MPEG MILLER PACIFIC ENGINEERING GROUP

PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED MALLARD POINTE RESIDENTIAL DEVELOPMENT
MALLARD ROAD
BELVEDERE, CALIFORNIA

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Job No. 3068.001

Prepared for:
Thompson/Dorfman Urban Residential Development
39 Forrest Street, Suite 201
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CERTIFICATION

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1.0 INTRODUCTION

This report presents the results of our Preliminary Geotechnical Investigation for the proposed Mallard Pointe Residential Development on Mallard Road in Belvedere, California. As shown on Figure 1, the project site is located on both sides of Mallard Road between Community Road and Belvedere Lagoon.

Our work was performed in accordance with our Agreement for Professional Services dated September 4, 2020. The purpose of our investigation was to explore the subsurface conditions and to develop preliminary geotechnical criteria for design and construction of the proposed improvements. The scope of our services includes:

- Review of available, published geologic mapping and geotechnical background information from our files, and any geologic/geotechnical background information supplied by you.
- Coordinate with Underground Service Alert (USA) to mark underground utilities in areas where we plan to conduct subsurface exploration.
- Subsurface exploration consisting of one day of Cone Penetration Tests (CPTs). We completed five CPTs that extended through the near-surface soils to a depth of about 100 feet below the ground surface or to refusal in firm soil or bedrock.
- Evaluation of relevant geologic hazards including seismic shaking, liquefaction, settlement, and other hazards.
- Preparing preliminary geotechnical recommendations and design criteria related to building foundations, site grading, temporary shoring, seismic design, and other geotechnical-related items.
- Preparing a Preliminary Geotechnical Investigation report which summarizes the referenced subsurface exploration, evaluation of relevant geologic hazards, and preliminary geotechnical recommendations and design criteria.

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

2.0 PROJECT DESCRIPTION

We understand that the project will consist of demolishing the existing structures and re-developing the property as a new residential development. The proposed development includes six single-family homes, five duplexes, and a two-story apartment building over parking. We anticipate that the new buildings would be wood frame structures with either wood frame or concrete slab-on-grade floors at the ground floor levels. Ancillary improvements are expected to include exterior hardscape/flatwork and asphalt paving, new underground utilities, new site drainage, landscaping, and other improvements "typical" of such developments. No detailed structural information is available at this time. However, we anticipate that the proposed structures will likely impose light to moderate foundation loads. A site plan showing the existing conditions and the approximate CPT locations is presented on Figure 2.

3.0 SITE CONDITIONS

3.1 Regional Geology

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

Regional geologic mapping (Rice, et al, 1976) indicates that the project site is underlain by artificial fill over Bay Mud (Geologic unit Qaf/Qm). Artificial fill typically consists of deposits of rock, soil, or Bay Mud placed by man upon natural surfaces. Bay Mud typically consists of thick deposits of unconsolidated, highly compressible silty clay. A regional geologic map of the site and vicinity is presented on Figure 3.

3.2 Seismicity

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

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

3.2.1 Regional Active Faults

An “active” fault is one that shows displacement within the last 11,000 years (i.e., Holocene) and has a reported average slip rate greater than 0.1 mm per year. The California Division of Mines and Geology has mapped various active and inactive faults in the region. These faults are shown in relation to the project site on the attached Active Fault Map, Figure 4. The nearest known active faults are the San Andreas, Hayward and San Gregorio Faults which are located roughly 13.3 kilometers southwest, 15.5 kilometers northeast, and 17.2 kilometers southwest of the site, respectively.

3.2.2 Historic Fault Activity

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

3.2.3 Probability of Future Earthquakes

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

Conclusions from the most recent UCERF3 and USGS indicate the highest probability of an earthquake with a magnitude greater than 6.7 originating on any of the active faults in the San Francisco Bay region by 2043 is assigned to the Hayward/Rodgers Creek Fault system. The Rodgers Creek Fault is located approximately 15.5 kilometers (9.6 miles) northeast of the site and is assigned a probability of 33 percent. The San Andreas Fault, located approximately 13.3 kilometers (8.3 miles) southwest of the site, is assigned a 22 percent probability of an earthquake with a magnitude greater than 6.7 by 2043. Additional studies by the USGS regarding the probability of large earthquakes in the Bay Area are ongoing. These current evaluations include data from additional active faults and updated geological data.

3.3 Surface Conditions

The project site (three parcels) is located on both sides of Mallard Road between Community Road and Belvedere Lagoon in Belvedere. Existing site elevations within the proposed residential development area range from approximately +7 to +15 feet MSL (mean sea level) based on Google Earth elevations. The project area is relatively flat and currently developed with existing residential structures and asphalt drive areas. Existing timber bulkheads (typically 2 to 3 feet high) are present along much of the Belvedere Lagoon frontage of the project site. A summary of our preliminary geotechnical evaluation of the existing bulkheads is provided in Appendix B.

3.4 Field Exploration

We explored the subsurface conditions near the proposed improvements on April 26, 2021 with five cone penetration tests (CPTs) at the approximate locations shown on Figure 2. The CPTs were excavated using an International Paystar 5000 by Middle Earth Geo Testing to depths ranging from 70 feet to 100 feet below the ground surface. The interpreted soil types, densities, strengths, and liquefaction potential are presented in Appendix A. Additional geotechnical exploration with borings, additional cone penetration testing, and laboratory testing of soil samples should be performed at a later date as part of a Phase 2 design level geotechnical investigation.

3.5 Subsurface Soil Conditions

The subsurface exploration generally confirms the regionally mapped geologic conditions at the site. The site is overlain by approximately 5 to 8-feet of medium dense to dense clayey sand to stiff sandy clay overlying about 20 to 70-feet of soft, compressible Bay Mud. Loose to dense silty sands were encountered beneath the silty clay (Bay Mud) to the maximum explored depth of 100.5-feet.

3.6 Groundwater

Groundwater was encountered in the CPTs at depths between 6 and 9-feet below the ground surface. Groundwater should generally be expected in on-site excavations deeper than about 5 feet below grade and may be shallower during the winter months or following periods of heavy rain or high tides.

4.0 GEOLOGIC HAZARDS

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

4.1 Fault Surface Rupture

Under the Alquist-Priolo Earthquake Fault Zoning Act, the California Division of Mines and Geology (now known as the California Geological Survey) produced 1:24,000 scale maps showing known active and potentially active faults and defining zones within which special fault studies are required. The nearest known active fault to the site is the San Andreas Fault located approximately 13.3 kilometers to the southwest. The site is not located within an Alquist-Priolo Special Studies Zone. We therefore judge the potential for fault surface rupture in the development area to be low.

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

4.2 Seismic Shaking

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

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

Table 1 – Deterministic Peak Ground Accelerations for Active Faults

Fault	Moment Magnitude for Characteristic Earthquake	Closest Estimated Distance (km)	Median Peak Ground Acceleration (g)	84% Peak Ground Acceleration (g)
San Andreas	7.9	13.3	0.30	0.49
Hayward	7.3	15.5	0.25	0.41
San Gregorio	7.4	17.7	0.24	0.40
Rodgers Creek	7.4	32.9	0.17	0.29
Contra Costa	6.5	31.6	0.12	0.20

Reference: Caltrans ARS Online v2.3.09 accessed on May 18, 2021. Site Class E= 180 (ft/sec)

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

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

Table 2 – Probabilistic Peak Ground Accelerations for Active Faults

Probability of Exceedance	Statistical Return Period	Magnitude	Peak Ground Acceleration (g)
2% in 50 years	2,475 years	7.23	0.72
10% in 50 years	475 years	7.11	0.46

Reference: USGS Unified Hazard Tool accessed on May 18, 2021. Site Class E= 180 (ft/sec)

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

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

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

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

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

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

We analyzed the potential for liquefaction utilizing the procedures outlined by Idriss and Boulanger (2014) within the liquefaction interpretation program, CLiq (Geologismiki, CLiq). The seismic event input into the model consisted of a magnitude 7.9 earthquake producing a PGA of 0.63 g, which corresponds to the PGA_M defined in ASCE 7-16. The results of our analyses indicate several liquefiable layers of various thicknesses underlie the site at various depths between 30 and 50-feet below the ground surface. The results of our liquefaction analyses are presented on Figures 7 through 11.

4.3.1 Post Liquefaction Settlement

Based on current post liquefaction settlement analyses procedures, settlement can occur in soils that exhibit a factor of safety against liquefaction of 2.0 or less. Utilizing the procedures outlined by Idriss and Boulanger (2014) approximately 0.5 to 1.5-inches of post liquefaction settlement may occur. It is noted that the estimated liquefaction induced settlement based on an analysis of CPT 2 is about 1.5 inches. Approximately 0.5 inches of liquefaction induced settlement is predicted based on an analysis of CPTs 1, 3, 4, and 5. We recommend that an exploratory boring should be positioned near CPT 2 during the Phase 2 design level geotechnical study to confirm or modify the predicted liquefaction induced settlement at this location. Differential settlement is estimated to be approximately one half of the estimated total settlement.

Additionally, we utilized the procedures outlined by Ozocak and Sert (2010) to calculate the Liquefaction Potential Index (LPI), which is a gauge to determine if liquefiable layers will impact the ground surface. LPI is a function of the thickness, depth, and factor of safety against liquefaction in the individual layers within a soil column. The resulting LPI value corresponds to a relative potential for surface deformation impacting the ground surface. Typically, an LPI value of zero indicates the liquefiable layer will not impact the ground surface; while a value less than 5 has a low probability, value between 5 and 15 have a moderate probability and an LPI value greater than 15 have a high probability of surface impact. The results of our liquefaction analyses indicate LPI between 1.0 and 5.0, suggesting a low probability of liquefaction effects impacting the ground surface.

Evaluation: *Less than significant with mitigation.*

Recommendation: *Shallow foundation systems should be designed to withstand up to 1.5-inches of total and 0.75-inches of liquefaction induced differential settlement, over 30-feet. If deep foundations are utilized, they should be designed to account for localized layers of reduced skin friction for seismic conditions. Flexible utility conduits and connections are recommended to reduce the likelihood of damage due to differential post-liquefaction settlements. Foundation design criteria to mitigate the effects of liquefaction are provided in Section 5.5 should be followed.*

4.4 Settlement

Significant settlement can occur when loads (fill or structures) are placed at sites that are located over Bay Mud. The project site is located over a relatively thick (from 10- to 70-foot deep) deposit of Bay Mud. The amount and rate of settlements are dependent on the amount of previous loads and additional (new) loads, previous loading history, the thickness of compressible material, and the inherent compressibility properties of the Bay Mud.

Differential settlements are also possible due to variations in the thickness of compressible Bay Mud, variations in new long-term loads (fill thickness or foundation loads) and variations in historic use of the land, i.e., old channels or low points through the site that may have required thicker fills, or previous “surcharges”, such as old structures or fill mounds. Because the site was filled long ago (about 70 years ago), the Bay Mud has completed a majority of the primary consolidation settlement under the loads from the existing fill and existing structures in areas where the Bay Mud thickness is less than about 30 feet. In areas underlain by more than about 30 feet of Bay Mud, several inches of additional primary consolidation settlement is expected in the future. Smaller secondary compression settlements (up to several inches) are also still occurring across the entire site area. Secondary compression occurs mostly after the primary consolidation settlement is complete, and occurs slowly over many decades.

Predicted site settlements (primary consolidation) caused by the original fill placed in 1950 are presented below in Table 3. Table 3 also presents the estimated settlement that will occur for different Bay Mud layer thicknesses over the next 50 years. Fifty years is a typical time span that is considered for the useful life of a residential structure, before significant repairs and renovation are needed.

Table 3– Predicted Site Settlements

Thickness of Bay Mud Layer	Total Predicted Settlement	Predicted Year of Settlement 99% Complete	Anticipated Settlement from 2022 to 2072
10 feet	27 inches	1960	0 inches
20 feet	35 inches	1990	0 inches
30 feet	41 inches	2030	0.4 inches
40 feet	45 inches	2070	2.6 inches
50 feet	49 inches	2130	5.5 inches
60 feet	52 inches	2210	7.5 inches
70 feet	54 inches	2310	8.5 inches

Although we do not anticipate significant site grading at the project site, for illustrative purposes, placing a uniform 1-foot-thick layer of normal weight soil fill (approximately 125 pounds per cubic foot) would result in about 7 inches of long-term settlement in an area underlain by 20-feet of Bay Mud. The predicted settlement increases to about 9 inches in an area underlain by 40-feet of Bay Mud and about 10 inches where 60 feet of Bay Mud exists. Linear extrapolation of settlement estimates is generally possible, such that doubling the thickness of new fill will similarly double the settlement amounts.

New building loads will also result in settlements, and the amount of settlement is proportional to the applied load. Differential settlements will also occur where fills terminate or where concentrated footing loads are applied.

It is noted that many nearby sites along the Belvedere Lagoon and other waterfront sites in Marin County share the same settlement risks as the project site due to the Bay Mud that typically underlies most waterfront properties in the region.

Evaluation: *Less than significant with mitigation.*

Recommendation: *Alternatives to reduce settlement/subsidence include minimizing the amount (weight) of new fill or foundation loads, using lightweight fill where grades must be raised, over-excavating to "offset" new loads, or designing the proposed structures to withstand total and differential ground settlements.*

Utilization of deep foundations would reduce expected total and differential settlements of structures to near-zero but would result in significant differential settlement between the structure and surrounding grades. The cost effectiveness and need for deep foundations for a given structure should be based on an evaluation of the settlement risk at the specific building location and the value of the building and costs associated with mitigation of settlement damage to the building.

Alternatively, a rigid shallow foundation system could be utilized, provided that the expected total and differential settlements are acceptable. Note that such settlements may result in damage to "brittle" surfaces such as stucco or plaster, and eventual re-leveling of foundations by pressure-grouting, underpinning, or other means will likely be necessary to correct differential settlements.

Placement of new loads at the site, especially fills, will induce additional settlement, and should generally be avoided in the interest of best performance of the new structures. Removal of existing fill soils and replacement with lightweight materials such as lava rock, cell-crete, or geofoam may be considered as a means of achieving a "net zero new load" condition, which would result in no net change to predicted future settlements. New utilities should be provided with flexible connections and emergency shut-off valves to reduce the risk of damage due to future settlements. Additional discussion and preliminary recommendations for site grading, settlement mitigation, and new foundations are presented in Section 5 of this report.

4.5 <u>Seismic Densification</u>

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

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

4.6 Expansive Soils

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

Additional exploration and laboratory testing should be performed to determine the expansive potential of surficial soils as part of a Phase 2 design-level report. We do not anticipate significant mitigation measures, and the “stiffened” foundations recommended in Section 5 will improve performance of structures over potentially expansive soils.

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

4.7 Lurching and Ground Cracking

Lurching and associated ground cracking can occur during strong ground shaking. The ground cracking generally occurs along the tops of slopes where stiff soils are underlain by soft deposits or along steep slopes or channel banks. Slopes adjacent to the site are currently relatively low and flat. If new fills are placed at the site, the risk of lurching and ground cracking near slopes would be increased. Provided new fills are not placed at the site, the risk of lurching and ground cracking impacting the new structures is low.

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

4.8 Erosion

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

Evaluation: *Less than significant with mitigation.*
Recommendation: *For new improvements at the site, careful attention should be paid to finished grades and the project Civil Engineer should design the site drainage system to collect surface water into a storm drain system that discharges water at appropriate locations. Re-establishment of vegetation*

on disturbed areas will also minimize erosion. Erosion control measures during and after construction should be in accordance with a prepared Storm Water Pollution Prevention Plan and should conform to the most recent version of the California Stormwater Quality Association, Stormwater Best Management Practice Handbook.

4.9 Slope Instability/Landsliding

Slope instability generally occurs on relatively steep slopes and/or on slopes underlain by weak materials. The project site lies on nearly level terrain, therefore, slope instability/landsliding is not considered a significant geologic hazard at the project site. Further studies of slope stability would be required if new fills are planned at the project site.

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

4.10 Flooding

The project site is located at about elevation +7 feet and is mapped as being within a 100-year flood zone (ArcGIS, 2021) as shown on Figure 12. Therefore, large scale flooding is considered a moderate hazard at the project site.

Evaluation: *Less than significant with mitigation.*
Recommendations: *Measures include designing floor elevations that will be above the local permitting agency minimums. Consideration should also be given to design of finished grades at the site so that adverse drainage conditions do not allow water to pond around or beneath structures. Additional geotechnical recommendations for site drainage are provided in Section 5.*

4.11 Tsunami and Seiche

Seiche and tsunamis are short duration, earthquake-generated water waves in large, enclosed bodies of water and the open ocean, respectively. The extent and severity of a seiche or tsunami would be dependent upon ground motions and fault offset from nearby active faults.

There have been eight credible local seiche events observed in San Francisco Bay between 1854 and 1906, six of which are attributed to earthquake activity and two to landslides. The Mare Island earthquake caused the largest seiche with 0.6-meter amplitude waves near Benicia and is attributed to slip on the Rodgers Creek fault. No confirmed seiche have been recorded in San Francisco Bay since 1906.

The project site is located on Belvedere Lagoon and approximately 2,000 feet east of Richardson Bay and is mapped as being within an inundation area as shown on Figure 13 (ArcGIS, 2021). Therefore, the risk of inundation is moderate at the project site.

Evaluation: *Less than significant with special engineering measures.*
Recommendations: *As for mitigation of flooding and sea-level rise hazards, mitigation measures should include designing finished floor elevations above predicted future Marin County tsunami runup elevations, considering the likelihood that sea level may rise several feet. Estimates of expected future*

settlements should also be considered in evaluating inundation potential. Careful consideration should be given to design of finished grades and site drainage to minimize the potential for damage due to flooding. A new floodwall with top-of-wall elevations above the predicted seiche runup elevation could be considered; however, we judge this is likely neither warranted nor cost-effective given the scope of the project and given that the floodwalls would need to extend along all adjacent properties to be effective. Additional discussion of expected future settlements and preliminary recommendations for site grading and drainage are presented in Section 5 of this report.

4.12 <u>Sea Level Rise</u>

Globally, sea levels are rising due to thermal expansion caused by the ocean warming and the melting of land-based ice such as glaciers and polar ice caps. Regionally and locally, sea level rise has the potential to influence the impact of coastal, riverine, and localized nuisance flooding. These may result in permanent inundation, more frequent and longer duration floods, shoreline erosion and overtopping, and elevated groundwater and increased salinity intrusion. The National Oceanic and Atmospheric Administration (NOAA) predicts that the sea level may rise as much as 1- to 3-feet within the next 30 years and even greater levels by 2100. The project site is located on Belvedere Lagoon and approximately 2,000 feet east of Richardson Bay at an elevation of between +7 and +15 feet above sea level. Therefore, the risk of sea level rise inundation is moderate at the project site. A detailed analysis of Sea Level Rise was beyond the scope of our evaluation but could be considered by a Civil Engineer and or consultation with the Planning Department.

Evaluation: *Less than significant with special engineering measures.*

Recommendations: *Mitigation measures should include designing finished floor elevations above predicted future Marin County flood elevation minimums, considering the likelihood that sea level may rise several feet. Estimates of expected future settlements should also be considered in evaluating flood potential. Careful consideration should be given to design of finished grades and site drainage to minimize the potential for damage due to flooding. A new floodwall with top-of-wall elevations above the predicted base flood elevation could be considered; however, we judge this is likely neither warranted nor cost-effective given the scope of the project and given that the floodwalls would need to extend along all adjacent properties to be effective. Additional discussion of expected future settlements and preliminary recommendations for site grading and drainage are presented in Section 5 of this report.*

5.0 <u>PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS</u>

Based on the results of our preliminary investigation, we conclude the site conditions are suitable for the proposed improvements. The primary geotechnical issues to address in design of the project are strong seismic shaking due to the close proximity of the San Andreas Fault, liquefaction and liquefaction induced settlement, settlement of soft clay deposits (Bay Mud), and flooding.

5.1 Seismic Design

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

The magnitude and character of these ground motions will depend on the particular earthquake and the site response characteristics. Based on the interpreted subsurface conditions and close proximity to the San Andreas, Hayward, and San Gregorio Faults, we recommend the CBC coefficients and site values shown in Table 4 below for use to calculate the design base shear of the new construction.

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

Table 4 – Preliminary 2019 California Building Code Seismic Design Criteria

Parameter	Design Value	
Site Latitude	37.8748°N	
Site Longitude	-122.4653°W	
Site Class	C	E
Spectral Response (short), S_S	1.50 g	1.50 g
Spectral Response (1-sec), S_1	0.60 g	0.60 g
Spectral Response (Short), S_{MS}	1.80 g	n/a
Design Spectral Response (short), S_{DS}	1.20 g	n/a
Short Period Site Coefficient, F_a	1.2	n/a
MCE_G PGA Adjusted, PGA_M	0.63 g	0.62 g

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

5.2 Preliminary Site Grading Considerations

Minor site grading is anticipated for the project, consisting chiefly of excavations for new foundations, drainage systems, and underground utilities. Given that placement of new fills will induce additional future settlements that could damage improvements, we generally do not recommend placing new fills at the site, and instead recommend achievement of planned finished-floor elevations by use of a concrete mat slab or post-tensioned slab-on-grade foundation system with interconnected perimeter and interior stem walls supporting elevated interior floors above a crawl space. Alternatively, a “traditional” continuous interconnected spread footing (“waffle slab”) foundation could be considered. Load balancing of the new buildings should be considered so as to not incur new loading at the site that would induce new settlements.

New flood or retaining walls a few feet high may be considered around the perimeter of the property. The loading associated with new flood walls is essentially negligible for the purpose of calculating future settlements. However, since raising exterior grades with normal-weight fill soils will induce significant new settlements within the property and potentially adjacent properties, we generally recommend that exterior grades be raised by use of Styrofoam blocks (which would result in essentially no new settlement), or with lightweight fill or cell-crete (which would include minor new settlements). Alternatively, framed wooden decks may be utilized. Given existing site elevations and likely shallow groundwater, Styrofoam may be susceptible to floating during high tides or flooding events, and either lava rock or cell-crete are likely more suitable for use as lightweight fill. Conceptually, we recommend perimeter walls consist of mechanically-stabilized earth (MSE) block-type walls (such as Versa-Lok or similar) which may be constructed on shallow footing/leveling-course foundations without the need for significant excavation. Additionally, reinforcing geogrid could be “pinned” between the block courses and extended across the full width of the yard to encourage uniform settlement in the event that new loads cannot be entirely offset by over-excavation as a result of shallow groundwater and low site grades.

Site excavations for new foundations, underground utilities, and other improvements will generally encounter 5- to 8-feet of variable fill material over soft Bay Mud. Based on our previous experience, the majority of site excavations can probably be performed with “traditional” grading equipment, such as medium-size dozers and excavators. Subsurface exploration and laboratory testing should be performed as part of a future design-level Investigation to confirm fill thicknesses, excavation conditions, and soil-type classifications. On a preliminary basis, onsite fill and Bay Mud soils should be considered “Type C” and will likely be prone to caving or collapse in open excavations.

Over-excavation and backfill may be required to stabilize the bottom of excavations where they “bottom” in Bay Mud. For new structures and pipelines, we recommend over-excavating a minimum of 12 inches below the bottom of the planned foundation or pipe flowline, placing stabilization fabric (Mirafi 500X or equivalent) on the soft Bay Mud, and placing ¾-inch lightweight lava rock to raise the grade to the specified subgrade elevation and reduce the risk of future settlement. Stabilization fabric should be wrapped over the top of the lava rock. Dewatering will likely be required in these areas to maintain dry working conditions, and also will likely need to be accomplished with submersible pumps, as dewatering wells are unlikely to be effective given the immediate proximity of San Francisco Bay.

5.3 Probable Foundation Types

In order to minimize the adverse impact of differential settlements, we recommend rigid shallow foundation systems should be used where predicted differential settlements are not excessive. In areas where predicted differential settlements are larger, deep foundation systems should be utilized.

Suitable shallow foundation systems include thick, heavily-reinforced mat slabs, grid foundations (consisting of continuous, interconnected footings), or post-tensioned slabs. Each of the shallow foundation systems should be designed to span areas of non-uniform support up to 15-feet in diameter and minimize the effects of post-construction differential settlements. Preliminary design criteria for mat and post-tensioned concrete slabs are shown in Table 5, while preliminary design criteria for shallow continuous footings are shown in Table 6.

Table 5 – Thickened Mat or Post-Tension Slab Design Criteria

Parameter	Design Value
Allowable Bearing Pressure	500 pounds per square foot
Modulus of Subgrade Reaction	100 psi per inch
Minimum Edge Thickness[1]	12 inches
Maximum Unsupported Interior Span[2]	15 feet
Maximum Unsupported Edge Cantilever	7 feet
Edge Moisture Variation, Center Lift	15 feet
Edge Moisture Variation, Edge Lift	7 feet
Differential Soil Movement, Edge & Center Lift	1.0 inches
Lateral Passive Resistance[3]	250 pounds per cubic foot

(1) Actual thickness, load distribution, and unsupported spans must be determined by the Structural Engineer to reduce deformations to acceptable levels.
(2) Assumes rigid slab behavior with idealized fixed end conditions.
(3) Equivalent fluid pressure. Neglect the upper six inches in calculating passive resistance unless confined by concrete or asphalt.

Table 6 – Shallow Continuous Footing Design Criteria

Parameter	Design Value
Minimum Width[1]	18 inches
Minimum Embedment[2]	18 inches
Allowable Bearing Pressure[3]	500 pounds per square foot
Base Friction Coefficient	0.30
Lateral Passive Resistance[4]	250 pounds per cubic foot
Maximum Unsupported Interior Span[5]	15 feet
Maximum Unsupported Edge Cantilever	7 feet

(1) Size foundations to maintain uniform bearing pressures, i.e., size footing widths to design loads instead of uniform foundation widths.
(2) Footings may need to be deeper if the Structural Engineer determines additional rigidity is required to evenly spread column loads.
(3) Dead plus live loads. May increase by 1/3 for total design loads, including wind and seismic.
(4) Equivalent fluid pressure. Neglect the upper six inches in calculating passive resistance unless confined by concrete or asphalt.
(5) Assumes rigid slab behavior with idealized fixed end conditions.

In areas of the site where the predicted total and differential post-construction settlements are unacceptable, a deep foundation system bearing on firm materials below the compressible Bay Mud could be considered for the new structures. In general, the areas of the site underlain by more than about 30 to 40 feet of Bay Mud (as shown on Figure 2) will experience larger differential settlements that may warrant the use of deep foundations. Suitable deep foundation options at the site could include auger-cast piers, torque-down piles, or helical piers. Driven piles are not recommended due to the noise and vibrations caused and the variable depth to achieve full pile capacity. Traditional drilled piers are also not recommended due to the high groundwater conditions and “squeezing” Bay Mud soils.

With adequately embedded auger cast piers, torque down piles, or helical piers, total and differential building settlements should be negligible. However, significant differential settlement between the building and exterior grades should be expected if deep foundations are used. Flexible utility conduits and connections are recommended to reduce the likelihood of damage at the interface with structures supported on deep foundations.

It should also be noted that construction of auger-cast piles and torque-down piles requires mobilization of very large equipment to the site. Helical piers are likely the most feasible and cost-effective option for deep foundation support. Based on our experience and previous exploration in the area, we estimate that helical piers could likely generate capacities on the order of 30-kips each at depths of about 100-feet below the ground surface. We anticipate that lateral passive resistance would need to be provided by concrete grade beams embedded a couple of feet below grade. Specific design criteria for deep foundation systems, if needed, will be provided in a future design level geotechnical report.

5.4 Interior Concrete Slabs-On-Grade

Interior concrete slabs should be reinforced with steel bars (not wire mesh) and should be designed by the Structural Engineer. Contraction joints should be incorporated in the concrete slab in both directions, no greater than 10 feet on center, and reinforcing bars should extend continuously through the control joints.

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

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

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

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

5.5 Exterior Concrete Slabs

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

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

5.6 Site and Foundation Drainage

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

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

5.7 Underground Utilities

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

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

5.8 Pavements

We have calculated thicknesses for asphalt pavements in accordance with Caltrans procedures for flexible pavement design. Our calculations assume an R-value of 10 and a range of Traffic Indices from 4.0 to 7.0 depending on the expected traffic loads for a twenty-year design life. The

R-value should be confirmed with future laboratory testing. In general, areas expected to experience loading from heavy vehicles should be designed using a higher Traffic Index, while parking areas and other lightly loaded areas can utilize a thinner pavement section based on a lower Traffic Index. The recommended pavement sections are presented in Table 7.

Table 7 – Preliminary Asphalt-Concrete Pavement Sections

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

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

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

Permeable pavements or pavers may be utilized in the project design to enhance the on-site infiltration of surface water runoff. More detailed recommendations for permeable pavements would be provided in a design level geotechnical report.

6.0 SUPPLEMENTAL GEOTECHNICAL SERVICES

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

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

7.0 LIMITATIONS

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

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8.0 LIST OF REFERENCES

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

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

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

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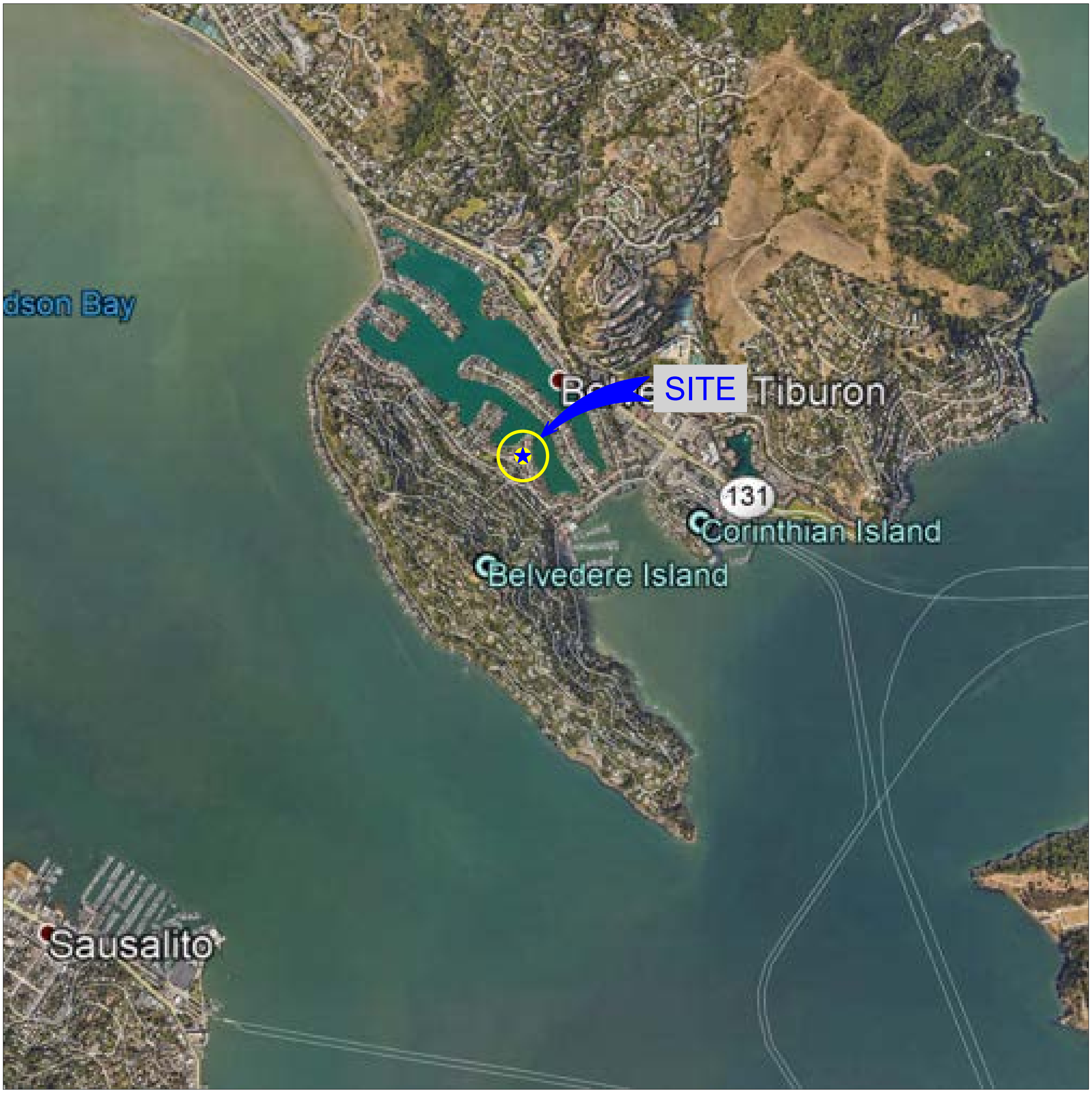
United States Geological Survey (2013), “The Uniform California Earthquake Rupture Forecast, Version 3 (UCERF3) – The Time Independent Model”, USGS Open-File Report 2013-1165, CGS Special Report 228, Southern California Earthquake Center Publication 1792.

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



SITE: LATITUDE, 37.8748°
LONGITUDE, -122.4651°

SITE LOCATION

N.T.S.

NORTH

REFERENCE: Google Earth, 2021

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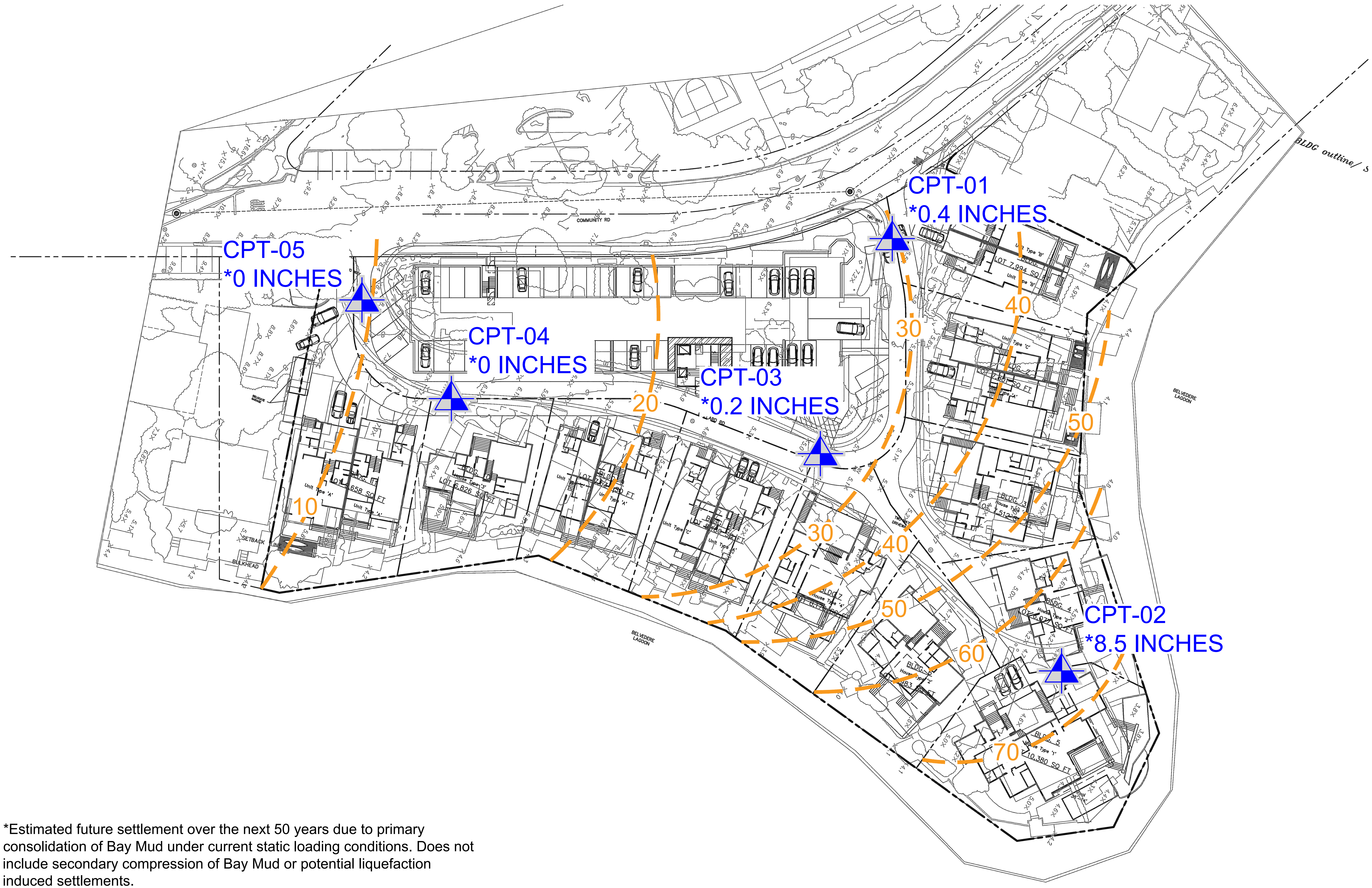
SITE LOCATION MAP

Mallard Pointe
Belvedere, California

Project No. 3068.001 Date: 5/18/2021

Drawn MMT
Checked

1
FIGURE



*Estimated future settlement over the next 50 years due to primary consolidation of Bay Mud under current static loading conditions. Does not include secondary compression of Bay Mud or potential liquefaction induced settlements.

Approximate location of CPT completed by MPEG, 2021

Approximate Bay Mud thickness contours

REFERENCE: BKF, "Mallard Pointe Site Plan" 6/9/21.

SCALE

0 30 60 120 FEET

NORTH

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SITE PLAN

Mallard Pointe
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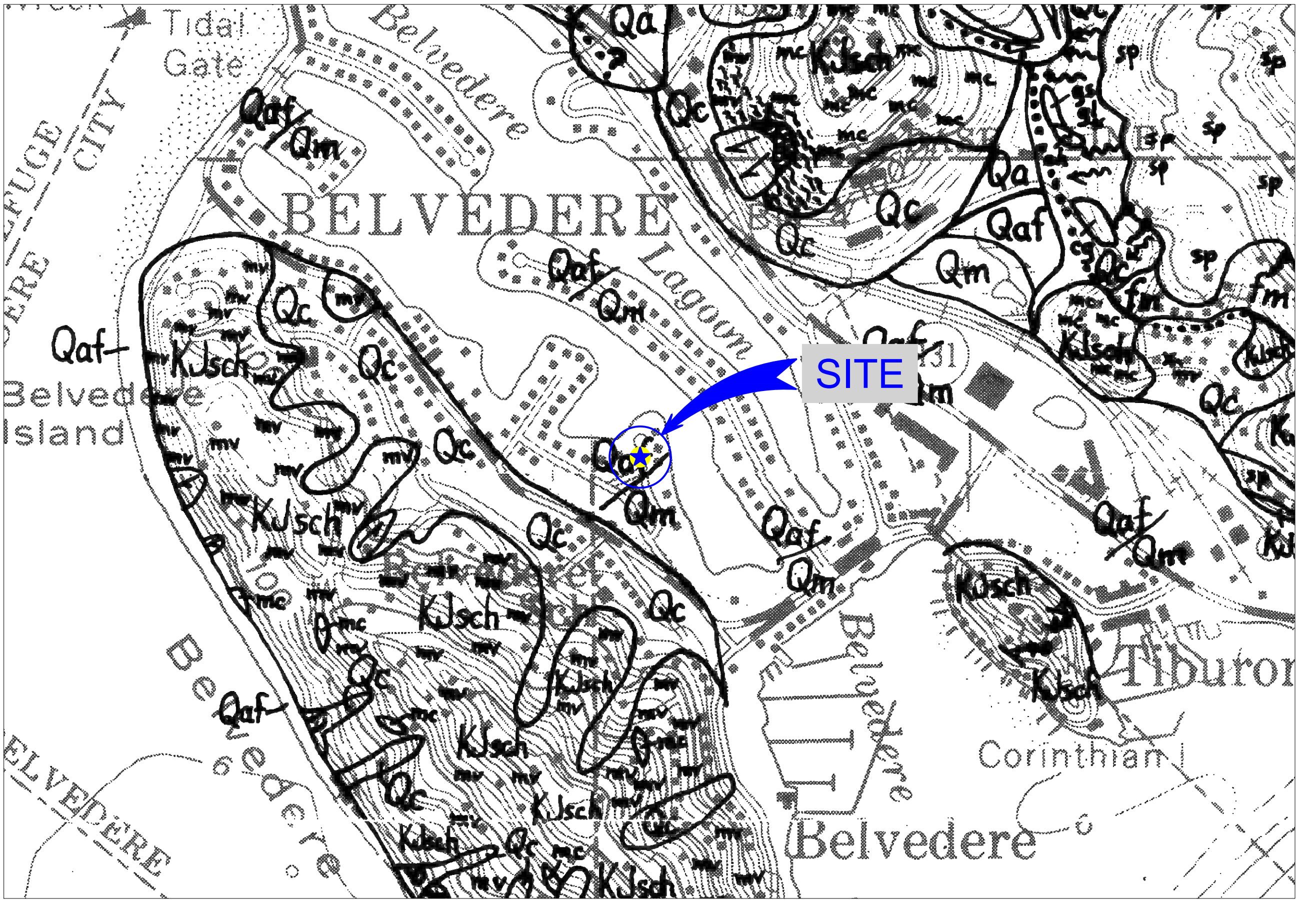
Project No. 3068.001 Date: 5/18/2021

Designed DSC
Drawn MMT
Checked MMT

2
FIGURE



REGIONAL GEOLOGIC MAP

(NOT TO SCALE)

NORTH

LEGEND

Qaf Artificial Fill - Deposits of rock, soil, garbage and trash, or bay mud placed by man upon natural surfaces, mostly for engineering purposes. Highly variable from place to place as to composition, degree of compaction, etc.

Qm Bay Mud - Marshlands, former marshlands, and mudflats bordering San Francisco and San Pablo Bays. Mostly at or below mean sea level; these are thick deposits of unconsolidated, low-density, semi-fluid, highly compressible, highly impermeable silty clay.

Qc Colluvium - Unsorted sands, silts, clays and weathered rock fragments accumulated on or at the base of slopes by natural gravitational or slope wash processes

KJsch Semi-schist, Phyllite, and Schist - Predominately slightly to well foliated or lineated metamorphosed sedimentary and volcanic rocks.

REFERENCE: Rice, S.J., and Smith, T.C., "Geology of the Tiburon Peninsula, Sausalito, and Adjacent Areas" *in* Geology for Planning in Central and Southeastern Marin County, California, California Department of Conservation, Division of Mines and Geology, Map Scale 1:12,000

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REGIONAL GEOLOGIC MAP

Mallard Pointe
Belvedere, California

Project No. 3068.001 Date: 5/18/2021

Drawn MMT
Checked

3
FIGURE

SITE COORDINATES
LAT. 37.8748°
LON. -122.4651°

SCALE
0 12.5 25 50 MILES

NORTH

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

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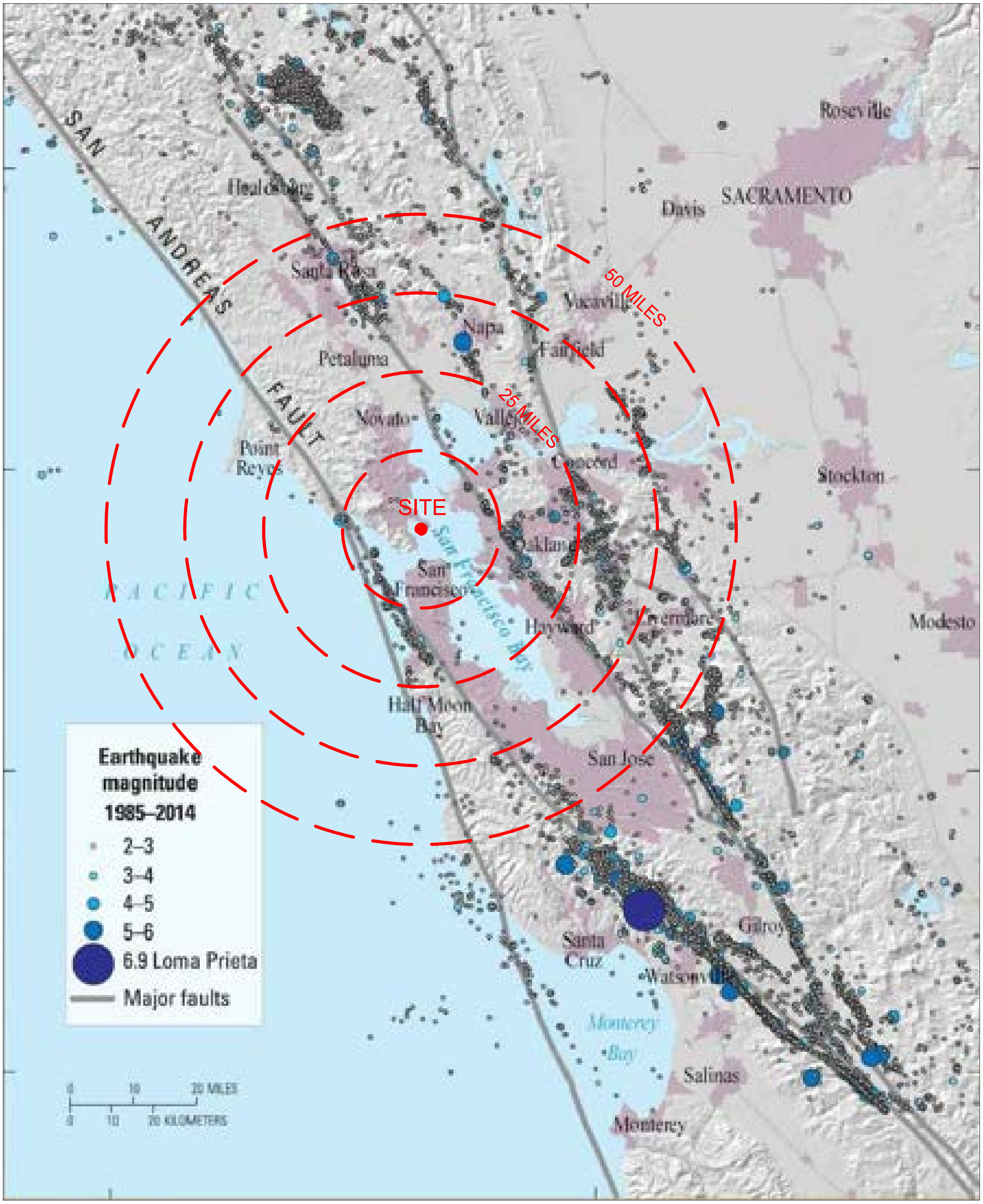
A CALIFORNIA CORPORATION, © 2018, ALL RIGHTS RESERVED FILENAME: 3068.001 Figures.dwg		Mallard Pointe Belvedere, California Project No. 3068.001 Date: 5/18/2021	Drawn MMT Checked	4 FIGURE



SITE COORDINATES
LAT. 37.8748°
LON. -122.4651°

SCALE
0 12.5 25 50 MILES

NORTH

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

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HISTORIC EARTHQUAKE ACTIVITY

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Belvedere, California

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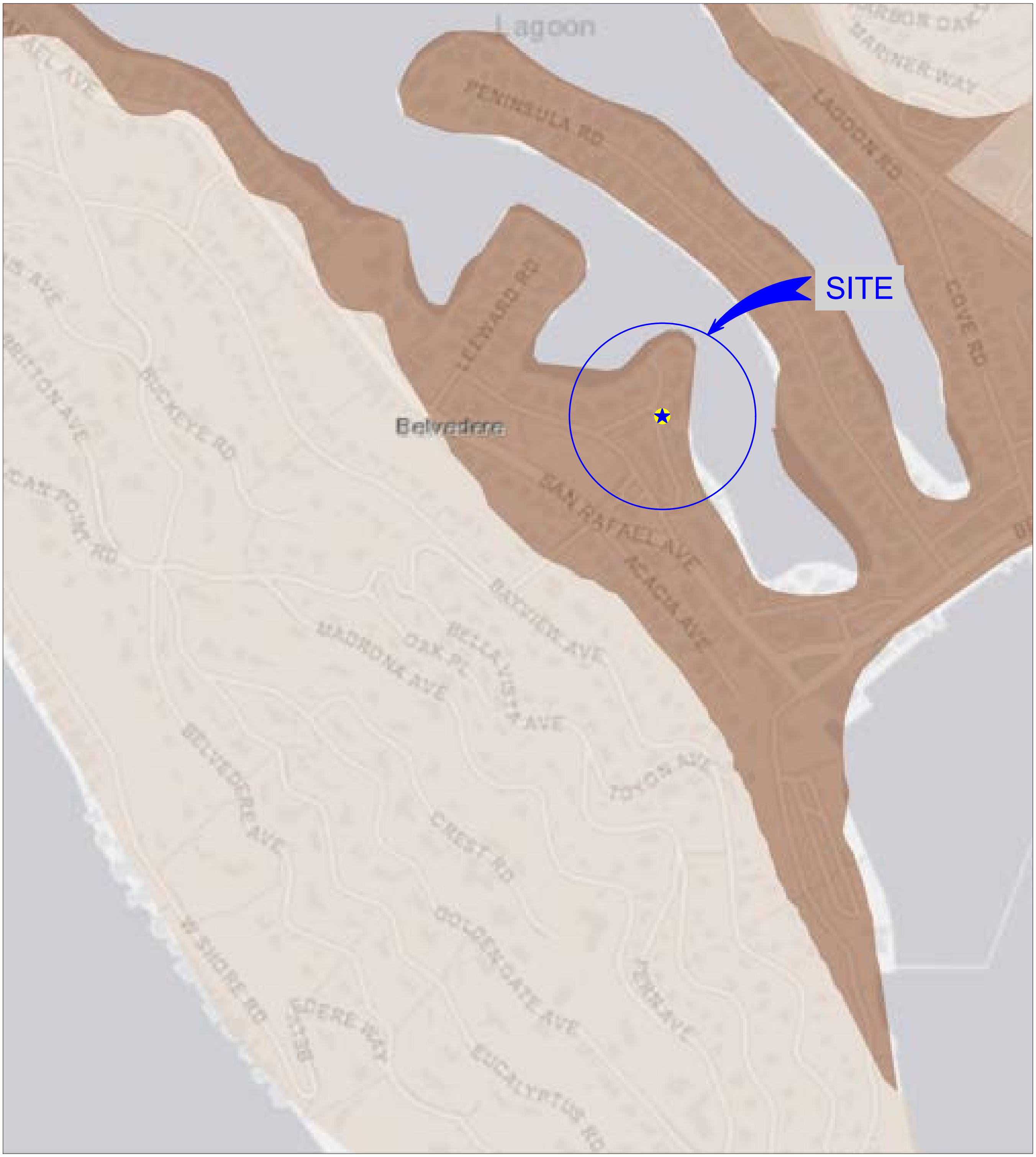
Checked

5
FIGURE



Susceptibility Level:

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

No Scale

NORTH

Reference: ABAG Hazard Map Viewer.

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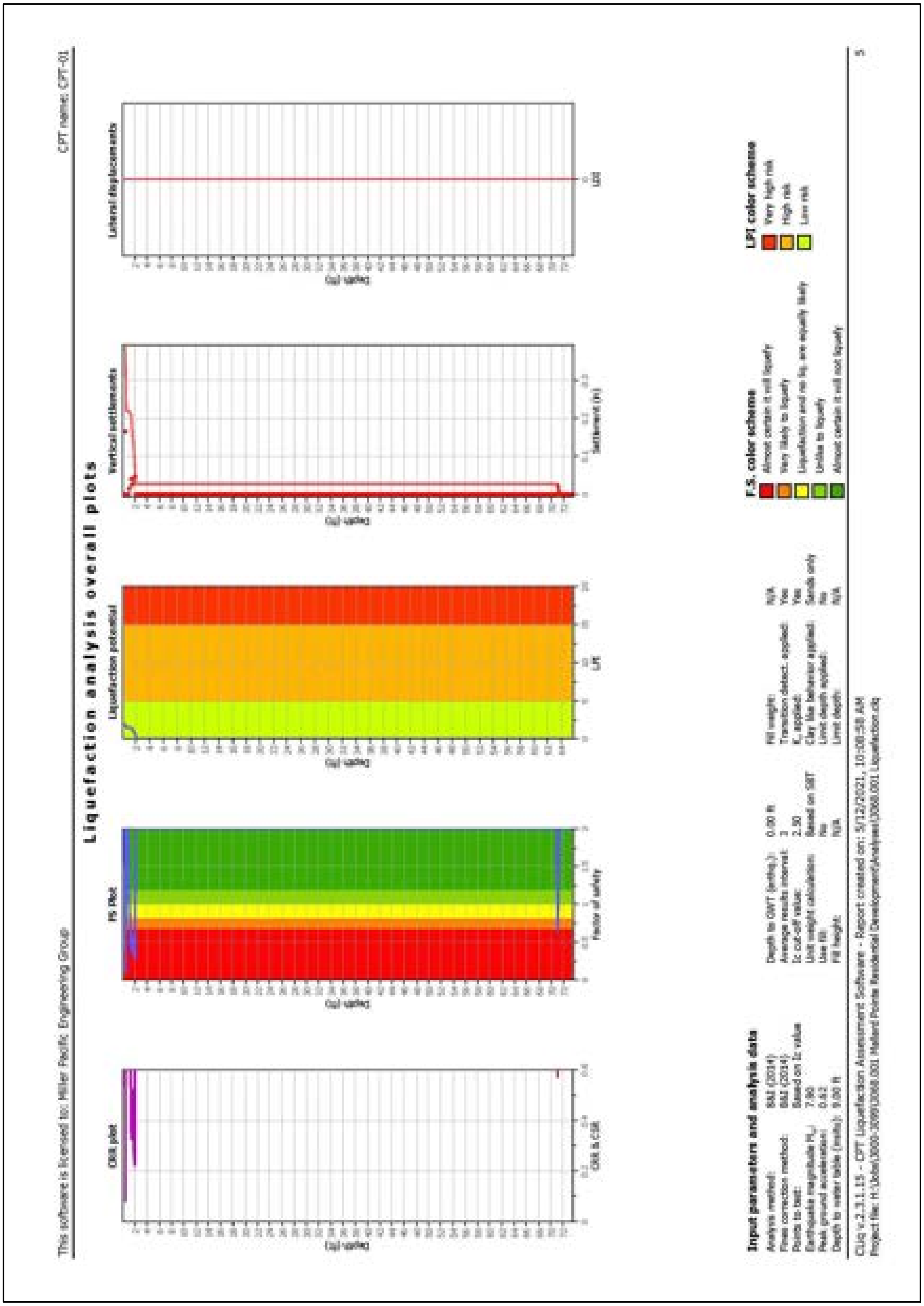

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CPT-01 LIQUEFACTION ANALYSIS

Mallard Pointe
Belvedere, California

Project No. 3068.001 Date: 5/18/2021

Drawn MMT
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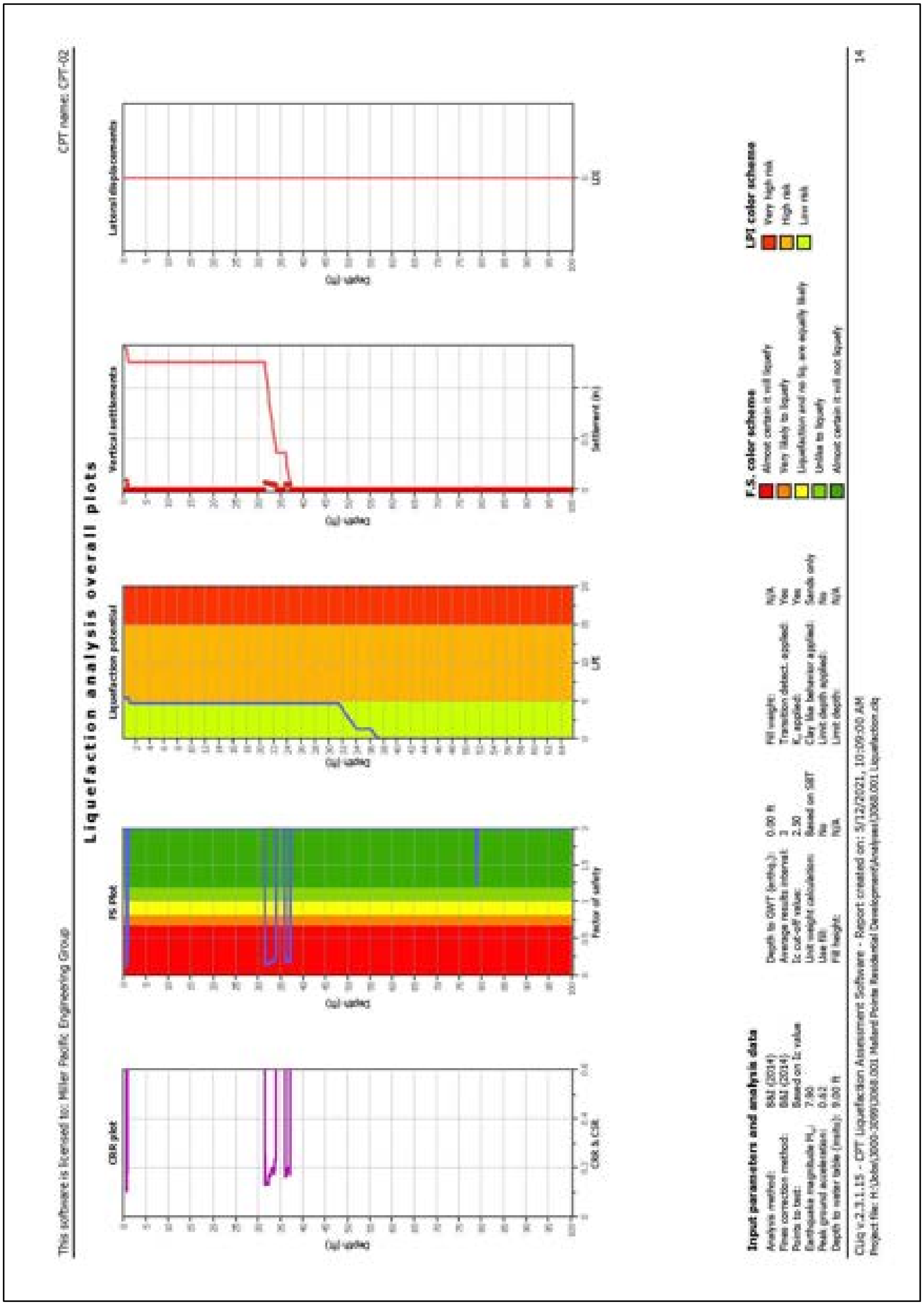
7
FIGURE



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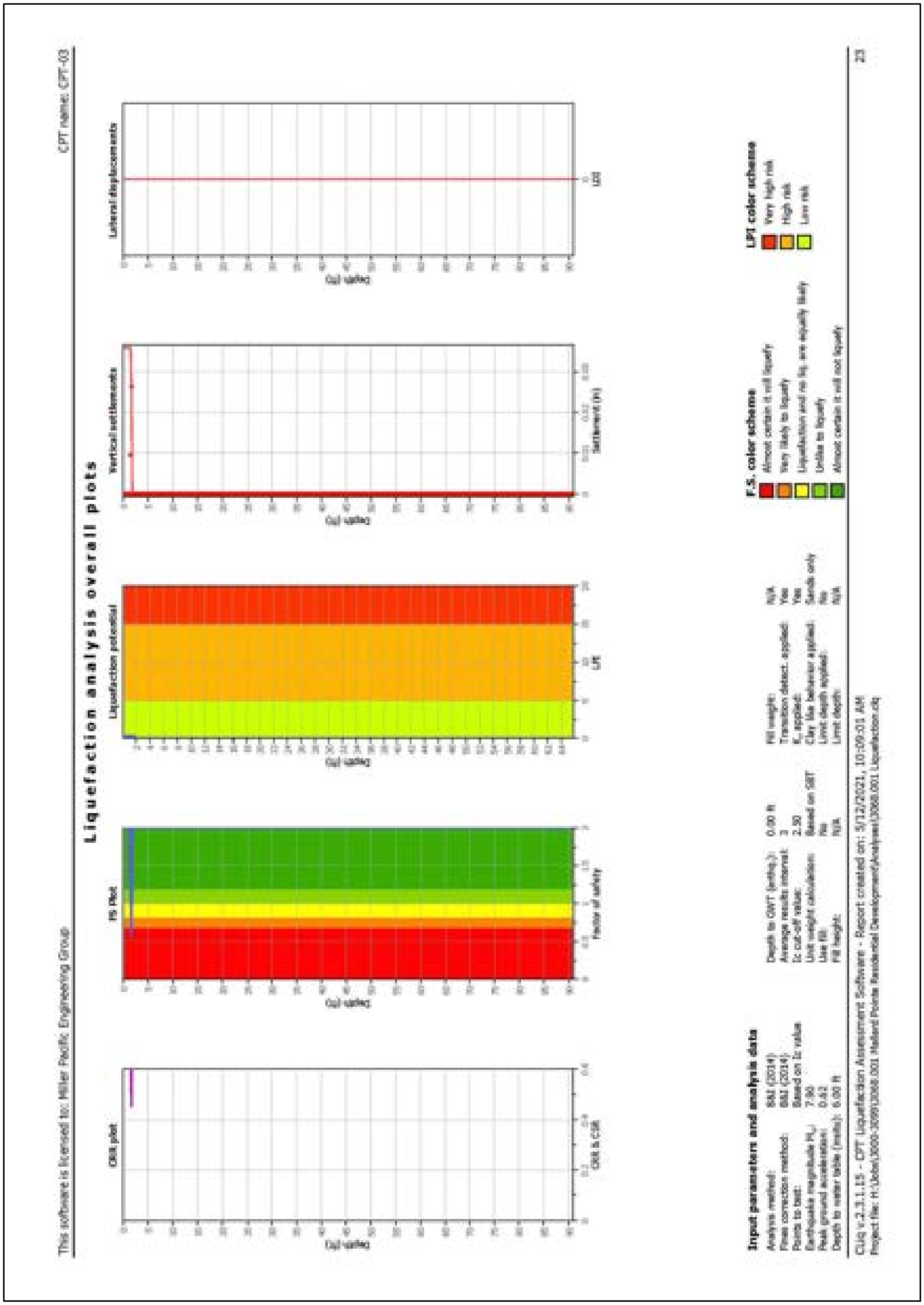
FILENAME: 3068.001 Figures.dwg	www.millerpac.com	Project No. 3068.001 Date: 5/18/2021	FIGURE



CPT-03 LIQUEFACTION ANALYSIS

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Project No. 3068.001 Date: 5/18/2021

Drawn MMT
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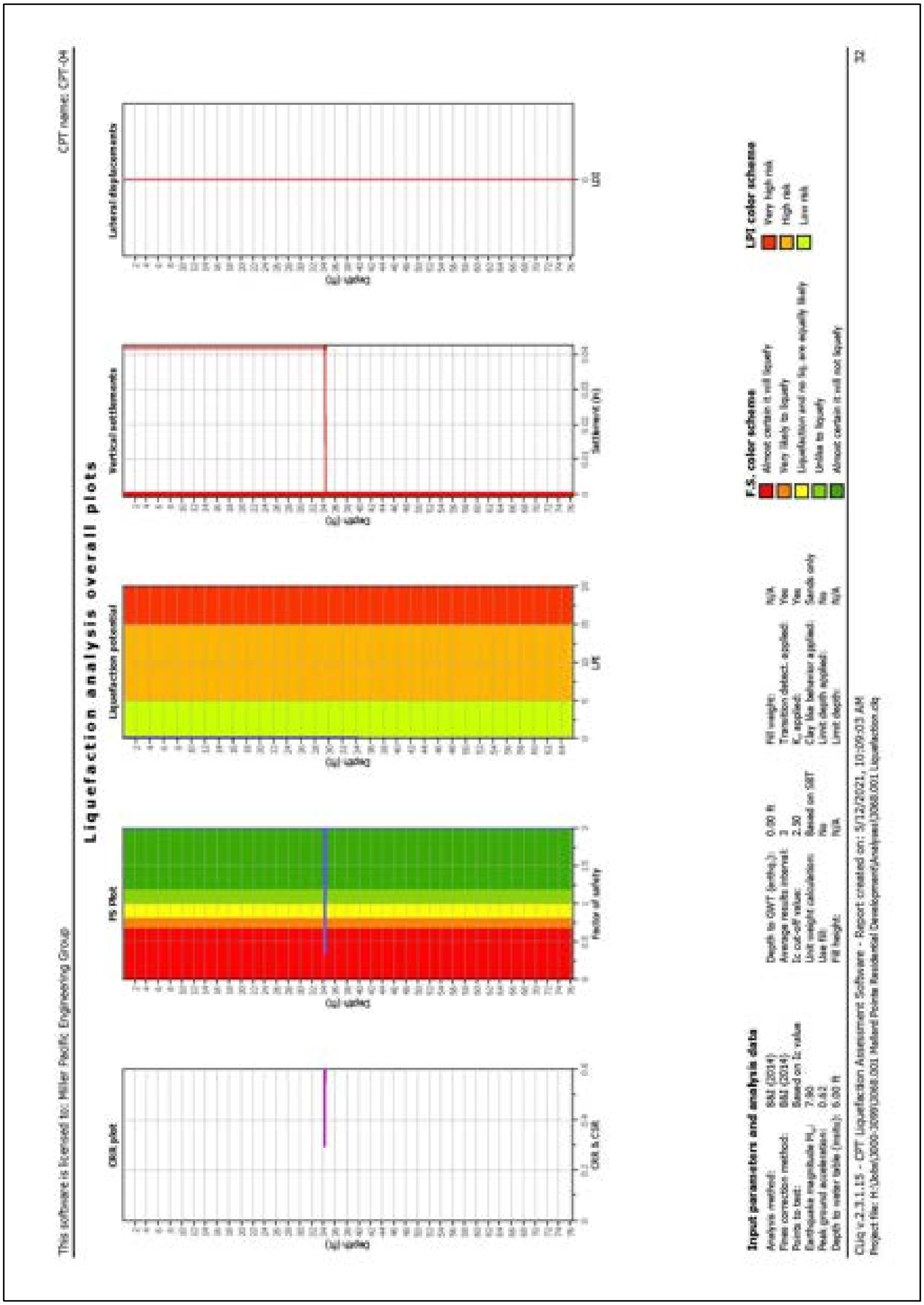
9
FIGURE



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CPT-04 LIQUEFACTION ANALYSIS

Mallard Pointe
Belvedere, California

Project No. 3068.001

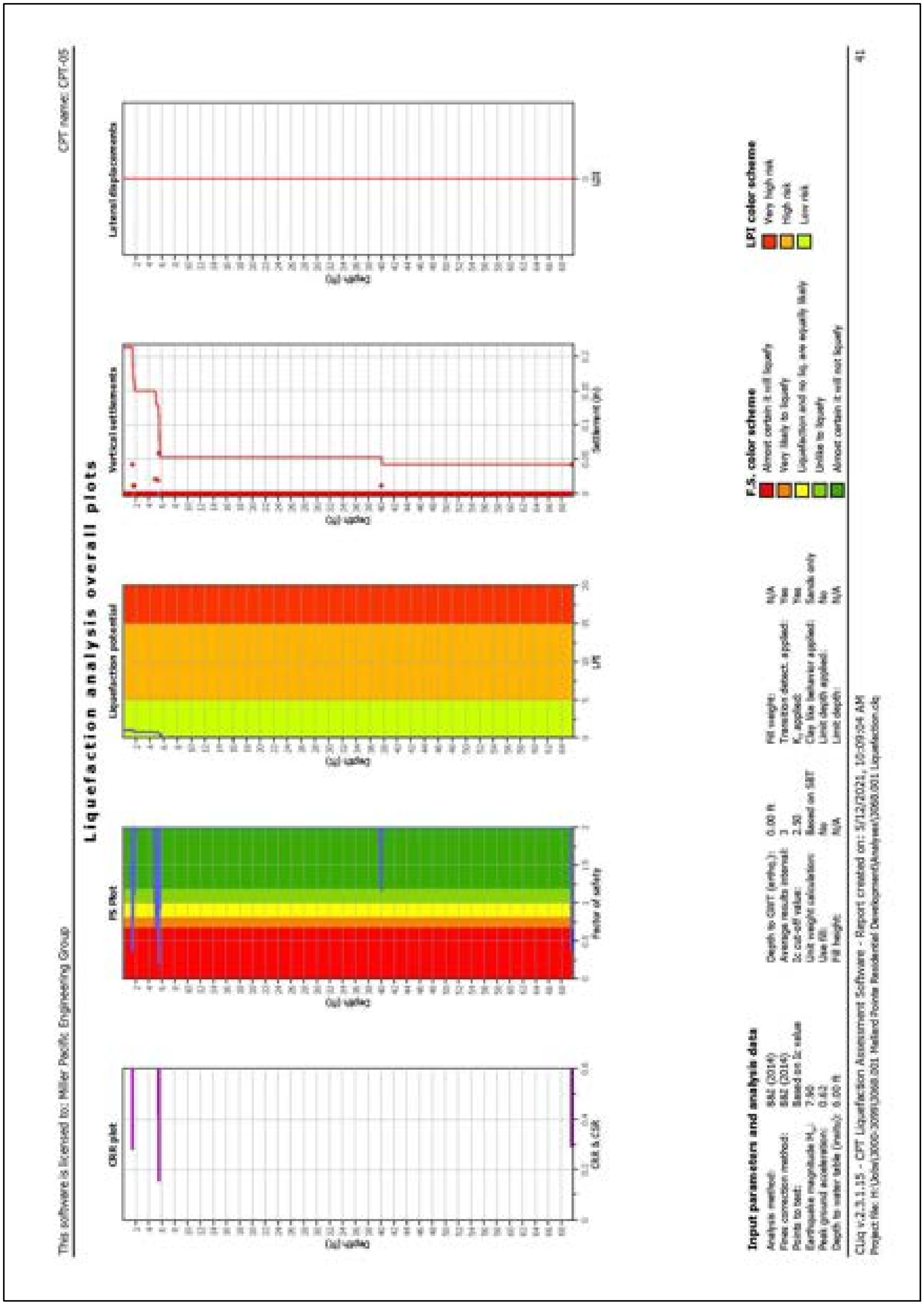
Date: 5/18/2021

Drawn MMT

Checked

10

FIGURE



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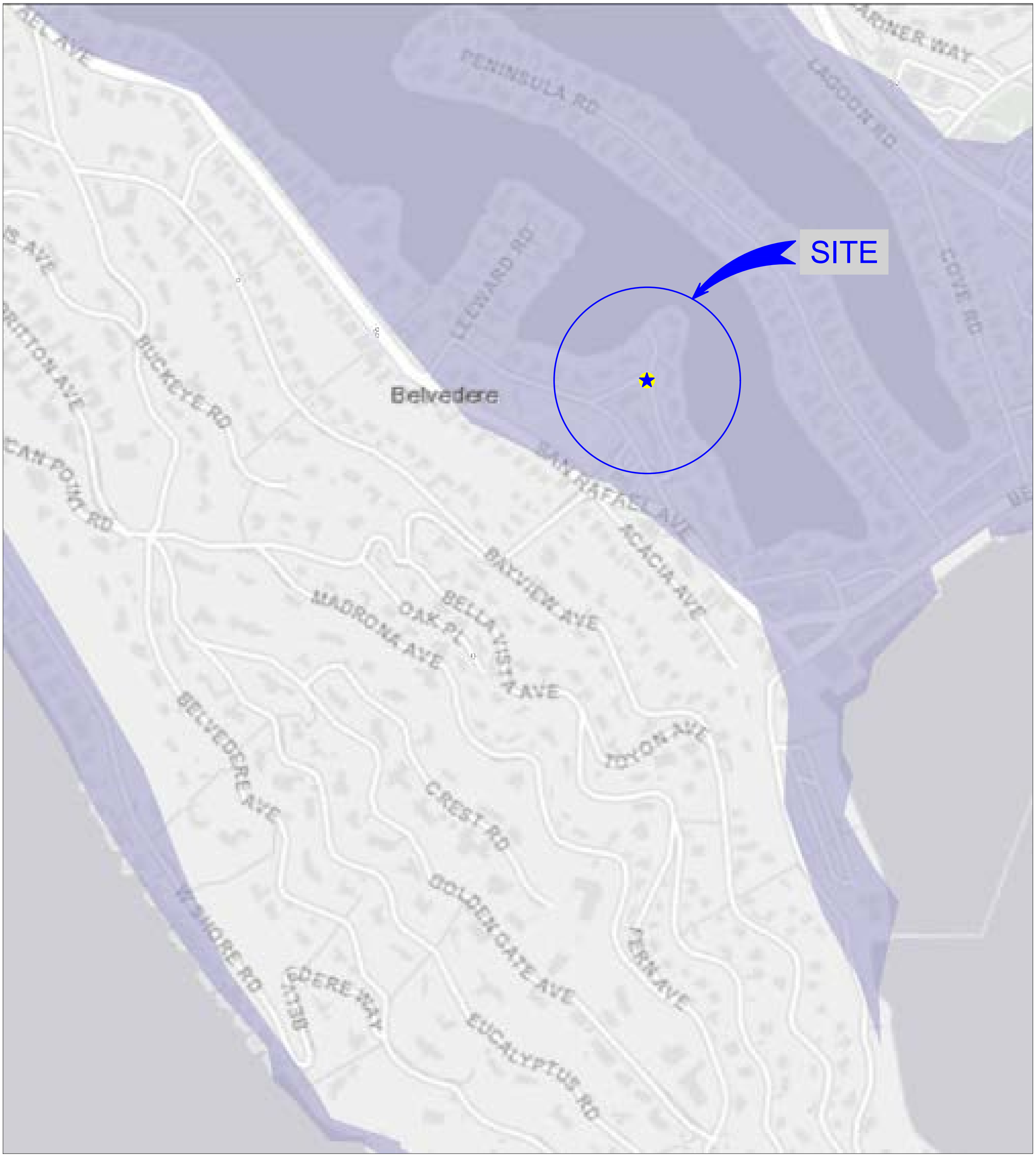
A CALIFORNIA CORPORATION, © 2018, ALL RIGHTS RESERVED	F 415 / 382-3450	11 FIGURE
FILENAME: 3068.001 Figures.dwg	www.millerpac.com	Project No. 3068.001 Date: 5/18/2021



Legend:

Tsunami Evacuation Zone

No Scale

NORTH

Reference: ABAG Hazard Map Viewer.

MPEG MILLER PACIFIC ENGINEERING GROUP

FILENAME: 3068.001 Figures.dwg

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TSUNAMI INUNDATION MAP

Mallard Pointe
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Project No. 3068.001 Date: 5/18/2021

Drawn MMT
Checked

12
FIGURE

FEMA Flood Hazard:

100-Year

500-Year

No Scale

NORTH

Reference: ABAG Hazard Map Viewer.

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FILENAME: 3068.001 Figures.dwg

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APPENDIX A
SUBSURFACE EXPLORATION AND LABORATORY TESTING

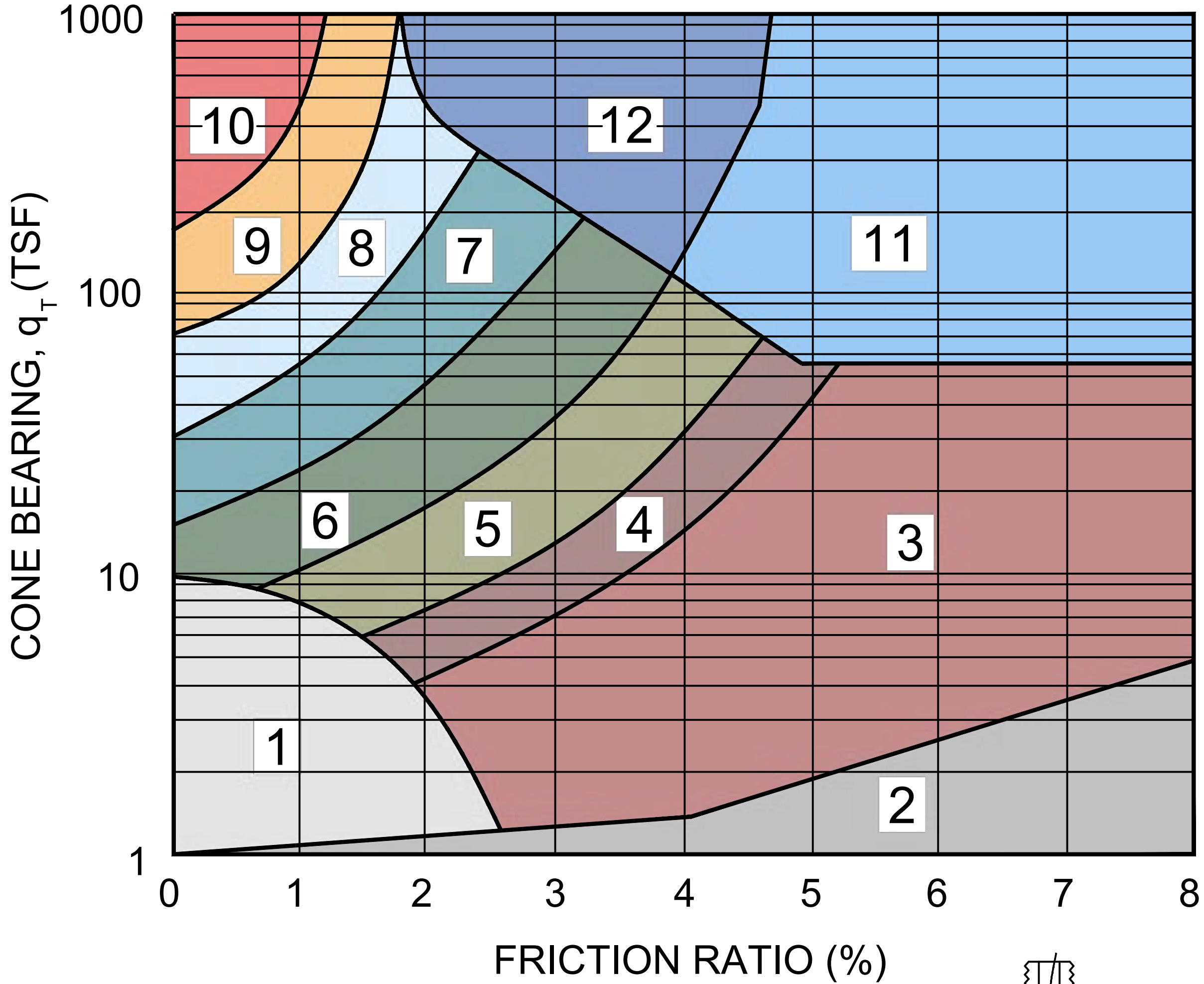
A. SUBSURFACE EXPLORATION

We performed five Cone Penetration Tests (CPT) on April 26, 2021 at the locations shown on the Site Plan, Figure 2. The CPT is a special exploration technique that provides a continuous profile of data throughout the depth of exploration. It is particularly useful in defining stratigraphy, relative soil strength and in assessing liquefaction potential.

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

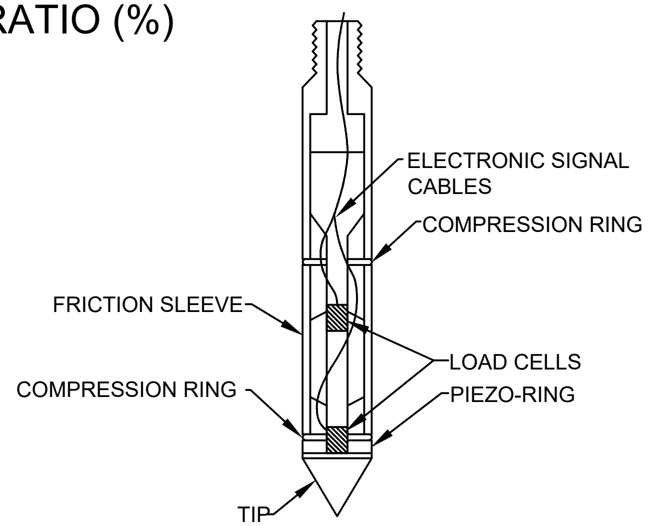
The recorded data is transferred to an in-office computer for reduction and analysis. The analysis of cone bearing and sleeve friction (i.e., friction ratio) indicates the soil type, the cone bearing alone indicates soil density or strength, and the pore pressure indicates the presence of clay. Variations in the data profile indicate changes in stratigraphy. This test method has been standardized and is described in detail by the ASTM Standard Test Method D3441 "Deep, Quasi-Static Cone and Friction Cone Penetration Tests of Soil." The interpretation of CPT data is illustrated on Figure A-1, and the CPT data logs are presented on Figures A-2 through A-6.

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



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

(*) Overconsolidated or Cemented



CONE PENETROMETER

(NO SCALE)

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

MPEG MILLER PACIFIC ENGINEERING GROUP

FILENAME: 3068.001 CPTs.dwg

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CPT SOIL INTERPRETATION CHART

Mallard Pointe
Belvedere, California

Project No. 3068.001 Date: 5/7/2021

Drawn MMT
Checked

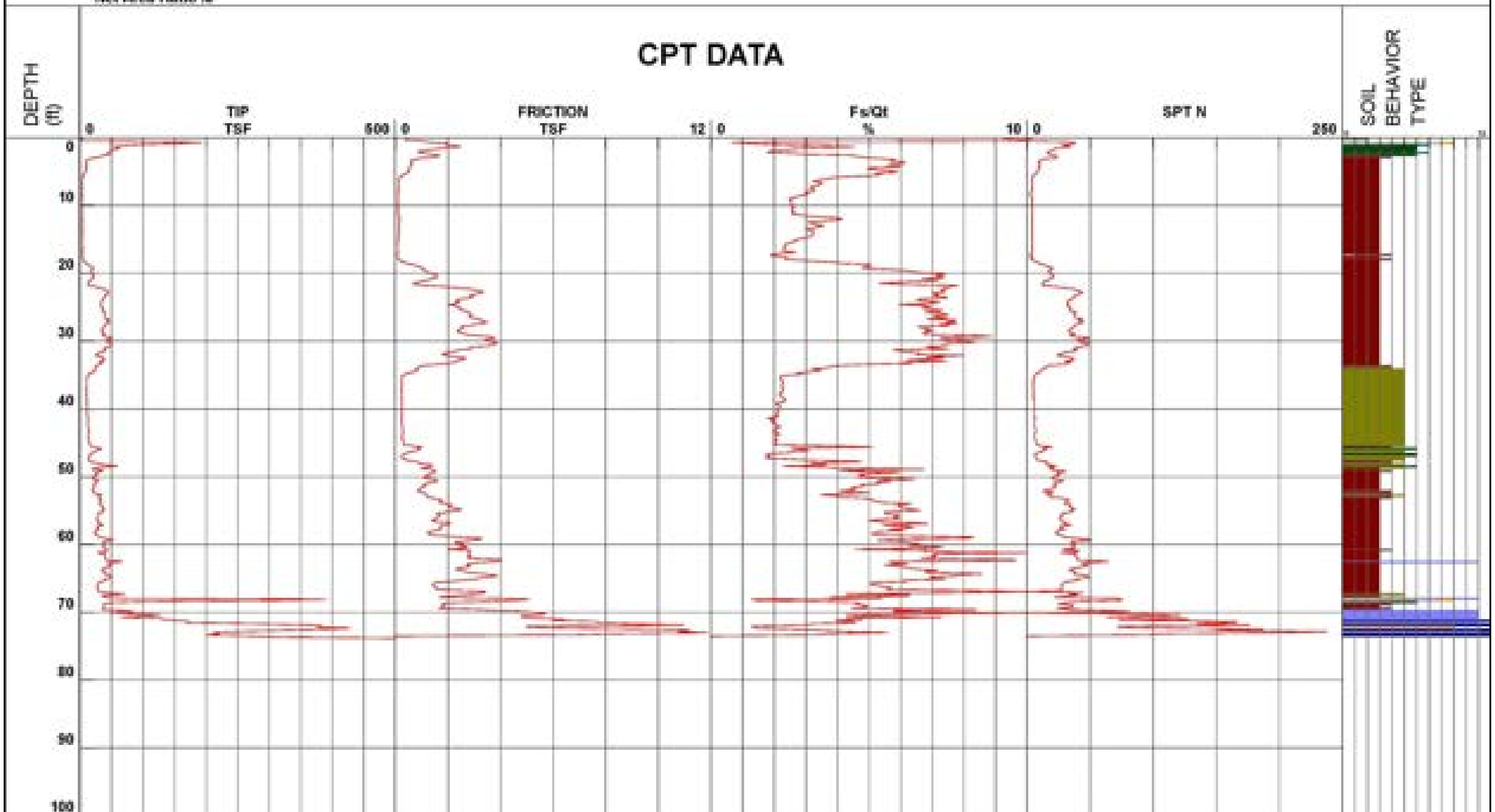
A-1
FIGURE

Miller Pacific Engineering

Project	Mallard Point	Operator	AJ-OO	Filename	SDF(525).cpt
Job Number	3068.001	Cone Number	DDG1587	GPS	
Hole Number	CPT-01	Date and Time	4/26/2021 8:52:39 AM	Maximum Depth	73.82 ft
EST GW Depth During Test			9.00 ft		

Net Area Ratio .8

CPT DATA



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

Cone Size 15cm squared

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

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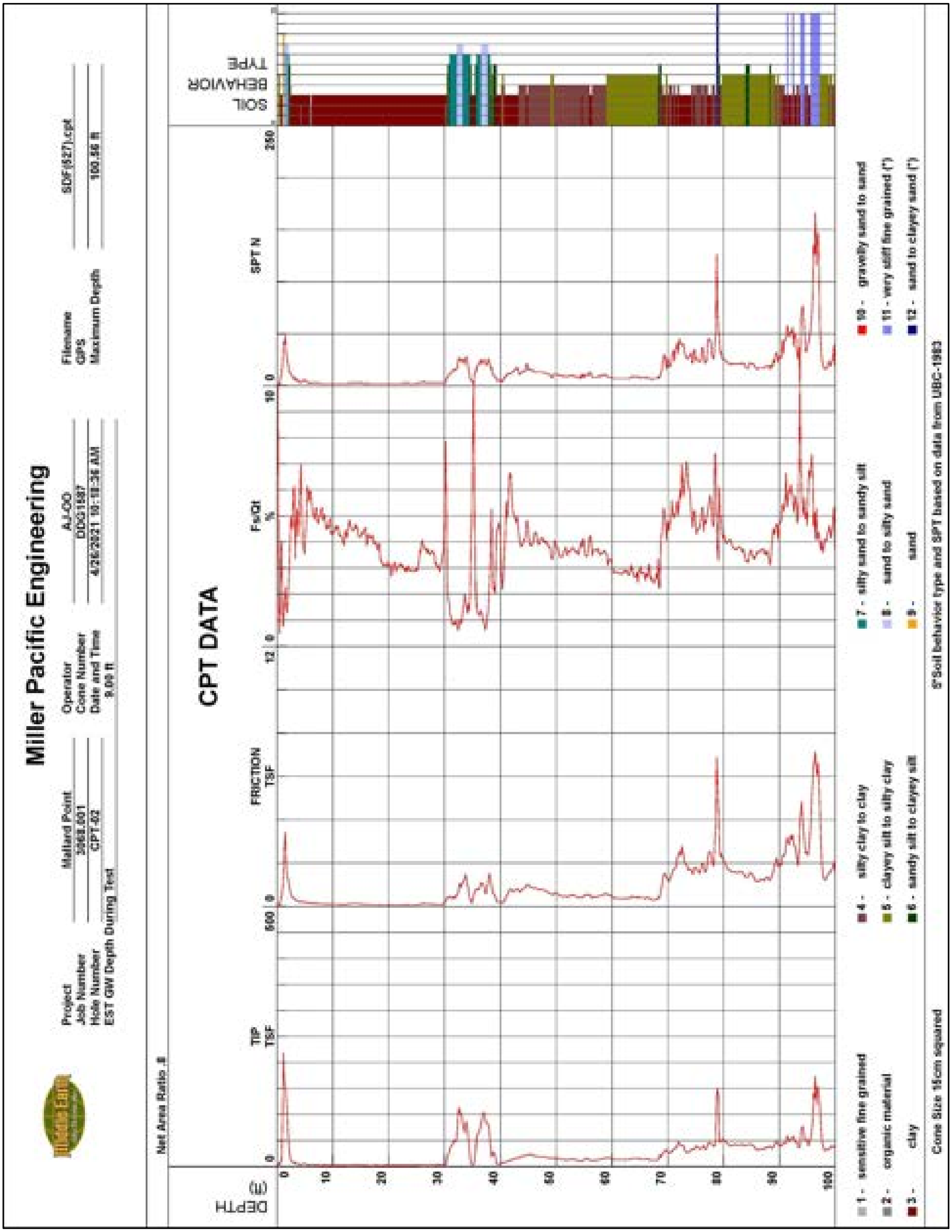
CPT-01 DATA

Mallard Pointe
Belvedere, California

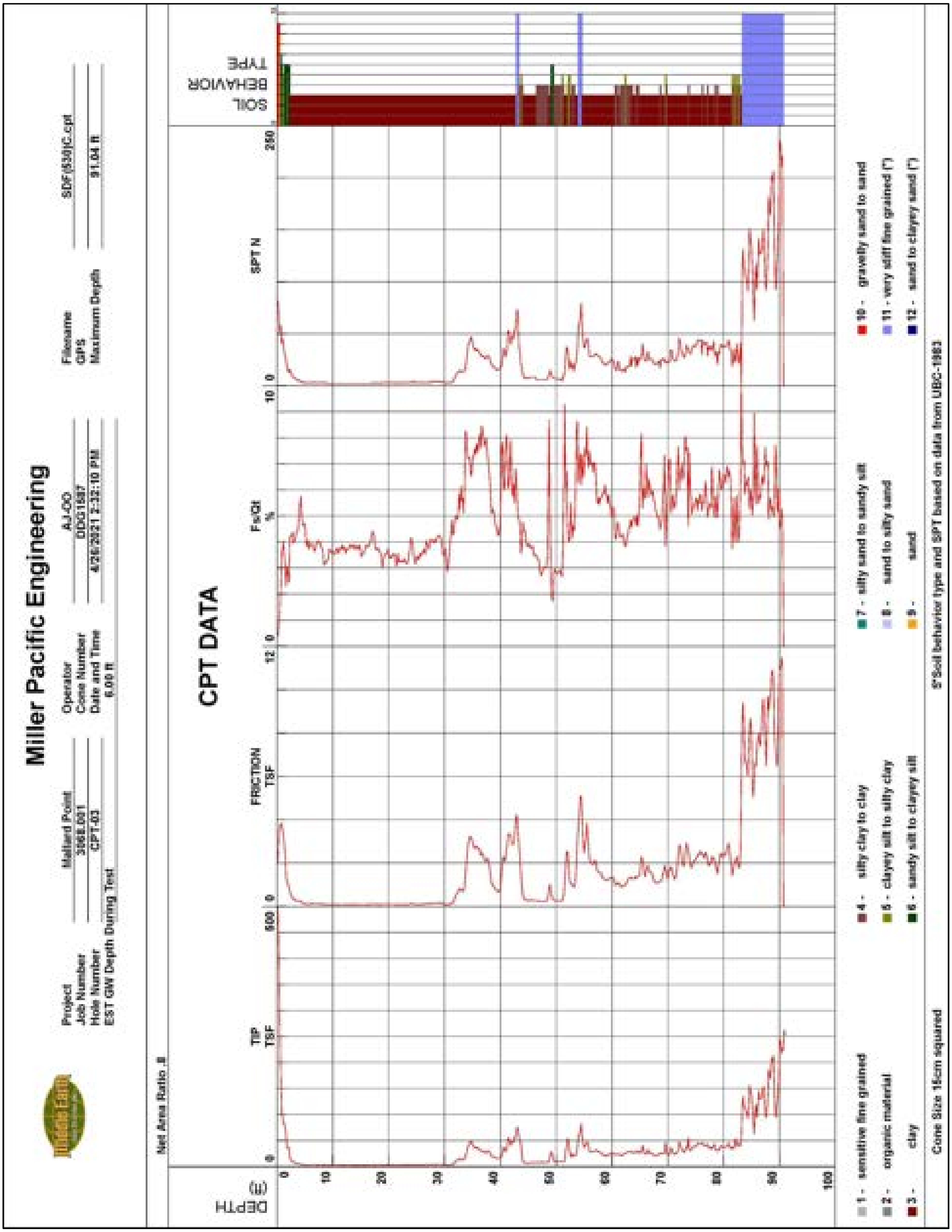
Project No. 3068.001 Date: 5/7/2021

Drawn MMT
Checked

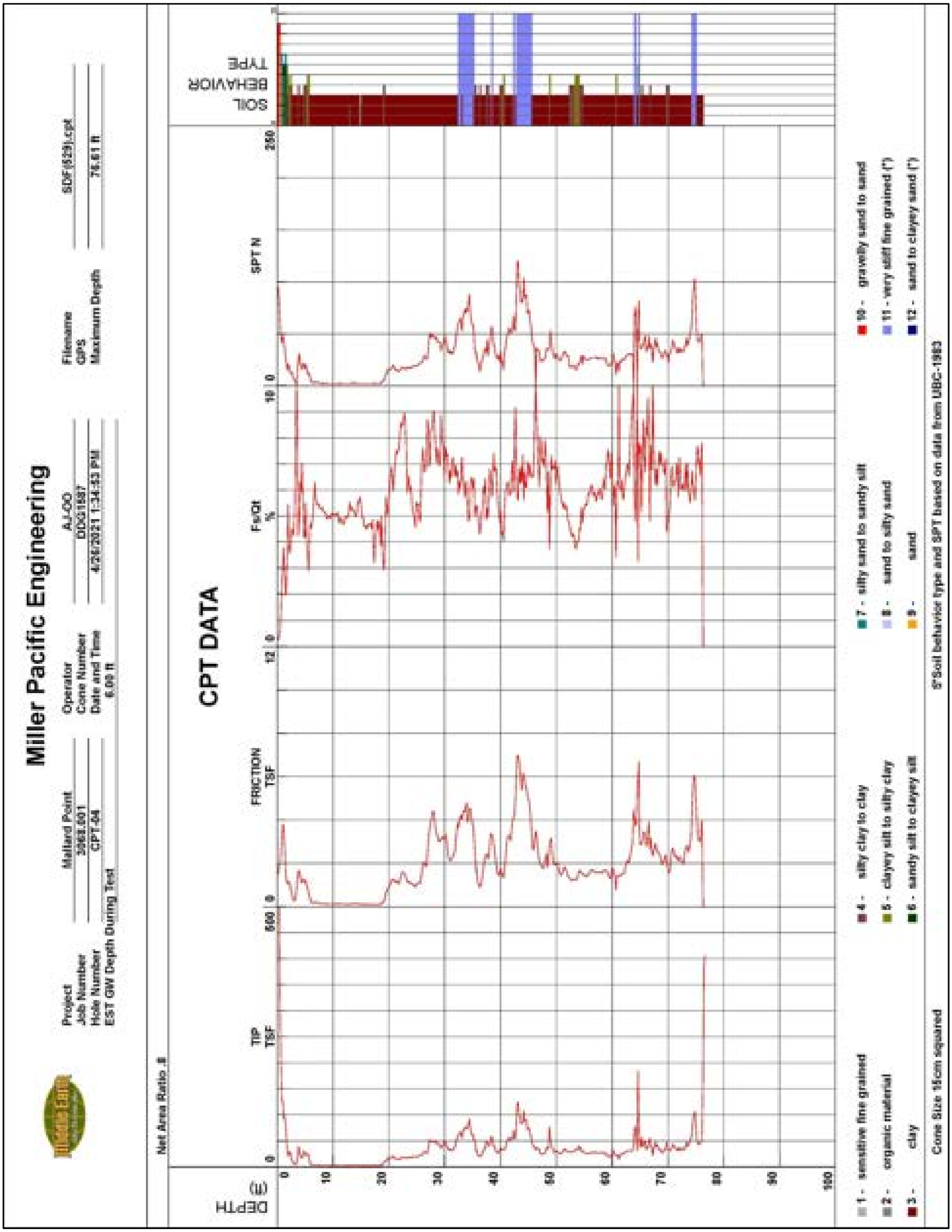
A-2
FIGURE



MILLER PACIFIC ENGINEERING GROUP	504 Redwood Blvd. Suite 220 Novato, CA 94947 T 415 / 382-3444 F 415 / 382-3450 www.millerpac.com	CPT-02 DATA
 FILENAME: 3068.001 CPTs.dwg		Mallard Pointe Belvedere, California Project No. 3068.001 Date: 5/7/2021 Drawn MMT Checked **A-3** FIGURE



MILLER PACIFIC ENGINEERING GROUP	504 Redwood Blvd.	CPT-03 DATA	
	Suite 220		
	Novato, CA 94947	Mallard Pointe Belvedere, California	Drawn MMT Checked
	T 415 / 382-3444		A-4 FIGURE
	F 415 / 382-3450		
 FILENAME: 3068.001 CPTs.dwg	www.millerpac.com	Project No. 3068.001 Date: 5/7/2021	



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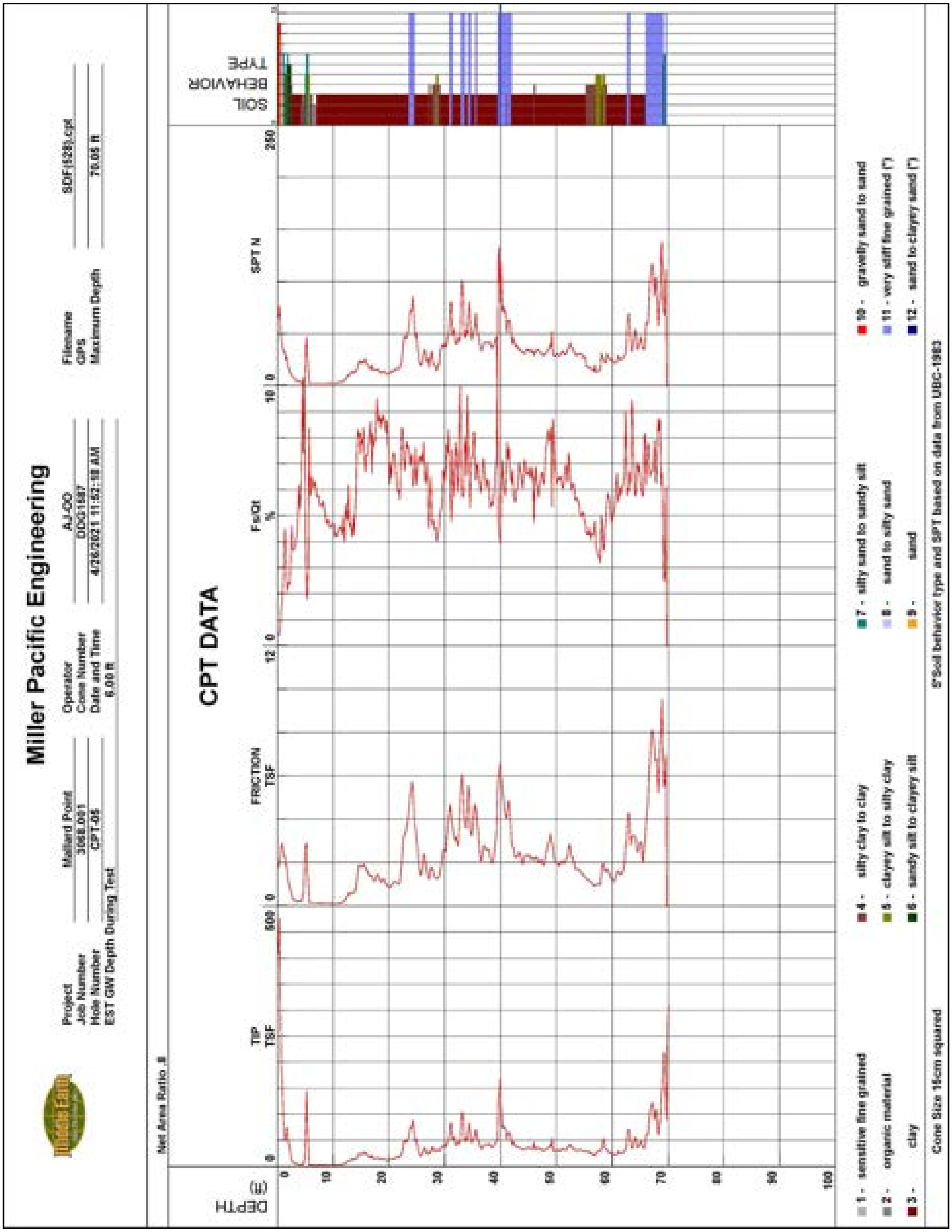
CPT-04 DATA

Mallard Pointe
Belvedere, California

Project No. 3068.001 Date: 5/7/2021

Drawn MMT
Checked

A-5
FIGURE



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FILENAME: 3068.001 CPTs.dwg

CPT-05 DATA

Mallard Pointe
Belvedere, California

Project No. 3068.001 Date: 5/7/2021

Drawn MMT
Checked

A-6
FIGURE

GeoTechnical Memorandum 01

To:	Thompson/Dorfman Urban	Project:	Mallard Pointe Bulkheads
From:	Scott Stephens, GE	cc:	
Date:	October 6, 2020	Job No.:	3068.001
Subject:	Geotechnical Evaluation of Existing Bulkheads		

This memorandum summarizes Miller Pacific Engineering Group's (MPEG) geotechnical inspection and evaluation regarding replacement of existing, old timber bulkheads around the Mallard Pointe residential development in Belvedere, CA. Our services are being provide per our scope and agreement dated September 17, 2020. The project area is the shoreline from 1 to 16 Mallard Road. The existing bulkheads separate fill materials placed to create rear yards and the Belvedere Lagoon.

Regional Geology

The site is located within the Coast Range Geomorphic Province of California. The regional bedrock geology consists of complexly folded, faulted, sheared, and altered sedimentary, igneous, and metamorphic rock of the Jurassic-Cretaceous age (65-190 million years ago) Franciscan Complex. For the last 15,000 years, the sea level has continually risen (due to melting of glaciers from the Wisconsin glaciation) and flooded the lower topography. For the last 8,000 years, silt and clay particles carried in suspension in floodwater, have been deposited in the San Francisco Bay to form the soft and highly compressible "Bay Mud." This process continues today. Regional geologic mapping by the California Division of Mines and Geology (CDMG) indicates the site consists of fill over Bay Mud. Based on the topography and geology maps, the Bay Mud is expected to start at San Rafael Ave and thicken towards the north. The depth and thickness of the Bay Mud under the project site is currently unknown.

Historic Aerial Photographs

We reviewed historic aerial photographs available from Pacific Aerial Surveys of Oakland, California to obtain information about the site history. Most of the development in the project area occurred in the late 1950's and early 1960's, with lesser levels of construction and other activity in the years before and after that period. The results of our review are outlined below.

<u>September 6, 1946 (AV-09-04-04)</u>
The current alignment of San Rafael Avenue is in place. There is some clearing and rough grading occurring in the residential development area. Belvedere Lagoon has not been dredged yet.

<u>November 8, 1950 (AV-41-06-31)</u>
Grading and fill placement appear mostly complete in the project area. Belvedere Lagoon has been dredged and is full of water.

<u>June 20, 1961 (AV-432-34-01)</u>
Grading is complete, subdivision improvements are in place and the homes have been constructed.

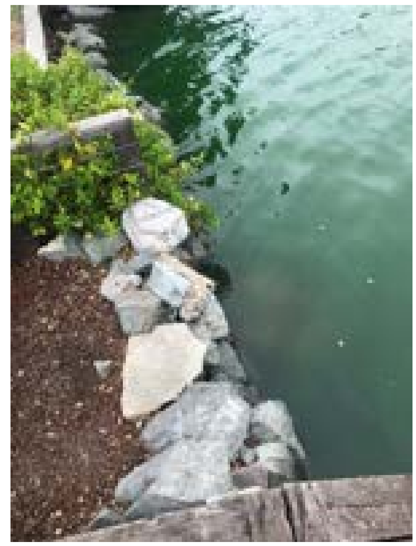
<u>July 2, 1970 (AV-957-08-12) and thereafter</u>
No significant changes.

Existing Bulkhead Conditions

We performed a site reconnaissance on August 27, 2020 to observe and evaluate the existing bulkhead conditions along the Belvedere Lagoon frontage of the subject properties. The initial intent of the inspection was to evaluate a global repair plan for the old, 2 to 3 feet high, wooden bulkheads using a new retaining structure on the water side of the bulkheads. Upon inspection, it became apparent that landscape improvements and site conditions in the rear yards were variable, and a majority of the old wood bulkheads had either been partially buried and supported by rip-rap, or in some cases, wooden bulkheads were removed and replaced entirely with rip rap. Examples of existing conditions are shown below.

1-3 Mallard Drive

6-8 Mallard Drive



11-12 Mallard Drive

It was also noted during our site inspection that generally adverse surface drainage conditions exists in most yards and posts that support decks in the lagoon are in poor condition.

Evaluation and Conclusions

Based on the existing conditions, it does not appear that a global repair is warranted or needed for the existing timber bulkheads. Considering the exposed height of the timber bulkheads is typically less than 12 inches, we recommend repair of the exposed bulkheads on an as-needed basis. The repairs should be made on a case-by-case basis as exposed wood rots and fails. Since repairs are expected be less than 3 feet high, engineering design is not required. The repairs should consider wave action, and thus be erosion resistant. For future repairs, we envision demolition and removal of the exposed wood, and then replacement with additional rip-rap, interlocking modular block walls or other forms of landscape improvements.

We hope this provides you with the information you require at this time. Please do not hesitate to contact us with any questions or concerns.

MILLER PACIFIC ENGINEERING GROUP

Scott Stephens
Geotechnical Engineer No. 2398
(Expires 6/30/21)