MILLER PACIFIC
ENGINEERING GROUP

GEOTECHNICAL INVESTIGATION
RIVERBEND RESIDENTIAL DEVELOPMENT
529 MADISON STREET
PETALUMA, CALIFORNIA

October 12, 2021

Project 2066.001

Prepared for:
Lenox Homes
3675 Mt. Diablo Boulevard, Suite 350
Lafayette, California 94549

Attention: Mr. Rick Rosenbaum, Project Manager

CERTIFICATION

This document is an instrument of service, prepared by or under the direction of the undersigned professionals, in accordance with the current ordinary standard of care. The service specifically excludes the investigation of radon, asbestos, toxic mold and other biological pollutants, and other hazardous materials. The document is for the sole use of the client and consultants on this project. Use by third parties or others is expressly prohibited without written permission. If the project changes, or more than two years have passed since issuance of this report, the findings and recommendations must be reviewed by the undersigned.

MILLER PACIFIC ENGINEERING GROUP
(a California corporation)

REVIEWED BY

Monica Thornton
Project Engineer

Daniel S. Caldwell
Geotechnical Engineer No. 2006
(Expires 9/30/23)

MILLER PACIFIC
ENGINEERING GROUP

GEOTECHNICAL INVESTIGATION
RIVERBEND RESIDENTIAL DEVELOPMENT
529 MADISON STREET
PETALUMA, CALIFORNIA

TABLE OF CONTENTS

MILLER PACIFIC ENGINEERING GROUP

FIGURES

APPENDIX A – REFERENCE SUBSURFACE EXPLORATION AND LABORATORY TESTING

APPENDIX B – SUBSURFACE EXPLORATION

APPENDIX C – MCE_R GROUND MOTION HAZARD ANALYSIS

GEOTECHNICAL INVESTIGATION
RIVERBEND RESIDENTIAL DEVELOPMENT
529 MADISON STREET
PETALUMA, CALIFORNIA

1.0 INTRODUCTION

This report summarizes Miller Pacific Engineering Group's (MPEG) Geotechnical Investigation for the planned residential subdivision located at 529 Madison Street in Petaluma, California. A Site Location Map is shown on Figure 1. The purpose of our Geotechnical Investigation is to explore the subsurface soil and groundwater conditions, evaluate geologic hazards that may affect the planned development, and provide geotechnical design criteria for the project. In accordance with our proposal dated February 5, 2021, we are providing our geotechnical engineering services in three phases: 1) Geotechnical Investigation for the proposed improvements, 2) supplemental consultation and geotechnical design review, and 3) construction observation and testing. This report completes our Phase 1 services and includes the following:

- Review of readily available published geologic and geotechnical reference data;
- Exploration of the subsurface conditions with six Cone Penetration Tests (CPTs);
- Laboratory testing of select samples to determine the pertinent engineering properties of the soil layers;
- Evaluation of geologic hazards and development of conceptual mitigation measures;
- Development of geotechnical recommendations and design criteria, including site grading and foundation design for the project; and,
- Preparation of this report summarizing our findings.

2.0 PROJECT DESCRIPTION

The proposed Riverbend residential development consists of 29 lots on approximately 3.4 acres of land. The proposed project is to include the development of two-story residential structures, along with new infrastructure including streets, underground utilities, and associated improvements. We anticipate that foundation loading associated with the proposed wood frame residential structures will be relatively light. The proposed residential structures will have a minimum setback of 50 feet from the top of the Petaluma River channel. The proposed improvements are shown on the attached Site Plan, Figure 2.

3.0 SITE CONDITIONS

3.1 Regional Geology

The project site lies within the Coast Ranges geomorphic province of California. Regional topography within the Coast Ranges province is characterized by northwest-southeast trending mountain ridges and intervening valleys that parallel the major geologic structures, including the San Andreas Fault System. The province is also generally characterized by abundant landsliding and erosion, owing in part to its typically high levels of precipitation and seismic activity.

The oldest rocks in the region are the sedimentary, igneous, and metamorphic rocks of the Mesozoic-age (225- to 65-million years old) Franciscan Assemblage. Within Sonoma County, Franciscan rocks are in fault contact with marine sedimentary rocks of the Great Valley Sequence which are of similar age. Locally, a variety of sedimentary and volcanic rocks of Tertiary (1.8- to 65-million years old) and Quaternary (less than 1.8-million years old) age unconformably overlie the basement rocks of the Franciscan Assemblage and Great Valley Sequence. Within Sonoma County, the late Miocene to Pliocene-age (approximately 2.6- to 11.6-million years old) Sonoma Volcanics comprise the majority of these rocks.

Tectonic deformation and erosion during late Tertiary and Quaternary time (the last several million years) formed the prominent coastal ridges and intervening valleys typical of the Coast Ranges province. The youngest geologic units in the region are Quaternary-age (last 1.8 million years) sedimentary deposits, including alluvial deposits which partially fill most of the valleys and colluvial deposits which typically blanket the lower portions of surrounding slopes.

Regional geologic mapping (Bezore, et al, 2002) indicates that the majority of the project site is underlain by alluvial terrace deposits of latest Holocene age. Terrace deposits are commonly composed of sands, gravels, silts, and minor clays and tend to be moderately to well sorted. A regional geologic map is presented on Figure 3.

3.2 Surface Conditions

The roughly 3.4-acre site is currently undeveloped and is bordered on the northeast by Edith Court, to the southeast by Madison Street, to the southwest by Clover Stornetta lands, and to the northwest by the Petaluma River. Historic maps indicate that a former structure was located near the Madison Street frontage of the property, which has since been demolished. The northwestern portion of the site is elevated three to four feet above that portion of the site adjacent to Madison Street. The elevated portion of the site is blanketed with an undocumented older granular fill layer which we understand was placed during previous Corps of Engineers construction work along the Petaluma River. The site supports a heavy growth of weeds and wild grasses, and some small shrubs and trees.

3.3 Field Exploration and Laboratory Testing

Miller Pacific Engineering Group (MPEG) conducted geotechnical investigations for previously planned developments at the Clover Site in 2001 and 2006. Seven exploratory borings were performed as a part of the 2001/2006 geotechnical studies. The previous MPEG boring logs are included in the attached Appendix A.

Subsurface exploration conducted by MPEG at the project site consisted of one Cone Penetration Test (CPT) performed on May 7, 2015, and five Cone Penetration Tests performed on August 25, 2021, at the approximate locations shown on the Site Plan, Figure 2.

The Cone Penetration Test (CPT) is an 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. A description of the CPT and CPT logs are described on Figures B-1 through B-7. Additionally, to aid in determining the site classification, we performed a shear wave velocity profile on two CPTs. The results are presented on Figure B-8 in Appendix B.

3.4 Subsurface Conditions

Subsurface data generally confirms the regionally mapped geology. The project site is mantled with between one and six feet of undocumented fill over the northwestern roughly three quarters of the site. The fill is composed of a mixture of sand and gravel, with minor amounts of silt and clay. The fill is undocumented and appears to be poorly compacted. Below the fill, the native soil consists of roughly four to eight feet of medium stiff to stiff brown to dark brown highly plastic (expansive) silty clay, underlain by layers of medium dense to dense clayey sand and gravelly sand, interbedded with layers of medium stiff to stiff sandy and silty clay (alluvial soil deposit).

Groundwater was encountered in most of the borings and CPTs at a depth between 9 and 15-feet below the ground surface. Groundwater levels may be shallower during the winter months or following periods of heavy rain. Temporary perched groundwater is anticipated within a few feet of the ground surface during and after periods of heavy rainfall. In general, the groundwater levels are anticipated to correspond relatively closely with the water level in the adjacent Petaluma River.

3.5 Seismicity

The project site is located within a seismically active region that includes the Central and Northern Coast Mountain Ranges. Several active faults are present in the area, including the Rodgers Creek, San Andreas, Maacama, Hayward, and San Gregorio Faults, among others. An "active" fault is defined as one that shows displacement within the last 11,000 years and, therefore, is considered more likely to generate a future earthquake than a fault that shows no evidence of recent rupture. The California Department of Conservation, Division of Mines and Geology has mapped various active and inactive faults in the region (CDMG, 1972 and 2000). These faults are shown in relation to the project site on the attached Active Fault Map, Figure 4. The Rodgers Creek Fault is the nearest known active fault and is located approximately 8.0 kilometers (4.9-miles) east of the site.

3.5.1 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. Two significant earthquakes have struck the Sonoma County area in recent history that have caused significant damage.

The first earthquake that caused significant damage was the 1906 San Francisco Earthquake (M7.9); which reportedly resulted in a Modified Mercalli Scale of IX (Lawson, 1908). The Modified Mercalli Intensity scale is based on observed damage and the public response during a seismic event. A Modified Mercalli Intensity of IX typically results in general public panic, damage to masonry buildings ranging from collapse to serious damage unless modern design, racked wood-framed structures, structures shifted off foundations; if not bolted to the foundation and broken underground utilities." Reported damage included multiple structural collapses, including Santa Rosa City Hall, and structures sliding off foundations. Additionally, 60 to 65-lives were lost as a result of the earthquake.

The second earthquake that caused significant structural damage was the 1969 (M5.6) Santa Rosa Earthquake. This earthquake reportedly resulted in a Modified Mercalli Intensity of VIII (Cloud et. al., 1970). A Modified Mercalli Intensity of VIII typically results in affected steering of cars, extensive damage to unreinforced masonry buildings, including partial collapse, fall of some masonry walls, twisting and falling of chimneys and monuments, structures shifted off foundations, if not bolted to the foundation; loose partition walls thrown out of plumb and broken tree branches. Reported damage included approximately 99-structures heavily damaged with many requiring abandonment. No deaths were associated with this earthquake.

3.5.2 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 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 (aka UCERF, UCERF2, and UCERF3, respectively). 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.

The 2003 study (UCERF) specifically analyzed fault sources and earthquake probabilities for the seven major regional fault systems in the Bay Area region of northern California. The 2008 study (UCERF2) applied many of the analyses used in the 2003 study to the entire state of California and updated some of the analytical methods and models. The most recent 2015 study (UCERF3) further expanded the database of faults considered and allowed for consideration of multi-fault ruptures, among other improvements.

Conclusions from the most recent UCERF3 indicate the highest probability of an M>6.7 earthquake on any of the active faults in the San Francisco Bay region by 2045 is assigned to the Hayward/Rodgers Creek Fault, located approximately 8.0-kilometers east of the site, at 33%. 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.

4.0 GEOLOGIC HAZARDS EVALUATION

The principal geologic hazards which could potentially affect the project site are strong seismic shaking from future earthquakes in the San Francisco Bay Region, liquefaction, lurching and ground cracking, and expansive soils. Other hazards, such as fault rupture and settlement are not considered significant at the site. More detailed discussion of each geologic hazard considered, their anticipated impacts, and recommended mitigation measures are discussed below.

4.1 Fault Surface Rupture

Under the Alquist-Priolo Earthquake Fault Zoning Act, the California Geological Survey (CDMG)/California Geologic Survey (CGS) (1972, 2000) produced 1:24,000 scale maps showing all known active faults and defining zones within which special fault studies are required. Based on currently available published geologic information, the project site is not located within an Alquist-Priolo Earthquake Fault Zone (CGS, 2000). The potential for fault surface rupture on the project site is therefore considered to be low.

Evaluation: *No significant impact.*
Recommendations: *No special engineering measures are required.*

4.2 Seismic Shaking

The site will likely experience seismic ground shaking from future earthquakes in the San Francisco Bay Area. Earthquakes along several active faults in the region, as shown on Figure 4, could cause moderate to strong ground shaking at the site.

4.2.1 Deterministic Seismic Hazard Analysis

Deterministic Seismic Hazard Analysis (DSHA) predicts the intensity of earthquake ground motions by analyzing the characteristics of nearby faults, distance to the faults and rupture zones, earthquake magnitudes, earthquake durations, and site-specific geologic conditions. Empirical relations (Abrahamson, Silva & Kamai, Boore, Stewart, Seyhan & Atkinson, Campbell & Borzognia, and Chiou & Youngs, (2014)) for the stiff soil subsurface conditions were utilized to provide approximate estimates of median peak site accelerations. A summary of the principal active faults affecting the site, their closest distance, moment magnitude of characteristic earthquake, probable median accelerations and plus one standard deviation (+1σ), peak ground accelerations (PGA) for earthquakes on faults near the site are shown in Table A.

TABLE A
DETERMINISTIC PEAK GROUND ACCELERATION
Riverbend Residential Development
529 Madison Street
Petaluma, California

Fault	Fault Distance[1]	Moment Magnitude[1]	Median PGA[1,2,3,4]	+1σ PGA[4]
Rodgers Creek	8 km	7.3	0.36 g	0.61 g
San Andreas	24 km	8.0	0.26 g	0.44 g
Maacama	32 km	7.4	0.17 g	0.29 g
Hayward	32 km	7.3	0.16 g	0.28 g
San Gregorio	38 km	7.4	0.15 g	0.26 g

1. Values determined using USGS Quaternary Fault and Fold Database, https://www.usgs.gov/natural-hazards/earthquake-hazards/faults, accessed September 15, 2021.
2. Values determined using Vs^{30} = 270 m/s for Site Class "D" in accordance with the 2019 CBC and ASCE-7-16. Note actual ground accelerations may be higher or lower depending on the exact location and underlying geologic conditions.

4.2.2 Probabilistic Seismic Hazard Analysis

Probabilistic Seismic Hazard Analysis (PSHA) 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 PGA for two separate probabilistic conditions, the 2% chance of exceedance in 50 years (2,475-year statistical return period) and the 10% chance of exceedance in 50 years (475-year statistical return period), utilizing the online USGS Unified Hazard Tool (USGS, 2021). The results of the probabilistic analyses are presented below in Table B.

TABLE B
PROBABILISTIC SEISMIC HAZARD ANALYSES
Riverbend Residential Development
529 Madison Street
Petaluma, California

	Statistical Return Period	Magnitude	PGA
2% in 50 years	2,475 years	7.1	0.81 g
10% in 50 years	475 years	7.1	0.46 g

Reference: USGS Unified Hazard Tool (2021)

The potential for strong seismic shaking at the project site is high. Due to its close proximity, the Rodgers Creek Fault (approximately 8.0 kilometers east of the site) presents the highest potential for strong ground shaking. The most significant adverse impact associated with strong seismic shaking is potential damage to structures and improvements.

Evaluation: *Less than significant with mitigation.*
Recommendations: *Minimum mitigation measures should include designing the structures and foundations in accordance with the most recent version of the California Building Code. Recommended seismic coefficients are provided in Section 5.2 of this report.*

4.3 Liquefaction Potential and Related Impacts

Liquefaction refers to the sudden, temporary loss of soil shear strength during strong ground shaking. Liquefaction-related phenomena include liquefaction-induced settlement, flow failure, and lateral spreading. These phenomena can occur where there are saturated, loose, granular deposits. Recent advances in liquefaction studies indicate that liquefaction can occur in granular materials with a high, 35 to 50%, fines content (soil particles that pass the #200 sieve), provided the fines exhibit a plasticity index of less than 7. Saturated granular layers were observed during our subsurface exploration. Additionally, regional mapping indicates the site lies in a zone of “high liquefaction susceptibility”, as shown on Figure 6.

4.3.1 Liquefaction Evaluation

To evaluate soil liquefaction, the seismic energy from an earthquake is compared with the ability of the soil to resist pore pressure generation, known as the Cyclic Resistance Ratio (CRR). The earthquake energy is termed the cyclic stress ratio (CSR) and is a function of the maximum considered earthquake peak ground acceleration (PGA) and depth. Soil resistance to liquefaction is based on its relative density, and the amount and plasticity of the fines (silts and clays). The relative density of cohesionless soil is correlated with the Standard Penetration Test (SPT) blow count data measured in the field and corrected for hammer efficiency, overburden, and percent fines to determine the $(N_1)_{60,CS}$ value. Cone Penetration Test data, corrected for overburden, can also be utilized to determine the relative density of a soils and subsequently its resistance to liquefaction.

We analyzed the potential for liquefaction utilizing the data from our borings and the procedures outlined by Idriss and Boulanger (2008 & 2010), considering a magnitude 7.3 earthquake producing a PGA of 0.75-g, which corresponds to the PGA_M value as defined in ASCE 7-10 Section 11.8.3. The liquefaction analysis software Cliq, developed by Geologismiki (2006), uses CPT data to evaluate liquefaction potential. The results of our liquefaction analyses are presented on Figures 7 through 12, and indicate several localized soil layers, ranging from a few inches to a few feet thick, may liquefy under a strong seismic event.

4.3.2 Post Liquefaction Settlement and Lateral Spread

We predicted the amount of post liquefaction settlement utilizing the procedures outlined by Idriss and Boulanger (2008, 2010 & 2014), which indicate post liquefaction settlement can occur in soils that exhibit a factor of safety against liquefaction of 2.0 or less. Based on our analyses, we predict up to about 2.0-inches of total settlement and 1.0-inch of differential settlement, over a horizontal distance of 30-feet, may occur during the design seismic event.

Based on the five CPT's conducted at the site, and the liquefaction analyses, it appears that a relatively continuous, variable thickness layer of soil between about 10-feet and 20-feet below the existing ground surface is susceptible to liquefaction. Due to the nearby proximity of the Petaluma River channel slope, there is a high risk of lateral movement of the upper 25-feet of soil beneath the project site toward the Petaluma River during a large seismic event. Based on our analyses, predicted ground surface lateral displacements of about one to five feet (depending on distance from the Petaluma River channel slope) may occur during the design seismic event.

Based on our analyses, as described above, it is our opinion that certain layers within the sand/gravel deposits may liquefy during a strong seismic event. Therefore, liquefaction and related settlement and lateral spread presents a high risk of damage to the planned improvements.

Evaluation:	*Less than significant with mitigation.*
Recommendations:	*Foundation systems should be designed to withstand up to 2.0-inches of total and 1.0-inch of differential settlement, over 30-feet. Foundation design criteria to mitigate the effects of liquefaction provided in Section 5.4 should be followed. Deep soil mixing should be utilized to strengthen a zone of soil along the Petaluma River frontage of the site to mitigate the risk of lateral spread during a strong seismic event.*

4.4 Seismically Induced Ground Settlement

Seismic ground shaking can induce settlement of 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. Subsurface exploration indicates the presence of some loose to medium dense sands above the groundwater level, including surficial undocumented fill and dredge materials. Therefore, the likelihood of damage to improvements at the site due to seismically induced ground settlement is high.

Evaluation: *Less than significant with mitigation.*
Recommendations: *Mitigation measures include removal/overexcavation of existing weak granular fill soil and replacement as compacted, engineered fill, and designing new foundations to span over localized areas of differential settlement.*

4.5 Cyclic Softening and Related Impacts

Cyclic softening refers to a loss of shear strength within a sensitive, cohesive, saturated, fine-grained soil (silt and clay) during a seismic event. The effects of cyclic softening can result in a reduction of the soil undrained shear strength that subsequently can cause a significant loss of bearing capacity or slope failures. Soft, sensitive, saturated, clay was not encountered during our subsurface exploration. Therefore, we judge that cyclic softening will not impact shallow foundation elements and presents a low risk of damage to the planned improvements.

Evaluation: *No significant impact.*
Recommendations: *No special engineering measures are required.*

4.6 Lurching and Ground Cracking

Lurching and associated ground cracking can occur during strong ground shaking. Lurching and ground cracking generally occurs along the tops of slopes where stiff soils are underlain by soft deposits or along steep channel banks. Lateral spreading generally occurs where liquefiable deposits flow towards a “free face”, such as channel banks, during an earthquake.

Conditions susceptible to lurching and ground cracking exist along the banks of the Petaluma River. Therefore, the potential for a negative impact to the project improvements is moderate to high.

Evaluation: *Less than significant with mitigation.*
Recommendations: *Structures and other improvements should be set-back from the top of the riverbank by 50 feet, when possible. Soil improvement methods, such as deep soil mixing, should be utilized to strengthen a zone of soil adjacent to the river channel.*

4.7 Erosion

Sandy soils on moderate slopes or clayey soils on steep slopes are susceptible to erosion when exposed to concentrated water runoff. The project site is relatively level (with the exception of the river channel slopes). We did not observe evidence of significant erosion on the project site or along the riverbank. Therefore, we consider the potential for erosion to adversely impact the proposed develop is low.

Evaluation: *Less than significant with mitigation.*
Recommendations: *Mitigation measures include designing a site drainage system to collect surface water and discharging it into an established storm drainage system. The project Civil Engineer of Architect is responsible for designing the site drainage system and, 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 Seiche and Tsunami

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 is not mapped (ABAG, 2021) as lying within a tsunami inundation zone. Therefore, seiche and tsunami events are not considered significant geologic hazards at the site.

Evaluation: *No significant impact.*
Recommendations: *No special engineering measures are required.*

4.9 Flooding

The project site is not mapped within a FEMA 100-year flood zone (ABAG, 2021). The site is located adjacent to the Petaluma River. Flood control improvements for the river are underway or have been completed. A detailed evaluation of the flooding potential at the project site and design of appropriate flood control and drainage improvements should be provided by the project Civil Engineer.

Evaluation: *Less than significant with mitigation.*
Recommendations: *The project Civil Engineer should evaluate the risk localized flooding and provide appropriate storm drain design.*

4.10 Dam Failure Inundation

Based on the Sonoma County Hazard Mitigation Plan Map (County of Sonoma, 2011) the site is not mapped in a Dam Failure Inundation zone. Therefore, the risk of inundation of the site from dam failure is judged low.

Evaluation: *No significant impact.*
Recommendations: *No special engineering measures are required.*

4.11 Expansive Soil

Expansive soils will shrink and swell with fluctuations in moisture content and are capable of exerting significant expansion pressures on building foundations, interior floor slabs, and exterior flatwork. Distress from expansive soil movement can include cracking of brittle wall coverings (stucco, plaster, drywall, etc.), racked door and/or window frames, and uneven floors and cracked slabs. Flatwork, pavements, and concrete slabs-on-grade are particularly vulnerable to distress due to their low bearing pressures. High plasticity expansive clayey soil is present near the existing ground surface in portions of the project area. Excavation and fill placement is anticipated during site grading operations which will change the current conditions. The risk of damage due to expansive soils is generally moderate to high.

Evaluation: *Less than significant with mitigation.*
Recommendations: *Site grading should be performed to remove or lime treat highly expansive soil within the upper three feet under the planned improvements. Alternatively, foundations should be designed to account for some expansive soil movement.*

4.12 Settlement/Subsidence

Significant settlement can occur when new loads are placed at sites that are located over soft compressible clays, such as bay mud. The amount and rate of settlement is dependent on the magnitude of additional new loads (i.e., new structures and/or new fill), the thickness of compressible material, and the inherent compressibility properties of the bay mud.

Our subsurface exploration did not reveal the presence of bay mud or other soft, compressible clay layers beneath the site. Therefore, the risk of settlement due to compressible clay is considered to be low.

Evaluation: *No significant impact.*
Recommendations: *No special engineering measures are required.*

4.13 Slope Instability/Landsliding

Weak soils and bedrock on moderate to steep slopes can move downslope due to gravity. Slope instability is often initiated or accelerated by soil saturation and groundwater pressure. Slope movement can vary from slow, shallow soil creep to large, sudden debris flows. Landslides can cause significant damage to structures and improvements. The project site is relatively level with the exception of the river channel slopes, and planned improvements are typically setback 50 feet from the top of the channel slopes.

Evaluation: *Less than significant with mitigation.*
Recommendations: *Structures and other improvements should be set-back from the top of the riverbank by 50 feet, when possible. Soil improvement methods, such as deep soil mixing, should be utilized to strengthen a zone of soil adjacent to the river channel near development areas.*

4.14 Soil Corrosion

Corrosive soil and seawater can damage buried metallic structures and underground utilities, deteriorate rebar reinforcement, and cause spalling of concrete. Laboratory corrosivity testing of the site soils was not included in our current scope of services; however, designers of site utilities and structural steel and concrete elements should account for a potentially corrosive environment. Considering the potential presence of brackish water in and around the project site, we judge the hazard due to corrosion to be moderate to high.

Evaluation: *Less than significant with mitigation.*
Recommendations: *The project Civil and Structural Engineer should specify materials that are resistant to corrosive soil or provide cathodic corrosion protection. At least 3-inches of concrete coverage should be provided over reinforcing steel. Underground utilities should be constructed of plastic or PVC pipe when possible; metallic piping should be avoided.*

4.15 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 Sonoma County, California, which is mapped in radon gas Zone 3 by the United States Environmental Protection Agency (USEPA, 2019). Zone 3 is classified by the EPA as exhibiting a "low" potential for Radon-222 gas with average predicted indoor screening levels less than 2 pCi/L. Therefore, the potential for hazardous levels of radon at the project site is low.

Evaluation: *No significant impact.*
Recommendations: *No special engineering measures are required.*

4.16 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 51 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 special engineering measures are required.*

4.17 Naturally Occurring Asbestos (NOA)

Naturally occurring asbestos is commonly found in association with serpentinite and associated ultramafic rock types. These rocks are a major constituent of the Franciscan Complex, which underlies vast portions of the greater San Francisco Bay Area. However, the project site is underlain by a thick layer of river terrace deposits. Therefore, the likelihood of naturally occurring asbestos negatively impacting the proposed project is low.

Evaluation: *No significant impact.*
Recommendations: *No special engineering measures are required.*

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 General

Based on our investigation and our experience with similar projects in the area, we conclude that, from a geotechnical standpoint, the site is suitable for the planned improvements. The primary geotechnical issues to address in the design of the project are strong seismic shaking, mitigation of the risk of lateral soil displacement towards the river channel during strong seismic shaking, design of foundations to account for potential liquefaction induced differential vertical settlements, existing undocumented fill, and expansive near surface soil.

5.2 Seismic Design

The project site is located in a seismically active area. Therefore, new structures should be designed in conformance with the seismic provisions of the California Building Code (CBC) to mitigate the potential effects of strong seismic ground shaking to the proposed structures. However, since the goal of the building code is protection of life safety, some structural damage may still occur during strong ground shaking.

The site is underlain by thick alluvial deposits. Minimum mitigation of ground shaking includes seismic design of the structures in conformance with the provisions of the most recent version (2019) of the California Building Code. The magnitude and character of these ground motions will depend on the particular earthquake and the site response characteristics. Based on the interpreted subsurface conditions and proximity of the Rodgers Creek and San Andreas Faults, we recommend the CBC coefficients and site values shown in Table C below to calculate the design base shear of the new construction.

TABLE C
2019 CBC FACTORS
Riverbend Residential Development
529 Madison Street
Petaluma, California

Factor Name	Coefficient	2019 CBC Site Specific Value
Site Class[1]	$S_{A,B,C,D,E,\text{ or }F}$	S_D
Site Coefficient	F_a	1.00
Site Coefficient	F_v	-
Spectral Acc. (short)	S_S	1.50 g
Spectral Acc. (1-sec)	S_1	0.60 g
Spectral Response (short)	SM_S	1.62 g
Spectral Response (1-sec)	SM_1	1.77 g
Design Spectral Response (short)	SD_S	1.08 g
Design Spectral Response (1-sec)	SD_1	1.18 g
MCE_G[2] PGA adjusted for Site Class	PGA_M	0.75 g
Seismic Design Category	A,B,C,D, or E	D

Notes:
1. Site Class D Description: Stiff soil profile with shear wave velocities between 600 and 1,200 ft/sec, standard blow counts between 15 and 50 blows per foot, and undrained shear strength between 1,000 and 2,000 psf.
2. Maximum Considered Earthquake Geometric Mean

5.3 Site Preparation and Grading

The general grading recommendations presented below are appropriate for construction in the late spring through fall months (dry season). From winter through the early spring months, on-site soils may be saturated due to rainfall and may be difficult to compact without drying by aeration or the addition of lime and/or cement (or a similar product) to dry the soils.

Site preparation and grading should conform to the recommendations and criteria outlined below. General recommendations for wintertime construction are provided later in this report.

A portion of the site is mantled by undocumented fill ranging in thickness from about one to six feet. The existing undocumented fill is generally granular, consisting of silts, sands, and gravels, and will generally qualify as select, nonexpansive fill. The existing fill overlies expansive dark brown silty clay native soil. In some areas of the site, the native soil is exposed at the existing ground surface.

We recommend that all existing undocumented fill should be overexcavated to expose the native dark brown silty clay topsoil. The clayey subgrade at the bottom of the overexcavated areas should then be scarified to a minimum depth of 8-inches, moisture conditioned to at least three percent over optimum moisture content, and compacted to a minimum relative compaction of 90 percent. “Potholing” should be undertaken during the grading operation to verify that no loose, dry, desiccated expansive clay is buried beneath new fill in building and pavement areas.

The undocumented fill that is overexcavated from beneath building and pavement areas will generally qualify for use as select, nonexpansive fill. We recommend that the upper 24 inches of soil on building pads, and extending at least five feet beyond the building lines in all directions, should consist of select, nonexpansive fill. Nonexpansive fill can include on site soil or imported soil, and is defined in Section 5.3.4.

5.3.1 Surface Preparation

Clear all trees, brush, roots, over-sized debris, loose stockpiled soils, and organic material from areas to be graded. Trees that will be removed (in structural areas) must also include removal of stumps and roots larger than two inches in diameter. Excavated areas (i.e., excavations for stump removal) should be restored with properly moisture conditioned and compacted fill as described in the following sections. Any loose soil or undocumented fill at subgrade will need to be excavated to expose firm natural soils (estimated depth one to six feet below existing site grades). Debris, rocks larger than four inches, and vegetation are not suitable for structural fill and should be removed from the site. Alternatively, vegetation strippings may be used in landscape areas.

5.3.2 Lime or Cement Treatment

As previously discussed, much of the near surface natural alluvial soil (upper one to five feet or more) consists of high plasticity silt and clay with a high expansive potential. To mitigate the expansive potential of the soil beneath buildings and pavement areas, the site may be lime or cement treated. Lime/cement treatment chemically alters the clay soils, resulting in a reduction in the inherent plasticity, a significant reduction in the shrink/swell potential, an improvement to workability (i.e., compaction), and an increase of the shear strength. If lime treatment is utilized during site grading, in structural areas we preliminarily recommend at least 5% high calcium lime (by weight) should be thoroughly mixed with the surficial soils (utilizing a 120 pcf soil density) to a depth of 24-inches beneath building footprint areas and extending at least 5-feet beyond the building footprint in all directions. Laboratory testing should be performed on representative samples prior to the lime treatment operation to establish the percentage, by dry weight, of lime to be used to ensure that the maximum plasticity index of the treated soil is 12 and the minimum pH is 12.4. The lime treating process will need to be conducted in at least two lifts, each lift having a thickness of no more than 18 inches. The lime should be thoroughly mixed into the native clayey soil using a rotary type mixer. The performance of lime stabilized soil is critically dependent on uniform mixing of the lime into the highly expansive soil and providing a proper curing period following amendment with the lime.

Pavement area subgrade soil can also be lime or cement treated to reduce the required thickness of Class 2 baserock in the pavement structural section. In this case, we recommend that the native subgrade soil beneath new asphalt pavement areas should be lime treated to a minimum depth of 18 inches below the subgrade level and extending at least 3-feet beyond the edge of pavement. Treated soils should then be compacted to at least 90% relative compaction in structural areas and 95% relative compaction in areas subject to vehicular loads.

5.3.3 Over Excavation

If lime treatment is not utilized to improve the soil conditions, the upper 24-inches of existing highly plastic and expansive soil below the subgrade in building areas (where present), and extending 5-feet beyond building areas, should be removed from the site. The excavated surface should be free of loose material and kept moist to prevent soil shrinkage. Non-expansive on site or imported fill, as described below in Section 5.3.4, should be placed, and compacted as described in Section 5.3.5.

5.3.4 Materials

Based on our laboratory testing, native on-site soil is typically highly plastic and expansive and is not suitable for use as select, nonexpansive fill unless the soil is lime treated. The existing undocumented on-site fills are typically granular and have low expansion potential and would generally be suitable for use as select, nonexpansive fill. If imported fill is required, the material shall be free of toxic contamination and shall consist of soil and rock mixtures that: (1) are free of organic material, (2) have a Liquid Limit less than 40 and a Plasticity Index of less than 12, (3) have a maximum particle size of four inches, and (4) have more than 50% retained on the No. 200 sieve. Any imported fill material shall be tested and inspected by the project geotechnical engineer prior to importing to the site to determine its suitability for use as fill material.

5.3.5 Compacted Fill

On-site fill, backfill, and scarified subgrades should be conditioned to at least 3% over the optimum moisture content. Properly moisture conditioned and cured on-site materials should subsequently be placed in loose horizontal lifts of 8 inches thick or less, and uniformly compacted to a minimum of 90% relative compaction. Relative compaction, maximum dry density, and optimum moisture content of fill materials should be determined in accordance with ASTM Test Method D 1557, "Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using a 10-lb. Rammer and 18-in. Drop."

5.4 Foundation Design

Provided that site preparation and grading are performed in accordance with the recommendations above, new building loads can be supported on post-tensioned slab foundation systems. The foundation systems should be designed to resist up to 1.0-inch of differential settlement over a horizontal distance of 30-feet. Foundation design criteria are shown in Table D.

TABLE D
SHALLOW FOUNDATION DESIGN CRITERIA
Riverbend Residential Development
529 Madison Street
Petaluma, California

Post-Tensioned Slab:

Minimum Post-Tensioned Slab Thickness	12 inches
Edge moisture variation (e_m) – Center Lift	9 feet
Edge moisture variation (e_m) – Edge Lift	5 feet
Differential soil movement (y_m) – Center Lift	1.5 inches
Differential soil movement (y_m) – Edge Lift	1.5 inches
Allowable bearing capacity: [1,2,3]	1,500 psf

Notes:

(1) Dead plus live loads. May increase by 1/3 for total design loads, including wind and seismic.

(2) Foundations to bear on compacted nonexpansive engineered fill, placed, and compacted in accordance with the recommendations presented in Section 5.3 of this report.

(3) Post-tensioned slab thickened slab edge should extend to a minimum depth of 12-inches below the rough pad grade to confine soil beneath the slab and to reduce storm water intrusion under the slab.

5.5 Site and Foundation Drainage

We recommend that the project Civil Engineer should design the building pad elevations so as to avoid flooding of the proposed structures and ponding of water near structures. We also recommend that the project Civil Engineer be responsible for design of site drainage systems, and that site drainage be carefully considered during design of finished grades.

We recommend that landscaped areas adjoining new structures be sloped downward at least 0.25 feet for 5 feet (5%) 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%). Roof gutter downspouts may discharge onto the pavements but should not discharge onto any landscaped areas. The gutter downspout discharge should be designed as an independent system and should not be connected to foundation or other subdrain discharge systems. Provide area drains for landscape planters adjacent to buildings and parking areas and collect downspout drainage into a nonperforated pipe collection system directed to a suitable discharge point.

Foundation drains consisting of perforated pipe within drain rock and filter fabric, or Caltrans Class II permeable material should be considered. Seepage should be collected into a nonperforated pipe system and conveyed to an appropriate discharge point as with other drainage. As noted above, the foundation drains should not be connected to the roof gutter discharge system. We recommend the use of Schedule 40 PVC, SDR-35 PVC, or equivalent materials for subdrain and nonperforated pipe construction; lower-strength and lesser-quality materials such as ABS plastic and corrugated or slotted pipe should be avoided for best future performance.

All site drainage should be conveyed via nonperforated pipe away from the development areas and discharged at an appropriate location unlikely to result in significant erosion. If no connection to an established storm drain system is available, then runoff should be conveyed to an established existing drainage channel. Ideally, the drainage system would be designed to reduce peak flow rates to pre-development conditions via use of bio-retention or detention basins and appropriately designed outflow works.

5.6 Concrete Slabs-On-Grade

We recommend that interior concrete slabs should be placed on a minimum 24-inch-thick layer of compacted lime treated soil or compacted nonexpansive soil fill over a moist compacted subgrade as previously described above.

To reduce (i.e., improve) interior moisture conditions, a minimum of four inches of clean, free draining, ¾-inch angular gravel should be placed beneath all interior concrete slabs to form a capillary moisture break. The drain rock must be placed on a properly moisture conditioned and compacted subgrade that has been approved by the Geotechnical Engineer. A 15-mil, or thicker, vapor barrier should be placed over the compacted drain rock. The vapor barrier shall meet the ASTM E 1745 Class A requirements and be installed per ASTM E 1643. Eliminating the capillary moisture break and/or vapor barrier may result in excess moisture intrusion through the floor slabs resulting in poor performance of floor coverings, mold growth, or other adverse conditions.

It should be pointed out that where the gravel capillary break layer is placed beneath floor slabs, there is a possibility that water will tend to collect in the gravel layer and become trapped. If this condition occurs, the potential for moisture problems at the surface of the slab will be increased. One method of minimizing the potential for this to occur would be to construct a subdrain trench through and just below the gravel layer so that water collected in this area can escape. The subdrain should extend at least 12 inches below the base of the slab and 6 inches below the bottom of the gravel layer, and would consist of a four-inch diameter, perforated pipe (Schedule 40 PVC) surrounded by gravel. The subdrain would connect to the gravel layer beneath the slab, and the pipe should lead (at a minimum one percent slope) to a storm drain or another suitable outlet point. The outlet pipe should transition to nonperforated pipe at a point three feet inside the perimeter footing of the structure. A compacted clayey soil plug or other type of moisture barrier should be used at the point where the outlet pipe penetrates the perimeter footing to prevent seepage from back-flowing into the underslab gravel layer.

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

To minimize expansive soil movement and damage, exterior concrete slabs should have a minimum thickness of five inches and should be underlain with at least 4-inches of Caltrans Class 2 Aggregate Base compacted to at least 92% relative compaction over 12 inches of compacted, nonexpansive fill over a moist compacted subgrade as previously described above. Additionally, contraction joints should be incorporated in the concrete slab in both directions, no greater than 10 feet on center and the reinforcing bars should extend through these control joints. For improved performance, exterior concrete slabs may be underlain with a thicker section of Caltrans Class 2 Aggregate Base compacted to at least 92% relative compaction.

5.7 Asphalt Pavements

Typically, asphalt pavement sections are designed utilizing two variables, the R-Value (a measure of the subgrade resistance) and the Traffic Index (a measure of the type and amount of daily traffic). Based on the subsurface conditions, we judge an R-Value of 5 is appropriate for the untreated on-site soil, and an R-value of 40 may be used for a lime treated soil subgrade. We anticipate the proposed pavement section in parking areas will be subjected to a moderate volume of daily vehicular loads (Traffic Index 5.0). Pavement areas used for bus or light truck traffic should be designed using a Traffic Index of 6.0. If new pavements will be required to support fire apparatus loading, we should be consulted for supplemental pavement design recommendations. Recommended pavement structural sections are provided in Table E.

TABLE E
RECOMMENDED PAVEMENT STUCTURAL SECTIONS
Riverbend Residential Development
529 Madison Street
Petaluma, California

Native soil subgrade, assumed R-value of 5:

Traffic Index	Asphaltic Concrete	Aggregate Baserock
5.0	3.0 inches	10.0 inches
6.0	3.5 inches	13.0 inches

Lime treated soil subgrade (minimum 18 inch), assumed R-value of 40:

Traffic Index	Asphaltic Concrete	Aggregate Baserock
5.0	3.0 inches	6.0 inches
6.0	3.5 inches	6.0 inches

Notes:

1.) Roughly equivalent performance is possible by substituting one inch of additional asphalt for two inches of aggregate base. Minimum asphalt thickness is shown.
2.) Section thicknesses based on Caltrans Flexible Pavement Design Procedures.

The aggregate baserock should conform to Caltrans Class 2 Aggregate Baserock (Class 2 AB) outlined in Section 26 of the Caltrans Standard Specifications. The Class 2 AB shall be placed in layers on a properly prepared and firm and unyielding subgrade as described in the previously discussed grading recommendations. The Class 2 AB should be compacted to at least 95% relative compaction. Additionally, the Class 2 AB section should be firm and unyielding when proof rolled under heavy construction equipment.

5.8 Utility Trench Excavations and Backfills

Excavations for utilities will most likely extend into medium stiff to stiff silty and clayey soils. Trench excavations having a depth of five feet or more that will be entered by workers must be sloped, braced, or shored in accordance with current Cal/OSHA regulations. On-site soils appear to be Type C. All excavations where collapse of excavation sidewall, slope or bottom could result in injury or death of workers, should be evaluated by the contractor's safety officer, and designated competent person prior to entering in accordance with current Cal/OSHA regulations.

Bedding materials for utility pipes should be well graded sand with 90 to 100% of particles passing the No. 4 sieve and no more than 5% finer than the No. 200 sieve. Provide the minimum bedding beneath the pipe in accordance with the manufacturer's recommendation, typically 3 to 6 inches. Trench backfill may consist of on-site soils, moisture conditioned to within 2% of the optimum moisture content, placed in thin lifts and compacted to a minimum of 90% relative compaction. Backfill for trenches within pavement areas should consist of non-expansive granular fill. Use equipment and methods that are suitable for work in confined areas without damaging utility conduits. Where utility lines cross under or through perimeter footings, they should be sealed to reduce moisture intrusion into the areas under the slabs and/or footings.

5.9 Wintertime Construction

Wintertime/wet weather site work is feasible during the construction phase of this project, provided that weather conditions do not adversely impact the planned grading and proper erosion control measures are implemented to prevent excessive silt and mud from entering the storm drain system. High soil moisture contents and muddy site conditions may impact placing fills, compacting subgrades, and excavating foundation trenches. Several alternatives may be considered to improve the site conditions to allow site work to proceed in rainy conditions:

- Prior to the onset of winter rains, maintain a drier site by covering the work area and any stockpiled materials with plastic membrane sheeting or other impermeable membrane. Where asphalt pavements, other hardscape or drainage improvements currently exist in work areas, consider leaving these improvements in place until the last possible moment to maintain a drier subgrade condition.

- Lime treat the subgrade soils when site work commences to "weatherproof" the site. The disadvantage to this alternative is that future landscaping will likely require excavation and replacement of the treated soils for acceptable plant growth.

- Finally, imported, drier fill materials could be used to stabilize the site. Soft or wet on-site materials could be excavated to firm materials and drier (preferably granular) soils with good drainage characteristics would be imported to restore site grades. This alternative might also require future excavation and replacement of landscaping soils.

If construction occurs relatively early in the winter, we judge the first option (covering the site prior to winter rains) could be an effective method of maintaining a workable site. When the construction schedule and weather conditions are known, we can meet with the project team to further discuss alternatives to continuation of wintertime construction.

6.0 SUPPLEMENTAL GEOTECHNICAL SERVICES

We must review the plans and specifications for the project when they are nearing completion to confirm that the intent of our geotechnical recommendations has been incorporated and provide supplemental recommendations, if needed. During construction, we must observe and test site grading, and observe foundation excavations for the structures and associated improvements to confirm that the soil conditions encountered during construction are consistent with the design criteria presented in this report.

7.0 LIMITATIONS

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 Lenox Homes and/or its assignees specifically for this project. No other warranty, expressed or implied, is made. Our evaluations and recommendations are based on the data obtained during our subsurface exploration program and our experience with soil conditions in this geographic area.

Our approved scope of work did not include an environmental assessment of the site. Consequently, this report does not contain information regarding the presence or absence of toxic or hazardous wastes in the soil and groundwater at the site.

The evaluations and recommendations do not reflect variations in subsurface conditions that may exist between boring locations or in unexplored portions of the site. Should such variations become apparent during construction, the general recommendations contained within this report will not be considered valid unless MPEG is given the opportunity to review such variations and revise or modify our recommendations accordingly. No changes may be made to the general recommendations contained herein without the written consent of MPEG.

We recommend that this report, in its entirety, be made available to project team members, contractors, and subcontractors for informational purposes and discussion. We intend that the information presented within this report be interpreted only within the context of the report as a whole. No portion of this report should be separated from the rest of the information presented herein. No single portion of this report shall be considered valid unless it is presented with and as an integral part of the entire report.

8.0 LIST OF REFERENCES

American Society of Civil Engineers, “Minimum Design Loads for Buildings and Other Structures,” (ASCE 7), 2016.

American Society for Testing and Materials, “2016 Annual book of ASTM Standards, Section 4, Construction, Volume 4.08, Soil and Rock; Dimension Stone; Geosynthetics,” ASTM, Philadelphia, 2016.

California Building Code, 2019 Edition, California Building Standards Commission, Sacramento, 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, 2020.

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

Cloud, W.R., Hill, D.M., Huffman, M.E., Jennings, C.W., McEvilly, T.V., Nason, R.D., Steinbrugge, K.V., Tocher, D. Unger, J.D., & Youd, T.L., “The Santa Rosa Earthquake of October 1969,” California Department of Mines and Geology, Volume 23 #3, March 1970.

California Division of Mines and Geology, Special Publication 42, “Alquist-Priolo Special Studies Zone Act,” 1972 (Revised 1988).

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.

Field, E.H. et al (2015), "Long-Term Time-Dependent Probabilities for the Third Uniform California Earthquake Rupture Forecast (UCERF3)", Bulletin of the Seismological Society of America, Volume 105, No. 2A, 33pp., April 2015, doi: 10.1785/0120140093.

Lawson, A.C., “The California Earthquake of April 18, 1906, Report of the State Earthquake Investigation Commission,” 1908.

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.

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.

Shahi, K.S. & Baker, J.W., “NGA Models for Ground Motion Directionality” May 2013.

United States Environmental Protection Agency (USEPA) (2019), EPA Map of Radon Zones in California, http://www.epa.gov/radon/states/california.html, accessed September 15, 2021.

United States Geologic Survey (USGS), “Documentation for the 2008 Update of the United States National Seismic Hazard Maps,” Open-File-Report 2008-1128, 2008.

United States Geological Survey (USGS), "Earthquake Probabilities in the San Francisco Bay Region, 2002 to 2031 – A Summary of Finding," The Working Group on California Earthquake Probabilities, Open File Report 99-517, 2003.

United States Geological Survey & SC/EC, "The Uniform California Earthquake Rupture Forecast, Version 2 (UCERF 2), 2007 Working Group on California Earthquake Probabilities, USGS Open File Report 2007-1437, CGS Special Report 203, SCEC Contribution #1138, 2008.

SITE: LATITUDE, 38.2417°
LONGITUDE, -122.6383°

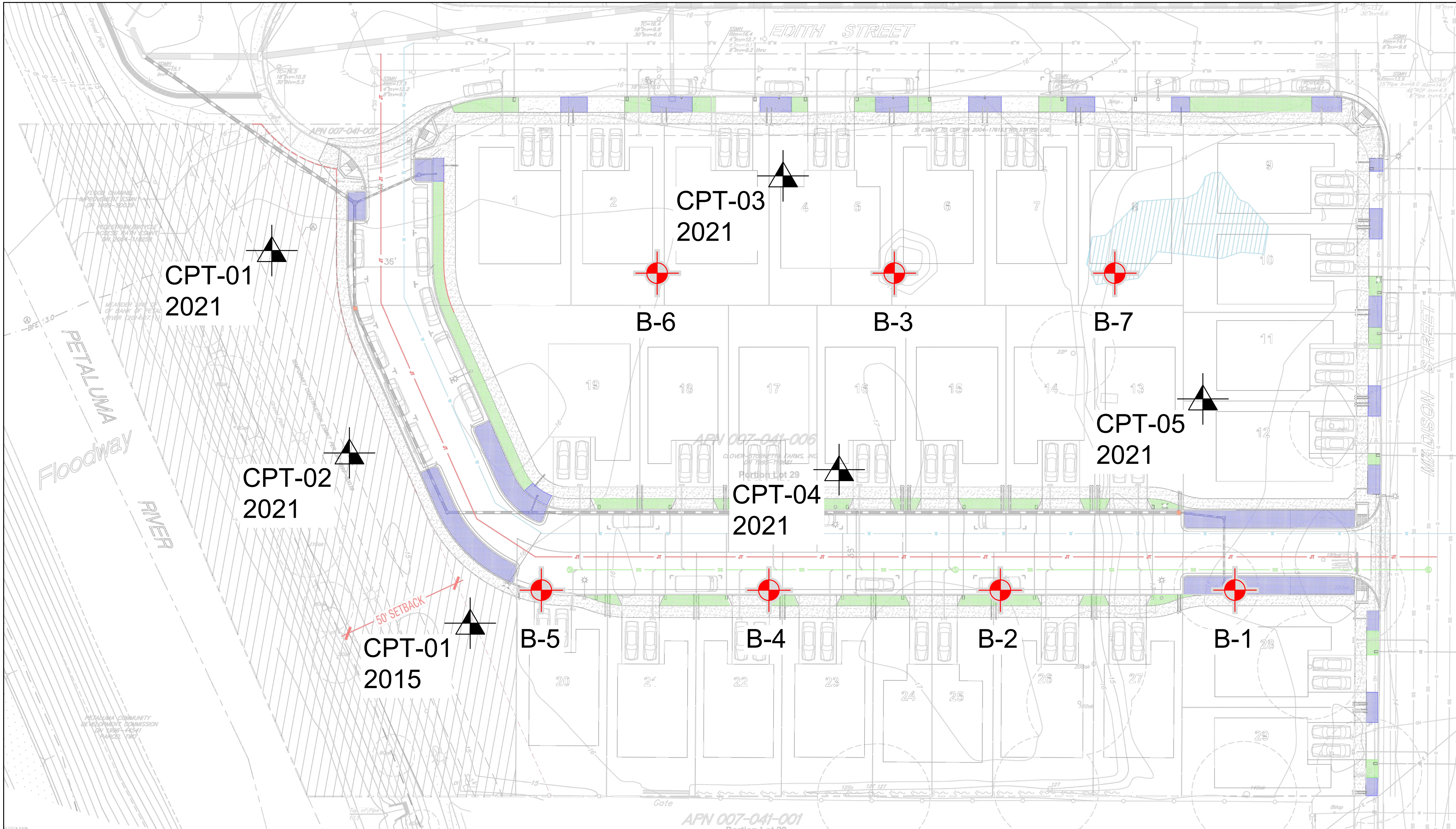
SITE LOCATION

N.T.S.

NORTH

REFERENCE: Google Earth, 2021

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
A CALIFORNIA CORPORATION, © 2018, ALL RIGHTS RESERVED FILENAME: 2066.001 Figures2021.dwg		Riverbend Development 529 Madison Street Petaluma, California Project No. 2066.001 Date: 9/15/2021
		Drawn MMT Checked
		1 FIGURE



Approximate location of boring completed by MPEG, 2001/2006

Approximate location of CPT completed by MPEG, 2015 and 2021

SCALE

0 20 40 80 FEET

NORTH

REFERENCE: Steven L. Lafranchi & Associates, Inc., Composite Exhibit, Clover Site, Sheet EX-1, 01.09.2019.

MPEG MILLER PACIFIC ENGINEERING GROUP

FILE: 2066.001 Figures2021.dwg

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

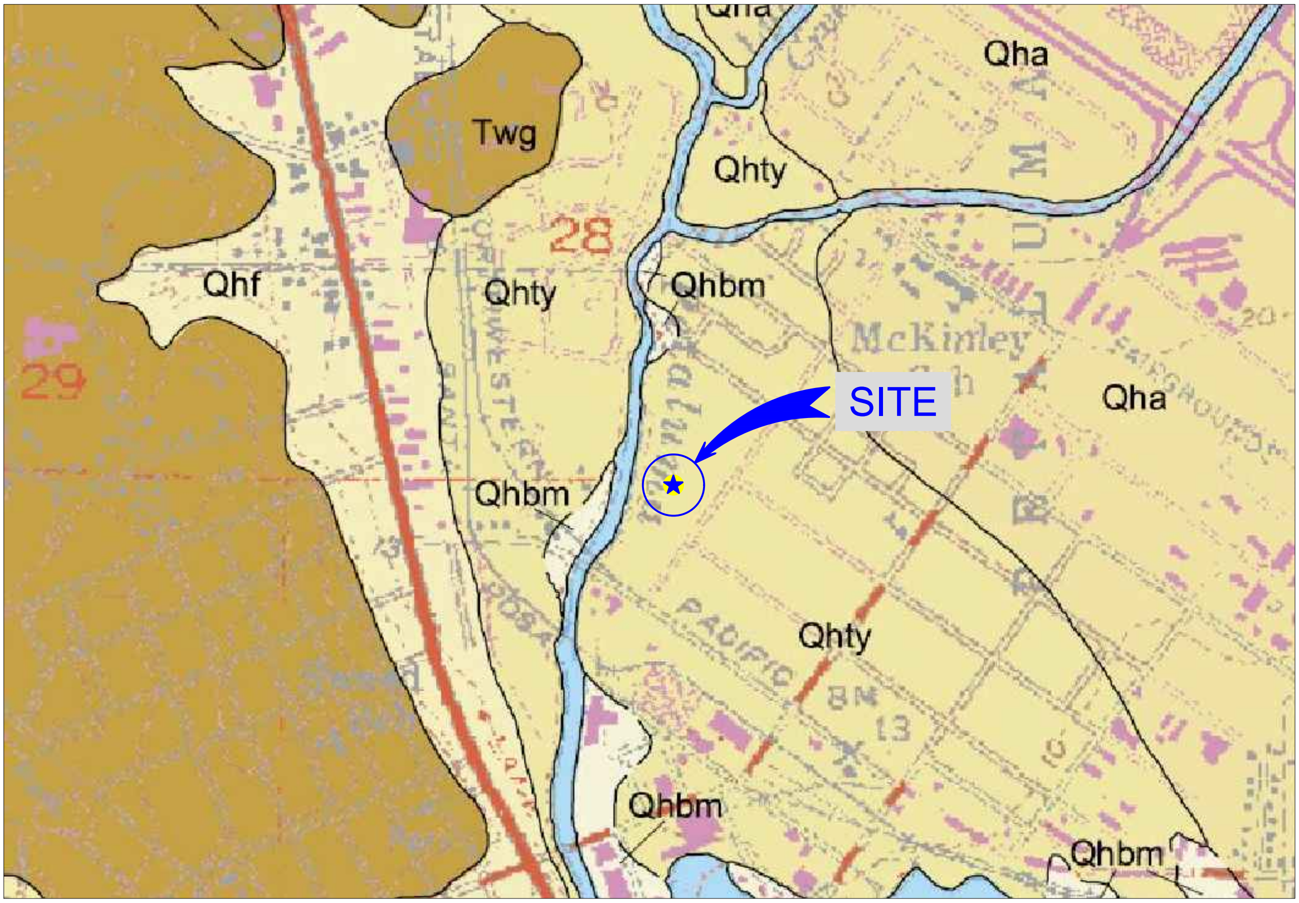
SITE PLAN

Riverbend Development
529 Madison Street
Petaluma, California

Project No. 2066.001 Date: 9/15/2021

Designed DSC
Drawn MMT
Checked DSC

2
FIGURE



REGIONAL GEOLOGIC MAP

(NOT TO SCALE)

NORTH

LEGEND

LEGEND:

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

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

MPEG MILLER PACIFIC ENGINEERING GROUP

FILENAME: 2066.001 Figures2021.dwg

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

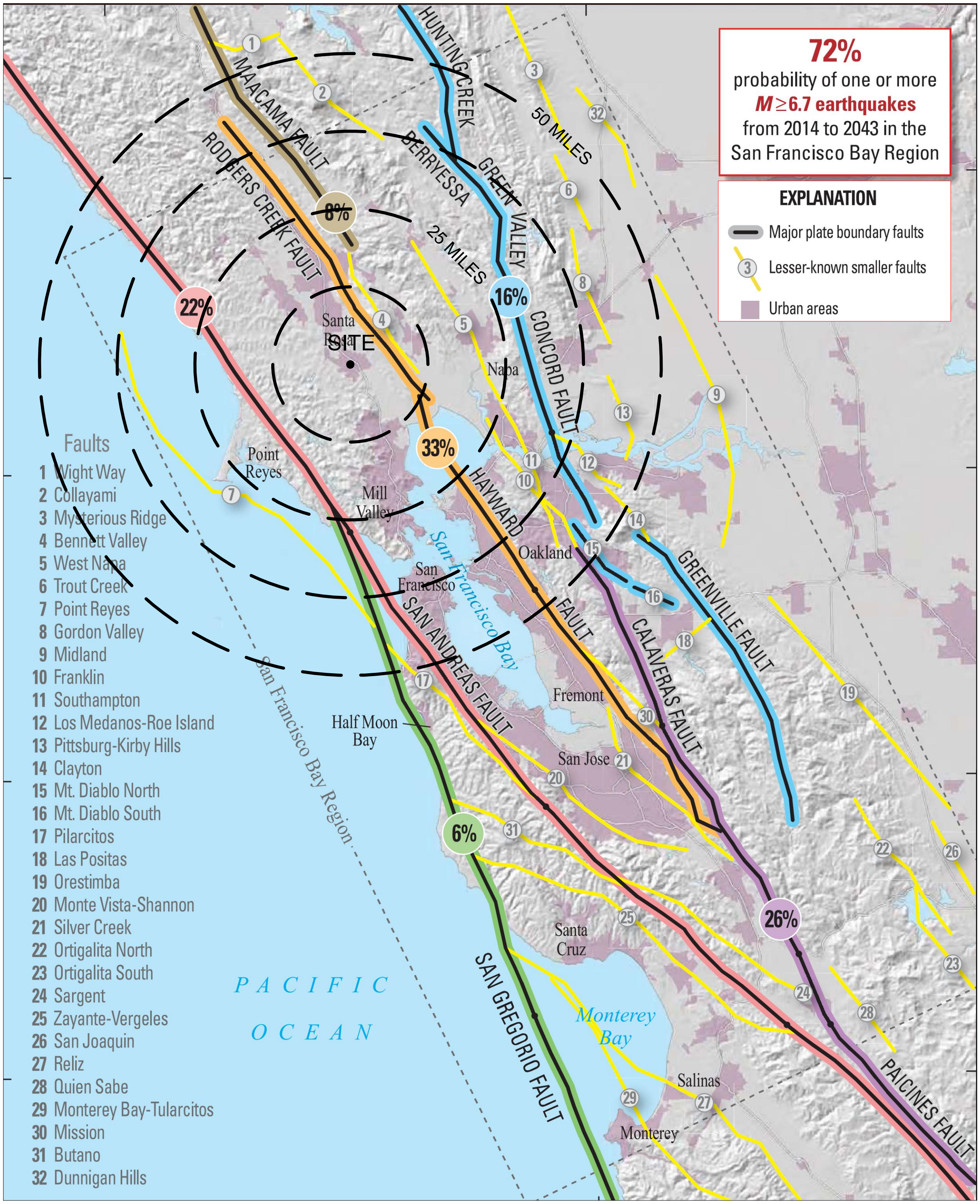
REGIONAL GEOLOGIC MAP

Riverbend Development
529 Madison Street
Petaluma, California

Project No. 2066.001 Date: 9/15/2021

Drawn MMT
Checked

3
FIGURE



SITE COORDINATES
LAT. 38.2417°
LON. -122.6383°

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).

MPEG MILLER PACIFIC ENGINEERING GROUP

FILENAME: 2066.001 Figures2021.dwg

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

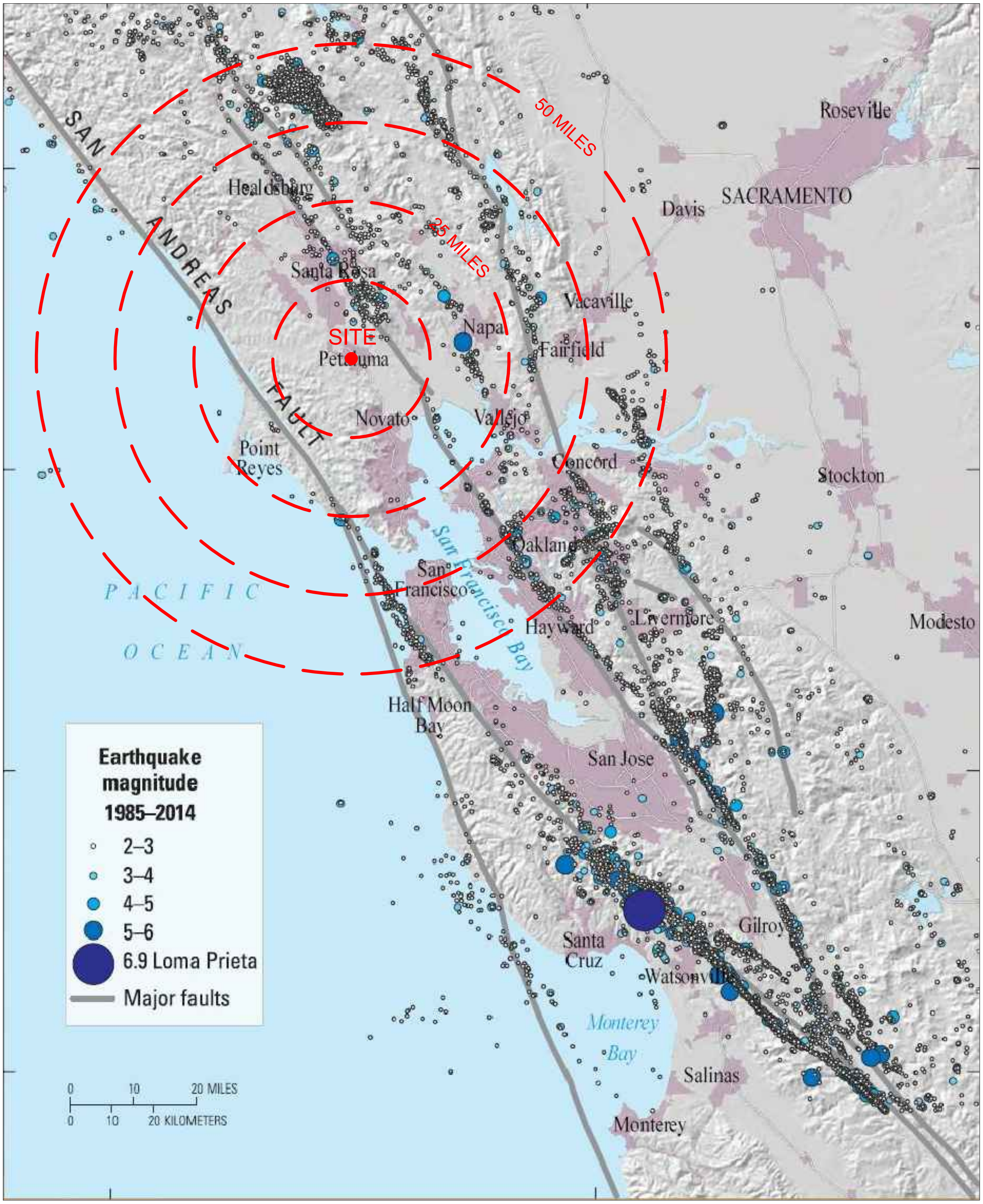
ACTIVE FAULT MAP

Riverbend Development
529 Madison Street
Petaluma, California

Project No. 2066.001 Date: 9/15/2021

Drawn MMT
Checked

4
FIGURE



SITE COORDINATES
LAT. 38.2417°
LON. -122.6383°

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

FILENAME: 2066.001 Figures2021.dwg

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

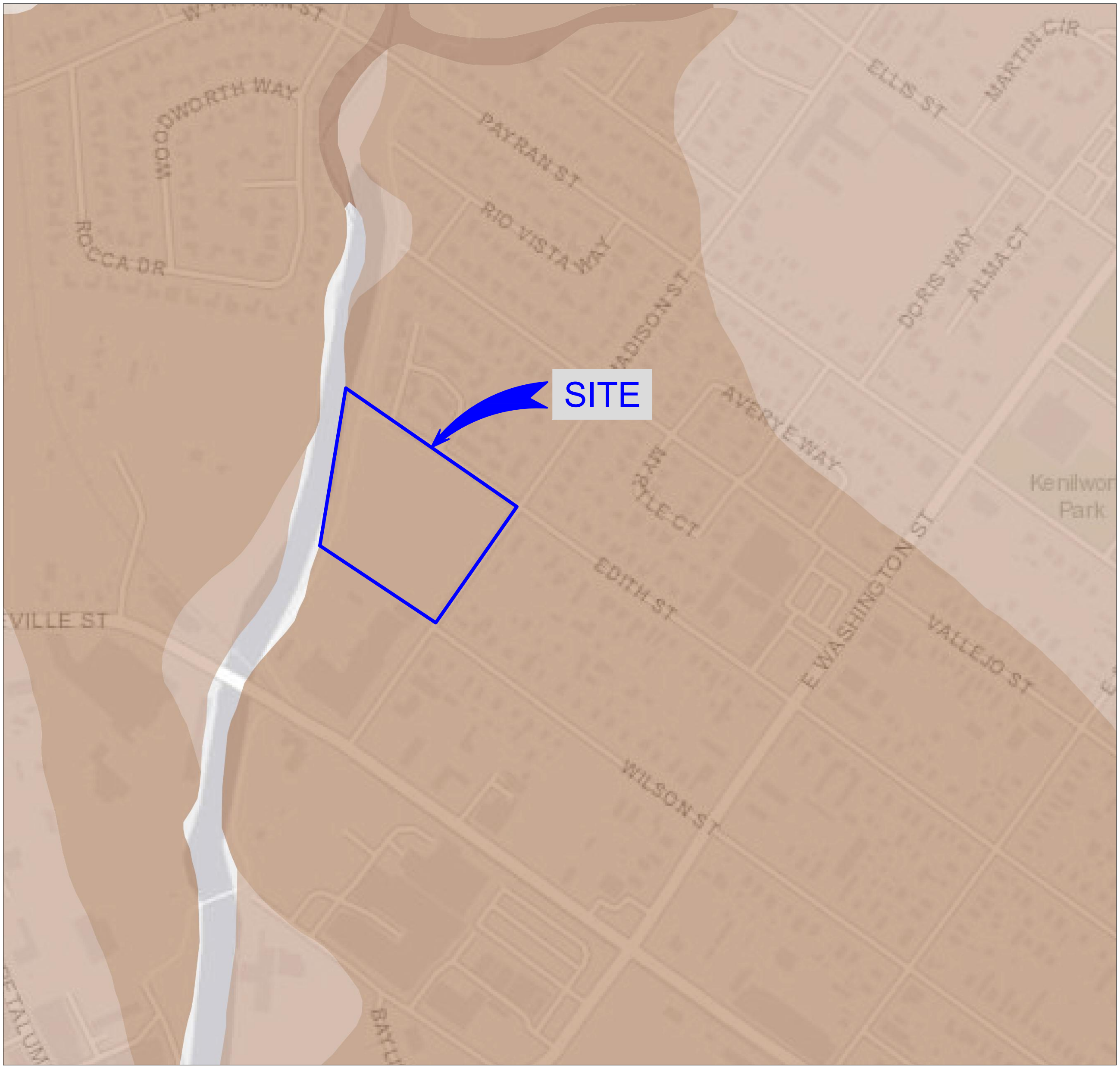
HISTORIC EARTHQUAKE ACTIVITY

Riverbend Development
529 Madison Street
Petaluma, California

Project No. 2066.001 Date: 9/15/2021

Drawn MMT
Checked

5
FIGURE



Earthquake Liquefaction Susceptibility	
Very low	High
Moderate	Very high

REFERENCE: ABAG, 2021

MILLER PACIFIC ENGINEERING GROUP

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

FILENAME: 2066.001 Figures2021.dwg

LIQUEFACTION SUSCEPTIBILITY MAP

Riverbend Development
529 Madison Street
Petaluma, California

Project No. 2066.001 Date: 9/15/2021

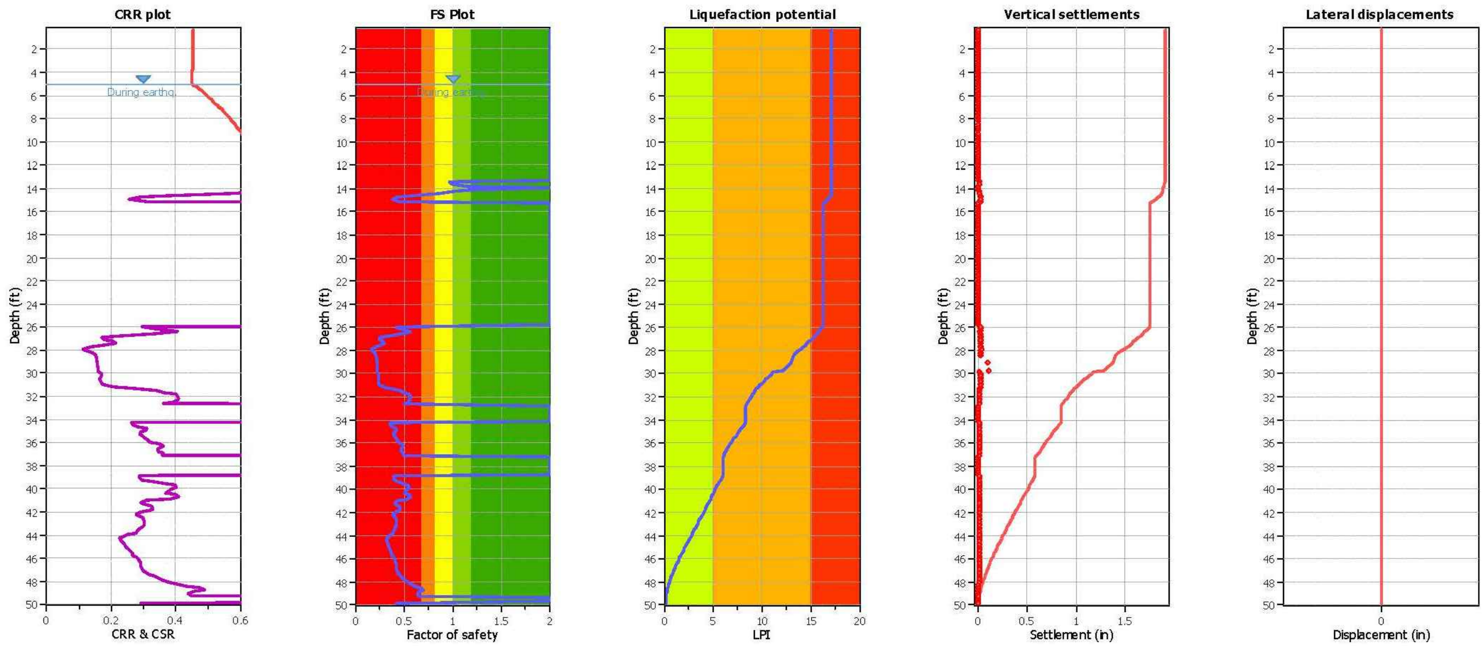
Drawn MMT
Checked

6
FIGURE

This software is licensed to: Miller Pacific Engineering Group

CPT name: CPT-02

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	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:	0.75	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: 9/15/2021, 11:35:32 AM
Project file: H:\Jobs\2000-2099\2066.001 River Bend Bridge\Analysis\2066.001 2021 CPT Liquefaction.clq

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

FILENAME: 2066.001 Figures2021.dwg

CPT-01 2015 LIQUEFACTION ANALYSIS PLOT

Riverbend Development
529 Madison Street
Petaluma, California

Project No. 2066.001 Date: 9/15/2021

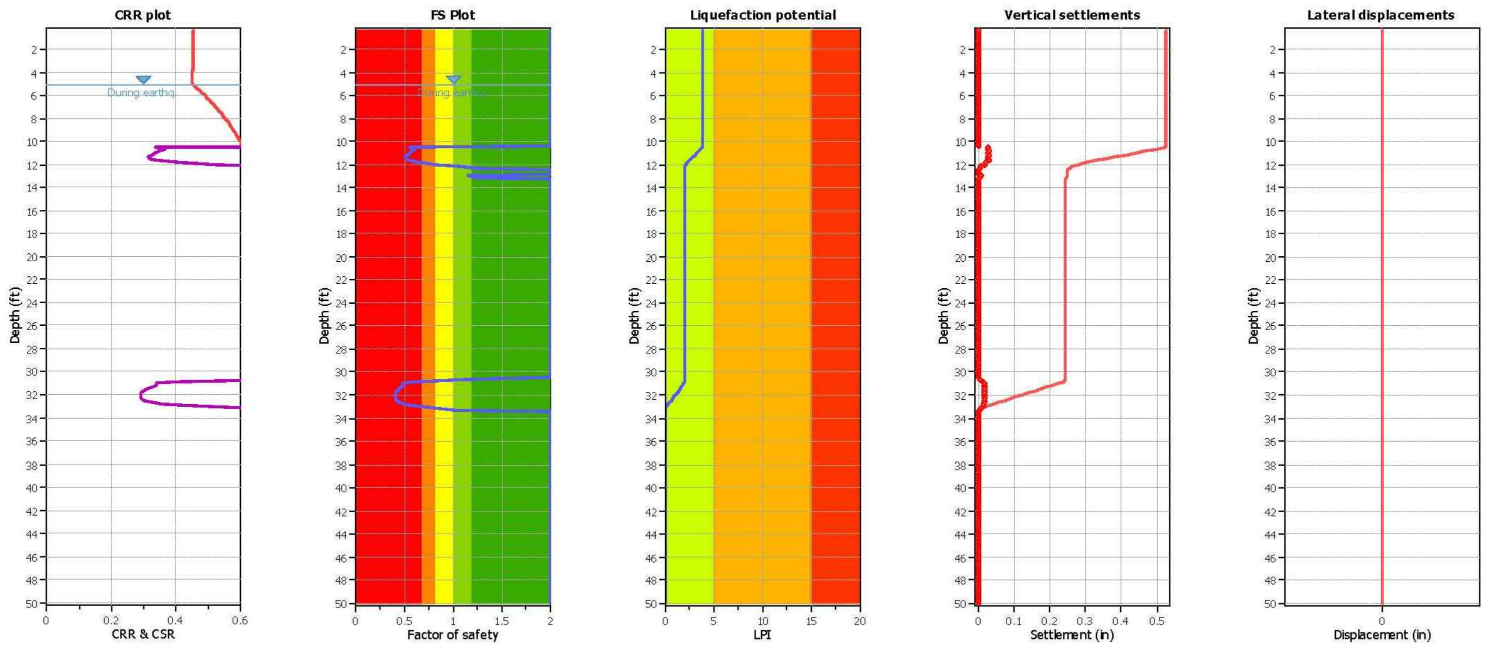
Drawn MMT
Checked

7
FIGURE

This software is licensed to: Miller Pacific Engineering Group

CPT name: CPT-01

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	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:	0.75	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

CLiq v.2.3.1.15 - CPT Liquefaction Assessment Software - Report created on: 9/9/2021, 11:51:16 AM
Project file: H:\Jobs\2000-2099\2066.001 River Bend Bridge\Analysis\2066.001 2021 CPT Liquefaction.clq

5

MILLER PACIFIC ENGINEERING GROUP

FILENAME: 2066.001 Figures2021.dwg

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

CPT-01 2021 LIQUEFACTION ANALYSIS PLOT

Riverbend Development
529 Madison Street
Petaluma, California

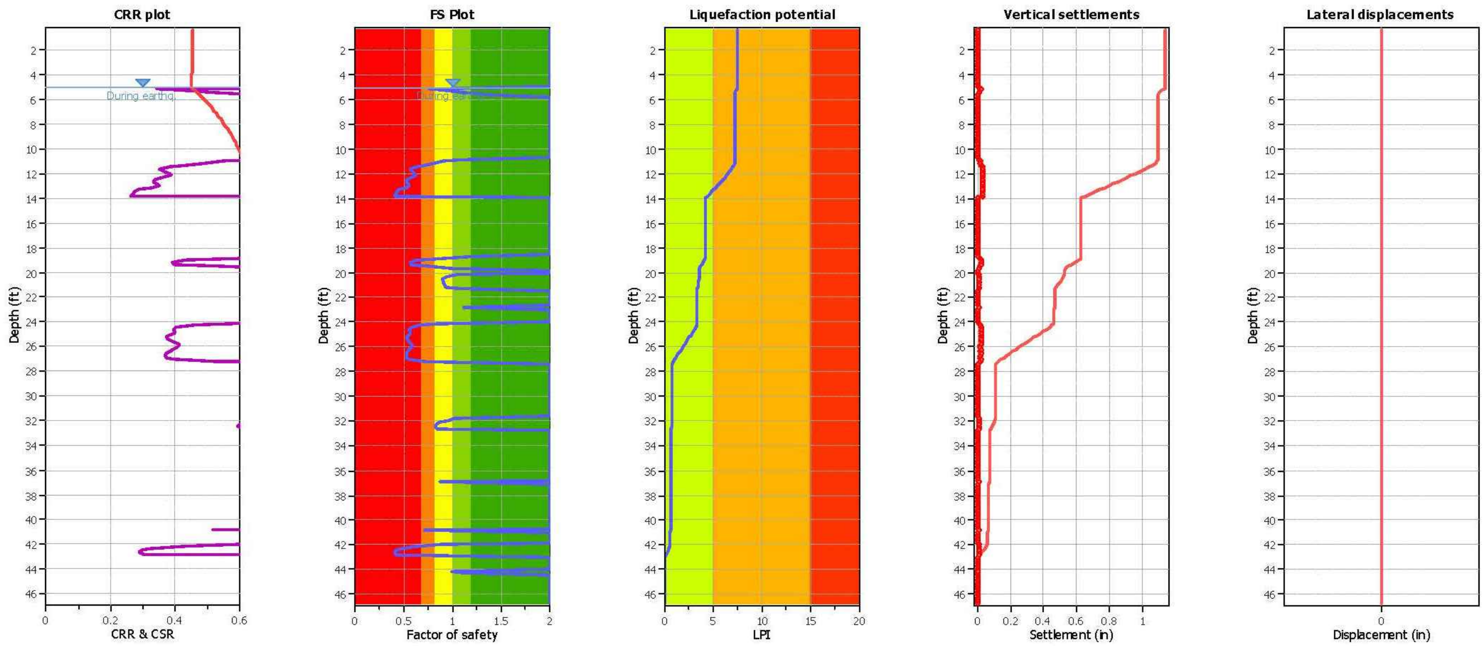
Project No. 2066.001 Date: 9/15/2021

Drawn MMT
Checked

8
FIGURE

CPT name: CPT-02

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	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:	0.75	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

CLiq v.2.3.1.15 - CPT Liquefaction Assessment Software - Report created on: 9/9/2021, 11:51:17 AM
Project file: H:\Jobs\2000-2099\2066.001 River Bend Bridge\Analysis\2066.001 2021 CPT Liquefaction.clq

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

FILENAME: 2066.001 Figures2021.dwg

CPT-02 2021 LIQUEFACTION ANALYSIS PLOT

Riverbend Development
529 Madison Street
Petaluma, California

Project No. 2066.001 Date: 9/15/2021

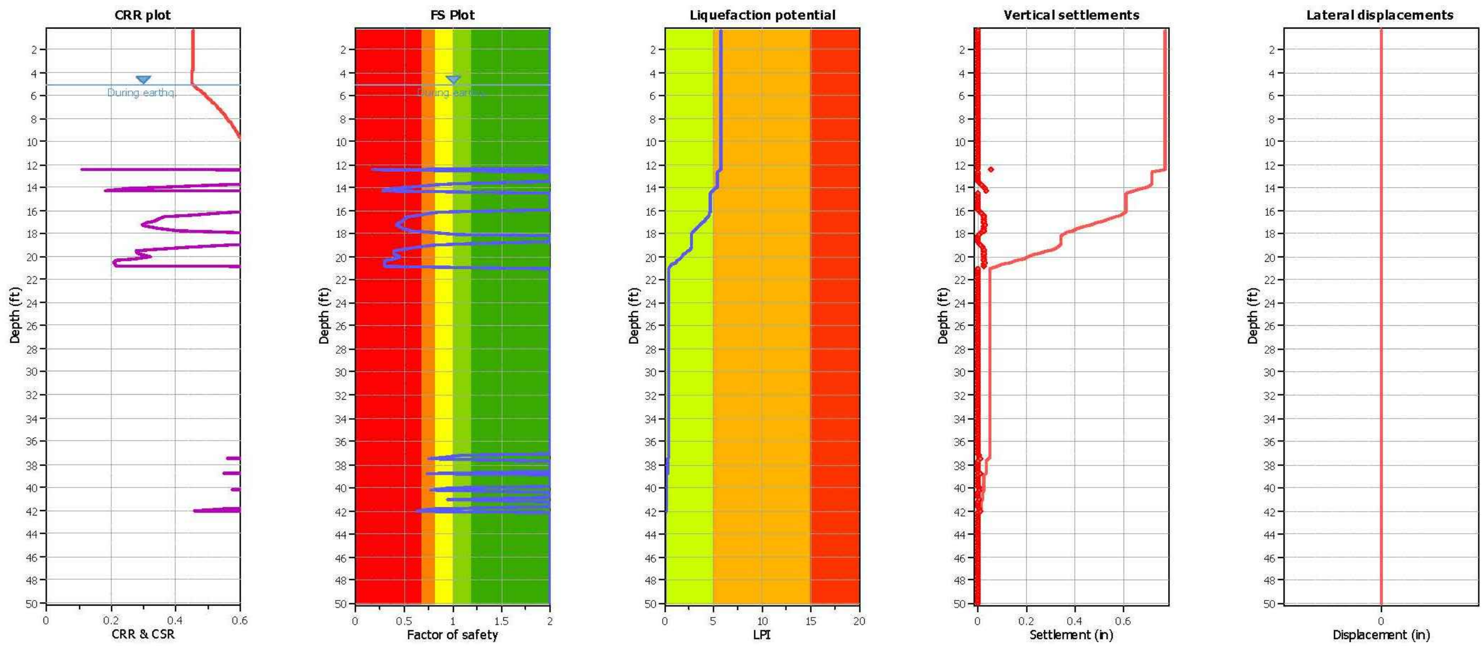
Drawn MMT
Checked

9
FIGURE

This software is licensed to: Miller Pacific Engineering Group

CPT name: CPT-03

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	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:	0.75	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	12.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

CLiq v.2.3.1.15 - CPT Liquefaction Assessment Software - Report created on: 9/9/2021, 11:51:18 AM
Project file: H:\Jobs\2000-2099\2066.001 River Bend Bridge\Analysis\2066.001 2021 CPT Liquefaction.clq

23

MILLER PACIFIC ENGINEERING GROUP

FILENAME: 2066.001 Figures2021.dwg

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

CPT-03 2021 LIQUEFACTION ANALYSIS PLOT

Riverbend Development
529 Madison Street
Petaluma, California

Project No. 2066.001

Date: 9/15/2021

Drawn MMT

Checked

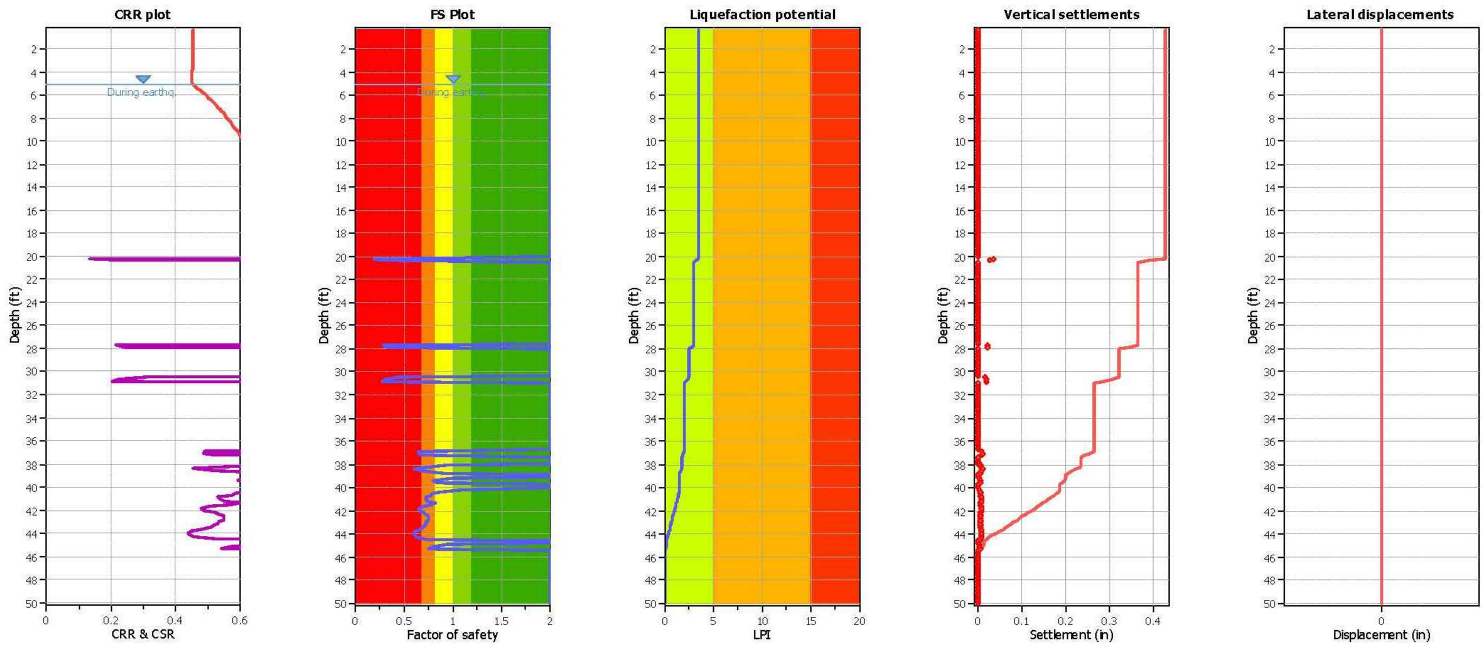
10

FIGURE

This software is licensed to: Miller Pacific Engineering Group

CPT name: CPT-04

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	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:	0.75	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	11.00 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: 9/9/2021, 11:51:19 AM
Project file: H:\Jobs\2000-2099\2066.001 River Bend Bridge\Analysis\2066.001 2021 CPT Liquefaction.clq

32

MILLER PACIFIC ENGINEERING GROUP

FILENAME: 2066.001 Figures2021.dwg

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

CPT-04 2021 LIQUEFACTION ANALYSIS PLOT

Riverbend Development
529 Madison Street
Petaluma, California

Project No. 2066.001 Date: 9/15/2021

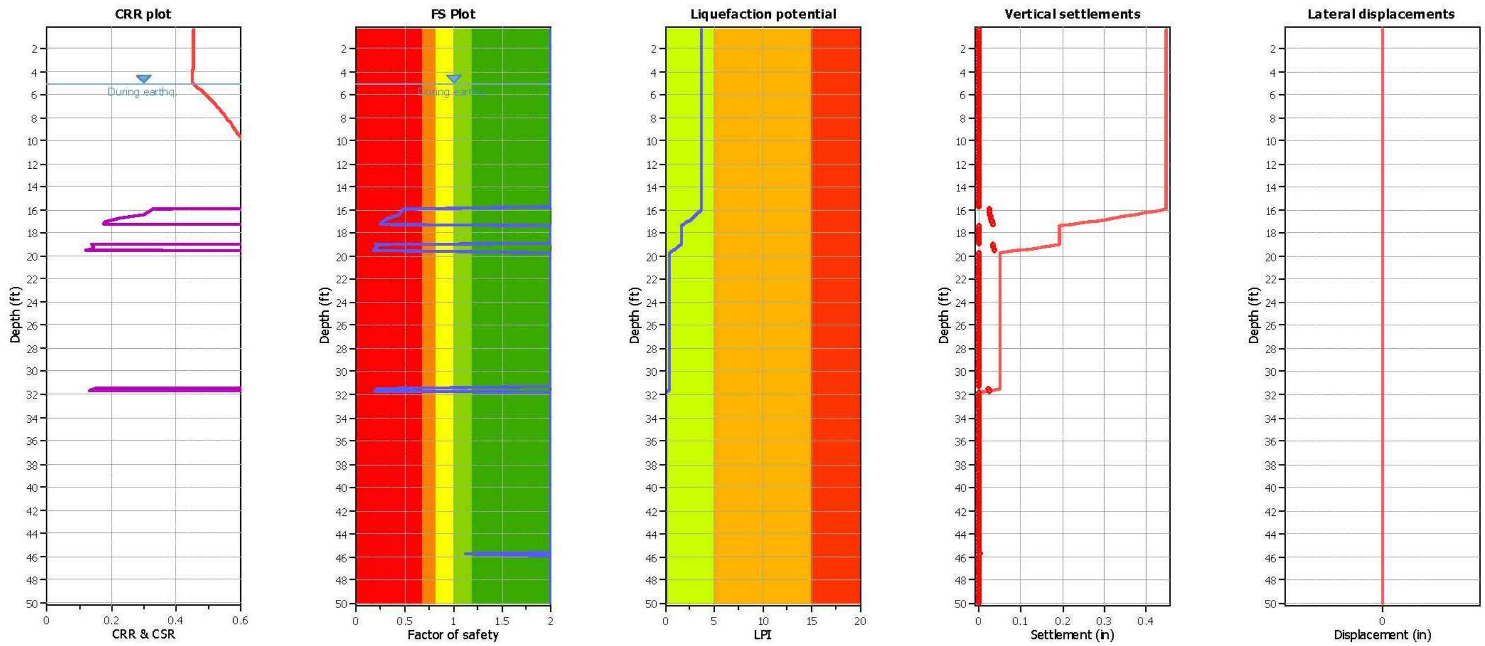
Drawn MMT
Checked

11
FIGURE

This software is licensed to: Miller Pacific Engineering Group

CPT name: CPT-05

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	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:	0.75	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	9.00 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: 9/9/2021, 11:51:20 AM
Project file: H:\Jobs\2000-2099\2066.001 River Bend Bridge\Analysis\2066.001 2021 CPT Liquefaction.clq

41

MILLER PACIFIC ENGINEERING GROUP

FILENAME: 2066.001 Figures2021.dwg

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

CPT-05 2021 LIQUEFACTION ANALYSIS PLOT

Riverbend Development
529 Madison Street
Petaluma, California

Project No. 2066.001 Date: 9/15/2021

Drawn MMT
Checked

12
FIGURE

MILLER PACIFIC
ENGINEERING GROUP

APPENDIX A
REFERENCE SUBSURFACE EXPLORATION

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS		SYMBOL	DESCRIPTION
COARSE GRAINED SOILS over 50% sand and gravel	CLEAN GRAVEL	GW	Well-graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines
	GRAVEL with fines	GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	CLEAN SAND	SW	Well-graded sands or gravely sands, little or no fines
		SP	Poorly-graded sands or gravely sands, little or no fines
	SAND with fines	SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
FINE GRAINED SOILS over 50% silt and clay	SILT AND CLAY liquid limit <50%	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
		CL	Inorganic clays of low to medium plasticity, gravely clays, sandy clays, silty clays, lean clays
		OL	Organic silts and organic silt-clays of low plasticity
	SILT AND CLAY liquid limit >50%	MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic clays of medium to high plasticity
HIGHLY ORGANIC SOILS		PT	Peat, muck, and other highly organic soils
ROCK			Undifferentiated as to type or composition

KEY TO BORING AND TEST PIT SYMBOLS

CLASSIFICATION TESTS

AL	ATTERBERG LIMITS TEST
SA	SIEVE ANALYSIS
HYD	HYDROMETER ANALYSIS
P200	PERCENT PASSING NO. 200 SIEVE
P4	PERCENT PASSING NO. 4 SIEVE

STRENGTH TESTS

TV	FIELD TORVANE (UNDRAINED SHEAR)
UC	LABORATORY UNCONFINED COMPRESSION
TXCU	CONSOLIDATED UNDRAINED TRIAXIAL
TXUU	UNCONSOLIDATED UNDRAINED TRIAXIAL
	UC, CU, UU = 1/2 Deviator Stress

SAMPLER TYPE

UNDISTURBED CORE SAMPLE: MODIFIED CALIFORNIA OR HYDRAULIC PISTON SAMPLE

STANDARD PENETRATION TEST SAMPLE

X DISTURBED OR BULK SAMPLE

ROCK OR CORE SAMPLE

NOTE: Test boring and test pit logs are an interpretation of conditions encountered at the location and time of exploration. Subsurface rock, soil and water conditions may differ in locations and with the passage of time. Lines defining interface between differing soil or rock description are approximate and may indicate a gradual transition.

FILE: SoilClass.dwg

Miller Pacific
ENGINEERING GROUP

SOIL CLASSIFICATION CHART
Clover Stornetta Subdivision
Petaluma, California

A-1

Project No. 872.02 | Date 02/16/06 | Approved By:

Figure

FRACTURING AND BEDDING

Fracture Classification	Spacing	Bedding Classification
Crushed	less than 3/4 inch	Laminated
Intensely fractured	3/4 to 2-1/2 inches	Very thinly bedded
Closely fractured	2-1/2 to 8 inches	Thinly bedded
Moderately fractured	8 to 24 inches	Medium bedded
Widely fractured	2 to 6 feet	Thickly bedded
Very widely fractured	greater than 6 feet	Very thickly bedded

HARDNESS

Low	Carved or gouged with a knife
Moderate	Easily scratched with a knife, friable
Hard	Difficult to scratch, knife scratch leaves dust trace
Very hard	Rock scratches metal

STRENGTH

Friable	Crumbles by rubbing with fingers
Weak	Crumbles under light hammer blows
Moderate	Indentations <1/8 inch with moderate blow with pick end of rock hammer
Strong	Withstands few heavy hammer blows, yields large fragments
Very strong	Withstands many heavy hammer blows, yields dust, small fragments

WEATHERING

Complete	Minerals decomposed to soil, but fabric and structure preserved
High	Rock decomposition, thorough discoloration, all fractures are extensively coated with clay, oxides or carbonates
Moderate	Fracture surfaces coated with weathering minerals, moderate or localized discoloration
Slight	A few stained fractures, slight discoloration, no mineral decomposition, no affect on cementation
Fresh	Rock unaffected by weathering, no change with depth, rings under hammer impact

NOTE: Test boring and test pit logs are an interpretation of conditions encountered at the location and time of exploration. Subsurface rock, soil and water conditions may differ in other locations and with the passage of time.

FILE: RockClass.dwg

Miller Pacific
ENGINEERING GROUP

ROCK CLASSIFICATION CHART
Clover Stornetta Subdivision
Petaluma, California

A-2

Project No. 872.02 Date 02/16/06 Approved By:

Figure

BORING 1

EQUIPMENT: Truck Mounted B-53

DATE: 10/23/01
ELEVATION: +6 feet
*REFERENCE: USGS Quad, Petaluma, CA, 1981

OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DESCRIPTION
					SILTY CLAY (CH) (ALLUVIUM) moist, stiff, high plasticity, dark brown
		40	19.1	104	
		53			CLAYEY SAND (SC) moist, medium dense to dense, tan
					SILTY SAND (SM) moist, medium dense to dense, mottled brown
		48			groundwater encountered at 9 feet
		31			GRAVELLY SAND (GM) wet, medium dense, mottled brown
		26			SILTY SAND (SM) wet, medium dense, mottled tan
					GRAVELLY SAND (GM) wet, medium dense, mottled brown
		50			SILTY SAND (SM) wet, mottled tan
					GRAVELLY SAND (GM) wet, medium dense, mottled brown
	1250 (UC)	28	20.6	107	SANDY CLAY (CL) moist, medium stiff, low to medium plasticity, dark gray
		30			Bottom of boring at 21 feet

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

FILE: 872.01.dwg

Miller Pacific
ENGINEERING GROUP

BORING LOG
Clover-Stornetta Dairy
Petaluma, California

Figure A-3

Project No. 872.01 | Date 11/14/01 | Approved By:

BORING 2

EQUIPMENT: Truck Mounted B-53

DATE: 10/23/01

ELEVATION: +6 feet

*REFERENCE: USGS Quad, Petaluma, CA, 1981

% PASSING #200	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DESCRIPTION
					GRAVELLY SAND (GP) (FILL) moist, loose, mottled gray to brown
		14	8.8		
7		9			
					SILTY CLAY (CH) (ALLUVIUM) moist, stiff, high plasticity, dark brown to black
	2500 (UC)	36	22.3	100	
		23			
					CLAYEY SAND (SC) moist, medium dense becoming loose, mottled tan to brown; groundwater encountered at 11 feet
		16			
32		11			
					Bottom of boring at 16.5 feet

DEPTH scale: meters 0–6; feet 0, 5, 10, 15, 20

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

FILE: 872.01.dwg

Miller Pacific ENGINEERING GROUP

BORING LOG
Clover-Stornetta Dairy
Petaluma, California

A-4

Project No. 872.01 | Date 11/14/01 | Approved By: | Figure

BORING 3

EQUIPMENT: Truck Mounted B-53

DATE: 10/23/01

ELEVATION: +6 feet

*REFERENCE: USGS Quad, Petaluma, CA, 1981

OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DEPTH meters	DEPTH feet	DESCRIPTION
					0	0	SILTY SAND (SM) (FILL) slightly moist, loose, mottled brown
		14	10.6	91			
					1		
		13					SILTY CLAY (CH) (ALLUVIUM) moist, medium stiff, high plasticity, brown
						5	
					2		
	1500 (UC)	31	26.5	97			
		25					same material except medium stiff to stiff, dark brown to black
					3	10	
							▽ groundwater encounterd at 11 feet
		32			4		CLAYEY SAND (SC) wet, medium dense becoming loose, mottled tan to brown
						15	
8		12			5		
							Bottom of boring at 16.5 feet
					6	20	

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

FILE: 872.01.dwg

Miller Pacific
ENGINEERING GROUP

BORING LOG
Clover-Stornetta Dairy
Petaluma, California

A-5

Project No. 872.01 | Date 11/14/01 | Approved By: | Figure

BORING 4

EQUIPMENT: Truck Mounted B-53

DATE: 10/23/01

ELEVATION: +6 feet

*REFERENCE: USGS Quad, Petaluma, CA, 1981

OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DEPTH meters / feet	SAMPLE	SYMBOL (3)	
					0 / 0			CLAYEY SAND (SC) (FILL) moist, loose, mottled brown
		12	12.5	89				
					1			
		22						SILTY CLAY (CH) (ALLUVIUM) moist, medium stiff, high plasticity, dark brown
					5			
					2			
	2000 (UC)	33	23.7	101				
		23			3 / 10			groundwater encountered at 10 feet CLAYEY SAND (SC) wet, medium dense becoming loose, grayish tan
23		14			4			
					15			GRAVELLY SAND (GC) wet,medium dense, mottled brown
		35			5			SILTY SAND (SM) wet, medium dense, mottled tan
								Bottom of boring at 16.5 feet
					6 / 20			

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

FILE: 872.01.dwg

Miller Pacific
ENGINEERING GROUP

BORING LOG
Clover-Stornetta Dairy
Petaluma, California

A-6

Project No. 872.01 Date 11/14/01 Approved By:

Figure

BORING 5

EQUIPMENT: Truck Mounted B-53

DATE: 10/23/01

ELEVATION: +6 feet

*REFERENCE: USGS Quad, Petaluma, CA, 1981

OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)
	4000 (UC)	24	21.1	97
		13		
	2500 (UC)	40	22.9	101
		27		
		16		

DEPTH: meters 0–6; feet 0, 5, 10, 15, 20

SAMPLE | SYMBOL (3)

GRAVELLY SAND (SP) (FILL)
dry, loose, gray

SANDY SILTY CLAY (CH) (ALLUVIUM)
moist, medium stiff to stiff, high plasticity, brown

SANDY CLAY (CL)
moist, stiff, medium plasticity, grayish-brown

∇ groundwater encountered at 10 feet

SILTY SAND (SM)
wet, medium dense, mottled tan to brown

CLAYEY SAND (SC)
wet, soft to medium dense, grayish tan

SANDY CLAY (CL)
moist, soft to medium stiff, medium plasticity, mottled tan to brown

CLAYEY SAND (SC)
wet, soft to medium dense, grayish tan

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

FILE: 872.01.dwg

BORING 6

EQUIPMENT: Tracked Rig with 6 in. Solid Flight Auger
DATE: 2/9/06
ELEVATION: 000-Feet*
*REFERENCE: Topo Map used for Elevation

OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DEPTH meters / feet	SAMPLE	SYMBOL (3)	DESCRIPTION
		32	21.7		0 / 0			SANDY SILTY CLAY (CL/CH) brown and gray mottled, moist, medium stiff to stiff,high plasticity clay, ~25% well graded sand (Alluvium)
	UC 970	25	29.8	91	1 / 5			SILTY CLAY (CL/CH) olive brown, moist, medium stiff-stiff, medium high plasticity (Alluvium)
		37	21.4	102	3 / 10			SANDY CLAY (CL/CH) tan, moist, medium dense (Alluvium)
	UC 200	54	19.3	106	4			WELL GRADED SAND WITH SILT (SM) tan, wet, dense (Alluvium)
					15 / 5 / 6 / 20			Bottom of Boring at 14 ft. Groundwater was not encountered.

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

FILE: xxx.dwg

Miller Pacific
ENGINEERING GROUP

BORING LOG
Clover Stornetta Subdivision
Petaluma, California

A-8

Project No. 872.02 | Date 02/16/06 | Approved By:

Figure

BORING 7

EQUIPMENT: Tracked Rig with 6 in. Solid Flight Auger
DATE: 2/9/06
ELEVATION: 000-Feet*
*REFERENCE: Topo Map used for Elevation

OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)
	UC 1150	42	30.4	90
	UC 2000	20/3"	21.5	100
P200 31.8	UC 660	36	23.7	101
	UC 175	50	23.5	99

DEPTH meters feet — SAMPLE — SYMBOL (3)

SILTY CLAY (CL/CH)
dark brown, moist, stiff, high plasticity clay (Alluvium)

SANDY CLAY (CL/CH)
tan, moist, medium dense (Alluvium)

SILTY SAND (SM)
tan, moist to wet, medium dense, fine sand (Alluvium)

CLAYEY/SILTY SAND (SC/SM)
grayish brown, wet, dense, fine sand, ~40% fines (Alluvium)

Bottom of Boring at 15 ft.
Groundwater at 12 ft. when auger removed.

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

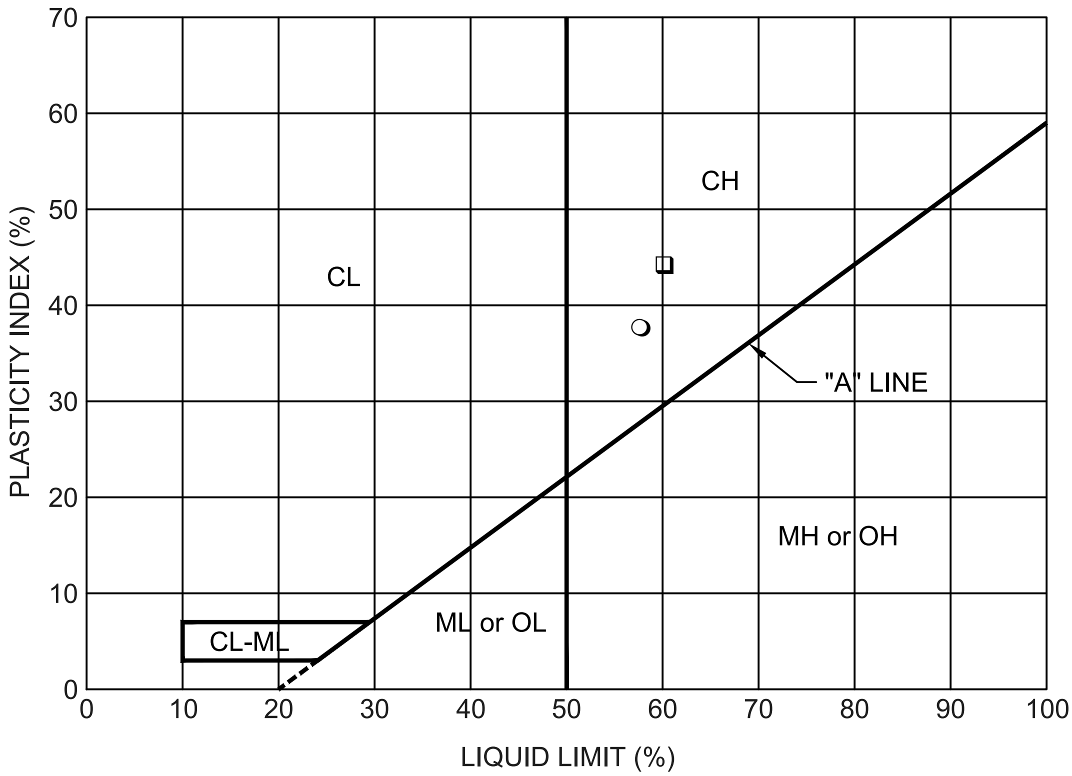
FILE: xxx.dwg

Miller Pacific
ENGINEERING GROUP

BORING LOG
Clover Stornetta Subdivision
Petaluma, California

A-9

Project No. 872.02 | Date 02/06/06 | Approved By: | Figure



SYMBOL	SAMPLE SOURCE	CLASSIFICATION	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTICITY INDEX (%)
❑	Boring 1 2.5 feet	SiltyClay (CH) dark brown	60	17	43
❍	Boring 4 3.5-5 feet	Silty Clay (CH) dark brown	58	21	37

REFERENCE: Liquid Limit, Plastic Limit, and Plasticity Index of Soils, ASTM D 4318

FILE: Plasticity Index.dwg

Miller Pacific
ENGINEERING GROUP

PLASTICITY CHART
Madison/Edith Subdivision
Petaluma, California

A-10

Project No. 872.02 | Date 03/03/06 | Approved By:

Figure

APPENDIX B
SUBSURFACE EXPLORATION

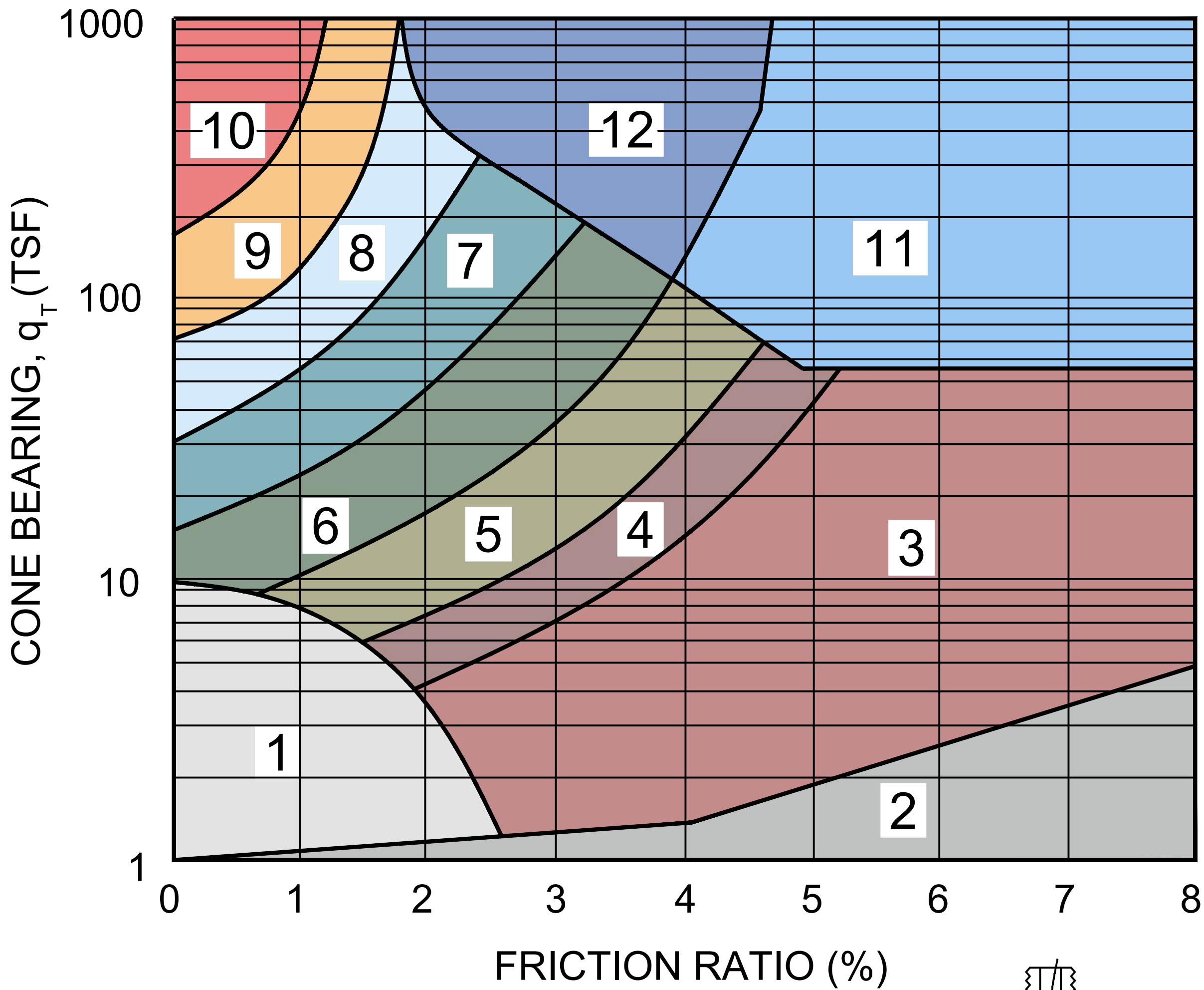
1.0 Cone Penetration Testing

We performed one Cone Penetration Test (CPT) on May 7, 2015, and five additional CPTs on August 25, 2021, at the approximate 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 B-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.

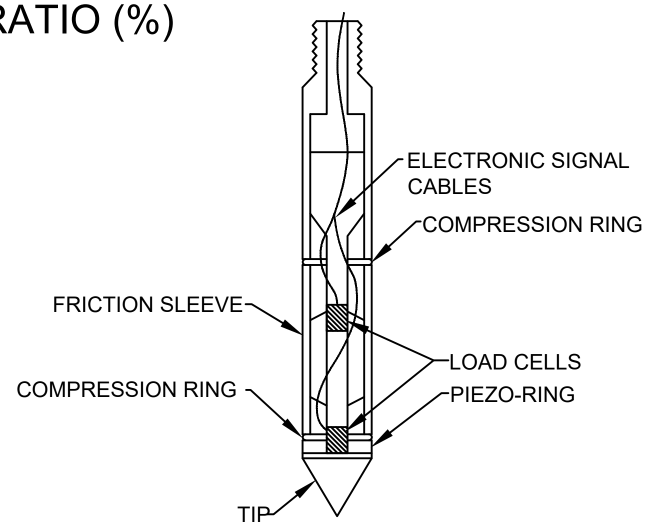
The recorded data is transferred to an in-office computer for reduction and analysis. The analysis of cone bearing and sleeve friction (i.e., friction ratio) indicates the soil type, the cone bearing alone indicates soil density or strength, and the pore pressure indicates the presence of clay. Variations 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 B-1, and the CPT data logs are presented on Figures B-2 through B-7.

The exploratory CPT logs, description of soils encountered, and the laboratory test data reflect conditions only at the locations of the borings at the time they were excavated or retrieved. 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

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

FILENAME: 2066.001 CPTs.dwg

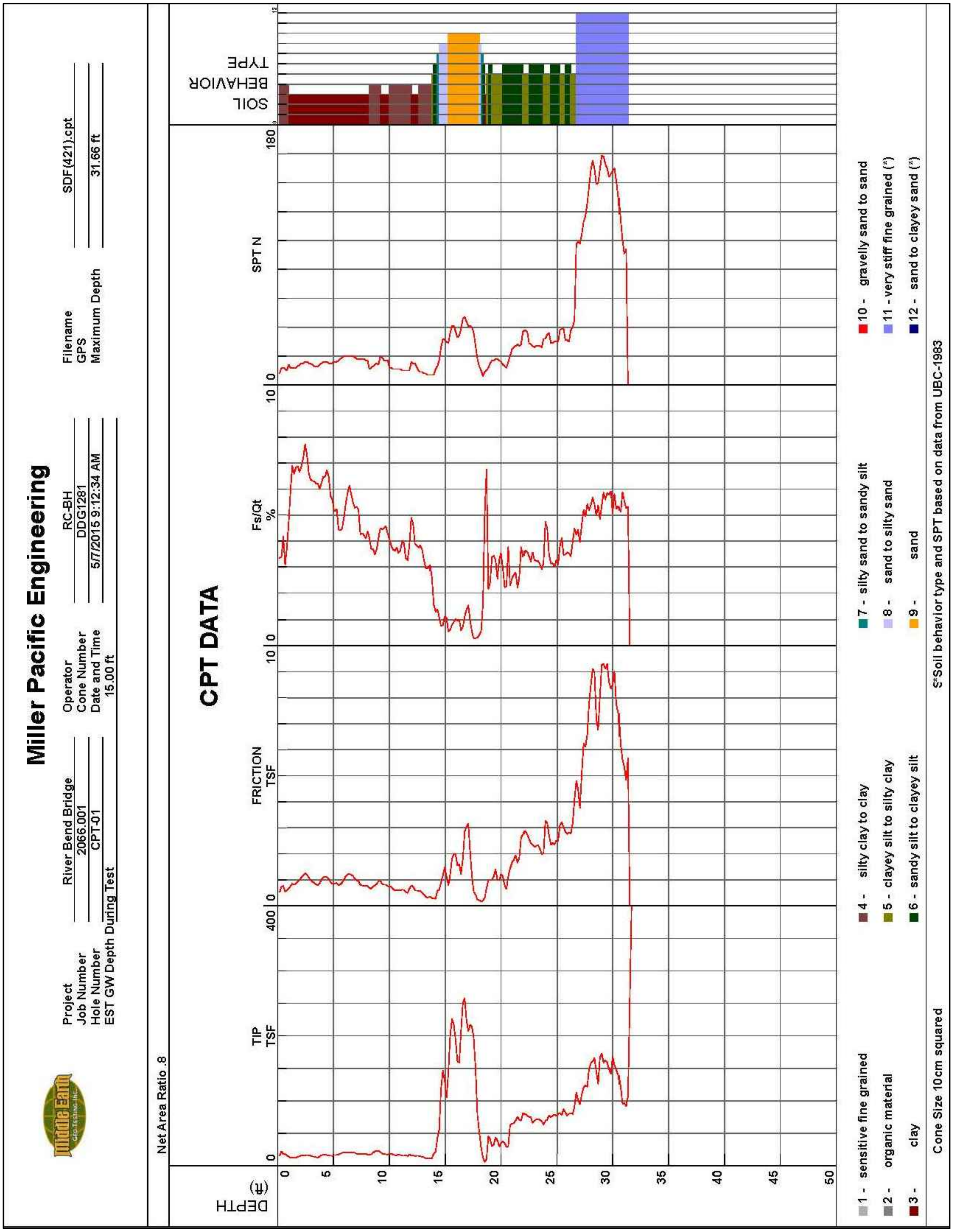
CPT SOIL INTERPRETATION CHART

Riverbend Development
529 Madison Street
Petaluma, California

Project No. 2066.001 Date: 9/15/2021

Drawn MMT
Checked

B-1
FIGURE



MPEG MILLER PACIFIC ENGINEERING GROUP

FILENAME: 2066.001 CPTs.dwg

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

CPT-01 2015 DATA

Riverbend Development
529 Madison Street
Petaluma, California

Project No. 2066.001 Date: 9/15/2021

Drawn MMT
Checked

B-2
FIGURE

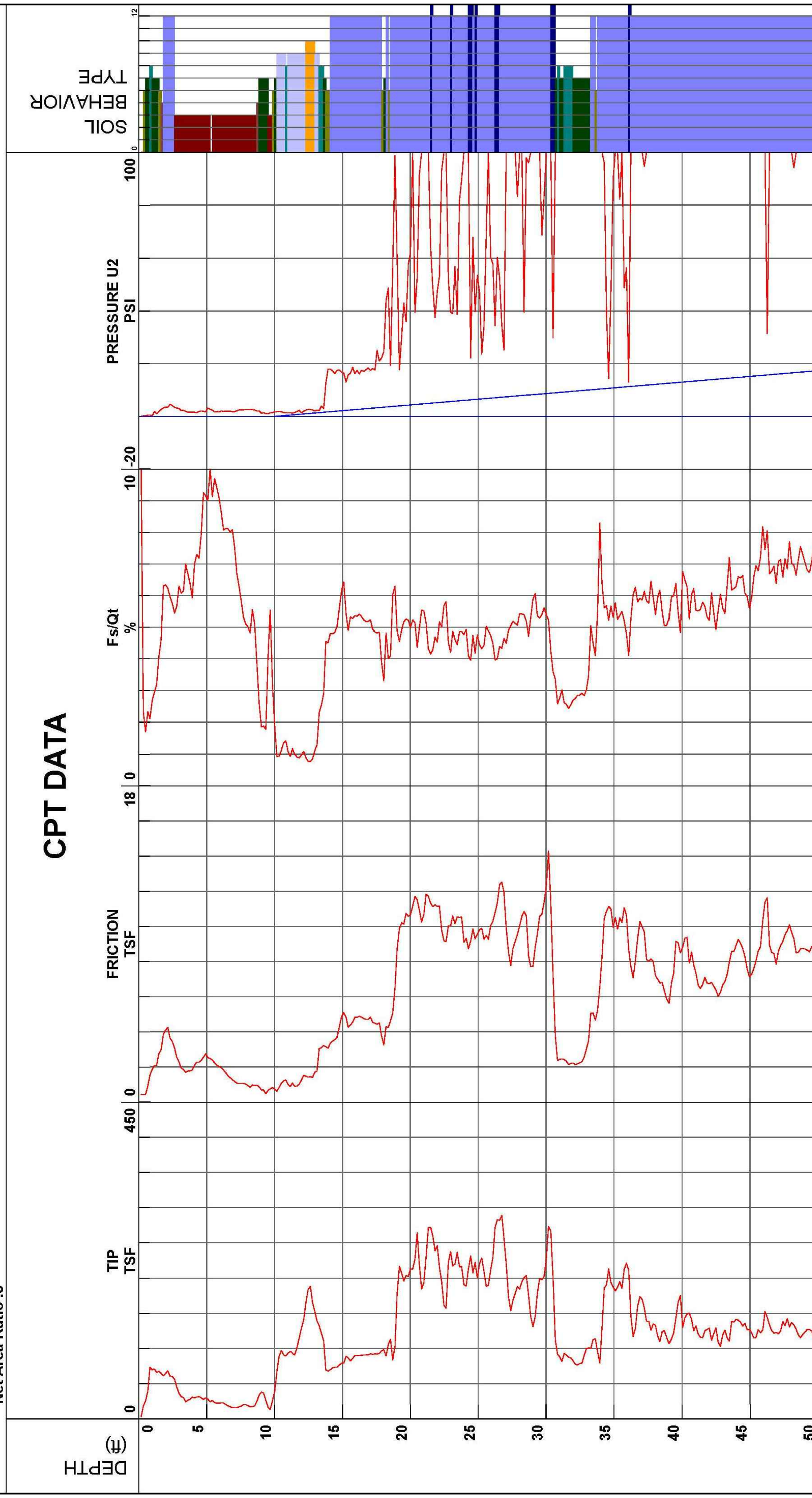
Miller Pacific Engineering

Project	Riverbend Subdivision	Operator	AJ-OO
Job Number	2066.001	Cone Number	DDG1596
Hole Number	CPT-01	Date and Time	8/25/2021 1:35:41 PM
EST GW Depth During Test	10.00 ft		

Filename	SDF(879).cpt
GPS	
Maximum Depth	50.52 ft

Net Area Ratio .8

CPT DATA



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

Cone Size 15cm squared

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

MPEG MILLER PACIFIC ENGINEERING GROUP

FILENAME: 2066.001 CPTs.dwg

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

CPT-01 2021 DATA

Riverbend Development
529 Madison Street
Petaluma, California

Project No. 2066.001 Date: 9/15/2021

Drawn MMT
Checked

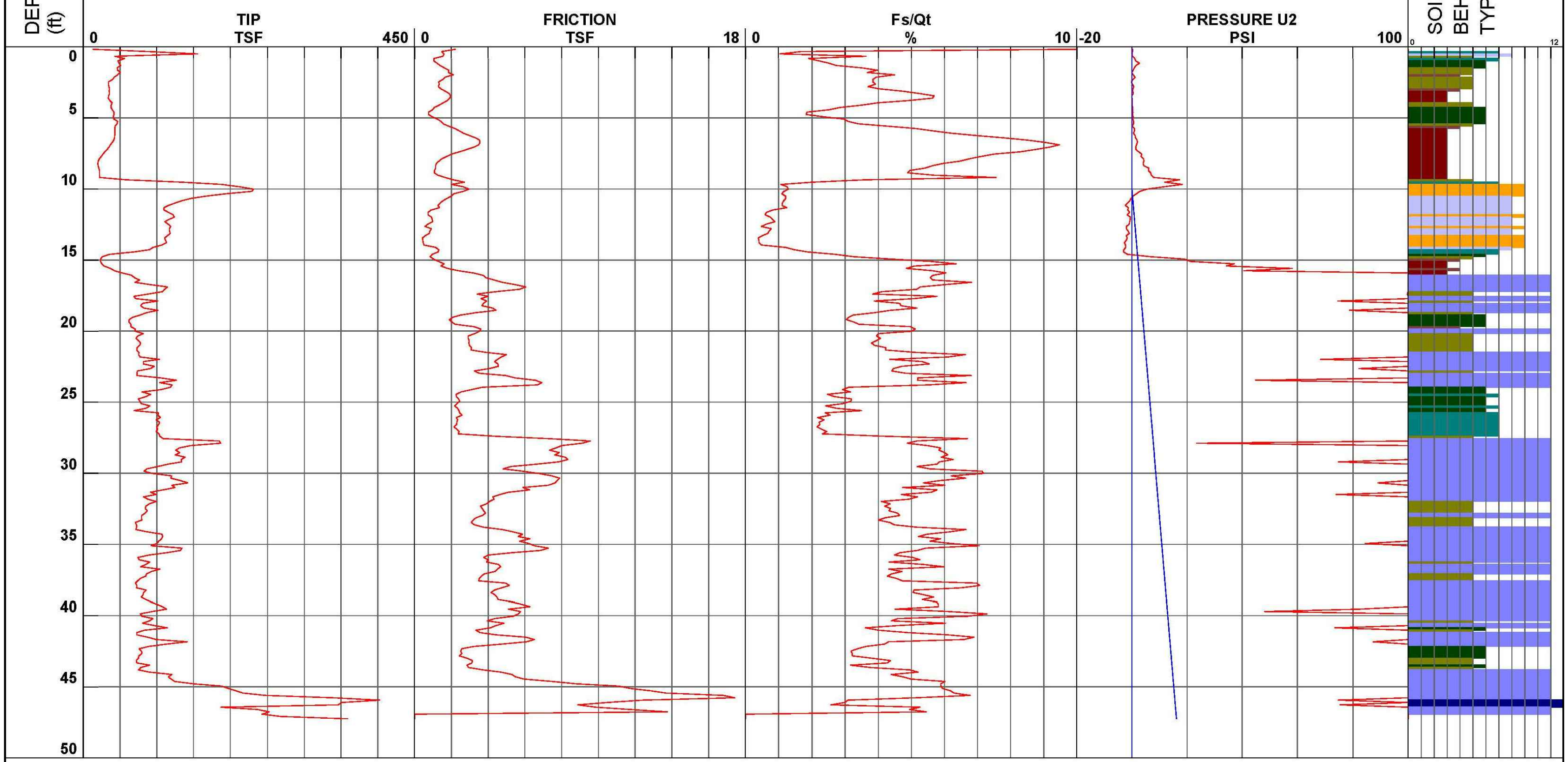
B-3
FIGURE

Miller Pacific Engineering

Project	Riverbend Subdivision	Operator	AJ-OO	Filename	SDF(878).cpt
Job Number	2066.001	Cone Number	DDG1596	GPS	
Hole Number	CPT-02	Date and Time	8/25/2021 12:45:06 PM	Maximum Depth	47.24 ft
EST GW Depth During Test	10.00 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

MPEG MILLER PACIFIC ENGINEERING GROUP

FILENAME: 2066.001 CPTs.dwg

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

CPT-02 2021 DATA

Riverbend Development
529 Madison Street
Petaluma, California

Project No. 2066.001 Date: 9/15/2021

Drawn MMT
Checked

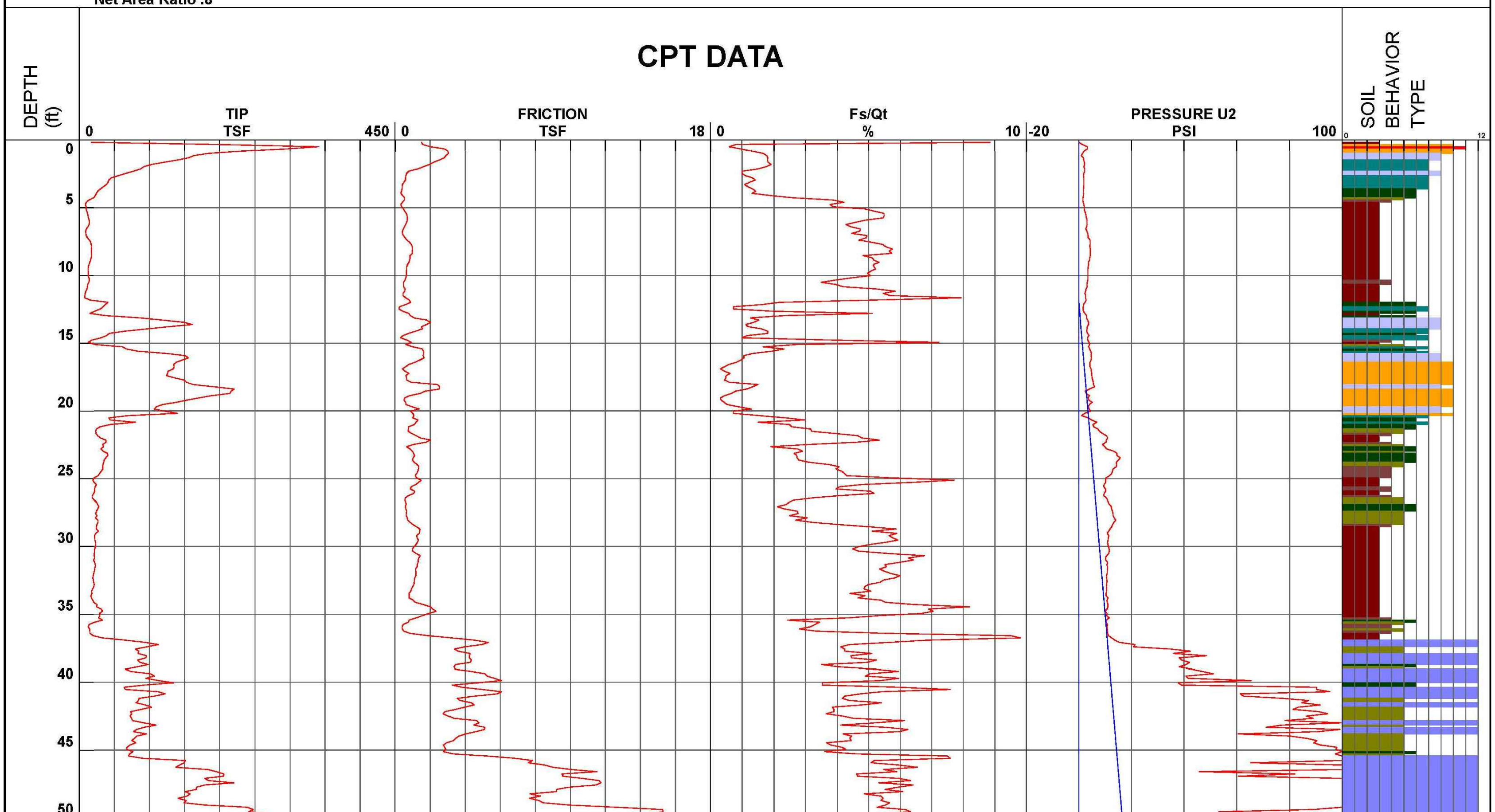
B-4
FIGURE

Miller Pacific Engineering

Project	Riverbend Subdivision	Operator	AJ-OO	Filename	SDF(880).cpt
Job Number	2066.001	Cone Number	DDG1596	GPS	
Hole Number	CPT-03	Date and Time	8/25/2021 2:17:20 PM	Maximum Depth	50.52 ft
EST GW Depth During Test	12.00 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

MPEG MILLER PACIFIC ENGINEERING GROUP

FILENAME: 2066.001 CPTs.dwg

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

CPT-03 2021 DATA

Riverbend Development
529 Madison Street
Petaluma, California

Project No. 2066.001 Date: 9/15/2021

Drawn MMT
Checked

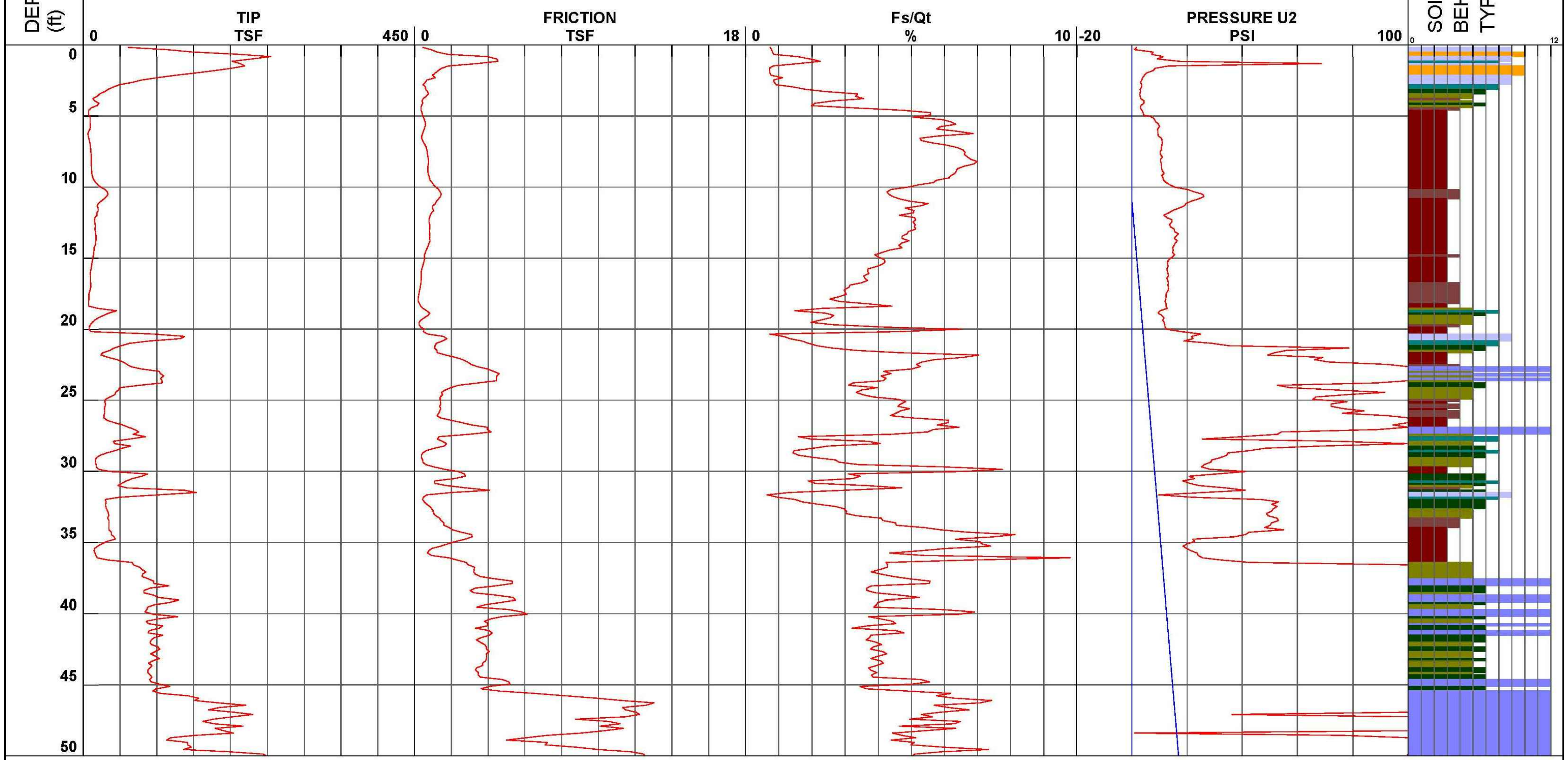
B-5
FIGURE

Miller Pacific Engineering

Project	Riverbend Subdivision	Operator	AJ-OO	Filename	SDF(877).cpt
Job Number	2066.001	Cone Number	DDG1596	GPS	
Hole Number	CPT-04	Date and Time	8/25/2021 11:08:46 AM	Maximum Depth	50.52 ft
EST GW Depth During Test		11.00 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 15cm squared

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

MPEG MILLER PACIFIC ENGINEERING GROUP

FILENAME: 2066.001 CPTs.dwg

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

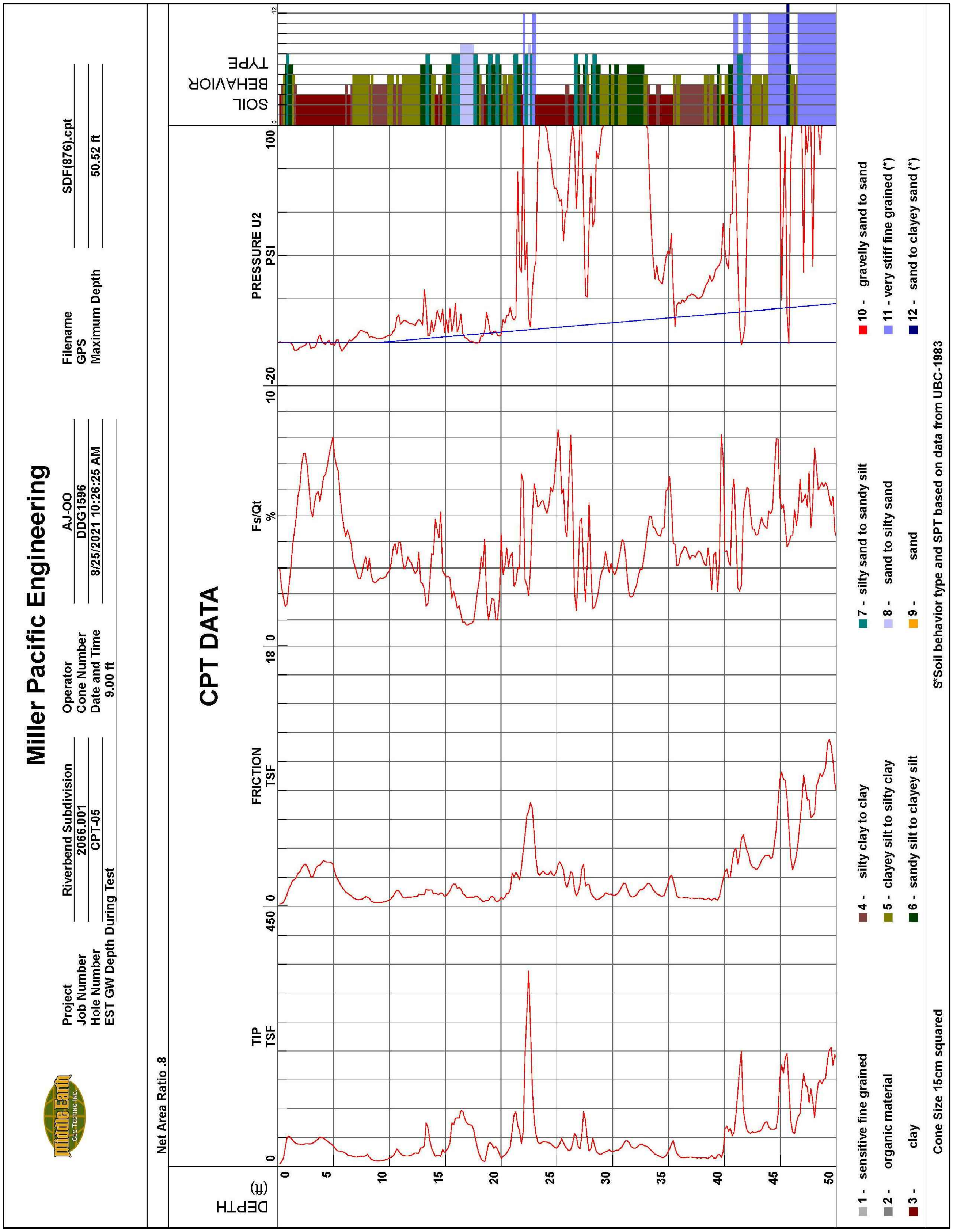
CPT-04 2021 DATA

Riverbend Development
529 Madison Street
Petaluma, California

Project No. 2066.001 Date: 9/15/2021

Drawn MMT
Checked

B-6
FIGURE



MPEG MILLER PACIFIC ENGINEERING GROUP

FILENAME: 2066.001 CPTs.dwg

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

CPT-05 2021 DATA

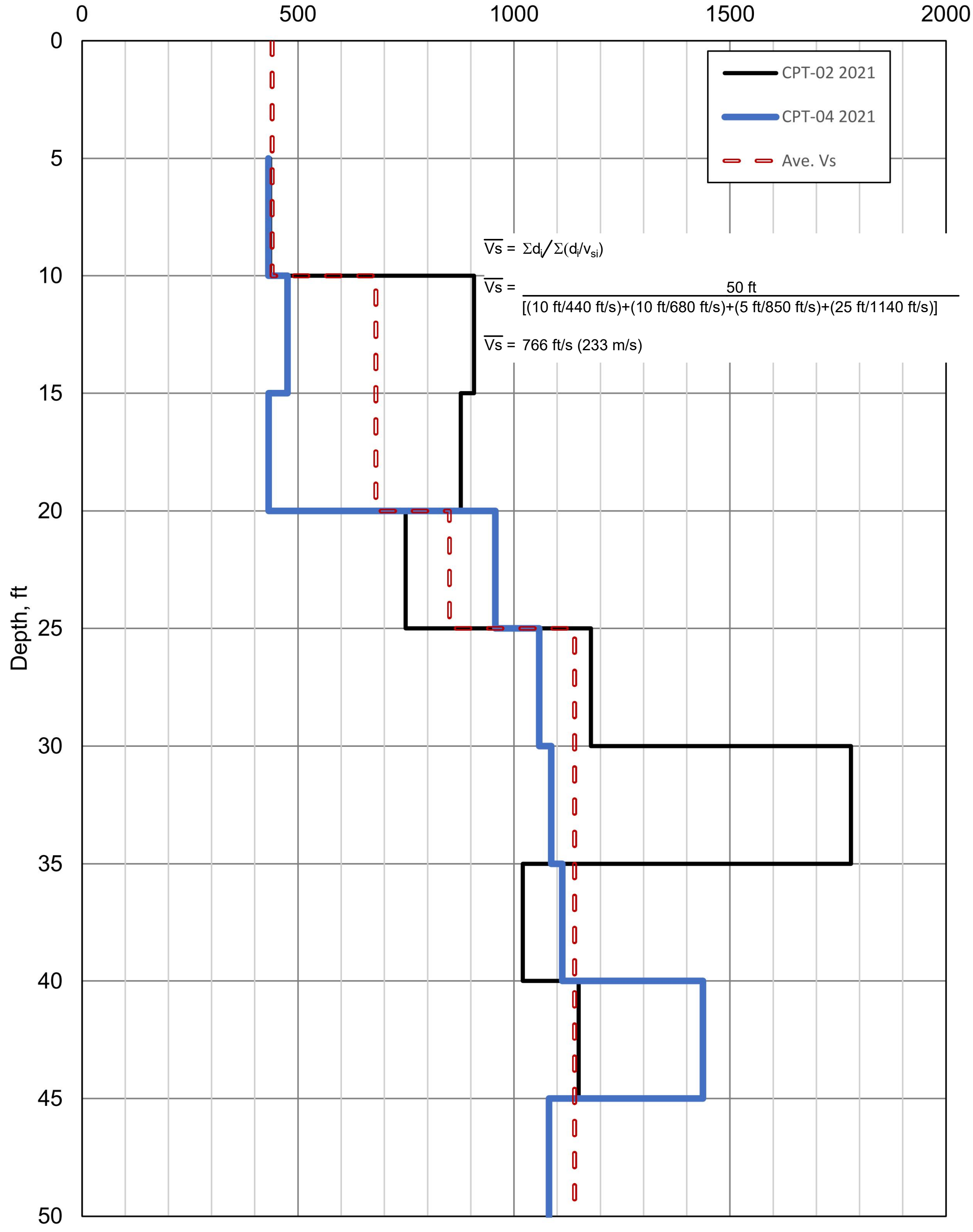
Riverbend Development
529 Madison Street
Petaluma, California

Project No. 2066.001 Date: 9/15/2021

Drawn MMT
Checked

B-7
FIGURE

Shear Wave Velocity vs. Depth



$\overline{Vs} = \Sigma d_i / \Sigma(d_i/v_{si})$

$\overline{Vs} = \dfrac{50\text{ ft}}{[(10\text{ ft}/440\text{ ft/s})+(10\text{ ft}/680\text{ ft/s})+(5\text{ ft}/850\text{ ft/s})+(25\text{ ft}/1140\text{ ft/s})]}$

$\overline{Vs} =$ 766 ft/s (233 m/s)

MPEG MILLER PACIFIC ENGINEERING GROUP

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

FILENAME: 2066.001 CPTs.dwg

SHEAR WAVE VELOCITY PROFILES

Riverbend Development
529 Madison Street
Petaluma, California

Project No. 2066.001 Date: 9/15/2021

Drawn MMT
Checked

B-8
FIGURE

APPENDIX C
RISK TARGETED MAXIMUM CONSIDERED
EARTHQUAKE (MCE_R) GROUND MOTION HAZARD ANALYSIS

Due to the presence of sandy soil layers beneath the building site that are prone to liquefaction, we judge the site should be classified as “Site Class F” per the 2019 California Building Code. However, per section 20.3.1 of the ASCE 7-16, an equivalent linear site-specific response analysis (i.e., SHAKE, DeepSoil, etc.) is not required if the proposed structure has a fundamental period of less than 0.5 seconds. We anticipate the proposed structures will have fundamental periods less than 0.5-seconds; therefore, based on the harmonic mean of the blow counts we recommend classifying the site as a “Site Class D”.

The ASCE 7-16 mapped spectral acceleration parameters at a period of 0.2-second, S_S, and 1.0-second, S_1, at the project site are 1.50 g and 0.60 g, respectively. Per ASCE 7-16 Table 11.4-1 a Site-Specific Ground Motion shall be developed per Section 11.4.8 for S_S values greater than 1.0 g for Site Class E sites and all cases for Site Class F sites. Additionally, a Site-Specific Ground Motion Hazard Analysis shall be performed per ASCE 7-16 Section 11.4.8 if the S_1 value is greater than 0.2 g for Site Class D, greater than 1.0 g for Site Class E, and all cases for Site Class F. Therefore, per ASCE 7-16 Section 11.4.8, we performed a Site-Specific Ground Motion Hazard Analysis per ASCE 7-16 Section 21.2, as described in the sections below.

Probabilistic (MCE_R) Ground Motions: Method 1

A probabilistic acceleration response spectrum, corresponding to a 2% chance of exceedance in 50-years (2,475 return period) was generated utilizing the United States Geologic Survey (USGS) online Unified Hazard Tool (https://earthquake.usgs.gov/hazards/interactive/, accessed 2021) for a Site Class D soil profile (V_{S30} = 260 m/s) an the Dynamic: Conterminous U.S. 2014 (v4.2.0) model. The accelerations given were modified by the risk coefficients C_{RS} and C_{R1}, 0.91 and 0.91, respectively. The accelerations were further converted to the probabilistic spectral response acceleration in the maximum horizontal response utilizing the procedures outlined by Shahi and Baker, 2013. These modifications to the probabilistic spectra correspond to a response with a risk targeted level of 1% probability of collapse within a 50-year period. The resulting probabilistic MCE_R values and spectra are presented on Figures C-1 and C-2, respectively.

Deterministic (MCE_R) Ground Motions

A deterministic acceleration response spectrum was generated utilizing the NGA attenuation models outlined by Abrahamson, Silva & Kamai (2014); Boore, Stewart, Seyhan & Atkinson (2014); Campbell & Borzognia (2014); and Chiou & Youngs (2014) NGA2 West models for a Site Class D (V_{S30} = 260 m/s). The geometric average of the 84th percentile spectral accelerations from the aforementioned attenuation relationships were modified for the probabilistic spectral response acceleration in the maximum horizontal direction, utilizing the procedures outlined by Shahi and Baker, 2013. The resulting deterministic MCER values and spectra are shown on Figures C-1 and C-2, respectively. The deterministic MCE_R spectra shall not be less than the Lower Limit Deterministic MCE_R Response Spectrum, as described in ASCE 7-16 Figure 21.2-1 which is tabulated and plotted on Figures C-1 and C-2, respectively.

Site Specific MCE_R

The site specific MCE_R spectral response acceleration at any period shall be taken as the lesser of the response accelerations from the probabilistic ground motions and the deterministic ground motions and is presented on Figure C-3. Additionally, per ASCE 7-16 Section 21.3, the design spectral response acceleration at any period is equal to 2/3^{rds} the MCE_R Response Spectrum, as shown on Figure C-3.

Per ASCE 7-16 Section 21.4, the MCE_R spectral response acceleration parameters shall be taken from the Site-Specific Spectrum defined as follows and are presented on Figure C-3:

- S_{DS} – The S_{DS} parameter shall be taken as 90% of the maximum spectral acceleration, S_a, obtained from the site-specific spectrum, at any period between 0.2 and 5.0-seconds. However, the values obtained shall not be less than 80% of the values determined in accordance with ASCE 7-16 Section 11.4.5.
- S_{D1} – The S_{D1} parameter shall be taken as the maximum value of the product, TS_a, for periods between 1.0 and 2.0-seconds for Site Class C and B sites; and periods between 1.0 and 5.0-seconds for Site Class D, E & F sites. However, the values obtained shall not be less than 80% of the values determined in accordance with ASCE 7-16 Section 11.4.5.
- S_{MS} – The S_{MS} parameter is equal to 1.5 times the S_{DS} value, but not less than 80% of the values determined in accordance with ASCE 7-16 Section 11.4.4.
- S_{M1} – The S_{M1} parameter is equal to 1.5 times the S_{D1} value, but not less than 80% of the values determined in accordance with ASCE 7-16 Section 11.4.4.

ASCE 7-16
SITE SPECIFIC RISK-TARGETED
MAXIMUM CONSIDERED EARTHQUAKE (MCE_R)

Project Name: Riverbend Residential Development
Project Numb: 2066.001

Latitude: 38.2417
Longitude: -122.6383

General Seismic Parameters
ASCE 7-16 Section 11.4

Site Class:	D
S_S (g):	1.50
S_1 (g):	0.60
F_a:	1.20
F_v:	N/A
T_L (sec):	12.0
C_{RS}:	0.91
C_{R1}:	0.91

Minimum Design Spectra Parameters
ASCE 7-16 Section 21.3

Site Class:	D	S_{MS} (g):	1.50
S_S (g):	1.50	S_{M1} (g):	1.50
S_1 (g):	0.60	S_{DS} (g):	1.00
F_a:	1.00	S_{D1} (g):	1.00
F_v:	2.50	T_0 (sec):	0.20
		T_S (sec):	1.00

Deterministic MCE Screening
ASCE 7-16 (Sup #1) 21.2.3

Fa:	1.00
1.2 x Fa (g):	1.20
Max PSHA (g):	2.20
DSHA Rqd.:	**YES**

Min. Deterministic MCE
ASCE 7-16 (Sup #1) 21.2.2

Fa:	1.00
1.5 x Fa (g):	1.50
Max DSHA (g):	1.79
Min MCE Rqd.:	**NO**

Probabilistic MCE
ASCE 7-16 Section 21.2.1 - Method 1

Period (sec)	Sa_{RotD50} (g)	$Sa_{RotD100}$ / Sa_{RotD50}	$Sa_{RotD100}$ (g)	C_R	Sa (g)
0.01	0.81	1.10	0.89	0.914	0.81
0.10	1.35	1.10	1.48	0.914	1.36
0.20	1.81	1.10	1.99	0.914	1.82
0.30	2.08	1.13	2.34	0.913	2.14
0.50	2.05	1.18	2.41	0.911	2.20
0.75	1.69	1.24	2.09	0.908	1.90
1.00	1.43	1.30	1.86	0.905	1.68
2.00	0.80	1.35	1.07	0.905	0.97
3.00	0.53	1.40	0.74	0.905	0.67
4.00	0.38	1.45	0.55	0.905	0.50
5.00	0.29	1.50	0.44	0.905	0.40

Deterministic MCE
NGA West2 2014 - 84th Percentile

Period (sec)	Sa_{RotD50} (g)	$Sa_{RotD100}$ / Sa_{RotD50}	$Sa_{RotD100}$ (g)
0.01	0.61	1.10	0.67
0.02	0.61	1.10	0.67
0.03	0.62	1.10	0.68
0.05	0.68	1.10	0.75
0.08	0.81	1.10	0.89
0.10	0.94	1.10	1.03
0.15	1.15	1.10	1.26
0.20	1.30	1.10	1.43
0.25	1.41	1.11	1.57
0.30	1.50	1.13	1.69
0.40	1.55	1.15	1.79
0.50	1.52	1.18	1.78
0.75	1.27	1.24	1.57
1.00	1.08	1.30	1.41
1.50	0.79	1.33	1.05
2.00	0.61	1.35	0.82
3.00	0.40	1.40	0.56
4.00	0.28	1.45	0.40
5.00	0.20	1.50	0.30
7.50	0.10	1.50	0.14
10.00	0.05	1.50	0.08

Scaled Deterministic MCE
ASCE 7-16 (Sup #1) 21.2.2

Period (sec)	Sa (g)
0.01	0.57
0.02	0.57
0.03	0.57
0.05	0.63
0.08	0.74
0.10	0.87
0.15	1.06
0.20	1.20
0.25	1.32
0.30	1.42
0.40	1.50
0.50	1.50
0.75	1.32
1.00	1.18
1.50	0.88
2.00	0.69
3.00	0.47
4.00	0.34
5.00	0.25
7.50	0.12
10.00	0.07

Site Specific MCE_R
ASCE 7-16 Section 21.2.3

Period (sec)	Sa (g)
0.01	0.67
0.02	0.67
0.03	0.68
0.05	0.75
0.08	0.89
0.10	1.03
0.15	1.26
0.20	1.43
0.25	1.57
0.30	1.69
0.40	1.79
0.50	1.78
0.75	1.57
1.00	1.41
1.50	1.05
2.00	0.82
3.00	0.56
4.00	0.40
5.00	0.30
7.50	0.14
10.00	0.08

Site-Specific Design Spectrum
ASCE 7-16 Section 21.3

Period (sec)	Sa (g)
0.01	0.45
0.02	0.45
0.03	0.45
0.05	0.50
0.08	0.59
0.10	0.69
0.15	0.84
0.20	0.95
0.25	1.05
0.30	1.13
0.40	1.19
0.50	1.19
0.75	1.05
1.00	0.94
1.50	0.70
2.00	0.55
3.00	0.37
4.00	0.27
5.00	0.20
7.50	0.10
10.00	0.05

80% General Response Spectrum
ASCE 7-16 Section 21.3

	Period (sec)	Sa (g)	80% Sa (g)
	0.01	0.43	0.34
	0.04	0.53	0.42
	0.07	0.62	0.50
	0.11	0.72	0.57
	0.14	0.81	0.65
	0.17	0.91	0.72
T_O =	0.20	1.00	0.80
T_S =	1.00	1.00	0.80
	1.31	0.76	0.61
	1.62	0.62	0.50
	1.92	0.52	0.42
	2.23	0.45	0.36
	2.54	0.39	0.32
	2.85	0.35	0.28
	3.15	0.32	0.25
	3.46	0.29	0.23
	3.77	0.27	0.21
	4.08	0.25	0.20
	4.38	0.23	0.18
	4.69	0.21	0.17
	5.00	0.20	0.16

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

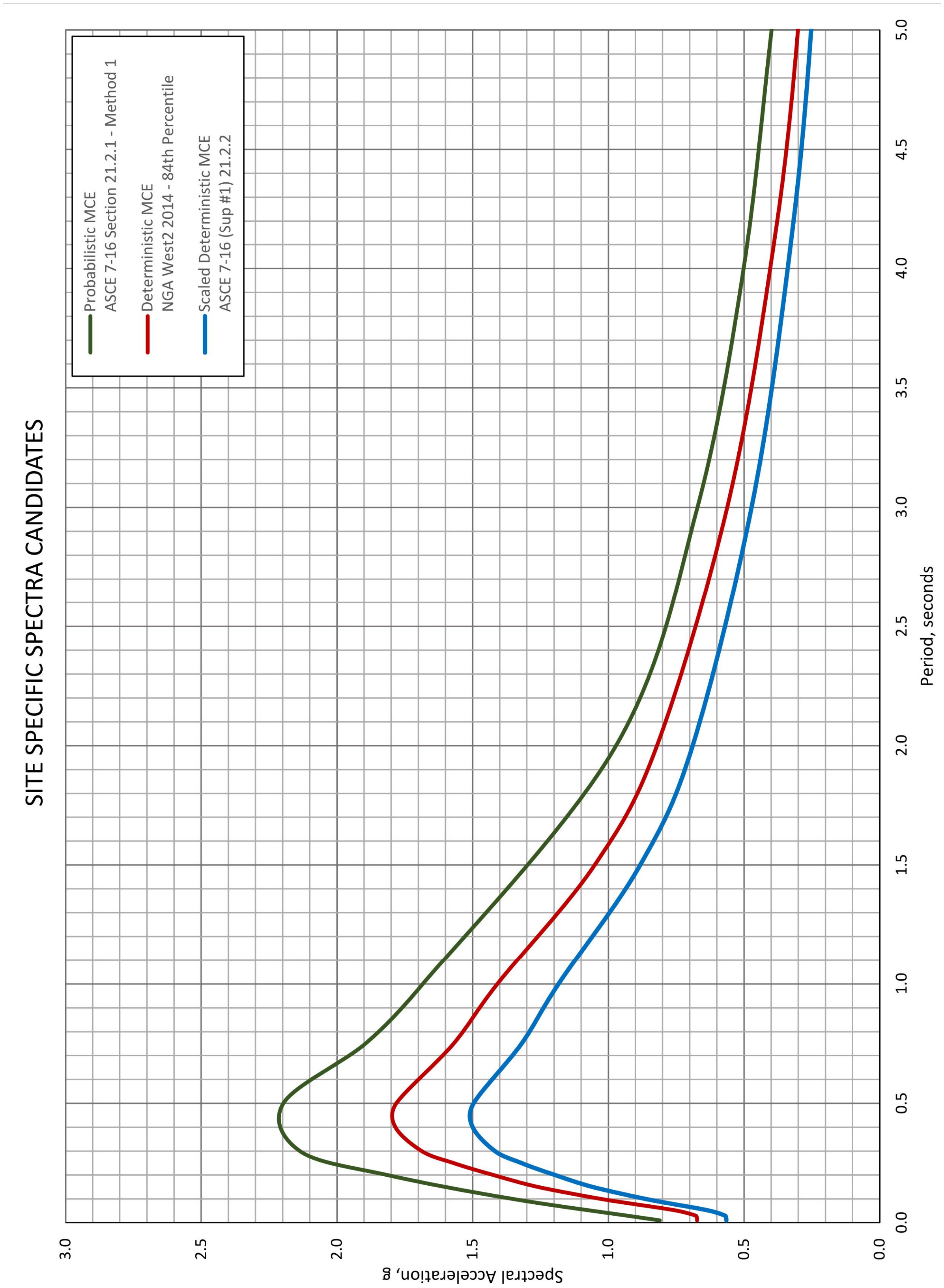
FILE: 2066.001 Site Specific.dwg

ASCE 7-16 MCEr CALCULATIONS

Riverbend Development
529 Madison Street
Petaluma, California
Project No. 2066.1001 Date: 9/15/2021

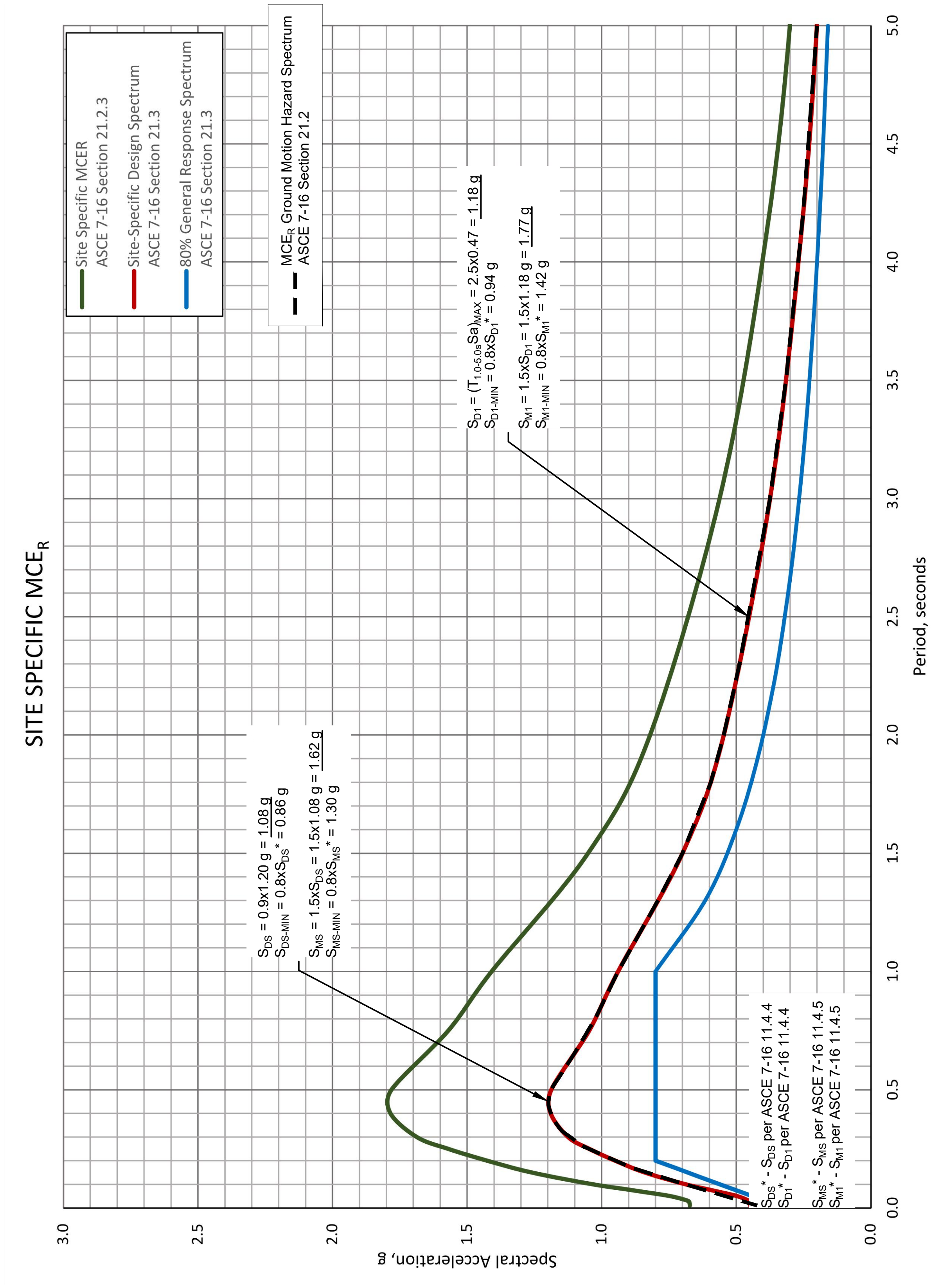
Drawn MMT
Checked

C-1
FIGURE



MPEG MILLER PACIFIC ENGINEERING GROUP	504 Redwood Blvd. Suite 220 Novato, CA 94947 T 415 / 382-3444 F 415 / 382-3450 www.millerpac.com	ASCE 7-16 MCEr CANDIDATE SPECTRA
		Riverbend Development 529 Madison Street Petaluma, California Project No. 2066.001 Date: 9/15/2021
		Drawn MMT Checked
		C-2 FIGURE

FILE: 2066.001 Site Specific.dwg



MILLER PACIFIC ENGINEERING GROUP

FILE: 2066.001 Site Specific.dwg

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

ASCE 7-16 MCEr DESIGN SPECTRUM

Riverbend Development
529 Madison Street
Petaluma, California

Project No. 2066.001 Date: 9/15/2021

Drawn MMT
Checked

C-3
FIGURE