CORNERSTONE
EARTH GROUP

TYPE OF SERVICES	Geotechnical Investigation
PROJECT NAME	MCWD A1/A2 Reservoirs and B/C Booster Pump Station
LOCATION	8th Street and 6th Avenue Marina, California
CLIENT	Schaaf and Wheeler
PROJECT NUMBER	187-55-1
DATE	December 29, 2020

GEOTECHNICAL

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TABLE OF CONTENTS

Type of Services	**Geotechnical Investigation**
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SECTION 1: INTRODUCTION

This geotechnical report was prepared for the sole use of Schaaf and Wheeler for the MCWD A1/A2 Reservoirs and B/C Booster Pump Station project located within the California State Monterey Bay campus near 8th Street and 6th Avenue in Marina, California. The location of the site is shown on the Vicinity Map, Figure 1 and Site Plan, Figure 2. For our use, we were provided with the following documents:

- A set of plans titled “Plans for the A1/A2 Reservoirs and B/C Booster Pump Station, Marina Coast Water District, CIP NO. GW – 0112,” prepared by Schaaf & Wheeler, dated September 9, 2020.
- A topographic map, Sheets 1 and 2, titled “Marina Coast Water District ‘A’ Reservoirs, Proposed Tank Site & Pipeline Alignment, Topographic Map, CSUMB, Marina, California,” prepared by Whitson Engineers, issued as draft for review only dated July 3, 2019.

As you know, in 2007 and 2008 the proposed reservoir tanks and pump station project had begun and were to be located east of the current proposed location. At that time, Cornerstone Earth Group performed a geotechnical investigation and presented our findings in a report titled “Geotechnical Investigation, MCWD A1/A2 Reservoir & B/C Booster Pump Station, 8th Street Cut-Off and 6th Ave, Marina, California,” dated July 19, 2007. This report has been updated to reflect changes to the project location and required design criteria.

1.1 PROJECT DESCRIPTION

The project site is within the California State University Monterey Bay campus located near 8th Street and 6th Avenue in Marina, California. The project will include two potable water storage tanks (A1/A2 Reservoirs) and a new pump station (B/C Booster Pump Station) to pump water from the A1/A2 Reservoirs to the existing B and C pressure zone reservoirs and distribution systems. Currently, both water storage tanks are planned to hold about 2 million gallons. Pipeline modifications and sequencing to the existing A, B, and C zone transmission pipelines and additional pipelines and appurtenances as required for replacing the existing B/C Booster Station and Sand Tank for a complete and operable storage and pumping system are also planned.

The planned reservoir tanks are to be ground supported, welded carbon steel designed and constructed in accordance with the American Water Works Association (AWWA) D100-11 standards. The tanks are to be constructed within the upper level of the site and are currently to have a bottom of tank/top of foundation Elevation 199 feet (NAVD88 datum). The tanks are to have a 114-foot diameter, have a top of roof Elevation 234.9 feet at the roof vent, have a high water surface Elevation 223.8 feet (corresponding up to a 24.8-foot height of water in each tank), and have a freeboard Elevation 231.4 feet. The tank walls are to be supported by a foundation around the outside ring of each tank and depending on the tank designer a foundation beneath the column in the center of each tank. Each tank steel bottom between the ring foundation and center column foundation is to be supported directly on the subgrade soils beneath.

The planned pump station is to be one-story, concrete/CMU frame construction, and designed using the 2019 California Building Code (CBC) standards. The pump station will be constructed into the existing slope between the upper and lower levels of the site with the top of pump station (roof) at Elevation 207 feet and the finish floor elevation at Elevation 190 feet (NAVD88 datum). We understand the building will be supported by a 1-foot and a 1-foot 7-inch-thick mat foundation with subgrade at about Elevation 188 to 189 feet.

The primary pipeline modifications in proximity to the proposed pump station will be installed within a pipe easement located within the sloped area between the upper and lower levels of the site. Other appurtenant utilities, landscaping and improvements necessary for site development are also planned.

Structural loads were provided by the Structural Engineer, TJC and Associates, Inc. for the proposed tanks and pump station. For the tanks, the global interior dead + fluid load is 1,575 pounds per square foot (psf). The perimeter footing loading is 1,950 psf (dead + roof live load + fluid) and 2,550 psf (dead + fluid + seismic). The interior center column load is 11.2 kips (dead + roof live load). For the pump station, the wall loads are about 1.8 kips per lineal foot and the average contact pressure beneath the mat foundation will be about 800 to 1000 psf. Grading is anticipated to include cuts and fills on the order of 2 to 4 feet for construction of the tanks. Grading for the pump station pad will include cuts up to about 10 to 12 feet and cuts up to about 10 feet for installation of the new distribution pipeline is anticipated.

An existing detention basin, located less than a ¼ mile to the northwest of the proposed tanks and pump station, is planned to be utilized for retention of tank water in the event the tanks would need to be emptied.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated April 11, 2019 and consisted of field and laboratory programs to evaluate physical and engineering properties of the subsurface soils, engineering analysis to prepare recommendations for site work and grading, tank and building foundations, lateral earth pressures for retaining wall design, temporary shoring, flatwork, and pavements, and preparation of this report. Brief descriptions of our exploration and laboratory programs are presented below.

1.3 EXPLORATION PROGRAM

Field exploration consisted of four borings drilled on June 26, 2019 with truck-mounted, hollow-stem auger drilling equipment. The borings were drilled to depths ranging from 21½ to 51½ feet. The borings were backfilled with cement grout in accordance with local requirements; exploration permits were obtained as required by local jurisdictions.

The approximate locations of our exploratory borings are shown on the Site Plan, Figure 2. Details regarding our field program are included in Appendix A.

1.3.1 Previous Exploration Program

Our 2007 field exploration consisted of five borings drilled on June 13 and 18, 2007, using hollow-stem auger drilling equipment. The borings were drilled to depths ranging from approximately 20 to 76½ feet. The approximate locations of our 2007 borings are shown on the Site Plan, Figure 2. Exploration logs from our 2007 borings are included in Appendix C.

1.4 LABORATORY TESTING PROGRAM

In addition to visual classification of samples, the laboratory program focused on obtaining data for foundation design and seismic ground deformation estimates. Testing included moisture contents, dry densities, and washed sieve analyses. Details regarding our laboratory program are included in Appendix B.

1.5 CORROSION EVALUATION

Two samples from our borings at depths of 1½ and 3½ feet were tested for saturated resistivity, pH, and soluble sulfates and chlorides. The results are presented in Appendix B. Additionally, four samples from our 2007 borings at depths of 1 to 4½ feet were tested as above. Results from these tests are presented in Appendix C. In general, the on-site soils can be characterized as very mildly corrosive to buried metal, and non-corrosive to buried concrete.

1.6 ENVIRONMENTAL SERVICES

Environmental services were not requested for this project. If environmental concerns are determined to be present during future evaluations, the project environmental consultant should review our geotechnical recommendations for compatibility with the environmental concerns.

SECTION 2: REGIONAL SETTING

2.1 GEOLOGICAL SETTING

The site is located within the coast range geomorphic province of central California. Throughout the Cenozoic Era central California has been affected by tectonic forces associated with lateral or transform plate motion between the North American and Pacific crustal plates, producing a complex system of northwest-trending faults - the San Andreas Fault system (Page, 1998). Uplift, erosion and subsequent re-deposition of sedimentary rocks within this province have been driven primarily by the northwest directed, strike-slip movement of the tectonic plates and the associated northeast oriented compressional stress. The northwest-trending coastal mountain ranges are the result of an orogeny (formation of mountains by the process of tectonic uplift) believed to have been occurring since the Pleistocene epoch (approximately 2-3 million years before present).

The portion of the Monterey Bay area where the site exists is within the Salina Block, which is bound by the San Andreas Fault on the east, and by the San Gregorio - Palo Colorado Fault to the west. The Salina block is composed of an elongate prism of granites and metamorphic rock types. The Salina basement complex is overlain primarily by marine sedimentary rocks of tertiary age and terrestrial rocks of Pliocene to Pleistocene age, and modern dune and alluvial deposits.

2.2 REGIONAL SEISMICITY

The San Francisco Bay area region is one of the most seismically active areas in the Country. While seismologists cannot predict earthquake events, geologists from the U.S. Geological Survey have recently updated earlier estimates from their 2015 Uniform California Earthquake Rupture Forecast (Version 3) publication. The estimated probability of one or more magnitude 6.7 earthquakes (the size of the destructive 1994 Northridge earthquake) expected to occur somewhere in the San Francisco Bay and Monterey Bay areas has been revised (increased) to 72 percent for the period 2014 to 2043 (Aagaard et al., 2016). The faults in the region with the highest estimated probability of generating damaging earthquakes between 2014 and 2043 are the Hayward (33%), Rodgers Creek (33%), Calaveras (26%), and San Andreas Faults (22%). In this 30-year period, the probability of an earthquake of magnitude 6.7 or larger occurring is 22 percent along the San Andreas Fault and 33 percent for the Hayward or Rodgers Creek Faults. As seen with damage in San Francisco and Oakland due to the 1989 Loma Prieta earthquake that was centered about 50 miles south, significant damage can occur at considerable distances. Higher levels of shaking and damage would be expected for earthquakes occurring at closer distances.

The faults considered capable of generating significant earthquakes are generally associated with the well-defined areas of crustal movement, which trend northwesterly. The table below presents the State-considered active faults within 25 kilometers of the site.

Table 1: Approximate Fault Distances

Fault Name	Distance	
	(miles)	(kilometers)
Rinconada	2.5	4.1
Monterey Bay - Tularcitos	5.7	9.1
San Gregorio	13.9	22.3
Zayante-Vergeles	14.8	23.8

A regional fault map is presented as Figure 3, illustrating the relative distances of the site to significant fault zones.

SECTION 3: SITE CONDITIONS

3.1 SITE BACKGROUND AND SURFACE DESCRIPTION

The proposed reservoir tanks and booster pump station project will be constructed on the former Fort Ord military base and is currently within the California State University Monterey Bay campus. The project area was formerly developed by the military and the previous structures have been demolished leaving an undeveloped area.

The proposed site for the reservoir tanks and booster pump station can be broken into two levels separated by a landscape berm. The upper and lower levels are generally open asphalt concrete areas, which serve as parking areas as well as storage areas for stockpiled debris and soil. The landscape berm is generally covered with ice plant, some large shrubs, and some large mature trees.

The upper level slopes downward from the southwest to the northeast from approximate Elevation 201 feet to Elevation 195 feet (NAVD88 datum). The landscape berm initially rises slightly about a foot above the upper level and then slopes down about 14 to 15 feet to the lower level at approximate Elevation 181 to 182 feet at the toe of slope. The berm slopes generally have an inclination of 2:1 (horizontal:vertical) or flatter.

The proposed reservoir tanks are to be located within the upper level and the pump station is to be located within the landscape berm adjacent to the northern end of the upper level. The utility corridor and pipe easement for the primary pipeline modifications in proximity to the pump station runs east-west within the northern side of landscaping berm adjacent to the lower level of the site.

Surface pavements within the upper and lower level generally consisted of 2 to 3 inches of asphalt concrete over 4 to 6 inches of aggregate base. Based on visual observations, the existing pavements are in poor to moderate shape with areas of significant alligator cracking.

3.1.1 Existing Detention Basin

The existing detention basin, which is to be utilized for dispersion of the reservoir tanks water in the event of a tank failure, is located less than a ¼ mile to the northwest of the proposed tanks and pump station and is bounded by 5th Avenue to the west, 8th Street to the north/northeast, and an abandoned road to the south/southeast. The bottom of existing basin extends about 10 to 15 feet below the surrounding roadways and is generally covered with ice plant, shrubs, weeds, and a few mature trees. The existing detention basin is shown on Figure 5.

3.2 SUBSURFACE CONDITIONS

Borings EB-1 and EB-2 were drilled in the lower parking lot to the north of the landscape berm. Boring EB-1 was near the toe of the landscape berm slope, adjacent to the north side of the proposed pump station. Boring EB-2 was offset slightly from the toe of the landscape berm and was adjacent to the pipe easement east of the proposed pump station. Below the surface pavement, Boring EB-1 encountered dense poorly graded sand to a depth of 9 feet, underlain by medium dense poorly graded sand with silt to a depth of 11 feet, underlain by dense poorly graded sand with silt to a depth of 17 feet, underlain by very dense poorly graded sand to the maximum depth explored of 30 feet beneath the surface. Below the surface pavement, Boring EB-2 encountered dense poorly graded sand to the maximum depth explored of 21½ feet below the surface.

Borings EB-3 and EB-4 were drilled in the upper parking lot to the south of the landscape berm. Boring EB-3 was adjacent to the south side of the proposed pump station and slightly north of the northern reservoir tank. Boring EB-4 was within the proposed footprint of the southern reservoir tank near the southern side of the tank. Below the surface pavement, Boring EB-3 encountered undocumented fill consisting of dense to very dense poorly graded sand to a depth of approximately 7 feet. The fill was underlain by generally medium dense poorly graded sand with silt to a depth of 17 feet, underlain by medium dense to very dense poorly graded sand to the maximum depth explored of 50 feet. Below the surface pavement, Boring EB-4 encountered undocumented fill consisting of dense silty sand and dense to very dense poorly graded sand to a depth of approximately 13 feet below the surface. The fill was underlain by medium dense to dense poorly graded sand with silt to a depth of 20 feet, underlain by loose to medium dense poorly graded sand with silt to a depth of 28 feet The poorly graded sand with silt was underlain by medium dense poorly graded sand to a depth of 32 feet, underlain by very dense poorly graded sand to a depth of 37 feet. The poorly graded sand was underlain by medium dense to dense poorly graded sand with silt to the maximum depth explored of 51½ feet. Figure 4, Cross Section A-A’, depicts the generalize soil profile in the location of the reservoir tanks and pump station.

3.2.1 Plasticity/Expansion Potential

The subsurface soils are silty and poorly graded sands with about 16 percent or less fines passing the No. 200 sieve. These soils are non-plastic and have very low expansion potential.

3.2.2 In-Situ Moisture Contents

Laboratory testing indicated that the in-situ moisture contents generally range from about 2 to 13 percent within the upper 20 feet, corresponding to about 8 percent below optimum to near the estimated laboratory optimum moisture.

3.3 GROUNDWATER

Groundwater was not encountered in our current borings drilled to a maximum depth of 51½ feet below existing grades. Additionally, groundwater was not encountered in our previous 2007 borings to a maximum depth of 76½ feet below site grades.

Groundwater levels are not currently mapped at the site by the State of California. We reviewed the GeoTracker website regarding groundwater depths in the site area. Based on our GeoTracker website search, there is no available data within the site area.

Fluctuations in groundwater levels occur due to many factors including seasonal fluctuation, underground drainage patterns, regional fluctuations, and other factors. Based on the available data, we anticipate groundwater to be at depths greater than 50 feet beneath the site.

3.4 CORROSION SCREENING

We tested two samples collected at depths of 1½ and 3½ feet for resistivity, pH, soluble sulfates, and chlorides. We also tested four samples collected at depths of 1 to 4½ feet during our 2007 investigation. The laboratory test results for our current and previous borings are summarized in Table 2A.

Table 2A: Summary of Corrosion Test Results

Sample Location	Depth (feet)	Soil pH[2]	Resistivity[3] (ohm-cm)	Chloride[4] (mg/kg)	Sulfate[5,6] (mg/kg)
EB-2	3½	7.3	45,617	7	82
EB-4	1½	6.8	20,890	6	249
EB-2[1]	1 to 2½	7.6	26,932	<2	16
EB-2[1]	3 to 4½	6.6	40,342	<2	13
EB-5[1]	1 to 2½	7.8	23,738	<2	20
EB-5[1]	3 to 4½	7.8	29,456	<2	16

Notes: [1] 2007 Boring
[2] ASTM G51
[3] ASTM G57 - 100% saturation
[4] ASTM 4327 / Cal 422-mod (2007 borings)
[5] ASTM 4327 / Cal 417-mod (2007 borings)
[6] 1 mg/kg = 0.0001 % by dry weight

Many factors can affect the corrosion potential of soil including moisture content, resistivity, permeability, and pH, as well as chloride and sulfate concentration. Typically, soil resistivity, which is a measurement of how easily electrical current flows through a medium (soil and/or water), is the most influential factor. In addition to soil resistivity, chloride and sulfate ion concentrations, and pH also contribute in affecting corrosion potential.

3.4.1 Preliminary Soil Corrosion Screening

Based on the laboratory test results summarized in Table 2A and published correlations between resistivity and corrosion potential, the soils are considered very mildly corrosive to buried metallic improvements (Chaker and Palmer, 1989).

In accordance with the 2019 CBC Section 1904.1, alternative cementitious materials for different exposure categories and classes shall be determined in accordance with ACI 318-19 Table 19.3.1.1, Table R19.3.1, and Table 19.3.2.1. Based on the laboratory sulfate test results, no cement type restriction is required, although, in our opinion, it is generally a good idea to include some sulfate resistance and to maintain a relatively low water-cement ratio. We have summarized applicable exposure categories and classes from ACI 318-19, Table 19.3.1.1 below in Table 2B.

Table 2B: ACI 318-14 Table 19.3.1.1 Exposure Categories and Classes

Freezing and Thawing (F)	Sulfate (S, soil)	In Contact with Water (W)	Corrosion Protection of Reinforcement (C)
F0[1]	S0[2]	W0[3]	C0[4]

1 (F0) “Concrete not exposed to freezing-and-thawing cycles” (ACI 318-19)
2 (S0) “Water soluble sulfate in soil, percent by mass is less than 0.10” (ACI 318-19)
3 (W0) “Concrete dry in service” (ACI 318-19)
4 (C0) “Concrete dry or protected from moisture” (ACI 318-19)

We recommend the structural engineer and a corrosion engineer be retained to confirm the above information and provide additional recommendations, as needed.

3.5 GROUNDWATER INFILTRATION WITHIN EXISTING DETENSION BASIN

3.5.1 Field Infiltration Tests

An infiltration test was performed to estimate the rate of infiltration in the soils within the bottom of the existing detention basin as shown on Figure 5. Infiltration testing was performed within this basin so that the design team can determine the feasibility of using the basin to drain the reservoir tanks in the event the tanks would need to be emptied.

One infiltration test, I-1, was performed within the bottom of the existing detention basin on June 27, 2019. The approximate location of the infiltration test is shown on Figure 5. We also performed three shallow excavations using a hand auger in proximity to our infiltration test. The shallow excavations extended to a depth of approximately 5 feet beneath the bottom of the existing basin. The soils encountered at these locations were fairly consistent with low fines contents and classified as generally poorly graded sands with silt.

The infiltration test was performed using a double-ring infiltrometer in accordance with ASTM D3385 test methods (constant head) at a depth of approximately 1 foot below the bottom of existing basin. The rings were embedded at about 6 inches below the exposed soil level, filled with approximately 4 inches of water and allowed to presoak for about 20 minutes before starting the test readings. Following presoaking, the infiltration test was conducted for approximately 1½ hours. A fairly constant infiltration rate was maintained during the last hour of testing. The test result is summarized in the following table.

Table 3: Double-Ring Infiltration Test Results

Test Location	Infiltration Rate (inches/hour)
I-1	27

The test result may not be truly indicative of the long-term, in-situ infiltration. Other factors including soil stratifications, heterogenous deposits, overburden stress, and other factors can influence infiltration results. We recommend that an appropriate factor of safety be considered for the design of infiltration systems at the site.

SECTION 4: GEOLOGIC HAZARDS

4.1 FAULT RUPTURE

As discussed in Section 2, several significant faults are located within 25 kilometers of the site; however, the site is not located within a State-designated Alquist Priolo Earthquake Fault Zone. As shown in Figure 3, no known surface expression of fault traces is thought to cross the site; therefore, fault rupture hazard is not a significant geologic hazard at the site.

4.2 ESTIMATED GROUND SHAKING

Moderate to severe (design-level) earthquakes can cause strong ground shaking, which is the case for most sites within the Central Coast Area. As mentioned, the pump station is to be designed and constructed in accordance with the 2019 California Building Code (CBC). Additionally, the reservoir tanks are to be designed and constructed in accordance with AWWA D100-11 standard. AWWA D100-11 standard indicates seismic design and parameters are to be in accordance with procedures outlined in ASCE 7-05. However, per our discussions with the structural engineer, TJC and Associates, Inc., we understand the tanks will also be designed in accordance with the 2019 CBC.

The 2019 CBC follows seismic design procedures outlined in ASCE 7-16. A peak ground acceleration (PGA_M) was estimated following the ground motion hazard analysis procedure presented in Chapter 21, Section 21.2 of ASCE 7-16 and Supplement No. 1 and determined in accordance with Section 21.5 of ASCE 7-16. For our analyses we used a PGA_M of 0.55g.

4.3 LIQUEFACTION POTENTIAL

The site is not currently mapped by the State of California but is within a zone mapped as having a low liquefaction potential (Rosenberg, 2001, and Stanford University, 2015). We screened the site for liquefaction during our site exploration by retrieving samples from the site to depths of at least 50 feet, performing visual classification on sampled materials, and performing various tests to further classify the soil properties.

During strong seismic shaking, cyclically induced stresses can cause increased pore pressures within the soil matrix that can result in liquefaction triggering, soil softening due to shear stress loss, potentially significant ground deformation due to settlement within sandy liquefiable layers as pore pressures dissipate, and/or flow failures in sloping ground or where open faces are present (lateral spreading) (NCEER 1998). Limited field and laboratory data is available regarding ground deformation due to settlement; however, in clean sand layers settlement on the order of 2 to 4 percent of the liquefied layer thickness can occur. Soils most susceptible to liquefaction are loose, non-cohesive soils that are saturated and are bedded with poor drainage, such as sand and silt layers bedded with a cohesive cap.

Soils with corrected “N” values greater than 30 are generally not considered liquefiable and can be pre-screened for liquefaction triggering. Some loose to medium dense soils with corrected “N” values less than 30 were encountered at locations and depths shallower than 50 feet across the site. However, as discussed, groundwater is not anticipated to be present within the upper 50 feet at the site. Based on this information, in our opinion, the potential for liquefaction to affect the proposed improvements is very low.

4.4 LATERAL SPREADING

Lateral spreading is horizontal/lateral ground movement of relatively flat-lying soil deposits towards a free face such as an excavation, channel, or open body of water; typically lateral spreading is associated with liquefaction of one or more subsurface layers near the bottom of the exposed slope. As failure tends to propagate as block failures, it is difficult to analyze and estimate where the first tension crack will form.

The potential for liquefaction to occur at the site is very low; therefore, in our opinion, the potential for lateral spreading is also very low.

4.5 SEISMIC SETTLEMENT/UNSATURATED SAND SHAKING

Loose to medium dense unsaturated sandy soils can settle during strong seismic shaking resulting in settlement of the ground surface and improvements. Seismic compression of unsaturated sand occurs due to rearrangement of soil particles during shaking and compression of the void space. The magnitude of volumetric compression of unsaturated sand is largely a function of seismic loading (effective shear strain and number of cycles) and the state of the soil (relative density and degree of saturation).

Our borings encountered loose to medium dense sands at varying depths across the site. We evaluated the potential for seismic compaction of the sand layers based on the work by Pradell (1998). Our analyses indicate that the sands may settle on the order of less than ¼ inch in the location of Borings EB-1 and EB-2 and on the order of ⅓ inch and 1⅓ to 1½ inch in the location of Borings EB-3 and EB-4, respectively, following strong seismic shaking.

4.6 SLOPE STABILITY

As mentioned, the site consists of an upper and lower level with a landscape berm between that transitions grades from the upper to lower levels. The northern end of the upper level is about 14 to 15 feet higher than the lower level at the toe of the landscape berm slope and the berm slopes have an inclination of 2:1 (horizontal:vertical) or flatter.

We performed a screening level static and seismic analysis of the slope through Cross Section A-A'. Computer assisted slope stability analysis was performed using the computer program SLIDE Version 9.001 and circular failure modes. Global and perimeter tank footing loading ranging from 1,575 psf to 2,550 psf was used in our analysis as provided by the structural engineer. Based on the current layout for the tanks and this loading, our screening level analysis of the existing slope indicates the slope to be stable for both seismic and static loading conditions, with factors of safety greater than 1.0 for seismic loading and greater than 1.5 for static loading.

4.7 TSUNAMI/SEICHE

The site is not mapped within a State-designated tsunami inundation area (CGS, 2009). The site is approximately 1⅓ miles inland from the Monterey Bay shoreline and is approximately 180 to 200 feet above mean sea level; therefore, the potential for inundation due to tsunami or seiche is considered low.

4.8 FLOODING

Based on our internet search of the Federal Emergency Management Agency (FEMA) flood map public database, the site is located within Zone X, described as an "Area of minimal flood hazard." We recommend the project civil engineer be retained to confirm this information.

SECTION 5: CONCLUSIONS

5.1 SUMMARY

From a geotechnical viewpoint, the project is feasible provided the concerns listed below are addressed in the project design. Descriptions of each concern with brief outlines of our recommendations follow the listed concerns.

- Potential for seismic and static settlement beneath reservoir tanks
- Presence of medium dense sands beneath pump station
- Undocumented fill
- Presence of cohesionless soils
- Retaining wall construction difficulties with cohesionless sands
- Differential movement at on-grade to on-structure transitions

5.1.1 Potential for Seismic and Static Settlement Beneath Reservoir Tanks

As discussed, our analysis of the unsaturated sand layers indicate there is a potential for about ⅓ to 1½ inches of settlement of the sand layers during a significant seismic event in the location of the proposed reservoir tanks. Differential seismic settlements are estimated to be on the order of ¾ to 1 over a horizontal distance of 50 feet.

As previously discussed, the average contact pressure beneath the tanks will be approximately 1,575 pounds per square foot (psf) with static pressures (dead + roof live load + fluid) beneath perimeter footing of 1,950 psf. Based on this loading, we estimate total static settlements beneath the center of the tanks to be on the order of 1½ to 2 inches and differential static settlements to be on the order of ¾ to 1 inch over a horizontal distance of 50 feet.

Based on our discussions with you, we understand the tanks can be designed for and accommodate these anticipated settlements. If design requirements change and the tanks are not able to be designed for the above settlements, additional recommendations and an alternative foundation will be required. Recommendations are presented in the "Foundations" section of this report.

5.1.2 Presence of Medium Dense Sands Beneath Pump Station

As discussed, the booster pump station finished floor will be at Elevation 190 feet and subgrade will be at about Elevation 188 to 189 feet. Based on these depths, we anticipate medium dense sands to be located beneath the building's subgrade. As such, we recommend the pump station pad be excavated to a minimum depth of Elevation 188 feet, processed, and compacted prior to construction of the pump station mat slab. Additional recommendations are presented in the "Booster Pump Station Pad Preparation" section of this report.

5.1.3 Undocumented Fill

As discussed, Borings EB-3 and EB-4 encountered 7 and 13 feet, respectively, of dense to very dense undocumented fill within the upper level of the site. Borings within the lower level did not encountered undocumented fills. As discussed, the booster pump station finished floor will be at Elevation 190 feet and subgrade for the building will be at about Elevation 188 to 189 feet. As shown on Cross Section A-A', Figure 4, fills are anticipated to be present within the existing landscape berm. These fills are anticipated to be variable in density and consistency within the berm area. Provided the pump station pad is over-excavated to a minimum depth of Elevation 188 feet as discussed above and the pad is processed and compacted as described in the

“Booster Pump Station Pad Preparation” section of this report, the fills remaining beneath the pump station pad will be reworked to a more uniform and consistent density.

For the reservoir tanks, the fills encountered appear to be dense to very dense. However, as records of the fill placement were not provided and the existing grades slope slightly down from the south to north, to have a more uniform thickness of new engineered fill beneath the tanks, we recommend the northern tank pad for Tank A1 and southern tank pad for Tank A2 be over-excavated to a minimum depth of Elevation 194 and 197 feet, respectively. The over-excavated subgrade should be processed and compacted prior to placement of additional fill up to the tank subgrade elevation. Additional recommendations are presented in the “Reservoir Tanks Pad Preparation” section of this report.

5.1.4 Presence of Cohesionless Soils

As mentioned, the site is underlain by cohesionless, sandy soils with low fines content. The sandy soil will likely not stand vertical when excavated and excavation sidewalls for foundations, utility trenches, temporary slopes, booster pump station excavation, etc. may cave in or accumulate significant amount of slough. Trenches for utilities and other excavations will likely have to be sloped to accommodate the potential caving and sloughing conditions. Grading and excavation contractors should be made aware of this condition and plan on forming footings, sloping trench sidewalls, preparing slab-on-grade subgrade just prior to concrete placement, and other similar construction issues as relates to temporary shoring, utility excavations, etc.

In addition, these types of soils are highly subject to erosion from wind and water. We recommend that new final slopes in sand be 3:1 (H:V) or flatter to limit erosion. All exposed surfaces should be vegetated or otherwise protected from erosion. These issues are addressed within the “Earthwork” and “Foundations” sections of this report.

5.1.5 Retaining Wall Construction Difficulties with Cohesionless Sands

The sands consist of fine to medium sands with fines generally less than 16 percent. These sands will likely not stand vertical when excavated. The contractor who will construct the retaining walls along the back and sides of the booster pump station will need to address this issue. If temporary slopes are not sloped, top-down construction and/or temporary vertical elements, or other techniques for face stability, will likely be required. Recommendations addressing this concern are presented in the “Earthwork” and “Retaining Walls” section of this report.

5.1.7 Differential Movement At On-grade to On-Structure Transitions

As discussed, we understand the booster pump station will be constructed below-grade into the existing landscape berm that transitions between the upper and lower levels of the site. Pavements will be constructed directly adjacent to the upside of the pump station wall and we understand trucks will likely be driving up to and adjacent to the building. Based on the latest plans provided, it does not appear improvements transition from on-grade support to overlying the below-grade wall of the building. However, if improvements will transition from on-grade to on-structure, varying amounts of settlement can be anticipated between the structure and the joining improvements supported on-grade due to difficulty in compacting retaining wall backfill, seasonal soil movement, differing response to vehicle loading, as well as other causes. As such, we recommend that retaining wall backfill be compacted to 95 percent where surface improvements are planned (see “Retaining Wall” section).

5.2 PLANS AND SPECIFICATIONS REVIEW

We recommend that we be retained to review the geotechnical aspects of the project structural, civil, and landscape plans and specifications, allowing sufficient time to provide the design team with any comments prior to issuing the plans for construction.

5.3 CONSTRUCTION OBSERVATION AND TESTING

As site conditions may vary significantly between the small-diameter borings performed during this investigation, we also recommend that a Cornerstone representative be present to provide geotechnical observation and testing during earthwork and foundation construction. This will allow us to form an opinion and prepare a letter at the end of construction regarding contractor compliance with project plans and specifications, and with the recommendations in our report. We will also be allowed to evaluate any conditions differing from those encountered during our investigation, and provide supplemental recommendations as necessary. For these reasons, the recommendations in this report are contingent of Cornerstone providing observation and testing during construction. Contractors should provide at least a 48-hour notice when scheduling our field personnel.

SECTION 6: EARTHWORK

6.1 SITE DEMOLITION

All existing improvements not to be reused for the current development, including all foundations, flatwork, pavements, utilities, and other improvements should be demolished and removed from the site. Recommendations in this section apply to the removal of these improvements, which may be present on the site, prior to the start of mass grading or the construction of new improvements for the project.

Cornerstone should be notified prior to the start of demolition, and should be present on at least a part-time basis during all backfill and mass grading as a result of demolition. Occasionally, other types of buried structures (wells, cisterns, debris pits, etc.) can be found on sites with prior development. If encountered, Cornerstone should be contacted to address these types of structures on a case-by-case basis.

Special care should be taken during the demolition and removal of existing improvements to minimize disturbance of the subgrade. Excessive disturbance of the subgrade, which includes either native or previously placed engineered fill, resulting from demolition activities can have serious detrimental effects on planned foundation and paving elements.

Existing foundations are typically mat-slabs, shallow footings, or piers/piles. If slab or shallow footings are encountered, they should be completely removed. If drilled piers are encountered, they should be cut off at an elevation at least 60-inches below proposed footings or the final subgrade elevation, whichever is deeper. The remainder of the drilled pier could remain in place. Foundation elements to remain in place should be surveyed and superimposed on the proposed development plans to determine the potential for conflicts or detrimental impacts to the planned construction. Following review, additional mitigation or planned foundation elements may need to be modified.

6.1.2 Abandonment of Existing Utilities

All utilities should be completely removed from within the planned pump station and reservoir tank areas. Utilities extending beyond the building and tank areas may be abandoned in place provided the ends are plugged with concrete, they do not conflict with planned improvements, and that the trench fills do not pose significant risk to the planned surface improvements.

The risk for owners associated with abandoning utilities in place include the potential for future differential settlement of existing trench fills, and/or partial collapse and potential ground loss into utility lines that are not completely filled with grout.

6.2 SITE CLEARING AND PREPARATION

6.2.1 Site Stripping

The site should be stripped of all surface vegetation, and surface and subsurface improvements within the proposed improvement areas. Demolition of existing improvements is discussed in detail below. Surface vegetation and topsoil should be stripped to a sufficient depth to remove all material greater than 3 percent organic content by weight. Based on our site observations, surficial stripping should extend about 4 to 6 inches below existing grade in the landscaping berm area to remove the vegetation except in areas where trees and large shrubs will be removed. Deeper excavations should be anticipated in these areas to remove the root balls.

6.2.2 Tree and Shrub Removal

Trees and shrubs designated for removal should have the root balls and any roots greater than ½-inch diameter removed completely. Mature trees are estimated to have root balls extending to depths of 2 to 4 feet, depending on the tree size. Significant root zones are anticipated to extend to the diameter of the tree canopy. Grade depressions resulting from root ball removal should be cleaned of loose material and backfilled in accordance with the recommendations in the “Compaction” section of this report. A Cornerstone representative should be present to provide geotechnical observation and testing during backfill of the excavations.

6.3 BOOSTER PUMP STATION PAD PREPARATION

As discussed, the booster pump station finished floor is to be at Elevation 190 feet and subgrade will be at about Elevation 188 to 189 feet. Due to the presence of medium dense sands, we recommend the pump station pad be over-excavated to a minimum depth of Elevation 188 feet and to a lateral distance of at least 3 feet beyond the pump station footprint. The exception would be along the retaining wall edge of the mat where the over-excavation may be cut to edge of mat. The exposed bottom of excavation at Elevation 188 feet should be scarified a minimum 12 inches, moisture conditioned, and compacted in accordance with the “Compaction” section of this report. We note that the subgrade soils are likely dry of optimum, as such, the contractor should be prepared to mix enough water into the soils to bring the soils near optimum moisture content. In the past, on similar projects with similar soil conditions, several rounds of mixing and watering were required to bring each lift to above optimum moisture content. In addition to the requirements presented in the “Compaction” section below, a performance specification for compaction of the base of the exposed subgrade should also consist of a minimum of five overlapping passes with a heavy-duty, vibratory smooth drum roller (such as a Dynapac CA5000, Volvo SD160 or an approved equivalent) that will exert a minimum of 25,000 ft-lbs of energy. Subgrade compaction should extend at least 3 feet beyond the pump station pad and perimeter of the foundation, except as noted above.

6.4 RESERVOIR TANKS PAD PREPARATION

As discussed, we understand the reservoir tank pads will be at Elevation 199 feet. Due to the presence of undocumented fills, we recommend the tank pads be over-excavated to a minimum depth of Elevation 194 and 197 feet for Tanks A1 and A2, respectively. The over-excavation should extend to a lateral distance of at least 5 feet beyond the tank footprints. Following over-excavation to the elevations mentioned above, the exposed subgrade should be scarified a minimum 12 inches, moisture conditioned, and compacted in accordance with the “Compaction” section of this report. In addition to the requirements presented in the “Compaction” section below, a performance specification for compaction of the base of the exposed subgrade should also consist of a minimum of five overlapping passes with a heavy-duty, vibratory smooth drum roller (such as a Dynapac CA5000, Volvo SD160 or an approved equivalent) that will exert a minimum of 25,000 ft-lbs of energy. Subgrade compaction should extend at least 5 feet beyond the tank pads and perimeter of the foundation.

Additional fill should then be placed and compacted in lifts in accordance with the “Compaction” section. We note that the subgrade soils are dry of optimum, as such, the contractor should be prepared to mix enough water into the soils to bring the soils above optimum moisture content. In the past, on similar projects with similar soil conditions, several rounds of mixing and watering were required to bring each lift to above optimum moisture content.

6.5 TEMPORARY CUT AND FILL SLOPES

The contractor is responsible for maintaining all temporary slopes and providing temporary shoring where required. Temporary shoring, bracing, and cuts/fills should be performed in accordance with the strictest government safety standards. On a preliminary basis, the upper

20 feet at the site may be classified as OSHA Soil Type C materials. Recommended soil parameters for temporary shoring, if required, are provided in the "Temporary Shoring" section of this report.

Excavations performed during site demolition and removal should be sloped at 1.5:1 (horizontal:vertical) within the upper 5 feet below building and reservoir/tank subgrade. Excavations extending more than 5 feet below building and tank subgrade and excavations in pavement and flatwork areas should be sloped in accordance with the OSHA soil classification.

6.6 BELOW-GRADE EXCAVATIONS

Below-grade excavations may be constructed with temporary slopes in accordance with the "Temporary Cut and Fill Slopes" section above if space allows. Alternatively, temporary shoring may support the planned pump station cut up to 12 feet. We have provided geotechnical parameters for shoring design in the section below. The choice of shoring method should be left to the contractor's judgment based on experience, economic considerations and adjacent improvements such as utilities, pavements, and foundation loads. Temporary shoring should support adjacent improvements without distress and should be the contractor's responsibility. A pre-condition survey including photographs and installation of monitoring points for existing site improvements should be included in the contractor's scope. We should be provided the opportunity to review the geotechnical parameters of the shoring design prior to implementation; the project structural engineer should be consulted regarding support of adjacent structures.

6.6.1 Temporary Shoring

Based on the site conditions encountered during our investigation, the cuts may be supported by soldier beams and tie-backs, braced excavations, soil nailing, or potentially other methods. If soil nailing is desired, the contractor should likely plan on limited sections where excavations may be left open, potentially constructing the nails through temporary sloped cuts, and other similar measures for sandy soil conditions. The use of hollow-core bar soil nails may be needed to address collapsing or caving soils.

Where shoring will extend more than about 10 feet, restrained shoring will most likely be required to limit detrimental lateral deflections and settlement behind the shoring. In addition to soil earth pressures, the shoring system will need to support adjacent loads such as construction vehicles and incidental loading, existing structure foundation loads, and street loading. We recommend that heavy construction loads (cranes, etc.) and material stockpiles be kept at least 15 feet behind the shoring. Where this loading cannot be set back, the shoring will need to be designed to support the loading. The shoring designer should provide for timely and uniform mobilization of soil pressures that will not result in excessive lateral deflections. Minimum suggested geotechnical parameters for shoring design are provided in the table below.

Table 4: Suggested Temporary Shoring Design Parameters

Design Parameter	Design Value
Minimum Lateral Wall Surcharge (upper 5 feet)	120 psf
Cantilever Wall – Triangular Earth Pressure	35 pcf
Restrained Wall – Uniform Earth Pressure	23H(1)(2)
Passive Pressure – Starting at 2 feet below the bottom of the adjacent excavation(3)	400 pcf up to 3,000 psf maximum uniform pressure

(1) H equals the height of the excavation; passive pressures are assumed to act over twice the soldier pile diameter
(2) The cantilever and restrained pressures are for drained designs. If undrained shoring is designed, an additional 62.4 pcf should be added for hydrostatic pressures.
(3) Bottom of adjacent excavation is bottom of mass excavation or bottom of footing excavation, whichever is deeper directly adjacent to the shoring element.
(4) If the reservoir tanks are constructed while temporary shoring is in-place and prior to construction of the pump station walls, the temporary shoring walls may need to be design for additional lateral wall surcharge from the tanks. Additional surcharge from the proposed tank is provided in the "Retaining Walls" section of this report.

If shotcrete lagging is used for the shoring facing, the permanent retaining wall drainage materials, as discussed in the "Wall Drainage" section of this report, will need to be installed during temporary shoring construction. At a minimum, 2-foot-wide vertical panels should be placed between soil nails or tiebacks that are spaced at 6-foot centers. For 8-foot centers, 4-foot-wide vertical panels should be provided. A horizontal strip drain connecting the vertical panels should be provided, or pass-through connections should be included for each vertical panel.

We performed our borings with hollow-stem auger drilling equipment and as such were not able to evaluate the potential for caving soils, which can create difficult conditions during soldier beam, tie-back, or soil nail installation; caving soils can also be problematic during excavation and lagging placement. The contractor is responsible for evaluating excavation difficulties prior to construction. Relatively clean sands were encountered during our exploration, pilot holes performed by the contractor may be desired to further evaluate caving soils prior to the finalization of the shoring budget.

In addition to anticipated deflection of the shoring system, other factors such as voids created by soil sloughing, and erosion of granular layers can create adverse ground subsidence and deflections. The contractor should attempt to cut the excavation as close to neat lines as possible; where voids are created they should be backfilled as soon as possible with sand, gravel, or grout.

As previously mentioned, we recommend that a monitoring program be developed and implemented to evaluate the effects of the shoring on adjacent improvements. All sensitive improvements should be located and monitored for horizontal and vertical deflections and distress cracking based on a pre-construction survey.

The above recommendations are for the use of the design team; the contractor in conjunction with input from the shoring designer should perform additional subsurface exploration they deem necessary to design the chosen shoring system. A California-licensed civil or structural engineer must design and be in responsible charge of the temporary shoring design. The contractor is responsible for means and methods of construction, as well as site safety.

6.7 SUBGRADE PREPARATION

After site clearing and demolition is complete, and prior to backfilling any excavations that extend below the excavation plane for subgrade resulting from over-excavation, the excavation subgrade and subgrade within areas to receive additional site fills, slabs-on-grade and/or pavements should be scarified to a depth of 12 inches, moisture conditioned, and compacted in accordance with the “Compaction” section below. We note that the subgrade soils are dry of optimum, as such, the contractor should be prepared to mix enough water into the soils to bring the soils above optimum moisture content. In the past, on similar projects with similar soil conditions, several rounds of mixing and watering were required to bring each lift to above optimum moisture content.

Sandy subgrades that are allowed to dry out after compaction will be subject to disturbance by both foot and vehicle traffic. In pavement areas, we recommend that aggregate base sections be placed immediately after the subgrade is prepared to reduce rework. In the building and reservoir tank areas, we recommend that subgrade compaction and proof rolling be performed within 24 hours of capillary break layer or slab-on-grade construction to reduce rework.

6.8 SUBGRADE STABILIZATION MEASURES

Native soil and fill materials consisting of sands and silty sands can become unstable due to high moisture content, whether from high in-situ moisture contents or from winter rains. When the moisture content increases over the laboratory optimum, it becomes more likely the materials will be subject to softening and yielding (pumping) from construction loading or become unworkable during placement and compaction.

The soils appear to range from below optimum to near optimum moisture content at the time of our drilling. However, during winter and spring, the soils may be significantly wetter. If construction is undertaken during the winter, spring, or wet periods, the contractor should anticipate drying native soils prior to reusing them as fill. During dryer periods, the contractor should anticipate moisture conditioning the soils prior to reusing them as fill. When the soils are wetter, repetitive rubber-tire loading, or other heavy or repetitive loads may de-stabilize the soils.

There are several methods to address potential unstable soil conditions and facilitate fill placement and trench backfill. Some of the methods are briefly discussed below. Implementation of the appropriate stabilization measures should be evaluated on a case-by-case basis according to the project construction goals and the particular site conditions.

6.8.1 Scarification and Drying

The subgrade may be scarified to a depth of 12 inches and allowed to dry to near optimum conditions, if sufficient dry weather is anticipated to allow sufficient drying. More than one round of scarification may be needed.

6.8.2 Removal and Replacement

As an alternative to scarification, the contractor may choose to over-excavate the unstable soils and replace them with dry on-site or import materials. A Cornerstone representative should be present to provide recommendations regarding the appropriate depth of over-excavation, whether a geosynthethic (stabilization fabric or geogrid) is recommended, and what materials are recommended for backfill.

6.8.3 Chemical Treatment

Where the unstable area exceeds about 5,000 to 10,000 square feet and/or site winterization is desired, chemical treatment with cement may be more cost-effective than removal and replacement, depending on access conditions. Recommended chemical treatment depths will typically range from 12 to 18 inches depending on the magnitude of the instability.

6.9 MATERIAL FOR FILL

6.9.1 Re-Use of On-site Soils

On-site soils with an organic content less than 3 percent by weight may be reused as general fill. General fill should not have lumps, clods or cobble pieces larger than 6 inches in diameter; 85 percent of the fill should be smaller than 2½ inches in diameter. Minor amounts of oversized material (smaller than 12 inches in diameter) may be allowed provided the oversized pieces are not allowed to nest together and the compaction method will allow for loosely placed lifts not exceeding 12 inches.

6.9.2 Potential Import Sources

Imported and non-expansive material should be inorganic with a Plasticity Index (PI) of 15 or less, and not contain recycled asphalt concrete where it will be used within the booster pump station areas. To prevent significant caving during trenching or foundation construction, imported material should have sufficient fines. Samples of potential import sources should be delivered to our office at least 10 days prior to the desired import start date. Information regarding the import source should be provided, such as any site geotechnical reports. If the material will be derived from an excavation rather than a stockpile, potholes will likely be required to collect samples from throughout the depth of the planned cut that will be imported. At a minimum, laboratory testing will include PI tests. Material data sheets for select fill materials (Class 2 aggregate base, ¾-inch crushed rock, quarry fines, etc.) listing current laboratory testing data (not older than 6 months from the import date) may be provided for our review without providing a sample. If current data is not available, specification testing will need to be completed prior to approval.

Environmental and soil corrosion characterization should also be considered by the project team prior to acceptance. Suitable environmental laboratory data to the planned import quantity should be provided to the project environmental consultant; additional laboratory testing may be required based on the project environmental consultant's review. The potential import source should also not be more corrosive than the on-site soils, based on pH, saturated resistivity, and soluble sulfate and chloride testing.

6.10 COMPACTION REQUIREMENTS

All fills, and subgrade areas where fill, slabs-on-grade, and pavements are planned, should be placed in loose lifts 8 inches thick or less and compacted in accordance with ASTM D1557 (latest version) requirements as shown in the table below. In general, sandy/gravelly soils should be compacted with vibratory equipment. Open-graded materials such as crushed rock should be placed in lifts no thicker than 18 inches and consolidated in place with vibratory equipment. Each lift of fill and all subgrade should be firm and unyielding under construction equipment loading in addition to meeting the compaction requirements to be approved. The contractor (with input from a Cornerstone representative) should evaluate the in-situ moisture conditions, as the use of vibratory equipment on soils with high moistures can cause unstable conditions. General recommendations for soil stabilization are provided in the "Subgrade Stabilization Measures" section of this report.

Table 5: Compaction Requirements

Description	Material Description	Minimum Relative[1] Compaction (percent)	Moisture[2] Content (percent)
General Fill (tank and pump station pads)	On-Site Granular Soils	95	Optimum
Oil Sand Cushion	Material Mix per AWWA D100-11 Standard	95	Optimum
Trench Backfill – Pipe Zone	On-Site Granular Soils or Imported Well-Graded Bedding and Shading	95	Optimum
Trench Backfill – Trench Zone	On-Site Granular Soils or Imported Non-Expansive Material – Paved Areas	95	Optimum
	On-Site Granular Soils or Imported Non-Expansive Material – Unpaved Areas	90	Optimum

Table 5 Continues

Table 5: Compaction Requirements (Continued)

Description	Material Description	Minimum Relative[1] Compaction (percent)	Moisture[2] Content (percent)
Trench Backfill – Street Zone	On-Site Granular Soils or Imported Non-Expansive Material	95	Optimum
Crushed Rock Fill (Pipe bedding and trench backfill)	Clean Crushed Rock	Consolidate In-Place	NA
Below-Grade Wall Backfill	Without Surface Improvements	90	Optimum
Below-Grade Wall Backfill	With Surface Improvements	95[4]	Optimum
Flatwork Subgrade	On-Site Granular Soils	90	Optimum
Flatwork Aggregate Base	Class 2 Aggregate Base[3]	90	Optimum
Pavement Subgrade	On-Site Granular Soils	95	Optimum
Pavement Aggregate Base	Class 2 Aggregate Base[3]	95	Optimum
Asphalt Concrete	Asphalt Concrete	95 (Marshall)	NA

1 – Relative compaction based on maximum density determined by ASTM D1557 (latest version)
2 – Moisture content based on optimum moisture content determined by ASTM D1557 (latest version)
3 – Class 2 aggregate base shall conform to Caltrans Standard Specifications, latest edition, except that the relative compaction should be determined by ASTM D1557 (latest version)
4 – Using light-weight compaction or walls should be braced

6.10.1 Construction Moisture Conditioning

The on-site sandy soils may dry out and ravel after initial compaction. The contractor should anticipate re-moisture conditioning (flooding is not recommended), or deep scarification, moisture conditioning, and re-compaction.

6.11 TRENCH BACKFILL

Pipeline lines constructed within public right-of-way should be trenched, bedded and shaded, and backfilled in accordance with the local or governing jurisdictional requirements (Marina Coast Water District Standard Plan, Standard Detail W-12), except as modified above. Utility lines in private improvement areas should be constructed in accordance with the following requirements unless superseded by other governing requirements.

Pipeline lines should be bedded and shaded to at least 12 inches over the top of the lines with crushed rock (⅜-inch-diameter or greater) or well-graded sand and gravel materials conforming to the pipe manufacturer's requirements. Open-graded shading materials should be consolidated in place with vibratory equipment and well-graded materials should be compacted to at least 95 percent relative compaction with vibratory equipment prior to placing subsequent backfill materials. Open-graded materials should be enclosed within filter fabric (Mirafi 140N or approved equivalent) to prevent migration of sand into the open graded material.

We recommend that trenches be excavated a minimum 12 inches beyond the outside of the pipe including bells. The pipe shading should be consolidated or compacted (depending on type of material) on the outside of the pipe in lifts to enable the material to be compacted under the pipe haunches.

General backfill over shading materials may consist of on-site native materials provided they meet the requirements in the “Material for Fill” section, and are moisture conditioned and compacted in accordance with the requirements in the “Compaction” section.

Where utility lines will cross perpendicular to strip footings, the footing should be deepened to encase the utility line, providing sleeves or flexible cushions to protect the pipes from anticipated foundation settlement, or the utility lines should be backfilled to the bottom of footing with sand-cement slurry or lean concrete. Where utility lines will parallel footings and will extend below the “foundation plane of influence,” an imaginary 1:1 plane projected down from the bottom edge of the footing, either the footing will need to be deepened so that the pipe is above the foundation plane of influence or the utility trench will need to be backfilled with sand-cement slurry or lean concrete within the influence zone. Sand-cement slurry used within foundation influence zones should have a minimum compressive strength of 75 psi.

6.12 PERMANENT FILL SLOPES

Fill slopes should be overbuilt and trimmed back, exposing engineered fill when complete. Fill placed on existing ground inclined at 5:1 or greater should be benched into the existing slope and a keyway constructed at the toe of the fill. Benches and keyways should be angled slightly into the slope (minimum 2 percent inclination).

Due to the highly erodible sandy soils, we recommend new final slopes in sand be 3:1 (H:V) or flatter to limit erosion. All exposed surfaces should be vegetated or otherwise protected from erosion. Refer to the “Erosion Control” section of this report for a discussion regarding protection of slope surfaces.

6.13 SITE DRAINAGE

Ponding should not be allowed adjacent to tank and building foundations, slabs-on-grade, or pavements. Hardscape surfaces should slope at least 2 percent towards suitable discharge facilities; landscape areas should slope at least 3 percent. Roof runoff should be directed away from building areas in closed pipes to storm drain or other retention or detention areas. Landscape drainage such as drain inlets and storm water filtration and/or infiltration trenches should be provided to collect and transmit storm water runoff to project storm drains, and/or detention or retention facilities.

6.14 PERMANENT EROSION CONTROL MEASURES

Grading will require periodic maintenance after construction to reduce the potential for erosion and sloughing. At a minimum all slopes should be vegetated by hydroseeding or other landscape ground cover. The establishment of vegetation will help reduce runoff velocities, allow some infiltration and transpiration, trap sediment within runoff, and protect the soil from raindrop impact. Depending on the exposed material type and the slope inclination, more aggressive erosion control measures may be needed to protect slopes for one or more winter seasons while vegetation is establishing. This may consist of straw matting, or erosion control blankets used in combination with planting.

Both construction and post-construction Storm Water Pollution Prevention Plans (SWPPPs) should be prepared for the project-specific requirements. We recommend that final grading plans be provided for our review.

SECTION 7: 2019 CBC SEISMIC DESIGN CRITERIA

We developed site-specific seismic design parameters in accordance with Chapter 16, Chapter 18 and Appendix J of the 2019 California Building Code (CBC) and Chapters 11, 12, 20, and 21 and Supplement No. 1 of ASCE 7-16. Per discussions with the structural engineer, we understand the reservoir tanks will be designed per AWWA D100-11 standards however will be in accordance with the 2019 CBC and ASCE 7-16 seismic parameters. We understand the pump station will also follow 2019 CBC and ASCE 7-16 seismic design.

7.1 SITE LOCATION AND PROVIDED DATA FOR 2019 CBC SEISMIC DESIGN

The project is located at latitude 36.656597° and longitude -121.796108°, which is based on Google Earth (WGS84) coordinates at the center of the site at 8th Street and 6th Avenue, Marina, California. The structural engineer provided that the structures have been assigned as Risk Category IV, resulting in a Seismic Importance Factor (I_e) of 1.50 in accordance with Table 1.5-2 of ASCE 7-16. The structural engineer also indicated that based on ASCE 7-16, Section 12.8.2.1 that the fundamental period of the CMU pump station building is estimated as 0.12 seconds. The period for the tanks have not been provided. The above values should be confirmed by the structural engineer.

7.2 SITE CLASSIFICATION – CHAPTER 20 OF ASCE 7-16

Code-based site classification and ground motion attenuation relationships are based on the time-weighted average shear wave velocity of the top approximately 100 feet (30 meters) of the soil profile (V_{S30}).

The “Seismic Coefficients” used to design buildings are established based on a series of tables and figures addressing different site factors, including the soil profile in the upper 100 feet below grade and mapped spectral acceleration parameters based on distance to the controlling seismic source/fault system. Based on our borings and our review of the local geology, the site is underlain by typical SPT “N” values between about 15 to 50 blows per foot. Based on the conditions encountered in our borings, and available geologic data, the site may be classified as Site Class D, described as a “stiff soil” profile. Because we used site-specific data from our explorations and laboratory testing, the site class should be considered as “determined” for the purposes of estimating the seismic design parameters from the code outlined below. Our site-specific ground motion hazard analysis considered a V_{S30} of 230 m/s (754 ft/s).

7.3 CODE-BASED SEISMIC DESIGN PARAMETERS

Code-based spectral acceleration parameters were determined based on mapped acceleration response parameters adjusted for the specific site conditions. Mapped Risk-Adjusted Maximum Considered Earthquake (MCE_R) spectral acceleration parameters (S_S and S_1) were determined using the ATC Hazards by Location website (https://hazards.atcouncil.org).

The mapped acceleration parameters were adjusted for local site conditions based on the average soil conditions for the upper 100 feet (30 meters) of the soil profile. Code-based MCE_R spectral response acceleration parameters adjusted for site effects (S_{MS} and S_{M1}) and design spectral response acceleration parameters (S_{DS} and S_{D1}) are presented in Table 6.

In accordance with Section 11.4.8 of ASCE 7-16, structures on Site Class D sites with mapped 1-second period spectral acceleration (S_1) values greater than or equal to 0.2 require a site-specific ground motion hazard analysis be performed in accordance with Section 21.2 of ASCE 7-16. ***Design seismic parameters determined by performing a Ground Motion Hazard Analysis per Section 21.2 of ASCE 7-16 are presented in Table 9. Recommended values in Table 6 should not be used for design unless in the judgement of the structural engineer an exception can be taken in accordance with Section 11.4.8 of ASCE 7-16.*** Values summarized in Table 6 are only used to determine Seismic Design Category and comparison with minimum code requirements for further use in our ground motion hazard analysis (GMHA).

Table 6: 2019 CBC Site Categorization and Site Coefficients

Classification/Coefficient	Design Value
Site Class	D
Site Latitude	36.656597°
Site Longitude	-121.796108°
Risk Category	IV
Seismic Design Category	To be determined by S.E.
Short Period Mapped Spectral Acceleration – S_S	1.417g
1-second Period Mapped Spectral Acceleration – S_1	0.512g
Short-Period Site Coefficient – F_a	1.0
Long-Period Site Coefficient – F_v	“Null”**
Short Period MCE Spectral Response Acceleration Adjusted for Site Effects – S_{MS}	1.417g
1-second Period MCE Spectral Response Acceleration Adjusted for Site Effects – S_{M1}	“Null”**
Short Period, Design Earthquake Spectral Response Acceleration – S_{DS}	0.945g

Table 6 Continues

Table 6: 2019 CBC Site Categorization and Site Coefficients (Continued)

Classification/Coefficient	Design Value
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	"Null"**
Long-Period Transition – T_L	12 seconds
Site Coefficient – F_{PGA}	1.1
Site Modified Peak Ground Acceleration – PGA_M	0.644g

Note: S.E. = Structural Engineer
**See site-specific analysis, see Section 11.4.8.

7.4 GROUND MOTION HAZARD ANALYSIS

Following Section 11.4.8 of ASCE 7-16, we performed a ground motion hazards analysis (GMHA) in accordance with Chapter 21, Section 21.2 of ASCE 7. We evaluated both Probabilistic MCE_R Ground Motions in accordance with Method 1 and Deterministic MCE_R Ground Motions to generate our recommended design response spectrum for the project.

Our analyses were performed using the USGS interface Unified Hazard Tool (UHT) based on the UCERF 3 Data Set, Building Seismic Safety Council (BSSC) Scenario Catalog 2014 event set (BSSC 2014), and the 2014 National Seismic Hazard Maps – Source Parameters (NSHMP deterministic event set). Additionally, we utilized the USGS program Response Spectra Plotter with combined models (Combined: WUS 2014 (4.1)).

Our analysis utilized the mean ground motions predicted by four of the Next Generation Attenuation West 2 (NGA-West 2) relationships: Boore-Atkinson (2013), Campbell-Bozorgnia (2013), Chiou-Youngs (2013), and Abrahamson-Silva (2013). Rotation factors (scale factors) were determined as specified in ASCE 7-16 Chapter 21, Section 21.2, to calculate the maximum rotated component of ground motions (ASCE, 2016).

7.4.1 Probabilistic MCE_R

We performed a probabilistic seismic hazard analysis (PSHA) per ASCE 7-16 Section 21.2.1. The probabilistic MCE acceleration response spectrum is defined as the 5 percent damped acceleration response spectrum having a 2 percent probability of exceedance in a 50-year period (2,475-year return period). The probabilistic MCE spectrum was multiplied by Risk Coefficients (CR) to determine the probabilistic MCE_R. We used Risk Coefficients (CR_S and CR_1) of 0.953 and 0.936, respectively, based on ASCE 7-16 Section 21.2.1.1 – Method 1 and the ATC website. Risk coefficients for the various periods are presented in Table 7, Column 3.

The resulting probabilistic MCE_R is presented on Figure 6 (red line). Spectral ordinates are tabulated in Table 7, Column 6.

7.4.2 Deterministic MCER

We performed deterministic seismic hazard analyses in accordance with ASCE 7-16 Section 21.2.2 and ASCE 7-16 Supplement No. 1. The deterministic MCE_R acceleration response spectrum is calculated as the largest 84th percentile ground motion in the direction of maximum horizontal response for each period for characteristic earthquakes on all known active faults within the region. The largest deterministic ground motion for all periods resulted from a M_w 8.05 earthquake on the San Andreas Fault, located approximately 30.6 kilometers from the site.

In accordance with Supplement No. 1 of ASCE 7-16, when the largest spectral response acceleration of the resulting deterministic ground motion response spectrum is less than $1.5F_a$ than the largest 84th percentile rotated response spectrum (Table 7, Column 4) shall be scaled by a single factor such that the maximum response spectral acceleration equals $1.5F_a$. For Site Classes A, B, C and D, F_a is determined using Table 11.4.1 with the value of S_s taken as 1.5; for Site Class E, F_a shall be taken as 1.0. When the largest spectral response acceleration of the probabilistic ground motion response of 21.2.1 is less than $1.2F_a$, the deterministic ground motion response spectrum does not need to be calculated.

As the largest probabilistic spectral response acceleration was determined to be 1.895 which is greater than 1.2 F_a, where F_a is taken as 1.0 from Table 11.4-1 in ASCE 7-16 Supplement No. 1, the 84th percentile rotated response spectrum was calculated as part of the deterministic analyses. The maximum spectral acceleration from the 84th percentile rotated response spectrum was then compared to $1.5F_a$ to determine if a scale factor needed to be applied. The deterministic MCE spectrum are tabulated in Table 7, Column 5. The deterministic MCE_R is presented graphically on Figure 6 (blue line).

7.4.3 Site-Specific MCE_R

The site-specific MCE_R is defined by ASCE 7-16 Section 21.2.3 as the lesser of the deterministic and probabilistic MCE_R's at each period. Spectral ordinates for the site-specific MCE_R are tabulated in Table 7, Column 7 and shown graphically on Figure 6 (dashed black line).

Table 7: Development of Site-Specific MCE$_R$ Spectrum

Period (seconds)	CBC General Spectrum (g)	Risk Coefficient	Det. 84th Percentile Rotated	Deterministic MCE$_R$ (g)	Probabilistic MCE$_R$ (g)	Site-Specific MCE$_R$ (g)
0.000	0.378	0.953	0.513	0.629	0.725	0.629
0.050	0.535	0.953	0.542	0.665	1.001	0.665
0.100	0.692	0.953	0.783	0.961	1.276	0.961
0.150	0.848	0.953	0.969	1.189	1.483	1.189
0.181	0.945	0.953	1.030	1.265	1.612	1.265
0.200	0.945	0.953	1.068	1.311	1.691	1.311
0.250	0.945	0.952	1.147	1.408	1.793	1.408
0.300	0.945	0.951	1.197	1.469	1.895	1.469
0.400	0.945	0.949	1.222	1.500	1.868	1.500
0.500	0.945	0.947	1.205	1.479	1.842	1.479
0.750	0.945	0.941	1.014	1.244	1.542	1.244
0.903	0.945	0.938	0.920	1.129	1.402	1.129
1.000	0.853	0.936	0.861	1.057	1.314	1.057
2.000	0.427	0.936	0.486	0.597	0.731	0.597
3.000	0.284	0.936	0.345	0.424	0.501	0.424
4.000	0.213	0.936	0.268	0.328	0.379	0.328
5.000	0.171	0.936	0.219	0.269	0.301	0.269

7.4.4 Design Response Spectrum

The Design Response Spectrum (DRS) is defined in ASCE 7-16 Section 21.3 as:

- two-thirds of the site-specific MCE$_R$, but
- not less than 80% of the general design response spectrum

Spectral accelerations corresponding to two-thirds of the MCE$_R$ are tabulated in Table 8, Column 2. Ordinates corresponding to 80% of the general Site Class D response spectrum are tabulated below in Table 8, Column 3. Ordinates of the site-specific DRS are tabulated in Table 8, Column 4. Development of the site-specific DRS is presented graphically on Figure 7 (dashed black line).

Table 8: Development of Site-Specific Design Response Spectrum

Period (seconds)	2/3 Site-Specific MCE_R (g)	80% CBC General Spectrum (g)	Design Response Spectrum (g)
0.000	0.419	0.302	0.419
0.050	0.443	0.428	0.443
0.100	0.641	0.553	0.641
0.150	0.793	0.679	0.793
0.181	0.843	0.756	0.843
0.200	0.874	0.756	0.874
0.250	0.938	0.756	0.938
0.300	0.979	0.756	0.979
0.400	1.000	0.756	1.000
0.500	0.986	0.756	0.986
0.750	0.829	0.756	0.829
0.903	0.753	0.756	0.756
1.000	0.704	0.683	0.704
2.000	0.398	0.341	0.398
3.000	0.282	0.228	0.282
4.000	0.219	0.171	0.219
5.000	0.179	0.137	0.179

7.5 DESIGN ACCELERATION PARAMETERS

Design acceleration parameters (S_{DS} and S_{D1}) were determined in accordance with Section 21.4 of ASCE 7-16. S_{DS} is defined as the design spectral acceleration at 90% of the maximum spectral acceleration, S_a, obtained from the site-specific spectrum, at any period within the range from 0.2 to 5 seconds, inclusive. S_{D1} is defined as the maximum value of the product, TS_a, for periods from 1 to 2 seconds for sites with $v_{s,30} > 1{,}200$ ft/s ($v_{s,30} > 365.76$ m/s) and for periods from 1 to 5 seconds for sites with $v_{s,30} \leq 1{,}200$ ft/s ($v_{s,30} \leq 365.76$ m/s).

Site-specific MCE_R spectral response acceleration parameters (S_{MS} and S_{M1}) are calculated as:

- 1.5 times the S_{DS} and S_{D1} values, respectively, but
- not less than 80% of the code-based values presented in Table 6.

Recommended design acceleration parameters are summarized in Table 9.

When using the Equivalent Lateral Force Procedure, ASCE 7-16 Section 21.4 allows using the spectral acceleration at any period (T) in lieu of S_{D1}/T in Eq. 12.8-3 and $S_{D1}T_L/T^2$ in Eq. 12.8-4.

The site-specific spectral acceleration at any period may be calculated by interpolation of the spectral ordinates in Table 8, Column 4.

Table 9: Site-Specific Design Acceleration Parameters

Parameter	Value
S_{DS}	0.900
S_{D1}	0.895
S_{MS}	1.350
S_{M1}	1.343

7.6 SITE-SPECIFIC MCE$_G$ PEAK GROUND ACCELERATION

We calculated the Site-Specific MCE$_G$ Peak Ground Acceleration (PGA$_M$) per ASCE 7-16 Section 21.5. The Site-Specific PGA$_M$ is calculated as the lesser of probabilistic and deterministic geometric mean PGA. The 2% in 50-year probabilistic geometric mean PGA is 0.692g. The deterministic PGA is considered the greater of the largest 84th percentile deterministic geometric mean PGA (0.466g) or one-half of the tabulated F_{PGA} value from ASCE 7-16 Table 11.8.1 with the value of PGA taken as 0.5g. For Site Class D, F_{PGA} is 1.100 and one-half of the F_{PGA} is 0.55g; therefore, the deterministic PGA is 0.55g. Additionally, the Site-Specific PGA$_M$ may not be less than 80% of the mapped PGA$_M$ determined from ASCE 7-16 Equation 11.8-1. The mapped PGA$_M$ for the site is 0.644g; 80% of PGA$_M$ is 0.515g.

Based on the above, the recommended Site-Specific PGA$_M$ for the site is 0.55g.

SECTION 8: FOUNDATIONS

8.1 SUMMARY OF RECOMMENDATIONS

In our opinion, the proposed reservoir tanks and booster pump station may be supported on shallow foundations provided anticipated settlements are tolerable and the recommendations in the “Earthwork” section and the sections below are followed.

8.2 MAT FOUNDATIONS – BOOSTER PUMP STATION

As discussed, we understand finished floor elevation for the pump station will be Elevation 190 feet. We also understand the building is proposed to be supported by a 1-foot and a 1-foot 7-inch thick mat foundation. Provided recommendations outlined in the “Earthwork” section of this report are followed, the booster pump station may be supported by a reinforced concrete mat foundation.

8.2.1 Reinforced Concrete Mat Foundation

As mentioned, we understand the average contact pressure beneath the mat foundation will be about 800 to 1,000 psf. We recommend the maximum allowable bearing pressure at heavier loaded areas of the mat be limited to 3,750 psf for combined dead plus live loads. The maximum bearing pressure may be increased by one-third for all loads, including wind or seismic. The maximum bearing pressure is a net value; the weight of the mat may be neglected for the portion of the mat extending below grade. Top and bottom reinforcing steel should be included as required to help span irregularities and differential settlement.

8.2.2 Mat Settlement

Based on an average areal pressure of 1,000 psf, we estimate total static settlement will be on the order of ½ inch or less near the center of the mat foundation and differential static settlement from the center to the edges would be about ¼ inch. In addition, we estimate that differential seismic movement from dry sand shaking will be on the order of ¼ inch across the mat foundation, resulting in a total estimated differential settlement on the order of ½-inch across the mat foundation. We recommend we be retained to review the final footing layout and loading and verify the settlement estimates above.

8.2.3 Mat Foundation Lateral Loading

Lateral loads may be resisted by friction between the bottom of mat foundation and the supporting subgrade, and also by passive pressures generated against the mat edges. An ultimate frictional resistance of 0.45 applied to the footing dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 400 pcf may be used in design. The structural engineer should apply an appropriate factor of safety (such as 1.5) to the ultimate values above. Where the mat is adjacent to landscape areas without hardscape, the upper 12 inches of soil should be neglected when determining passive pressure capacity.

8.2.4 Mat Foundation Construction Considerations

Sandy, cohesionless soils with low fines content are present. These soils will become easily disturbed during mat construction activities. As such, we recommend compaction and proof rolling of the mat foundation subgrade just prior to steel placement and mat construction. Contractors may consider placement of a protection layer (rat slab/aggregate base layer) to protect prepared subgrade from disturbance during mat construction.

8.2.5 Moisture Protection Considerations for Mat Foundations

The following general guidelines for concrete mat construction where floor coverings are planned are presented for the consideration by the developer, design team, and contractor. These guidelines are based on information obtained from a variety of sources, including the American Concrete Institute (ACI) and are intended to reduce the potential for moisture-related problems causing floor covering failures, and may be supplemented as necessary based on project-specific requirements. The application of these guidelines or not will not affect the geotechnical aspects of the mat foundation performance.

- Place a 10-mil vapor retarder conforming to ASTM E 1745, Class C requirements or better directly below the concrete mat; the vapor retarder should extend to within 12 to 18 inches from the mat edges and be sealed at all seams and penetrations in accordance with manufacturer's recommendations and ASTM E 1643 requirements. For mats 12 inches thick or less, a 4-inch-thick capillary break, consisting of ½- to ¾-inch crushed rock with less than 5 percent passing the No. 200 sieve, should be placed below the vapor retarder and consolidated in place with vibratory equipment.

- The concrete water:cement ratio should be 0.45 or less. Mid-range plasticizers may be used to increase concrete workability and facilitate pumping and placement.

- Water should not be added after initial batching unless the slump is less than specified and/or the resulting water:cement ratio will not exceed 0.45.

- Where floor coverings are planned, all concrete surfaces should be properly cured.

- Water vapor emission levels and concrete pH should be determined in accordance with ASTM F1869-98 and F710-98 requirements and evaluated against the floor covering manufacturer's requirements prior to installation.

8.3 SHALLOW RING AND SPREAD FOOTING FOUNDATIONS – RESERVOIR TANKS

Provided the reservoir tanks can tolerate the anticipated settlements discussed below, the tank ring foundations and center column foundation can be supported by shallow ring and spread footing foundations.

8.3.1 Ring and Spread Footings

The ring and spread footings should bear entirely on properly prepared subgrade in accordance with the "Earthwork" section of this report, be at least 12 inches wide, and extend at least 24 inches below the lowest adjacent grade. Bottom of footing is based on lowest adjacent grade, defined as the deeper of the following: 1) bottom of the tank subgrade, or 2) finished exterior grade, excluding landscaping topsoil.

Footings constructed to the above dimensions and in accordance with the "Earthwork" recommendations of this report are capable of supporting maximum allowable bearing pressures of 3,000 psf for combined dead plus live loads, and 4,000 psf for all loads including wind and seismic. These pressures are based on factors of safety of 2.0 and 1.5 applied to the ultimate bearing pressure for dead, dead plus live, and all loads, respectively. These pressures are net values; the weight of the footing may be neglected for the portion of the footing extending below grade (typically, the full footing depth). Top and bottom mats of reinforcing steel should be included in continuous footings to help span irregularities and differential settlement.

8.3.2 Ring and Spread Footing Settlement

Structural loads for the reservoir tanks were provided by the structural engineer as 1,950 psf (dead + roof live load + fluid) for the perimeter ring foundation, 11.2 kips (dead + roof liv load) for the interior center column, and interior average contact pressure of 1,575 psf (dead + fluid) pressure beneath the bottom of tank between the interior column and perimeter foundation.

Based on this loading and the allowable bearing pressures presented above, we estimate that the total static settlement will be on the order of 1½ to 2 inches at the center of the tank, with post-construction differential settlement of about ¾ to 1 inch over a horizontal distance of 50 feet. In addition, we estimate that differential seismic movement from dry sand shaking will be on the order of ¾ to 1 inch between independent foundation elements, resulting in a total estimated differential movement on the order of 1½ to 2 inches between independent foundation elements, or a horizontal distance of 50 feet. We recommend we be retained to review the final footing layout and loading and verify the settlement estimates above.

As mentioned, based on our discussions with you, we understand the tanks can be designed for and accommodate the anticipated settlements discussed above. If it is determined the tanks are not able to be designed for the above settlements, additional recommendations and an alternative foundation will be required.

8.3.3 Lateral Loading

Lateral loads may be resisted by friction between the bottom of footing and the supporting subgrade, and also by passive pressures generated against footing sidewalls. An ultimate frictional resistance of 0.45 applied to the footing dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 400 pcf may be used in design. The structural engineer should apply an appropriate factor of safety (such as 1.5) to the ultimate values above. The upper 12 inches of footing embedment should be neglected for passive resistance for exterior footings unless the perimeter of the tank is paved.

8.3.4 Oiled Sand Cushion

As discussed, we understand the steel bottom of the tanks between the ring foundation and center column foundation will be supported directly on the subgrade soils beneath. We also understand a 6-inch oiled sand cushion will be placed between the subgrade and tank bottom. The 6-inch oiled sand cushion should be placed and compacted on the subgrade prepared in accordance with the “Earthwork” recommendations of this report. The oiled sand cushion should be compacted to a minimum 95 percent relative compaction as outlined in the “Compaction” section of this report and should be designed in accordance with AWWA D100-11 requirements.

8.3.5 Ring and Spread Footing Foundation Construction Considerations

Where utility lines cross perpendicular to the ring foundation footings, the footing should be deepened to encase the utility line, providing sleeves or flexible cushions to protect the pipes from anticipated foundation settlement, or the utility lines should be backfilled to the bottom of footing with sand-cement slurry or lean concrete. Where utility lines parallel footings and will extend below the "foundation plane of influence," an imaginary 1:1 plane projected down from the bottom edge of the footing, either the footing will need to be deepened so that the pipe is above the foundation plane of influence or the utility trench will need to be backfilled with sand-cement slurry or lean concrete within the influence zone. Sand-cement slurry used within foundation influence zones should have a minimum compressive strength of 75 psi.

Footing excavations should be filled as soon as possible or be kept moist until concrete placement by regular sprinkling to prevent desiccation. Additionally, the footings will be in sands with low fines content and will likely slough and not stand vertical. Excavation sidewalls may need to be sloped to a minimum 1.5:1 (horizontal:vertical) inclination where footings or Stay-Form or similar may need to be placed within footing excavations as they are excavated during construction of the foundation elements. In addition, depending on how the excavations are cut, if the footing subgrade is loosened, the footing bottoms will need to be re-compacted in place to at least 90 percent relative compaction. A Cornerstone representative should observe all footing excavations prior to placing reinforcing steel and concrete. If there is a significant schedule delay between our initial observation and concrete placement, we may need to re-observe the excavations.

SECTION 9: PEDESTRIAN PAVEMENTS

9.1 EXTERIOR FLATWORK

Exterior concrete flatwork subject to pedestrian traffic only should be at least 4 inches thick and supported on compacted subgrade prepared in accordance with the "Earthwork" recommendations of this report. Flatwork that will be subject to heavier or frequent vehicular loading should be designed in accordance with the recommendations in the "Vehicular Pavements" section below. To help reduce the potential for uncontrolled shrinkage cracking, adequate expansion and control joints should be included. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness. Flatwork should be isolated from adjacent foundations or retaining walls.

SECTION 10: VEHICULAR PAVEMENTS

10.1 ASPHALT CONCRETE

The following asphalt concrete pavement recommendations tabulated below are based on the Procedure 608 of the Caltrans Highway Design Manual, estimated traffic indices for various pavement-loading conditions, and on a design R-value of 20. The design R-value was chosen based on our engineering judgement considering the soil type and variable surface conditions.

Table 8: Asphalt Concrete Pavement Recommendations, Design R-value = 20

Design Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base* (inches)	Total Pavement Section Thickness (inches)
4.0	2.5	5.5	8.0
4.5	2.5	7.0	9.5
5.0	3.0	7.0	10.0
5.5	3.0	9.0	12.0
6.0	3.5	9.5	13.0
6.5	4.0	10.5	14.5

*Caltrans Class 2 aggregate base; minimum R-value of 78

Frequently, the full asphalt concrete section is not constructed prior to construction traffic loading. This can result in significant loss of asphalt concrete layer life, rutting, or other pavement failures. To improve the pavement life and reduce the potential for pavement distress through construction, we recommend the full design asphalt concrete section be constructed prior to construction traffic loading. Alternatively, a higher traffic index may be chosen for the areas where construction traffic will be using the pavements.

10.2 PORTLAND CEMENT CONCRETE DRIVEWAYS

The exterior Portland Cement Concrete (PCC) pavement recommendations tabulated below are based on methods presented in the Portland Cement Association (PCA) design manual (PCA, 1984). We have provided a few pavement alternatives as an anticipated Average Daily Truck Traffic (ADTT) was not provided. An allowable ADTT should be chosen that is greater than what is expected for the development.

Table 9: PCC Pavement Recommendations, Design R-value = 20

Allowable ADTT	Minimum PCC Thickness (inches)
4	5.0
57	5.5
480	6.0

The PCC thicknesses above are based on a concrete compressive strength of at least 3,500 psi, supporting the PCC on at least 6 inches of Class 2 aggregate base compacted as recommended in the "Earthwork" section, and laterally restraining the PCC with curbs or concrete shoulders. Adequate expansion and control joints should be included. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness.

SECTION 11: RETAINING WALLS

11.1 STATIC LATERAL EARTH PRESSURES

The structural design of any site retaining wall should include resistance to lateral earth pressures that develop from the soil behind the wall, any undrained water pressure, and surcharge loads acting behind the wall. Provided a drainage system is constructed behind the wall to prevent the build-up of hydrostatic pressures as discussed in the section below, we recommend that the walls with level backfill be designed for the following pressures:

Table 10: Recommended Lateral Earth Pressures

Wall Condition	Lateral Earth Pressure*	Additional Surcharge Loads
Unrestrained – Cantilever Wall	35 pcf	⅓ of vertical loads at top of wall
Restrained – Braced Wall	35 pcf + 8H** psf	½ of vertical loads at top of wall

* Lateral earth pressures are based on an equivalent fluid pressure for level backfill conditions
** H is the distance in feet between the bottom of footing and top of retained soil

The below-grade walls for the pump station should be designed as restrained walls. If adequate drainage cannot be provided behind the wall, an additional equivalent fluid pressure of 40 pcf should be added to the values above for both restrained and unrestrained walls for the portion of the wall that will not have drainage. Damp proofing or waterproofing of the walls may be considered where moisture penetration and/or efflorescence are not desired.

11.1.1 Surcharge Loading on Pump Station South Wall from Reservoir Tank

Based on plans provided, the northern reservoir tank is 24 feet south of the southern wall for the pump station. Additionally, global interior dead + fluid load is 1,575 psf beneath the tank with 1,950 psf (dead + roof live load + fluid) and 2,550 psf (dead + fluid + seismic) beneath the perimeter tank footing. With these pressures, current layout of the tanks and pump station, bottom of tank elevation at Elevation 199 feet, finished grade about Elevation 198 to 199 feet on south side of pump station, and pump station finished floor Elevation 190 feet, the pump station southern wall will be subjected to addition surcharge loading from the northern reservoir tank. Based on the above understanding and assumptions, we recommend the pump station southern wall be designed for an additional 20 psf surcharge load in the upper 5 feet of the wall (Elevation 194 to 199 feet), an additional 55 psf surcharge load at a depth of 5 to 10 feet (Elevation 189 to 194 feet), and an additional 80 psf surcharge load below a depth of 10 feet (Elevation 189 feet). We recommend we be retained to review the final site layout and tank loading, and verify the surcharge loading.

11.2 SEISMIC LATERAL EARTH PRESSURES

The 2019 California Building Code (CBC) states that lateral pressures from earthquakes should be considered in the design of basements and retaining walls. We developed seismic earth pressures for the proposed below-grade pump station walls using interim recommendations generally based on refinement of the Mononobe-Okabe method (Lew et al., SEAOC 2010). We checked the result of the total seismic increment when added to the recommended active earth pressure against the recommended fixed (restrained) wall earth pressures. Because the wall is restrained, or will act as a restrained wall, and will be designed for 35 pcf (equivalent fluid pressure) plus a uniform earth pressure of 8H psf, based on current recommendations for seismic earth pressures, it appears that active earth pressures plus a seismic increment do not exceed the fixed wall earth pressures. Therefore, an additional seismic increment above the design earth pressures is not required as long as the walls are designed for the restrained wall earth pressures recommended above in accordance with the CBC.

11.3 WALL DRAINAGE

11.3.1 At-Grade Site Walls

Adequate drainage should be provided by a subdrain system behind all walls. This system should consist of a 4-inch minimum diameter perforated pipe placed near the base of the wall (perforations placed downward). The pipe should be bedded and backfilled with Class 2 Permeable Material per Caltrans Standard Specifications, latest edition. The permeable backfill should extend at least 12 inches out from the wall and to within 2 feet of outside finished grade. Alternatively, ½-inch to ¾-inch crushed rock may be used in place of the Class 2 Permeable Material provided the crushed rock and pipe are enclosed in filter fabric, such as Mirafi 140N or approved equivalent. The upper 2 feet of wall backfill should consist of compacted on-site soil. The subdrain outlet should be connected to a free-draining outlet or sump.

Miradrain, Geotech Drainage Panels, or equivalent drainage matting can be used for wall drainage as an alternative to the Class 2 Permeable Material or drain rock backfill. Horizontal strip drains connecting to the vertical drainage matting may be used in lieu of the perforated pipe and crushed rock section. The vertical drainage panel should be connected to the perforated pipe or horizontal drainage strip at the base of the wall, or to some other closed or through-wall system such as the TotalDrain system from AmerDrain. Sections of horizontal drainage strips should be connected with either the manufacturer’s connector pieces or by pulling back the filter fabric, overlapping the panel dimples, and replacing the filter fabric over the connection. At corners, a corner guard, corner connection insert, or a section of crushed rock covered with filter fabric must be used to maintain the drainage path.

Drainage panels should terminate 18 to 24 inches from final exterior grade. The Miradrain panel filter fabric should be extended over the top of and behind the panel to protect it from intrusion of the adjacent soil.

11.3.2 Below-Grade Walls

Miradrain, AmerDrain or other equivalent drainage matting may be used for wall drainage where below-grade walls are temporarily shored and the shoring will be flush with the back of the permanent walls. The drainage panel should be connected at the base of the wall by a horizontal drainage strip and closed or through-wall system such as the TotalDrain system from AmerDrain.

Sections of horizontal drainage strips should be connected with either the manufacturer's connector pieces or by pulling back the filter fabric, overlapping the panel dimples, and replacing the filter fabric over the connection. At corners, a corner guard, corner connection insert, or a section of crushed rock covered with filter fabric must be used to maintain the drainage path.

Drainage panels should terminate 18 to 24 inches from final exterior grade unless capped by hardscape. The drainage panel filter fabric should be extended over the top of and behind the panel to protect it from intrusion of the adjacent soil. If the shoring system will be offset behind the back of permanent wall, the drainage systems discussed in the "At-Grade Site Walls" section may also be used.

11.4 BACKFILL

Where surface improvements will be located over the retaining wall backfill, backfill placed behind the walls should be compacted to at least 95 percent relative compaction using light compaction equipment. Where no surface improvements are planned, backfill should be compacted to at least 90 percent. If heavy compaction equipment is used, the walls should be temporarily braced.

Consideration should be given to the transitions from on-grade to on-structure. Subslabs or other methods for reducing differential movement of flatwork or pavements across this transition should be included in the project design.

11.5 FOUNDATIONS

Retaining walls for the pump station building may be supported on a continuous spread footing designed in accordance with the recommendations presented in the "Foundations" section of this report.

At this time, we are not aware of other retaining walls for the project. If other site retaining walls are proposed, we should be consulted to provide additional recommendations as recommendations may vary based on location due to the potential for undocumented fills and loose to medium dense native soils with the potential for dry sand seismic settlements.

SECTION 12: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of Schaaf and Wheeler specifically to support the design of the MCWD A1/A2 Reservoirs and B/C Booster Pump Station project in Marina, California. The opinions, conclusions, and recommendations presented in this report have been formulated in accordance with accepted geotechnical engineering practices that exist in Northern California at the time this report was prepared. No warranty, expressed or implied, is made or should be inferred.

Recommendations in this report are based upon the soil and groundwater conditions encountered during our subsurface exploration. If variations or unsuitable conditions are encountered during construction, Cornerstone must be contacted to provide supplemental recommendations, as needed.

Schaaf and Wheeler may have provided Cornerstone with plans, reports and other documents prepared by others. Schaaf and Wheeler understands that Cornerstone reviewed and relied on the information presented in these documents and cannot be responsible for their accuracy.

Cornerstone prepared this report with the understanding that it is the responsibility of the owner or his representatives to see that the recommendations contained in this report are presented to other members of the design team and incorporated into the project plans and specifications, and that appropriate actions are taken to implement the geotechnical recommendations during construction.

Conclusions and recommendations presented in this report are valid as of the present time for the development as currently planned. Changes in the condition of the property or adjacent properties may occur with the passage of time, whether by natural processes or the acts of other persons. In addition, changes in applicable or appropriate standards may occur through legislation or the broadening of knowledge. Therefore, the conclusions and recommendations presented in this report may be invalidated, wholly or in part, by changes beyond Cornerstone's control. This report should be reviewed by Cornerstone after a period of three (3) years has elapsed from the date of this report. In addition, if the current project design is changed, then Cornerstone must review the proposed changes and provide supplemental recommendations, as needed.

An electronic transmission of this report may also have been issued. While Cornerstone has taken precautions to produce a complete and secure electronic transmission, please check the electronic transmission against the hard copy version for conformity.

Recommendations provided in this report are based on the assumption that Cornerstone will be retained to provide observation and testing services during construction to confirm that conditions are similar to that assumed for design, and to form an opinion as to whether the work has been performed in accordance with the project plans and specifications. If we are not retained for these services, Cornerstone cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of Cornerstone's report by others. Furthermore, Cornerstone will cease to be the Geotechnical-Engineer-of-Record if we are not retained for these services.

SECTION 13: REFERENCES

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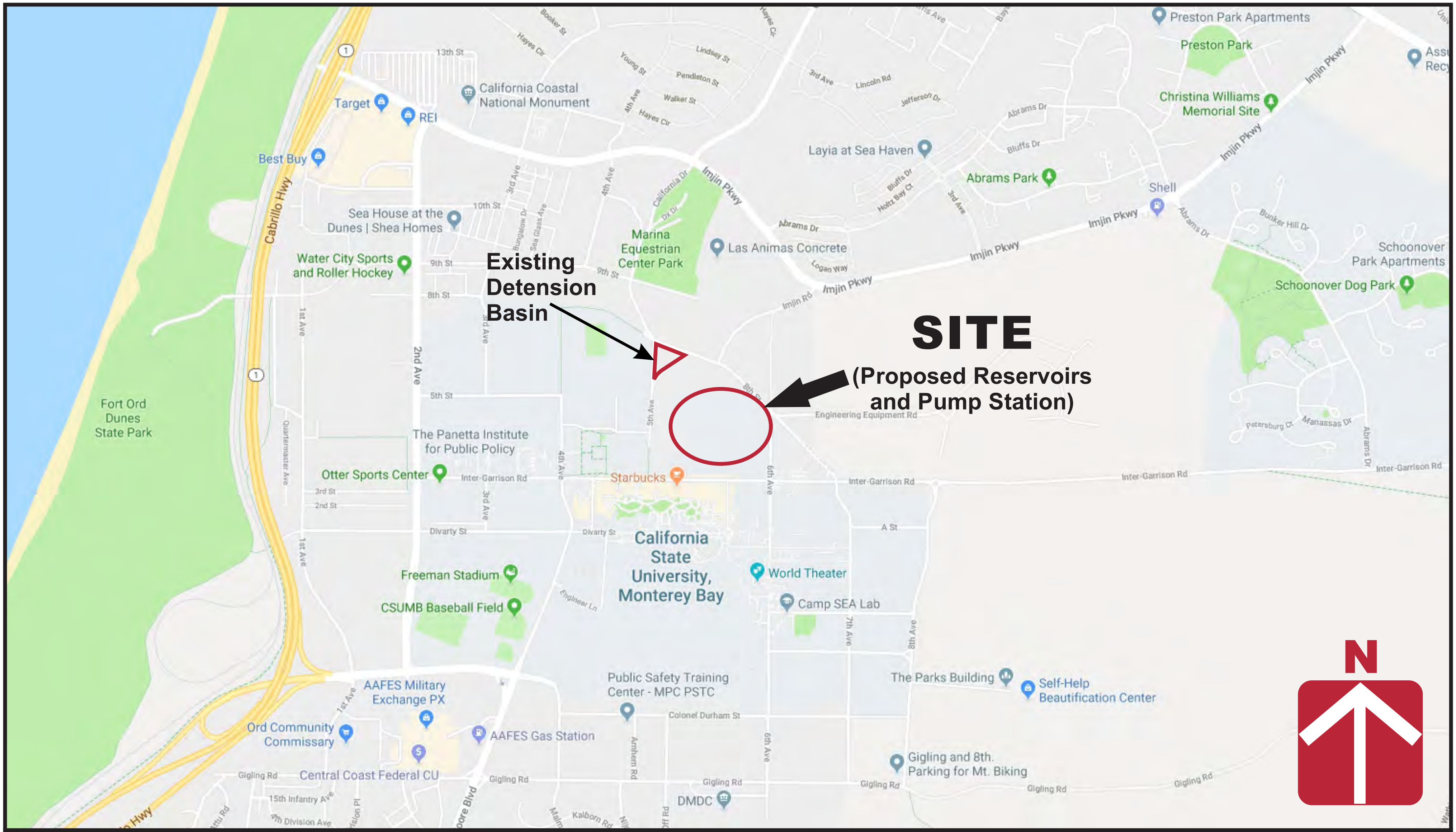

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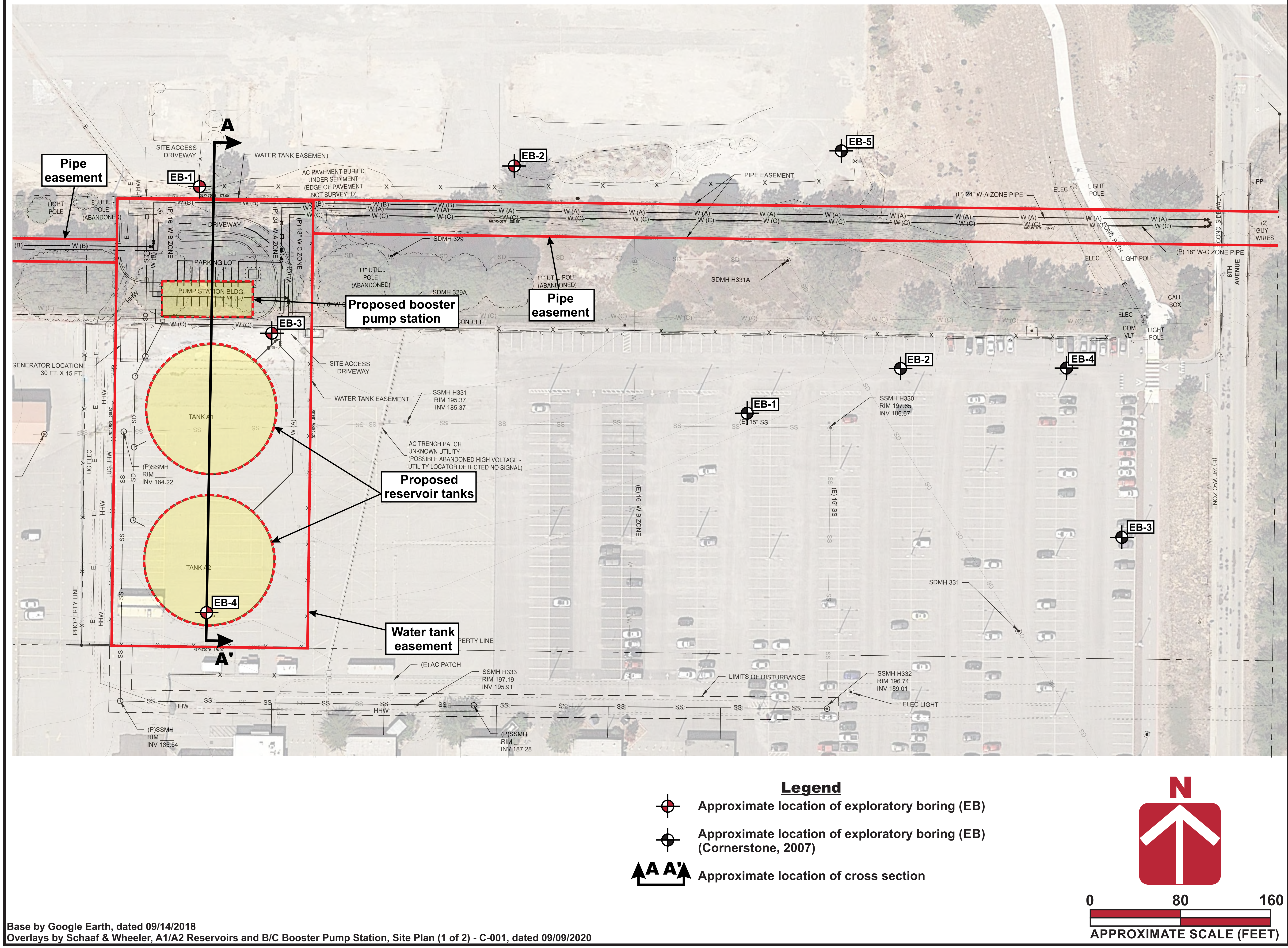
Vicinity Map

MCWD A1/A2 Reservoirs and B/C Booster Pump Station Marina, CA

Project Number	187-55-1
Figure Number	Figure 1
Date	December 2020
Drawn By	RRN



Legend

- Approximate location of exploratory boring (EB)
- Approximate location of exploratory boring (EB) (Cornerstone, 2007)
- A A' Approximate location of cross section

Base by Google Earth, dated 09/14/2018

Overlays by Schaaf & Wheeler, A1/A2 Reservoirs and B/C Booster Pump Station, Site Plan (1 of 2) - C-001, dated 09/09/2020

Project Number 187-55-1

Figure Number Figure 2

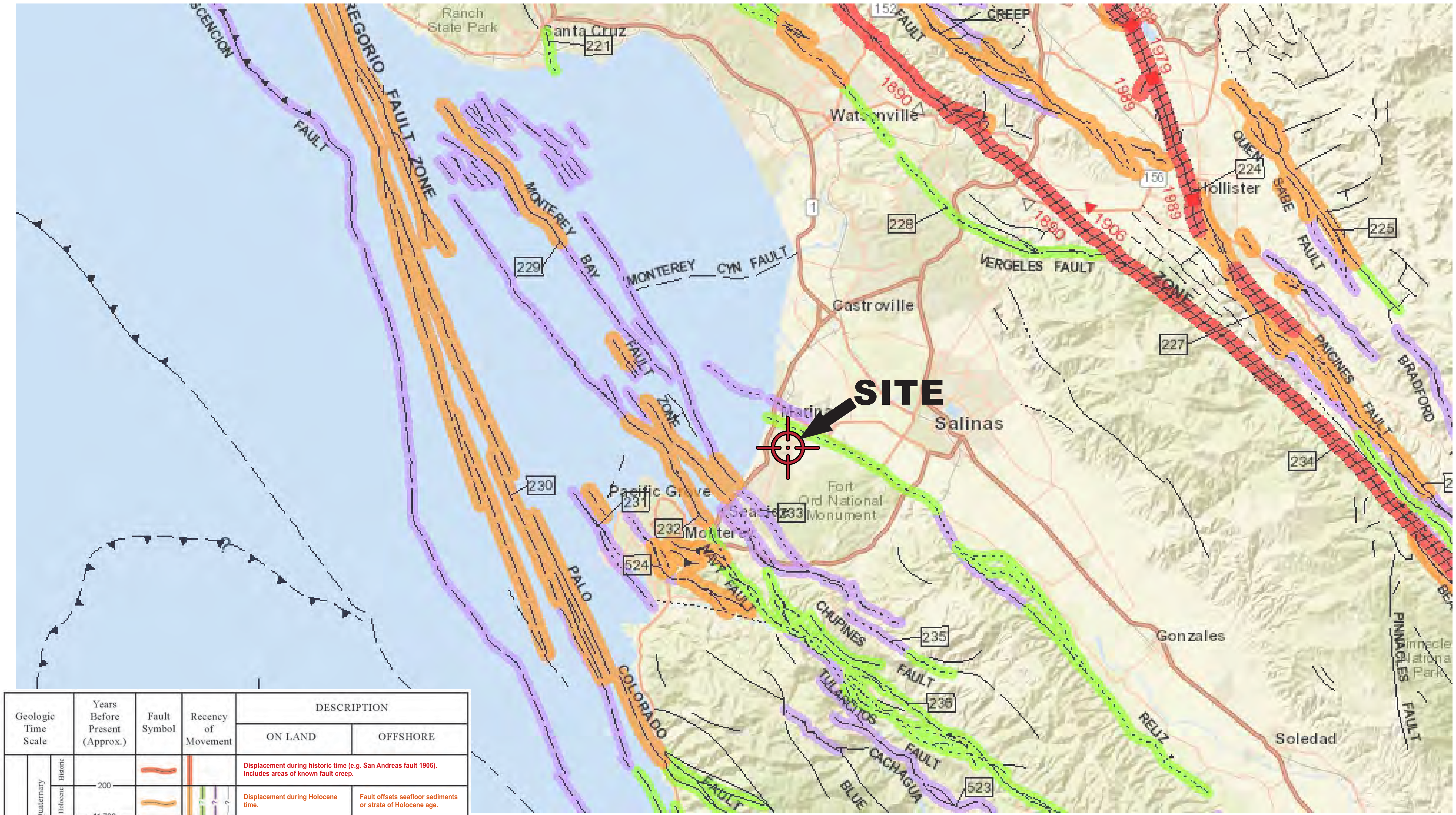
Date December 2020

Drawn By RRN

Site Plan

MCWD A1/A2 Reservoirs and B/C Booster Pump Station Marina, CA

CORNERSTONE EARTH GROUP



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION	
						ON LAND	OFFSHORE
Quaternary	Late Quaternary	Historic		🟥		Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		Holocene	200	🟧	? ? ?	Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
		Pleistocene	11,700	🟩	? ?	Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
	Early Quaternary		700,000	🟪	?	Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Pilo-Pleistocene age.	Fault cuts strata of Quaternary age.
Pre-Quaternary			1,600,000 4.5 billion (Age of Earth)	~		Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.

Base by California Geological Survey - 2010 Fault Activity Map of California (Jennings and Bryant, 2010)

N

0 5 10

APPROXIMATE SCALE (MILES)

Project Number 187-55-1

Figure Number Figure 3

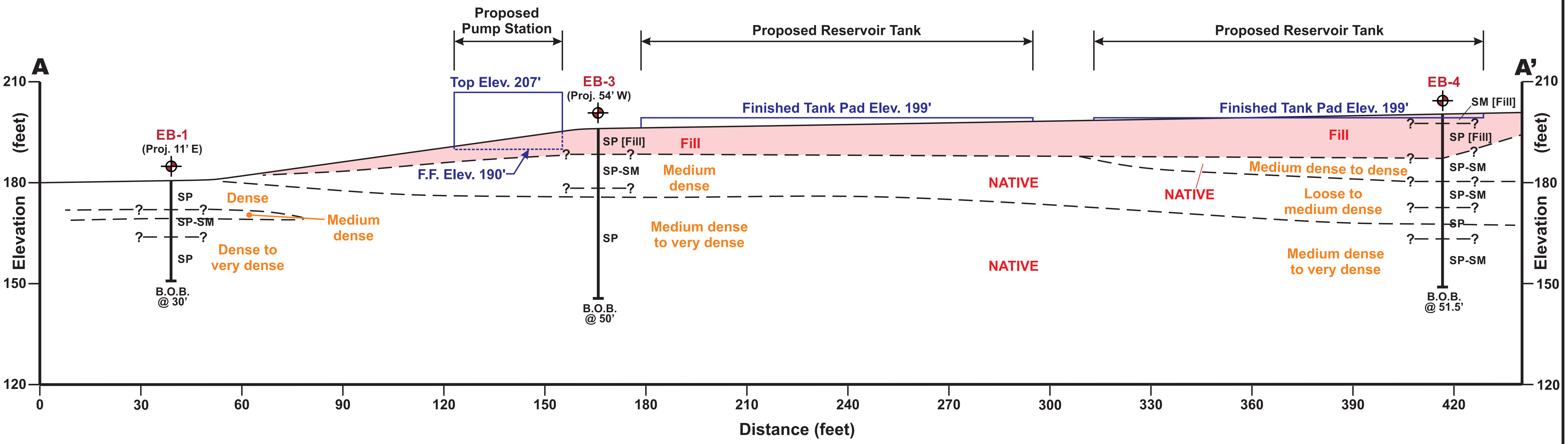
Date December 2020

Drawn By RRN

Regional Fault Map

MCWD A1/A2 Reservoirs and B/C Booster Pump Station
Marina, CA

CORNERSTONE EARTH GROUP



Section A-A'

(View Looking East)
1"=30' H:V

Symbols

SP	Poorly Graded Sand
SM	Silty Sand
SP-SM	Poorly Graded Sand with Silt
⊕	Approximate location of exploratory boring (EB)

Notes:
1) Surficial fills associated with existing pavements, landscaping or utilities are not shown.
2) The subsurface profile is conceptual and is based on limited subsurface data obtained from widely spaced borings. Actual subsurface conditions may vary significantly between borings.
3) See Figure 2 for location of cross section.
4) Referenced elevations based on NAVD88 datum.

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Cross Section A-A'

**MCWD A1/A2 Reservoirs and
B/C Booster Pump Station
Marina, CA**

Project Number
187-55-1

Figure Number
Figure 4

Date
December 2020

Drawn By
RRN

Legend

🟥 Approximate location of infiltration test (I)

0 60 120
APPROXIMATE SCALE (FEET)

Base by Google Earth, dated 11/15/2018

Existing Detention Basin - Infiltration Testing Location

MCWD A1/A2 Reservoirs and B/C Booster Pump Station
Marina, CA

Project Number 187-55-1
Figure Number Figure 5
Date December 2020
Drawn By RRN

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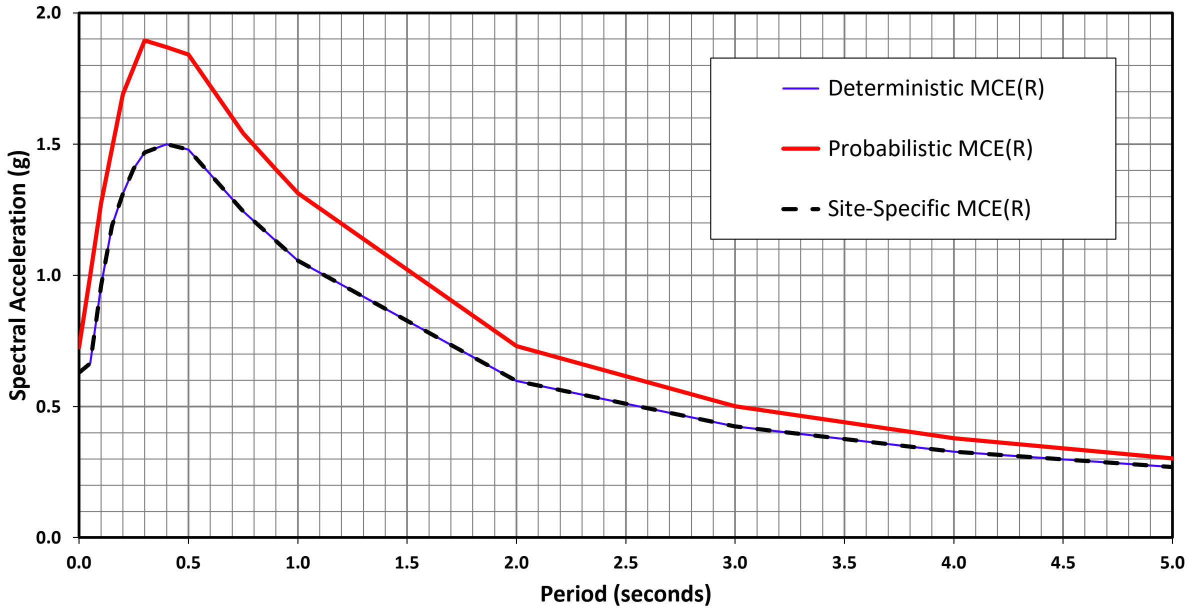

The Site-Specific Maximum Considered Earthquake (MCE_R) is defined as the lesser of the following at all periods:

- Deterministic MCE_R – maximum 84th percentile deterministic, or
- Probabilistic MCE_R – defined as the 2,475–year ground motion.

Site-Specific MCE_R	
Period (Seconds)	Spectral Acceleration (g)
0.00	0.629
0.05	0.665
0.10	0.961
0.15	1.189
0.18	1.265
0.20	1.311
0.25	1.408
0.30	1.469
0.40	1.500
0.50	1.479
0.75	1.244
0.90	1.129
1.00	1.057
2.00	0.597
3.00	0.424
4.00	0.328
5.00	0.269

References:

ASCE/SEI 7-16: Minimum Design Loads and Associated Criteria for Buildings and Other Strutures with Supplement No. 1.
2019 California Building Code, Title 24, Part 2, Volume 2

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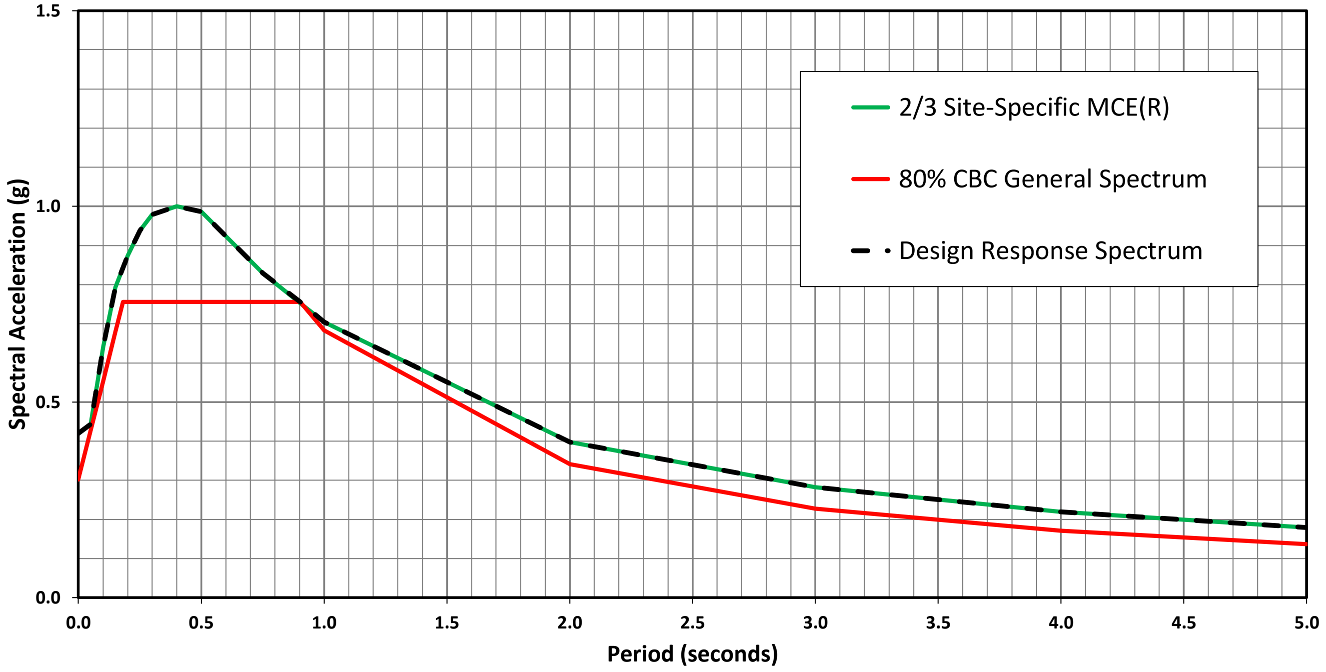
MCE_R RESPONSE SPECTRA

FIGURE 6

MCWD A1/A2 Reservoirs and
B/C Booster Pump Station
Marina, California

PROJECT NO. 187-55-1

December 22, 2020 MJS



The Site-Specific Design Response Spectrum per Section 21.2, 21.3 and 21.4 of ASCE 7-16 is defined as the greater of the following at all periods:

- 2/3 of the Site-Specific MCE_R, or
- 80% of the CBC General Spectrum.

Design Response Spectra	
Period (Seconds)	Spectral Acceleration (g)
0.00	0.419
0.05	0.443
0.10	0.641
0.15	0.793
0.18	0.843
0.20	0.874
0.25	0.938
0.30	0.979
0.40	1.000
0.50	0.986
0.75	0.829
0.90	0.756
1.00	0.704
2.00	0.398
3.00	0.282
4.00	0.219
5.00	0.179

Site Design	Design Values
Site Class (Per Chapter 20 ASCE 7-16)	D
Shear Wave Velocity, V_{S30} (m/sec)	230
Site Latitude (degrees)	36.656597
Site Longitude (degrees)	-121.796108
Risk Category	IV
Building Period (sec)	0.12
Importance Factor, I_e	1.5
[1]Site Specific PGA_M (g)	0.550

[1] Lower of Deterministic and Probabilistic, but not less than 80% of mapped value of FM x PGA, determined in accordance with Section 21.5 of ASCE 7-16.

Design Acceleration Parameters[1]	
S_{DS}	0.900
S_{D1}	0.895
S_{MS}	1.350
S_{M1}	1.343

References:

ASCE/SEI 7-16: Minimum Design Loads and Associated Criteria for Buildings and Other Strutures with Supplement No. 1.
2019 California Building Code, Title 24, Part 2, Volume 2

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DESIGN RESPONSE SPECTRA

FIGURE 7

MCWD A1/A2 Reservoirs and
B/C Booster Pump Station
Marina, California

PROJECT NO. 187-55-1

December 22, 2020 | MJS

APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using truck-mounted, hollow-stem auger drilling equipment. Four 8-inch-diameter exploratory borings were drilled on June 26, 2019 to depths ranging from 21½ to 51½ feet. The approximate locations of exploratory borings are shown on the Site Plan, Figure 2. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). Boring logs, as well as a key to the classification of the soil, are included as part of this appendix.

Boring locations were approximated using existing site boundaries, a hand held GPS unit, and other site features as references. Boring elevations were estimated based on the topographic map provided by Whitson Engineers. The locations and elevations of the borings should be considered accurate only to the degree implied by the method used.

Representative soil samples were obtained from the borings at selected depths. All samples were returned to our laboratory for evaluation and appropriate testing. The standard penetration resistance blow counts were obtained by dropping a 140-pound hammer through a 30-inch free fall. The 2-inch O.D. split-spoon sampler was driven 18 inches and the number of blows was recorded for each 6 inches of penetration (ASTM D1586). 2.5-inch I.D. samples were obtained using a Modified California Sampler driven into the soil with the 140-pound hammer previously described. Unless otherwise indicated, the blows per foot recorded on the boring log represent the accumulated number of blows required to drive the last 12 inches. The various samplers are denoted at the appropriate depth on the boring logs.

Attached boring logs and related information depict subsurface conditions at the locations indicated and on the date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these boring locations. The passage of time may result in altered subsurface conditions due to environmental changes. In addition, any stratification lines on the logs represent the approximate boundary between soil types and the transition may be gradual.

UNIFIED SOIL CLASSIFICATION (ASTM D-2487-10)

MATERIAL TYPES	CRITERIA FOR ASSIGNING SOIL GROUP NAMES			GROUP SYMBOL	SOIL GROUP NAMES & LEGEND	
COARSE-GRAINED SOILS >50% RETAINED ON NO. 200 SIEVE	GRAVELS >50% OF COARSE FRACTION RETAINED ON NO 4. SIEVE	CLEAN GRAVELS <5% FINES	Cu>4 AND 1<Cc<3	GW	WELL-GRADED GRAVEL	▨
			Cu>4 AND 1>Cc>3	GP	POORLY-GRADED GRAVEL	▨
		GRAVELS WITH FINES >12% FINES	FINES CLASSIFY AS ML OR CL	GM	SILTY GRAVEL	▨
			FINES CLASSIFY AS CL OR CH	GC	CLAYEY GRAVEL	▨
	SANDS >50% OF COARSE FRACTION PASSES ON NO 4. SIEVE	CLEAN SANDS <5% FINES	Cu>6 AND 1<Cc<3	SW	WELL-GRADED SAND	▨
			Cu>6 AND 1>Cc>3	SP	POORLY-GRADED SAND	▨
		SANDS AND FINES >12% FINES	FINES CLASSIFY AS ML OR CL	SM	SILTY SAND	▨
			FINES CLASSIFY AS CL OR CH	SC	CLAYEY SAND	▨
FINE-GRAINED SOILS >50% PASSES NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT<50	INORGANIC	PI>7 AND PLOTS>"A" LINE	CL	LEAN CLAY	▨
			PI>4 AND PLOTS<"A" LINE	ML	SILT	▥
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OL	ORGANIC CLAY OR SILT	▤
	SILTS AND CLAYS LIQUID LIMIT>50	INORGANIC	PI PLOTS >"A" LINE	CH	FAT CLAY	▨
			PI PLOTS <"A" LINE	MH	ELASTIC SILT	▥
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OH	ORGANIC CLAY OR SILT	▤
HIGHLY ORGANIC SOILS		PRIMARILY ORGANIC MATTER, DARK IN COLOR, AND ORGANIC ODOR		PT	PEAT	▨

OTHER MATERIAL SYMBOLS

▨	Poorly-Graded Sand with Clay	▨	Sand
▨	Clayey Sand	▥	Silt
▥	Sandy Silt	▨	Well Graded Gravelly Sand
▩	Artificial/Undocumented Fill	▥	Gravelly Silt
▨	Poorly-Graded Gravelly Sand	■	Asphalt
▨	Topsoil	▨	Boulders and Cobble
▨	Well-Graded Gravel with Clay		
▨	Well-Graded Gravel with Silt		

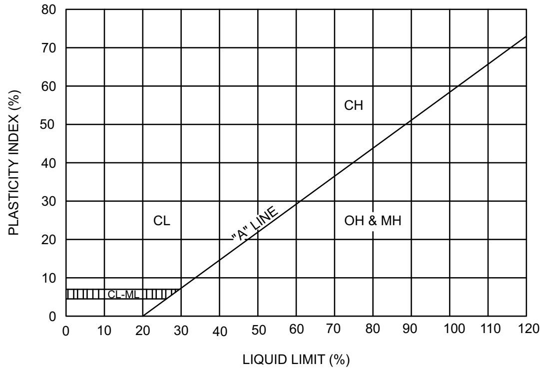
SAMPLER TYPES

☒	SPT	■	Shelby Tube
⧗	Modified California (2.5" I.D.)	○	No Recovery
▯	Rock Core	✋	Grab Sample

ADDITIONAL TESTS

CA	-	CHEMICAL ANALYSIS (CORROSIVITY)	PI	-	PLASTICITY INDEX
CD	-	CONSOLIDATED DRAINED TRIAXIAL	SW		SWELL TEST
CN	-	CONSOLIDATION	TC	-	CYCLIC TRIAXIAL
CU	-	CONSOLIDATED UNDRAINED TRIAXIAL	TV	-	TORVANE SHEAR
DS	-	DIRECT SHEAR	UC	-	UNCONFINED COMPRESSION
PP	-	POCKET PENETROMETER (TSF)	(1.5)	-	(WITH SHEAR STRENGTH IN KSF)
(3.0)	-	(WITH SHEAR STRENGTH IN KSF)			
RV	-	R-VALUE	UU	-	UNCONSOLIDATED UNDRAINED TRIAXIAL
SA	-	SIEVE ANALYSIS: % PASSING #200 SIEVE			
▼	-	WATER LEVEL			

PLASTICITY CHART



PENETRATION RESISTANCE
(RECORDED AS BLOWS / FOOT)

SAND & GRAVEL		SILT & CLAY		
RELATIVE DENSITY	BLOWS/FOOT*	CONSISTENCY	BLOWS/FOOT*	STRENGTH** (KSF)
VERY LOOSE	0 - 4	VERY SOFT	0 - 2	0 - 0.25
LOOSE	4 - 10	SOFT	2 - 4	0.25 - 0.5
MEDIUM DENSE	10 - 30	MEDIUM STIFF	4 - 8	0.5-1.0
DENSE	30 - 50	STIFF	8 - 15	1.0 - 2.0
VERY DENSE	OVER 50	VERY STIFF	15 - 30	2.0 - 4.0
		HARD	OVER 30	OVER 4.0

* NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 INCHES TO DRIVE A 2 INCH O.D. (1-3/8 INCH I.D.) SPLIT-BARREL SAMPLER THE LAST 12 INCHES OF AN 18-INCH DRIVE (ASTM-1586 STANDARD PENETRATION TEST).

** UNDRAINED SHEAR STRENGTH IN KIPS/SQ. FT. AS DETERMINED BY LABORATORY TESTING OR APPROXIMATED BY THE STANDARD PENETRATION TEST, POCKET PENETROMETER, TORVANE, OR VISUAL OBSERVATION.

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LEGEND TO SOIL DESCRIPTIONS

Figure Number
A-1

BORING NUMBER EB-1

CORNERSTONE EARTH GROUP

PROJECT NAME MCWD A1/A2 Reservoirs & B/C Booster Pump Station
PROJECT NUMBER 187-55-1
PROJECT LOCATION 8th Street and 6th Avenue, Marina, CA
DATE STARTED 6/26/19 **DATE COMPLETED** 6/26/19
GROUND ELEVATION 180.5 FT +/- **BORING DEPTH** 30 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 36.656840° **LONGITUDE** -121.796500°
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
GROUNDWATER LEVELS:
LOGGED BY JLC
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION
180.5 180.3	0	2 inches asphalt concrete over 4 inches aggregate base
180.0		**Poorly Graded Sand (SP)** dense, moist, yellowish brown, fine sand
	5	
171.5		**Poorly Graded Sand with Silt (SP-SM)** medium dense, moist, yellowish brown to brown, fine sand
	10	becomes dense
	15	
163.5		**Poorly Graded Sand (SP)** very dense, moist, light brown, fine sand
	20	
	25	
150.5	30	Bottom of Boring at 30.0 feet.

N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
63	MC-1B	104	5		
46	SPT				
64	MC-3B	101	3		1
39	SPT				
46	MC-5B	104	7		
35	SPT				
32	SPT-7		13		
36	SPT				
72	SPT-9		4		
59	SPT-10		4		
72	SPT				

UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
1.0 2.0 3.0 4.0

CORNERSTONE EARTH GROUP

BORING NUMBER EB-2

PAGE 1 OF 1

PROJECT NAME MCWD A1/A2 Reservoirs & B/C Booster Pump Station
PROJECT NUMBER 187-55-1
PROJECT LOCATION 8th Street and 6th Avenue, Marina, CA
DATE STARTED 6/26/19 **DATE COMPLETED** 6/26/19
GROUND ELEVATION 182.5 FT +/- **BORING DEPTH** 21.5 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 36.656867° **LONGITUDE** -121.795550°
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
GROUNDWATER LEVELS:
LOGGED BY JLC
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL; 1.0 2.0 3.0 4.0

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
182.5	0	3 inches asphalt concrete over 4 inches aggregate base							
182.3									
181.9		**Poorly Graded Sand (SP)** dense, moist, yellowish brown, fine sand							
			22	SPT-1		7			
			42	MC-2B	103	5		1	
	5								
			26	SPT					
			50	MC-4B	103	4			
	10		32	SPT					
			32	SPT-6		4			
			43	SPT					
	15								
			39	SPT-8		2			
	20		38	SPT					
161.0		Bottom of Boring at 21.5 feet.							
	25								
	30								

BORING NUMBER EB-3

PAGE 1 OF 2

CORNERSTONE EARTH GROUP

PROJECT NAME MCWD A1/A2 Reservoirs & B/C Booster Pump Station
PROJECT NUMBER 187-55-1
PROJECT LOCATION 8th Street and 6th Avenue, Marina, CA
DATE STARTED 6/26/19 **DATE COMPLETED** 6/26/19
GROUND ELEVATION 195 FT +/- **BORING DEPTH** 50 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 36.656476° **LONGITUDE** -121.796277°
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
GROUNDWATER LEVELS:
LOGGED BY JLC
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
1.0 2.0 3.0 4.0

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
195.0 194.8	0	2 inches asphalt concrete over 4 inches aggregate base						
194.5		**Poorly Graded Sand (SP) [Fill]** very dense, moist, brown and yellowish brown mottled, fine sand	48	SPT-1		5		
			50 4"	MC-2B	108	5		
	5							
		becomes dense	33	SPT				
188.0		**Poorly Graded Sand with Silt (SP-SM)** medium dense, moist, brown to light brown, fine sand	30	MC-4B	99	4		9
	10		17	SPT-5		2		
			16	SPT-6		2		6
		becomes dense, color changes to light brown	36	SPT				
	15							
		becomes medium dense	23	SPT-8		3		8
178.0		**Poorly Graded Sand (SP)** medium dense, moist, light brown, fine sand						
			26	SPT-9		2		
	20							
		becomes dense	40	SPT				
		becomes very dense	50 6"	SPT-11		2		
	25							
		becomes medium dense	32	SPT-12		4		
	30							
164.0			33	SPT				
163.5								

Continued Next Page

PROJECT NAME MCWD A1/A2 Reservoirs & B/C Booster Pump Station

PROJECT NUMBER 187-55-1

PROJECT LOCATION 8th Street and 6th Avenue, Marina, CA

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ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf ○ HAND PENETROMETER △ TORVANE ● UNCONFINED COMPRESSION ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL 1.0 2.0 3.0 4.0
163.5			**Poorly Graded Sand (SP)** dense, moist, light brown, fine sand							
	35			49	SPT-14		4			
	40			52	SPT					
	45			52	SPT-16		3			
			some medium sand	60	SPT					
145.0	50		Bottom of Boring at 50.0 feet.							
	55									
	60									
	65									

BORING NUMBER EB-4

PAGE 1 OF 2

CORNERSTONE EARTH GROUP

PROJECT NAME MCWD A1/A2 Reservoirs & B/C Booster Pump Station
PROJECT NUMBER 187-55-1
PROJECT LOCATION 8th Street and 6th Avenue, Marina, CA
DATE STARTED 6/26/19 **DATE COMPLETED** 6/26/19
GROUND ELEVATION 200.5 FT +/- **BORING DEPTH** 51.5 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 36.655799° **LONGITUDE** -121.796484°
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
GROUNDWATER LEVELS:
LOGGED BY JLC
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
1.0 2.0 3.0 4.0

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
200.5 200.3	0	2 inches asphalt concrete over 6 inches aggregate base						
199.8		**Silty Sand (SM) [Fill]** dense, moist, brown, fine sand	40	MC-1B	110	7		16
197.5		**Poorly Graded Sand (SP) [Fill]** dense, moist, light brown and brown mottled, fine sand	34	SPT				
	5	becomes very dense	50 4"	MC-3B	106	4		
		becomes dense	39	SPT				
	10		78	MC-5B	104	4		3
187.5		**Poorly Graded Sand with Silt (SP-SM)** dense, moist, brown and light brown mottled, fine sand	30	SPT				
	15	becomes medium dense	23	SPT-7		5		12
			21	SPT				
180.5	20	**Poorly Graded Sand with Silt (SP-SM)** loose, moist, brown to light brown, fine sand	12	SPT-9		6		
		becomes medium dense	11	SPT-10		5		7
	25		15	SPT				
172.5		**Poorly Graded Sand (SP)** medium dense, moist, light brown, fine sand	18	SPT-12		5		
	30		25	SPT				
169.0								

Continued Next Page

CORNERSTONE EARTH GROUP

PROJECT NAME MCWD A1/A2 Reservoirs & B/C Booster Pump Station
PROJECT NUMBER 187-55-1
PROJECT LOCATION 8th Street and 6th Avenue, Marina, CA

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UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
1.0 2.0 3.0 4.0

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
169.0			**Poorly Graded Sand (SP)** medium dense, moist, light brown, fine sand							
			becomes very dense	61	SPT-14		3			
	35									
163.5			**Poorly Graded Sand with Silt (SP-SM)** medium dense, moist, light brown, fine sand							
				30	SPT					
	40									
				28	SPT-16		8		5	
			becomes dense	45	SPT					
	45									
			becomes medium dense	30	SPT-18		6			
	50									
				33	SPT					
149.0			Bottom of Boring at 51.5 feet.							
	55									
	60									
	65									

APPENDIX B: LABORATORY TEST PROGRAM

The laboratory testing program was performed to evaluate the physical and mechanical properties of the soils retrieved from the site to aid in verifying soil classification.

Moisture Content: The natural water content was determined (ASTM D2216) on 32 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

Dry Densities: In place dry density determinations (ASTM D2937) were performed on 10 samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Washed Sieve Analyses: The percent soil fraction passing the No. 200 sieve (ASTM D1140) was determined on 10 samples of the subsurface soils to aid in the classification of these soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Corrosion: Two soluble sulfate determinations (ASTM D4327), resistivity tests (ASTM G57), chloride determinations (ASTM D4327), and pH determinations (ASTM G51) were performed on two representative samples of the subsurface soils. Results of these tests are attached to this appendix.

COOPER
TESTING LABORATORY

Corrosivity Tests Summary

CTL # 640-1328 **Date:** 7/10/2019 **Tested By:** PJ **Checked:** PJ
Client: Cornerstone Earth Group **Project:** Marina Pump Station **Proj. No:** 187-55-1
Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate		pH	ORP		Sulfide	Moisture	Soil Visual Description
			As Rec.	Min	Sat.	mg/kg	mg/kg	%		(Redox)		Qualitative	At Test	
						Dry Wt.	Dry Wt.	Dry Wt.		E_H (mv)	At Test	by Lead	%	
Boring	Sample, No.	Depth, ft.	ASTM G57	Cal 643	ASTM G57	ASTM D4327	ASTM D4327	ASTM D4327	ASTM G51	ASTM G200	Temp °C	Acetate Paper	ASTM D2216	
EB-2	2A	3.5	-	-	45,617	7	82	0.0082	7.3	-	-	-	5.7	Yellowish Brown Silty SAND
EB-4	1A	1.5	-	-	20,890	6	249	0.0249	6.8	-	-	-	8.0	Yellowish Brown Silty SAND

APPENDIX C: CORNERSTONE EARTH GROUP 2007 EXPLORATION LOGS AND LABORATORY DATA

APPENDIX A – FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using truck-mounted drilling equipment. Five 8-inch diameter exploratory borings were drilled on June 13 and 18, 2007 to depths of 20 to 50 feet. The approximate locations of exploratory borings are shown on the Site Plan, Figure 2. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). Boring logs, as well as a key to the classification of the soil, are included as part of this appendix.

Boring locations were approximated using existing site features site features as references using a hand held tape measure. Boring elevations were determined by interpolation of spot elevations shown on the boundary survey plan by others. The locations and elevations of the borings should be considered accurate only to the degree implied by the method used.

Representative soil samples were obtained from the borings at selected depths. All samples were returned to our laboratory for evaluation and appropriate testing. The standard penetration resistance blow counts were obtained by dropping a 140-pound hammer through a 30-inch free fall. The 2-inch O.D. split-spoon sampler was driven 18 inches and the number of blows was recorded for each 6 inches of penetration (ASTM D1586). 2.5-inch I.D. samples were obtained using a Modified California Sampler driven into the soil with the 140-pound hammer previously described. Unless otherwise indicated, the blows per foot recorded on the boring log represent the accumulated number of blows required to drive the last 12 inches. The various samplers are denoted at the appropriate depth on the boring logs.

Attached boring logs and related information depict subsurface conditions at the locations indicated and on the date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these boring locations. The passage of time may result in altered subsurface conditions due to environmental changes. In addition, any stratification lines on the logs represent the approximate boundary between soil types and the transition may be gradual.

UNIFIED SOIL CLASSIFICATION (ASTM D-2487-98)

MATERIAL TYPES	CRITERIA FOR ASSIGNING SOIL GROUP NAMES			GROUP SYMBOL	SOIL GROUP NAMES & LEGEND	
COARSE-GRAINED SOILS >50% RETAINED ON NO. 200 SIEVE	GRAVELS >50% OF COARSE FRACTION RETAINED ON NO 4. SIEVE	CLEAN GRAVELS <5% FINES	Cu>4 AND 1<Cc<3	GW	WELL-GRADED GRAVEL	▨
			Cu>4 AND 1>Cc>3	GP	POORLY-GRADED GRAVEL	▨
		GRAVELS WITH FINES >12% FINES	FINES CLASSIFY AS ML OR CL	GM	SILTY GRAVEL	▨
			FINES CLASSIFY AS CL OR CH	GC	CLAYEY GRAVEL	▨
	SANDS >50% OF COARSE FRACTION PASSES ON NO 4. SIEVE	CLEAN SANDS <5% FINES	Cu>6 AND 1<Cc<3	SW	WELL-GRADED SAND	▨
			Cu>6 AND 1>Cc>3	SP	POORLY-GRADED SAND	▨
		SANDS AND FINES >12% FINES	FINES CLASSIFY AS ML OR CL	SM	SILTY SAND	▨
			FINES CLASSIFY AS CL OR CH	SC	CLAYEY SAND	▨
FINE-GRAINED SOILS >50% PASSES NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT<50	INORGANIC	PI>7 AND PLOTS>"A" LINE	CL	LEAN CLAY	▨
			PI>4 AND PLOTS<"A" LINE	ML	SILT	▨
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OL	ORGANIC CLAY OR SILT	▨
	SILTS AND CLAYS LIQUID LIMIT>50	INORGANIC	PI PLOTS >"A" LINE	CH	FAT CLAY	▨
			PI PLOTS <"A" LINE	MH	ELASTIC SILT	▨
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OH	ORGANIC CLAY OR SILT	▨
HIGHLY ORGANIC SOILS		PRIMARILY ORGANIC MATTER, DARK IN COLOR, AND ORGANIC ODOR		PT	PEAT	▨

OTHER MATERIAL SYMBOLS

▨	Poorly-Graded Sand with Clay	▨	Sand
▨	Clayey Sand	▨	Silt
▨	Sandy Silt	▨	Well Graded Gravelly Sand
▨	Artificial/Undocumented Fill	▨	Gravelly Silt
▨	Poorly-Graded Gravelly Sand	■	Asphalt
▨	Topsoil	▨	Boulders and Cobble
▨	Well-Graded Gravel with Clay		
▨	Well-Graded Gravel with Silt		

SAMPLER TYPES

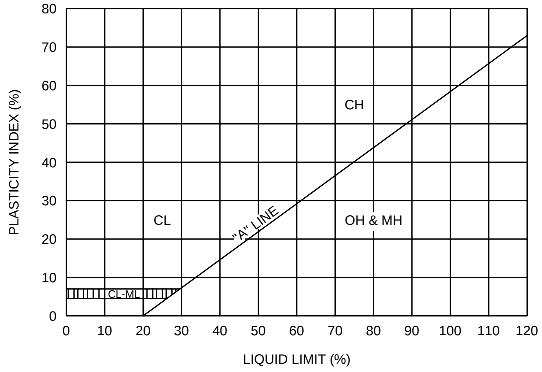
☒	SPT	■	Shelby Tube
⧗	Modified California (2.5" I.D.)	○	No Recovery
▯	Rock Core		

ADDITIONAL TESTS

CA - CHEMICAL ANALYSIS (CORROSIVITY)
CD - CONSOLIDATED DRAINED TRIAXIAL
CN - CONSOLIDATION
CU - CONSOLIDATED UNDRAINED TRIAXIAL
DS - DIRECT SHEAR
PP - POCKET PENETROMETER (TSF)
(3.0) - (WITH SHEAR STRENGTH IN KSF)
RV - R-VALUE
SA - SIEVE ANALYSIS: % PASSING #200 SIEVE
▼ - WATER LEVEL

PI - PLASTICITY INDEX
SW SWELL TEST
TC - CYCLIC TRIAXIAL
TV - TORVANE SHEAR
UC - UNCONFINED COMPRESSION
(1.5) - (WITH SHEAR STRENGTH IN KSF)
UU - UNCONSOLIDATED UNDRAINED TRIAXIAL

PLASTICITY CHART



PENETRATION RESISTANCE
(RECORDED AS BLOWS / FOOT)

SAND & GRAVEL		SILT & CLAY		
RELATIVE DENSITY	BLOWS/FOOT*	CONSISTENCY	BLOWS/FOOT*	STRENGTH** (KSF)
VERY LOOSE	0 - 4	VERY SOFT	0 - 2	0 - 0.5
LOOSE	4 - 10	SOFT	2 - 4	0.5 - 1.0
MEDIUM DENSE	10 - 30	MEDIUM STIFF	4 - 8	1.0 - 2.0
DENSE	30 - 50	STIFF	8 - 15	2.0 - 4.0
VERY DENSE	OVER 50	VERY STIFF	15 - 30	4.0 - 8.0
		HARD	OVER 30	OVER 8.0

* NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 INCHES TO DRIVE A 2 INCH O.D. (1-3/8 INCH I.D.) SPLIT-BARREL SAMPLER THE LAST 12 INCHES OF AN 18-INCH DRIVE (ASTM-1586 STANDARD PENETRATION TEST).

** UNDRAINED SHEAR STRENGTH IN KIPS/SQ. FT. AS DETERMINED BY LABORATORY TESTING OR APPROXIMATED BY THE STANDARD PENETRATION TEST, POCKET PENETROMETER, TORVANE, OR VISUAL OBSERVATION.

CORNERSTONE
EARTH GROUP

LEGEND TO SOIL DESCRIPTIONS

Figure Number
A-1

BORING NUMBER EB-1

PAGE 1 OF 2

CORNERSTONE EARTH GROUP

PROJECT NAME MCWD Reservoir and Pump Station
PROJECT NUMBER 142-1-1
PROJECT LOCATION Marina
DATE STARTED 6/18/07 **DATE COMPLETED** 6/18/07
GROUND ELEVATION 196 FT +/- **BORING DEPTH** 45 ft.
DRILLING CONTRACTOR EGI
LATITUDE **LONGITUDE**
DRILLING METHOD 8" HSA
GROUND WATER LEVELS:
LOGGED BY SEF
∇ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-VALUE (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT, PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE
196.0	0	2 inches AC over 6 inches aggregate base						
195.5		**Silty Sand (SM) [Fill]** loose, moist, dark brown	7	SPT-1		8		14
194.0		**Silty Sand (SM) [Fill]** loose, moist, light brown to brown, mottled	4	SPT-2		6		15
	5		7	SPT-3		9		16
190.0		**Silty Sand (SM) [Fill]** loose, moist, brown to yellow brown, mottled	11	SPT-4		10		15
186.5	10	**Poorly Graded Sand (SP) [Native]** loose, moist, yellow brown	10	SPT-5		7		6
182.0	15	**Poorly Graded Sand (SP)** dense, moist, yellow brown	35	SPT-6		3		3
	20		36	SPT-7		3		
	25		28	SPT-8		6		
	30		58	SPT-9		4		
161.0	35		48	SPT-10		4		

UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
1.0 2.0 3.0 4.0 5.0

Continued Next Page

BORING NUMBER EB-1

CORNERSTONE EARTH GROUP

PROJECT NAME MCWD Reservoir and Pump Station
PROJECT NUMBER 142-1-1
PROJECT LOCATION Marina

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UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
1.0 2.0 3.0 4.0 5.0

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-VALUE (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT, PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
161.0	35	▒	**Poorly Graded Sand (SP)** very dense, moist, yellow brown							
		▒		85	☒ SPT-11		3			
	40	▒								
		▒		70	☒ SPT-12		3			
151.0	45		Bottom of Boring at 45.0 feet.							
	50									
	55									
	60									
	65									
	70									
	75									

BORING NUMBER EB-2

PAGE 1 OF 1

PROJECT NAME MCWD Reservoir and Pump Station
PROJECT NUMBER 142-1-1
PROJECT LOCATION Marina
DATE STARTED 6/13/07 **DATE COMPLETED** 6/13/07
GROUND ELEVATION 198 FT +/- **BORING DEPTH** 31.5 ft.
DRILLING CONTRACTOR EGI
LATITUDE **LONGITUDE**
DRILLING METHOD 8" HSA
GROUND WATER LEVELS:
LOGGED BY SEF
∇ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
1.0 2.0 3.0 4.0 5.0

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-VALUE (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT, PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
198.0 197.6	0		2 inches AC over 4 1/2 inches aggregate base **[Fill]**							
			Poorly Graded Sand (SP) [Native] medium dense, moist, yellow brown, fine grained	14	SPT-1		6			
				14	SPT-2		5		4	
	5			13	SPT-3		4			
				22	SPT-4		5			
	10			28	SPT-5		4		7	
187.0			**Poorly Graded Sand (SP)** dense, moist, yellow brown, fine grained	44	SPT-6		4			
	15			56	SPT-7		3			
	20			45	SPT-8		3			
	25			45	SPT-11		4			
	30			48	SPT-12		5			
166.5			Bottom of Boring at 31.5 feet.							
	35									

CORNERSTONE EARTH GROUP

BORING NUMBER EB-3

PAGE 1 OF 3

PROJECT NAME MCWD Reservoir and Pump Station
PROJECT NUMBER 142-1-1
PROJECT LOCATION Marina
DATE STARTED 6/13/07 **DATE COMPLETED** 6/18/07
GROUND ELEVATION 204 FT +/- **BORING DEPTH** 76.5 ft.
DRILLING CONTRACTOR EGI
LATITUDE **LONGITUDE**
DRILLING METHOD 8" HSA
GROUND WATER LEVELS:
LOGGED BY SEF
∇ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-VALUE (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT, PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf ○ HAND PENETROMETER △ TORVANE ● UNCONFINED COMPRESSION ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL 1.0 2.0 3.0 4.0 5.0
204.0	0		2 inches AC over 6 inches aggregate base							
203.5			**Poorly Graded Sand (SP-SM) [Fill]** dense, moist, orange, medium to fine grained, mottled	51	MC-2	95	4		7	
201.5			at 2 1/2 feet becomes medium dense, yellow brown	18	SPT-3		2		3	
	5		**Sand (SP) [Native]** medium dense, slightly moist, yellow brown, fine grained	34	MC-5	87	2		3	
				24	SPT-6		2		3	
	10			38	MC-7		2		2	
190.0			at 14 feet becomes dense							
	15		**Poorly Graded Sand (SP)** dense to very dense, slightly moist, yellow brown, fine grained	57	SPT-9		2			
	20			37	SPT-10		2			
	25		**Poorly Graded Sand (SP)** dense, slightly moist, yellow brown, fine grained	63	SPT-11		2			
175.0	30		**Poorly Graded Sand (SP)** dense, moist, yellow brown, fine grained	39	SPT-12		7			
169.0	35									

Continued Next Page

BORING NUMBER EB-3

PROJECT NAME MCWD Reservoir and Pump Station
PROJECT NUMBER 142-1-1
PROJECT LOCATION Marina

UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
1.0 2.0 3.0 4.0 5.0

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-VALUE (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT, PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
169.0	35			68	☒ SPT-13		4			
167.5			stopped at 36 1/2 feet switched to B-61 on 6/18/07 **Poorly Graded Sand (SP)** dense to very dense, slightly moist, yellow brown, medium grained							
	40			40	☒ SPT-14		3			
	45			64	☒ SPT-15		3			
	50			57	☒ SPT-16		3			
	55			65	☒ SPT-17		2			
	60			42	☒ SPT-18		2			
	65			55	○ NR-19					
	70			50/6"	☒ SPT-20		2			
128.6	75									

Continued Next Page

BORING NUMBER EB-3

PAGE 3 OF 3

CORNERSTONE EARTH GROUP

PROJECT NAME MCWD Reservoir and Pump Station
PROJECT NUMBER 142-1-1
PROJECT LOCATION Marina

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ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-VALUE (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT, PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf ○ HAND PENETROMETER △ TORVANE ● UNCONFINED COMPRESSION ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL 1.0 2.0 3.0 4.0 5.0
128.6 127.5			**Poorly Graded Sand (SP)** dense to very dense, slightly moist, yellow brown, medium grained becomes red brown	43	☒ SPT-21		9			
			Bottom of Boring at 76.5 feet.							
	80									
	85									
	90									
	95									
	100									
	105									
	110									
	115									

BORING NUMBER EB-4

PAGE 1 OF 2

PROJECT NAME MCWD Reservoir and Pump Station
PROJECT NUMBER 142-1-1
PROJECT LOCATION Marina
DATE STARTED 6/18/07 **DATE COMPLETED** 6/18/07
GROUND ELEVATION 202 FT +/- **BORING DEPTH** 40 ft.
DRILLING CONTRACTOR SGF
LATITUDE **LONGITUDE**
DRILLING METHOD 8" HSA
GROUND WATER LEVELS:
LOGGED BY SEF
∇ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-VALUE (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT, PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
202.0	0	2 inches AC over 6 inches aggregate base **[Fill]**							
201.5		**Poorly Graded Sand (SP) [Native]** medium dense, slightly moist, yellow brown	12	SPT-1		7			
			14	SPT-2		6			
	5		18	SPT-3		6			
		becomes dense	31	SPT-4		4			
	10		30	SPT-5		4			
			27	SPT-6		3			
	15								
			31	SPT-7		3			
	20								
			30	SPT-8		5			
	25								
			35	SPT-9		5			
	30								
			42	SPT-10		5			
167.0	35								

UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
1.0 2.0 3.0 4.0 5.0

Continued Next Page

CORNERSTONE
EARTH GROUP

BORING NUMBER EB-4

PAGE 2 OF 2

PROJECT NAME MCWD Reservoir and Pump Station

PROJECT NUMBER 142-1-1

PROJECT LOCATION Marina

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
1.0 2.0 3.0 4.0 5.0

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-VALUE (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT, PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
167.0	35		**Poorly Graded Sand (SP)** moist, dense, yellow brown, medium grained							
				38	☒ SPT-11		5			
162.0	40		Bottom of Boring at 40.0 feet.							
	45									
	50									
	55									
	60									
	65									
	70									
	75									

BORING NUMBER EB-5

CORNERSTONE EARTH GROUP

PROJECT NAME MCWD Reservoir and Pump Station
PROJECT NUMBER 142-1-1
PROJECT LOCATION Marina
DATE STARTED 6/13/07 **DATE COMPLETED** 6/13/07
GROUND ELEVATION 184 FT +/- **BORING DEPTH** 21.5 ft.
DRILLING CONTRACTOR EGI
LATITUDE **LONGITUDE**
DRILLING METHOD 8" HSA
GROUND WATER LEVELS:
LOGGED BY SEF
∇ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION
184.0	0	2 inches AC over 6 inches aggregate base
183.5		**[Fill]** **Poorly Graded Sand (SP) [Native]** medium dense, slighty moist, yellow brown, fine grained sand
172.0		**Poorly Graded Sand (SP)** dense, slighty moist, yellow brown
162.5		Bottom of Boring at 21.5 feet.

N-VALUE (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT, PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
17	☒ SPT-1		3		5	
14	☒ SPT-2		3		3	
15	☒ SPT-3		3			
19	☒ SPT-4		2			
21	☒ SPT-5		2			
30	☒ SPT-6		3			
33	☒ SPT-7		3			
38	☒ SPT-8		3			

UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
1.0 2.0 3.0 4.0 5.0

APPENDIX B – LABORATORY TEST PROGRAM

The laboratory testing program was performed to evaluate the physical and mechanical properties of the soils retrieved from the site to aid in verifying soil classification.

Moisture Content

The natural water content was determined (ASTM D2216) on fifty seven samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

Dry Densities

In place dry density determinations (ASTM D2937) were performed on two samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depth.

Washed Sieve Analyses

The percent soil fraction passing the No. 200 sieve (ASTM D1140) was determined on 15 samples of the subsurface soils to aid in the classification of these soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

APPENDIX C – SITE CORROSIVITY EVALUATION

JDH Corrosion Consultants
Incorporated

July 13, 2007

Cornerstone Earth Group
1259 Oakmead Parkway
Sunnyvale, California 94085

Attention: **Mr. Scott E. Fitinghoff, P.E., G.E.**
Principal Engineer

Subject: **Site Corrosivity Evaluation**
MCWD Pump Station
Project No. 142-1-1

Dear Scott,

In accordance with your request, we have reviewed the laboratory soils data and the in-situ soil resistivity data for the above referenced project site. Our evaluation of these results and our corresponding recommendations for corrosion control for the above referenced project foundations and buried site utilities are presented herein for your consideration.

SOIL TESTING & ANALYSIS

Soil Chemical Analysis

Four (4) soil samples from the project site were chemically analyzed for corrosivity by ***Cooper Testing Laboratories***. Each sample was analyzed for chloride and sulfate concentration, pH, resistivity at 100% saturation and moisture percentage. The test results are presented in Cooper Testing Laboratories *Corrosivity Test Summary* dated 7/5/07. The results of the chemical analysis were as follows:

Soil Laboratory Analysis

Chemical Analysis	Range of Results	Corrosion Classification*
Chlorides	<2 mg/kg	Non-corrosive
Sulfates	13 - 20 mg/kg	Non-corrosive **
pH	6.6 – 7.8	Non- corrosive**
Moisture (%)	3.1 – 6.1	Not-applicable
Resistivity at 100% Saturation	23,700 – 40,300 ohm-cm	Non-corrosive

* With respect to bare steel or ductile iron
** With respect to mortar coated steel

In-Situ Soil Resistivities

One in-situ soil resistivity measurement was performed at the project site at depths of 2.5’, 5’, 10’, 15’and 20’ and 100% of the results indicate essentially non-corrosive conditions for all soil layers to a depth of 20’ below grade. These results are consistent with the boring logs also provided for our review.

DISCUSSION

Reinforced Concrete Foundations

Due to the low levels of water-soluble sulfates in these soils, special sulfate resistant cement is not required for concrete structures placed into these soils. Sulfate resistant concrete as recommended in the Uniform Building Code (UBC) for soils containing less than 0.10% water-soluble sulfate in soil by weight shall be used.

Underground Metallic Pipelines

The soils at the project site are considered to be “non-corrosive” to ductile/cast iron, steel and dielectric coated steel based on the saturated resistivity measurements, in-situ soil resistivity measurement, pH levels and water soluble sulfate levels. Therefore, no special requirements for corrosion control are required for buried metallic utilities at this site. However, all underground pipelines should be electrically isolated from above grade structures, reinforced concrete structures and copper lines in order to avoid potential galvanic corrosion problems.

LIMITATIONS

> *The conclusions and recommendations contained in this report are based on the information and assumptions referenced herein. All services provided herein were performed by persons who are experienced and skilled in providing these types of services and in accordance with the standards of workmanship in this profession. No other warrantees or guarantees, expressed or implied, are provided.*

We thank you form the opportunity to be of service to ***Cornerstone Earth Group*** on this project and trust that you find the enclosed information satisfactory. If you have any questions or if we can be of any additional assistance, please feel free to contact us at (925) 927-6630.

Respectfully submitted,

J. Darby Howard, Jr.

REGISTERED PROFESSIONAL ENGINEER
J. DARBY HOWARD, JR.
No. CR 1055
Exp. June 30, 2009
CORROSION
STATE OF CALIFORNIA

J. Darby Howard, Jr., P.E.
JDH Corrosion Consultants, Inc.
Principal
Cc: File 27105

Corrosivity Test Summary

CTL # 640-021 **Date:** 7/5/2007 **Tested By:** PJ **Checked:** PJ

Client: Cornerstone **Project:** MCWD Pump Station **Proj. No:** 142-1-1

Remarks:

Sample Location or ID			Resistivity @ 15.5 ^{o}C (Ohm-cm)			Chloride	Sulfate-(water soluble)		pH	ORP	Sulfide	Moisture	Soil Visual Description
Boring	Sample, No.	Depth, ft.	As Rec.	Minimum	Saturated	mg/kg	mg/kg	%		(Redox)	Qualitative	%	
						Dry Wt.	Dry Wt.	Dry Wt.		mv	by Lead	At Test	
			ASTM G57	Cal 643	ASTM G57	Cal 422-mod.	Cal 417-mod.	Cal 417-mod.	ASTM G51	SM 2580B	Acetate Paper	ASTM D2216	
EB- 2	1	1-2.5	-	-	26,932	<2	16	0.0016	7.6	-	-	6.1	Yellowish Brown SAND
EB- 2	2	3-4.5	-	-	40,342	<2	13	0.0013	6.6	-	-	5.9	Yellowish Brown SAND
EB- 5	1	1-2.5	-	-	23,738	<2	20	0.0020	7.8	-	-	3.6	Olive Brown SAND, trace Gravel
EB- 5	2	3-4.5	-	-	29,456	<2	16	0.0016	7.8	-	-	3.1	Olive Brown SAND, trace Gravel