon-222 Gas

Radon-222 is a product of the radioactive decay of uranium-238 and raduim-226, which occur naturally in a variety of rock types, mainly phosphatic shales, but also in other igneous, metamorphic, and sedimentary rocks. While low levels of radon gas are common, very high levels, which are typically caused by a combination of poor ventilation and high concentrations of uranium and radium in the underlying geologic materials, can be hazardous to human health.

The project site is located in Contra Costa County, California, which is mapped in radon gas Zone 2 by the United States Environmental Protection Agency (USEPA, 2020). Zone 2 is classified by the EPA as exhibiting a “moderate” potential for Radon-222 gas with average predicted indoor screening levels ranging from 2 to 4 pCi/L. Therefore, the potential for hazardous levels of radon at the project site is moderate.

Evaluation: *No significant impact with mitigation.*
Recommendation: *Installation of a vapor barrier (as would be required to reduce moisture transmission into interior spaces) should be considered to reduce the risk of radon exposure.*

4.15 Volcanic Eruption

Several active volcanoes with the potential for future eruptions exist within northern California, including Mount Shasta, Lassen Peak, and Medicine Lake in extreme northern California, the Mono Lake-Long Valley Caldera complex in east-central California, and the Clear Lake Volcanic Field, located in Lake County approximately 72 miles north of the project site. The most recent volcanic eruption in northern California was at Lassen Peak in 1917, while the most recent eruption at the nearest volcanic center to the project site, the Clear Lake Volcanic Field, was about 10,000 years ago. All of northern California's volcanic centers are currently listed under "normal" volcanic alert levels by the USGS California Volcano Observatory (USGS, 2019a). While the aforementioned volcanic centers are considered "active" by the USGS, the likelihood of damage to the proposed improvements due to volcanic eruption is generally low.

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

4.16 Naturally Occurring Asbestos (NOA)

Asbestos is a generic term for a group of naturally occurring fibrous minerals which, when airborne, can be hazardous to the respiratory system. Crystals of asbestiform minerals may become liberated from the host rock and airborne during crushing or grading operations. Asbestiform minerals are regulated by the California Environmental Protection Agency, Air Resources Board (CARB).

The most common asbestos mineral is chrysotile, although other minerals such as tremolite and actinolite take the same fibrous crystal form. While relatively rare in occurrence, these minerals are commonly associated with ultramafic and related metamorphic rocks. These most often include serpentinite but may also include low- to high-grade schists such as chlorite schist and blueschist.

The project site is underlain by alluvial soils and weathered sedimentary bedrock, and no evidence indicative of ultramafic or related rock types was observed at the site or in surrounding areas. Therefore, the likelihood that significant quantities of NOA exist at the site is remote.

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

4.17 Hazardous Materials

While environmental testing for hazardous materials was beyond the scope of our services, we did observe enclosures containing HVAC units and other industrial equipment may contain hazardous materials. The site appears to have previously been utilized as a quarry for a period of time, appears to have been the site of several episodes of undocumented filling, and groundwater records from the nearby Arco station indicate ongoing monitoring for the presence of environmental contaminants. Therefore, we judge the potential for hazardous materials being present on the project site, currently or in the future, is moderate.

Evaluation: *Less than significant with mitigation.*
Recommendation: *The site should comply with all local, state, and federal guidelines to minimize the exposure to hazardous materials. We recommend that the potential for hazardous materials and other environmental contaminants be assessed by a suitably qualified professional.*

5.0 PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our investigation, we conclude the site conditions are suitable for the proposed improvements. The primary geotechnical considerations for the project will include determining whether active faults exist at the site and, if so, providing adequate setbacks for new structures; designing the improvements to resist strong seismic ground shaking; and providing uniform foundation support and adequate mitigation for potential seismically-induced differential settlements.

As previously noted, this report is preliminary in nature and additional subsurface exploration, laboratory testing, and engineering evaluation will be required to provide specific future settlement estimates and to develop design-level criteria for new foundations and other geotechnical items. A subsurface Fault Trench Investigation should also be performed prior to final design to assess the potential for fault surface rupture and comment on the suitability of the planned building envelopes and foundation designs. Preliminary geotechnical recommendations and development guidelines are presented in the following sections to aid in project planning and development.

5.1 Seismic Design

The project site is located in a seismically active area. New structures should be designed in conformance to the seismic provisions of the California Building Code (CBC) to mitigate the potential effects of strong seismic ground shaking to the proposed structures. As previously discussed, the site is underlain by liquefiable soils, thus, the site should be classified as a Site Class “F”. Per the provisions outlined in the CBC, if the proposed structures have a natural period of 0.5 seconds or less, then the site may be classified as either Site Class “D” or “E”. If structures will have a fundamental period in excess of 0.5-seconds, a site-specific seismic analysis will be required. Note also that the discovery of active faults at or near the site would also require site-specific seismic design analysis. The values presented in Table 3 for short-period structures should be confirmed based on supplemental exploration.

Table 3 – Preliminary 2019 California Building Code Seismic Design Criteria

Parameter	Design Value
Site Class	D
Site Latitude	37.9941°N
Site Longitude	-122.2849°W
Spectral Response (short), S_S	1.985 g
Spectral Response (1-sec), S_1	0.751 g
Site Coefficient, F_a	1.2
Site Coefficient, F_V	n/a

Reference: USGS US Seismic Design Maps, accessed on August 24, 2020.

5.2 Site Grading

Significant grading (cut or fill placement) is not expected at the project site. 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 pavements, old foundations, over-sized debris, and organic material from areas to be graded. Debris, rocks larger than six inches, and vegetation are not suitable for structural fill and should be removed from the site. Existing foundations and utilities which are to be abandoned as part of the work should be removed from structural areas. In non-structural areas, utilities could be abandoned in place provided cement grout completely fills any void in the utility.

Where fills or other structural improvements are planned, the subgrade surface should be scarified to a depth of 8 inches, moisture conditioned to above the optimum moisture content, and compacted to at least 90 percent relative compaction. Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density, as determined by ASTM D1557. Subgrade preparation should extend a minimum of 5 feet beyond the planned building envelope in all directions. The subgrade should be firm and unyielding when proof-rolled with heavy, rubber-tired construction equipment. If soft, wet or otherwise unsuitable materials are encountered at subgrade elevation during construction, we can provide supplemental recommendations to address the specific condition.

5.2.2 Excavations

Based on our subsurface exploration, site excavations will encounter a combination of loose to medium dense sand and medium stiff clay in the upper 10 feet. Groundwater should be anticipated below a depth of about 10-feet. The native soils generally classify as OSHA Type “C” soils. The sands may exhibit running behavior above the groundwater table and flowing behavior below the groundwater table when exposed in unsupported excavations.

Where excavations exceed 5-feet and cannot be sloped in accordance with OSHA regulations, temporary support will be required to ensure the safety of workers and to reduce the potential for failure of the excavation sidewalls and damage to surrounding improvements. Excavation stability and the structural design of temporary shoring should be made the sole responsibility of the Contractor. For excavations deeper than 10-feet, the design of temporary dewatering systems should be made the sole responsibility of the Contractor.

5.2.3 Fill Materials, Placement and Compaction

Fill materials should consist of non-expansive materials that are free of organic matter, have a Liquid Limit of less than 40 (ASTM D 4318), a Plasticity Index of less than 20 (ASTM D 4318), and a minimum R-value of 20 (California Test 301). The fill material should contain no more than 50 percent of particles passing a No. 200 sieve and should have a maximum particle size of 4 inches. Onsite soils may be suitable for use as fill provided, they meet the criteria specified above. Any imported fill material needs to be tested to determine its suitability.

Fill materials should be moisture conditioned to above the optimum moisture content prior to compaction. Properly moisture conditioned fill materials should subsequently be placed in loose, horizontal lifts of 8 inches-thick or less and uniformly compacted to at least 90 percent relative compaction. Where fill thicknesses are greater than 5 feet, fill materials should be compacted to at least 92 percent relative compaction. In pavement areas, the upper 12 inches of fill should be compacted to at least 95 percent relative compaction. The maximum dry density and optimum moisture content of fill materials should be determined in accordance with ASTM D1557.

5.3 Preliminary Foundation Design

The preliminary drawings indicate new buildings may be up to five stories in height. The new structures are expected to induce moderate foundation loads. The site is underlain by variable thick alluvial deposits which appear to range from about 10- to 40-feet thick or more. As discussed previously, differential settlements of up to 4-inches may be possible during a seismic event; however, the magnitude of predicted settlements will need to be refined based on additional soil borings, laboratory testing and evaluation of planned building loads. If refined settlement estimates are slightly to significantly lower, then shallow foundations (such as heavy concrete mat or post-tensioned slabs) are probably appropriate for the project – if predicted differential settlements are confirmed via future soil borings and laboratory testing, then a deep foundation system will be recommended. Various potential foundation systems, assuming refined settlement estimates remain unchanged, are discussed below.

Deep Foundations

A deep foundation system which derives its support from weathered bedrock below any potentially liquefiable soils should provide good long-term performance with minimal anticipated settlement. There are several alternative deep foundation systems which may be considered for the project, including traditional cast-in-drilled-hole (CIDH) concrete piles, displacement auger-cast piles (ACP), driven concrete or steel piles, and “torque-down” piles (TDP). While each system should provide comparable performance and settlement mitigation, variations in cost and constructability should be considered during selection of the “preferred” system. For preliminary design, we anticipate that deep foundation elements may develop capacities on the order of 70-kips each at depths between about 25- and 70-feet (east to west). Higher capacities can be achieved at increased depths.

Traditional CIDH pier excavations may need to be cased or slurry supported to avoid collapse given the relatively shallow water table and potential for unpredictable zones of loose granular soils susceptible to caving. Additionally, CIDH pier construction will result in the need for off haul and disposal of drilling spoils which, depending on the results of environmental testing performed for the project, may incur additional hazardous material testing, transport, and disposal costs.

Full or partial displacement auger-cast piles (ACP) avoids this complexity by effectively using the hollow drill auger to laterally displace soils during drilling, provide temporary support of soils and pumping concrete directly into the excavation as the auger is withdrawn. ACP requires the use of specialty equipment. While not as much as CIDH construction, ACP construction will still result in the need for some spoils off haul.

Driven piles, which do not require spoils off haul or stabilization of excavations, have been successfully utilized on projects of similar scale in the area. Given the site’s location in a central business district and the noise, vibration, and general disturbance inherent to the pile-driving process needs to be considered.

Torque-down piles (TDP) consists of full-displacement large-diameter welded steel pipe fitted with a proprietary conical tip and helical auger flight. The pile is effectively “screwed” into the ground by a large drilling rig until such depth is reached that the required pile load capacity is achieved.

Vertical capacity is achieved via a combination of skin friction and end bearing, while lateral capacity is gained through the bending moment of the pipe. Lateral capacity may be increased by filling the TDP with concrete upon installation. The main advantages of the system include the avoidance of constructability considerations such as excavation casing, de-watering, noise, and vibrations.

Shallow Foundations with Ground Improvement

As an alternative to deep foundations in the event that predicted differential settlements remain as stated herein following future laboratory testing, ground improvement may be considered as a means of allowing the use of shallow foundations. Ground improvement options could include Drill Displacement Columns (DDC), which involve drilling to design depths and injecting Controlled Low Strength Material (CLSM) under pressure to create large diameter, well-defined compaction columns that effectively increase soil strength. DDC does not structurally connect to the foundation; as such, structures would need to incorporate shallow foundations. Other ground-improvement options include deep soil-cement mixing (DSCM), whereby Portland cement is mixed continuously with soil to create overlapping columns if increased strength, and “geopiers” or stone columns, where columns of dense aggregate are created by “vibro-replacement”, which reinforces soft native soils.

Based on our experience with similar projects, we judge that, given the scope of the proposed development, it is likely that differential settlement can be reduced by implementation of one of the above ground-improvement techniques. Bearing capacities on improved ground would be on the order of 5,000 psf, as opposed to about 2,000 psf on unimproved ground. Pursuit of a ground-improvement scheme would require supplemental laboratory testing and significant quality-control testing during site grading and construction.

Upon selection of the “preferred” foundation system, we can consult with the project Structural Engineer and provide additional recommendations and design criteria as needed for foundation construction. The foundation design for new structures will primarily depend on building loads and layouts, and the results of supplemental exploratory boring and laboratory testing.

5.4 Retaining Walls

Retaining walls are not currently anticipated for the development. If needed, basement walls and site retaining walls should be preliminarily designed to resist lateral pressures from earth and surcharge loads, as shown in Table 5. Retaining walls that can slightly deflect at the top can be designed using the unrestrained criteria shown below. Walls that are structurally connected and not allowed to deflect (e.g., tied-back walls) are restrained and are commonly designed using a uniform active earth pressure distribution rather than an equivalent fluid pressure.

Table 5 – Preliminary Active Earth Pressure for Retaining Wall Design

Backfill Inclination[1]	Unrestrained[2,3]	Restrained[3,4]
Level	40 pcf	30 x H psf
3:1	50 pcf	35 x H psf
2:1	60 pcf	40 x H psf

Notes:
(1) Interpolate earth pressures for intermediate slopes
(2) Equivalent fluid pressure
(3) Wall design should account for a seismic surcharge of 15 x H (in psf) in addition to active pressure
(4) Rectangular distribution, H is wall height in feet

Wall drainage is required for all retaining walls taller than 3 feet. Wall drainage should consist of Caltrans Class 1B permeable material within filter fabric or Caltrans Class 2 permeable material. A composite drainage panel such as Miradrain 6000 (or approved equivalent) could also be used. The drainage should be collected in a 4-inch perforated PVC drain line at the base of the wall and discharged to an appropriate discharge location. The permeable material should extend at least 12 inches from the back of the wall and be continuous from the bottom of the wall to within 12 inches of the ground surface.

5.5 Interior Concrete Slabs-On-Grade

Reinforced concrete slab-on-grade floors are judged to be appropriate for the site conditions. The concrete slabs-on-grade may be poured monolithically or separated with a cold joint. We recommend that interior concrete slabs have a minimum thickness of 5 inches and be reinforced with steel reinforcing bars (not mesh) with rebar extending through crack control joints. Slabs should be placed on a moist subgrade to reduce potential for future expansive behavior. The project Structural Engineer should specifically design the concrete slabs, including locations of crack control joints.

To reduce the potential for moisture to move upward through the slab, a four-inch layer of clean, free draining, ¾-inch angular gravel should be placed beneath interior concrete slabs to form a capillary moisture break. The gravel 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, should be placed over the compacted base rock. The vapor barrier shall meet the ASTM E1745 Class A requirements and be installed per ASTM E1643. 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.

We note that over time, placing sand between the vapor barrier and concrete is becoming less common because of elevated interior moisture contents. If sand is used, it should be dry, and if it is not used, the slab should be carefully designed with a lower water-cement ratio (generally less than 0.45) since eliminating the sand can cause cracking or "curling" of the new concrete. For slabs that are not sensitive to moisture vapor, we recommend at least four inches of Class 2 aggregate base (Caltrans, 2015) compacted to 95 percent relative compaction.

5.6 Exterior Concrete Slabs

Exterior concrete walkway slabs and other concrete slabs that are not subjected to vehicle loads should be a minimum of 4 inches thick and underlain with 4 inches or more of Class 2 aggregate base. The aggregate base 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 base 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 5 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.

5.7 Site and Foundation Drainage

New grading could result in adverse drainage patterns causing water to pond around the development. 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 area and outlets should be designed to reduce erosion. Site drainage improvements should be connected into an established storm drainage system.

5.8 Underground Utilities

Excavations for utilities will generally encounter a combination of loose to dense sand and soft to medium stiff 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.

5.9 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 for subgrade soils 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 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 6.

Table 6 – Preliminary Asphalt-Concrete Pavement Sections

Traffic Index[1]	Asphalt Concrete (inches)	Aggregate Base (inches)
4.0	3.0	7.0
5.0	3.5	8.0
6.0	4.5	10.5
7.0	5.0	13.0

(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 (soil borings and exploratory trenches) and laboratory testing will be required once building details are better defined (e.g., structural loads, extent of excavation, etc.) to develop design-level geotechnical recommendations and criteria for the project 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. 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 the project Owner and/or their 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 soils in this geographic area.

8.0 LIST OF REFERENCES

American Society of Civil Engineers (ASCE) (2016), "Minimum Design Loads for Buildings and Other Structures" (2016 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, (2019) "2019 Annual Book of ASTM Standards, Section 4, Construction, Volume 4.08, Soil and Rock; Dimension Stone; Geosynthetics," ASTM, Philadelphia.

Association of Bay Area Governments, "Resilience Program: Bay Area Hazards Maps". Online, date accessed September 17, 2020.

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

California Department of Conservation, California Geologic Survey, "Digital Images of the Alquist Priolo Maps" http://www.conservation.ca.gov/cgs/rghm/ap/Pages/Index.aspx, 2018.

California Department of Transportation (Caltrans) (2018), 2018 Standard Specifications.

California Department of Transportation (Caltrans) (2020), Highway Design Manual, 7th Edition.

California Department of Transportation (Caltrans), "Corrosion Guidelines Version 1.0," September 2015.

California Division of Mines and Geology, Special Publication 42, "Alquist-Priolo Special Studies Zone Act," 1972 (Revised 2018).

California State Water Resources Control Board (2020), "Geotracker (web-based groundwater resources database), https://documents.geotracker.waterboards.ca.gov/esi/uploads/geo_report/6426409244/T0601300401.PDF, accessed September 15, 2020.

Campbell, K., Bozorgnia, Y., "NGA Ground Motion Model for the Geometric Mean Horizontal Component of PGA, PGV, PGD and 5% Damped Linear Elastic Response Spectra for Periods Ranging from 0.01 to 10 s," EERI Earthquakes Spectra, Volume 24, Number 1, February 2008.
Chiou, B. and Youngs, R., "An NGA Model for the Average Horizontal Component of Peak Ground Motion and Response Spectra," EERI Earthquakes Spectra, Volume 24, Number 1, February 2008.

City of Pinole, "General Plan Update, Potential Geologic Hazards Map, Figure 9.1" November 2010.

Dibblee, Thomas W. Jr. (1980) "Geologic Map of the Richmond Quadrangle, Alameda and Contra Costa Counties, California.", California Division of Mines and Geology Open File Report 80-1100.

Idriss, I.M. & Boulanger, R.W. "Soil Liquefaction during Earthquakes", Earthquake Engineering Research Institute Monograph 12, 2008.

Idriss, I.M. & Boulanger, R.W. "SPT-Based Liquefaction Triggering Procedures" Department of Civil and Environmental Engineering, College of Engineering, University of California at Davis, UCD/GCM-10/02, December 2010.

Kleinfelder (2011), "Subsurface Fault Investigation, Pinole Valley High School Campus, located at 2900 Pinole Valley Road in Pinole, California", File No. 117463/3, dated August 9, 2011.

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

Ozocak, A & Sert, S., "Evaluation of liquefaction risk by a revised LPI approach," 2nd International Symposium on Cone Penetration Testing, Huntington Beach, CA, USA. Volume 2&3: Technical Papers, Session 3: Applications, Paper No. 3-30, 2010.

Phelps, G.A. et al (2008), "Three-Dimensional Map of the Hayward Fault Zone, San Francisco Bay Region, California", United States Geological Survey Scientific Investigations Map 3045, Version 1.0.

Robertson, P.K. (2009) "Performance Based Earthquake Design using the CPT", Proceedings of the International Conference on Performance-Based Design in Earthquake Geotechnical Engineering (IS-Tokyo 2009), 15-18 June 2009.

Southern California Earthquake Center (SCEC), "Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction Hazards in California," University of Southern California, March 1999.

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.

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.

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.

Williams, P.L. et al (1995), "Geophysical Detection of Fault Geometry, Activity, and Recurrence Behavior: Rodgers Creek and Pinole Faults, California" (DRAFT in preparation for publication), *in* Very-High Profile Resolution Profiling Across Pacific Coast Faults, McEvilly, T.V. ed, University of California, Berkeley Seismograhic Stations, USGS NEHRP Award No. 1434-94-G-2406, https://books.google.com/books/about/Very_High_Resolution_Profiling_Across_Pa.html?id=dJxOAQAAIAAJ.

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

SITE: LATITUDE, 37.9941°
LONGITUDE, -122.2849°

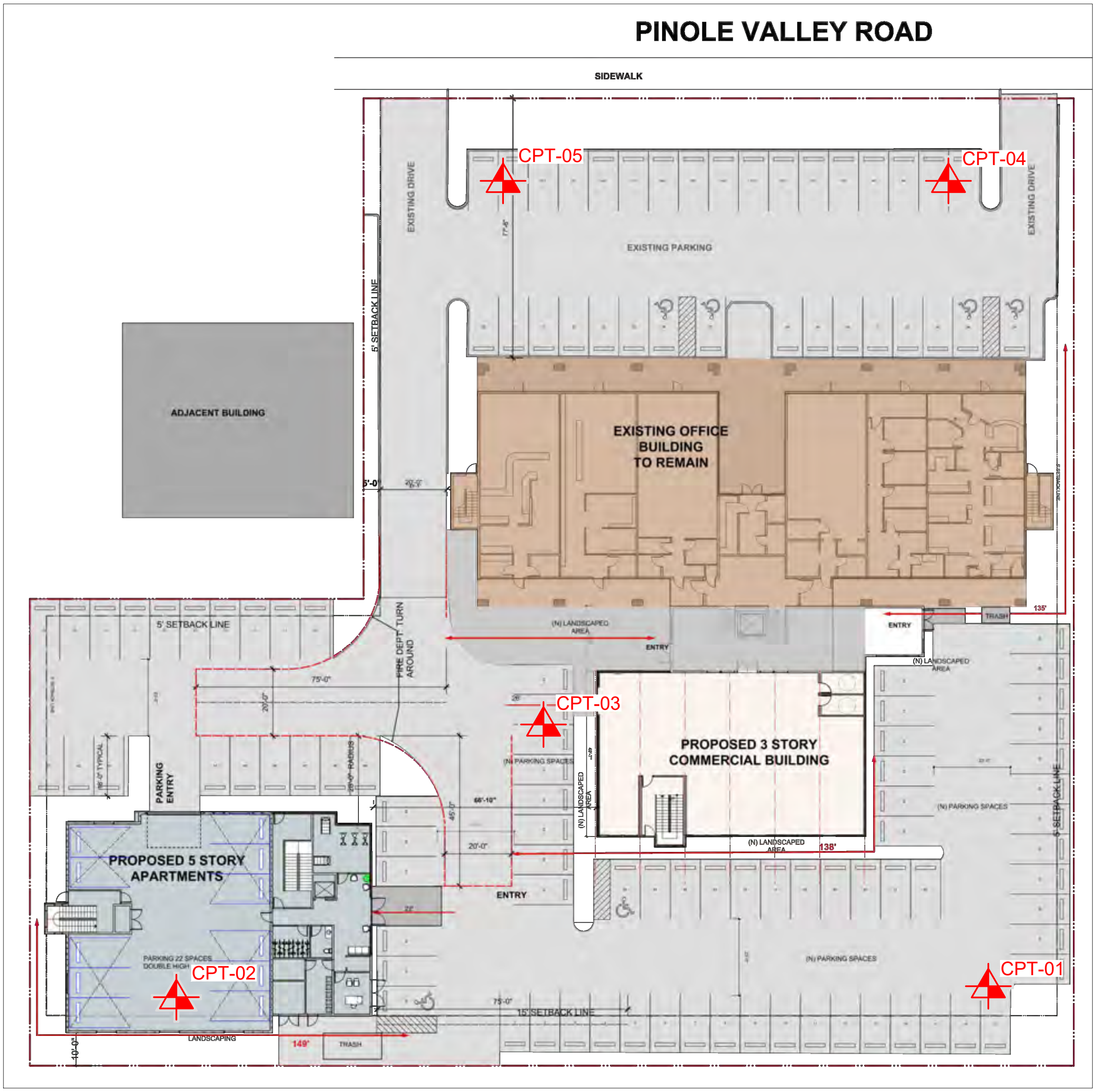
SITE LOCATION

N.T.S.

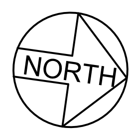
NORTH

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		
 FILENAME: 3039.001 Figures.dwg		BCRE Project 2801 Pinole Valley Road Pinole, California Project No. 3039.001 Date: 9/17/2020	Drawn MMT Checked	1 FIGURE



SITE PLAN



SCALE

0 25 50 100 FEET

Approximate location of CPT completed by MPEG, 2020

REFERENCE: ch x tld, "Commercial Office Addition & New Apartment Building", Site Plan, Sheet A0-1.0.

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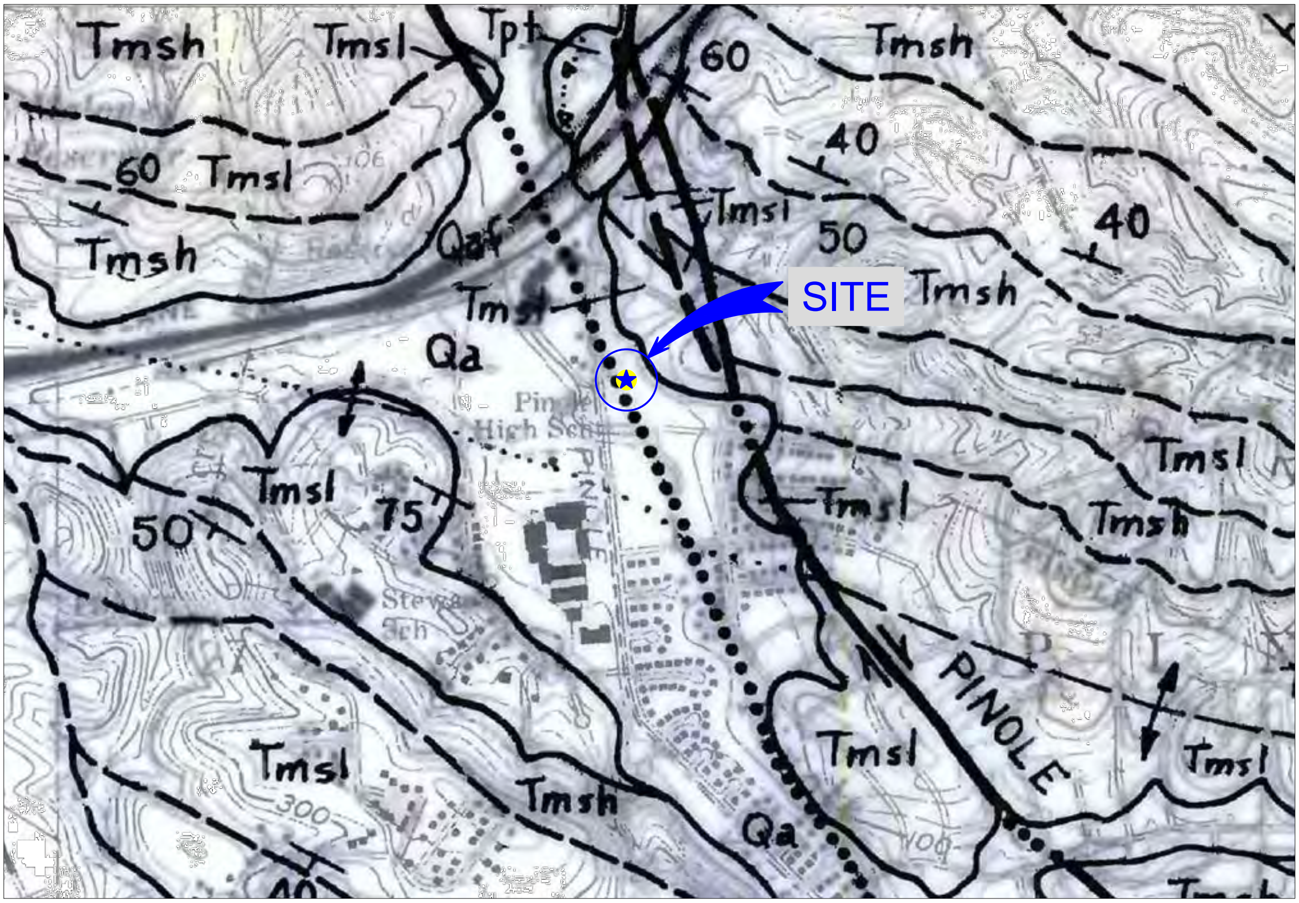
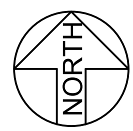
 FILENAME: 3039.001 Figures.dwg		BCRE Project 2801 Pinole Valley Road Pinole, California Project No. 3039.001 Date: 9/17/2020	Drawn MMT Checked	2 FIGURE





REGIONAL GEOLOGIC MAP

(NOT TO SCALE)

LEGEND:

Qbm	Bay Mud - Highly compressible, highly expansive marine clays and silts
Qa	Alluvium - Poorly to moderately sorted sands, gravels, silts, and clay
Tmsl	Siltstone - Massive, locally sandy
Tmsh	Shale - Light gray, massive to platy, siliceous to sandy
— — —	Geologic Contact, dashed where appropriate
▬▬	Fault

REFERENCE: Dibblee, Thomas W. (1980), "Preliminary Geologic Map of the Richmond Quadrangle, Alameda and Contra Costa Counties, California." Department of the Interior, USGS, Open File Report 80-1100, Scale 1:24,000.

MPEG MILLER PACIFIC ENGINEERING GROUP

FILENAME: 3039.001 Figures.dwg

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REGIONAL GEOLOGIC MAP

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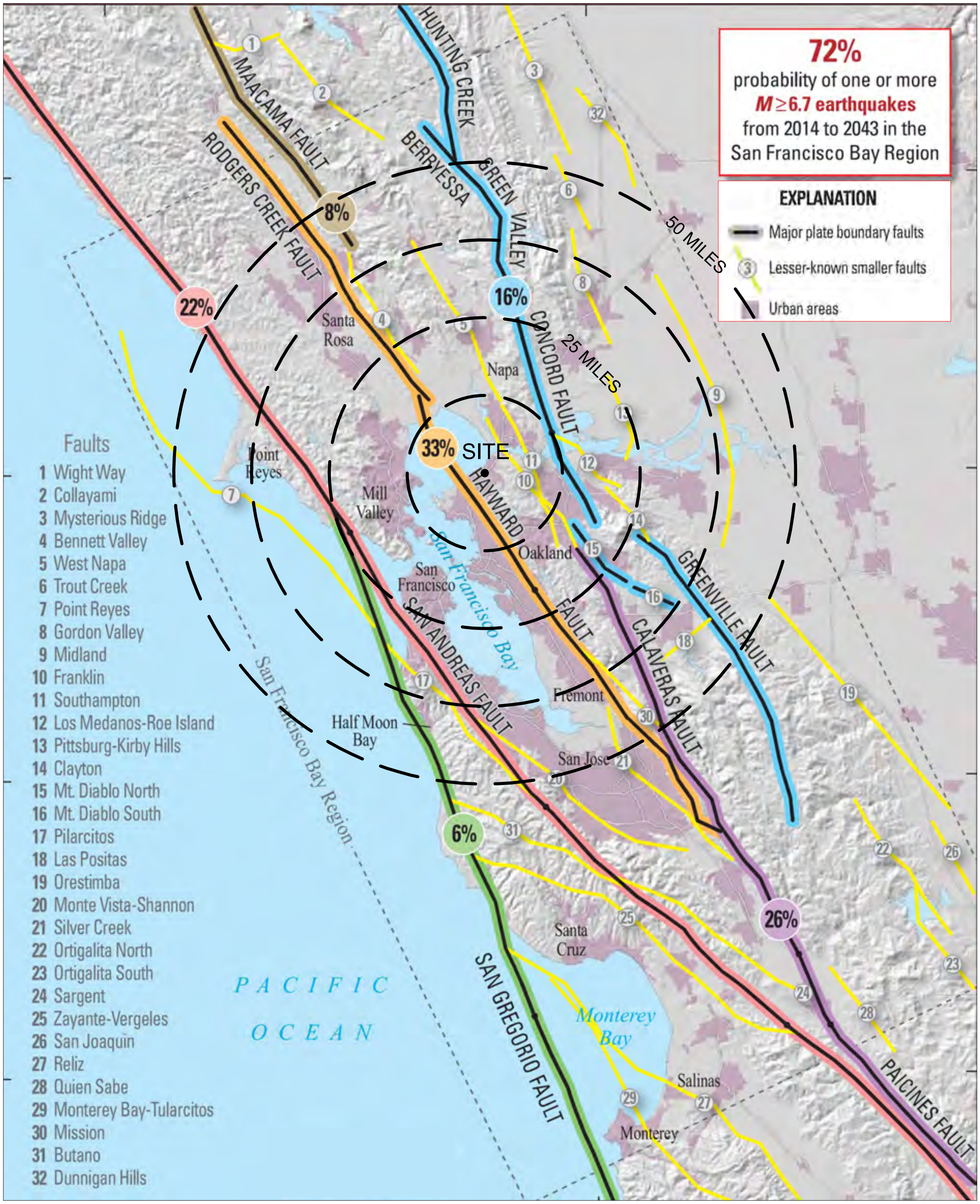
3
FIGURE



SITE COORDINATES
LAT. 37.9941°
LON. -122.2849°

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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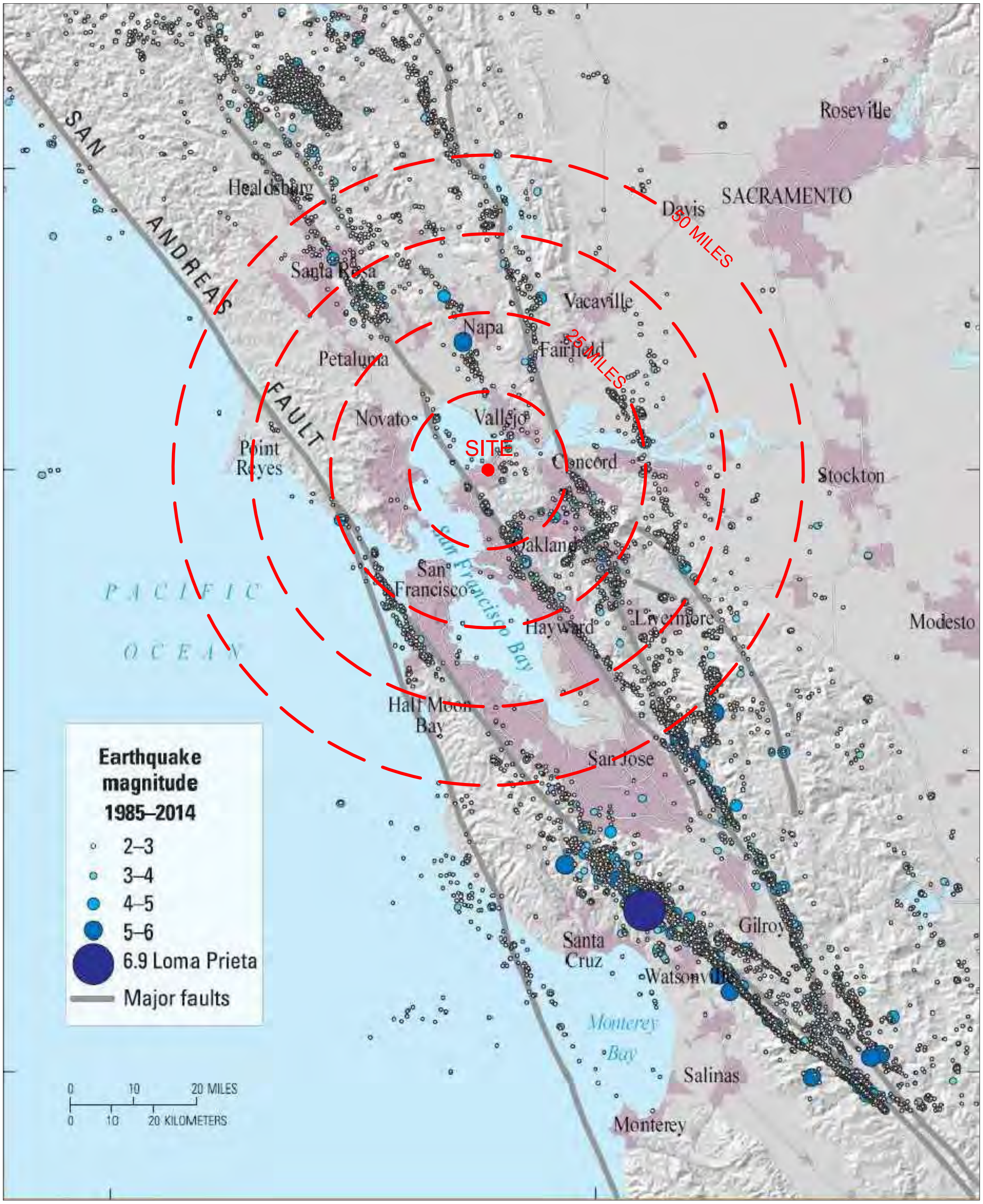
4
FIGURE



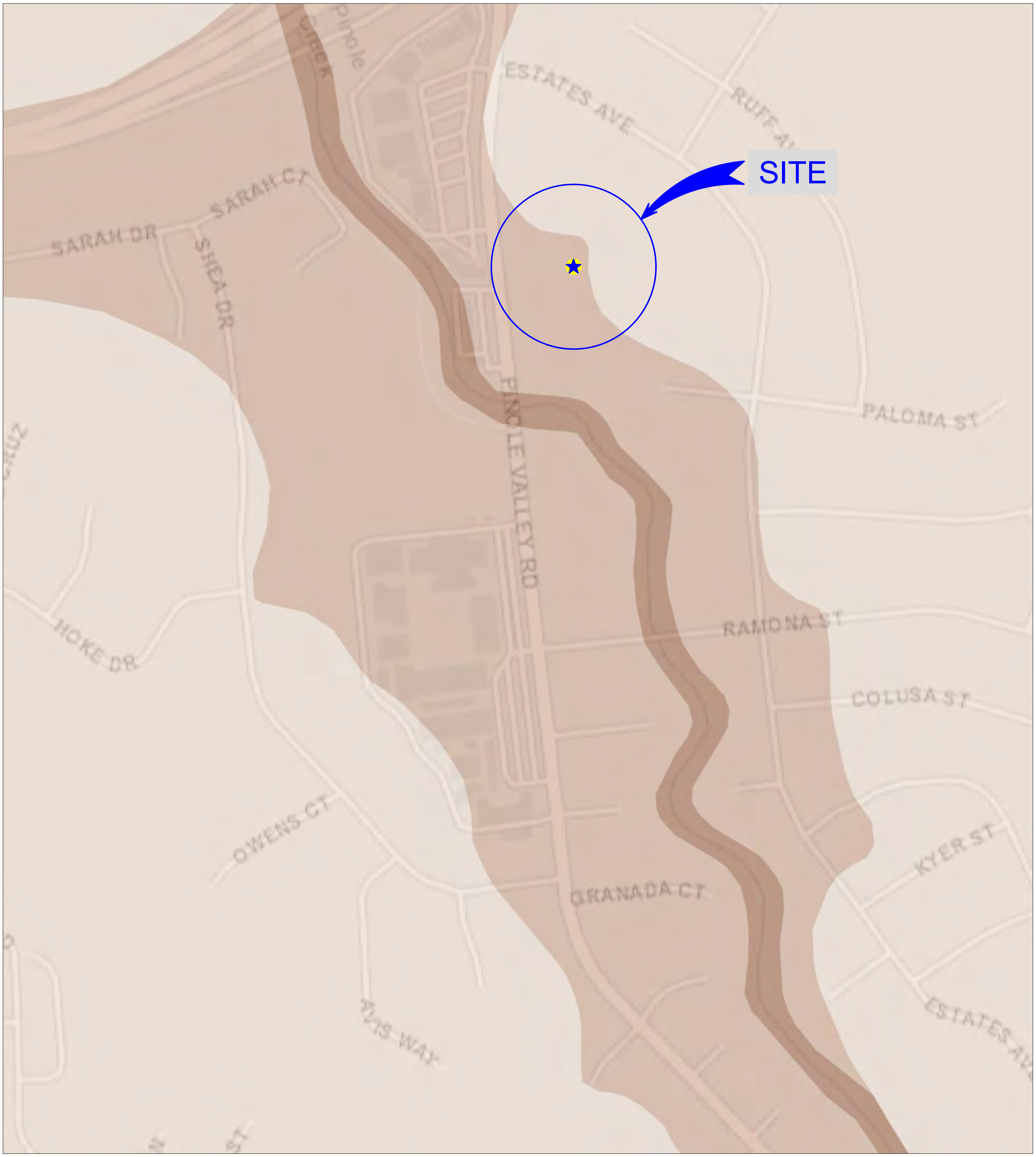
SITE COORDINATES
LAT. 37.9941°
LON. -122.2849°

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).

MPEG MILLER PACIFIC ENGINEERING GROUP	504 Redwood Blvd. Suite 220 Novato, CA 94947 T 415 / 382-3444 F 415 / 382-3450 www.millerpac.com	HISTORIC EARTHQUAKE ACTIVITY		
FILENAME: 3039.001 Figures.dwg		BCRE Project 2801 Pinole Valley Road Pinole, California Project No. 3039.001 Date: 9/17/2020	Drawn MMT Checked	5 FIGURE



Susceptibility Level:

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

No Scale

NORTH

Reference: ABAG Hazard Map Viewer.

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FILENAME: 3039.001 Figures.dwg

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LIQUEFACTION SUSCEPTIBILITY MAP

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6
FIGURE

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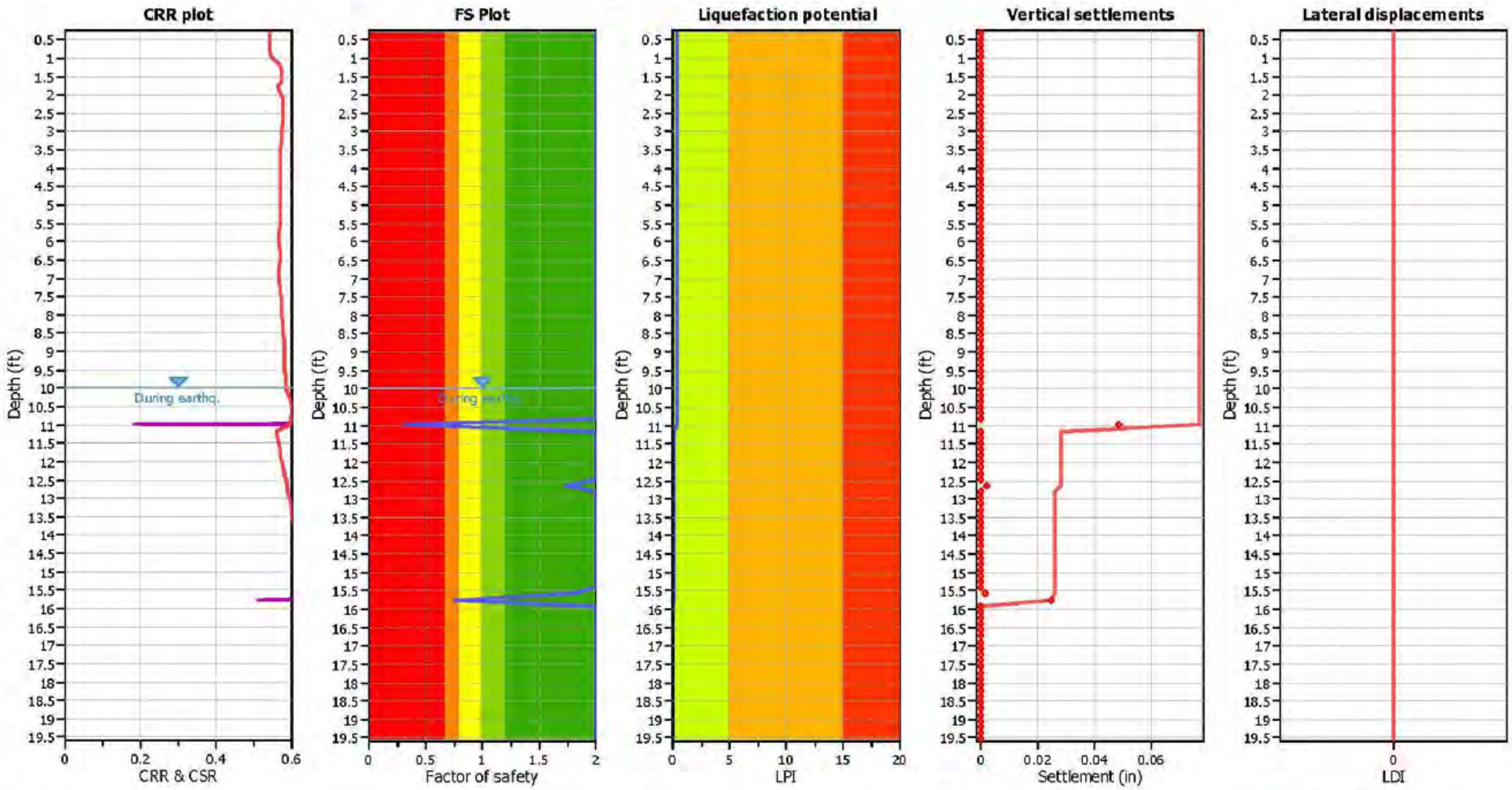
CPT name: CPT-01

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	10.00 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:	Sands only
Peak ground acceleration:	0.99	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	20.00 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: 9/17/2020, 10:30:29 AM
Project file: H:\Jobs\3000-3099\3039.001 2801 Pinole Valley Road Eval\Phase 1 Preliminary Investigation\Analyses\3039.001Cliq.clq

5

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FILENAME: 3039.001 Figures.dwg

CPT-01 LIQUEFACTION RESULTS

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7
FIGURE

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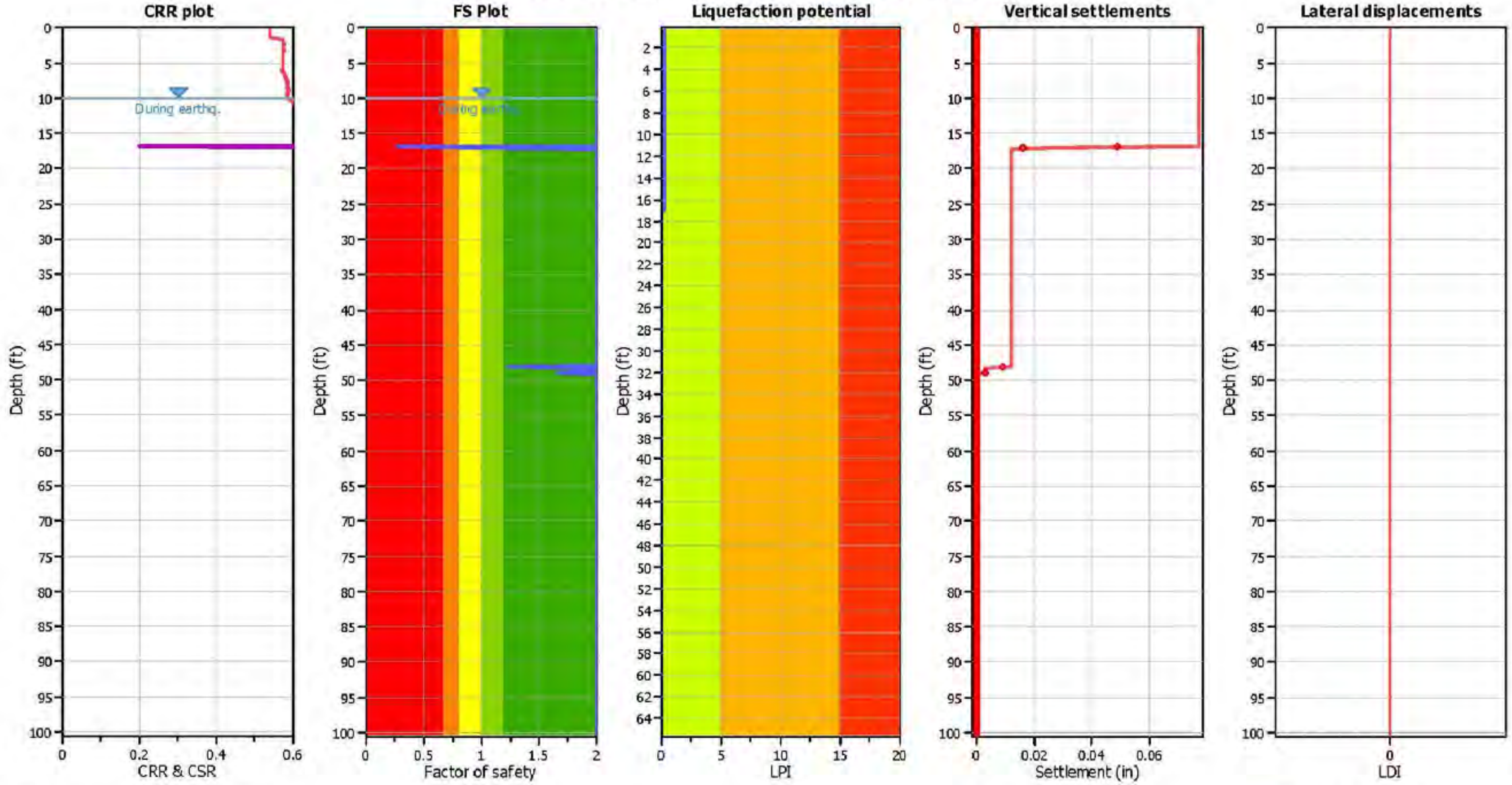
CPT name: CPT-02

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	10.00 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:	Sands only
Peak ground acceleration:	0.99	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	15.00 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: 9/17/2020, 10:30:31 AM

Project file: H:\Jobs\3000-3099\3039.001 2801 Pinole Valley Road Eval\Phase 1 Preliminary Investigation\Analyses\3039.001Cliq.clq

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CPT-02 LIQUEFACTION RESULTS

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FIGURE 8

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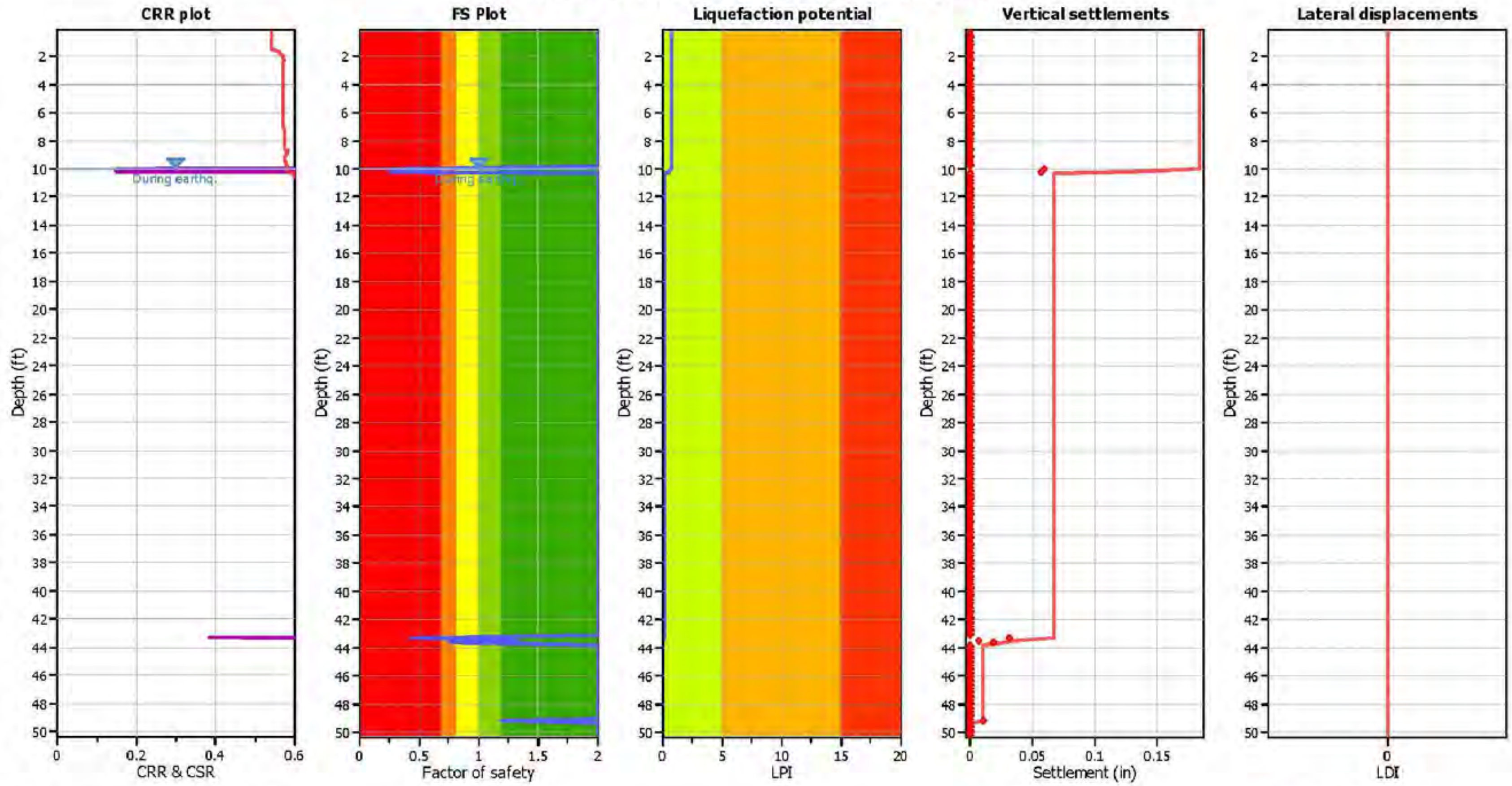
CPT name: CPT-03

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	10.00 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:	Sands only
Peak ground acceleration:	0.99	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	15.00 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: 9/17/2020, 10:30:33 AM
Project file: H:\Jobs\3000-3099\3039.001 2801 Pinole Valley Road Eval\Phase 1 Preliminary Investigation\Analyses\3039.001Cliq.clq

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CPT-03 LIQUEFACTION RESULTS

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9
FIGURE

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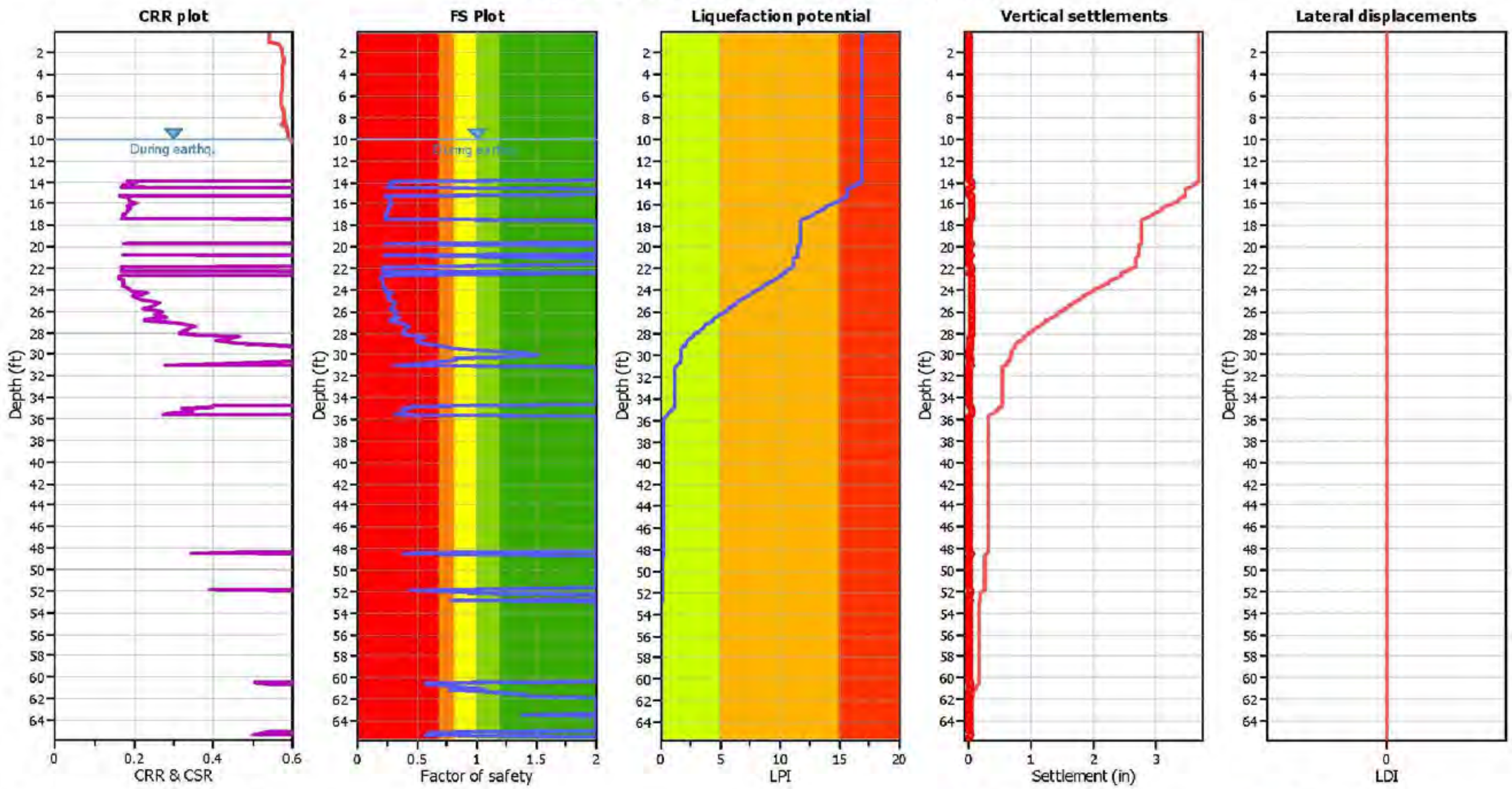
Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	10.00 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:	Sands only
Peak ground acceleration:	0.99	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	14.00 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: 9/17/2020, 10:30:34 AM
Project file: H:\Jobs\3000-3099\3039.001 2801 Pinole Valley Road Eval\Phase 1 Preliminary Investigation\Analyses\3039.001Cliq.clq

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CPT-04 LIQUEFACTION RESULTS

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10
FIGURE

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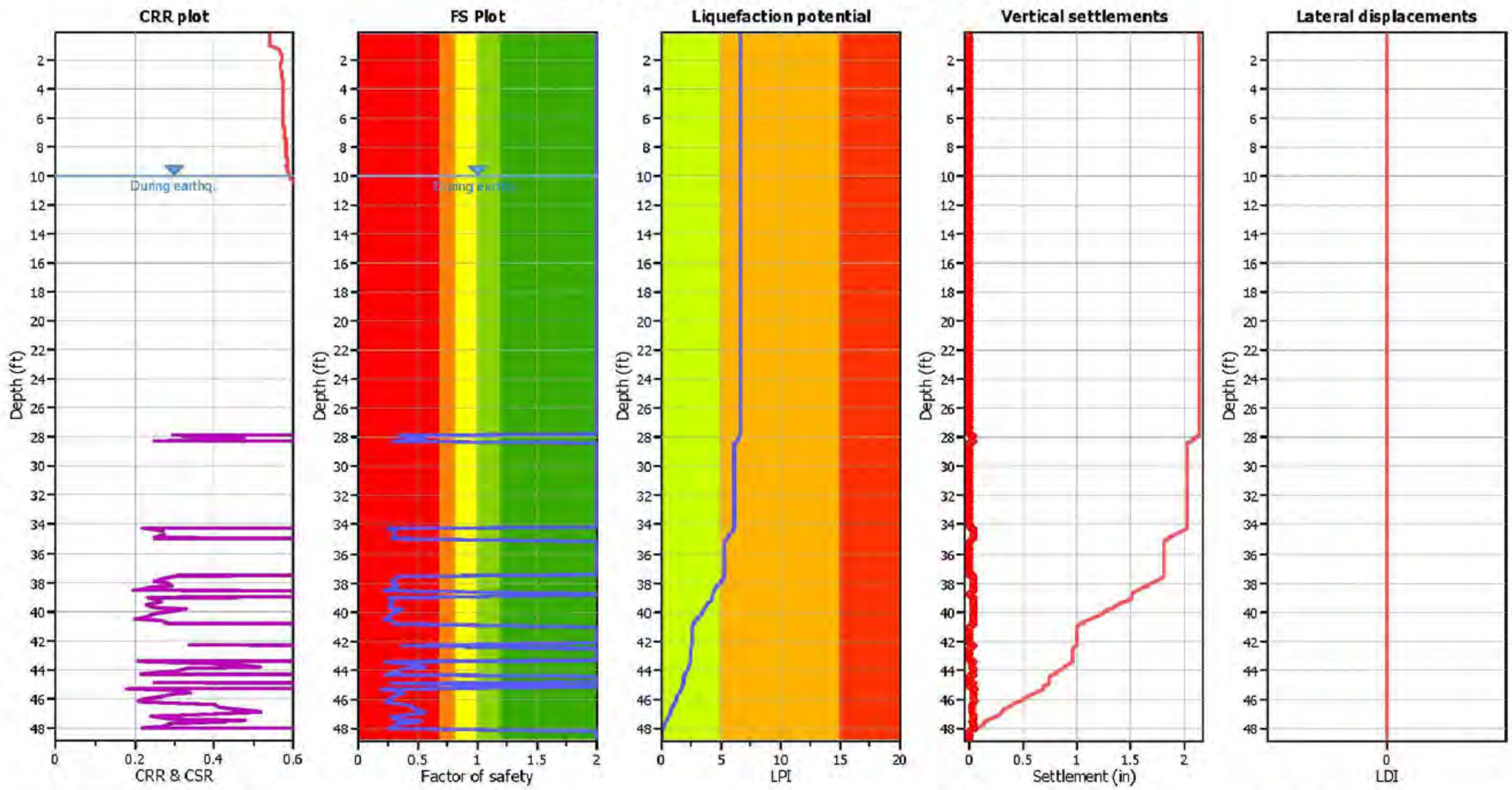
CPT name: CPT-05

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	10.00 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:	Sands only
Peak ground acceleration:	0.99	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	18.00 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: 9/17/2020, 10:30:36 AM

Project file: H:\Jobs\3000-3099\3039.001 2801 Pinole Valley Road Eval\Phase 1 Preliminary Investigation\Analyses\3039.001Cliq.clq

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CPT-05 LIQUEFACTION RESULTS

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11
FIGURE

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APPENDIX A
SUBSURFACE EXPLORATION AND LABORATORY TESTING

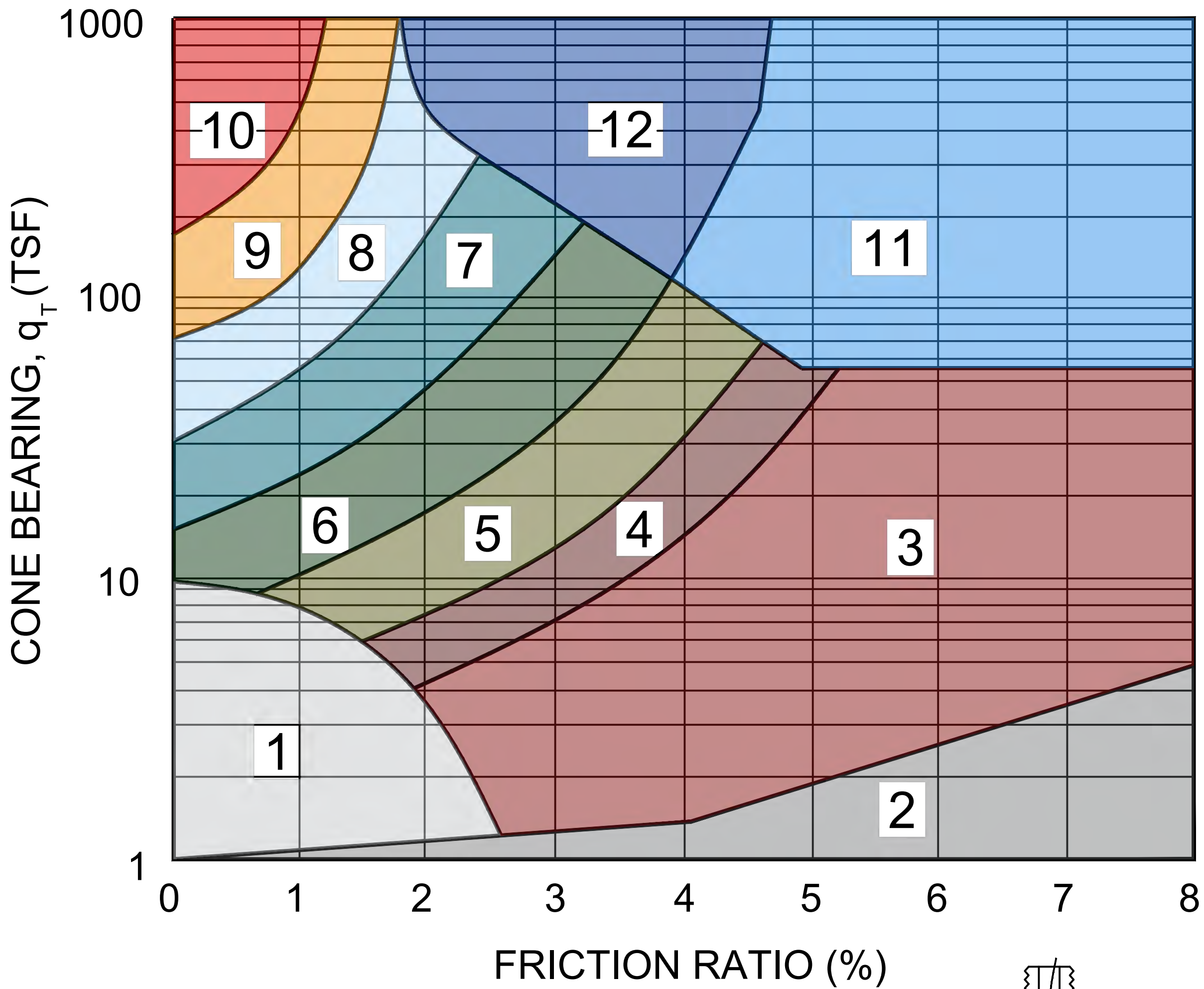
A. SUBSURFACE EXPLORATION

We performed five Cone Penetration Tests (CPT) on August 17, 2020 at the locations shown on the Site Plan, Figure 2. 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.

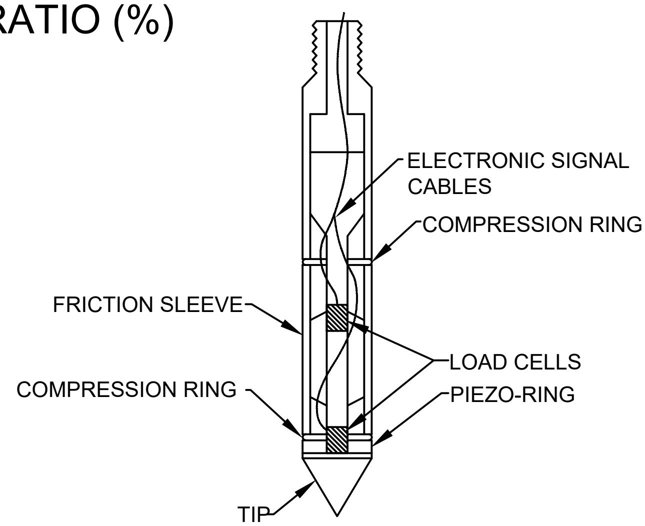
The recorded data is transferred to an in-office computer for reduction and analysis. The analysis of cone bearing and sleeve friction (i.e. friction ratio) indicates the soil type, the cone bearing alone indicates soil density or strength, and the pore pressure indicates the presence of clay. Variations 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-6.

The exploratory CPT logs, description of soils encountered, and the laboratory test data reflect conditions only at the location of the boring 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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CPT SOIL INTERPRETATION CHART

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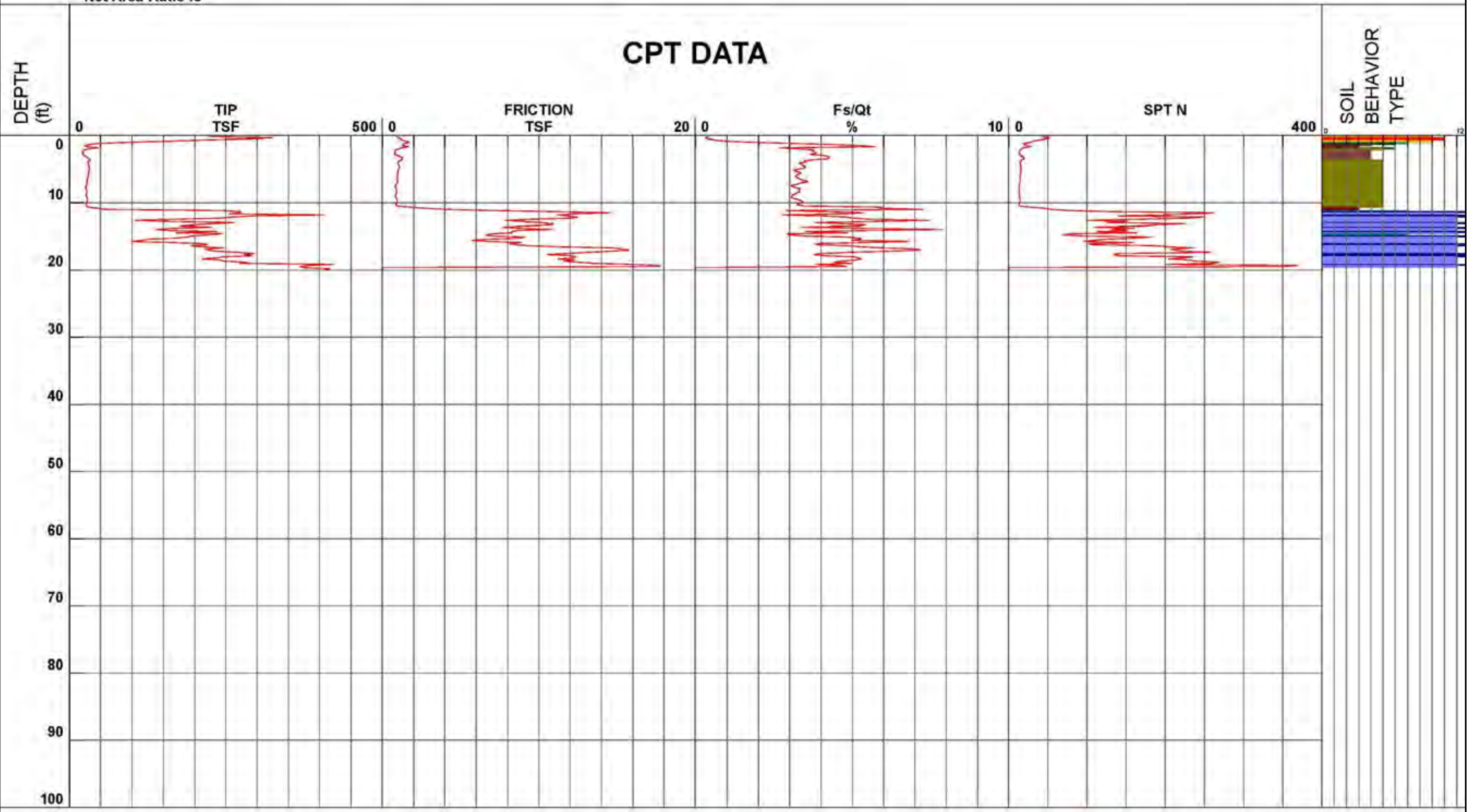
A-1
FIGURE

Miller Pacific Engineering

Project	BCRE Development	Operator	JM-AJ	Filename	SDF(003).cpt
Job Number	3039.001	Cone Number	DDG1496	GPS	
Hole Number	CPT-01	Date and Time	8/17/2020 9:27:43 AM	Maximum Depth	20.01 ft
EST GW Depth During Test		20.00 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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CPT-01 DATA

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Drawn MMT
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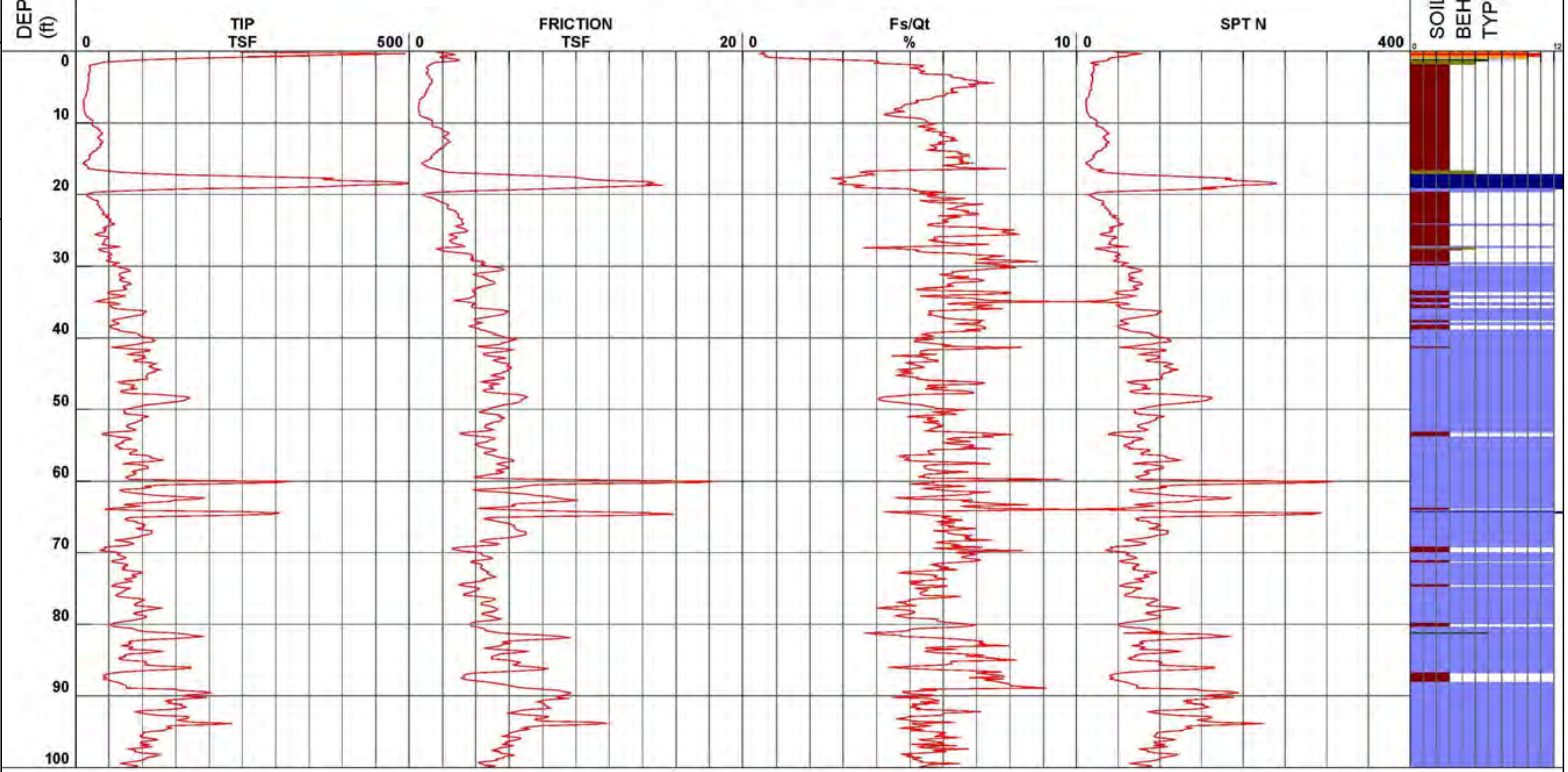
A-2
FIGURE

Miller Pacific Engineering

Project	BCRE Development	Operator	JM-AJ	Filename	SDF(005).cpt
Job Number	3039.001	Cone Number	DDG1496	GPS	
Hole Number	CPT-02	Date and Time	8/17/2020 11:14:12 AM	Maximum Depth	100.72 ft
EST GW Depth During Test	15.00 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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CPT-02 DATA

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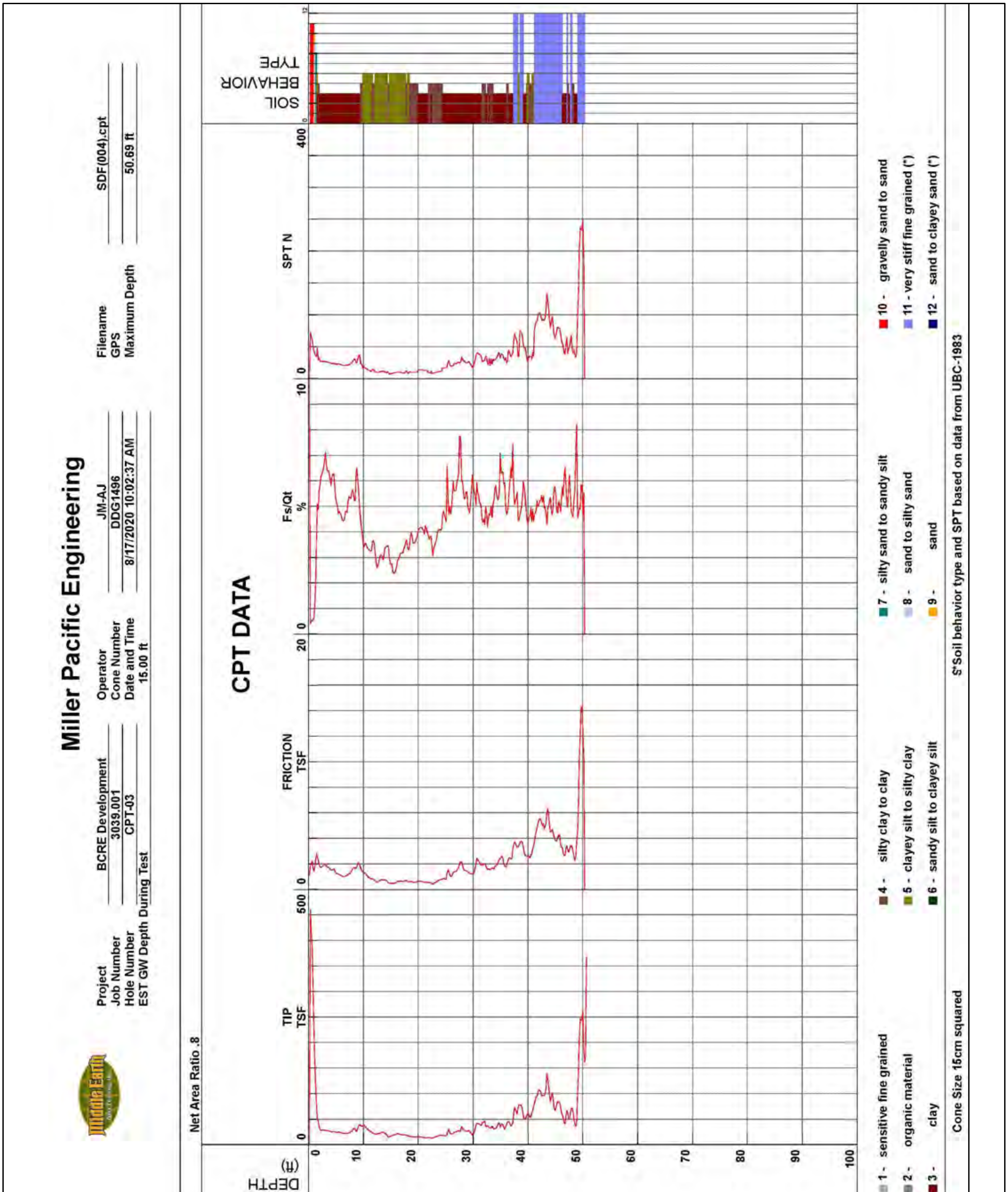
A-3
FIGURE



FILENAME: 3039.001 CPTs.dwg

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CPT-03 DATA

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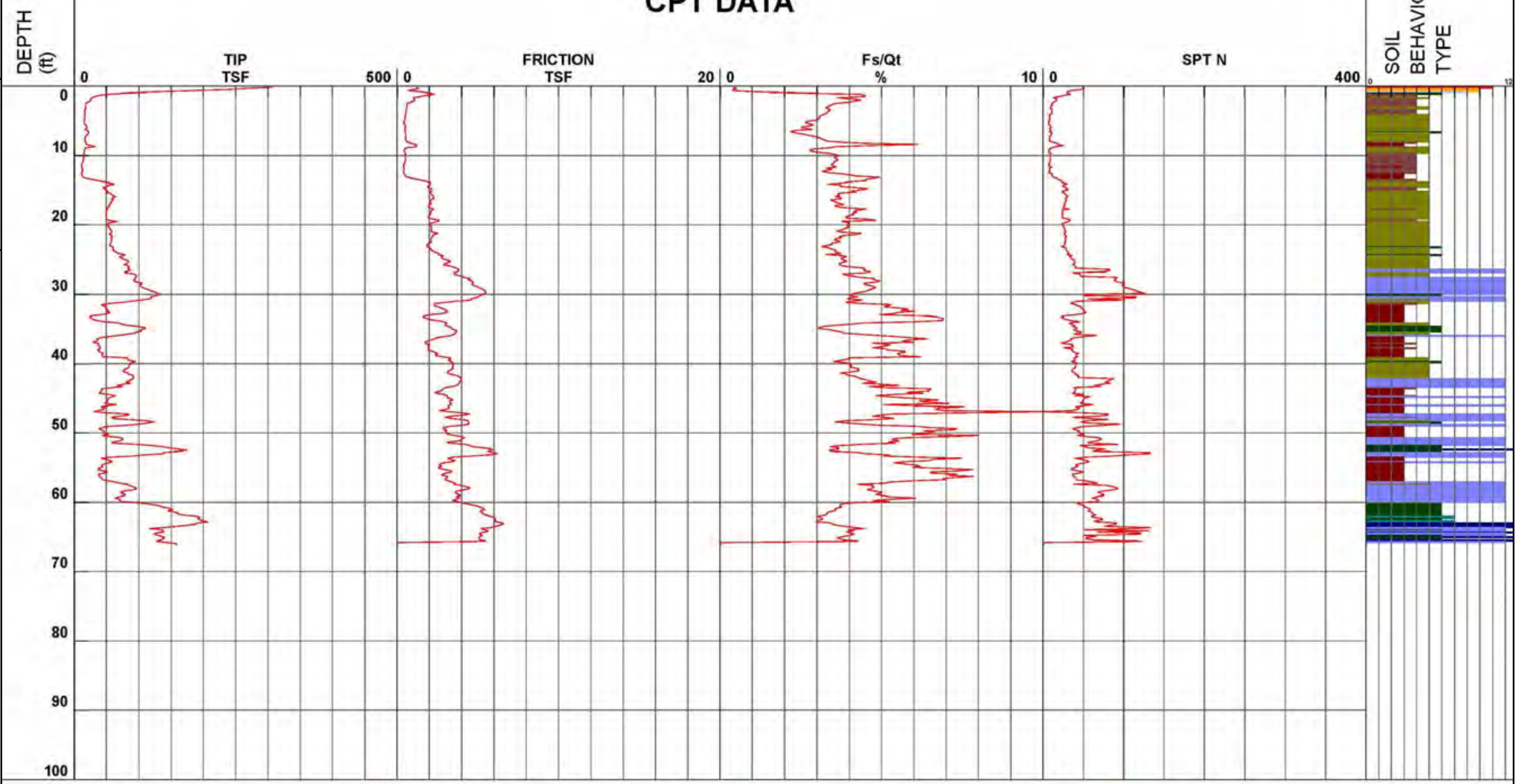
A-4
FIGURE

Miller Pacific Engineering

Project	BCRE Development	Operator	JM-AJ	Filename	SDF(002).cpt
Job Number	3039.001	Cone Number	DDG1496	GPS	
Hole Number	CPT-04	Date and Time	8/17/2020 8:26:14 AM	Maximum Depth	66.11 ft
EST GW Depth During Test	14.00 ft				

Net Area Ratio .8

CPT DATA



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

Cone Size 15cm squared

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

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CPT-04 DATA

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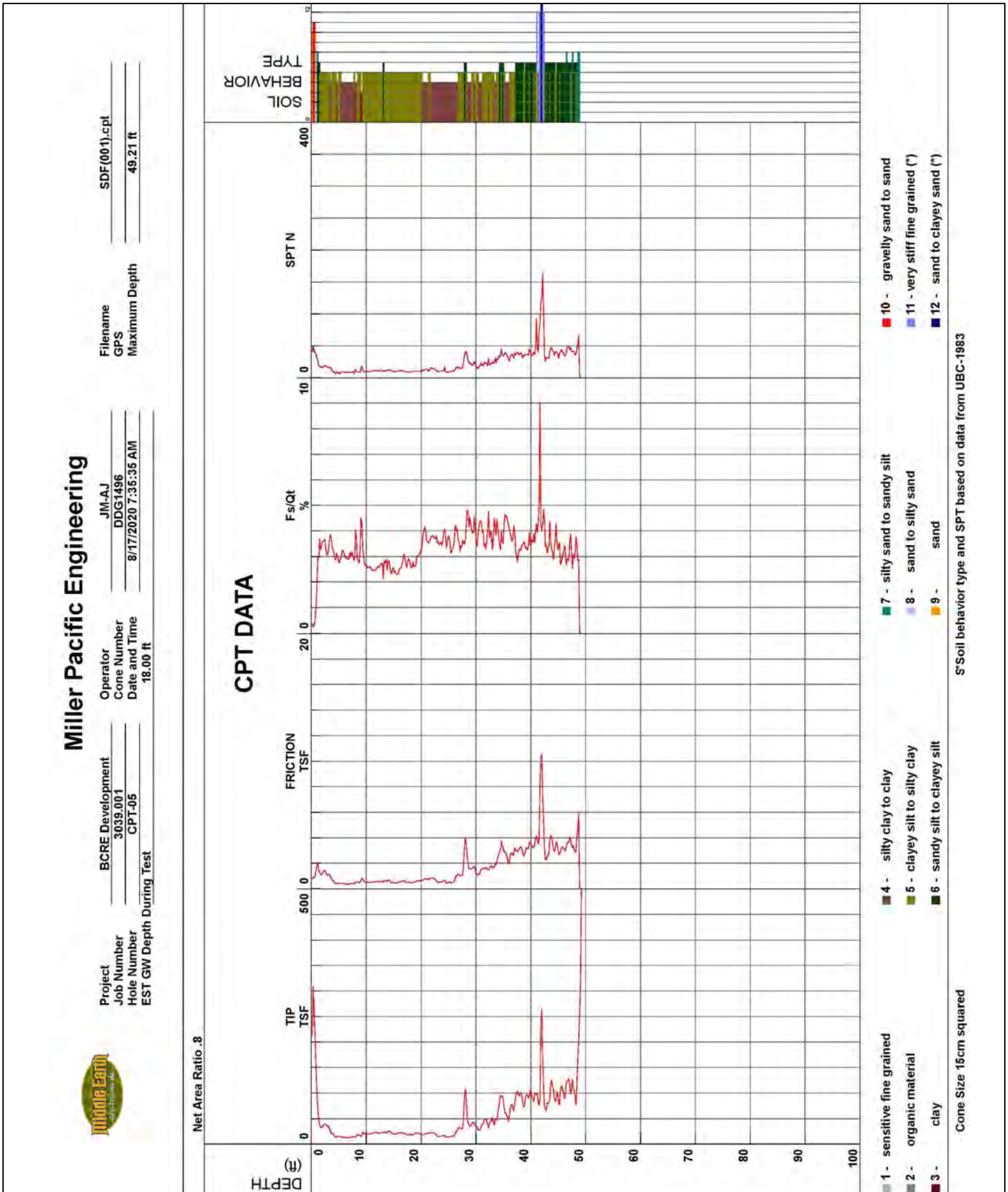
A-5
FIGURE



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FILENAME: 3039.001 CPTs.dwg

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CPT-05 DATA

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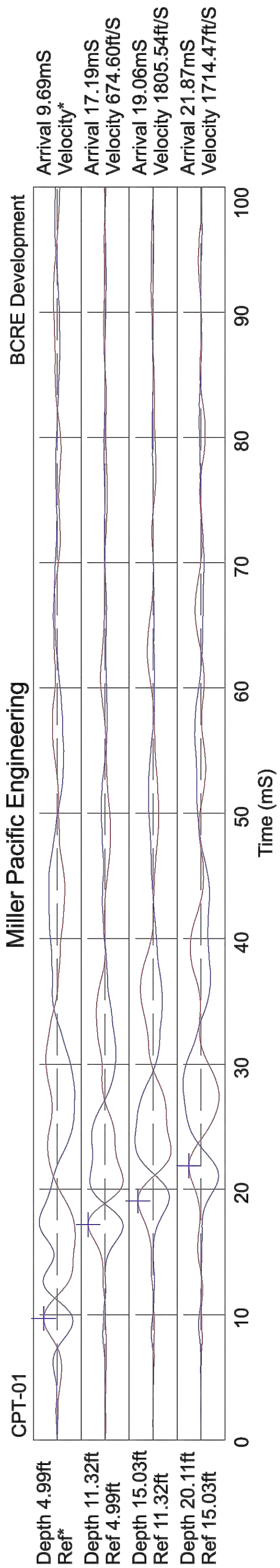
Pinole, California

Project No. 3039.001 Date: 9/17/2020

Drawn MMT
Checked

A-6
FIGURE



Hammer to Rod String Distance (ft): 5.83
* = Not Determined

COMMENT:

MPEG
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Novato, CA 94947
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FILENAME: 3039.001 CPTs.dwg

CPT-01 SHEAR WAVE VELOCITY DATA

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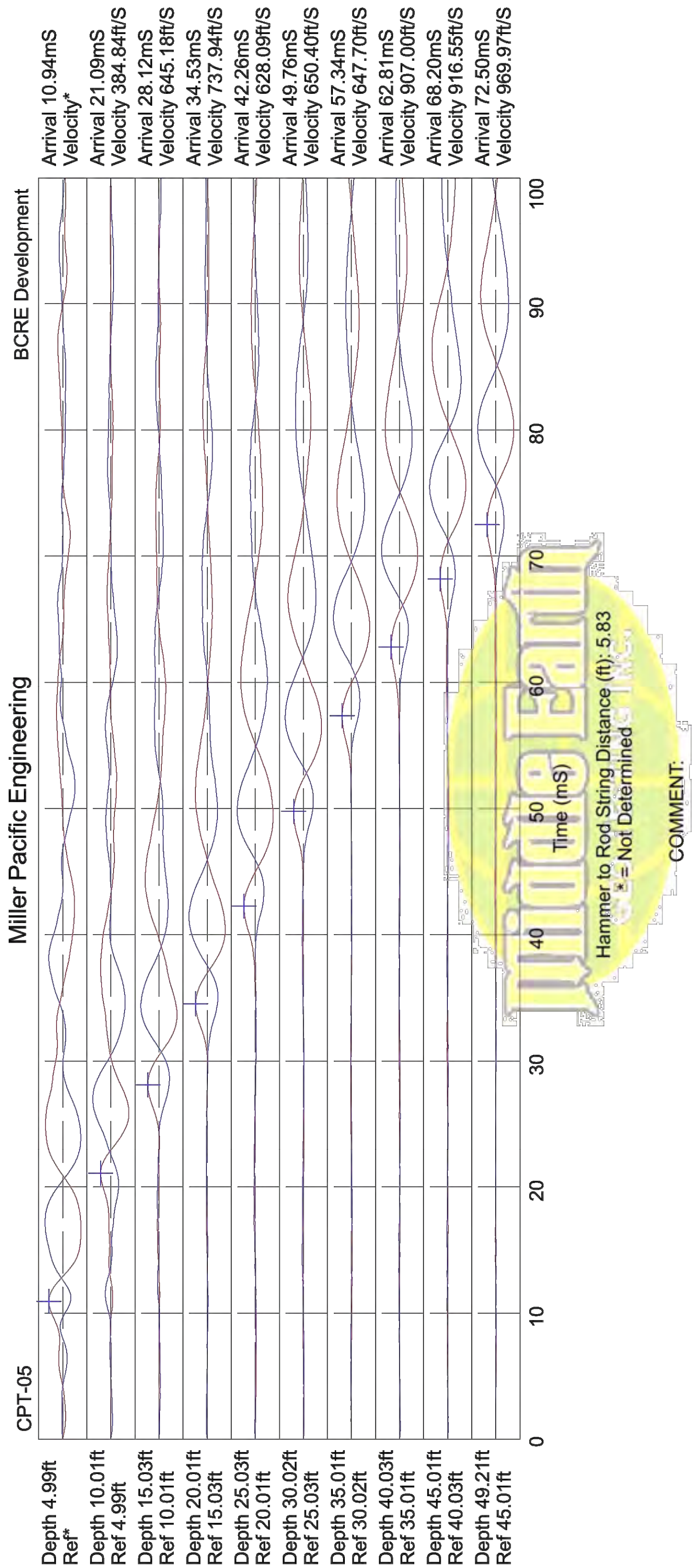
Project No. 3039.001 Date: 9/17/2020

Drawn MMT
Checked

A-7
FIGURE



MILLER PACIFIC ENGINEERING GROUP	504 Redwood Blvd. Suite 220 Novato, CA 94947 T 415 / 382-3444 F 415 / 382-3450 www.millerpac.com	CPT-05 SHEAR WAVE VELOCITY DATA
A CALIFORNIA CORPORATION, © 2016, ALL RIGHTS RESERVED FILENAME: 3039.001 CPTs.dwg		BCRE Project 2801 Pinole Valley Road Pinole, California Project No. 3039.001 Date: 9/17/2020
		Drawn MMT Checked
		A-8 FIGURE

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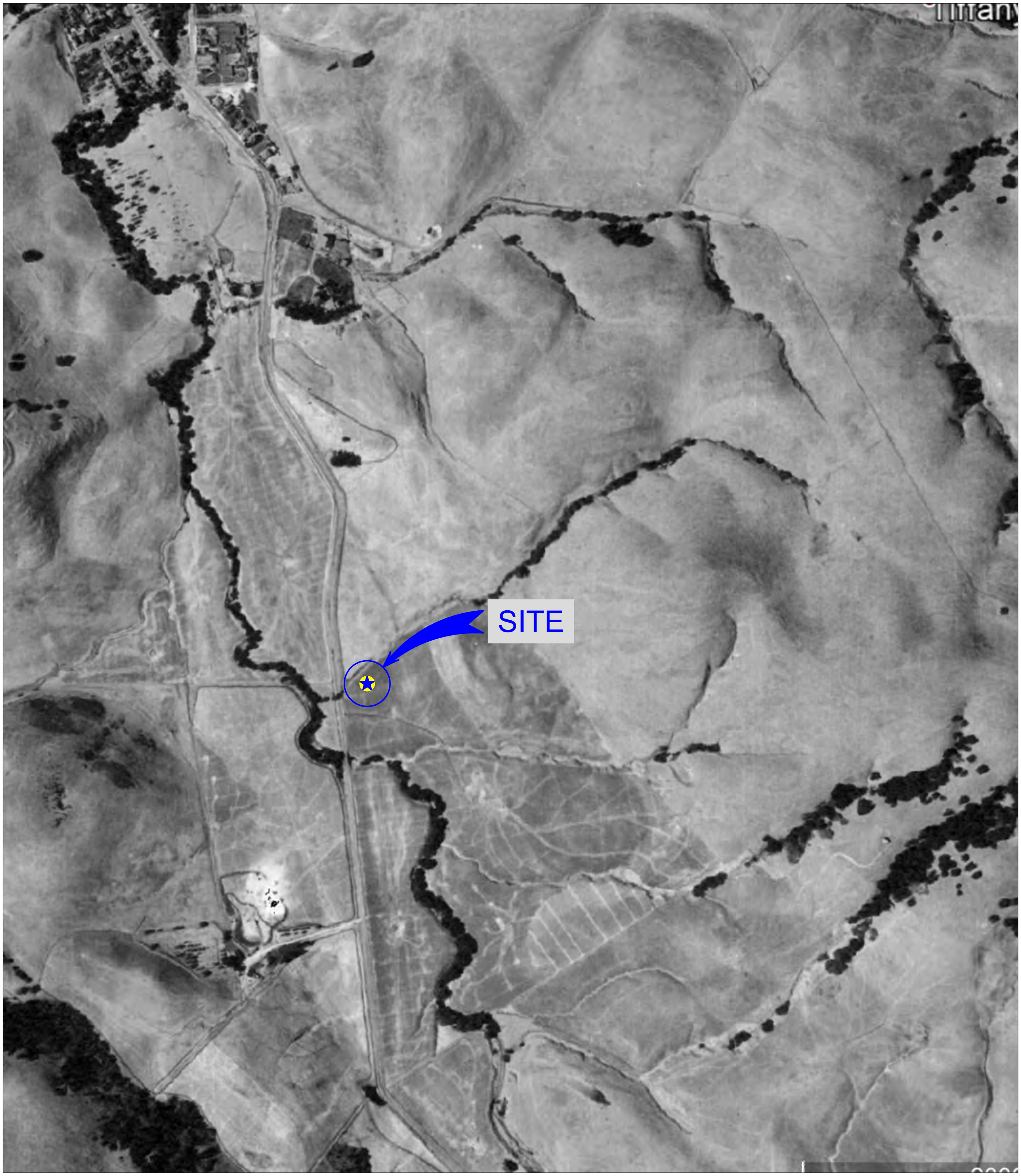
APPENDIX B
HISTORIC AERIAL PHOTOGRAPHS



SCALE

0 350 700 1400 FEET

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FILENAME: 3039.001 Aerial Photos.dwg

GOOGLE EARTH IMAGE - 1939

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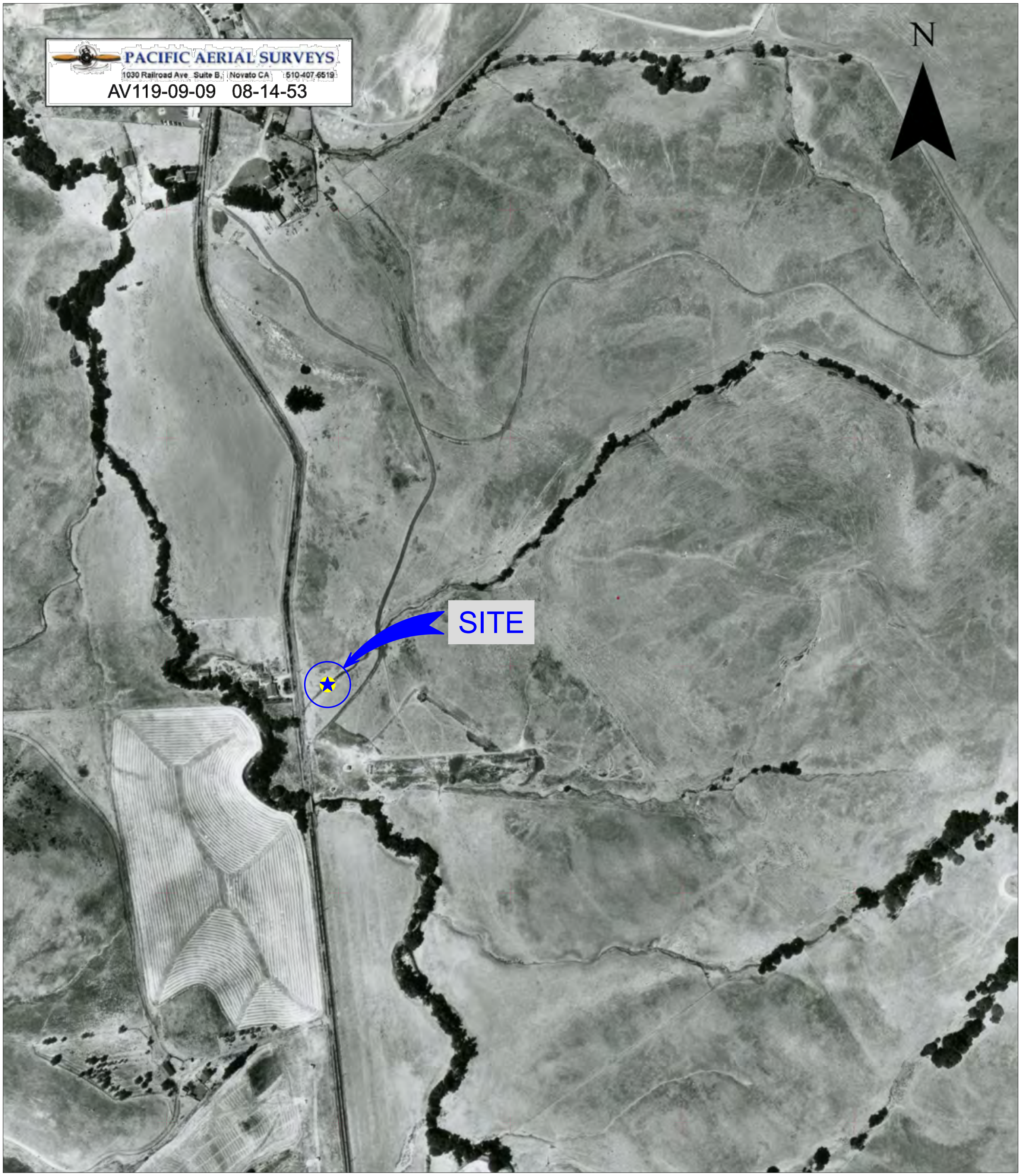
Drawn MMT
Checked

B-1
FIGURE



SCALE

0 350 700 1400 FEET

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AERIAL PHOTOGRAPH - 8/14/1953

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B-2
FIGURE

SCALE

0 350 700 1400 FEET

MPEG MILLER PACIFIC ENGINEERING GROUP

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AERIAL PHOTOGRAPH - 5/3/1957

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B-3
FIGURE

SCALE

0 350 700 1400 FEET

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AERIAL PHOTOGRAPH - 3/1/1958

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B-4
FIGURE

SCALE

0 350 700 1400 FEET

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AERIAL PHOTOGRAPH - 6/30/1959

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B-5
FIGURE

SCALE

0 350 700 1400 FEET

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FILENAME: 3039.001 Aerial Photos.dwg

AERIAL PHOTOGRAPH - 7/25/1963

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B-6
FIGURE

SCALE

0 350 700 1400 FEET

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FILENAME: 3039.001 Aerial Photos.dwg

AERIAL PHOTOGRAPH - 4/20/1966

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Checked

B-7
FIGURE

SCALE

0 350 700 1400 FEET

MPEG
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AERIAL PHOTOGRAPH - 4/10/1968

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Drawn MMT
Checked

B-8
FIGURE