GEOTECHNICAL INVESTIGATION
TREATMENT PLANT UPGRADE PROJECT
SAUSALITO-MARIN CITY SANITARY DISTRICT
SAUSALITO, CALIFORNIA

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Project 1028.04

Prepared For:
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CERTIFICATION

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MILLER PACIFIC ENGINEERING GROUP
(a California corporation)

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I. INTRODUCTION

This report presents the results of our Geotechnical and Geologic Investigation for the planned Treatment Plant Upgrades to the Sausalito-Marin City Sanitary Districts treatment plant in Sausalito, California. A project site location map is presented on Figure 1. This report is intended for the exclusive use of RMC Water and Environment and the design team for this project and site. No other use is authorized without the express written consent of Miller Pacific Engineering Group.

The scope of our Phase 1 services is described in our proposal letter dated February 8, 2013 and includes the following geotechnical services:

- Review of available published geologic mapping and geotechnical background data;
- Subsurface exploration with 7 soil borings;
- Laboratory testing of select samples;
- Evaluation of geologic hazards and respective mitigation measures;
- Geotechnical evaluation and analyses;
- Development of seismic design criteria and recommendations in accordance with the 2010 California Building Code;
- Development of geotechnical design criteria and recommendations for site preparation and grading, site drainage, foundations, retaining walls, new cut and fill slopes, temporary shoring, underground utilities, pavements, and other geotechnical items; and
- Preparation of this report.

Issuance of this report completes our Phase 1 services. Supplemental services are anticipated to include geo-civil design of new retaining walls and cut slopes, plan review and consultation, and observation and testing during construction.

II. PROJECT DESCRIPTION

The project consists of constructing a new primary sedimentation tank and a new headworks structure with screening and grit removal facilities, new material handling areas, and a new truck turntable. The lower portion of the existing access road will be realigned to the north to accommodate the new sedimentation tank. Ancillary improvements will include realignment of the existing influent sewer pipe, secondary and tertiary improvements within the existing treatment plant area, minor remodeling of the administration building, and a minimum of 0.6 million gallons new equalization storage.

Extensive grading and relatively tall site retaining walls (up to approximately 30-feet high) will be required to facilitate access road realignment and construction of the new sedimentation tank, headworks structure and other improvements at the steeply-sloping, heavily developed waterfront site. A site plan indicating the extent of the proposed improvements is shown on Figure 2.

The design team for the SMCSD treatment plant improvements includes the Sausalito-Marin City Sanitary District (Owner), RMC Water and Environment (Project Manager & Civil Engineer), Burks Toma (Architect), Royston, Hanamoto, Alley & Abey (RHAA) (Landscape Architect), and TJC and Associates, Inc. (Structural & Electrical Engineers).

III. SITE CONDITIONS

A. Regional Geology

The project site is located in the Coast Ranges geomorphic province of California, which is typified by generally northwest-trending ridges and intervening valleys formed as a result of movement along a group of northwest-trending fault systems, principally the San Andreas Fault. Bedrock geology within Marin County is dominated by sedimentary, igneous, and metamorphic rocks of the Jurassic-Cretaceous age Franciscan Complex. Sandstone and shale comprise the majority of Franciscan rock types, while less common rocks include chert, serpentinite, basalt, greenstone, and exotic low- to high-grade metamorphic rocks, including phyllite, schist, and eclogite.

The project site is located on the western shore of the San Francisco Bay, just north of Fort Baker and the Golden Gate Bridge. Regional geologic mapping indicates the site lies within a colluvial swale, flanked to the north and south by Franciscan chert and greenstone bedrock. Colluvial deposits typically are composed of unsorted soil and rock debris, transported downslope by gravity and natural weathering processes. Within the Marin Headlands and Sausalito areas, chert is typically red-brown with thin gray shale interbeds, closely fractured to crushed, hard, and strong.

Greenstone is metamorphosed basaltic rock and can range from very hard and strong where fresh to friable where deeply weathered. A regional geologic map is presented on Figure 3.

B. Seismicity

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

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

1. Active Faults in the Region - Such earthquakes could occur on any of several active faults within the region. An "active" fault is one that shows displacement within the last 11,000 years (i.e. Holocene) and has a reported average slip rate greater than 0.1 mm per year. The California Division of Mines and Geology (1998) has mapped various active and inactive faults in the region. These faults, defined as either California Building Code Source Type "A" or "B," are shown in relation to the project site on the attached Active Fault Map, Figure 4.

2. Historic Fault Activity - Numerous earthquakes have occurred in the region within historic times. The results of our computer database search indicate that 36 earthquakes (Richter Magnitude 5.0 or larger) have occurred within 100 kilometers (62 miles) of the site area between 1769 and 2013. The five most significant historic earthquakes to affect the project site are summarized in Table A.

TABLE A
SIGNIFICANT EARTHQUAKE ACTIVITY
SMCSD – Treatment Plant Upgrades
Sausalito, California – TAKE FROM 1206.08

Historic Richter Magnitude[1]	Year[1]	Distance[1]	Estimated Peak Acceleration[2,3]
8.2	1906	16 km	0.26 g
6.8	1836	25 km	0.11 g
7.0	1838	28 km	0.11 g
6.8	1868	37 km	0.08 g
5.3	1870	17 km	0.06 g

(1) USGS (2011)
(2) Abrahamson & Silva (2008), Boore & Atkinson (2008), Campbell & Bozorgnia (2008), Chiou & Youngs (2008), Idriss (2008)
(3) Values determined using Vs^{30} = 762 m/s for Site Class "B" (Rock) in accordance with 2010 CBC.

3. Probability of Future Earthquakes – The site will likely experience moderate to strong ground shaking from future earthquakes, originating on active faults in the San Francisco Bay region. The historical records do not directly indicate either the maximum credible earthquake or the probability of such a future event. To evaluate earthquake probabilities in California, the USGS has assembled a group of researchers into the "Working Group on California Earthquake Probabilities" (2003, 2008a) to estimate the probabilities of earthquakes on active faults. In these studies, potential sources were analyzed considering fault geometry, geologic slip rates, geodetic strain rates, historic activity, and micro-seismicity, to arrive at estimates of earthquakes of various magnitudes on a variety of faults in California.

   The 2003 study specifically analyzed fault sources and earthquake probabilities for the seven major regional fault systems in the Bay Area region of northern California. In addition, the probabilities of "background earthquakes" were included. These background earthquakes are not associated with the identified fault systems and may occur on lesser faults (i.e., West Napa) or previously unknown faults (i.e., the 1989 Loma Prieta and 2000 Mt. Veeder - Napa earthquakes). The 2008 study applied many of the analyses used in the 2003 study to the entire state of California and updated some of the analytical methods and models.

   When the probabilities on all seven fault systems and the background earthquakes are combined mathematically, the mean probability of a M>6.7 earthquake in northern

California is about 93%. Additionally, probabilities of a M>6.7 event on the nearest mapped active faults by 2038 (Hayward-Rodgers Creek Fault System and San Andreas Fault) are 31% and 21%, respectively. Additional studies by the USGS regarding the probability of large earthquakes in the Bay Area are ongoing. These current evaluations include data from additional active faults and updated geological data.

C. Surface Conditions

The project site lies at the east edge of the Marin Headlands, along the western shore of San Francisco Bay between Cavallo Point to the south and the mouth of Richardson Bay to the north. The site is bounded to the east by San Francisco Bay, to the west by East Road, and to the north and south by steep, undeveloped, east-facing hillsides. Slopes at the site are generally east-facing and inclined between about 2:1 (horizontal:vertical) in the lower reaches of the site, flattening to near 3:1 in the upslope portions closer to East Road. In the extreme northeastern portion of the site, near-vertical bluffs rise between 10 and 20 feet from the San Francisco Bay shoreline to the moderately steep slopes above. The site lies roughly at the confluence of two moderately-developed colluvial swales emanating from the ridgeline upslope to the west. The proposed development area is sited in the northern portion of the site.

The site is currently developed as a wastewater treatment facility, and existing improvements include a variety of buildings, tanks, and underground utilities across the property as shown on Figure 2. Utilities entering and exiting the facility typically traverse the slope above the access road at the north end of the property and are marked with a series of low wooden bulkheads staked to the ground. An existing storm drain outfall is located on the slope above the planned primary sedimentation tank with an eroded gully that flow down towards the planned improvements.

Bluffs along the shoreline at the north end of the site, below the base of the main access road, are inclined near-vertical and expose hard, strong pillow basalt and greenstone (metamorphosed basaltic rocks) near the waterline. Thinly-bedded, tightly-folded, rust-red "ribbon" chert is exposed higher in the bluffs, and commonly occurs with very thin gray shale interbeds. Chert varies from hard and strong where fresh to friable where deeply weathered. Aside from the bluffs and low cut slopes near the shoreline, only scattered outcrops of highly weathered chert exist at the site, mainly in older, eroded cuts around existing structures. Surface soils across the remainder of the site generally consist of medium dense sands and medium stiff clays. We understand through discussions with facility personnel that, in the vicinity of the proposed development area, these soils may have been derived from excavations during construction of the nearby East Road tunnel and "loose-dumped" over the slope at the time of tunnel construction. Therefore, surface soils at the site may consist of a combination of colluvium, residual soil, and "undocumented" fill.

No evidence of "global" slope instability or landsliding was observed at the site, nor are any landslides shown on available regional geologic maps. However, we did note evidence of soil "creep" on some of the steeper slopes around the site. Additionally, many of the wooden bulkheads which mark incoming and outgoing utilities are undermined and/or failed as a result of generally poor surface drainage and resulting erosion of loose surface soils.

D. Field Exploration and Laboratory Testing

Subsurface exploration consisted of seven soil borings at the approximate locations shown on Figure 2, performed on June 11-12, 2013. Borings were excavated to depths between 10.5 and 22.5-feet using portable, hydraulic-powered drilling equipment equipped with 4-inch solid flight augers. Borings 1 and 7 (as shown on Figure 2) were subsequently deepened on June 19, 2013 using a portable hydraulic drill rig equipped with a 2.5-inch air hammer bit. Materials encountered were logged by our Field Geologist and select samples retained for laboratory testing. A brief explanation of the terms and methods used during field exploration activities is presented on Figures A-1 and A-2, Soil and Rock Classification Charts. Boring logs are shown on Figures A-3 through A-12.

Laboratory tests included determination of moisture content, in-situ density, unconfined compressive strength, and percentage of particles passing the No. 200 (75-µm) sieve. Laboratory test results are presented on the boring logs. The field exploration and laboratory testing program is discussed in further detail in Appendix A.

E. Subsurface Conditions

The results of our subsurface exploration generally confirm the regionally- and locally-mapped geology as referenced and described above. The project site is underlain chiefly by relatively competent chert and greenstone bedrock beneath a thin layer of colluvial and residual soils. Locally thick fill soils were found to exist near the southwest corner of the planned headworks structure. In general, chert bedrock was found to overlie the greenstone and be more highly fractured, while greenstone was observed to generally be very hard and strong. Local geologic conditions are shown on Figure 2 and geologic cross-sections along B-B' and D-D' are presented on Figure 5. Subsurface conditions indicate that foundation excavations for the new primary sedimentation tank and headworks building will be founded in chert and/or greenstone bedrock. Additionally, it should be noted that significant quantities of hard greenstone rock will need to be excavated to achieve the planned foundation grades. Additional discussion of site grading and excavation is presented in Section V of this report.

Boring 1, located near the southwest corner to the planned headworks structure, encountered approximately 11-feet of fill and colluvial soils composed of medium dense to dense clayey and silty sand with lesser gravel. Residual soils composed of stiff clay with sand and gravel were encountered between 11- and 18-feet. Hard, strong, highly weathered chert bedrock was encountered between 18- and 29-feet, and very hard and strong greenstone bedrock was encountered from 29- to 33-feet, where Boring 1 was terminated.

Boring 2, located near the southeastern corner to the planned headworks building, encountered about 3-feet of medium dense clayey sand colluvium underlain by stiff clay with sand and gravel to a depth of 9-feet. At 9-feet, hard, strong, highly weathered chert bedrock was encountered, and Boring 2 was terminated at a depth of 15.25-feet.

Boring 3 was located near the center of the planned headworks building, and encountered about 5-feet of medium dense to dense retaining wall backfill composed of medium dense to dense clayey sand with lesser gravel. Beneath the fill, hard, strong chert bedrock was encountered to a depth of 10.5-feet, where Boring 3 was terminated.

Borings 4, 5 and 6, located generally in the vicinity of the planned upslope access road retaining wall and primary sedimentation tank, encountered between 1- and 5-feet of stiff to very stiff sandy clay residual soils underlain by hard, moderately strong to strong chert bedrock. Borings 4 through 6 were terminated at depths between 12.0 and 12.5-feet.

Boring 7 was located near the north end of the planned access road retaining wall and encountered less than 3-feet of medium dense clayey sand residual soil underlain by hard, moderately strong chert bedrock to a depth of about 10-feet. At 10-feet, hard, very strong greenstone was encountered, and Boring 7 was terminated at a depth of 43.5-feet.

Significant groundwater was encountered in Boring 7 at a depth of about 38-feet (roughly +14 MSL), and was measured at a depth of 13-feet following completion of the boring. It is likely that this groundwater is confined to a fracture or dormant fault plane or geologic contact within the bedrock. Groundwater was not encountered in any of the other borings during our exploration. Because Boring 7 was not left open for an extended period of time, a stabilized depth to groundwater may not have been recorded. Based on our experience with similar sites in the area, groundwater may be expected within a few feet of the upper bedrock contact and could be encountered along fractures, faults, and contacts within the bedrock. In general, shallower groundwater should be expected during the winter months and following periods of heavy rain.

III. GEOLOGIC HAZARDS

A. General

This section identifies potential geologic hazards at the property site, their significant adverse impacts, and recommended mitigation measures. We judge the significant geologic hazards at the project site are seismic ground shaking, erosion, and slope instability/landsliding. More detailed evaluation of these and other commonly-considered geologic hazards are presented below.

B. Fault Surface Rupture

Under the Alquist-Priolo Special Studies Zone Act, the California Division of Mines and Geology (CDMG 2000) produced 1:2,000 scale maps showing all known active faults in California. The nearest known active faults, the San Andreas and San Gregorio faults, lie approximately 7 miles west and 9 miles southwest of the site, respectively, and the site is not located within an Alquist-Priolo Special Studies Zone. The potential for fault surface rupture at the site is low.

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

C. Seismic Shaking

The site will experience seismic ground shaking similar to other areas in the seismically active San Francisco Bay Area. Earthquakes along several active faults in the region, as shown on Figure 4, could cause moderate to strong ground shaking at the site. The intensity of earthquake motion will depend on the characteristics of the generating fault, distance to the fault and rupture zone, earthquake magnitude, earthquake duration, and site-specific geologic conditions.

Deterministic methods use empirical attenuation relations to provide approximate estimates of median peak ground accelerations. A summary of the active faults that could most significantly affect the project site, their maximum credible magnitude, closest distance to the site, and probable peak ground accelerations are summarized in Table B.

TABLE B
ESTIMATED PEAK GROUND ACCELERATION
SMCSD – Treatment Plant Upgrades
Sausalito, California

Fault	Moment Magnitude For Characteristic Earthquake[1]	Closest Estimated Distance (kilometers)[1]	Median Peak Ground Acceleration (g)[1,2]
San Andreas	8.0	11	0.32
San Gregorio	7.4	15	0.22
Hayward	7.3	18	0.18
Rodgers Creek	7.3	36	0.10
Calaveras	6.9	38	0.08

(1) California Department of Transportation (Caltrans), Caltrans ARS Online v2.2.06, http://dap3.dot.ca.gov/ARS_Online/, accessed June 15, 2013.
(2) Values determined using Vs^{30} = 760 m/s for Site Class "B" (Rock) in accordance with 2010 CBC/2005 ASCE-7.

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

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

Evaluation: *Less than significant with mitigation.*

Mitigation: *New improvements should be designed in accordance with the latest edition (2010) of the California Building Code. Recommended seismic design criteria are presented in Section V of this report.*

D. Liquefaction Potential

Liquefaction refers to the sudden, temporary loss of soil strength during strong ground shaking. This phenomenon can occur where there are saturated, loose, granular (sandy) deposits subjected to seismic shaking. Liquefaction-related phenomena include settlement, flow failure, and lateral spreading. Loose granular deposits were not observed during our exploration and foundations for new improvements will likely bear directly on bedrock. Therefore, the potential for damage due to liquefaction is low.

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

E. Seismically-Induced Ground Settlement

Ground shaking can induce settlement of loose, granular soils above the water table. Loose granular soils were not encountered in our exploratory borings. Therefore, the risk of seismically-induced ground settlement at the site is low.

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

F. Lurching and Ground Cracking

Lurching and associated ground cracking can occur during strong ground shaking. The ground cracking generally occurs along the tops of slopes where stiff soils are underlain by soft or loose deposits or along steep channel banks. These conditions generally do not exist at the site, and the risk of damage due to lurching or ground cracking is low.

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

G. Erosion

Sandy soils on moderately steep slopes or clayey soils on steep slopes are susceptible to erosion when exposed to concentrated surface water flow. The potential for erosion is increased when established vegetation is disturbed or removed during normal construction activity. The project site is underlain by a thin to moderately thick cover of loose to medium dense fill and colluvial soils on moderate slopes, and many slopes at the site exhibit evidence of active erosion. Therefore, the risk of erosion at the site is moderate.

Evaluation: *Less than significant with mitigation.*
Mitigation: *The project Civil Engineer should design site drainage to collect and convey surface water to an appropriate discharge location, ideally into an established storm drain system. Re-establishing vegetation on disturbed areas will minimize erosion. Erosion control measures during and after construction should conform to the most recent version of the Erosion and Sediment Control Field Manual (California Regional Water Quality Control Board, 2002).*

H. Seiche and Tsunami

Seiche and tsunamis are short duration earthquake-generated water waves in enclosed bodies of water and the open ocean, respectively. The extent and severity of a seiche would be dependent upon ground motions and fault offset from nearby active faults. Tsunami inundation mapping by the California Emergency Management Agency (CalEMA) indicates that the site lies on the margin of an inundation zone along the San Francisco Bay shoreline as shown on Figure 6. Because the site is located at low elevation and immediately adjacent to San Francisco Bay, the risk of damage due to seiche or tsunami is considered moderate.

Evaluation: *Less than significant with mitigation.*

Mitigation: *The project team should consider the potential for inundation by seiche or tsunami and design new improvements appropriately. In particular, tanks or other structures designed to impound wastewater or other hazardous materials should be designed to withstand potential short-term hydrostatic pressures on exterior walls due to inundation. These facilities should also be planned at sufficient elevation to reduce the likelihood of overtopping by seiche or tsunami that could result in dispersion of hazardous materials into the adjacent bay.*

I. Flooding

The adverse impact from flooding is water damage to structures and furnishings. The majority of the project site is located on moderately- to steeply-sloping terrain and is not located within a FEMA flood zone. The shoreline area of the site will be subject to periodic inundation as a result of tidal and flooding activity, and the proposed improvements are located at low elevation and immediately adjacent to San Francisco Bay. Therefore, the likelihood of large-scale flooding at the site is moderate.

Evaluation: *Less than significant with mitigation.*

Mitigation: *The project team should consider the potential for inundation by floodwater and design new improvements appropriately as discussed above. Careful attention should be given to design of finished grades to avoid ponding of water around structures and small-scale flooding at the site. Site drainage recommendations are presented in Section V of this report.*

J. Settlement

Consolidation of soft and/or compressible soils can cause settlement of structures and other surface improvements. The project site is generally underlain by a thin to moderately-thick layer of medium dense/stiff to very stiff sandy and clayey residual soils over hard bedrock.

Preliminary project plans indicate the new structures will be constructed on level pads created entirely by excavating into the hillside. Therefore, the risk of damage due to settlement is generally low provided that foundations bear directly on bedrock.

Evaluation: *Less than significant.*

Mitigation: *Foundations for new structures should bear on firm bedrock beneath any colluvial and residual soils. If new foundations are planned to span cut-fill transitions, bearing support should be derived from deep foundations extending through the fills and/or native soils into firm bedrock materials. Additional discussion of cut-fill construction and foundation design is presented in Section V of this report.*

K. Expansive Soil

Expansive soil occurs when clay particles interact with water causing volume changes in the clay soil. The clay soil swells when saturated and contracts when dried. This phenomenon generally decreases in magnitude with increasing confinement pressure at depth. These volume changes may damage lightly loaded foundations, retaining walls and shallow improvements. Expansive soils also cause soil creep on sloping ground.

The greenstone and chert bedrock which underlies the site commonly weathers to expansive clay minerals. Clayey soils encountered during our exploration were observed to be generally of low to moderate plasticity, and we noted evidence of localized soil creep during our site reconnaissance. Therefore, we judge the risk of damage due to expansive soils at the site is moderate.

Evaluation: *Less than significant with mitigation.*

Mitigation: *Foundations for new structures should bear on firm bedrock beneath any potentially expansive soils. If significant expansive soils are encountered during construction where other lightly-loaded structures, such as exterior flatwork, are planned, we should be consulted to provide supplemental recommendations.*

L. Slope Instability and Landsliding

Available geologic mapping does not indicate the presence of any landslides in the immediate vicinity of the project site, nor was any evidence of large-scale or “global” instability or landsliding observed during our site reconnaissance and exploration. We did note evidence of active erosion and “creep” of loose surficial soils on steeper slopes throughout the site. Therefore, while the risk of large-scale landsliding is generally low, the risk of smaller-scale sliding or slow-moving slope creep is moderate.

Evaluation: *Less than significant with mitigation.*

Mitigation: *New improvements should be founded on firm materials beneath any creep-prone deposits, and new retaining walls should be designed to withstand increased active pressures due to slope creep. Recommendations for foundation and retaining walls design are presented in Section V of this report.*

V. CONCLUSIONS AND RECOMMENDATIONS

A. Conclusions

Based on the results of our investigation, research and evaluation, it is our professional opinion that development of the project site is feasible from a geotechnical standpoint. The primary geotechnical considerations for the project include mitigation of strong seismic ground shaking, providing uniform foundation support for new structures, and cost-effective design of new retaining structures and other improvements. Recommendations and design criteria for these and other considerations are presented in the following sections.

B. Site Preparation and Grading

Extensive grading, consisting chiefly of excavation, will be required to construct level building pads for the new primary sedimentation tank and headworks structure and for realignment of the lower portion of the access road. Based on proposed foundation and roadway grades, cuts up to about 30-feet will be required. Site grading should be performed in accordance with the following recommendations:

1. Site Preparation – Clear all trees, brush, roots, over-sized debris, and organic material from areas to be graded. Trees that will be removed (in structural areas) must also include removal of stumps and roots larger than four inches in diameter. Existing foundations should be removed or cut-off 3 feet below planned subgrade elevations. Excavated areas (i.e., excavations for foundation removal) below planned subgrade should be restored with properly moisture conditioned and compacted fill as described in the following sections. Underground utilities may be abandoned in place provided the utility is completely filled with neat cement grout. Any loose soil or rock at subgrade will need to be excavated to expose firm natural soils or bedrock. Debris, rocks larger than six inches and vegetation are not suitable for structural fill and should be removed from the site. Alternatively, vegetation strippings may be used in landscape areas.

   Where fills or other structural improvements are planned on level ground, the subgrade surface should be scarified to a depth of about eight inches, moisture conditioned to above the optimum moisture content, and compacted to a minimum of 90 percent relative compaction (ASTM D-1557). Subgrade areas exposing bedrock need not be scarified and recompacted. Relative compaction, maximum dry density, and optimum moisture content of fill materials should be determined in accordance with ASTM Test Method D 1557, "Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using a 10-lb. Rammer and 18-in. Drop." If soft, wet or otherwise unsuitable materials are encountered at the subgrade elevation during construction, we will provide supplemental recommendations/field directives to address the specific condition.

2. Excavations – Site excavations for new foundations, underground utilities, and other improvements will generally encounter colluvial and/or residual soil deposits of variable thickness over relatively shallow chert and greenstone bedrock. Based on observations of drilling rates with both auger and pneumatic equipment, we judge that the majority of excavations within the chert bedrock can be reasonably accomplished with “traditional” grading equipment such as moderate to large dozers and excavators. In contrast, the underlying greenstone, which will likely be encountered near the base of the planned foundation excavations for the new primary sedimentation tank and headworks structure, as shown on Figure 5, was observed to be typically hard and strong to very strong. Drilling rates with pneumatic equipment, which typically provides superior performance and efficiency in hard rock, were reduced by about 50% in those borings that encountered greenstone bedrock. Therefore, this material is likely to require the use of specialized excavation techniques, such as “hoe-ramming”, low-impact blasting or rock splitting using expansive chemical grouts to excavate. Therefore, we recommend including a line item for hard rock excavation in the project bid documents, where “hard rock” is defined as material which cannot be excavated at a reasonable production rate with equipment typically used for excavation work in similar terrain, such as a Caterpillar 330 or equivalent excavator equipped with a bucket, thumb, and “rock” teeth. If hard rock is encountered during construction which prohibits excavation to the required depths, we should be consulted to observe conditions and revise our recommendations and/or design criteria, as appropriate.

3. Fill Materials and Compaction – Fill should be placed on a prepared subgrade as described above. The fill material shall be non-expansive materials free of organic matter, have a Liquid Limit of less than 40, a Plasticity Index of less than 20, minimum R-value of 20, and conform to the gradation limits shown below in Table C. Shallow excavations in fill, colluvial, or residual soils and weathered chert bedrock will likely yield clayey to gravelly mixtures that are suitable for re-use as fill, while deeper excavations a, particularly where greenstone or relatively unweathered chert is encountered, may yield cobbles or boulders that require substantial processing to meet the gradation requirements shown in Table C, depending on the equipment and methods used.

   Fill materials should be placed in loose horizontal lifts no greater than eight inches thick. Structural fills less than five feet thick should be moisture conditioned above the optimum moisture content and uniformly compacted to a minimum of 90 percent relative compaction. Structural fills in excess of five feet should be moisture conditioned above the optimum moisture content and uniformly compacted to a minimum of 92 percent relative compaction to reduce the potential for significant settlements. In non-structural (landscape) areas fill compaction may be reduced to at least 85 percent. The upper twelve inches of pavement subgrade (i.e., access roads and driveways) should be compacted to a minimum of 95 percent relative compaction to provide a smooth, uniform, and unyielding surface when proof-rolled with heavy rubber-tire construction equipment.

TABLE C
IMPORTED FILL GRADATION LIMITS
SMCSD – Treatment Plant Upgrades
Sausalito, California

Particle Size	Percent Finer by Dry Weight
4 inch	100
No. 4 sieve	20 - 100
No. 200 sieve	0 - 50

4. Fill Slope Construction – If new fills are planned on sloping surfaces steeper than 8:1 (horizontal:vertical), they should be founded on keyways and benches excavated into stable bedrock. Keyway depths will be determined during construction, but we anticipate keyways would extend a minimum of 3-feet into firm bedrock. Subsurface drainage should be provided for all keyway excavations and intermediate benches prior to fill placement or as determined in the field by the Geotechnical Engineer. A typical hillside fill construction detail is presented on Figure 7.

   Fill slopes should be inclined no steeper than 2:1. Fill slopes steeper than 2:1 will require internal reinforcement and need to be specifically designed. If fill slopes steeper than 2:1 are planned, we should be consulted to provide additional recommendations and design criteria.

5. Permanent and Temporary Cut Slopes – Temporary (steeper) cut slopes will be required during construction until retaining walls are constructed and backfilled. For planning purposes, these cut slopes in soils and soft rock should be inclined at 1:1 (horizontal:vertical), based on an OSHA Type "B" soil profile. Temporary cut slopes in hard rock may be inclined at 0.5:1, and even steeper slopes may be possible where favorable geologic conditions are encountered. Geologic inspection during excavation

will be required to verify that the above recommendations are appropriate for the conditions encountered.

Performance of temporary cut slopes will be heavily dependent on the amount of time the cut is unsupported, seepage and surface runoff over the face, bedding and fracture planes of rock and soil materials, and other factors. The steeper (temporary) cut slopes may exhibit some sloughing, especially during wet weather conditions, and cleanup of soil and rock debris at the base of slopes may be required. We recommend the project grading contractor be responsible for the performance of temporary cut slopes, and we should be present intermittently during construction to verify that the above recommendations remain appropriate for actual conditions encountered.

Top down construction with soil nail walls would allow for vertical excavation and provides lateral support as the excavation deepens. Temporary vertical cuts for the wall should not exceed 5 to 6 feet without lateral support from soil nails and shotcrete facing.

Permanent cut slopes excavated into soil/soft rock and competent bedrock should be inclined no steeper than 2:1 and 1:1, respectively. Concrete lined v-ditches should be provided 5-feet back from the top of the cut slope. Additionally, the top of the cut slope should be trimmed and rounded to reduce the potential of minor sloughing at the grade break.

Properly-designed and -constructed cut slopes should perform as well as adjacent slopes. However, rock conditions in this geologic area are variable, not totally predictable and may therefore need modification during construction. Periodic slope maintenance after construction, such as the cleanup of rock debris, may be required.

C. Seismic Design

Minimum mitigation of ground shaking includes seismic design of the structures in conformance with the provisions of the most recent version (2010) of the California Building Code. The magnitude and character of these ground motions will depend on the particular earthquake and the site response characteristics. Based on the interpreted subsurface conditions and close proximity of the San Andreas and San Gregorio Faults, we recommend the CBC coefficients and site values shown in Table D below to calculate the design base shear of the new construction. To determine site seismic coefficients, we used the USGS Earthquake Ground Motion Parameters Java application, Version 5.1.0, using the latitude and longitude shown on Figure 4.

TABLE D
2010 CBC SEISMIC DESIGN FACTORS
SMCSD – Treatment Plant Upgrades
Sausalito, California

Factor Name	Coefficient	CBC Table/ Figure	Site Specific Value[1]
Site Class[2]	$S_{A,B,C,D,E,\text{ or }F}$	1613.5.2	S_B
Spectral Acc. (short)	S_s	1613.5(3)	1.50 g
Spectral Acc. (1-sec)	S_1	1613.5(4)	0.68 g
Site Coefficient	F_a	1613.5.3(1)	1.0
Site Coefficient	F_v	1613.5.3(2)	1.0

1) Values determined in accordance with the 2005 ASCE-7 standard.
2) Soil Profile Type S_B Description: Rock, Shear Wave Velocity between 2,500 and 5,000 feet per second, Standard Penetration blow counts greater than 50, and undrained shear strength greater than 2,000 psf.

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

D. Foundation Design Criteria

Preliminary project plans indicate that new foundations for the planned improvements will be constructed entirely in “cut” areas, and we judge that shallow foundations bearing directly on competent bedrock will provide adequate support for these structures. If structures will span cut/fill transitions, deep foundations such as drilled, cast-in-place concrete piers with interconnected grade beams should be utilized in fill areas to reduce the risk of damage due to total and differential settlements. Deep foundations may also be required where the planned improvements would otherwise impose damaging surcharge loads on existing facilities if shallow foundations were utilized. All deep foundations should extend through any soil materials to derive uniform bearing support for new structures from the underlying bedrock. Design criteria for both shallow and deep foundation systems are presented in Table E.

TABLE E
FOUNDATION DESIGN CRITERIA
SMCSD – Treatment Plant Upgrades
Sausalito, California

Shallow Footings[1]:	
Minimum Width:	12 inches
Minimum Embedment into Competent Bedrock[2]:	12 inches
Allowable bearing pressure	
Dead Plus Live Loads[3, 4]	5,000 psf
Base Friction Coefficient:	0.40
Lateral Passive Resistance[5]	
Bedrock:	450 pcf
Drilled Piers:	
Minimum Diameter:	18 inches
Minimum Embedment into Competent Bedrock[6]:	5 feet
Skin Friction[7]	
Fill, Colluvial, and Residual Soils:	Ignore
Bedrock:	4,000 psf
Lateral Passive Resistance[5,8]:	
Fill, Colluvial, and Residual Soils:	Ignore
Bedrock:	450 pcf

Notes:
(1) In weathered bedrock ("cut") areas, load all shallow foundations to similar bearing pressures, i.e. size footing widths to design loads instead of uniform foundation widths.
(2) Maintain minimum 7-feet horizontal confinement from the face of adjacent slopes.
(3) For fractured and weathered bedrock, may be increase for hard bedrock based on rock conditions observed during construction.
(4) May increase by 1/3 for total design loads (including wind and seismic). All foundations to bear directly on firm bedrock.
(5) Equivalent Fluid Pressure, not to exceed 10 times value in psf.
(6) Minimum depth may be reduced if hard rock is encountered, to be determined by the Geotechnical Engineer during construction.
(7) Uplift resistance is equal to 80% of the total skin friction.
(8) Apply values over effective width of 2 pier diameters.

Reinforced concrete slab-on-grade interior floors are also judged to be appropriate for the site. The concrete slabs-on-grade may be poured monolithically to the foundations or separated with a cold joint. We recommend that concrete slabs have a minimum thickness of 5 inches and be reinforced with steel reinforcing bars (not mesh). Concrete slabs that cross cut/fill or hard rock/natural soil transitions may experience small differential settlements that could cause cracking in the area of the transition.

E. Retaining Wall Design Criteria

Retaining walls will be utilized to create level building pads for the planned improvements and support the tall cut along the upslope side of the re-aligned access road. Preliminary plans indicate retaining walls will range to a maximum height of about 30-feet. For cost-effective construction, we recommend that site retaining walls consist of shotcrete-faced walls supported with soil-nails or rock anchors where cuts are planned. Alternatively, overexcavation and construction of conventional cast-in-place reinforced concrete walls could be used. However, temporary shoring of cut slopes will significantly complicate the construction of these types of walls.

Steeper, temporary slopes (than those discussed in Section V.B.5) may be possible during dry conditions and for short term excavations, such as cuts for soil-nail wall construction. However, adversely-bedded rock or seepage/weak soils near the ground surface may require flattening the temporary slopes. Five to six foot high vertical cuts should generally be feasible for construction of wall segments.

To reduce the risk of seepage through the below-grade walls (such as at the upslope side of the planned headworks building), construction of shotcrete shoring walls could be considered, followed by placement of waterproofing and construction of a permanent concrete retaining wall. All tiebacks and soil nails should be provided with double corrosion protection and a representative number of the tiebacks and soil nails need to be performance and proof-tested as determined by the engineer.

Retaining walls that can deflect at the top, such as landscape walls, can be designed using the unrestrained criteria shown in Table F. Walls that are structurally connected at the top and not allowed to deflect, such as basement or tied-back walls, are considered restrained. Restrained conditions are commonly designed using a uniform earth pressure distribution rather than an equivalent fluid pressure. Lateral support can be obtained from either passive soil resistance (i.e. keyways) or frictional sliding resistance of footings or from tiebacks. In addition to the soil loads, the retaining walls should be designed to resist temporary seismic loads.

TABLE F
RETAINING WALL DESIGN CRITERIA
SMCSD – Treatment Plant Upgrades
Sausalito, California

Foundation

Refer to the foundation design criteria in Table E.

Lateral Earth Pressure	Unrestrained[1,2]	Restrained[1,3]
Level Ground	45 pcf	25 X H psf
2:1 Slope	65 pcf	40 X H psf
Seismic Surcharge[3]	15 X H psf	

Soil Nails/Tiebacks

	Phi[4]	C (psf)[5]	Gamma (pcf)[6]
Fill/Colluvium/Residual Soil	32	350	130
Chert/Greenstone Bedrock	40	2,000	140

Min. Diameter Grouted Holes:	6 inches
Skin Friction:	
Fill/Colluvium/Residual Soil:	1,000 psf
Bedrock:	3,000 psf

Notes:
(1) Interpolate earth pressures for intermediate slopes.
(2) Equivalent fluid pressure.
(3) Rectangular uniform pressure distribution (H = height of wall).
(4) Angle of Internal Friction, effective stress, unitless
(5) Apparent (effective) Cohesion, for seismic conditions 500 psf of additional cohesion may be included for both materials
(6) Unit Weight of Soil
(7) Tiebacks and soil nails should be designed for load-testing up to 150% of the design load. Load testing to be performed in general accordance with the procedures recommended by the Post-Tensioning Institute (1996).

All walls higher than 3-feet require drainage to prevent the build-up of hydrostatic pressure. Either Caltrans Class 1B permeable material within filter fabric, drainage panels, or Caltrans Class 2 permeable material can be used. The project Architect should design a water-proofing system for walls adjacent to living space. The drainage should be collected in 4-inch, perforated, Schedule 40 PVC drain line placed at the base of the wall or discharged through weep-holes in the case of soil nail or cast-in-place concrete walls. Seepage collected in the drains should be conveyed in a closed pipe system to a suitable discharge outlet well away from the structures.

To maintain the wall drainage system, clean-outs must be provided for perforated pipes at the upstream end. Sweep fittings should be used at all major changes in direction. A typical retaining wall drain detail is shown on Figure 8. Retaining wall backfill should be compacted in accordance with the recommendations presented in site grading.

F. Concrete Slabs-On-Grade

Concrete slab-on-grade interior floors are deemed suitable for the site, pursuant to the discussion in Section V(D) above. Slabs-on-grade may be poured monolithically with the foundation or may be separated by a cold joint. We recommend interior concrete slabs be at least five inches thick and reinforced with steel bars (not wire mesh). A thicker slab with heavier reinforcing may be desirable to reduce potential slab cracking. Additionally, contraction joints should be incorporated in the concrete slab in both directions, no greater than 10-feet on center, and the reinforcing bars should extend through the control joints.

To improve interior moisture conditions, a five-inch layer of clean, free draining, 3/4-inch angular gravel or crushed base rock should be placed beneath the interior concrete slabs to form a capillary moisture break. The base rock must be placed on a properly moisture conditioned and compacted subgrade that has been approved by the Geotechnical Engineer. A plastic membrane vapor barrier, 15 mils or thicker, should be placed over the compacted base rock. The vapor barrier shall meet the ASTM E 1745 Class A requirements and be installed per ASTM E 1643. Eliminating the capillary moisture break and/or plastic vapor barrier may result in excess moisture intrusion through the floor slabs resulting in poor performance of floor coverings, mold growth or other adverse conditions.

Some interior flooring manufacturers will void their warranties if wet sand is utilized under the concrete slab. The sand layer may be eliminated from the section provided the vapor barrier (such as Stego Wrap™ or approved equivalent) is strong enough to withstand the construction of a concrete slab and is installed per ASTM E 1643.

Exterior concrete slabs should be at least 4-inches (100 mm) thick and reinforced as described above for interior slabs. Exterior concrete slabs shall be underlain with 4 inches (100 mm) or more of Caltrans Class 2 Aggregate Base compacted to at least 92 percent relative compaction. Some movement should be expected for exterior concrete slabs as the underlying soils react to seasonal moisture changes. If superior performance is desired, the exterior slabs can be thickened, reinforced as described above for interior slabs and/or underlain with a thicker aggregate base layer.

G. Site and Foundation Drainage

Careful consideration should be given to design of finished grades at the site. We recommend that the building areas be raised slightly and that the adjoining landscaped areas be sloped downward at least 0.25 feet for 5 feet (5 percent) from the perimeter of building foundations. Where hard surfaces, such as concrete or asphalt adjoin foundations, slope these surfaces at least 0.10 feet in the first 5 feet (2 percent). Area drains can be provided for landscape planters adjacent to buildings and parking areas and downspouts should discharge into a solid pipe collection system. Site drainage should be discharged away from the building area, preferably into an established storm drainage system.

Foundation drains should be constructed along upslope portions of the residence foundation and crawl space drains should be provided at the base of slopes to minimize the likelihood of water ponding in the crawl space. A typical foundation drainage detail is shown on Figure 9.

I. Pavements

New pavements will be required for the new access roads and driveways. We have calculated preliminary pavement sections in accordance with Caltrans procedures for flexible pavement design (2000). We have provided a range of Traffic Indices (TI) from 4 to 6 depending on the expected traffic loads for a twenty-year design life. We have estimated an R-value of 20 for the preliminary pavement design. The recommended pavement section is presented in Table G. Where slope grades exceed 15 percent, we recommend grooved concrete pavement be used.

TABLE G
PAVEMENT DESIGN CRITERIA
SMCSD – Treatment Plant Upgrades
Sausalito, California

	T.I.	Asphalt Concrete	Aggregate Base	Subgrade
Light passenger vehicles/parking	4	2.5 inches	7.0 inches	95% R.C.
Low truck traffic	5	3.0 inches	8.0 inches	95% R.C.
Frequent light truck traffic	6	4.0 inches	9.0 inches	95% R.C.
Frequent heavy truck traffic	7	5.0 inches	10.0 inches	95% R.C.

In general, 2-inches of aggregate base may be substituted for 1-inch of asphalt concrete if thinner structural sections are desired. Before placement of access road and driveway fills, an R-value test shall be performed on any import material to verify that it exceeds the design R-value of 20. The upper 8 inches of subgrade in pavement areas must be scarified, moisture conditioned to near the optimum water content, and then compacted to a minimum 95 percent relative compaction (ASTM D1557). The compacted surface must also be non-yielding when proof-rolled with heavy construction equipment.

The base rock should consist of compacted Class 2 Aggregate Base (Caltrans 2010), or approved alternate, compacted to achieve at least 95 percent relative compaction and a non-yielding surface when proof-rolled with heavy construction equipment.

J. Underground Utilities

Trench excavations having a depth of five feet or more must be excavated and shored in accordance with Cal/OSHA regulations. Pursuant to Cal/OSHA classifications, most on-site soils would be classified as Type B, while areas exposing hard rock may be classified as Type A. Utility trenches should be backfilled with soil compacted to at least 90 percent relative compaction. A minimum of 4-inches of sand (or other approved pipe bedding material) should be placed in the bottom of the trench excavation. The sand should be continuous around the utility pipe and extend at least 4-inches above the top of the pipe. The sand should be compacted to 90 percent relative compaction (R.C.).

Intermediate backfill above the sand to the subgrade elevation may be select on-site material or Caltrans Class 2 Aggregate Base. The backfill materials should be placed in uniform lifts no thicker than 8-inches, moisture conditioned to near optimum moisture content, and compacted to the specified degree of compaction. Within roadway and driveway areas, the intermediate backfill should be compacted to at least 90 percent relative compaction with the uppermost 12-inches compacted to 95 percent relative compaction, in accordance with ASTM D-1557. In non-structural (landscape) areas, the trench backfill should be constructed to at least 85 percent relative compaction.

VI. SUPPLEMENTAL GEOTECHNICAL SERVICES

We should review project plans as they near completion to verify that the intent of our geotechnical recommendation has been sufficiently incorporated and provide supplemental recommendations if needed. During construction, we should observe and test the geotechnical portions (site grading and preparation, foundations, retaining structures, and site drainage) of the project to confirm that subsurface conditions are as expected and the contractors work is performed in accordance with the contract documents.

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APPENDIX A

SUBSURFACE EXPLORATION AND LABORATORY TESTING

A. Soil and Rock Classification Systems

We have classified soil materials for engineering purposes in general conformance with ASTM Standard D 2488, "Field Identification and Description of Soils (Visual-Manual Procedure)" and the Unified Soil Classification System. These systems enable geotechnical engineers to correlate soil stratigraphy and compare physical soil properties. The soil classification system and symbols used for the soil borings and in discussions throughout this report are briefly explained on Figures A-1, Soil Classification Chart, and A-2, Rock Classification Chart.

B. Field Exploration and Sampling

We explored subsurface conditions at the site on June 11-12, 2013 and on June 19, 2013 with seven soil borings excavated at the locations shown on Figure 2. The purpose of the soil borings was to determine the subsurface soil and rock profile, examine the materials encountered, and obtain representative samples for laboratory testing. The exploration was performed under the technical supervision of our Field Geologist who examined and logged the soil materials encountered and obtained samples.

Soil borings were excavated at the locations shown on Figure 2 using a portable hydraulic-powered drill rig equipped with 4.0-inch diameter solid flight augers. Additional rock drilling was done using a 2.5-inch diameter pneumatic hammer bit (air hammer). Relatively “undisturbed” samples were collected from the soil borings using a 2.5-inch inside diameter, split-barrel “Modified California” sampler equipped with 2.5-inch by 6-inch brass liners and a 2.0-inch inside diameter “Standard Penetration Test” (SPT) sampler. The samplers were driven using a 140-pound hammer falling approximately 30-inches. Boring Logs are shown on Figures A-3 through A-10.

C. Laboratory Testing

We conducted laboratory tests on selected “undisturbed” samples to verify field identifications and to evaluate engineering properties. The following laboratory tests were conducted in accordance with the ASTM standard test method cited:

- Laboratory Determination of Water (Moisture Content) of Soil, Rock, and Soil-Aggregate Mixtures, ASTM D 2216,
- Density of Soil in Place by the Drive-Cylinder Method, ASTM D 2937;
- Unconfined Compressive Strength of Cohesive Soil, ASTM D 2166; and
- Amount of Material in Soils Finer than No. 200 (75-µm) Sieve, ASTM D 1140.

Laboratory test results are shown on the boring logs. The exploratory boring logs, descriptions of soils encountered and the laboratory test data reflect conditions only at the location of the excavation at the time they were excavated or retrieved. Conditions may differ at other locations and may change with the passage of time due to a variety of causes including natural weathering, climate, and changes in surface and subsurface drainage.