MILLER PACIFIC ENGINEERING GROUP

UPDATED GEOTECHNICAL INVESTIGATION
SAUSALITO MARIN CITY SANITARY DISTRICT
COLOMA PUMP STATION
SAUSALITO, CALIFORNIA

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Job No. 2213.004

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CERTIFICATION

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1.0 INTRODUCTION

This report presents the results of our updated Geotechnical Investigation for the Sausalito Marin City Sanitary District's (SMCSD) Coloma Pump Station Project in Sausalito, California. The project site is located just south of the intersection of Coloma Street and Bridgeway, as shown on the Site Location Map, Figure 1.

Our work was performed in accordance with our Agreement for Professional Services dated April 11, 2017. The purpose of our Geotechnical Investigation was to explore subsurface conditions and to develop geotechnical criteria for design and construction of the pump station improvements. The scope of our services includes:

- Review of readily available geotechnical and geologic reference materials, including the preliminary geotechnical evaluation prepared by DCM Consulting, Inc. (DCM, 2014).
- Exploration of subsurface conditions with two borings located within the general vicinity of the pump station.
- Geotechnical laboratory testing to estimate pertinent engineering properties of the soils encountered during our exploration.
- Environmental laboratory testing for volatile organic compounds, semi-volatile organic compounds, total petroleum hydrocarbons, CAM 17 metals and corrosivity.
- Evaluation of relevant geologic hazards including seismic shaking, settlement, liquefaction and other hazards.
- Preparing geotechnical recommendations and design criteria related to foundations, lateral pressures, temporary support of excavations, earthwork, trench backfill, new pavement sections, seismic design, and other geotechnical-related items.
- Preparation of this report which summarizes our subsurface exploration and laboratory testing programs, evaluation of relevant geologic hazards, and geotechnical recommendations and design criteria.

This report was completed under our Phase 2 services for the project. Subsequent phases of work may include geotechnical plan review and observation and testing of geotechnical-related work items during construction.

2.0 PROJECT DESCRIPTION

Based on our discussions with the project team and review of Preliminary Drawings (Carollo, 2019), we understand the planned improvements generally consist of constructing a new pump station within the City of Sausalito right-of-way just south of Coloma Street. The circular structure will be located largely within the footprint of the existing City of Sausalito's Whiskey Springs Pump Station and will occupy an approximately 16-foot-diameter footprint with an invert depth of about 26.5 feet below the existing ground surface. A new underground valve vault, electrical building and flow meter vault will be constructed south of the pump station with invert depths of about 7.5 feet, 13 feet and 7 feet below grade, respectively. The planned improvements also include constructing a new transformer and generator pad adjacent to the new pump station.

Other improvements are expected to include constructing new sewer pipelines and manholes which connect to the existing and proposed facilities. Temporary shoring will be required to facilitate construction of the new underground improvements. A new driveway will also be constructed along the western property boundary to provide access to the site from Coloma Street. The proposed improvements are shown on the 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 Jurassic-Cretaceous age (190- to 65-million years old) Franciscan Complex. Within Marin County, 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 locally overlie the basement rocks of the Franciscan Complex. 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.

The project site is located within lower-lying terrain near the western margins of the San Francisco Bay. Regional geologic mapping by the California Division of Mines and Geology (CDMG, 1976) indicates the site is underlain by artificial fill over Bay Mud and is located just north of a mapped boundary between Bay Mud and colluvial soils. Franciscan mélange is mapped near the base of the hillside located several hundred feet south of the site. A Regional Geologic Map and descriptions of the mapped geologic units are shown on Figure 3.

3.2 Seismicity

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

Faults are seldom single cracks in the earth's crust but are typically comprised 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.

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 (1998) has mapped various active and inactive faults in the region. These faults, defined as either California Building Code Source Type “A” or “B,” are shown in relation to the project site on the attached Active Fault Map, Figure 4. The nearest known active faults to the site are the San Andreas and San Gregorio Faults located approximately 10.1 kilometers (6.3 miles) and 13.6 kilometers (8.5 miles) southwest of the site, respectively.

3.3 Surface Conditions

The project area is located within the City of Sausalito right-of-way along the western side of Bridgeway, just south of the Coloma Street intersection. The site is bordered to the east by Bridgeway, to the north by Coloma Street, to the south by public landscaped areas, and to the west by a private townhome development. The ground surface is covered with grass, trees and sidewalks and an asphalt-paved driveway which is oriented roughly north/south along the western property boundary. The site is gently sloping with surface elevations ranging from about 13 to 17 feet[1].

The existing Whiskey Springs Pump Station is located within the proposed Coloma Pump Station footprint and will be removed as part of the planned improvements. The concrete structure occupies an approximately 27-foot by 17-foot footprint and is embedded roughly 23 feet below grade with an

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[1] Elevation references correspond to NAVD88.

invert elevation of about – 5 feet. A portable standby generator is located adjacent to the existing pump station within the asphalt-paved driveway. A storm drain junction box is located near the north end of the site and receives flow from two 18-inch corrugated metal pipe storm drains aligned within Coloma Street. The junction box discharges to a 53-inch by 83-inch elliptical concrete storm drain that extends northeast into the adjacent intersection. Various other underground utilities exist within and around the project area, as shown on Figure 2.

3.4 Site History

In addition to available geotechnical and geologic reference information, we reviewed several historic topographic maps to assess site development history and changes in surface topography. The maps were obtained from the United States Geological Survey website and were recorded between 1895 and 2015 at a variety of scales. The maps indicate the site is located within previous tidal marshlands which were filled prior to the 1950's as part of reclamation and development of the surrounding shoreline areas.

3.5 Field Exploration and Geotechnical Laboratory Testing

We explored subsurface conditions near the proposed force main alignment on July 12, 2017 with two borings at the approximate locations shown on Figure 2. The borings were excavated using track-mounted drilling equipment to approximate depths ranging from 16.5 to 26.5 feet below the ground surface. The borings were logged by our Field Geologist and samples were obtained for classification and laboratory testing. We prepared boring logs based on soil descriptions in the field, as well as visual examination and testing of the soil samples in our laboratory. The boring logs are presented in Appendix A.

Geotechnical laboratory testing of soil samples from the exploratory borings included determination of moisture content, dry density, unconfined compressive strength and the amount of material passing a No. 200 sieve. The results of our laboratory tests are presented on the boring logs and our laboratory testing program is discussed in greater detail in Appendix A.

3.6 Environmental Laboratory Testing

We composited soil samples collected at various depths from Borings 1 and 2 and delivered them to Analytical Sciences of Petaluma, California. The composite sample was tested for the presence of volatile organic compounds (VOC), semi-volatile organic compounds (SVOC), total petroleum hydrocarbons (TPH) and 17 metals identified by California Administrative Manual (CAM) as hazardous material when found to exceed a total threshold limit concentration (TTLC) or soluble threshold limit concentration (STLC) as specified by the California Code of Regulations (CCR), Title 22. The results of the environmental laboratory testing are presented in Appendix B.

While many of the CAM 17 metals were detected, the test results indicate the levels are below the TTLC. The results of our CAM 17 and TPH testing are presented in Appendix B. The results of our environmental testing indicate that onsite soils are generally not considered hazardous toxic waste in accordance with CAM regulations.

3.7 Subsurface Conditions and Groundwater

Based on our field exploration, subsurface conditions are generally consistent with geologic mapping, and consist of artificial fill over Bay Mud with fill thickness at the boring locations ranging from about 9 to 10 feet. The fill encountered in our borings is heterogeneous and contains variable amounts of clay, silt, sand, and gravel. While not directly observed, the fill may also contain cobbles, boulders, wood, organic material, and other debris which could not be retrieved within the samples or detected by the relatively small diameter borings. The Bay Mud encountered in our borings generally consists of very soft to soft silty clay of high plasticity.

Groundwater was encountered in both borings at about 12 feet below ground surface. Because the borings were not left open for an extended period of time, a stabilized depth to groundwater may not have been observed. Groundwater elevations fluctuate seasonally and with changes in tidal elevations and higher groundwater levels may be present during periods of intense rainfall and/or high tide.

A cursory search of the State Water Resources Control Board's Geotracker website indicates that groundwater monitoring wells were installed as part of various environmental studies near the Shell Service Station located roughly 450 feet northwest of the project site (Cambria, 2003). The monitoring data from these studies indicates the groundwater elevation at the well locations typically varied from about 3 to 10 feet below ground surface which corresponds to approximate elevations of +1 to +8 feet.

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, potential liquefaction of the sandy fill soils, settlement due to consolidation of the soft Bay Mud, and potentially corrosive soil and groundwater conditions. Other geologic hazards are judged less than significant with regard to the proposed project. Each significant geologic hazard considered is discussed in further detail in the following paragraphs.

4.1 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, and probable peak ground accelerations are summarized in Table 1. The calculated accelerations should only be considered as reasonable estimates. Many factors (soil conditions, orientation to the fault, etc.) can influence the actual ground surface accelerations.

Table 1 – Estimated Peak Ground Accelerations for Principal Active Faults

Fault	Moment Magnitude for Characteristic Earthquake	Closest Estimated Distance (km)	Median Peak Ground Acceleration (g)
San Andreas	8.0	10	0.30
San Gregorio	7.4	13	0.25
Hayward	7.3	17	0.21
Rodgers Creek	7.3	33	0.14
Calaveras	6.9	40	0.10

Reference: Caltrans ARS Online v2.3.09 accessed on August, 2017 using V_{s30} = 180 m/s.

The potential for strong seismic shaking at the project site is high. Due to their proximity and historic rates of activity, the San Andreas 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 the pump station and related improvements. Measures to mitigate the effects of ground shaking should, as a minimum, include using flexible connections where pipelines connect to the new structures and designing new structures to resist seismic loads as discussed in Section 5.1.

4.2 Liquefaction and Related Effects

Liquefaction refers to the sudden, temporary loss of soil strength during strong ground shaking. The strength loss occurs as a result of the build-up of excess pore water pressures and subsequent reduction of effective stress. While liquefaction most commonly occurs in saturated, loose, granular deposits, recent studies indicate that it can also occur in materials with relatively high fines content provided the fines exhibit lower plasticity. The effects of liquefaction can vary from cyclic softening resulting in limited strain potential to flow failure which cause large settlements and lateral ground movements. Lateral spreading refers to a specific type of liquefaction-induced ground failure characterized primarily by horizontal displacement of surficial soil layers as a consequence of liquefaction of a subsurface granular layer (Youd, 1995). Lateral spreads generally move down gentle slopes or slip toward a free face such as an incised river channel.

As shown on Figure 5, regional liquefaction hazard maps indicate the majority of the site is mapped within a zone of very high susceptibility to liquefaction (Association of Bay Area Governments, 2017). The fill encountered in our borings consists of an approximately 9- to 10-foot-thick layer of loose to medium dense sandy soils which contain variable amounts of gravel, clay and silt. We evaluated liquefaction potential for these soils in general accordance with the procedures outlined by Idriss and Boulanger (2010). We assumed a groundwater depth of seven feet below ground surface (approximate elevation of +8 feet) and a peak ground acceleration[2] (PGA) of 0.51 g. The results of our liquefaction analyses are shown on Figure 6 and indicate the fill is generally susceptible to liquefaction under the design PGA. The analysis further indicates that if liquefaction does occur, post-liquefaction settlements of up to about two inches should be expected. Minimum mitigation measures should include using flexible connections where pipelines connect to the new structures and designing new structures to accommodate the estimated post-liquefaction settlements as discussed in Section 5.3.

4.3 Settlement

Significant settlement can occur when new loads are applied to soft, compressible soils such as the Bay Mud encountered in the project borings. The rate and magnitude of potential settlements are dependent on the new loads that are applied, the presence of drainage layers, the thickness of compressible material, the inherent compressibility properties of the soils, and other factors. There are two modes of settlement of the Bay Mud: primary consolidation and secondary compression. Consolidation settlement often takes decades to complete, whereas secondary compression is generally a fraction of the total settlement but occurs over a much longer time.

Considering the Bay Mud thickness within the project area is less than about 10 feet, and that nearly 70 years has passed since the existing fill was placed, primary consolidation settlement due to the fill placement is expected to be essentially complete. However, additional settlements may be induced due to new structural loads from the planned improvements. We estimated the magnitude of these additional settlements using the preliminary layout shown on Figure 2 and structural loads presented in Table 2. Our analysis assumes the Bay Mud is normally consolidated with a compression index (Cc, strain-based) of 0.31 and recompression index (Cr, strain-based) of 0.05.

[2] The PGA used for liquefaction analysis is based on the MCE_G peak ground acceleration adjusted for site effects, calculated in accordance with ASCE 7 (ASCE, 2010).

Table 2 – Preliminary Structural Loads

Structure	Plan Dimensions	Embedment Depth	Contact Pressure
Coloma Pump Station	16.3-ft-diameter	28 feet	560 psf
Valve Vault	17.5 feet by 12.3 feet	8.5 feet	750 psf
Generator Pad	29.5 feet by 12.0 feet	At Grade	550 psf
Electrical Building	28.0 feet by 20.0 feet	14.2 feet	715 psf
Flowmeter Vault	13.7 feet by 9.0 feet	8.3 feet	915 psf

Reference: Preliminary dimensions and loads based on information provided by Carollo.

Based on the planned excavation depths, new structural loads from underground structures (e.g., the pump station, electrical building, vaults, manholes, etc.) will likely be balanced by the soil that is removed during excavation. Additionally, the pump station will likely be founded on firm bedrock which was encountered at about 20 feet below ground surface in the project borings. Therefore, consolidation settlement due to the proposed underground structures is expected to be negligible. For new above-grade structures founded on shallow foundations, we estimate up to about 2 inches of total settlement may occur due to the new loads. The new structures should be designed to account for the estimated settlements, as discussed further in Section 5.3.

4.4 Corrosive Soils

Corrosive soil can damage buried metallic structures, cause concrete spalling, and deteriorate rebar reinforcement. Per Caltrans Corrosion Guidelines (2012), a soil is considered corrosive if one or more of the following conditions exist:

- Chloride concentration is 500 ppm or greater
- Sulfate concentration is 2,000 ppm or greater
- pH is 5.5 or less

Soil samples collected from both borings at various depths were composited and tested to evaluate pH, electrical resistivity, chloride and sulfate contents. These laboratory test results are presented in Appendix B. The results of our corrosivity testing show the soil has a pH of 7.5, a chloride concentration of 2200 parts per million (ppm), and a sulfate concentration of 190 ppm. The corrosion test results indicate the soils are considered corrosive per Caltrans guidelines.

Minimum mitigation measures should include designing concrete structures in accordance with applicable durability requirements outlined in ACI 318. Metallic components should incorporate protective coatings or other measures aimed at improving corrosion resistance. We recommend that a qualified corrosion engineer be retained to review the laboratory test results and to provide additional mitigation measures as required.

5.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our subsurface exploration, we judge that construction of the proposed pump station and related improvements is feasible from a geotechnical standpoint. Primary geotechnical considerations for the project include: excavation through variable geologic conditions which include loose to medium dense, saturated sandy fill, soft Bay Mud and Franciscan bedrock; providing appropriate temporary support for excavations; providing for proper pipe bedding and backfill for excavations; and designing new structures to account for seismic loads and potential settlement due to liquefaction and/or consolidation of the soft Bay Mud. Additional discussion and recommendations addressing these and other considerations are presented in the following sections.

5.1 Seismic Design

Minimum mitigation of ground shaking includes seismic design of new structures in conformance with the provisions of the most recent edition (2016) 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 San Andreas and San Gregorio Faults, we recommend the CBC coefficients and site values shown in Table 3 be used for the design of the new pump station and related improvements.

Table 3 – 2016 California Building Code Seismic Design Criteria

Parameter	Design Value
Site Class	E
Site Latitude	37.868°N
Site Longitude	-122.503°W
Spectral Response (short), S_S	1.500 g
Spectral Response (1-sec), S_1	0.651 g
Site Coefficient, F_a	0.9
Site Coefficient, F_V	2.4

Reference: USGS US Seismic Design Maps accessed on August 18, 2017.

5.2 Earthwork

Earthwork for the pump station and related improvements should be performed in accordance with the following recommendations:

5.2.1 Subgrade Preparation

Clear pavements, over-sized debris, and organic material from areas to be graded. Debris, rocks larger than six inches, and vegetation are not suitable for structural fill and should be removed from the site. Trees that are located within the building areas should be removed and the root systems excavated. Existing foundations and utilities which are to be abandoned as part of the work should be removed from structural areas. In non-structural areas, utilities could be abandoned in place in many cases provided cement grout completely fills any void in the utility.

Where structural improvements are planned, the subgrade surface should be scarified to a depth of eight inches, moisture conditioned to slightly above the optimum moisture content, and compacted to at least 90 percent relative compaction. Within pavement areas, the subgrade should be compacted to at least 95 percent relative compaction. Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density, as determined by ASTM D1557. Areas exposing bedrock at subgrade elevation need not be scarified and compacted. The subgrade should be firm and unyielding when proof-rolled with heavy, rubber-tired construction equipment. If soft, wet or otherwise unsuitable materials are encountered at subgrade elevation during construction, we will provide supplemental recommendations to address the specific condition.

5.2.2 Excavations

Excavations will generally encounter fill consisting of loose to medium dense, clayey sand over very soft to soft Bay Mud. Based on the borings, weathered Franciscan bedrock will also likely be encountered where excavations extend more than about 20 feet below ground surface. While not encountered in our borings, the backfill around and below the existing pump station may also include relatively permeable materials.

In unsupported excavations, the sandy fill soils will be susceptible to flowing below groundwater and running to fast raveling above groundwater. The Bay Mud will be susceptible to squeezing while the Franciscan bedrock will exhibit firm behavior. Definitions of the various ground behaviors are presented in the Tunnelman's Ground Classification for Soils, Figure 7. In accordance with OSHA soil type designations, the fill and Bay Mud are considered "Type C" soils whereas the Franciscan bedrock is considered "Type B". Temporary support for excavations should be installed to ensure the safety of workers and to reduce the potential for damage to surrounding areas. Shoring and temporary support of excavations is discussed in further detail in Section 5.4

Based on our subsurface exploration, we judge the majority of site excavation can be performed with typical equipment, such as medium-size excavators. However, Franciscan bedrock often contains inclusions and zones of harder, more resistant rock which may require specialized techniques or equipment to excavate (e.g. jackhammers or hydraulic breakers). Therefore, we recommend inclusion of a line item and clear definition for "hard rock excavation" in the project bid documents. If hard rock is encountered during construction which prohibits excavation to the required depths, we should be consulted to observe conditions and revise our recommendations and/or design criteria as appropriate.

5.2.3 Excavation Bottom Stabilization

Based on the fill thicknesses observed during our subsurface exploration, the bottom of excavations for the new improvements may extend through the fill soils and into the underlying Bay Mud. In areas where excavations extend below the top of the Bay Mud or where excavation bottoms are soft, loose, or otherwise unstable, we recommend the excavation bottoms be overexcavated a minimum of 12 inches below the planned invert elevation and backfilled with drain rock. The drain rock should be completely wrapped with a geotextile filter fabric consisting of Mirafi FW300 or an approved equivalent.

5.2.4 Fill Materials

Unless otherwise recommended by SMCSD or 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. Class 2 aggregate base may also be used provided it conforms to the most recent version of Caltrans Standard Specifications and is compacted as described below. Shovel slicing or other methods should be used to ensure embedment materials are uniformly compacted around the haunches of the pipe. Alternatively, controlled low strength material (CLSM) may be used for pipe bedding and embedment. Provide the minimum bedding thickness beneath the pipe in accordance with the manufacturer’s recommendations (typically 3 to 6 inches).

Fill materials used for structural backfill should consist of non-expansive materials that are free of organic matter, have a Liquid Limit of less than 40 (ASTM D 4318), a Plasticity Index of less than 20 (ASTM D 4318), and have a minimum R-value of 20 (California Test 301). The fill material should contain no more than 50 percent of particles passing a No. 200 sieve and should have a maximum particle size of 4 inches. Some of the onsite fill soils may be suitable for re-use as trench backfill. The Bay Mud is not suitable for use as fill and should be removed from the site.

5.2.5 Fill Placement and Compaction

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

5.3 <u>Foundation and Pump Station Structural Design</u>

Design criteria for the new pump station and other underground structures are detailed on Figure 8. A buoyant uplift force will develop when the water level within the structure is lower than the exterior groundwater level. The design engineer will need to determine the maximum differential between the exterior and interior water levels. Resistance to uplift includes the weight of the structure plus the skin friction on the exterior of the structure. If necessary, the uplift resistance can be increased by structurally extending the foundation beyond the limits of the walls. Alternatively, helical anchors could be utilized to provide uplift resistance.

The walls of the pump station are expected to be restrained at the top and bottom which prevents lateral deflection. This type of wall is subject to a uniform active pressure distribution instead of the equivalent fluid pressure that is typically used for cantilevered walls. In addition, the walls need to withstand seismic and surcharge loading and hydrostatic forces due to potential differential water levels inside and outside of the wet well, as shown on Figure 8. For design of the pump station and other buried structures, we recommend that the groundwater level be taken as equal to the ground surface elevation.

The generator, transformer and other above-grade improvements may be supported on mat slabs provided they are designed to account for settlement due to liquefaction and consolidation of the Bay Mud. This should include up to 3 inches of total settlement and 1.5 inches of differential settlement over a span of 25 feet. If mat slab foundations are utilized, they should be designed using a modulus of subgrade reaction of 200 pci. Mat slabs should be designed using an allowable bearing pressure of 2,000 psf for soil and 4,000 psf for bedrock. Lateral loads for mat slabs can be resisted using a base friction coefficient of 0.35 and a passive resistance of 300 pcf. The slab subgrade should be prepared as discussed under Section 5.2.1. If excavations expose soft, unstable soils beneath the slabs, the bottom of the excavation should be prepared as discussed under Section 5.2.3.

If the new above-grade structures are not capable of accommodating the estimated settlements, they should be supported on deep foundations that extend into bedrock. While various deep foundation systems are feasible, helical piles are suitable for supporting the structural loads and are likely relatively cost-effective as compared to other systems. We estimate the helical piles should achieve vertical load capacities of about 35 kips provided they are advanced to bear on firm bedrock. Helical piles are limited in providing lateral resistance due to their relatively small diameter. Therefore, lateral loads will need to be resisted through base friction and passive resistance of embedded portions of the new structures. Battered helical piles could also be considered if additional lateral capacity is required.

Drilled piers are also an appropriate deep foundation alternative. However, the loose, granular fill soils and soft Bay Mud may result in caving or squeezing conditions and the use of casing, drilling slurry or other means of supporting the pier excavation during drilling and concrete placement should be anticipated. If used, drilled piers should be embedded a minimum of five feet into bedrock and should be designed using an allowable skin friction of 300 psf for soil and 1,000 psf for bedrock to resist vertical loads. Tip resistance should be neglected in calculating the vertical capacity for drilled piers.

The capacity of a drilled pier foundation system to resist lateral loading is a function of the soil type, foundation diameter, pier layout, allowable displacement and other factors. We estimated the lateral capacity of a drilled pier foundation for the new at-grade equipment pad using the computer software *Group 2016* by Ensoft, Inc. and the soil parameters shown in Table 4. Our analysis considered 24-inch-diameter drilled piers with a 1.5-ft-thick mat slab configured as shown in Appendix C and the three loading conditions presented below. The calculated deflections, bending moment and shear for each pile under the various loading conditions are summarized in Appendix C.

Case 1: Estimating the displacements resulting from a total lateral load of 51 kips.

Case 2: Estimating the total lateral load that would result in a displacement of 0.25 inches.

Case 3: Estimating the total lateral load that would result in a displacement of 0.50 inches.

Table 4 – Soil Parameters for Lateral Pile Analysis

Depth (feet)	General Soil Type	Unit Weight (pcf)	Friction Angle (deg)	Undrained Shear Strength (psf)	Subgrade Modulus (pci)	Strain at 50% Deflection (%)
0 to 10	Sandy Soil	115	30	0	20	-
10 to 20	Soft Clay	95	0	300	30	2.0
20+	Bedrock	130	0	4,000	2,000	0.5

5.4 Temporary Support of Excavations

Temporary support of excavations will be required to ensure the safety of workers and to reduce the potential for failure of the excavation sidewalls and damage to surrounding improvements. While a variety of systems are available, shoring that applies positive pressure and immediate support to the side walls of the excavation will be more effective in controlling ground movements and reducing the risk of damage to nearby utilities and structures. A “watertight” shoring system should be utilized to minimize the need for dewatering which could result in an increase in the effective stress of the overlying fill soils and subsequent settlement of the soft Bay Mud.

The selection, design, installation, monitoring, and removal of temporary shoring should be the responsibility of the Contractor in accordance with their means and methods. The selected support system should be designed to resist lateral pressures from earth, groundwater and construction surcharge loads. As a minimum, shoring systems should be designed based on the criteria provided in Figure 8.

5.5 Instrumentation and Monitoring

Given that deep excavations will be required for construction of the planned improvements, a preconstruction survey should be performed to document the condition of the existing pump station, adjacent apartment building and other nearby existing improvements. The survey should include video documentation of the buildings and surrounding areas and establishing survey control points on the ground surface and nearby structures. The baseline elevations of the monitoring points should be compared with survey readings taken during construction to determine if any ground movements occur.

If construction activities are expected to generate relatively high levels of ground-borne vibrations (e.g. from sheet pile installation, pile driving, etc.), vibration monitoring should be performed to evaluate potential impacts on adjacent structures. The susceptibility of damage to structures due to ground-borne vibrations depends on the type of vibration (i.e. transient or continuous), the distance from the vibration source, the age and condition of the structure, soil and groundwater conditions, the type of construction (e.g. wood-framed, steel, architectural finishes, etc.) and various other factors. A study by Whiffen (1971) established damage thresholds to structures due to continuous vibrations, as summarized in Table 5. While these thresholds may be used for general guidance, project-specific thresholds should be established based on criteria that is developed for specific structures.

Table 5 – Damage Thresholds due to Continuous Vibration

PPV (in/sec)	Design Value
0.4 – 0.6	Architectural damage and possible minor structural damage
0.2	Threshold at which there is a risk of architectural damage to normal dwelling houses (houses with plastered walls and ceilings).
0.1	Virtually no risk of architectural damage to normal buildings
0.08	Recommended upper limit of vibration to which ruins and ancient monuments should be subjected.
0.006 – 0.019	Vibration unlikely to cause damage

5.6 New Pavements

New pavements will be required for the pump station driveway and for trenches that extend into traffic areas. We have provided preliminary pavement design in accordance with Caltrans procedures for flexible pavement (Caltrans, 2015). We assumed Traffic Index values ranging from 4 to 7 depending on the expected traffic loads for a twenty-year design life. For our preliminary design, we assumed an R-value of 20 and 50 which are generally consistent with R-values for select fill and Class 2 aggregate subbase, respectively. During construction, we should test the backfill materials to confirm the R-value of the backfill material is consistent with our assumed values. The preliminary recommended pavement sections are presented in Table 6.

Table 6 – Preliminary Asphalt-Concrete Pavement Sections

Traffic Index	Select Fill Backfill (R-Value = 20)		Class 2 Aggregate Subbase (R-Value = 50)	
	Asphalt Thickness (inches)	Class 2 Aggregate Base Thickness (inches)	Asphalt Thickness (inches)	Class 2 Aggregate Base Thickness (inches)
4	3.0	5.0	2.5	4.0
5	3.5	7.0	3.0	5.0
6	4.0	9.0	3.5	6.0
7	5.0	10.0	4.0	7.0

The subgrade in new pavement areas should be prepared as discussed under Section 5.2.1. The Class 2 aggregate base and asphalt-concrete should conform to the most recent version of Caltrans Standard Specifications and should be compacted to at least 95% relative compaction. Additionally, the 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 thickness may be increased.

6.0 SUPPLEMENTAL GEOTECHNICAL SERVICES

We must review the plans and specifications when they are nearing completion to confirm that the intent of our recommendations has been incorporated and to provide supplemental recommendations as needed. During construction, we must inspect geotechnical items relating to earthwork, foundations and new pavement construction. We should observe the excavations for the new pump station and other improvements, foundation construction, proper moisture conditioning of soils, fill placement and compaction, and other geotechnical-related work items.

7.0 LIMITATIONS

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

8.0 LIST OF REFERENCES

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

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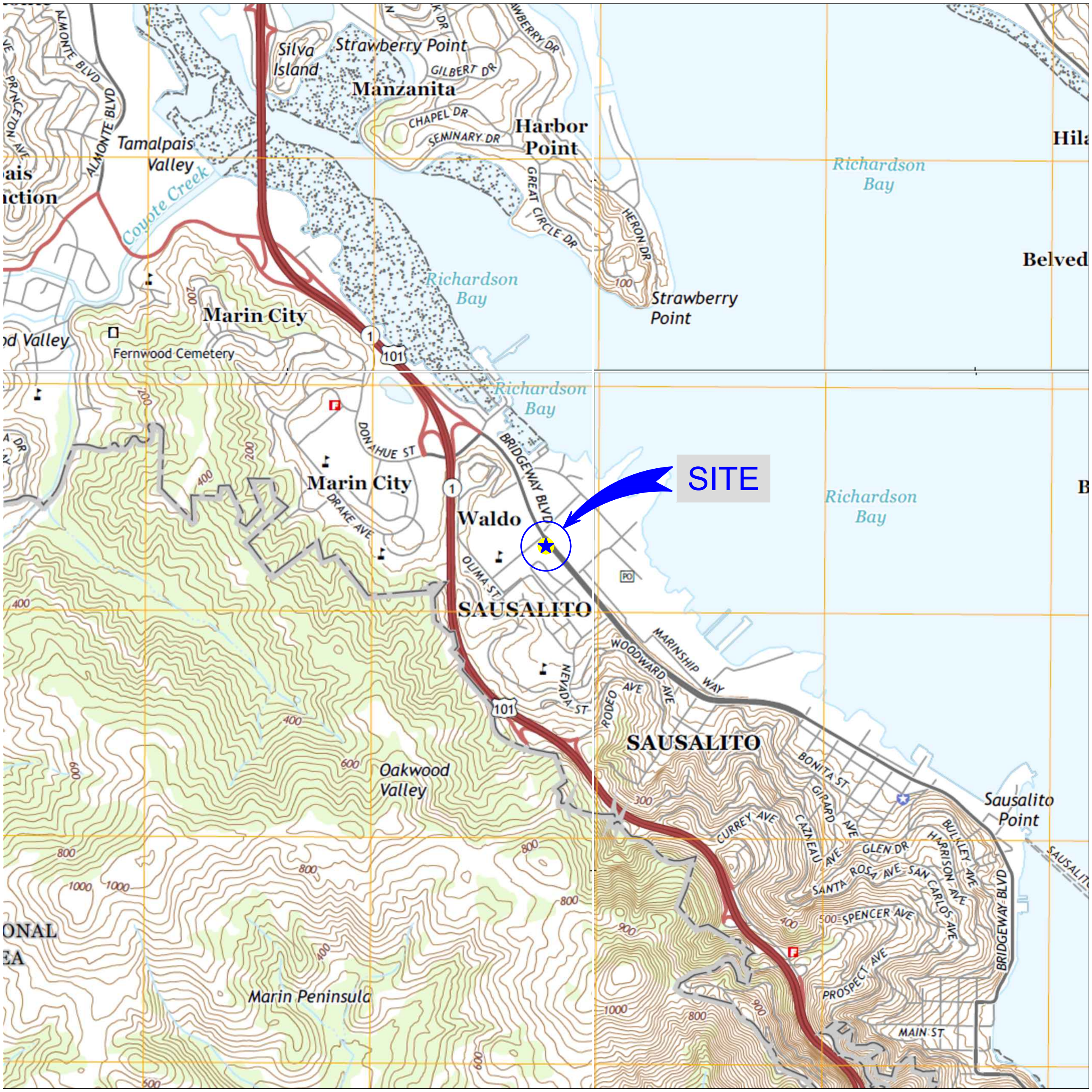

SITE LOCATION

(NO SCALE)

NORTH

REFERENCE: USGS, Point Bonita, San Francisco North, San Rafael, and San Quentin Quadrangle, 7.5' Series, 2015.

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SITE LOCATION MAP

SMCSD Coloma Pump Station
Sausalito, California

Project No. 2213.004

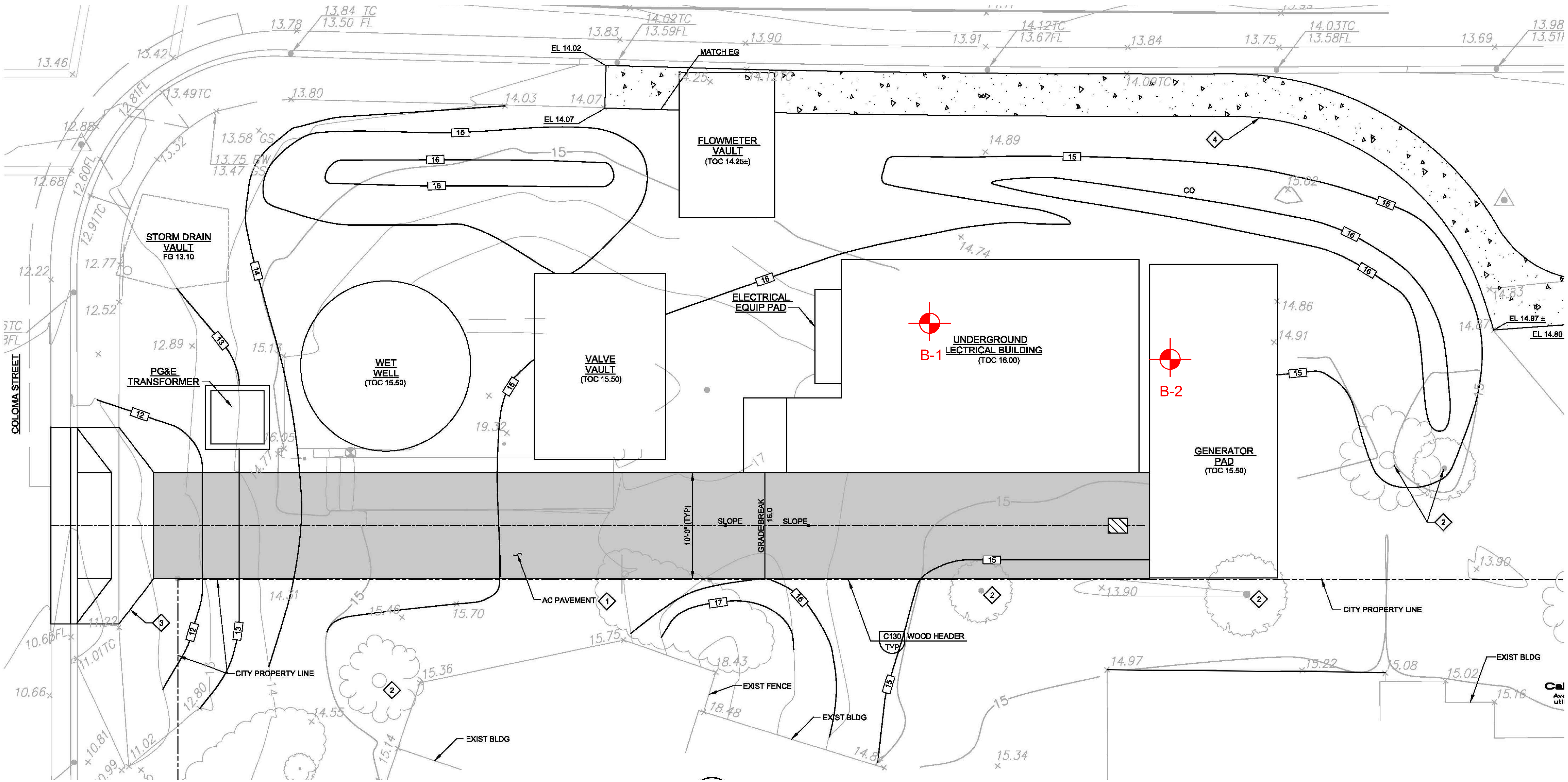
Date: 8/20/2017

Drawn RCA

Checked

1

FIGURE



SITE PLAN

(SCALE: 1" = 10'-0")

LEGEND:

APPROXIMATE LOCATION OF TEST BORING BY MILLER PACIFIC, 2017

REFERENCE: Carollo, "Sausalito-Marin City Sanitary District, Coloma Pump Station Improvements Project, Sheet C02 (50% Submittal), dated April 2019.

NORTH

SCALE

0 5 10 20 FEET

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SITE PLAN

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Project No. 2213.004 Date: 8/24/2017

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Drawn
RCA
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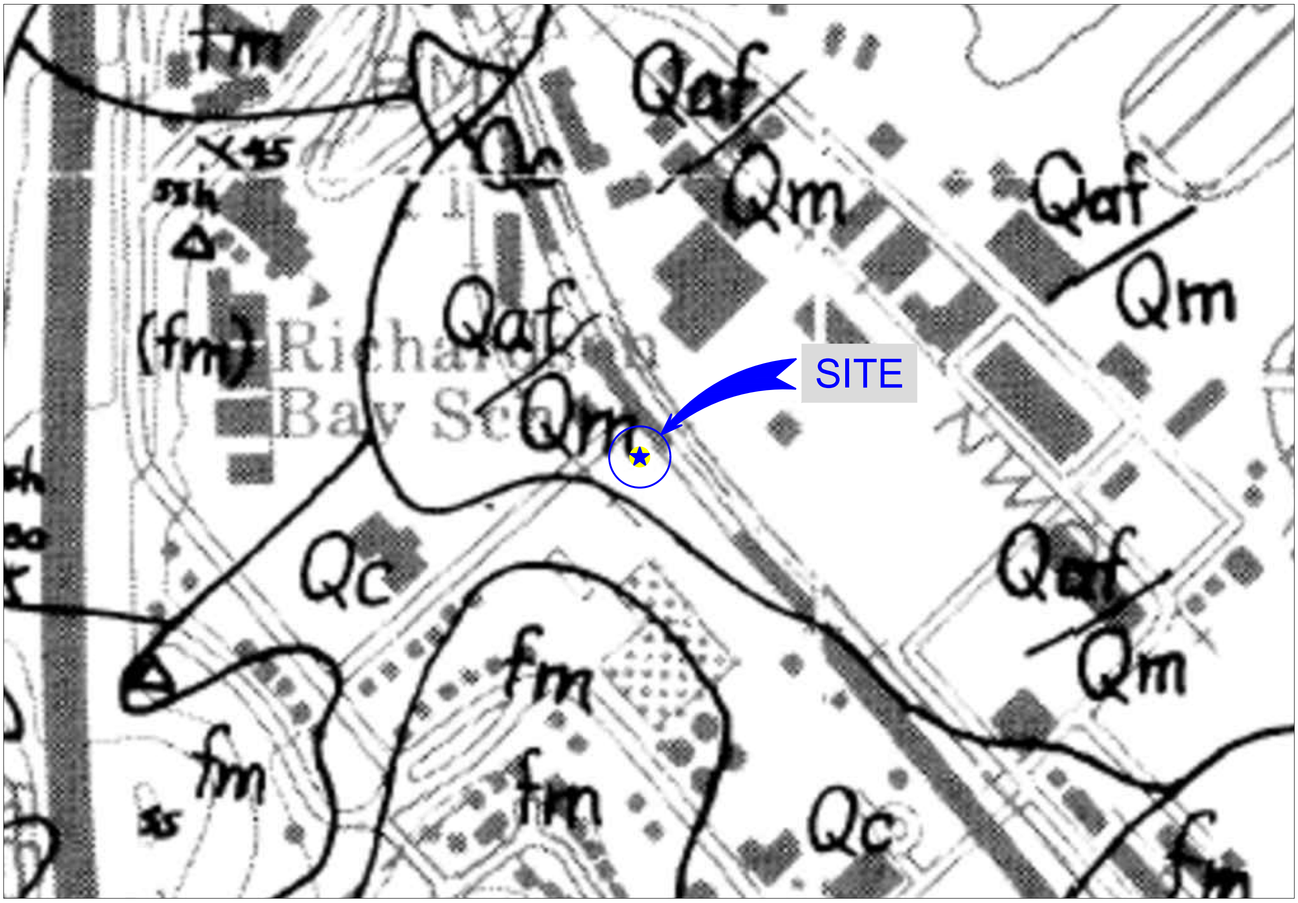
2
FIGURE



REGIONAL GEOLOGIC MAP

(NOT TO SCALE)

NORTH

LEGEND

Qaf **Artificial Fill** - Deposits of rock, soil, garbage and debris placed by man on natural surfaces, mostly for engineering purposes. Highly variable from place to place as to composition, degree of compaction, etc.

Qm **Bay Mud** - Marshlands and mudflats bordering San Francisco and San Pablo Bays. Deposits of unconsolidated, low density, semi-fluid, highly compressible silty clay.

Qc **Colluvium** - Unconsolidated and unsorted soil material and weathered rock fragments accumulated on or at the base of slopes by natural gravitational forces.

fm **Franciscan Melange** - A tectonic mixture of resistant rock types, principally sandstone, greenstone, chert, and serpentine, embedded in a matrix of pervasively sheared rock material

Reference: Rice, Salem J., Smith, Theodore C., (1976), "Geology of the Tiburon Peninsula, Sausalito, and Adjacent Areas, Marin County, California." California Department of Mines and Geology, Scale 1:12,000.

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REGIONAL GEOLOGIC MAP

SMCSD Coloma Pump Station
Sausalito, California

Project No. 2213.004 Date: 8/20/2017

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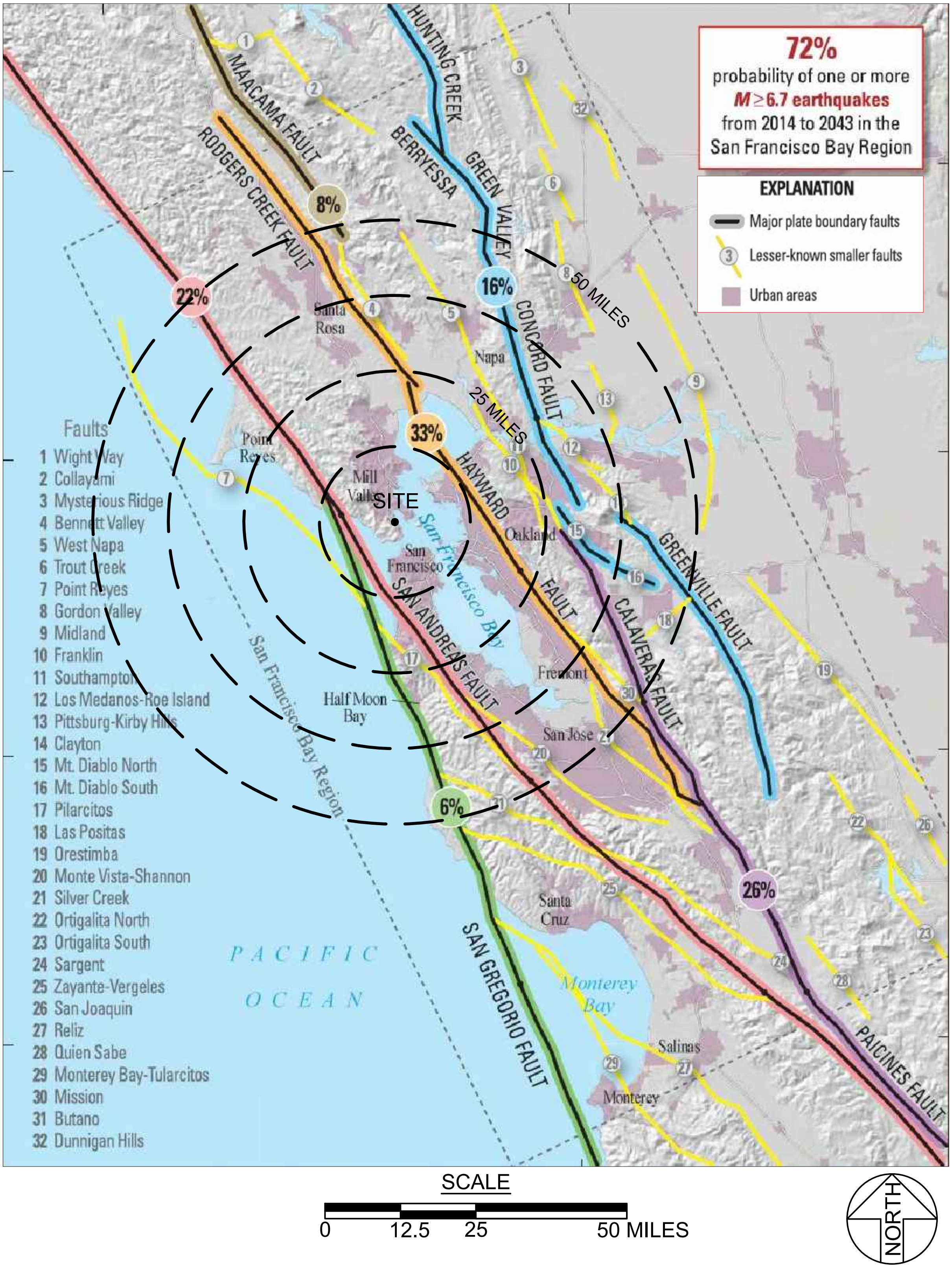
3
FIGURE



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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		SMCSD Coloma Pump Station Sausalito, California	Drawn NGK Checked	4 FIGURE
		Project No. 2213.004 Date: 8/20/2017		

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LIQUEFACTION SUSCEPTIBILITY

(NO SCALE)

Liquefaction Susceptibility Hazard

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

REFERENCE: Association of Bay Area Governments (http://gis.abag.ca.gov/website/Hazards/?hlyr=liqSusceptibility), accessed August 20, 2017.

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LIQUEFACTION SUSCEPTIBILITY MAP

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Sausalito, California

Project No. 2213.004 Date: 8/20/2017

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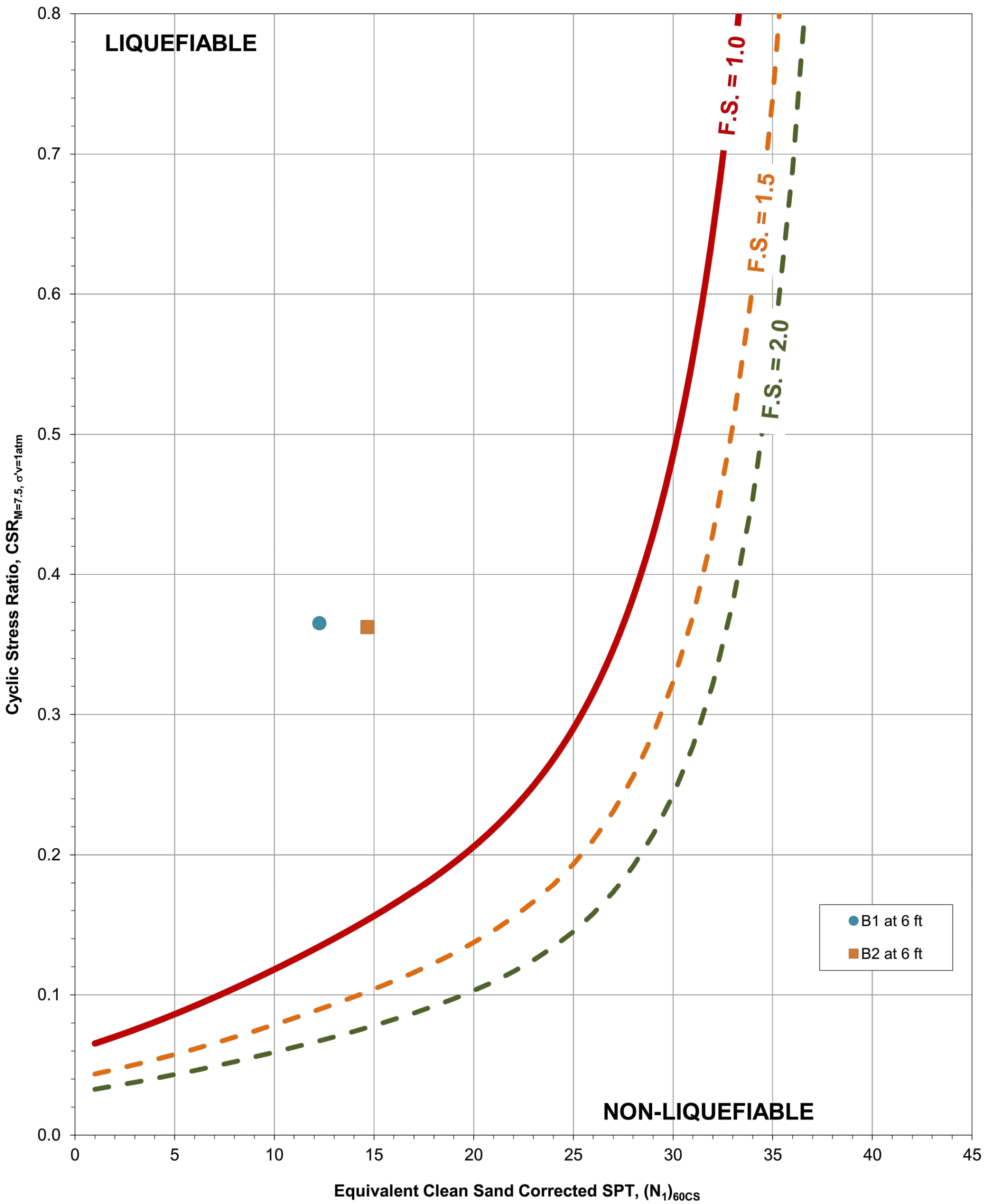
5
FIGURE

Liquefaction Analysis
(Idriss, I.M.& Boulanger, R.W., 2008 & 2010)



MPEG MILLER PACIFIC ENGINEERING GROUP	504 Redwood Blvd. Suite 220 Novato, CA 94947 T 415 / 382-3444 F 415 / 382-3450 www.millerpac.com	LIQUEFACTION ANALYSES SMCSD Coloma Pump Station Sausalito, California Project No. 2213.004 Date: 8/20/2017	Drawn RCA Checked	6 FIGURE

FILENAME: 2213.004 Figures.dwg

Tunnelman's Ground Classification for Soils[1]

Classification		Behavior	Typical Soil Types
Firm		Heading can be advanced without initial support, and final lining can be constructed before ground starts to move.	Loess above water table; hard clay, marl, cemented sand and gravel when not highly overstressed.
Raveling	Slow raveling ----- Fast raveling	Chunks or flakes of material begin to drop out of the arch or walls sometime after the ground has been exposed, due to loosening or to over-stress and "brittle" fracture (ground separates or breaks along distinct surfaces, opposed to squeezing ground). In fast raveling ground, the process starts within a few minutes, otherwise the ground is slow raveling.	Residual soils or sand with small amounts of binder may be fast raveling below the water tale, slow raveling above. Stiff fissured clays may be slow or fast raveling depending upon degree of overstress.
Squeezing		Ground squeezes or extrudes plastically into tunnel, without visible fracturing or loss of continuity, and without perceptible increase in water content. Ductile, plastic yield and flow due to overstress.	Ground with low frictional strength. Rate of squeeze depends on degree of overstress. Occurs at shallow to medium depth in clay of very soft to medium consistency. Stiff to hard clay under high cover may move in combination of raveling at excavation surface and squeezing at depth behind surface.
Running	Cohesive - running ----- Running	Granular materials without cohesion are unstable at a slope greater than their angle of repose (+/- 30° – 35°). When exposed at steeper slopes they run like granulated sugar or dune sand until the slope flattens to the angle of repose.	Clean, dry granular materials. Apparent cohesion in moist sand, or weak cementation in any granular soil, may allow the material to stand for a brief period of raveling before it breaks down and runs. Such behavior is cohesive-running.
Flowing		A mixture of soil and water flows into the tunnel like a viscous fluid. The material can enter the tunnel from the invert as well as from the face, crown, and walls, and can flow for great distances, completely filling the tunnel in some cases.	Below the water table in silt, sand, or gravel without enough clay content to give significant cohesion and plasticity. May also occur in highly sensitive clay when such material is disturbed.
Swelling		Ground absorbs water, increases in volume, and expands slowly into the tunnel.	Highly preconsolidated clay with plasticity index in excess of about 30, generally containing significant percentages of montmorillonite.

1 Modified by Heuer (1974) from Terzaghi (1950)

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TUNNELMAN'S GROUND CLASSIFICATION FOR SOILS

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Sausalito, California

Project No. 2213.004 Date: 8/20/2017

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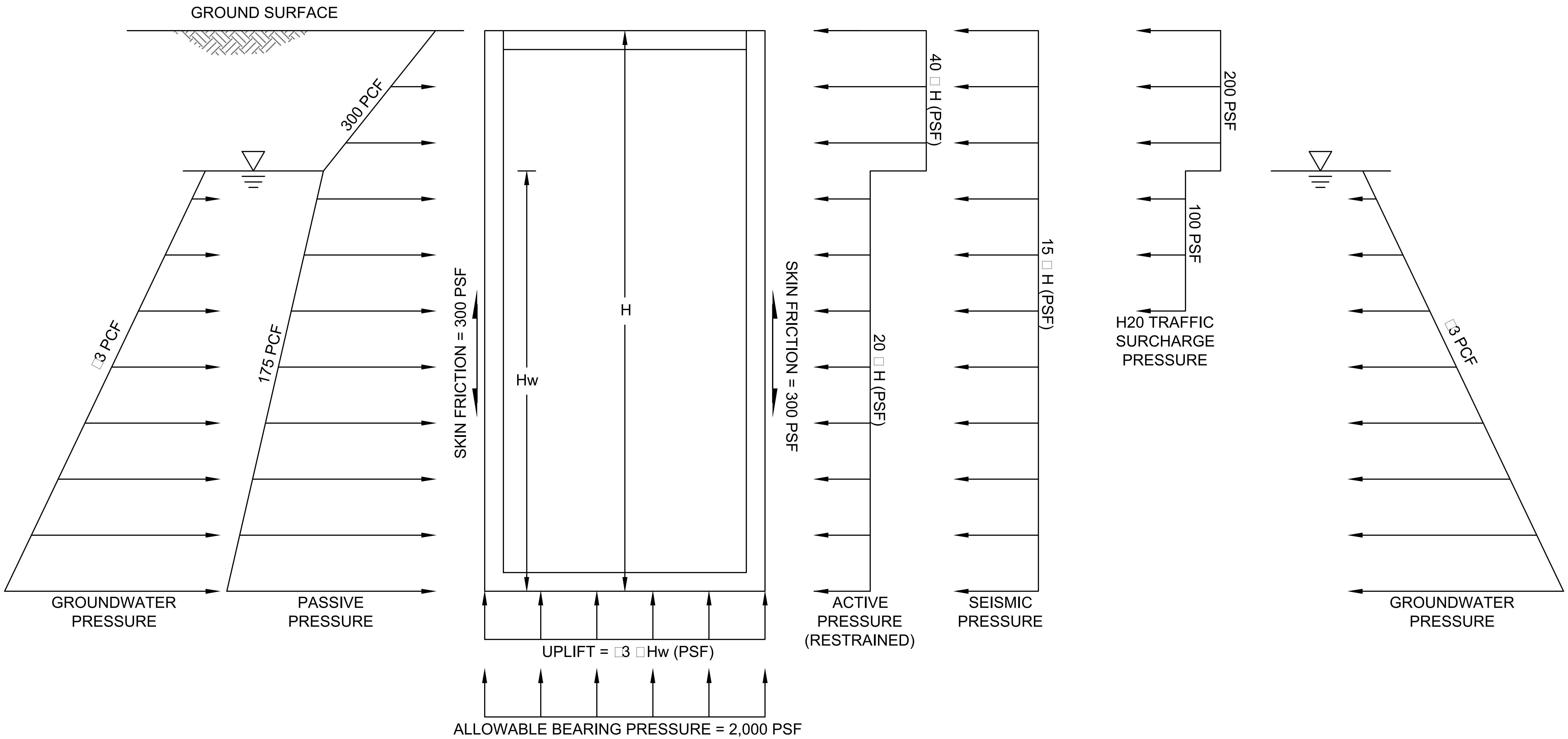
7
FIGURE



NOTES:

1. Assume groundwater is at the ground surface for design of permanent structures.
2. Surcharge pressures shown are □ased on H20 Traffic Loads. Other surcharge loads (e.g. due to construction traffic, soil stockpiles, structural loads, etc) may occur and should □e applied □y the designer as applica□le.

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FILE: 2213.004 Earth Pressure.dwg

PUMP STATION DESIGN CRITERIA

SMCSD Coloma Pump Station
Sausalito, California

Project No. 2213.004 Date: 8/3/201□

Designed
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8
FIGURE

APPENDIX A
SUBSURFACE EXPLORATION AND GEOTECHNICAL LABORATORY TESTING

A. SUBSURFACE EXPLORATION

We explored subsurface conditions with two exploratory borings drilled with track-mounted equipment on July 12, 2017 at the approximate locations shown on the Site Plan, Figure 2. The exploration was conducted under the technical supervision of our Field Geologist who examined and logged the soil materials encountered and obtained samples. The subsurface conditions encountered in the test borings are summarized and presented on the Boring Logs, Figures A-3 through A-5.

"Undisturbed" samples were obtained using a 3-inch diameter, split-barrel Modified California Sampler with 2.5 by 6-inch tube liners or a Standard Penetration Test (SPT) Sampler. The samplers were driven by a 140-pound hammer at a 30-inch drop. The number of blows required to drive the samplers 18 inches was recorded and is reported on the boring logs as blows per foot for the last 12 inches of driving. The samples obtained were examined in the field, sealed to prevent moisture loss, and transported to our laboratory

B. GEOTECHNICAL LABORATORY TESTING

We conducted geotechnical laboratory tests on selected intact samples to classify soils and to estimate engineering properties. The following laboratory tests were conducted in general accordance with the ASTM standard test method cited:

- Laboratory Determination of Water (Moisture Content) of Soil, Rock, and Soil-Aggregate Mixtures, ASTM D 2216
- Density of Soil in Place by the Drive-Cylinder Method, ASTM D 2937
- Unconfined Compressive Strength of Cohesive Soil, ASTM D 2166
- Amount of Material in Soils Finer than No. 200 (75-µm) Sieve, ASTM D1140
- Corrosivity as pH, EPA 904.
- Conductivity, SM 2510B
- Chlorides and Sulfates, EPA 300

The moisture content, dry density, amount of material passing a No. 200 sieve and unconfined compression test results are shown on the exploratory boring logs. The exploratory boring logs, description of soils encountered and the laboratory test data reflect conditions only at the location of the boring at the time they were excavated 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.

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 gravelly sands, little or no fines
		SP	Poorly-graded sands or gravelly 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

PI	PLASTICITY INDEX
LL	LIQUID LIMIT
SA	SIEVE ANALYSIS
HYD	HYDROMETER ANALYSIS
P200	PERCENT PASSING NO. 200 SIEVE
P4	PERCENT PASSING NO. 4 SIEVE

SAMPLER TYPE

- MODIFIED CALIFORNIA
- STANDARD PENETRATION TEST
- THIN-WALLED / FIXED PISTON
- HAND SAMPLER
- ROCK CORE
- X DISTURBED OR BULK SAMPLE

NOTE: Test boring and test pit logs are an interpretation of conditions encountered at the excavation location during the time of exploration. Subsurface rock, soil or water conditions may vary in different locations within the project site and with the passage of time. Boundaries between differing soil or rock descriptions are approximate and may indicate a gradual transition.

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 DRIVING RESISTANCE

Modified California and Standard Penetration Test samplers are driven 18 inches with a 140-pound hammer falling 30 inches per blow. Blows for the initial 6-inch drive seat the sampler. Blows for the final 12-inch drive are recorded onto the logs. Sampler refusal is defined as 50 blows during a 6-inch drive. Examples of blow records are as follows:

25	sampler driven 12 inches with 25 blows after initial 6-inch drive
85/7"	sampler driven 7 inches with 85 blows after initial 6-inch drive
50/3"	sampler driven 3 inches with 50 blows during initial 6-inch drive or beginning of final 12-inch drive

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SOIL CLASSIFICATION CHART

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Sausalito, California

Project No. 2213.004 Date: 8/2/2017

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A-1
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.

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ROCK CLASSIFICATION CHART

SMCSD Coloma Pump Station
Sausalito, California

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Project No. 2213.004 Date: 8/2/2017

A-2
FIGURE

BORING 1

EQUIPMENT: Track-Mounted Drill Rig with 4.0-inch Solid Flight Auger
DATE: 7/12/17
ELEVATION: 15 - feet*
*REFERENCE: Google Earth, 2016

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)	Description	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	DRILLING RATE (FT/MIN.)
0	0			Sandy Clayey GRAVEL (GC) light brown, dry to slightly moist, medium dense, gravel-sized chert, sandstone and shale fragments [Fill]						
1				Clayey SAND with Gravel (SC) brown and gray, moist, medium dense, fine to coarse grained [Fill]	32	120	8.6		P200 29.5	
2	5				10	123	12			
3	10			Sandy CLAY (CH) greenish-brown, moist, soft, medium to high plasticity [Bay Mud]	5	110	19.5	UC 250		
4										
5	15			Silty CLAY (CH) dark gray, wet, very soft, high plasticity abundant organics [Bay Mud]	3	52	84.4	UC 325		
6	20			Sandstone mottled gray, brown and orange, moderate hardness, moderate strength, moderately to highly weathered [Bedrock]						

∇ Water level encountered during drilling
▼ Water level measured after drilling

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

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BORING LOG

SMCSD Coloma Pump Station
Sausalito, California

Project No. 2213.004 Date: 8/2/2017

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A-3
FIGURE

BORING 1 (CONTINUED)

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)	DESCRIPTION	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	DRILLING RATE (MIN/FT)
	20			Sandstone mottled gray, brown and orange, moderate hardness, moderate strength, moderately to highly weathered [Bedrock]	92	123	14.9			
7										7.0
	25			grades to strong	50/3"					
8				Bottom of boring at 26.0 feet Groundwater encountered at 12.0 feet during drilling						
9	30									
10										
	35									
11										
12	40									

Water level encountered during drilling

Water level measured after drilling

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

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BORING LOG

SMCSD Coloma Pump Station
Sausalito, California

Project No. 2213.004 Date: 8/2/2017

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A-4
FIGURE

BORING 2

EQUIPMENT: Track-Mounted Drill Rig with 4.0-inch Solid Flight Auger
DATE: 7/12/17
ELEVATION: 15 - feet*
*REFERENCE: Google Earth, 2016

DEPTH (feet)	DEPTH (meters)	DESCRIPTION	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	DRILLING RATE (FT/MIN.)
0	0	Clayey SAND with Gravel (SC) brown with yellow and gray mottling, moist, medium dense, fine to coarse sand, subangular gravel [Fill]						
	1		20	121	8.8			
5		grades loose	7	117	14.9	UC 700	P200 41.0	
	2							
	3	SILTY CLAY (CH) gray, wet, soft, high plasticity clay, trace sand and organics [Bay Mud]						
10			8	107	22.6	UC 525		
▽	4		7					
15	5	grades very soft, saturated	3	50	84.2	UC 350		
		Bottom of boring at 16.5 feet. Groundwater encountered at 12.5 feet during drilling.						
20	6							

▽ Water level encountered during drilling
▼ Water level measured after drilling

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

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: 2213.004 BL.dwg

BORING LOG

SMCSD Coloma Pump Station
Sausalito, California

Project No. 2213.004 Date: 8/2/2017

Drawn NGK
Checked

A-5
FIGURE

APPENDIX B

ENVIRONMENTAL LABORATORY TEST RESULTS

Analytical Sciences

Report Date: July 26, 2017

Laboratory Report

Eric Dabanian
Miller Pacific Engineering - Novato
504 Redwood Blvd., Suite 220
Novato, CA 94947

Project Name: **Carollo - Coloma Pump Station** **2213-004**

Lab Project Number: **7071401**

This 14 page report of analytical data has been reviewed and approved for release.

M Peters

Michele Peters

Laboratory Director

P.O. Box 750336
Petaluma, CA 94975-0336
Telephone: (707) 769-3128

110 Liberty Street
Petaluma, CA
94952

Volatile Hydrocarbons by GC/MS

Lab#	Sample ID	Compound Name	Result (µg/kg)	RDL (µg/kg)
7071401-01	**B1 3.5', 5.5', 10.5' & 15.5' B2 3.5', 6', 11.5' & 15.5'**	Gasoline	ND	500
		Dichlorodifluoromethane (F-12)	ND	2.0
		Chloromethane	ND	2.0
		Vinyl chloride	ND	2.0
		Chloroethane (CE)	ND	2.0
		Bromomethane	ND	2.0
		Trichlorofluoromethane (F-11)	ND	2.0
		Trichlorotrifluoroethane (F-113)	ND	2.0
		1,1-Dichloroethene (1,1-DCE)	ND	2.0
		Methylene chloride	ND	2.0
		trans-1,2-Dichloroethene	ND	2.0
		1,1-Dichloroethane (1,1-DCA)	ND	2.0
		cis-1,2-Dichloroethene (c1,2-DCE)	ND	2.0
		2,2-Dichloropropane	ND	2.0
		Chloroform (THM1)	ND	2.0
		Bromochloromethane	ND	2.0
		1,1,1-Trichloroethane (TCA)	ND	2.0
		1,2-Dichloroethane (EDC)	ND	2.0
		1,1-Dichloropropene	ND	2.0
		Carbon tetrachloride	ND	2.0
		Benzene	ND	2.0
		Trichloroethene (TCE)	ND	2.0
		1,2-Dichloropropane (DCP)	ND	2.0
		Dibromomethane	ND	2.0
		Bromodichloromethane (THM2)	ND	2.0
		cis-1,3-Dichloropropene	ND	2.0
		Toluene	ND	2.0
		1,1,2-Trichloroethane	ND	2.0
		1,3-Dichloropropane	ND	2.0
		Dibromochloromethane (THM3)	ND	2.0
		Tetrachloroethene (PCE)	ND	2.0
		1,2-Dibromoethane (EDB)	ND	2.0
		Chlorobenzene	ND	2.0
		1,1,1,2-Tetrachloroethane	ND	2.0
		Ethylbenzene	ND	2.0
		m,p-Xylene	ND	2.0
		Styrene	ND	2.0
		o-Xylene	ND	2.0
		Bromoform (THM4)	ND	2.0
		1,1,2,2-Tetrachloroethane	ND	2.0
		Isopropylbenzene	ND	2.0
		1,2,3-Trichloropropane	ND	2.0
		Bromobenzene	ND	2.0
		n-Propyl Benzene	ND	2.0
		2-Chlorotoluene	ND	2.0
		4-Chlorotoluene	ND	2.0
		1,3,5-Trimethylbenzene	ND	2.0

Volatile Hydrocarbons by GC/MS

Lab#	Sample ID	Compound Name	Result (µg/kg)	RDL (µg/kg)
7071401-01	**B1 3.5', 5.5', 10.5' & 15.5' B2 3.5', 6', 11.5' & 15.5'**	tert-Butylbenzene	ND	2.0
		1,2,4-Trimethylbenzene	ND	2.0
		sec-Butylbenzene	ND	2.0
		1,3-Dichlorobenzene	ND	2.0
		1,4-Dichlorobenzene	ND	2.0
		1,2-Dichlorobenzene	ND	2.0
		p-Isopropyltoluene	ND	2.0
		n-Butylbenzene	ND	2.0
		1,2-Dibromo-3-chloropropane	ND	2.0
		1,2,4-Trichlorobenzene	ND	2.0
		Naphthalene	ND	2.0
		Hexachlorobutadiene	ND	2.0
		1,2,3-Trichlorobenzene	ND	2.0
		Tertiary Butyl Alcohol (TBA)	ND	25
		Methyl tert-Butyl Ether (MTBE)	ND	2.0
		Di-isopropyl Ether (DIPE)	ND	2.0
		Ethyl tert-Butyl Ether (ETBE)	ND	2.0
		Tert-Amyl Methyl Ether (TAME)	ND	2.0

Surrogates	Result (µg/kg)	% Recovery	Acceptance Range (%)
Dibromofluoromethane	53.1	107	70-130
Toluene-d8	46.5	94	70-130
4-Bromofluorobenzene	47.9	96	70-130

Date Sampled:	07/12/17	Date Analyzed:	07/14/17	QC Batch:	B016854
Date Received:	07/14/17	Method:	EPA 8260B		

TPH Diesel & Motor Oil

Lab#	Sample ID	Compound Name	Result (mg/kg)	RDL (mg/kg)
7071401-01	**B1 3.5', 5.5', 10.5' & 15.5' B2 3.5', 6', 11.5' & 15.5'**	Diesel	ND	5.0
		Motor Oil	ND	50

Date Sampled:	07/12/17	Date Analyzed:	07/20/17	QC Batch:	B016849
Date Received:	07/14/17	Method:	EPA 8015B		

Corrosivity as pH

Lab#	Sample ID	Compound Name	Result (pH Units)	RDL (pH Units)
7071401-01	**B1 3.5', 5.5', 10.5' & 15.5' B2 3.5', 6', 11.5' & 15.5'**	pH	7.52	1.00

Date Sampled:	07/12/17	Date Analyzed:	07/18/17	QC Batch:	B016885
Date Received:	07/14/17	Method:	EPA 9040		

CAM Metals

Lab#	Sample ID	Compound Name	Result (mg/kg)	RDL (mg/kg)
7071401-01	**B1 3.5', 5.5', 10.5' & 15.5' B2 3.5', 6', 11.5' & 15.5'**	Antimony (Sb)	ND	5.0
		Arsenic (As)	5.7	1.5
		Barium (Ba)	110	2.0
		Beryllium (Be)	ND	0.50
		Cadmium (Cd)	ND	0.50
		Chromium (Cr)	64	6.0
		Cobalt (Co)	12	1.5
		Copper (Cu)	25	2.0
		Lead (Pb)	18	3.0
		Molybdenum (Mo)	ND	1.0
		Nickel (Ni)	89	8.0
		Selenium (Se)	ND	5.0
		Silver (Ag)	ND	1.0
		Thallium (Tl)	ND	5.0
		Vanadium (V)	32	2.0
		Zinc (Zn)	66	5.0

Date Sampled:	07/12/17	Date Analyzed:	07/25/17	QC Batch:	B016894
Date Received:	07/14/17	Method:	EPA 6010B		

Mercury

Lab#	Sample ID	Compound Name	Result (mg/kg)	RDL (mg/kg)
7071401-01	**B1 3.5', 5.5', 10.5' & 15.5' B2 3.5', 6', 11.5' & 15.5'**	Mercury (Hg)	0.12	0.10

Date Sampled:	07/12/17	Date Analyzed:	07/25/17	QC Batch: B016895
Date Received:	07/14/17	Method:	EPA 7471A	

Resistivity

Lab#	Sample ID	Compound Name	Result (ohms-cm)	RDL (ohms-cm)
7071401-01	**B1 3.5', 5.5', 10.5' & 15.5' B2 3.5', 6', 11.5' & 15.5'**	Resistivity	330	10

Date Sampled:	07/12/17	Date Analyzed:	07/18/17	QC Batch: B016885
Date Received:	07/14/17	Method:	SM 2510 B-2011	

Anions

Lab#	Sample ID	Compound Name	Result (mg/kg)	RDL (mg/kg)
7071401-01	**B1 3.5', 5.5', 10.5' & 15.5' B2 3.5', 6', 11.5' & 15.5'**	Chloride	2200	200
		Sulfate as SO4	190	10

Date Sampled:	07/12/17	Date Analyzed:	07/25/17	QC Batch: B016905
Date Received:	07/14/17	Method:	EPA 300.0	

Notes and Definitions

RDL	Reporting Detection Limit
ND	Analyte NOT DETECTED at or above the reporting detection limit (RDL)
RPD	Relative Percent Difference
NR	Not Reported

Analytical Sciences
P.O. Box 750336, Petaluma, CA 94975-0336
110 Liberty Street, Petaluma, CA 94952
(707) 769-3128

CHAIN OF COUSTODY

7071401

LAB PROJECT NUMBER: Eric Dabanian

PROJECT NAME: Carollo - Coloma Pump Station

PROJECT NUMBER: 2213-004

CLIENT INFORMATION		BILLING INFORMATION	
COMPANY NAME:	Miller Pacific Engineering	CONTACT:	Laura
ADDRESS:	504 Redwood Blvd., Suite 200	COMPANY NAME:	Miller Pacific Engineering
	Novato, CA 94947	ADDRESS:	504 Redwood Blvd., Ste. 200
CONTACT:			Novato, CA 94947
PHONE #:	415-382-3444 FAX: 415-382-3450	PHONE #:	415-382-3444
EMAIL:		FAX #:	415-382-3450

TURNAROUND TIME (CHECK ONE)

MOBILE LAB ____

SAME DAY ____ 24 HOURS ____

48 HOURS ____ 72 HOURS ____

5 DAYS ____ NORMAL X

GEOTRACKER EDF: ___ Y ___ N

GLOBAL ID: ________

COOLER TEMPERATUR

________ ° C

COC

PAGE ___ OF ___

ITEM	CLIENT SAMPLE I.D.	DATE SAMPLED	TIME	MATRIX	# CONT.	PRESV. YES/NO	volatile hydrocarbons	semi-volatile hydrocarbons	cam 17 metals	total petroleum hydrocarbons, diesel & oil	total petroleum hydrocarbons, gasoline	corrosivity suite, sulfate, chloride, ph, resistivity								COMMENTS	LAB SAMPLE #
1	B1 3.5' & 5.5'	12-Jul	10am	soil	1	no	x	x	x	x	x	x								composite all samples	01
2	B1 10.5' & 15.5'	12-Jul	11am	soil	1	no	x	x	x	x	x	x								ie only one set of tests	
3	B2 3.5' & 6'	12-Jul	12noon	soil	1	no	x	x	x	x	x	x									
4	B2 11.5'	12-Jul	1pm	soil	1	no	x	x	x	x	x	x									
5	B2 15.5'	12-Jul	2pm	soil	1	no	x	x	x	x	x	x									
6																					
7																					
8																					
9																					
10																					

SIGNATURES

Sampled By: Zoe Stephens

Reliquished: [illegible] Date: 7/14/17 Time: 8:00

RECEIVED BY LABORATORY: SIGNATURE: [illegible] DATE: 7.14.17 TIME: 8:00

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"When Quality Counts"
1534 Willow Pass Road, Pittsburg, CA 94565-1701
Toll Free Telephone: (877) 252-9262 / Fax: (925) 252-9269
http://www.mccampbell.com / E-mail: main@mccampbell.com

Analytical Report

Client:	Analytical Sciences	**WorkOrder:**	1707577
Date Received:	7/17/17 16:00	**Extraction Method:**	SW3550B
Date Prepared:	7/18/17	**Analytical Method:**	SW8270C
Project:	7071401; Carollo-Coloma Pump Station	**Unit:**	mg/Kg

Semi-Volatile Organics

Client ID	Lab ID	Matrix	Date Collected	Instrument	Batch ID
B1-3.5', 5.5', 10.5', 15.5', B2-3.5', 6', 11	1707577-001A	Soil	07/12/2017 10:00	GC21	142151

Analytes	Result	RL	DF	Date Analyzed
Acenaphthene	ND	0.50	2	07/18/2017 16:30
Acenaphthylene	ND	0.50	2	07/18/2017 16:30
Acetochlor	ND	0.50	2	07/18/2017 16:30
Anthracene	ND	0.50	2	07/18/2017 16:30
Benzidine	ND	2.6	2	07/18/2017 16:30
Benzo (a) anthracene	ND	0.50	2	07/18/2017 16:30
Benzo (a) pyrene	ND	0.50	2	07/18/2017 16:30
Benzo (b) fluoranthene	ND	0.50	2	07/18/2017 16:30
Benzo (g,h,i) perylene	ND	0.50	2	07/18/2017 16:30
Benzo (k) fluoranthene	ND	0.50	2	07/18/2017 16:30
Benzyl Alcohol	ND	2.6	2	07/18/2017 16:30
1,1-Biphenyl	ND	0.50	2	07/18/2017 16:30
Bis (2-chloroethoxy) Methane	ND	0.50	2	07/18/2017 16:30
Bis (2-chloroethyl) Ether	ND	0.50	2	07/18/2017 16:30
Bis (2-chloroisopropyl) Ether	ND	0.50	2	07/18/2017 16:30
Bis (2-ethylhexyl) Adipate	ND	0.50	2	07/18/2017 16:30
Bis (2-ethylhexyl) Phthalate	ND	0.50	2	07/18/2017 16:30
4-Bromophenyl Phenyl Ether	ND	0.50	2	07/18/2017 16:30
Butylbenzyl Phthalate	ND	0.50	2	07/18/2017 16:30
4-Chloroaniline	ND	1.0	2	07/18/2017 16:30
4-Chloro-3-methylphenol	ND	0.50	2	07/18/2017 16:30
2-Chloronaphthalene	ND	0.50	2	07/18/2017 16:30
2-Chlorophenol	ND	0.50	2	07/18/2017 16:30
4-Chlorophenyl Phenyl Ether	ND	0.50	2	07/18/2017 16:30
Chrysene	ND	0.50	2	07/18/2017 16:30
Dibenzo (a,h) anthracene	ND	0.50	2	07/18/2017 16:30
Dibenzofuran	ND	0.50	2	07/18/2017 16:30
Di-n-butyl Phthalate	ND	0.50	2	07/18/2017 16:30
1,2-Dichlorobenzene	ND	0.50	2	07/18/2017 16:30
1,3-Dichlorobenzene	ND	0.50	2	07/18/2017 16:30
1,4-Dichlorobenzene	ND	0.50	2	07/18/2017 16:30
3,3-Dichlorobenzidine	ND	1.0	2	07/18/2017 16:30
2,4-Dichlorophenol	ND	0.50	2	07/18/2017 16:30
Diethyl Phthalate	ND	0.50	2	07/18/2017 16:30
2,4-Dimethylphenol	ND	0.50	2	07/18/2017 16:30
Dimethyl Phthalate	ND	0.50	2	07/18/2017 16:30
4,6-Dinitro-2-methylphenol	ND	2.6	2	07/18/2017 16:30

(Cont.)

NELAP 4033ORELAP

Angela Rydelius, Lab Manager

Analytical Report

Client:	Analytical Sciences	**WorkOrder:**	1707577
Date Received:	7/17/17 16:00	**Extraction Method:**	SW3550B
Date Prepared:	7/18/17	**Analytical Method:**	SW8270C
Project:	7071401; Carollo-Coloma Pump Station	**Unit:**	mg/Kg

Semi-Volatile Organics

Client ID	Lab ID	Matrix	Date Collected	Instrument	Batch ID
B1-3.5', 5.5', 10.5', 15.5', B2-3.5', 6', 11	1707577-001A	Soil	07/12/2017 10:00	GC21	142151

Analytes	Result	RL	DF	Date Analyzed
2,4-Dinitrophenol	ND	13	2	07/18/2017 16:30
2,4-Dinitrotoluene	ND	0.50	2	07/18/2017 16:30
2,6-Dinitrotoluene	ND	0.50	2	07/18/2017 16:30
Di-n-octyl Phthalate	ND	1.0	2	07/18/2017 16:30
1,2-Diphenylhydrazine	ND	0.50	2	07/18/2017 16:30
Fluoranthene	ND	0.50	2	07/18/2017 16:30
Fluorene	ND	0.50	2	07/18/2017 16:30
Hexachlorobenzene	ND	0.50	2	07/18/2017 16:30
Hexachlorobutadiene	ND	0.50	2	07/18/2017 16:30
Hexachlorocyclopentadiene	ND	2.6	2	07/18/2017 16:30
Hexachloroethane	ND	0.50	2	07/18/2017 16:30
Indeno (1,2,3-cd) pyrene	ND	0.50	2	07/18/2017 16:30
Isophorone	ND	0.50	2	07/18/2017 16:30
2-Methylnaphthalene	ND	0.50	2	07/18/2017 16:30
2-Methylphenol (o-Cresol)	ND	0.50	2	07/18/2017 16:30
3 & 4-Methylphenol (m,p-Cresol)	ND	0.50	2	07/18/2017 16:30
Naphthalene	ND	0.50	2	07/18/2017 16:30
2-Nitroaniline	ND	2.6	2	07/18/2017 16:30
3-Nitroaniline	ND	2.6	2	07/18/2017 16:30
4-Nitroaniline	ND	2.6	2	07/18/2017 16:30
Nitrobenzene	ND	0.50	2	07/18/2017 16:30
2-Nitrophenol	ND	2.6	2	07/18/2017 16:30
4-Nitrophenol	ND	2.6	2	07/18/2017 16:30
N-Nitrosodiphenylamine	ND	0.50	2	07/18/2017 16:30
N-Nitrosodi-n-propylamine	ND	0.50	2	07/18/2017 16:30
Pentachlorophenol	ND	2.6	2	07/18/2017 16:30
Phenanthrene	ND	0.50	2	07/18/2017 16:30
Phenol	ND	0.50	2	07/18/2017 16:30
Pyrene	ND	0.50	2	07/18/2017 16:30
Pyridine	ND	0.50	2	07/18/2017 16:30
1,2,4-Trichlorobenzene	ND	0.50	2	07/18/2017 16:30
2,4,5-Trichlorophenol	ND	0.50	2	07/18/2017 16:30
2,4,6-Trichlorophenol	ND	0.50	2	07/18/2017 16:30

(Cont.)

NELAP 4033ORELAP

Angela Rydelius, Lab Manager

McCampbell Analytical, Inc.
"When Quality Counts"

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Toll Free Telephone: (877) 252-9262 / Fax: (925) 252-9269
http://www.mccampbell.com / E-mail: main@mccampbell.com

Analytical Report

Client:	Analytical Sciences	**WorkOrder:**	1707577
Date Received:	7/17/17 16:00	**Extraction Method:**	SW3550B
Date Prepared:	7/18/17	**Analytical Method:**	SW8270C
Project:	7071401; Carollo-Coloma Pump Station	**Unit:**	mg/Kg

Semi-Volatile Organics

Client ID	Lab ID	Matrix	Date Collected	Instrument	Batch ID
B1-3.5', 5.5', 10.5', 15.5', B2-3.5', 6', 11	1707577-001A	Soil	07/12/2017 10:00	GC21	142151

Analytes	Result	RL	DF	Date Analyzed
Surrogates	**REC (%)**	**Limits**		
2-Fluorophenol	73	30-130		07/18/2017 16:30
Phenol-d5	69	30-130		07/18/2017 16:30
Nitrobenzene-d5	78	30-130		07/18/2017 16:30
2-Fluorobiphenyl	75	30-130		07/18/2017 16:30
2,4,6-Tribromophenol	42	16-130		07/18/2017 16:30
4-Terphenyl-d14	82	30-130		07/18/2017 16:30

Analyst(s): REB

Analytical Comments: a3

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Toll Free Telephone: (877) 252-9262 / Fax: (925) 252-9269
http://www.mccampbell.com / E-mail: main@mccampbell.com

Glossary of Terms & Qualifier Definitions

Client: Analytical Sciences
Project: 7071401; Carollo-Coloma Pump Station
WorkOrder: 1707577

Quality Control Qualifiers

F2	LCS/LCSD recovery and/or RPD is out of acceptance criteria.

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Toll Free Telephone: (877) 252-9262 / Fax: (925) 252-9269
http://www.mccampbell.com / E-mail: main@mccampbell.com

Glossary of Terms & Qualifier Definitions

Client: Analytical Sciences
Project: 7071401; Carollo-Coloma Pump Station
WorkOrder: 1707577

Glossary Abbreviation

%D	Serial Dilution Percent Difference
95% Interval	95% Confident Interval
DF	Dilution Factor
DI WET	(DISTLC) Waste Extraction Test using DI water
DISS	Dissolved (direct analysis of 0.45 μm filtered and acidified water sample)
DLT	Dilution Test (Serial Dilution)
DUP	Duplicate
EDL	Estimated Detection Limit
ERS	External reference sample. Second source calibration verification.
ITEF	International Toxicity Equivalence Factor
LCS	Laboratory Control Sample
MB	Method Blank
MB % Rec	% Recovery of Surrogate in Method Blank, if applicable
MDL	Method Detection Limit
ML	Minimum Level of Quantitation
MS	Matrix Spike
MSD	Matrix Spike Duplicate
N/A	Not Applicable
ND	Not detected at or above the indicated MDL or RL
NR	Data Not Reported due to matrix interference or insufficient sample amount.
PDS	Post Digestion Spike
PDSD	Post Digestion Spike Duplicate
PF	Prep Factor
RD	Relative Difference
RL	Reporting Limit (The RL is the lowest calibration standard in a multipoint calibration.)
RPD	Relative Percent Deviation
RRT	Relative Retention Time
SPK Val	Spike Value
SPKRef Val	Spike Reference Value
SPLP	Synthetic Precipitation Leachate Procedure
ST	Sorbent Tube
TCLP	Toxicity Characteristic Leachate Procedure
TEQ	Toxicity Equivalents
WET (STLC)	Waste Extraction Test (Soluble Threshold Limit Concentration)

Analytical Qualifiers

a3	Sample diluted due to high organic content.

Analytical Sciences
P.O. Box 750336, Petaluma, CA 94975-0336
110 Liberty Street, Petaluma, CA 94952
(707) 769-3128
Fax (707) 769-8093

CHAIN OF CUSTODY

1707577

SUBCONTRACT LAB: **McCampbell Analytical**
AS Project Name: Carollo - Coloma Pump Station
AS Project Number: 7071401

CLIENT INFORMATION	
Company Name:	**ANALYTICAL SCIENCES**
Address:	**P.O. BOX 750336**
	PETALUMA, CA 94975-0336
Contact:	**Michele Peters**
Phone #:	**707-769-3128**
Fax #:	**707-769-8093**

email: analytical110@aol.com

TURNAROUND TIME (check one)			
Same Day		72 Hours	
48 Hours		24 Hours	
5 Days		Normal	X

EDT Required: **YES** **NO**
System ID:

Page 1 of 1

Item	Client Sample ID	Date Sampled	Time	Matrix	# Cont.	Presv. Y/N	EDB by 504	HAA by 552	8270	8081	8082	Cyanide	624	625	8141	8151				Comments	Lab Sample #
1	B1-3.5', 5.5', 10.5', 15.5 B2-3.5', 6', 11.5', 15.5'	7/12/17	1000	Soil	1	N			X											7071401	-01
2																					
3																					
4																					
5																					
6																					
7																					
8																					
9																					
10																				**Normal TAT**	

SIGNATURES

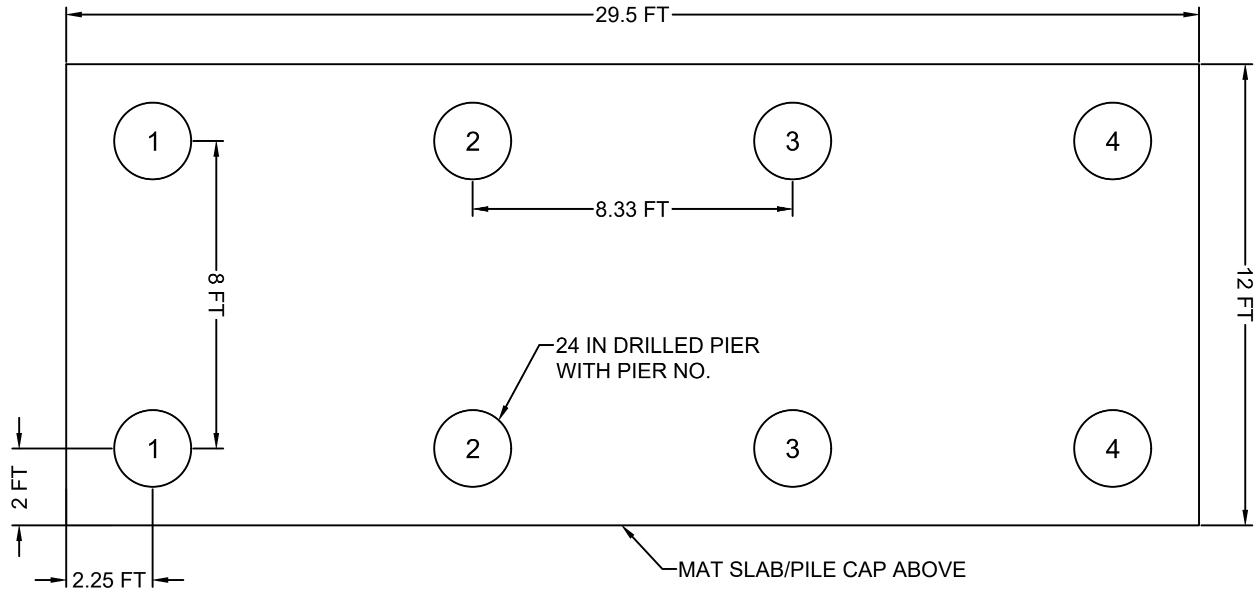
Relinquished By: Signature	Date	Time	Received By: Signature	Date	Time
[signature]	7/17	850	[signature]	7-21-17	0830
[signature]	7-17-17	160	[signature]	7/17/17	1600

4.2°

MILLER PACIFIC ENGINEERING GROUP

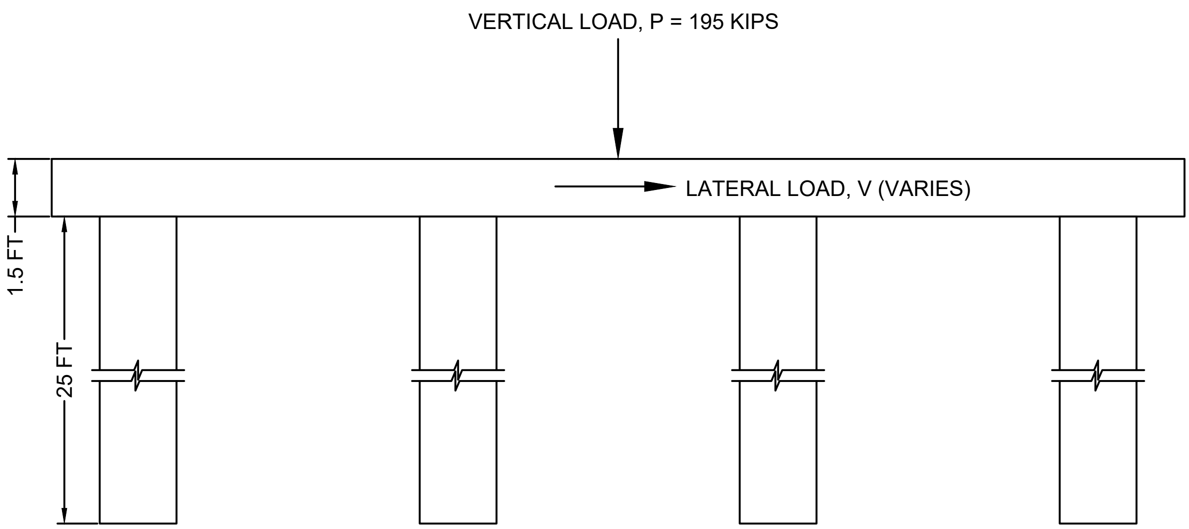
APPENDIX C

RESULTS OF LATERAL PILE ANALYSIS



EQUIPMENT PAD PLAN

(NO SCALE)



EQUIPMENT PAD SECTION

(NO SCALE)

MPEG MILLER PACIFIC ENGINEERING GROUP
504 Redwood Blvd.
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FILE: 2213.004 Pier Layout.dwg

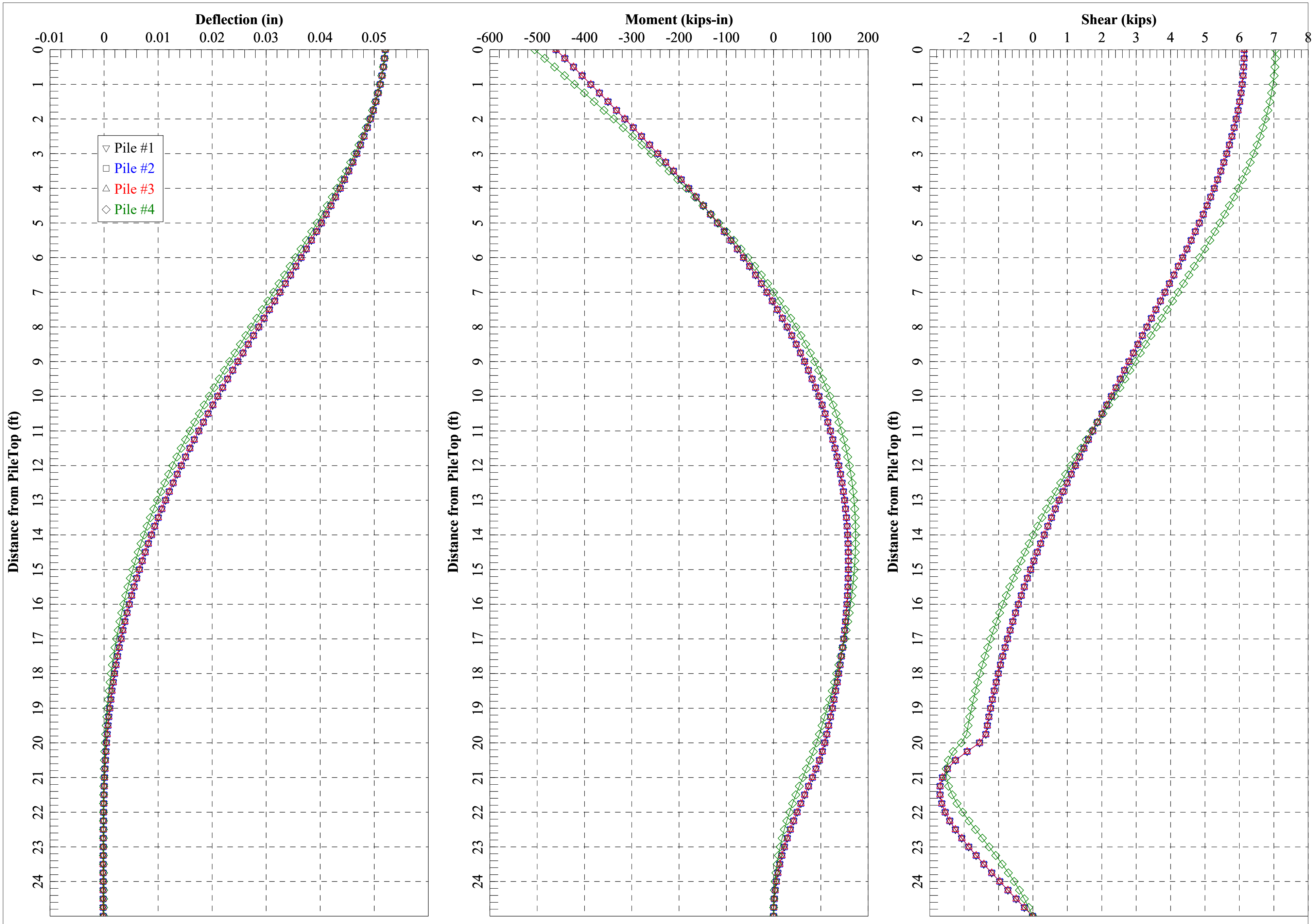
PIER LAYOUT & LOADING - EAST-WEST DIRECTION

SMCSD Coloma Pump Station
Sausalito, California

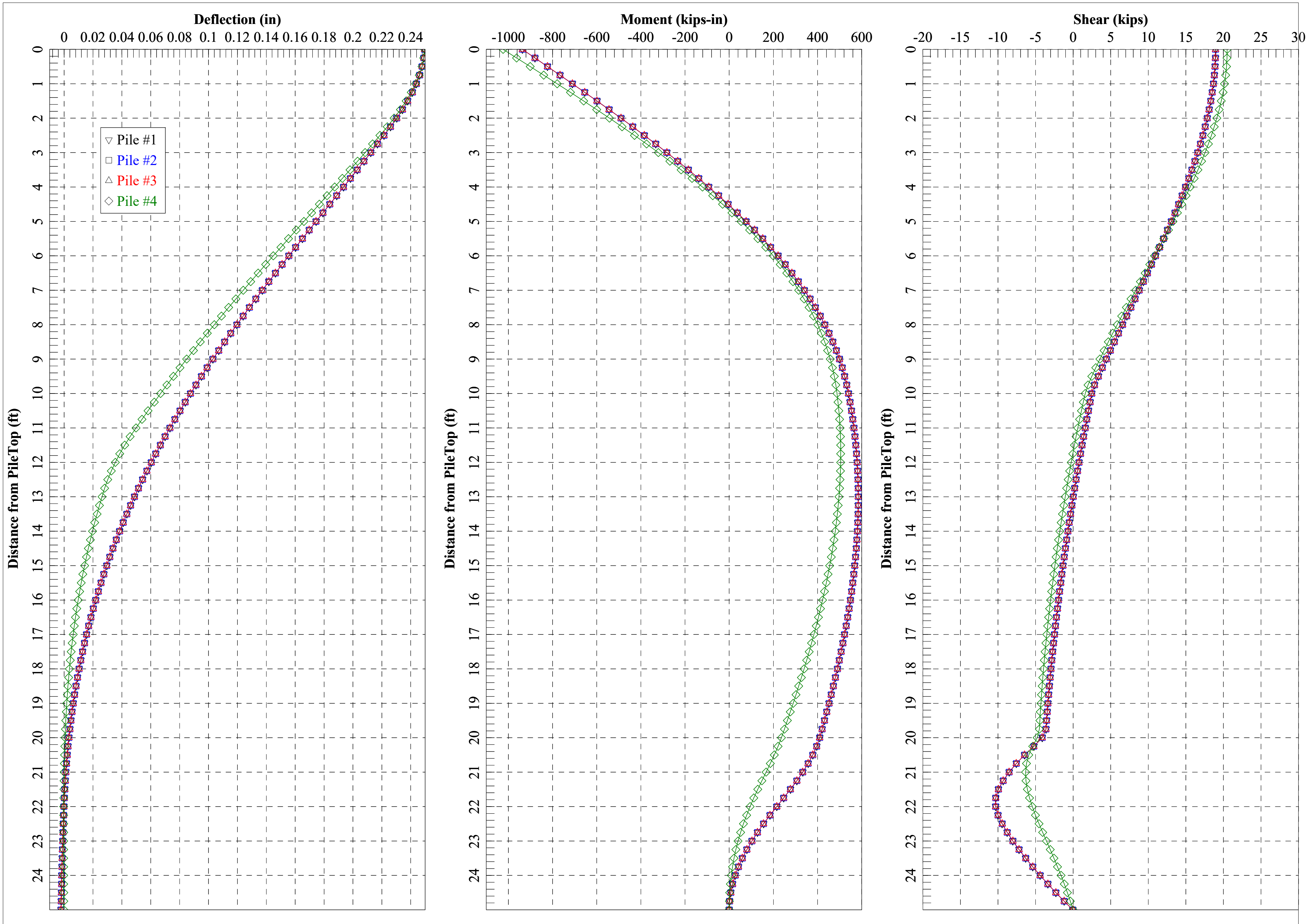
Drawn RCA
Checked

Project No. 2213.004 Date: 7/8/2019

C-1
FIGURE



24 in, E-W, P = 195 kips, V = 51 kips



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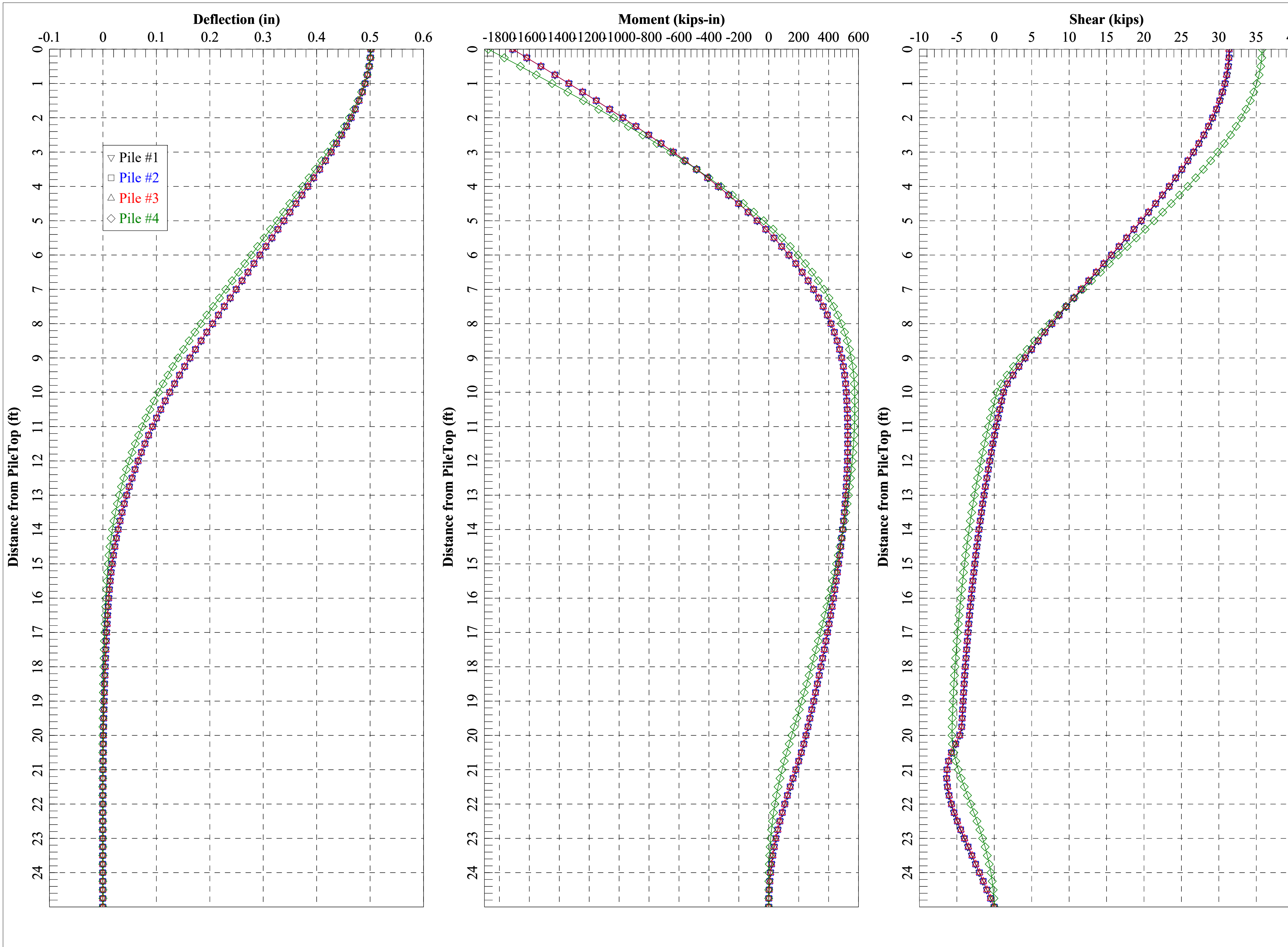

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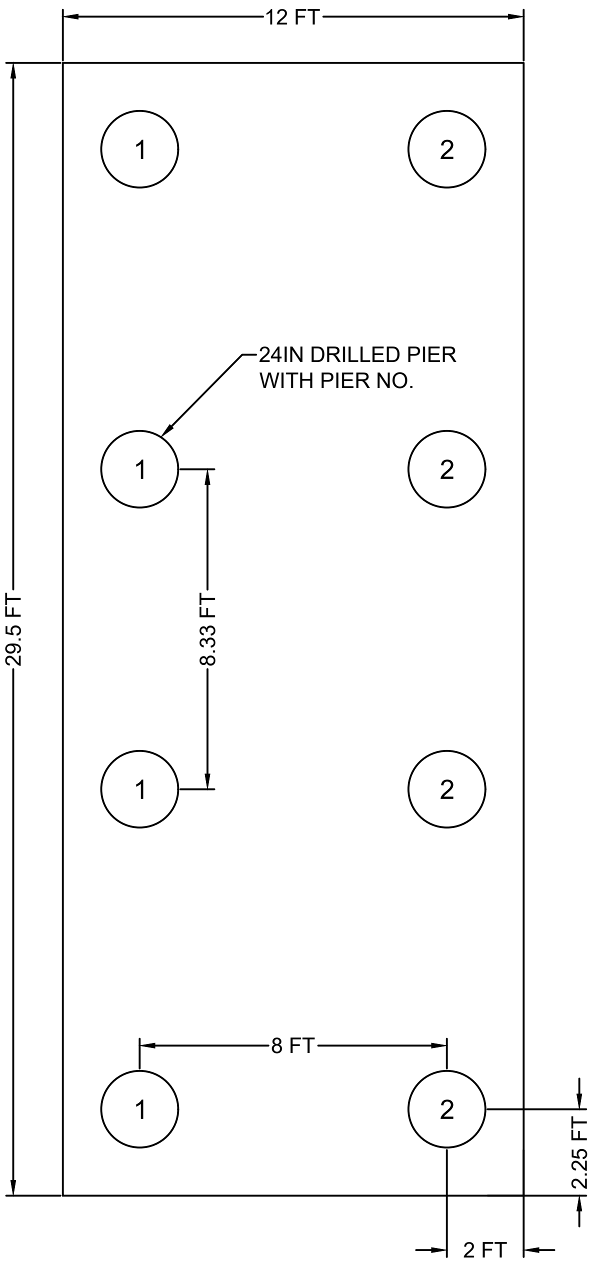
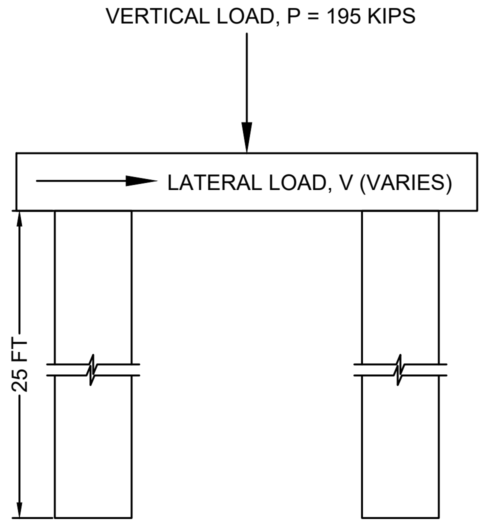


EQUIPMENT PAD PLAN
(NO SCALE)



EQUIPMENT PAD SECTION
(NO SCALE)

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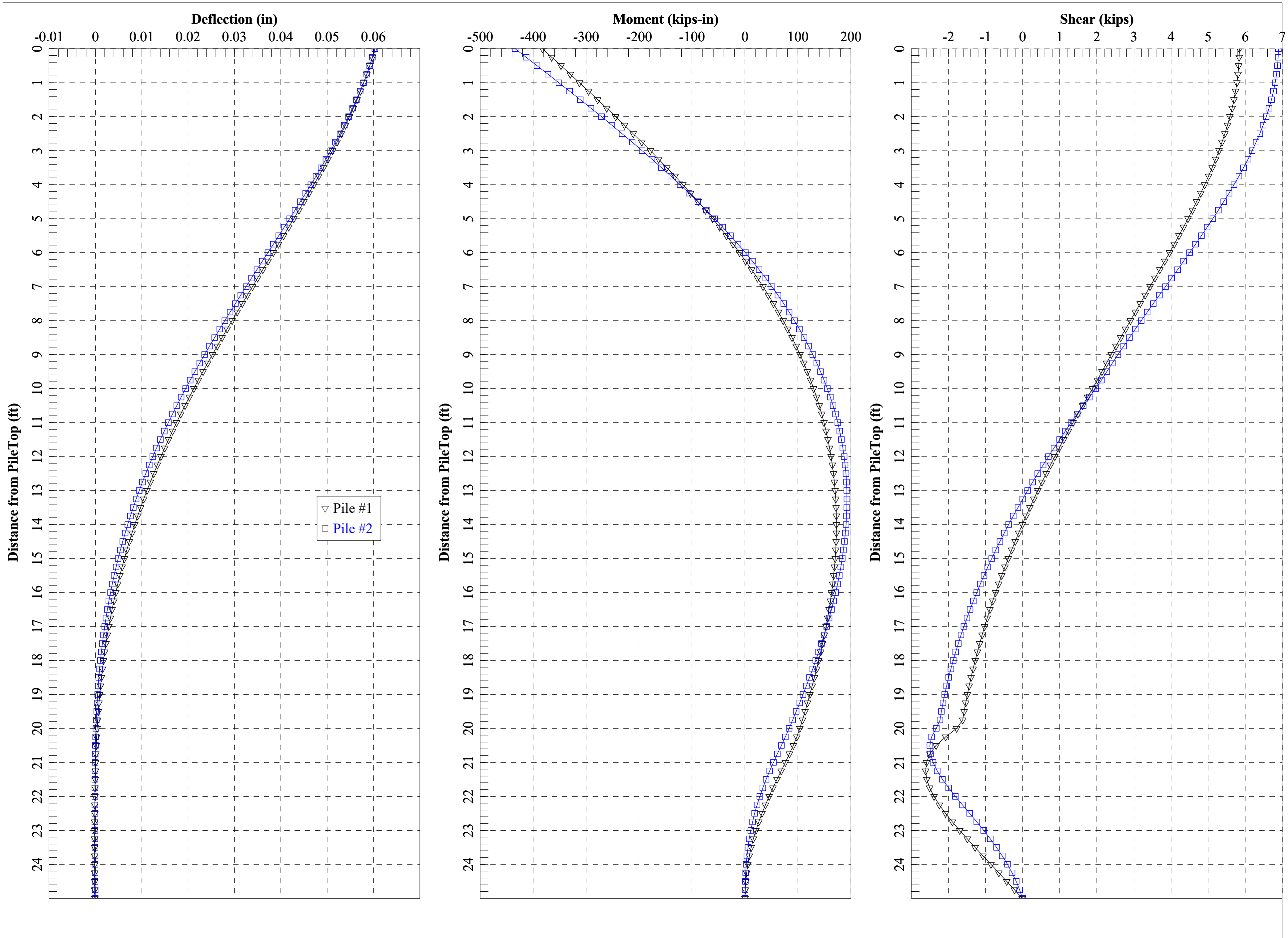
PIER LAYOUT & LOADING - NORTH-SOUTH DIRECTION

SMCSD Coloma Pump Station
Sausalito, California

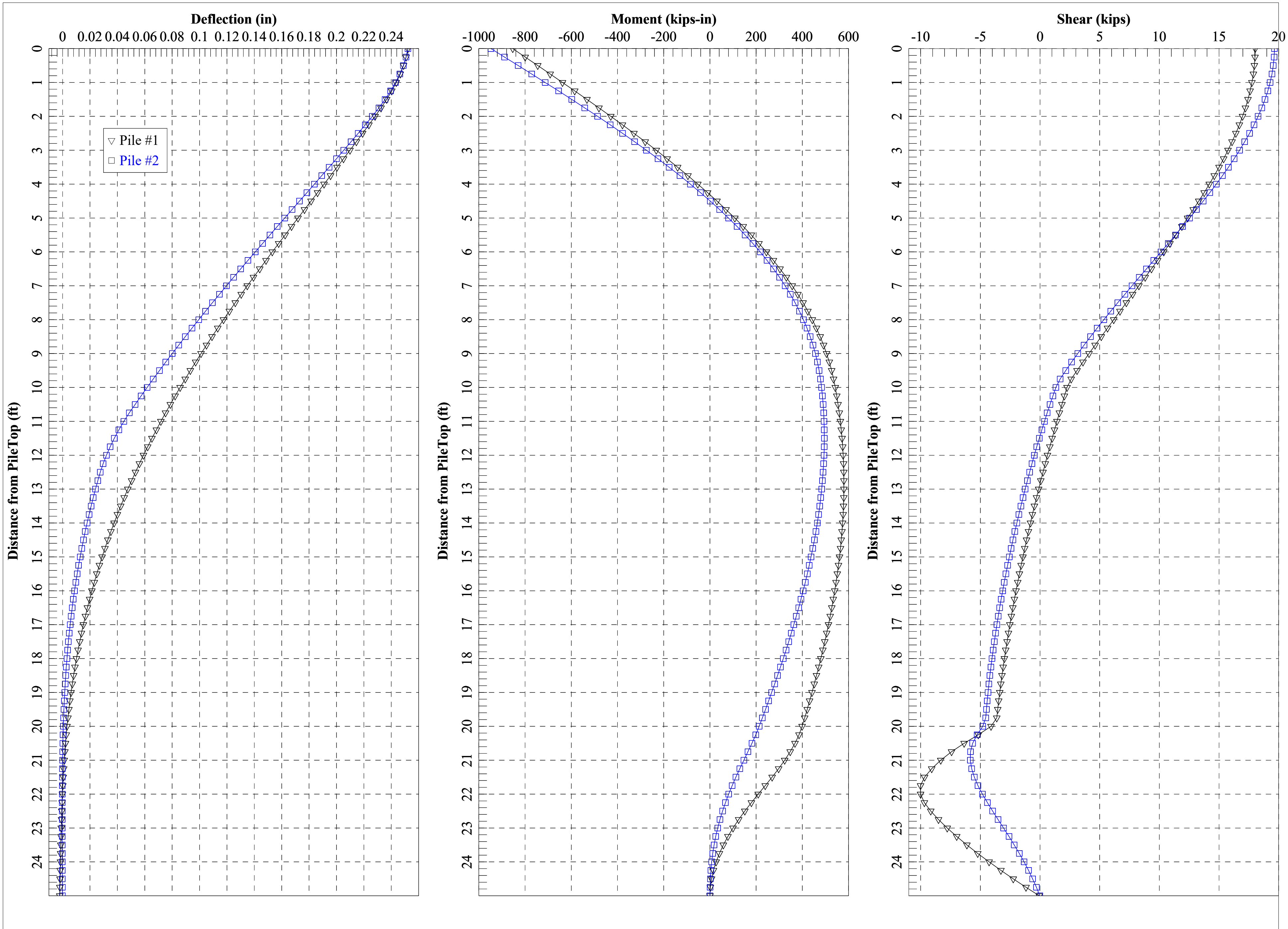
Project No. 2213.004 Date: 7/8/2019

Drawn RCA
Checked

C-2
FIGURE



24 in, N-S, P 195 kips, V = 51 kips



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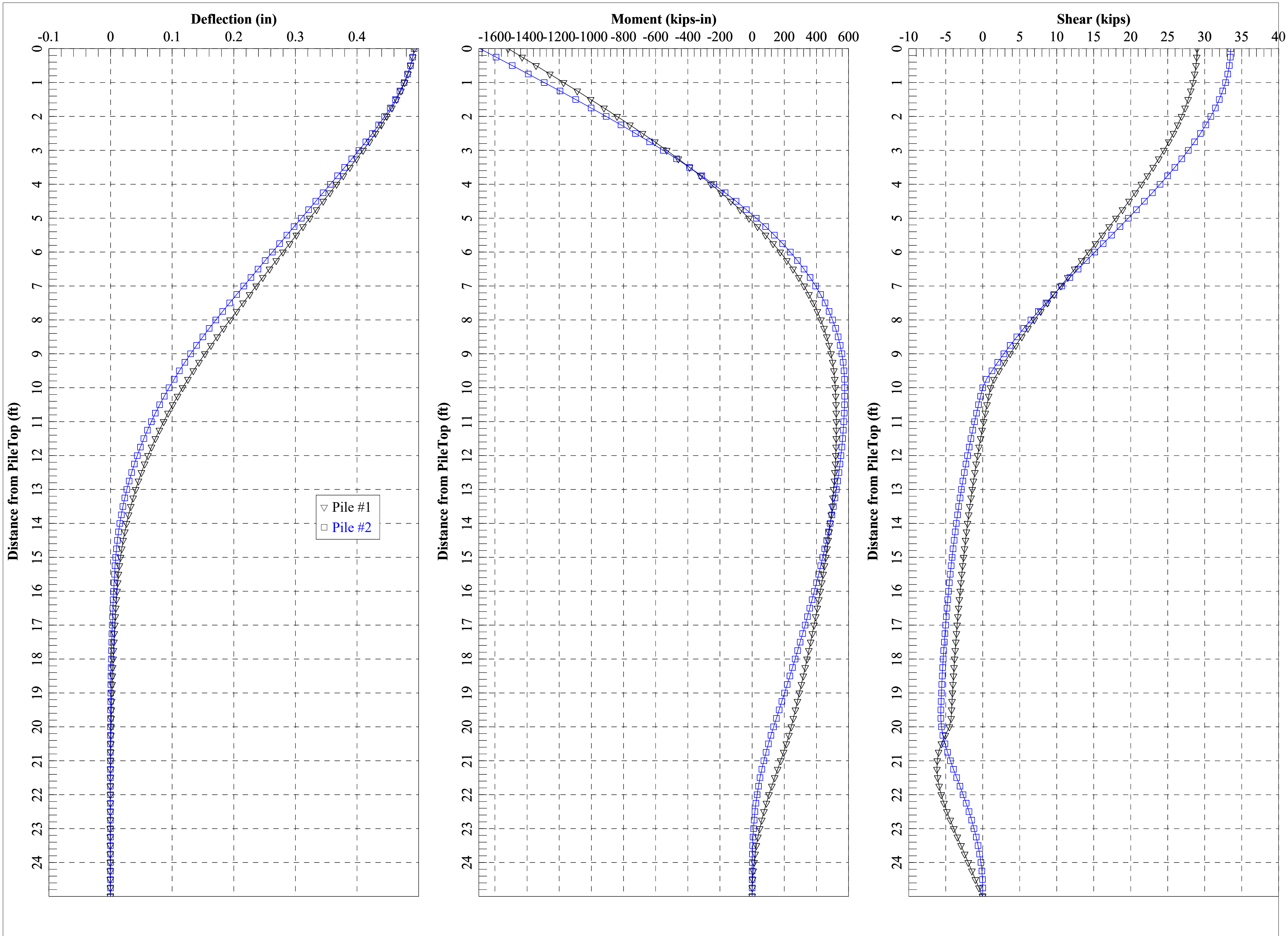

24 in, N-S, P = 195 kips, V = 250 kips

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MILLER PACIFIC
ENGINEERING GROUP

July 7, 2020
File: 2213.004altr.doc

Carollo Engineers, Inc.
2700 Ygnacio Valley Road, Suite 300
Walnut Creek, California 94598

Attn: James Wickstrom

Re: Addendum No. 1
2019 CBC Seismic Design Criteria
SMCSD Coloma Pump Station
Sausalito, California

This Addendum No. 1 to our Geotechnical Investigation Report dated July 5, 2019[1] provides updated seismic design criteria for the Sausalito Marin City Sanitary District's Coloma Pump Station planned near the intersection of Coloma Street and Bridgeway in Sausalito. The site is located close to several active faults, including the San Andreas and San Gregorio Faults which are located approximately 10.1 kilometers (6.3 miles) and 13.6 kilometers (8.5 miles) southwest of the site, respectively. The seismic design criteria provided in our report was based upon the 2016 California Building Code (CBC) and ASCE 7-10 which were the governing codes at the time our report was issued. We understand the new structures will be designed using the 2019 CBC and ASCE 7-16. Therefore, updated criteria based on the current codes are presented herein.

The site is underlain by roughly ten feet of artificial fill over Bay Mud. Moderately to highly weathered sandstone was encountered beneath the Bay Mud at about 20 feet below ground surface. Based on these subsurface conditions and the proximity to several active faults, the site is classified as "Site Class E". We performed a site-specific ground motion hazard analysis in general accordance with Section 21.2 of ASCE 7-16. The results of our analysis are presented on Figures 1 to 3 and the recommended seismic design criteria is summarized in Table 1.

[1] Miller Pacific Engineering Group, "Geotechnical Investigation, Sausalito Marin City Sanitary District, Coloma Pump Station, Sausalito, California", July 5, 2019.

Table 1 – 2019 California Building Code Seismic Design Criteria

Parameter	Design Value
Site Class	E
Risk Category	III
Site Latitude	37.868°N
Site Longitude	-122.503°W
Spectral Acceleration at Short Periods, S_S	1.50 g
Spectral Acceleration at Period of 1 sec, S_1	0.60 g
Adjusted Spectral Acceleration at Short Periods, S_{MS}	1.56 g
Adjusted Spectral Acceleration at Period of 1 sec, S_{M1}	3.39 g
Design Spectral Acceleration at Short Periods, S_{DS}	1.04 g
Design Spectral Acceleration at Period of 1 sec, S_{D1}	2.26 g
Adjusted MCE_G Peak Ground Acceleration, PGA_M	0.67 g

We appreciate the opportunity to be of continued service on this project. If we can be of further assistance or should there be any questions or concerns regarding our recommendations, please call.

Very truly yours,
MILLER PACIFIC ENGINEERING GROUP

REVIEWED BY:

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

Project Name: SMCSD - Coloma Pump Station
Project Numb: 2213.004

Latitude: 37.8680
Longitude: -122.5030

General Seismic Parameters
ASCE 7-16 Section 11.4

Site Class:	E
S_S (g):	1.50
S_1 (g):	0.60
F_a:	N/A
F_v:	N/A
T_L (sec):	12.0
C_{RS}:	0.92
C_{R1}:	0.90

Minimum Design Spectra Parameters
ASCE 7-16 Section 21.3

Site Class:	E	S_{MS} (g):	1.50
S_S (g):	1.50	S_{M1} (g):	2.40
S_1 (g):	0.60	S_{DS} (g):	1.00
F_a:	1.00	S_{D1} (g):	1.60
F_v:	4.00	T_0 (sec):	0.32
		T_S (sec):	1.60

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

Fa:	1.00
1.2 x Fa (g):	1.20
Max PSHA (g):	2.14
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):	1.72
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	0.73	1.10	0.80	0.916	0.74
0.10	1.15	1.10	1.27	0.916	1.16
0.20	1.53	1.10	1.68	0.916	1.54
0.30	1.81	1.13	2.04	0.914	1.86
0.50	2.01	1.18	2.36	0.910	2.14
0.75	1.80	1.24	2.23	0.905	2.02
1.00	1.67	1.30	2.17	0.900	1.95
2.00	1.24	1.35	1.67	0.900	1.50
3.00	0.90	1.40	1.26	0.900	1.13
4.00	0.66	1.45	0.96	0.900	0.86
5.00	0.50	1.50	0.75	0.900	0.68

Deterministic MCE
NGA West2 2014 - 84th Percentile

Period (sec)	Sa_{RotD50} (g)	$Sa_{RotD100}$ / Sa_{RotD50}	$Sa_{RotD100}$ (g)
0.01	0.56	1.10	0.61
0.02	0.54	1.10	0.59
0.03	0.54	1.10	0.59
0.05	0.54	1.10	0.60
0.08	0.62	1.10	0.69
0.10	0.72	1.10	0.79
0.15	0.87	1.10	0.96
0.20	1.00	1.10	1.10
0.25	1.12	1.11	1.25
0.30	1.25	1.13	1.41
0.40	1.40	1.15	1.61
0.50	1.46	1.18	1.72
0.75	1.37	1.24	1.69
1.00	1.32	1.30	1.72
1.50	1.20	1.33	1.59
2.00	1.07	1.35	1.44
3.00	0.79	1.40	1.11
4.00	0.58	1.45	0.85
5.00	0.44	1.50	0.66
7.50	0.22	1.50	0.33
10.00	0.12	1.50	0.18

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

Period (sec)	Sa (g)
0.01	0.53
0.02	0.51
0.03	0.52
0.05	0.52
0.08	0.60
0.10	0.69
0.15	0.83
0.20	0.96
0.25	1.09
0.30	1.23
0.40	1.41
0.50	1.50
0.75	1.48
1.00	1.50
1.50	1.39
2.00	1.26
3.00	0.97
4.00	0.74
5.00	0.57
7.50	0.29
10.00	0.16

Site Specific MCE_R
ASCE 7-16 Section 21.2.3

Period (sec)	Sa (g)
0.01	0.61
0.02	0.59
0.03	0.59
0.05	0.60
0.08	0.69
0.10	0.79
0.15	0.96
0.20	1.10
0.25	1.25
0.30	1.41
0.40	1.61
0.50	1.72
0.75	1.69
1.00	1.72
1.50	1.59
2.00	1.44
3.00	1.11
4.00	0.85
5.00	0.66
7.50	0.33
10.00	0.18

Site-Specific Design Spectrum
ASCE 7-16 Section 21.3

Period (sec)	Sa (g)
0.01	0.41
0.02	0.39
0.03	0.39
0.05	0.40
0.08	0.46
0.10	0.53
0.15	0.64
0.20	0.73
0.25	0.83
0.30	0.94
0.40	1.08
0.50	1.15
0.75	1.13
1.00	1.14
1.50	1.06
2.00	0.96
3.00	0.74
4.00	0.56
5.00	0.44
7.50	0.22
10.00	0.12

80% General Response Spectrum
ASCE 7-16 Section 21.3

	Period (sec)	Sa (g)	80% Sa (g)
	0.01	0.42	0.34
	0.06	0.52	0.41
	0.11	0.61	0.49
	0.17	0.71	0.57
	0.22	0.81	0.65
	0.27	0.90	0.72
T_O =	0.32	1.00	0.80
T_S =	1.60	1.00	0.80
	1.86	0.86	0.69
	2.12	0.75	0.60
	2.38	0.67	0.54
	2.65	0.60	0.48
	2.91	0.55	0.44
	3.17	0.50	0.40
	3.43	0.47	0.37
	3.69	0.43	0.35
	3.95	0.40	0.32
	4.22	0.38	0.30
	4.48	0.36	0.29
	4.74	0.34	0.27
	5.00	0.32	0.26

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ASCE 7-16 MCEr CALCULATIONS

SMCSD Coloma Pump Station
Sausalito, California

Project No. 2213.004 Date: 7/1/2020

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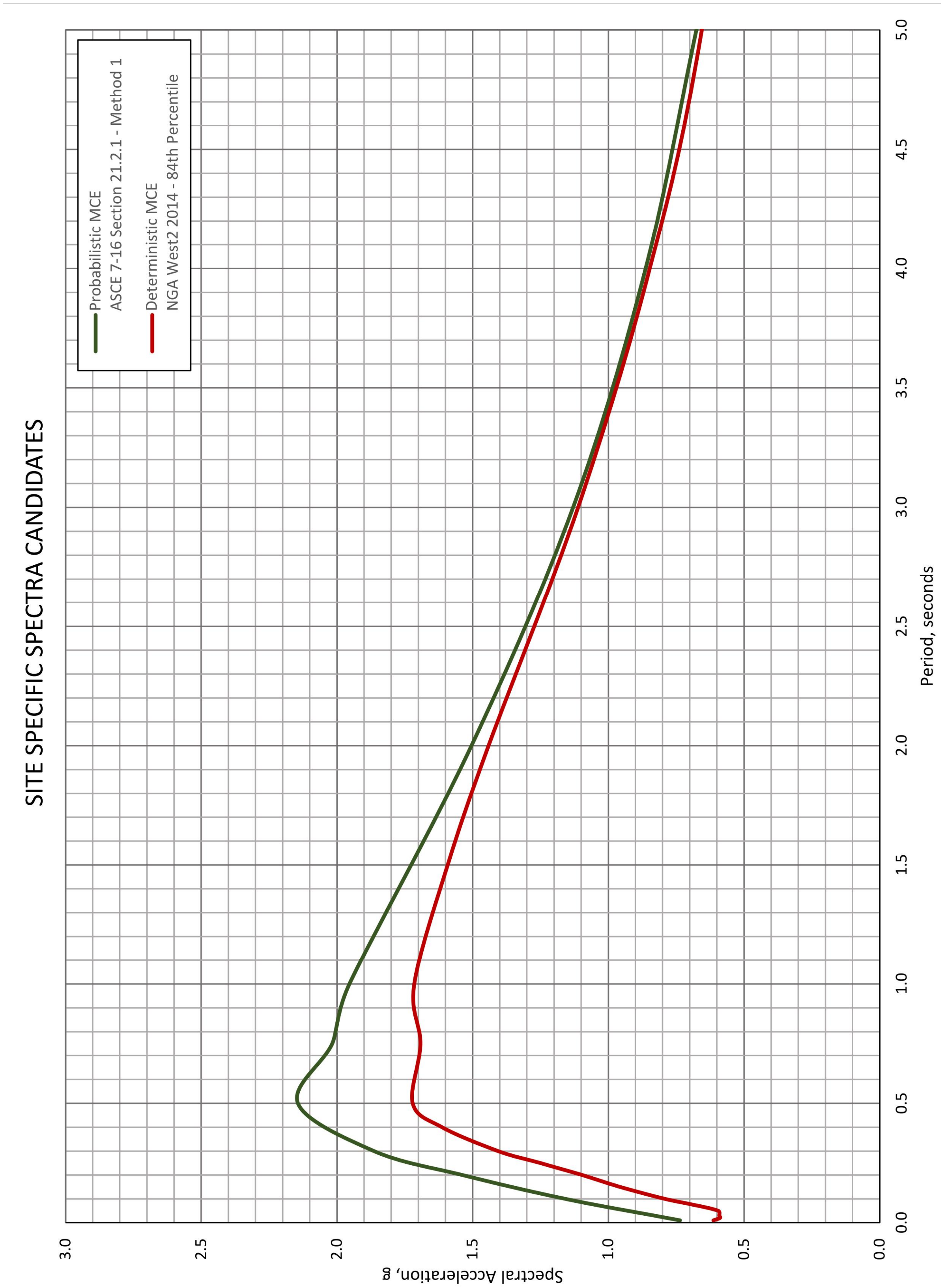
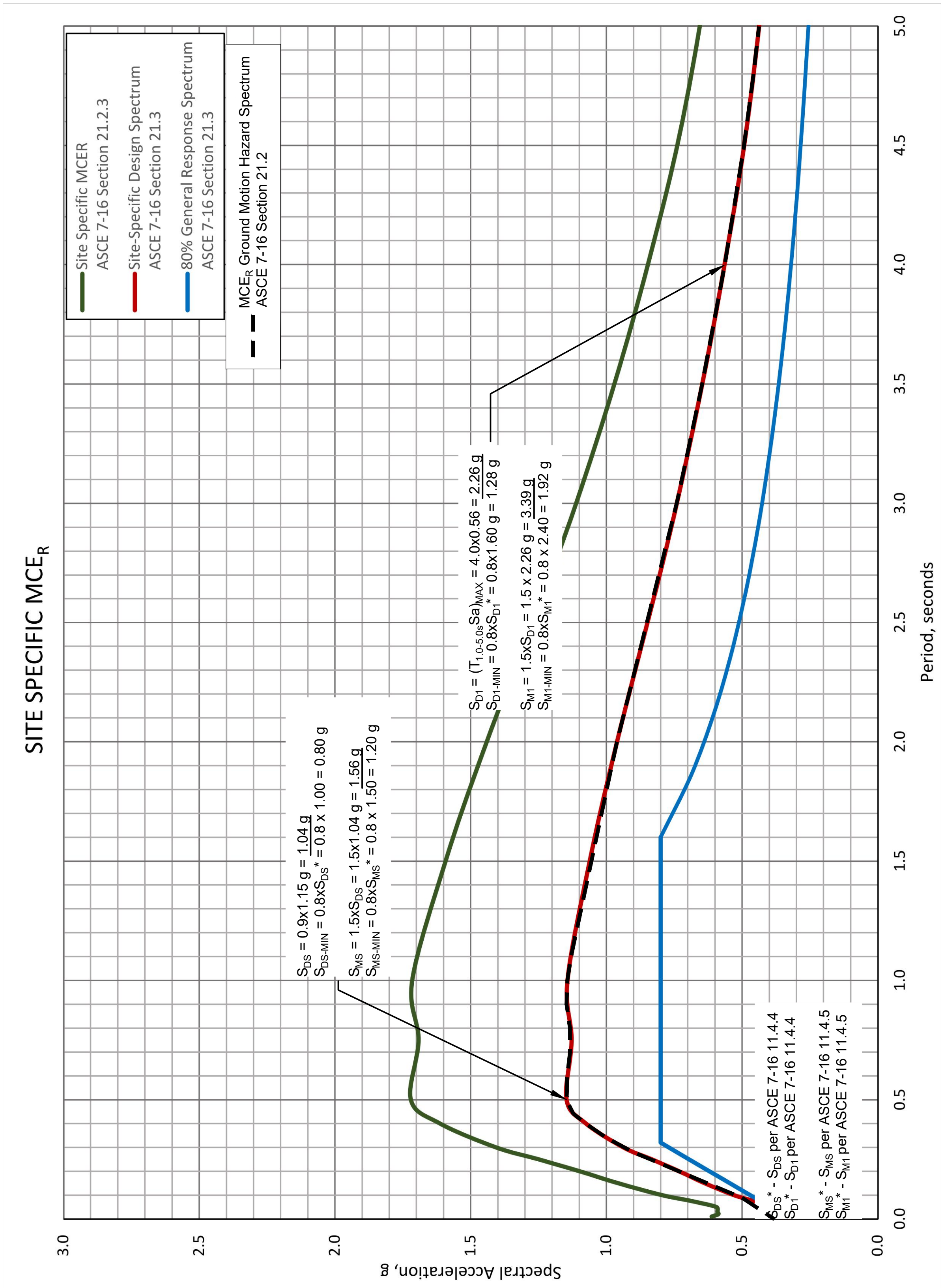
FIGURE 1



MPEG MILLER PACIFIC ENGINEERING GROUP	504 Redwood Blvd. Suite 220 Novato, CA 94947 T 415 / 382-3444 F 415 / 382-3450 www.millerpac.com	ASCE 7-16 MCEr CANDIDATE SPECTRA SMCSD Coloma Pump Station Sausalito, California Project No. 2213.004 Date: 7/1/2020	Drawn RCA Checked 2 FIGURE

FILE: 2213.004 2019 CBC Seismic Design Parameters.dwg



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ASCE 7-16 MCEr DESIGN SPECTRUM

SMCSD Coloma Pump Station
Sausalito, California

Project No. 2213.004 Date: 7/1/2020

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3
FIGURE