BSK ASSOCIATES

UPDATED LIMITED GEOLOGICAL SITE ASSESSMENT AND GEOTECHNICAL RECOMMENDATIONS REPORT CASTLEWOOD REDWOOD TANKS REPLACEMENT PLEASANTON, CALIFORNIA

BSK PROJECT NO. G16-062-11L

PREPARED FOR:

PAKPOUR CONSULTING GROUP, INC.
5776 STONERIDGE MALL ROAD
PLEASANTON, CALIFORNIA 94588

June 20, 2016

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June 20, 2016 **BSK Job No. G16-062-11L**

Pakpour Consulting Group, Inc.
5776 Stoneridge Mall Road
Pleasanton, California 94588

ATTENTION: Mr. Brandon Laurie, PE (blaurie@pcgengr.com)

SUBJECT: **Updated Limited Geological Site Assessment and Geotechnical Recommendations Report**
Castlewood Redwood Tanks Replacement
Pleasanton, California

Dear Mr. Laurie:

BSK Associates (BSK) is pleased to submit our updated limited geological site assessment and geotechnical recommendations report for the above referenced project. The enclosed report describes the geotechnical investigation performed and presents our geotechnical recommendations for foundations, retaining walls, earthwork, and pavements for the project.

In summary, it is our opinion that the tank site is suitable for the proposed construction provided that the geotechnical recommendations presented herein are followed for design and construction of the project. The main geotechnical concerns for the project are as follows:

1. The high potential for the tank site to be subjected to significant seismic ground shaking during a future earthquake on the Calaveras or other active faults in the region,
2. The high potential for the colluvium layer underlying the site to experience shallow landsliding (including seismically-induced) in the future, and
3. The potential for differential settlement to occur along cuts transitioning from fill/colluvium into conglomerate bedrock or due to differential thickness of fill/colluvium.

To address the first concern, we have included the results of our site-specific ground motion analysis, so that the structural engineer can incorporate it into the design of the new tank(s). We have also included dynamic seismic lateral earth pressures that can be used to design retaining walls. In order to address the other two concerns, we recommend that new water tank(s) be founded on a pier-supported mat foundation if they are located approximately where the existing tanks are located. Alternatively, if the location of the new tank(s) is shifted further to the west (upslope) so that they bear entirely on the conglomerate bedrock, they may be supported on mat foundations only. However, this could result in higher retaining walls and additional off haul material than currently planned. If cantilevered retaining walls are used at the tank site, they may be supported on continuous spread footings if they are founded on conglomerate bedrock. Otherwise, such walls should be supported on CIDH piers interconnected by a grade beam. Information on the investigative methods previously performed by others and our specific recommendations for design and construction of the project are contained in this report.

Conclusions and recommendations presented in the enclosed report are based on limited subsurface investigation and laboratory testing programs. Consequently, variations between anticipated and actual subsurface conditions may be found in localized areas during construction. If significant variation in the subsurface conditions is encountered during construction, BSK should review the recommendations presented herein and provide supplemental recommendations, if necessary.

Additionally, design plans should be reviewed by our office prior to their issuance for conformance with the general intent of our recommendations presented in the enclosed report.

We appreciate the opportunity of providing our services to you on this project and trust this report meets your needs at this time. If you have any questions concerning the information presented, please contact us at (925) 315-3151.

Sincerely,

BSK Associates Inc.

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Table of Contents

Plates and Appendices

1. INTRODUCTION

This report presents the results of our updated limited geological site assessment and geotechnical recommendations report for the Castlewood Redwood Tanks Replacement project in Pleasanton, California, hereafter referred to as "the project". A Vicinity Map showing the location of the tank site is presented on Plate 1. Our services have been performed for and coordinated with Pakpour Consulting Group (Pakpour).

Previous studies have been performed at the tank site by others as described in the "Previous Studies" section of this report. The seismic parameters provided in the previous reports were based on outdated versions of the AWWA, ASCE 7, and CBC codes. The site-specific ground motion analysis presented in the "Earthquake Ground Motion (2013 California Building Code)" section of this report is based on the 2013 CBC, ASCE 7-10, and AWWA D100-11 and D110-13 standards, which are considered current as of the date of this updated report.

This report contains a description of our findings, conclusions, and recommendations for the project. Note that our study relied on field and laboratory data, findings, and recommendations previously presented by others as discussed later in this report. We previously provided preliminary recommendations for this project in a draft memorandum entitled *Preliminary Geotechnical/Geological Recommendations, Castlewood Redwood Tanks Replacement, Pleasanton, California* dated May 10, 2016. The findings, conclusions, and recommendations presented in this report supersede those provided in our May 10, 2016 memorandum. Note that this report was originally issued on June 15, 2016, but has been revised to incorporate minor review comments by Pakpour.

1.1 Project Description

The Castlewood Service Area intends to replace two existing 100,000-gallon redwood storage water tanks located at the County of Alameda's Zone 2 site in the Castlewood Development in Pleasanton. The attached Site Plan, Plate 2, shows the two existing tanks. The tanks sit on the east facing slope of the Pleasanton ridge. These tanks have leaked extensively in the past. We understand there is a serious concern about the structural integrity of these tanks. A structural analysis of the tanks indicates both tanks lack mechanical fasteners and positive anchorage. Thus, these tanks are at risk of moving off their foundations and/or structurally failing during a significant seismic event.

The project will consist of demolishing and replacing the existing tanks with one larger or two smaller steel or concrete tank(s) meeting current AWWA D100 or D110 and 2013 CBC standards and having a storage capacity that matches or exceeds that of the existing tanks. The new tank(s) are expected to have diameter(s) ranging from about 30 to 50 feet. The anticipated maximum height of water to be stored in the tank(s) will be 20 feet. We understand that the new tank(s) will be considered an essential facility used for fire protection and emergencies in addition to regular water storage distribution. Accessibility to the tank site will also be improved via construction of a paved driveway and a pad around the tank(s), improved site drainage, and cut slope retention via construction of a retaining wall behind (upslope) of the new tank(s). The retaining wall is anticipated to be up to about 10 feet high. Retaining wall types being considered for the project include cantilevered, soldier pile and lagging (possibly with tiebacks), and soil nail wall.

Grading within the limits of the existing tanks and gravel driveway leading to the tanks is expected to be limited to cuts of 2 feet deep or less and fills less than 1 foot high. Cuts up to about 20 feet deep are expected for the planned retaining wall, the planned cut slope behind it, and portions of the new tank(s) along the west (upslope) side of the tank site. Existing and new underground utility lines are expected to be up to 5 feet deep.

If the actual project differs significantly from that described above, specifically if the grading differs from that we assumed above, we should be contacted to review and/or revise our conclusions and recommendations presented in this report.

1.2 Purpose and Scope of Services

The purpose of our study was to perform an updated limited geological site assessment and provide geotechnical recommendations for the design and construction of the project. The scope of services, as outlined in our March 15, 2016 proposal (File Number: GL15-11383) and April 22, 2016 amendment request letter (File Number: G16-062-11L), consisted of an updated limited geological assessment, pre-report consultation, laboratory testing, engineering analysis, site-specific ground motion analysis, and preparation of this report. Our study relied on the field and laboratory data, findings, conclusions, and recommendations previously presented by others as discussed later in this report as well.

Our study specifically excludes the assessment of site environmental characteristics, particularly those involving hazardous substances. Our scope of services did not include evaluation of contaminants in the soil, water, or air.

2. SITE INVESTIGATION

2.1 Previous Studies

Previous studies were performed at the tank site in 2007/2008, 2009, and 2012 by Cotton, Shires & Associates and Treadwell & Rollo (now Langan Treadwell Rollo). These studies were presented in the following documents, which are listed chronologically:

- *Geotechnical Investigation, California Water Service Company, Castlewood Tanks – Zone 2, Pleasanton, California*, dated January 18, 2008, by Cotton, Shires & Associates (File No. E0357);
- *Geologic and Geotechnical Services, Castlewood County Services Area, Redwood Tanks, Alameda County, California*, dated January 12, 2009, by Treadwell & Rollo (File No. 4916.1); and
- *Preliminary Geologic Hazards Evaluation and Subsurface Conditions, Castlewood Service Area Tanks and Pump Stations*, dated January 17, 2012 by Treadwell & Rollo (File No. 731575901).

Appendix A includes an Engineering Geologic Map (Plate 1) showing the location of the 2007 borings drilled at the tank site by Cotton, Shires & Associates (CSA), a description of their field investigation, boring logs, Triaxial Consolidated Undrained test results, and the Engineering Geologic Cross Section A-A' (Figure 4) showing a subsurface cross-section of the tank site inferred from the boring logs. Appendix A also includes two figures from the 2009 Treadwell & Rollo (T&R) study depicting the tank site's potential seismic hazards.

Throughout this updated report, we have incorporated relevant portions of the findings and conclusions from the above-referenced reports.

2.2 Current Study

A Certified Engineering Geologist (CEG) and a registered Geotechnical Engineer (GE) from BSK performed a geologic site reconnaissance on April 6, 2016 to observe existing surface conditions and exposed geologic features. During our reconnaissance, we collected two bulk samples of the surficial soils (upper 1 foot below the ground surface) for laboratory testing purposes. The approximate location of these samples (labeled as S-1 and S-2) are shown on Plate 2. Note that the locations of the surficial samples were estimated by our field representative based on rough measurements from existing features at the tank site. As such the sample locations should be considered approximate to the degree implied by the methods used.

2.3 Laboratory Testing

Laboratory tests were performed on selected soil samples to evaluate their physical characteristics and engineering properties. The previous and current laboratory testing program included dry density and moisture content, Atterberg limits, consolidated-undrained triaxial compression (TXCU), and Resistance (R)-Value testing. Some of the testing was performed by Cooper Testing Labs of Palo Alto, California. Some of the laboratory test results are presented on the individual boring logs in Appendix A. The results of the Atterberg limits, TXCU, and R-Value tests are also presented graphically in Appendices A and B.

Analytical testing was performed as part of our study on the surficial soil sample obtained from sampling location S-2, to assist in evaluating the corrosion potential of the on-site soils. The corrosivity testing was performed by CERCO Analytical of Concord, California using ASTM methods as described in CERCO Analytical's report. The corrosion results are presented at the end of Appendix B.

3. SITE CONDITIONS

3.1 Site Description

The tank site is situated near a sharp bend along Castlewood Drive, on a northeast facing slope, approximately halfway up the northwest/southeast trending Pleasanton Ridge. The two existing 100,000-gallon redwood tanks sit on a pad that appears to have been constructed by cutting into the hillside and filling the downslope side. The northernmost tank appears to be supported on shallow spread footings/stem walls and has a crawl space underneath it, while the southern tank appears to be supported on a mat foundation. The western perimeter of the tanks is surrounded by cantilevered retaining walls that appear to be up to 5 feet high.

Access to the tanks is provided via a gravel driveway that extends to the south of the tanks and connects to Castlewood Drive. Based on the elevation contours provided to us by Pakpour on June 1, 2016, the pad elevation is approximately 895 feet and the natural slopes to the west and north of the tank site range from about 2½H:1V (horizontal to vertical) to 1.6H:1V. A man-made cut slope immediately west of the tanks appears to have a gradient as steep as about 0.7H:1V, while the fill slope immediately east of the tanks appears to have a gradient as steep as 1H:1V. The slopes surrounding the tank site are covered by trees and sparse undergrowth vegetation.

3.2 Geologic Setting

As part of our evaluation of the geologic setting for the tank site area, a CEG from BSK performed a field reconnaissance of the tank site on April 6, 2016 to examine the current surficial site conditions. Our CEG also reviewed available geologic maps, publications, and historic aerial photographs for the tank site as discussed in the following subsections of this report.

3.2.1 Area Geology

The tank site is located in the Coastal Range geomorphic province that is characterized by north-south trending ridges and valleys that are typically highly folded with numerous faults. The tank site is located on a northeast-facing slope flanking the Pleasanton Ridge that consists of uplifted and folded Cretaceous sedimentary rocks. As shown on the Geology Map, Plate 3, these units strike north/northwest and dip to the west at approximately 30 to 60 degrees according to Dibblee and Minch (2005)[1]. The tank site is located on units mapped by Dibblee and Minch as Cretaceous age Panoche Formation described as clay shale or claystone, dark gray, micaceous, bedded, and includes a few thin sandstone layers. West and upslope of the tank site are Panoche Formation sandstone and conglomerate. Dibblee and Minch mapped a large landslide approximately 100 to 200 feet north of the tank site with a debris flow direction to the northeast, away from the site. The Calaveras fault was mapped by Dibble just below the tank site, approximately 500 feet to the northeast, with fault contacts between the Panoche Formation and landslide deposits east and downslope of the tank site.

As shown on Plate 1 in Appendix A, Cotton, Shires & Associates (CSA, 2008) identified dormant, active and old landslides southwest, northwest and east of the tank site. Based on our review of aerial photographs and observations made during our April 6, 2016 site reconnaissance, the landslides mapped by CSA on Plate 1 appear reasonable. The 2008 CSA subsurface investigation indicated that the tank site is located on approximately 6 to 13 feet of fill or colluvium overlying conglomerate and sheared claystone bedrock.

3.2.2 Site Reconnaissance

Our CEG and GE performed a site reconnaissance on April 6, 2016 to examine the tank site area for signs of slope instability, mass wasting, and other visible geologic hazards. The existing tanks were observed to be on a cut/fill pad with outcrops of colluvium Panoche Formation siltstone/claystone and conglomerate visible along its western (upslope) margins and fill along its eastern (downslope) margins. The slope above the tanks appeared to be colluvium with some fill from the road cut above the tanks. Similar to what was reported in the 2008 CSA report, we observed localized areas of instability in the cut slope immediately behind of the northern tank. The slope failures were shallow and consisted of slow moving slumps. About 2 feet of debris resulting from these failures appeared to have overtopped the existing short retaining wall behind the northern wall and accumulated between the wall and the tank foundation. The tanks were observed to be leaking, with the water draining downslope to the northeast. We also observed an erosional gully within the fill slope near the northern edge of the pad that appears to be the result of prolonged erosion from the water leak and possibly surface runoff. The features presented on Plate 1 (see Appendix A) of the 2008 CSA report appear to generally representative of current site conditions observed during our site reconnaissance.

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[1] Dibblee, T.W., and Minch, J.A. (2005), Geologic Map of the Dublin Quadrangle, Contra Costa and Alameda Counties, California: Dibblee Geological Foundation, Dibblee Foundation Map DF-164, scale 1:24,000.

3.2.3 Aerial Photograph Review

A review of historic aerial photographs for the tank site area was performed to evaluate the site's and surrounding property's geomorphic features for evidence of slope stability, drainage issues, and/or surface faulting. Stereo pairs of aerial photographs were obtained from Quantum Spatial, Inc. of Novato, California for review. The review included examination using a Leitz stereograph of the following aerial photographs:

LIST OF AERIAL PHOTOGRAPHS REVIEWED				
Film ID	Line	Frame	Scale	Date
KAV9015	13	18,19	1:7200	5/2/2005
AV6100	127	43,44,45	1:12000	6/29/1999
AV4625	27	39,40,41	1:12000	7/5/1994
AV3368	24	49	1:12000	8/18/1988
AV550	14,15	27	1:36000	7/22-23/1963
AV253	26	48	1:12000	5/4/1957
AV253	25	42	1:12000	5/16/1957

Based on our review of the above historical aerial photographs, there are several landforms that appear to be older landslides located southwest and north of the tank site. Evidence of older or active landslides that would impact the tank site were not observed on the reviewed aerial photographs. Erosional down-cutting features in gullies were observed upslope and to the southeast and downslope and to the south. Except for the erosional gully mentioned in the "Site Reconnaissance" section above, the other erosional features we observed near the tank site did not appear to be located in areas that could adversely impact the tank site.

3.3 Subsurface Conditions

On November 12 and December 7, 2007, CSA drilled a total of four (4) soil borings (labeled CSA/SD-1 through CSA/SD-4) within the tank site as shown on Plate 1 in Appendix A. The borings extended to depths of approximately 11½ to 47 feet below the ground surface (BGS). According to these borings, the eastern margins of the existing tank site are underlain by up to approximately 7 feet of fill consisting of loose to medium dense sand and medium stiff to very stiff (i.e., firm to hard) clay and silt. Below the fill, the borings encountered about 6 to 12 feet of colluvium consisting predominately of stiff to very stiff (i.e., firm to hard) sandy silt, but also containing clay and gravel. Underlying the colluvium, the borings encountered hard conglomerate bedrock with some interbedded layers of dense silty sand and weak, sheared claystone. Conglomerate is a coarse-grained sedimentary rock composed of rounded fragments within a matrix of finer grained material. The boring logs describe the conglomerate as rounded cobbles and gravel within a sand matrix.

Figure 4 in Appendix A presents a geologic cross-section of the subsurface conditions inferred by CSA from their 2007 borings. The location of this cross-section is shown on Plate 1 in Appendix A. Note that this cross-section is for illustrative purposes only and is based on the extrapolation and interpolation between and beyond the borings drilled by CSA in 2007 and the surficial observations made by CSA in their 2008 investigation. Therefore, this cross-section should be considered approximate. Actual subsurface conditions may vary and will need to be confirmed during grading by a qualified engineering geologist working for the Geotechnical Engineer-of-Record.

Atterberg limits testing performed by us on a sample collected from the upper 1 foot at sampling location S-2 resulted in a liquid limit (LL) of 38 and a plasticity index (PI) of 18. These results appear to be consistent with the Atterberg limits performed by CSA (as shown on their boring logs) at depths of approximately 3½ to 7½, which resulted in LL values of 39 to 42 and in PI values of 18 to 21. These results are indicative of soils with moderate expansion potential when subjected to changes in moisture content.

Free groundwater was encountered in boring CSA/SD-4 at a depth of approximately 47 feet BGS. It should be noted that groundwater levels can fluctuate several feet depending on factors such as seasonal rainfall, groundwater withdrawal, and construction activities on this or adjacent properties.

The above is a general description of soil and groundwater conditions encountered at the tank site in the previous borings by CSA. For a more detailed description of the soils encountered, refer to the boring log data in Appendices A.

It should be noted that subsurface conditions can deviate from those conditions encountered at the boring locations. If significant variation in the subsurface conditions is encountered during construction, it may be necessary for BSK to review the recommendations presented herein and recommend adjustments as necessary.

4. DISCUSSIONS AND CONCLUSIONS

Based on the results of our study, it is our opinion that the planned tank replacement project is feasible geotechnically and that the tank site may be developed as presently planned. This conclusion is based on the assumption that the recommendations presented in this report will be incorporated into the design and construction of this project.

Additional discussions of the conclusions drawn from our study, including general recommendations, are presented below. Specific recommendations regarding geotechnical design and construction aspects for the project are presented in the "Recommendations" section of this report.

4.1 Geologic and Seismic Hazards

4.1.1 Faulting and Seismic Shaking

The tank site and the San Francisco Bay Area are seismically dominated by the active San Andreas Fault system. This fault system movement is distributed across a complex system of generally strike-slip, right-lateral parallel and sub-parallel faults including, among others, the San Andreas, Hayward, and Calaveras faults. Nearby major active faults[2] include the Calaveras fault located approximately 500 feet to the northeast, the Hayward fault located approximately 9 kilometers (km) to the southwest, and the Greenville fault located approximately 20 km to northeast of the tank site. These faults are shown on the Map of Major Faults and Earthquake Epicenters in the San Francisco Bay Area, Figure 7, in Appendix A.

As shown on the Fault Zone Map[3], Plate 4, the tank site is located within an Alquist-Priolo Earthquake Fault Zone. According to the 1972 Alquist-Priolo Fault Zoning (AP) Act, a structure for human occupancy cannot be placed over the trace of an active fault (defined as having ruptured in the last 11,000 years) and must be set back from the fault (generally 50 feet). However, the AP act allows local agencies to be more restrictive than the law requires. We are not aware that fault trenching has been performed at the tank site in the past and Alameda County, the agency having jurisdiction over the tanks, did not require fault trenching for this current project. The 2008 CSA report indicated that the mapped location of the Calaveras fault at the base of the slope, approximately 500 feet to the northeast of the tank site, appears reasonable from a geomorphic evaluation of the area. Even though published maps do not

[2] An active fault is a fault that has ruptured in the last 11,000 years.
[3] California Division of Mines and Geology (1982), Special Studies Zones, Dublin Quadrangle, January 1, 1982.

indicate that the Calaveras fault crosses the tank site, the possibility that trace(s) of the Calaveras fault cross the site cannot be precluded without performing a fault trench investigation.

We expect the tank site to be subjected to substantial ground shaking due to a major seismic event on the active faults in the Bay Area and surrounding regions during the design life of the project. According to a recent study[4], there is a 63 percent probability that one or more magnitude M6.7 or greater earthquakes will occur in the San Francisco Bay Area between 2007 and 2036.

As has been demonstrated recently by the 1989 (M6.9) Loma Prieta, the 1994 (M6.7) Northridge, and the 1995 (M6.9) Kobe earthquakes, earthquakes of this magnitude range can cause severe ground shaking and significant damage to modern urban environments. Therefore, the design of the new tank(s) should incorporate the seismic design parameters presented in the "Earthquake Ground Motion (2013 California Building Code)" section of this report.

4.1.2 Landslides and Potential for Slope Failure

The tank site sits on the east flank of Pleasanton Ridge, which is an area with significant older deep-seated landslides. This area is labeled as "Area of Massive Landslides" on Plate 4. These landslides cover large portions of the slope below the crest of the Pleasanton Ridge and are modified by erosion and contain smaller younger landslides. These large slides consist primarily of sheared and broken Great Valley Sequence rocks that slid over the Calaveras Fault and cover Tertiary rocks and Pleistocene gravels at the base of the slope[5]. As shown on the Regional Seismic Hazard Zones Map[6], Figure 6, in Appendix A, the tank site is also located within an earthquake-induced landslide hazard zone.

The 2008 CSA report did not map any landslides within the tank site (refer to Plate 1 in Appendix A). However, the CSA report concluded that the potential for future shallow landslides (including seismically-induced) to occur at the tank site is high. Shallow landslides in this area could be triggered within the colluvium and fill layers by excessive precipitation

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[4] Field, E.H., Miler, K.R., and the 2007 Working Group on California Earthquake Probabilities (2008), Forecasting California's Earthquakes – What Can We Expect in the Next 30 Years?: U.S. Geological Survey, Fact Sheet 2008-3027, 4 p. (http//:pubs.usgs.gov/fs/2008/3027/).

[5] California Geological Survey (2008), Seismic Hazard Zone Report for the Dublin 7.5-Minute Quadrangle, Alameda County, California, Seismic Hazard Zone Report 112.

[6] California Geological Survey (2008), Seismic Hazards Zones, Dublin Quadrangle, August 27, 2008.

combined with poor drainage and/or strong ground shaking during an earthquake. Such ground movement could disrupt future access to the tanks and result in damage to underground utility lines traversing the colluvium and fill layers. CSA also indicated that they were unable to characterize the potential risk of a large, deep-seated landslide extending below the tank site. While regional geologic mapping by Majmundar (1996)[7] identifies the tank site as being underlain by a large landslide, other geologic maps, such as Dibblee and Minch (2005) and Graymer et al. (1996)[8], do not. In order to better define the potential risks from a large, deep-seated landslide at the tank site, additional subsurface exploration would be necessary, including deep, large-diameter borings. The CSA landslide assessment appears to be reasonable based on our review of the currently available data.

As pointed out in the 2009 report by T&R, the long-term stability of many hillside areas is difficult to predict. A hillside will remain stable only as long as the existing slope equilibrium (i.e., stability) is not disturbed by natural processes or by the acts of man. Landslides can be activated by a number of natural processes, such as loss of support at the bottom of a slope by stream erosion or the reduction of soil strength by an increase in the groundwater level or saturation of the surficial soil by excessive precipitation. Negative effects caused by man may include improper grading activities resulting in poor drainage and/or excessive loading of slopes, the introduction of excess water through irrigation, and improperly designed or constructed leach fields.

4.1.3 Expansive Soils

Laboratory test data (refer to the “Subsurface Conditions” section of this report), indicates that the near-surface soils encountered in the 2007 CSA borings and sampling location S-2 have a moderate expansive potential when subjected to changes in moisture content. Mitigation of expansive soil behavior is recommended by underlying exterior concrete flatwork with “non-expansive” fill and moisture conditioning of the subgrade soils as discussed in the “Recommendations” sections of this report.

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[7] Majmundar, H. H. (1995), Landslide Hazards in the Hayward Quadrangle and Parts of the Dublin Quadrangle, Alameda and Contra Costa Counties, California, Landslide Hazards Identification Ma No. 37, scale 1:24,000. DMG OFR95-14.

[8] Graymer, R. W., Jone, D. L., and Brabb, E. E. (1996), Preliminary Geologic Map Emphasizing Bedrock Formation in Alameda County, California: Derived from the Digital Database Open-File 96-252.

4.1.4 Liquefaction and Lateral Spread

Soil liquefaction is a condition where saturated, granular soils undergo a substantial loss of strength and deformation due to pore pressure increase resulting from cyclic stress application induced by earthquakes. In the process, the soil acquires mobility sufficient to permit both horizontal and vertical movements if the soil mass is not confined. Soils most susceptible to liquefaction are saturated, loose, clean, uniformly graded, and fine-grained sand deposits. If liquefaction occurs, foundations resting on or within the liquefiable layer may undergo settlements and/or a loss of bearing capacity.

Due to the presence of shallow bedrock at the tank site and the depth of groundwater (47 feet BGS according to boring CSA/SD-4), we conclude that the potential for liquefaction to occur at the site to be very low. As shown on the Regional Seismic Hazard Zones Map, Figure 6, in Appendix A, this is consistent with the seismic hazard mapping by the California Geological Survey (CGS) for the Dublin quadrangle, which shows the tank site outside the zone of potential liquefaction.

Lateral spread is a potential hazard commonly associated with liquefaction where extensional ground cracking and settlement occur as a response to lateral migration of subsurface liquefiable material. These phenomena typically occur adjacent to free faces such as slopes, creek channels, and levees. Because the liquefaction potential at the tank site is considered to be very low, we conclude that the potential for lateral spread to affect the site is low.

4.1.5 Dynamic Compaction/Seismic Settlement

Another type of seismically induced ground failure, which can occur as a result of seismic shaking, is dynamic compaction, or seismic settlement. Such phenomena typically occur in unsaturated, loose granular material or uncompacted fill soils. The potential for dynamic compaction settlement to occur in the conglomerate bedrock is considered low. However, based to our analysis[9], the portion of the tank site underlain by fill and colluvium could experience up to about ½-inch of dynamic compaction during a design-level earthquake. This settlement would be in addition to the elastic settlement discussed in the "Anticipated Settlements" section below. Differential dynamic compaction settlement is expected to be up to about two-thirds of the total value discussed above and to occur over a horizontal distance of approximately 30 feet.

[9] Tokimatsu, K. and Seed, H. B. (1987), Evaluation of Settlements in Sands Due to Earthquake Shaking, Journal of Geotechnical Engineering, ASCE, Vol. 113, No. 8, August, pp. 861-878.

4.2 Foundation Considerations

4.2.1 Foundation Type

Due to the variable thickness of fill and colluvium underlying the existing tank pad, the potential for shallow landsliding in the colluvium, and the potential for differential settlement of the surficial soils and between cuts transitioning from fill/colluvium to conglomerate bedrock, a cast-in-drilled-hole (CIDH) pier-supported mat foundation should be used to support the new tank(s). However, it is possible that, in order to resist the lateral loads discussed later in this report, the piers may need to be sized so large that the design becomes unfeasible. If the location for the new tank(s) is shifted further to the west (upslope) so that they bear entirely on the conglomerate bedrock, they may be instead supported on mat foundations only.

Cantilevered retaining wall(s) may be supported on continuous spread footings if they are founded on conglomerate bedrock that extends at least 7 feet laterally towards any nearby slopes. Otherwise, such walls should be supported on CIDH piers interconnected by a grade beam.

4.2.2 Adjacent Underground Utilities

Where footings or mat foundations are located near underground utilities (existing or new), the foundations should extend below a 1H:1V plane projected upward from the bottom of the underground utility to avoid surcharging it. Otherwise, the utility line should be evaluated to confirm it can handle the surcharge load, be relocated so it is not surcharged by the nearby foundation, or the trench backfill portion below the zone being surcharged should consist of a 2-sack mix of sand-cement slurry. Underground utility plans should be reviewed by BSK prior to trenching for conformance to these requirements.

4.2.3 Anticipated Settlements

We expect CIDH pier-supported structures to experience very little to no settlement. We estimate elastic settlement will be less than ½-inch for mat foundations and footings bearing on conglomerate bedrock. Most of this settlement is expected to occur during construction as the loading is applied or as the tanks are filled to capacity. Differential elastic settlement is expected to be about half of the total estimated elastic settlement over a horizontal distance of about 30 feet.

4.3 Retaining Wall

A retaining wall is planned behind (upslope) of the new tank(s). Wall types currently being considered include cantilevered, soldier pile and lagging (possibly with tiebacks), and soil nail wall. Caution should be exercised during construction of the retaining wall to reduce the risk of undermining the slope. For this reason, we do <u>not</u> recommend using a keystone-type or mechanically stabilized earth (MSE) wall because these types of walls require geogrid to anchor the wall facing. Use of geogrid would require a considerable amount of cutting behind the wall, which could adversely impact the segment of Castlewood Drive immediately upslope of the tank site and also expose the site to a higher risk of shallow landsliding in the colluvium layer during construction.

To help collect and dispose of surface water behind the retaining wall, we recommend that a concrete-lined ditch be constructed immediately behind the planned retaining wall. The ditch should discharge directly into a catch basin or another appropriate drainage inlet other than the wall drain. If weep holes are used for the wall drain, a concrete-lined ditch should also be installed along the toe of the wall and this ditch should also discharge into catch a basin. **The long-term maintenance and periodic clearing of these ditches is imperative to the design life of the retaining wall.**

If a cantilevered retaining wall is selected, we anticipate that a temporary back cut gradient of 1H:1V may be temporarily stable during construction. However, the actual temporary back cut gradient should be assessed during construction under the observation of a qualified engineering geologist working for the Geotechnical Engineer-of-Record. If signs of instability are observed during construction, the back cut excavation should be immediately halted until the engineering geologist has an opportunity to observe the back cut and provide input. Regardless of the temporary gradient used, the exposure time[10] of the back cuts should be minimized during construction, especially if grading activities take place during the winter (i.e., wet) season. As the backfill for the wall is being placed, the back cut should be benched back a minimum of 1 foot laterally from the back cut face. The back cut should be benched back at a maximum vertical interval of 1 foot. The gradient at the surface of the wall backfill should not exceed 2H:1V. However, we understand that steeper surface backfill gradients may be required near the northern and southern margins of the wall because the existing gradients in these areas are up to 1.7H:1V. If the zone having a backfill with a surface gradient steeper than 2H:1V is wider than about 2 feet, than that portion of the wall should be designed using the lateral

[10] Length of time between when the cuts are excavated and the new wall is backfilled.

earth pressures associated with a sloped backfill up to 1.7H:1V (refer to Table 3 later in this report).

Note that minor, surficial slope instabilities could occur along the surface and boundaries of the retaining wall backfill. Such slope instabilities tend to develop over time in the upper 2 to 3 feet below the ground surface and are generally associated with soil erosion and rilling rather than mass movement. Such erosional features could propagate and lead to larger slope failures if not corrected and if some erosion control and slope maintenance is not performed in an appropriate timeframe. For this reason, we recommend that a long-term maintenance program be implemented for the tank site.

To reduce the risk of debris generated from soil erosion and rilling to overtop the retaining wall, we recommend that the wall extend at least a couple feet above the ground surface upslope of the wall to form a screen wall. Periodic cleaning behind this screen wall should be performed as part of the long-term maintenance program recommended above for the tank site or accumulated soil could eventually overtop the screen wall.

4.4 Cut Slope

To reduce the potential for future shallow landsliding at the tank site, all permanent cuts should be supported by retaining walls and the retained cut slope should be no steeper than 2H:1V. Preliminary grading plans prepared by Pakpour show a proposed cut along the existing slope located immediately behind (upslope) of the new tank(s). We understand the purpose of this cut is to reduce (i.e., make it flatter) the existing slope gradients. This will result in cuts of up to about 20 feet deep and overall gradients of 2H:1V or flatter behind the planned retaining wall except at the northern and southern margins of the cut, where we understand the existing slope gradients are up to 1.7H:1V. Note that where the slope gradient is steeper than 2H:1V, there will be an increased risk of instability.

Care should be exercised by the contractor during grading of this slope to avoid overcutting below design finished grades. In the event the slope is overcut and a nominal amount of fill is placed, the Geotechnical Engineer-of-Record should be contacted to assess what measures, if any, should be implemented during placement of the fill. Such measures may include excavation of keyway(s), installation of subdrain line(s), benching into the cut slope as the fill is placed and compacted, and overbuilding the fill laterally and then cutting it back to allow for proper compaction of the finished slope face.

4.5 Grading Along the Eastern Margins of the Existing Tank Site

We recommend that grading of the eastern margins of the existing tank site be kept to a minimum as that area is underlain by 10+ feet of fill and colluvium that are susceptible to shallow landsliding. If it is necessary to add more than one foot of fill to that side of the tank site, we recommend using geogrid reinforcement within the outboard (downslope) portion of the fill instead of using a short retaining wall to retain the fill. Miragrid® 2XT biaxial geogrid or a USA manufactured equivalent should be used to reinforce the fill. The outboard gradient of the fill should not exceed 1.5H:1V. The geogrid should be installed according to the manufacturer's recommendations. For estimating purposes, the first layer of geogrid should be located 1 foot above the base of the new fill and be spaced at a maximum of 1-foot vertical intervals. The geogrid should extend laterally a minimum of 4 feet back from the face of the slope. The eastern (downslope) margins of the fill should be overbuilt laterally by 1 foot by the contractor and then trimmed back to design grades to expose a firm and well-compacted slope face. The geogrid reinforcement does not need to extend into the 1-foot lateral overbuilt zone. This requirement should be included in the notes section of the grading plans for the project. To lower the potential for erosion to occur along the outboard side of the fill, we recommend that a face wrap such as Miramesh® GR or equivalent be placed between successive geogrid layers.

The above geogrid layout and configuration is only preliminary and will need to be finalized on a case-specific basis. Most geogrid manufacturers, such as Tencate, can help the designer choose the final geogrid layout at no additional charge. BSK should review the final geogrid layout prior to the start of construction.

As an alternative to geogrid reinforcement, a keystone-type or MSE wall could be used to support the outboard side of the fill. However, there would be a risk that future shallow landsliding downslope of the eastern margins of the tank site could undermine the base of such a wall. If the project owner cannot tolerate this risk, a short cantilevered wall supported on CIDH piers interconnected by a grade beam could be used in this area instead. However, the piers would need to extend into bedrock as discussed in the "Pier-Supported Mat Foundation" section of this report.

4.6 Site Drainage

Proper site drainage is important for the long-term performance of the planned tanks, pavements, and concrete flatwork. As previously noted, an erosional gully that appears to be the result of prolonged erosion caused by the water leaking from the existing tanks and possibly surface runoff is present at the northern edge of the tank site. Therefore, it is important that the tank site area be graded to provide proper drainage away from foundations and slopes towards storm drain inlets and concrete lined ditches. The site should generally be graded so as to carry surface water away from the tank and retaining wall foundations at a minimum of 2 percent in paved areas and 5 percent in landscaped areas to a minimum of 10 feet laterally from these structures, where achievable. All roof gutters should be connected directly into a storm drainage system, or drain onto an impervious surface (not splash blocks) that drain away from the structure, provided that a safety hazard is not created.

Surface water ponding should not be allowed anywhere on the tank site during or after construction. Continuous, raised asphalt or concrete curbs should be constructed along the east shoulder of the portion of Castlewood Drive located immediately upslope of the tank site and along the east margins of the proposed paved driveway. For enhanced protection of the proposed cut slope from erosion and potential instability caused by saturation of the surficial soils, consideration should be given to installing a V-ditch at the crest of this slope (near the east shoulder of the portion of Castlewood Drive located immediately upslope of the tank site) so that surface runoff is diverted away from the slope.

Landscaping for the project should consist of drought resistant trees and vegetation that requires a minimum amount of watering. Otherwise, there is a risk that irrigation water at the site could trigger a future shallow landslide. We recommend consulting with a landscape specialist and/or arborist during selection of the type and layout of the landscaping for the project.

4.7 Underground Utility Lines

Due to the potential for differential movement due to elastic and dynamic compaction settlement, we recommend that flexible joints be installed along the transition zone of underground pipelines where they cross between fill/colluvium to conglomerate bedrock. Flexible joints should also be used where underground utility lines connect to mat foundations supported on CIDH piers. Depending on how much vertical offset these joints can handle, multiple joints installed in series may be required.

5. RECOMMENDATIONS

Presented below are recommendations for the design of the tank foundation, retaining wall, seismic considerations, earthwork, pavement, and construction considerations for this project.

5.1 Tank Foundation

5.1.1 Pier-Supported Mat Foundation

The CIDH piers should derive their vertical load capacities through skin friction on the side of the piers only in conglomerate bedrock. For resistance to uplift loads, the weight of the piers, the mat, and the empty tank(s), and the skin friction between the piers in conglomerate bedrock may be used. **Skin friction should be neglected for the portion of the piers extending from a depth of 2 feet into the conglomerate bedrock to the top of the piers.** An allowable skin friction value of 500 pound per square foot (psf) may be used to resist downward loads below a depth of 2 feet into the conglomerate bedrock. A one-third increase is permitted for downward transient loading, such as wind and seismic. The dead plus live load friction resistance includes a safety factor of at least 2 and the total design downward frictional resistance of about 650 psf (including wind and seismic) includes a safety factor of at least 1½. Uplift loads for short-term conditions should not exceed 2/3 of the allowable downward skin friction. The piers should have a minimum depth of 10 feet into the conglomerate bedrock. The piers should have a minimum diameter of 18 inches and should be spaced at least 3 diameters apart, center to center, or skin friction reductions may be necessary.

The top of the CIDH piers should be structurally connected to the mat foundation, which should be designed to distribute all the vertical and lateral loads applied to the mat by the tank(s) directly to the piers. **Bearing capacity and lateral resistance of the mat should be neglected.**

In addition to the lateral loads imposed by the water tank(s), the piers should be designed to resist a lateral creep load (equivalent fluid pressure) of 45 pounds per cubic foot (pcf) applied against the upslope side of the mat foundation from the top of the piers to the top of the conglomerate bedrock.

Resistance to lateral loads for CIDH piers can be provided by passive resistance against the face of piers using an allowable equivalent fluid pressure of 350 pcf up to a maximum of 2,000 psf acting against the piers below a depth of 2 feet into the conglomerate bedrock. The passive resistance may be applied to a width of twice the diameter of the piers. Piers should be spaced at least 6 diameters apart, center to center, in the direction of loading or lateral resistance capacity reductions may be necessary. The passive pressure value includes a factor of safety of at least 1.5. **Passive pressure should be neglected for the portion of the piers extending from a depth of 2 feet into the conglomerate bedrock to the top of the piers.**

If the structural engineer desires to instead analyze the lateral load resistance of the CIDH piers using LPILE, we recommend that a dense "Sand" P-Y curve soil model be used to model the conglomerate bedrock. A total unit weight of 125 pcf, a friction angle of 38 degrees, and a soil modulus of 225 pci may be used in the analysis. The zone extending from the top of the piers to a depth of 2 feet into the conglomerate bedrock should be modeled as the pier sticking out of the ground. The lateral loading discussed above should be used in the LPILE analysis.

Based on Figure 4 in Appendix A, an average depth of 10 feet may be assumed for the combined fill/colluvium layers underneath the existing tanks.

5.1.2 Mat Foundation

If the tank(s) are shifted to the west so that the mat foundation would bear directly on the conglomerate bedrock, the mat should have a minimum embedment depth at the edges of 36 inches. An allowable bearing pressure of 2,500 psf may be used for dead and long term live loads. The allowable bearing pressure value may be increased by 1/3 for short term seismic and wind loads. Bearing capacity values include a factor of safety of at least 2.

During construction, any portion of the mat foundation excavations and an area extending 7 feet from the outer edge of the mats that do not expose conglomerate bedrock, should be overexcavated until bedrock is exposed. The bottom of the resulting excavation should be keyed and benched as directed by the Geotechnical Engineer-of-Record's engineering geologist during construction, and then backfilled with a 2-sack sand-cement slurry. The keyway should be embedded at least 5 feet into bedrock unless otherwise indicated by the Geotechnical Engineer-of-Record' engineering geologist.

Lateral loads may be resisted by a combination of friction between the bottom of the mat and the supporting bedrock and by passive resistance acting against the sides of the mat. The frictional and passive resistance may be assumed in design to act concurrently. An allowable friction coefficient of 0.40 between the bottom of the mat and supporting bedrock may be used. For passive resistance, an allowable equivalent fluid pressure (unit weight) of 350 pcf may be used. The friction and passive values include factors of safety of about 1½.

Passive resistance in the upper foot of bedrock cover below finished grades should be neglected unless the ground surface is confined by concrete flatwork, pavement, or other such positive protection.

5.1.3 Construction Considerations

5.1.3.1 Mat Foundations

Concrete for mat foundations should be placed neat against undisturbed conglomerate bedrock or sand-cement slurry. It is important that mat foundation excavations not be allowed to dry before placing concrete. The excavations should be periodically moistened until concrete placement. During excavation of the mat foundation, if the conglomerate bedrock exposed at the bottom of the excavation becomes disturbed, it should be properly compacted to a firm and stable condition. Refer to the “Earthwork” section of this report for compaction requirements. If desired, a leveling course consisting of a 2- to 3-inch thick rat slab or a 4- to 6-inch thick layer of compacted Caltrans Class 2 aggregate base may be placed at the bottom of the mat foundation.

5.1.3.2 CIDH Piers

Because CIDH piers, if used, will extend into hard conglomerate bedrock containing cobbles and gravel within a sand matrix, difficult drilling should be anticipated. Therefore, heavy-duty drilling equipment will most likely be necessary to drill the piers to the design depth. Temporary casing of the pier holes may be necessary during construction to reduce the risk of caving of coarse grained materials encountered in the 2007 CSA borings. Therefore, the contractor installing the piers should be prepared to handle unstable pier hole conditions. If temporary casing is used during construction, it should consist of smooth walled steel casing. Corrugated metal pipe (CMP) should not be used as casing because it has a tendency to create voids or disturbed zones during removal.

We recommend that steel reinforcement and concrete be placed within about 4 to 6 hours upon completion of each pier hole and that holes be poured the same day they are drilled to reduce the potential for caving of the granular soils. The soils exposed in the holes should not be allowed to dry prior to the placement of concrete, since such drying could have an adverse impact on the performance of the piers. The bottom of the pier holes should be cleaned such that no more than two inches of loose soil remains in the hole prior to the placement of concrete. A concrete mix with a low water/cement ratio should be used in the construction of the piers to reduce shrinkage of the concrete. To increase the fluidity of the mix for improved consolidation and bond with the reinforcing steel, increased slump may be desirable. If this is the case, the slump should be increased via use of a plasticizer, rather than by adding water to the mix, because a low water to cement ratio is desired for shrinkage control. The steel reinforcement should be centered in the pier holes. Concrete used for pier construction should be discharged vertically into the pier holes to reduce aggregate segregation. Under no circumstances should concrete be allowed to free-fall against either the steel reinforcement or the sides of the excavation during construction.

If water is present during concrete placement, either the water needs to be pumped out or the concrete needs to be placed into the hole using tremie methods. If tremie methods are used, the end of the tremie pipe must remain below the surface of the in-place concrete at all times. In order to develop the design skin friction value provided above, concrete used for pier construction should have a slump of 4 to 6 inches if placed in a dry shaft without temporary casing, and from 6 to 8 inches if temporary casing is used.

Unit prices for temporary casing, dewatering, placement of concrete using tremie methods, and contingencies for slower than anticipated drilling should be obtained during bidding.

5.1.4 Construction Observation and Testing

All foundation excavations, including CIDH piers (if applicable), should be monitored by a representative of BSK during construction, including periodic observation by our engineering geologist. The purpose of such observation would be to:

- Check bottom conditions prior to placing steel reinforcement and concrete, including confirming that the subsurface conditions encountered are consistent with our recommendations, the adequacy of the supporting materials exposed, and moisture control;
- Check the overall foundation dimensions against the project plans and our recommendations;
- Check the need to overexcavate the bottom and area adjacent to the foundation excavations and backfill with sand-cement slurry if a mat foundation only is used to support the tanks; and
- Perform compaction testing of the bottom of the mat foundation excavations (if applicable).

5.2 Earthquake Ground Motion (2013 California Building Code)

5.2.1 Site Class

Based on Section 1613.3.2 of the 2013 California Building Code (CBC), the Site shall be classified as Site Class A, B, C, D, E or F based on the Site soil properties and in accordance with Chapter 20 of ASCE 7-10. Based on the previous investigation by Cotton, Shires & Associates, presented in their report dated January 18, 2008, the site is located on soil consisting of colluvium to a depth of approximately 13 feet BGS. Below the colluvium is bedrock material consisting of Cretaceous Conglomerate and Claystone. These rock units were identified by Dibblee and Minch (2005) as belonging to the Upper Cretaceous Panoche Formation. Based on the thickness of the soil mantle, we classify the Site Class as C (very dense soil and soft rock).

5.2.2 Seismic Design Criteria

The 2013 CBC utilizes ground motion based on the Risk-Targeted Maximum Considered Earthquake (MCER) that is defined in the 2013 CBC as the most severe earthquake effects considered by this code, determined for the orientation that results in the largest maximum response to horizontal ground motions and with adjustment for targeted risk. Ground motion parameters in the 2013 CBC are based on ASCE 7-10, Chapter 11.

The United States Geologic Survey (USGS) has prepared maps presenting the Risk-Targeted MCE spectral acceleration (5% damping) for periods of 0.2 seconds (SS) and 1.0 seconds (S1). The values of S_S and S_1 can be obtained from the USGS Ground Motion Parameter Application available at: http://earthquake.usgs.gov/designmaps/us/application.php

Table 1 below presents the spectral acceleration parameters produced for Site Class C by the USGS Ground Motion Parameter Application and Chapter 16 of the 2013 CBC based on ASCE 7-10.

Table 1 - Spectral Acceleration Parameters
Risk Targeted Maximum Considered Earthquake

Criteria	Value		Reference
MCE Mapped Spectral Acceleration (g)	S_S = 2.400	S_1 = 0.911	USGS Mapped Value
Site Coefficients (Site Class C)	F_a = 1.000	F_v = 1.300	ASCE Table 11.4
Site Adjusted MCE Spectral Acceleration (g)	S_{MS} = 2.400	S_{M1} = 1.184	ASCE Equations 11.4.1-2
Design Spectral Acceleration (g)	S_{DS} = 1.600	S_{D1} = 0.790	ASCE Equations 11.4.3-4

5.2.3 Seismic Design Category

Because the tank(s) are considered an essential facility, they should be classified as Risk Category IV per Table 1604.5 of the 2013 CBC. The long period spectral response acceleration coefficient, S_1, presented in the table above is greater than 0.75g. Therefore, a Seismic Design Category F should be assigned to the project per Section 1613.3.5 of the 2013 CBC.

5.2.4 Geometric Mean Peak Ground Acceleration

Per Section 1803.5.12 of the 2013 CBC, the peak ground acceleration (PGA) utilized for dynamic seismic lateral earth pressures and liquefaction, shall be based on a site specific study (ASCE 7-10, Section 21.5) or ASCE 7-10, Section 11.8.3. The USGS Ground Motion Parameter Application based on ASCE 7-10, Section 11.8.3 produced the values shown in Table 2 below based on Site Class C.

Table 2 - Geometric Mean Peak Ground Acceleration
Maximum Considered Earthquake

Criteria	Value	Reference
Mapped Peak Ground Acceleration (g)	PGA = 0.935	USGS Mapped Value
Site Coefficients (Site Class C)	F_{PGA} = 1.000	ASCE Table 11.8-1
Geometric Mean PGA (g)	PGA_M = 0.935	ASCE Equations 11.8-1

5.2.5 Site-Specific Ground Motion Analysis

As requested by the project structural engineer, a site-specific ground motion hazard analysis was performed in accordance with ASCE 7-10 Chapter 21, Section 21.2 and modified to meet the criteria in AWWA 100-11 and AWWA 110-13. Our ground motion analysis includes:

1. Determination of risk-targeted maximum considered earthquake (MCER) ground motion, deterministic MCER ground motion, and probabilistic MCER ground motion;

2. Determination of site-specific maximum considered earthquake geometric mean (MCEG) peak ground acceleration;

3. Scaling of the design response spectrum was performed to reflect the 0.5% damped response based on Damping Scaling Factors (DSFs) presented in AWWA 100-11 and AWWA 110-13, which use a DSF of 1.5 to scale from 5% to 0.5% damping; and

4. The analysis was performed according to the requirements of ASCE 7-10, Sections 21.2 through 21.5.

5.2.6 Deterministic MCER Ground Motion

Estimates of the MCE deterministic ground motion were computed using the software program EZ-Frisk (Version 7.65) developed by Risk Engineering. The EZ-Frisk analysis indicates that the Calaveras Fault source would produce the highest ground motion at the site from a deterministic standpoint. At periods above 0.1 second and below 0.5 seconds, the California Gridded dominates the ground motion.

Site-specific ground motions can be influenced by the types of faulting, magnitudes of the earthquakes, and local soil conditions. Ground Motion Prediction Equations (GMPE) account for these effects and are used to make estimates of ground motion at a site resulting from a scenario earthquake. Many GMPEs have been developed to estimate the variation of spectral acceleration with earthquake magnitude and distance from the site to the source of an earthquake. Next Generation Attenuation of Ground Motion relationships were developed by the Pacific Earthquake Engineering Research (PEER) Center that presented GMPEs for shallow crustal earthquakes in Western North America.

The 84th percentile of the maximum rotated component ground motion values were computed using four different Next Generation Attenuation relationships (NGAs). Distant Cascadia sources did not significantly increase acceleration values. The site ground motion is dominated by numerous local faults, therefore Cascadia sources were not included in the analysis. The acceleration values from each of four attenuation relationships were averaged using equal weight. The following attenuation relationships were used in the analysis:

- Boore-Atkinson (2008)[11] NGA Maximum Rotated Horizontal Component
- Campbell-Bozorgnia (2008)[12] NGA Maximum Rotated Horizontal Component
- Chiou-Youngs (2008)[13] NGA Maximum Rotated Horizontal Component

[11] Boore, D.M. and Atkinson, G.M. (2008), Ground-Motion Prediction Equations for the Average Horizontal Component of PGA, PGV, and 5%-Damped PSA at Spectral Periods between 0.01s and 10.0s, Earthquake Spectra 24:1, pp. 99-138.

[12] Campbell, K.W., and Bozorgnia, Y. (2008), NGA Ground Motion Model for the Geometric Mean Horizontal Component of PGA, PGV, PGD and 5% Damped Linear Elastic Response Spectra for Periods Ranging from 0.01 to 10 s, Earthquake Spectra 24:1, pp. 139-171.

- Abrahamson-Silva (2008)[14] NGA Maximum Rotated Horizontal Component

The analysis included seismic sources, based on the 2008 USGS fault model, within 200 kilometers of the site.

Amplification was accounted for in the analysis using the shear wave velocity (Vs) of 2,237 fps (682m/s) estimated from the average shear wave velocity of 13 feet of alluvium and 87 feet of Panoche Formation (Wills, 1998)[15]. In addition, some of the GMPEs require input for Z1.0 (defined as the depth in meters to a layer with Vs = 1,000 m/s) and Z2.5 (depth in km to a layer with Vs= 2,500 m/s). These two parameters intend to capture the basin effect on site response. The Z1.0 parameter is estimated to be 400 meters based on relationships established for Mesozoic sedimentary rocks. The project site is located in the East Bay Trough with an estimated depth of 3 kilometers to 2,500 m/s for older Mesozoic sedimentary rocks (Brocher, 2005)[16].

As specified in ASCE 7-10, Section 21.2.2, the deterministic spectral acceleration values representing the MCER are taken as the 84th percentile of the maximum rotated component 5% damped spectral accelerations. The deterministic response spectra are plotted on Plate 5.

5.2.7 Deterministic Lower Limit

ASCE 7-10, Section 21.2.2 specifies that the ordinates of the deterministic MCER ground motion response spectrum shall not be taken lower than the deterministic lower limits where:

SaM = 1.5Fa and SaM = 0.6(Fv/T), S_S = 1.5 and S_1 = 0.6

Per Tables 11.4-1 and 11.4-2 of ASCE 7-10, Site Class C, Fa = 1.00 and Fv = 1.30

The MCER deterministic lower limits using the above parameters and the 84th percentile deterministic site specific response spectrum adjusted using the deterministic lower limits are shown on Plate 5.

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13 Chiou, B. and Youngs, R. (2008), An NGA Model for the Average Horizontal Component of Peak Ground Motion and Response Spectra, Earthquake Spectra, 24:1, pp. 173-215.

14 Abrahamson, N. and Silva, W. (*2008*), Summary of the Abrahamson & Silva NGA Ground-Motion Relations. Earthquake Spectra: February 2008, Vol. 24, No. 1, pp. 67-97.

15 Wills, C.J., Silva, W. (1998), Shear-Wave Velocity Characteristics of Geologic Units in California, Earthquake Spectra, Volume 14, No. 3, 1998.

16 Brocher, T.M., (2005), Compressional and Shear Wave Velocity Versus Depth in the San Francisco Bay Area, California: Rules for USGS Bay Area Velocity Model 05.0.0, USGS Open-File Report 05–1317.

5.2.8 Probabilistic MCER Ground Motion

The probabilistic MCER ground motion was determined using the method in ASCE 7-10, Section 21.2.1.1. Estimates of the MCER probabilistic ground motion were computed using the software program EZ-Frisk (Version 7.65) developed by Risk Engineering. The analysis included active faults within 200 km of the site. Mean maximum rotated component acceleration values were computed using the same attenuation relationships and soil amplification as specified in the deterministic analysis above. The acceleration values from each of attenuation relationships were averaged using equal weight. The probabilistic MCE spectral acceleration values based upon our analysis are plotted on Plate 6.

As specified in ASCE 7-10, Section 21.2.1.1, the MCER ground motion was developed by adjusting the spectral acceleration values using the risk coefficients CRS (1.006) and CR1 (0.969) obtained from the USGS Ground Motion Parameter Application. The risk targeted MCER probabilistic spectrum based upon our analysis is plotted on Plate 7.

5.2.9 Design Response Spectrum (5% Damping)

As shown on Plate 8, the MCER deterministic spectrum is less than the probabilistic spectrum at all periods except 8 seconds. According to ASCE 7-10, Section 21.2.3, the lesser spectral values were used to construct the design spectrum. The site-specific design response spectrum is taken as 2/3 of the MCER spectral values. As shown on Plate 9, the site-specific design spectrum was adjusted such that values are greater than 80% of the general design spectrum and should be utilized for design (5% Damping).

5.2.10 Design Response Spectrum (0.5% Damping)

Plate 10 presents the adjustments to meet criteria in AWWA 100-11 and AWWA 110-13 for the Convective Component, 0.5% Damping. Scaling of the design response spectrum was performed to reflect the 0.5% damped response based on Damping Scaling Factors (DSFs) presented in AWWA 100-11 and AWWA 110-13, which use a DSF of 1.5 to scale from 5% to 0.5% damping.

5.2.11 Site-Specific MCE Geometric Mean (MCEG) Peak Ground Acceleration

Per ASCE 7-10, Section 21.5 the site-specific MCEG peak ground acceleration, PGAM, was taken as the lesser of the probabilistic geometric mean peak ground acceleration and the deterministic geometric mean peak ground acceleration. The site-specific MCEG peak ground acceleration should be greater than 80 percent of the general PGAM.

The probabilistic and deterministic seismic hazard analyses were performed using EZ-FRISK (Version 7.65) as described above using relationships without the maximum component option. Instead, the geometric mean values from the attenuation relationships were used.

5.2.12 Probabilistic MCEG Peak Ground Acceleration

The probabilistic geometric mean peak ground acceleration with a two percent probability of exceedance within a 50-year period was calculated to be 1.068g.

5.2.13 Deterministic MCEG Peak Ground Acceleration

The largest 84th percentile geometric mean peak ground acceleration for characteristic earthquakes on all known active faults within the site region was calculated as 0.902g. This value is greater than 0.5*FPGA, where FPGA=1.0 for a PGA=0.50g as stipulated in ASCE 7-10, Section 21.5.2. The controlling seismic source for the PGA is the Calaveras Fault (Mw=7.03).

5.2.14 Site-Specific MCEG Peak Ground Acceleration

The lesser value of the geometric mean probabilistic and deterministic peak ground accelerations is the deterministic value, which is 0.902g. This value is greater than 80 percent of the PGAM determined from ASCE 7-10, Section 11.8-1 (see Table 2 above). Therefore, 0.902g should be used as the PGA value for the project site.

5.3 Retaining Walls

5.3.1 Lateral Earth Pressures

Lateral earth pressures are presented in Table 3 below, and are expressed as equivalent fluid pressures (unit weights) in units of pcf. In addition to these earth pressures, the designer should add hydrostatic pressures behind the walls unless a drainage system is installed behind the walls. For cantilevered and soldier pile walls, a triangular lateral earth pressure distribution should be used for active and passive conditions. For tieback and soil nail walls, the lateral earth pressure distributions presented on Figure 5.5.5.7.2b-1 of the Caltrans Bridge Design Specifications (dated August 2004) should be used. For this figure, $P_a=(k_a)x(\gamma_s)x(H)$, where P_a is the maximum ordinate of pressure diagram in psf, k_a is the active lateral earth pressure coefficient presented in Table 3 below, γ_s is the total unit weight of soil in in pcf, and H is the wall design height in feet.

Table 3 – Recommended Lateral Earth Pressures for Walls Up to 15 feet in Height				
Description	**Level Backfill up to 6H:1V**[1]	**Sloped Backfill up to 3H:1V**[1]	**Sloped Backfill up to 2H:1V**[1]	**Sloped Backfill up to 1.7H:1V**[1,3]
Active Earth Pressure (flexible walls)[2]	45 pcf	50 pcf	60 pcf	70 pcf
Active Earth Pressure Coefficient (flexible walls)[2]	0.36	0.4	0.48	0.56
At-rest Earth Pressure (restrained walls)[2]	70 pcf	75 pcf	90 pcf	105 pcf

1. Horizontal to vertical
2. Expressed as an equivalent fluid pressure. Does not include hydrostatic pressures that might be caused by groundwater or water trapped behind the wall.
3. The gradient at the surface of the wall backfill should not exceed 2H:1V. However, we understand that steeper surface backfill gradients may be required near the northern and southern margins of the wall because the existing gradients in these areas are up to 1.7H:1V. If the zone having a backfill with a surface gradient steeper than 2H:1V is wider than about 2 feet, than that portion of the wall should be designed using the lateral earth pressures associated with a sloped backfill up to 1.7H:1V presented in this table.

5.3.2 Seismic Wall Pressures

According to Section 1803.5.12 of the 2013 California Building Code (CBC), dynamic seismic lateral earth pressures need to be included in the design of foundation walls and retaining walls supporting more than 6 feet of backfill height. We recommend using seismic pressures of 28H and 55H psf (where H is the height of the wall in feet) for flexible and restrained walls, respectively. A uniform rectangular pressure distribution with the resultant force acting at the mid-height of the wall may be used.

5.3.3 Wall Drainage

Retaining walls higher than 2 feet should be well-drained to reduce the potential for hydrostatic pressures to develop behind the walls. A typical drainage system for a cantilevered wall may consist of a 1- to 2-foot wide zone of Caltrans Class 2 Permeable material immediately behind the wall with a perforated pipe at the base of the wall discharging to a storm drain or other appropriate discharge facility via gravity flow. As an alternative, a prefabricated drainage board may be used in lieu of the Class 2 Permeable material. Where conditions allow for the use of weep holes, they may be used in lieu of the perforated pipe. The holes should be a minimum of 3 inches in diameter, and spaced at 8 feet or less on-center. Filter fabric or wire mesh should be placed over the holes at the backside of the wall to inhibit the permeable material, if used in lieu of a drainage board, from washing through the holes. **The drainage zone behind retaining walls should be capped with a minimum 12-inch thick layer of properly compacted onsite soil to reduce the risk of surface runoff discharging into the wall drain.**

Drains for soldier pile/tieback and soil nail walls typically consist of installing geocomposite strip drains behind the lagging or shotcrete facing of the wall at regular spacing (typically the horizontal spacing between nails). The bottom of the geocomposite drains are typically connected to a pipe that discharges into a collector pipe that in turn discharges to a storm drain system via gravity flow. Alternatively, the bottom of the geocomposite drains can be connected to weep holes similar to those described above.

5.3.4 Surcharge Loads

For surcharge loads imposed on the walls, a rectangular distribution with a uniform pressure equal to one-third of the surcharge pressure should be used for an unrestrained wall (active earth pressure condition). Surcharge loads caused by vehicular and/or construction traffic adjacent to the walls may be assumed to consist of a rectangular distributed uniform pressure of 100 psf acting over a depth of 10 feet below the ground surface. The wall designer should evaluate whether this surcharge is appropriate for the expected traffic loading. Additional analyses during design may be needed to evaluate the effects of non-uniform surcharge loads such as point loads, line loads, or other such presently undefined surcharge loads. In that case, we should be consulted for supplemental geotechnical recommendations.

5.3.5 Cantilevered Walls

Cantilevered retaining walls may be supported on continuous spread footings if they are founded on conglomerate bedrock that extends at least 7 feet laterally towards any nearby slopes. Otherwise, such walls should be supported on CIDH piers interconnected by a grade beam. The CIDH piers should be designed in accordance with the recommendations presented in the “Pier-Supported Mat Foundation” section of this report.

The footings should extend to at least 36 inches below finished subgrade. An allowable bearing pressure of 2,500 pounds per square feet (psf) may be used for dead and long term live loads. The allowable bearing pressure value may be increased by 1/3 for short term seismic and wind loads. Bearing capacity values include a factor of safety of at least 2. The footings should have a minimum width of 12 inches.

Concrete for the retaining wall footing should be placed neat against undisturbed conglomerate bedrock. It is important that the footing excavation not be allowed to dry before placing concrete. The excavation should be periodically moistened until concrete placement. During excavation of the footing, if the conglomerate bedrock exposed at the bottom of the excavation becomes disturbed, it should be properly compacted to a firm and stable condition. Refer to the "Earthwork" section of this report for compaction requirements.

Lateral loads may be resisted by a combination of friction between the bottom of the footings and the supporting subgrade and by passive resistance acting against the footing. The frictional and passive resistance may be assumed in design to act concurrently. An allowable friction coefficient of 0.40 between the bottom of the footing and supporting conglomerate subgrade may be used. For passive resistance, an allowable equivalent fluid pressure (unit weight) of 350 pcf may be used. The friction and passive values include factors of safety of about 1½.

Passive resistance in the upper foot of bedrock cover below finished grades should be neglected unless the ground surface is confined by concrete slabs, pavements, or other such positive protection.

5.3.6 Soldier Pile/Tieback Walls

Because the planned retaining wall will be permanent, we recommend that that timber lagging not be used due to its limited service life. For this reason, we recommend that the face of the walls be protected using precast concrete lagging, reinforced shotcrete, or similar long-term lagging. We anticipate the soldier piles will be pre-drilled and will be encased in concrete, and will behave like CIDH piers. For this reason, we expect the soldier piles will derive their load capacity from skin friction between the concrete and the surrounding conglomerate bedrock. Refer to the "Pier-supported Mat Foundation" section of the report for axial capacity and resistance to lateral load recommendations.

Tiebacks may be installed through the soldier piles. Tiebacks consist of the active reinforcement (i.e., post-tensioning/prestressing) of the soil behind the cut by installing closely spaced, near horizontal ground anchors that are subsequently encased in grout as the excavation proceeds. Typically, tiebacks are comprised of a steel tendon that is inserted into a hole drilled at an angle of between 15 to 30 degrees below a horizontal plane into the cut face and subsequently filled with concrete grout. While the entire length of soil nails are considered bonded to the surrounding soil/bedrock as discussed in the next section of this report, only a portion of the total length (referred to as the bonded length) of the tieback tendons is bonded to the surrounding soil/bedrock. Corrosion protection for the tiebacks and other corrosion sensitive components of the wall should be included in the wall design. The corrosivity test results we performed are summarized in the "Corrosion" section of this report and are presented at the end of Appendix B.

Based on our interpretation of the subsurface data presented in the 2008 CSA report and FHWA (1999)[17], we recommend that the tieback design parameters shown in Table 4 below be used for design of the tiebacks.

Table 4 – Tieback Design Parameters

Soil/Bedrock Unit	Total Unit Weight (pcf)[1]	Friction Angle (degrees)[1]	Cohesion (psf)[1]	Preliminary Ultimate Bond Stress[2] (psf)
Colluvium	125	25	100	500
Conglomerate	125	38	0	2,000

1. Based on our interpretation of the subsurface data presented in Cotton, Shires & Associates January 18, 2008 report.
2. Based on typical values provided in Table 7 of FHWA (1999). These values need to be confirmed and validated in the field via proof, creep, and performance tests during construction.

5.3.7 Soil Nail Walls

Soil nailing consists of the passive reinforcement (i.e., no post-tensioning) of the soil behind the cut by installing closely spaced, near horizontal soil nails that are subsequently encased in grout as excavation proceeds. Typically, soil nails are comprised of a steel tendon that is inserted into a hole drilled at an angle of between 10 to 20 degrees below a horizontal plane into the cut face and subsequently filled with concrete grout. The entire length of soil nails are considered bonded to the surrounding soil/bedrock. Following excavation and nail installation, a protective facing is applied to the cut to prevent erosion. Corrosion protection for the soil nail steel tendons and other corrosion sensitive components of the wall should be included in the wall design. The corrosivity test results we performed are summarized in the "Corrosion" section of this report and are presented at the end of Appendix B.

[17] Federal Highway Administration (1999), Geotechnical Circular No. 4 – Ground Anchors and Anchored Systems, FHWA-IF-99-015, dated June 1999.

Based on our interpretation of the subsurface data presented in the 2008 CSA report and FHWA (2015)[18], we recommend that the soil nail design parameters shown in Table 5 below be used for design of the soil nails.

Table 5 – Soil Nail Design Parameters

Soil/Bedrock Unit	Total Unit Weight (pcf)[1]	Friction Angle (degrees)[1]	Cohesion (psf)[1]	Preliminary Ultimate Bond Strength[2] (psf)
Colluvium	125	25	100	500
Conglomerate	125	38	0	2,000

1. Based on our interpretation of the subsurface data presented in Cotton, Shires & Associates January 18, 2008 report.
2. Based on typical values provided in Tables 4.4a and 4.4b of FHWA (2015). These values need to be confirmed and validated in the field via verification, proof, and creep tests during construction.

5.3.8 Design and Testing of Soldier Pile/Tieback or Soil Nail Walls

Due to the level of detail required in the design of soldier pile/tieback and soil nail walls, we recommend that these types of walls be design-build by a specialty contractor based on the requirements of FHWA (1999, 2015). The contractor selected should provide proof of at least 5 years of continuous experience designing and constructing these types of walls under similar subsurface conditions and accessibility constraints as those found at the project site. The design-build contractor selected should determine whether additional subsurface investigation beyond that presented in 2008 CSA report is necessary to design and construct these walls. Prior to the start of construction, the geotechnical aspects of the design calculations, plans, and specifications for the wall should be reviewed by BSK Associates. However, such review should not be construed as relieving the design-build contractor from having full responsibility of the design and construction of the wall.

The preliminary ultimate bond stress/strength values indicated above for tiebacks and soil nails were derived based on past experience with similar subsurface conditions and typical values provide in FHWA (1999, 2015). These preliminary design values should be confirmed and validated during construction.

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[18] Federal Highway Administration (2015), Geotechnical Engineering Circular No. 7 – Soil Nail Walls – Reference Manual, FHWA-NHI-14-007, dated February 2015.

The wall designer will be responsible for designing the actual tiebacks/soil nails and their horizontal/vertical layout/spacing, design of soldier piles (if used), design of wall protective facing, and developing acceptance criteria for proof, creep, and performance testing (for tiebacks) and verification, proof, and creep testing (for soil nails) during construction. The wall design should be modified as needed during construction based on the testing, such as adding more tiebacks/soil nails, decreasing the spacing between soil nails, adding more rows of tiebacks, etc.

For tiebacks, performance testing should be performed on at least the first two anchors installed for each soil/bedrock unit and on a minimum of 2 percent of the remaining production tiebacks thereafter. The remaining tiebacks that are not performance tested should be proof tested. All tiebacks should be creep tested.

For soil nail walls, at least 2 verification tests should be performed for each soil/bedrock unit and at least 5 percent of the production soil nails should be proof tested. All soil nails that undergo verification and proof testing should be creep tested.

5.3.9 MSE/Keystone-Type Walls

An MSE or keystone-type wall may <u>only</u> be used along the eastern margins of the planned asphalt paved driveway. Note that there would be a risk that future shallow landsliding downslope of this area could undermine the base of such walls.

The MSE or keystone-type wall should be designed to resist the lateral earth pressures provided in Table 3 above. Based on the our experience and the laboratory testing performed for the tank site, we estimate an internal friction angle (phi) of about 25 degrees, a cohesion (C) of about 100 psf, and a moist unit weight of approximately 125 pounds per cubic feet (pcf) for the backfill behind this the wall. These soil properties may be used in the design of the wall provided it is backfilled with properly compacted and moisture conditioned onsite soils per the recommendations contained in the “Earthwork” section of this report. The MSE/keystone-type wall designer should review and evaluate whether these soils are suitable for the design of the wall.

Portions of the wall higher than 2 feet should be well-drained to reduce hydrostatic pressure. A typical drainage system consists of a 1- to 2-foot wide zone of Caltrans Class 2 Permeable material immediately <u>behind</u> the reinforced soil mass with a perforated pipe at the base of the wall discharging to a storm drain or other appropriate discharge facility. As an alternative, a prefabricated drainage system may be used in lieu of the Class 2 Permeable material.

5.3.10 Construction Observation and Testing

Retaining wall construction should be monitored by a representative of BSK during construction, including periodic observation by our engineering geologist. Depending on the type(s) of retaining wall constructed, the purpose of such observation would include some of the following:

- Observe temporary back cut and check for signs of instability;
- Check bottom conditions of footing excavation prior to placing steel reinforcement and concrete, including confirming that the subsurface conditions encountered are consistent with our recommendations, the adequacy of the supporting materials exposed, and moisture control;
- Check the overall foundation dimensions against the project plans and our recommendations;
- Perform compaction testing of the bottom of the footing excavations and retaining wall backfill;
- Observe geogrid placement; and
- Observe load testing of tiebacks or soil nail tendons.

5.4 Exterior Flatwork

Exterior concrete flatwork at grade will be constructed on soils subject to swell/shrink cycles. Some of the adverse effects of swelling and shrinking can be reduced with proper moisture treatment. The intent is to reduce the fluctuations in moisture content by moisture conditioning the soils, sealing the moisture in, and controlling it. Near-surface soils to receive exterior concrete flatwork should be moisture conditioned according to the recommendations in the “Earthwork” section of this report. In addition, all exterior flatwork should be supported on a minimum of 6 inches of "non-expansive" fill. Where concrete flatwork is to be exposed to vehicle traffic, the 6 inches of "non-expansive" fill should consist of Caltrans Class 2 aggregate base meeting the requirements of Section 26 of the 2015 Caltrans Standard Specifications.

Exterior flatwork will be subjected to edge effects due to the drying out of subgrade soils. To protect against edge effects adjacent to unprotected areas, such as undeveloped areas of the tank, lateral cutoffs consisting of inverted curbs, water barrier, or similar are recommended. Cutoffs should extend a minimum of 2 inches below the “non-expansive” section.

Due the presence of moderately expansive soils near the site surface, flatwork should have control joints (i.e., weakened plane joints) spaced no more than 8 feet on centers. Prior to construction of the flatwork, the 6 inches of "non-expansive" fill should be moisture conditioned to near optimum moisture content. If the "non-expansive" fill is not covered within about 30 days after placement, the soils below this material will need to be checked to confirm that their moisture content is at least 2 percent over optimum. If the moisture is found to be below this level, the flatwork areas will need to be soaked until the proper moisture content is reached. Where flatwork is adjacent to curbs, reinforcing bars should be placed between the flatwork and the curbs. Expansion joint material should be used between flatwork and buildings.

5.5 Demolition

5.5.1 Existing Utilities

Active or inactive utilities within the construction area should be protected, relocated, or abandoned. Pipelines that are 2 inches in diameter or less may be left in place beneath the tank foundations provided they are cut off and capped at the foundation perimeters. Pipelines larger than 2 inches in diameter within the planned tank limits should be removed or filled with a 1-sack sand-cement slurry mix. Active utilities to be reused should be carefully located and protected during demolition and during construction.

5.5.2 Excavation and Backfill of Existing Foundations and Below-Grade Structures

All existing foundations and below-grade structures to be abandoned should be demolished and removed. The resulting excavations should then be properly backfilled with compacted engineered fill per the requirements of the “Earthwork” section of this report especially within areas underneath and extending within 5 feet laterally from the new tank limits. A BSK representative should observe and test the compaction of for earthwork activities during construction.

5.5.3 Reuse of Onsite Concrete

Although we find it unlikely that this would be cost effective for this project due to the relatively small amount of concrete present at the tank site, existing concrete may be pulverized for use as general engineered fill onsite if it meets the gradation requirements discussed in the “Re-Use of Onsite Soils and Imported Fill Material” section of this report.

5.6 Earthwork

Earthwork at the site will generally consist of subgrade preparation and placement of "non-expansive" fill and aggregate base for exterior flatwork and pavements, excavation, removal, and backfill of existing tank foundations, retaining walls, and underground utility lines, excavation of new tank foundations, retaining wall excavation and backfill, cut slope excavation, and excavation and backfill of new underground utility lines. We anticipate that the required grading within the limits of the existing tanks and gravel driveway leading to the tanks will consist of cuts of 2 feet deep or less and fills less than 1 foot high. Cuts up to about 20 feet deep are expected for the planned retaining wall, the planned cut slope behind it, and portions of the new tank(s) along the west (upslope) side of the tank site. Existing and new underground utility lines are expected to be up to 5 feet deep. BSK should review the final grading plans for conformance to our design recommendations prior to construction bidding. In addition, it is important that a representative of BSK observe and evaluate the adequacy of the supporting materials exposed under structures, concrete flatwork, and pavements. In general, soft/loose or unsuitable materials encountered should be overexcavated, removed, and replaced with compacted engineered fill material.

Site preparation and grading for this project should be performed in accordance with the site-specific recommendations provided below. A summary of compaction requirements for this project is presented in Exhibit 1 in Appendix C. Additional earthwork recommendations are presented in related sections of this report.

5.6.1 Site Preparation and Grading

Prior to the start of grading and subgrade preparation operations, the site should first be cleared and stripped to remove all surface vegetation, organic laden topsoil and debris generated during the demolition of existing tank foundations, retaining walls, and underground utilities within the site. Stripped topsoil may be stockpiled for later use in landscaping areas; however, this material should not be reused for engineered fill.

Any buried tree stumps, roots, or major root systems thicker than approximately 1-inch in diameter, abandoned foundations, septic tanks and leach field lines, uncovered during site stripping and/or grading activities should be removed. Unit prices for removal of such material should be obtained during bidding.

Following stripping and removal of deleterious materials, the site should be scarified to a minimum depth of 12 inches, moisture conditioned, and recompacted as indicated in Appendix

C, Exhibit 1. Scarification and recompaction should extend laterally a minimum of 5 feet beyond the limits of structures (defined as the outside perimeter of tank walls or foundation outer limits, whichever results in the greatest structure envelope) and 3 feet beyond the edge of flatwork and pavements, where achievable. All fills should be compacted in lifts of 8-inch maximum uncompacted thickness. A summary of compaction requirements for the project is presented in Exhibit 1. Laboratory maximum dry density and optimum moisture content relationships should be evaluated based on ASTM Test Designation D1557 (latest edition).

Due to the moderate expansion potential of the near-surface soils at the site, proper moisture conditioning is very important. After subgrade soils are properly moisture conditioned, their moisture content should be maintained until they are covered by improvements. This may require periodic moisturizing of the subgrade soils.

All site preparation and fill placement should be observed by a BSK representative. It is important that, during the stripping and scarification process, our representative be present to observe whether any undesirable material is encountered in the construction area and whether exposed soils are similar to those encountered during the 2008 CSA subsurface investigation.

5.6.2 Re-Use of Onsite Soil and Imported Fill Material

The onsite soils and conglomerate bedrock are suitable for re-use as general engineered fill and backfill provided vegetation, organic materials, and deleterious matter are removed. A BSK representative should be present onsite during grading to visually confirm the suitability of the soil to be used as fill and backfill. Particles larger than 3 inches within the onsite soils and conglomerate (if encountered) should either be removed and disposed offsite or broken down to 3 inches or less prior to using the soil as engineered fill. Nesting (i.e., concentration) of larger particles should be avoided to reduce the potential that this could create voids and allow future settlement in the overlying fill/backfill.

Maximum particle size for fill material should be limited to 3 inches, with at least 90 percent by weight passing the 1-inch sieve. Proper granular bedding and shading should be used beneath and around new utilities. Where imported “non-expansive” fill is required, it should be granular in nature, adhere to the above gradation recommendations, and conform to the minimum criteria presented in Table 6 below.

Table 6 - "Non-Expansive" Fill Criteria	
Plasticity Index	15 or less
Liquid Limit	Less than 30%
% Passing #200 Sieve	8 % – 40%

Highly pervious materials such as pea gravel or clean sands are not recommended because they permit transmission of water to the underlying soils. Imported fill material should not be any more corrosive than the onsite soils and should not be classified as being more corrosive than "moderately corrosive." Prior to transporting proposed imported materials to the site, the contractor should make representative samples of the material available to the Geotechnical Engineer-of-Record at least 10 working days in advance to allow the engineer enough time to confirm the material meets the above requirements. All onsite or imported fill material should be compacted to the recommendations provided for engineered fill in Exhibit 1.

5.6.3 Volume Change of Excavated/Compacted Soils

We anticipate on the order of a 20 to 30 percent volume increase of soil that is excavated from cuts at the site and subsequently transported offsite. If the material is compacted onsite or at its offsite destination to a minimum of 90 percent compaction (assuming ASTM D1557, latest edition), then we expect a volume change of on the order of +/- 10 percent from the original insitu volume. As an example, if 100 cubic yards are excavated, it could result in on the order of 120 to 130 cubic yards during transportation. If the material is in turn compacted to 90 percent compaction at the destination, it could result in 90 to 110 cubic yards of compacted material. Note that the above estimates do not take into consideration volume loss associated with the removal of gravel/cobbles/boulders larger than 3 inches, roots, and organic matter from the excavated material.

5.6.4 Weather/Moisture Considerations

If earthwork operations and construction for this project are scheduled to be performed during the rainy season (usually November to May) or in areas containing saturated soils, provisions may be required for drying of soil or providing admixtures, such as lime-treatment, to the soil prior to compaction. Conversely, additional moisture may be required during dry months. Water trucks should be made available in sufficient numbers to provided adequate water during earthwork operations.

5.6.5 Excavation and Backfill

We anticipate that excavations for the cut slope, foundations, retaining walls, and utility trenches can be made with heavy-duty excavators, dozers, or similar earthwork equipment. Excavations extending into hard conglomerate bedrock may require the use of hydraulic hammers and similar equipment to break up and excavate the rock. Where trenches or other excavations are extended deeper than 5 feet, the excavation may become unstable and should be evaluated to monitor stability prior to personnel entering the trenches. Shoring or sloping of any trench wall may be necessary to protect personnel and to provide stability. All trenches and excavations should conform to the current OSHA requirements for work safety. It is the contractor's responsibility to follow OSHA temporary excavation guidelines and grade the slopes with adequate layback or provide adequate shoring and underpinning of existing structures and improvements, as needed. Slope layback and/or shoring measures should be adjusted as necessary in the field to suit the actual conditions encountered, in order to protect personnel and equipment within excavations.

Care should be taken during construction to reduce the impact of trenching on adjacent structures and pavements (if applicable). Excavations should be located so that no structures, foundations, and slabs, existing or new, are located above a plane projected 1H:1V (horizontal to vertical) upward from any point in an excavation, regardless of whether it is shored or unshored.

Free groundwater was observed at a depth of 47 BGS in boring CSA/SD-4. However, the actual depth at which groundwater may be encountered in trenches and excavations may vary. As a minimum, provisions should be made to ensure that conventional sump pumps used in typical trenching and excavation projects are available during construction in case substantial runoff water accumulates within the excavations as a result of wet weather conditions.

Backfill for trenches and other small excavations beneath flatwork should be compacted as noted in Exhibit 1. Special care should be taken in the control of utility trench backfilling under structures and flatwork areas. Poor compaction may cause excessive settlements resulting in damage to overlying structures and flatwork.

Utility trenches located in landscaped areas should be capped with a minimum of 12 inches of compacted onsite top soils.

5.7 Asphalt Pavement

Pavements for this project will consist of the new asphalt concrete driveway for the tank site. We have made our pavement designs assuming the pavement subgrade soil will be similar to the near surface soils described in the boring logs. The near surface soils at the tank site appear to be moderately expansive and are therefore expected to have a low Resistance (R) Value. We ran R-Value testing on a sample collected from the upper 1 foot at sampling location S-1, which resulted in an R-Value of 27. Due to the potential variability of the fines content contained in the surficial soils at the tank site, we recommend using an R-Value of 10 for design of the asphalt concrete pavement section(s) for the project.

Pavement designs for various Traffic Indices (TIs) based on an R-Value of 10 are presented in Table 7 below. Each TI represents a different level of use. The owner or designer should determine which level of use best reflects the project and select appropriate pavement sections. The recommended pavement sections are presented in the table below and include a factor of safety of 0.2 feet as per the Caltrans Design Manual.

Table 7 – Pavement Design Recommendations (R-Value = 10)

Traffic Index	AC[1] (inches)	Class 2 AB[2] (inches)
5.0	2.5	10
5.5	3	11
6.0	3	12.5

1. Asphalt Concrete
2. Caltrans Class 2 Aggregate Base (Minimum R-Value = 78)

We recommend that the subgrade soil over which the pavement sections are to be placed be moisture conditioned and compacted according to the recommendations in Exhibit 1. Subgrade preparation should extend a minimum of 3 feet laterally beyond the back of curb or edge of pavement, where achievable.

Paved areas should be sloped and drainage gradients maintained to carry all surface water to appropriate collection points. Surface water ponding should not be allowed anywhere on the tank site during or after construction. We recommend that the pavement section be isolated from non-developed areas and areas of intrusion of irrigation water from landscaped areas. Concrete curbs should extend a minimum of 2 inches below the aggregate base and into the subgrade to provide a barrier against drying of the subgrade soils, or reduction of migration of landscape water, into the pavement section. Weep holes spaced at 4 feet on centers should also be provided. In lieu of the weep holes, a more effective system is to install a subdrain behind the curbs.

In addition, we recommend that all pavements conform to the following criteria:

- All trench backfills underneath pavements, including utility and sprinkler lines, should be properly placed and adequately compacted to provide a stable subgrade, in accordance with the compaction recommendations in Exhibit 1;
- An adequate drainage system should be provided to prevent surface water or subsurface seepage from saturating the subgrade soil;
- The asphalt concrete, aggregate base, and aggregate subbase materials should conform to Caltrans Specifications, latest edition; and
- Placement and compaction of pavements should be performed in accordance to appropriate Caltrans procedures.

5.8 Storm Water Infiltration

Storm runoff regulations require pretreatment of runoff and infiltration of storm water to the extent feasible. Typically, this results in the use of bioretention areas, vegetated swales, infiltration trenches, or permeable pavement near or within parking lots and at the location of roof run-off collection. These features are not well suited to fine-grained soils (silts and clays) because these soils have relatively low permeability and require significant time for infiltration to occur. In addition, allowing water to pond on expansive soils will cause the soils to swell, which can cause distress to adjacent pavements, slabs, and lightly loaded structures. Also, allowing water to pond near the near the toe and crest of a slope could cause slope instability and failure. **Therefore, we recommend that storm water infiltration be <u>excluded</u> from the design of this project due to the susceptibility of the tank site to shallow landsliding. Pervious pavements should also be <u>excluded</u> from the design of the project.**

5.9 Corrosion

A soil sample was collected during our field investigation at a depth of approximately 0 to 1 foot BGS at sampling location S-2 and was submitted for corrosion testing. The sample was tested by CERCO Analytical, a State-certified laboratory in Concord, California, for redox potential, pH, resistivity, chloride content, and sulfate content in accordance with ASTM test methods. The test results are presented at the end of Appendix B. Also included is the evaluation by CERCO Analytical of the corrosion test results. Because we are not corrosion specialists, we recommend that a corrosion specialist be consulted for advice on proper corrosion protection for underground piping which will be in contact with the soils and other design details.

Based upon the resistivity measurements, the sample tested is classified as "moderately corrosive" by CERCO Analytical. They recommend that all buried iron, steel, cast iron, ductile iron, galvanized steel, and dielectric coated steel or iron be properly protected against corrosion depending upon the critical nature of the structure. They also recommend all buried metallic pressure piping, such as ductile iron firewater pipelines, should be protected against corrosion.

The above are general discussions. A more detailed investigation may include more or fewer concerns, and should be directed by a corrosion expert. BSK does not practice corrosion engineering. Consideration should also be given to soils in contact with concrete that will be imported to the tank site during construction, such as topsoil and landscaping materials. For instance, any imported soil materials should not be any more corrosive than the onsite soils and should not be classified as being more corrosive than "moderately corrosive." Also, onsite cutting and filling may result in soils contacting concrete that were not anticipated at the time of this investigation.

5.10 Plan Review and Construction Observation

We recommend that BSK be retained by the Client to review the final foundation and grading plans and specifications before they go out to bid. It has been our experience that this review provides an opportunity to detect misinterpretation or misunderstandings of our recommendation prior to the start of construction.

Variations in soil types and conditions are possible and may be encountered during construction. To permit correlation between the soil data obtained during this investigation and the actual soil conditions encountered during construction, we recommend that BSK be retained to provide observation and testing services during site earthwork and foundation construction. This will allow us the opportunity to compare actual conditions exposed during construction with those encountered in the previous investigations/studies and our April 6, 2016 geologic site reconnaissance and to provide supplemental recommendations if warranted by the exposed conditions. Earthwork should be performed in accordance with the recommendations presented in this report, or as recommended by BSK during construction. BSK should be notified at least two weeks prior to the start of construction and prior to when observation and testing services are needed.

6. ADDITIONAL SERVICES AND LIMITATIONS

6.1 Additional Services

The review of plans and specifications, and field observation and testing during construction by BSK are an integral part of the conclusions and recommendations made in this report. If BSK is not retained for these services, the client will be assuming BSK's responsibility for any potential claims that may arise during or after construction due to the misinterpretation of the recommendations presented herein. The recommended tests, observations, and consultation by BSK during construction include, but are not limited to:

- review of plans and specifications;
- observations of site grading, including stripping and engineered fill placement;
- observation of cut slope and back cut excavation by a qualified engineering geologist;
- observation of retaining wall construction and load testing (if applicable);
- observation of foundation excavations; and
- in-place density testing of fills, backfills, and finished subgrades.

6.2 Limitations

The recommendations contained in this report are based on our field observations and previous subsurface explorations, current and previous laboratory tests, review of available geologic maps and publications, review of previous studies for the tank site, and our present knowledge of the proposed construction. It is possible that soil and bedrock conditions could vary between or beyond the points explored. If soil and bedrock conditions are encountered during construction that differ from those described herein, we should be notified immediately in order that a review may be made and any supplemental recommendations provided. If the scope of the proposed construction, including the proposed loads or structural locations, changes from that described in this report, our recommendations should also be reviewed.

We prepared this report in substantial accordance with the generally accepted geotechnical engineering practice as it exists in the tank site area at the time of our study. No warranty, either express or implied, is made. The recommendations provided in this report are based on the assumption that an adequate program of tests and observations will be conducted by BSK during the construction phase in order to evaluate compliance with our recommendations. Other standards or documents referenced in any given standard cited in this report, or otherwise relied upon by the author of this report, are only mentioned in the given standard;

they are not incorporated into it or "included by reference", as that latter term is used relative to contracts or other matters of law.

This report may be used only by the Client and only for the purposes stated within a reasonable time from its issuance, but in no event later than two (2) years from the date of the report, or if conditions at the tank site have changed. If this report is used beyond this period, BSK should be contacted to evaluate whether site conditions have changed since the report was issued.

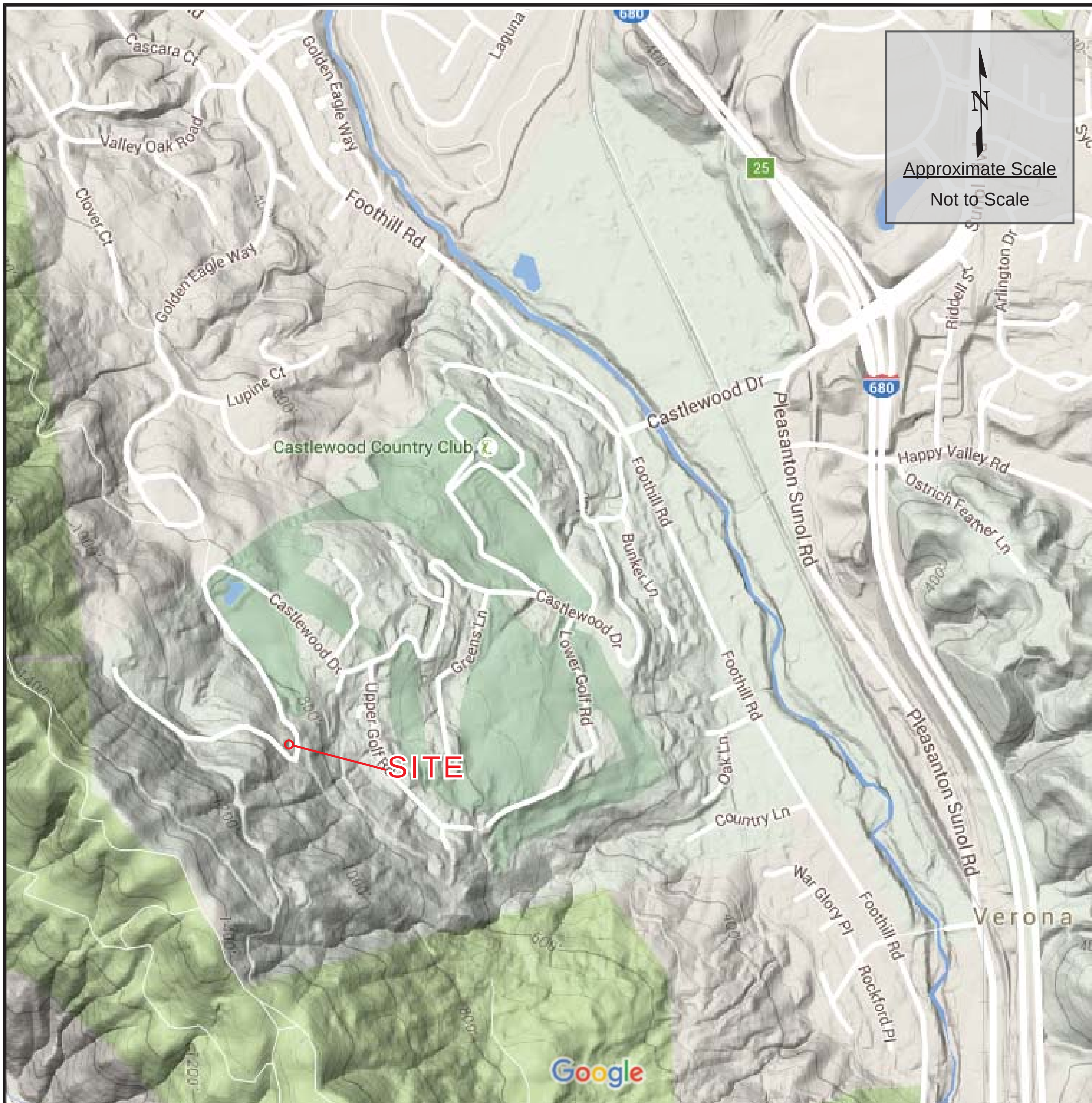
Also, land or facility use, on and off-site conditions, regulations, or other factors may change over time, and additional work may be required with the passage of time. Based on the intended use of the report, BSK may recommend that additional work be performed and that an updated report be issued.

The scope of work for this study and geotechnical report did not include environmental assessments or evaluations regarding the presence or absence of wetlands at this site.

BSK provided recommendations for this project based on the subsurface exploration by others. We understand that BSK will be given the opportunity to perform a formal geotechnical review of the final project plans and specifications. In the event BSK is not retained to review the final project plans and specifications to evaluate if our recommendations have been properly interpreted, we will assume no responsibility for misinterpretation of our recommendations.

We recommend that all earthwork during construction be monitored by a representative from BSK, including site preparation, cut slope and back cut excavation, retaining wall construction and load testing (if applicable), foundation excavation, placement of engineered fill, and trench backfill. The purpose of these services would be to provide BSK the opportunity to observe the actual soil and bedrock conditions encountered during construction, evaluate the applicability of the recommendations presented in this report to the soil and bedrock conditions encountered, and recommend appropriate changes in design or construction procedures if conditions differ from those described herein.

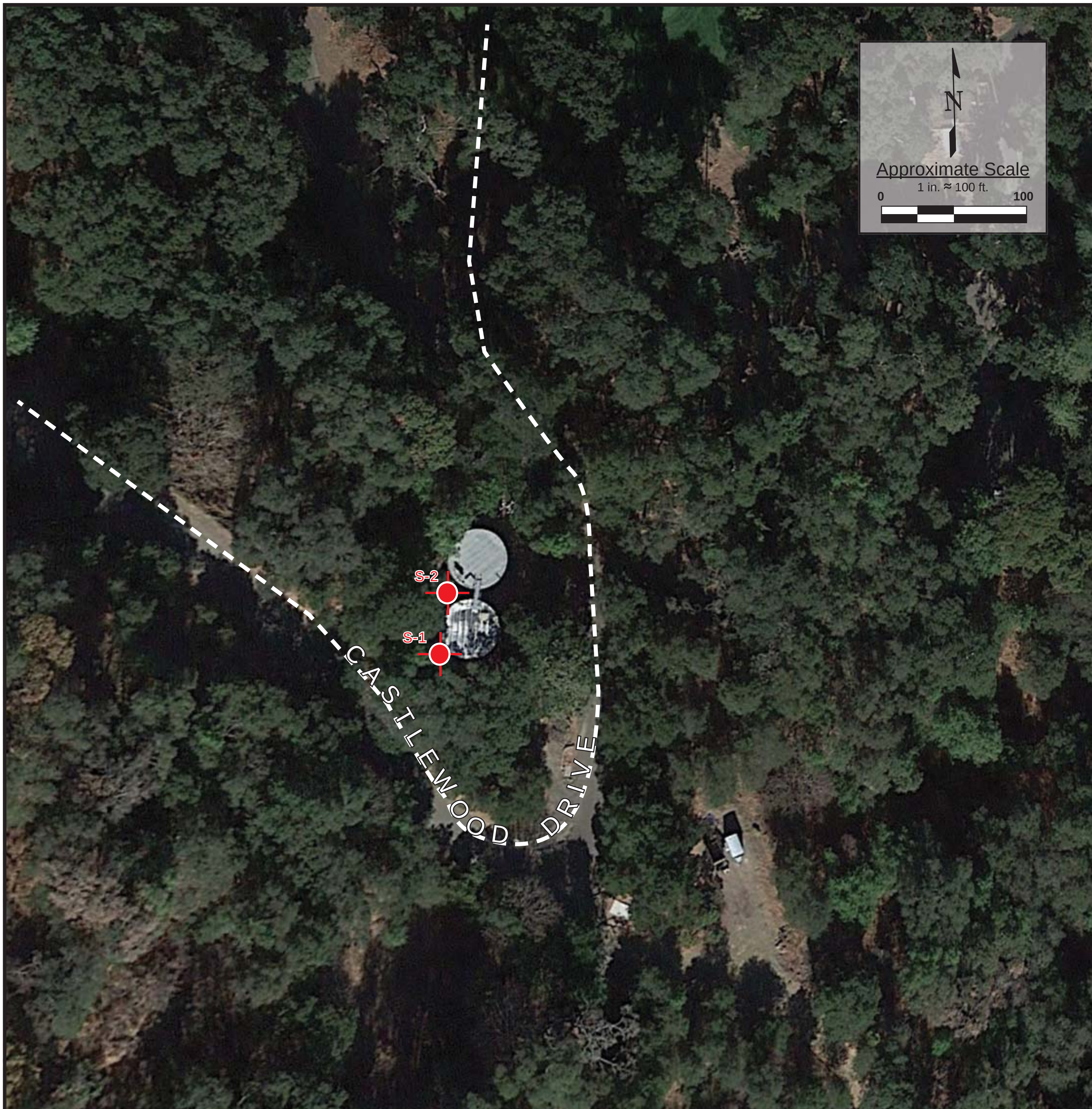
PLATES



References: 1. https://www.google.com/maps, 2016

The information included on this graphic representation has been compiled from a variety of sources and is subject to change without notice. BSK makes no representations or warranties, express or implied, as to accuracy, completeness, timeliness, or rights to the use of such information. This document is not intended for use as a land survey product nor is it designed or intended as a construction design document. The use or misuse of the information contained on this graphic representation is at the sole risk of the party using or misusing the information.

BSK ASSOCIATES			
	PROJECT NO. G16-062-11L	VICINITY MAP	PLATE
	DRAWN: 06/08/16		1
	DRAWN BY: D. Tower		
	CHECKED BY: C. Melo	Castlewood Redwood Tanks Replacement Pleasanton, California	
	FILE NAME: SitePlan.indd		



Reference: http://earth.google.com, 2016

Note: Locations are Approximate

Legend

S-1 Surficial Sample Locations (by BSK, 2016)

BSK ASSOCIATES

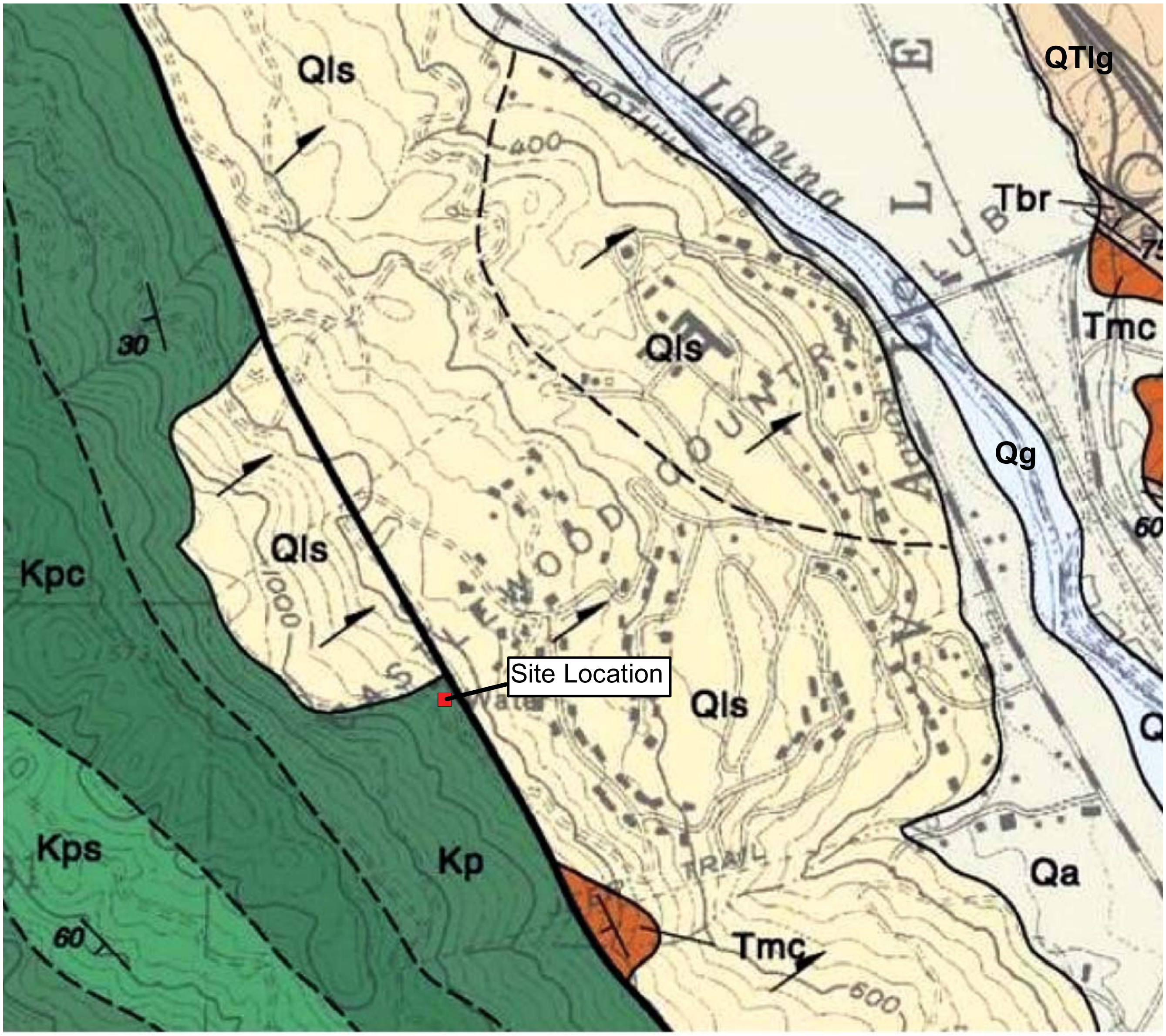
PROJECT NO.	G16-062-11L
DRAWN:	06/08/16
DRAWN BY:	D. Tower
CHECKED BY:	C. Melo
FILE NAME:	SitePlan.indd

SITE PLAN

Castlewood Redwood Tanks Replacement
Pleasanton, California

PLATE

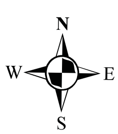
2



Reference: Dibblee, T.W., and Minch, J.A. (2005), Geologic Map of the Dublin Quadrangle, Contra Costa and Alameda Countries, California: Dibblee Geological Foundation, Dibblee Foundation Map DF-164, Scale 1:24,000.

LEGEND

Qa - Alluvial gravel, sand and clay of valley areas (Holocene)
Qg - Alluvial gravel, sand and clay of Arroyo Laguna (Holocene)
Qls - Landslide Rubble (Holocene/Pleistocene)
QTlg - Livermore Gravel (Pleistocene)
Tmc - Monterey Formation (Miocene)
Tbr - Briones Formation (Miocene) -- Sandstone, siltstone, conglomerate and shell breccia
Kp - Panoche Formation Claystone (Cretaceous)
Kpc - Panoche Formation Conglomerate (Cretaceous)
Kps - Panoche Formation Sandstone (Cretaceous)



BSK ASSOCIATES

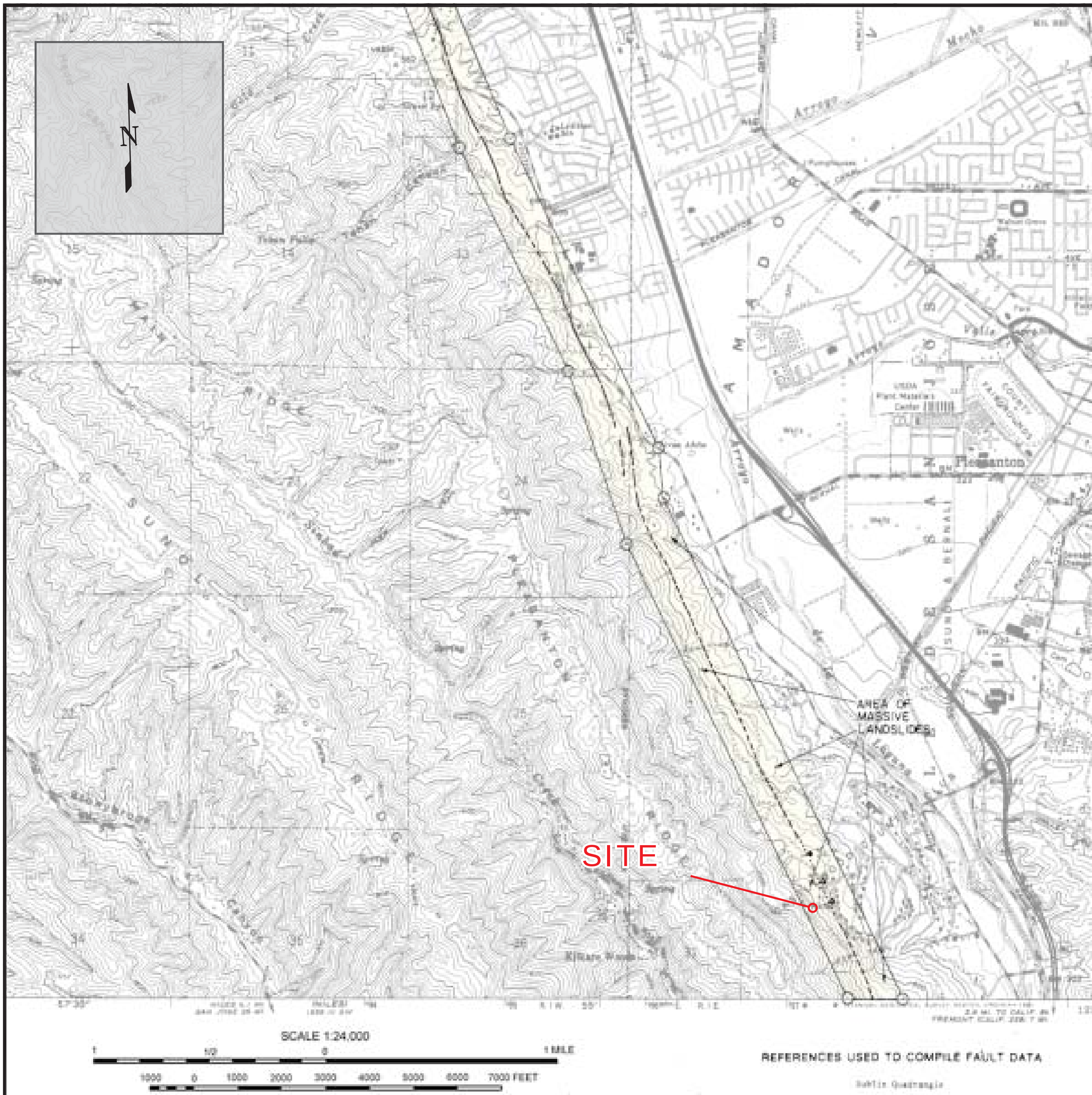
PROJECT NO.	G16-062-11L
DRAWN:	06/08/16
DRAWN BY:	D. Tower
CHECKED BY:	C. Melo
FILE NAME:	SitePlan.indd

GEOLOGIC MAP

Castlewood Redwood Tanks Replacement
Pleasanton, California

PLATE

3

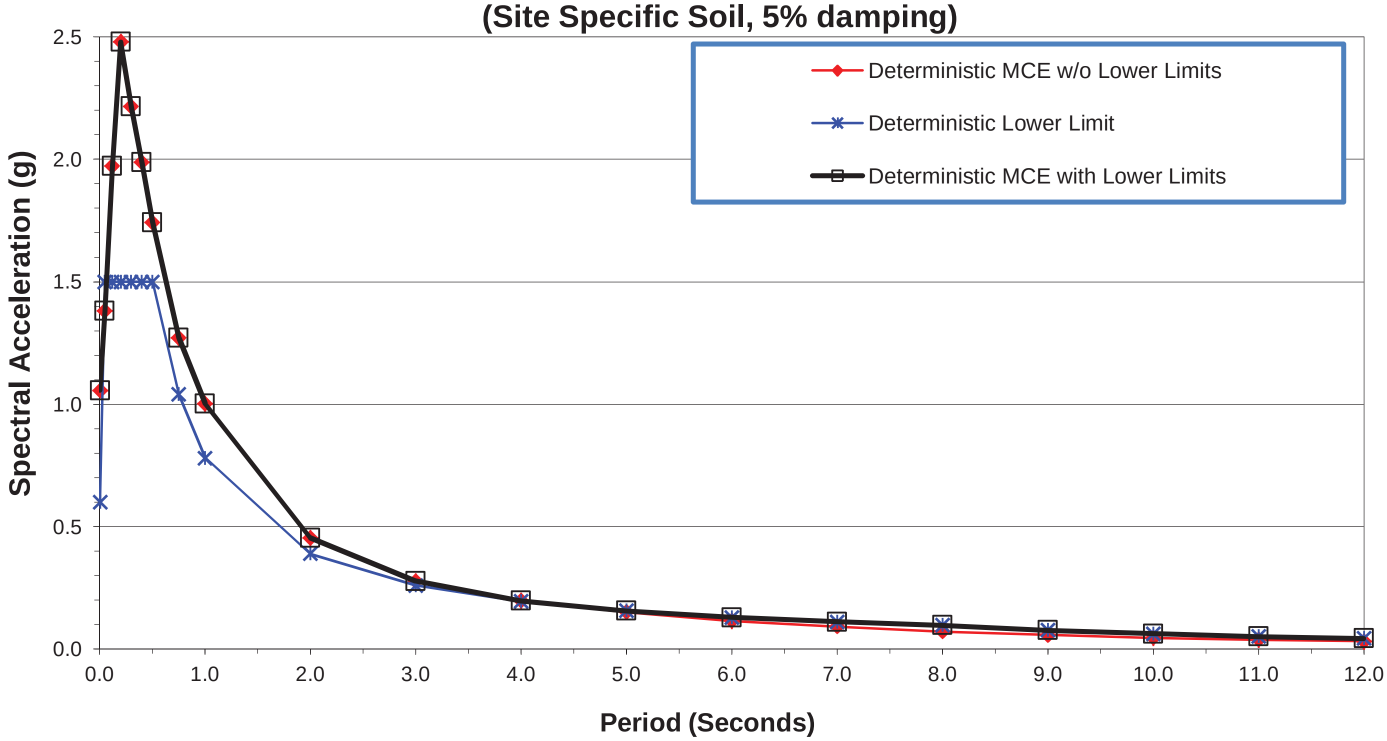


Reference: California Division of Mines and Geology (1982),Special Studies Zones, Dublin Quadrangle, January 1, 1982

BSK ASSOCIATES

PROJECT NO. G16-062-11L	FAULT ZONE MAP	**PLATE** 4
DRAWN: 06/08/16		
DRAWN BY: D. Tower		
CHECKED BY: C. Melo	**Castlewood Redwood Tanks Replacement Pleasanton, California**	
FILE NAME: LabPlates.indd		

Deterministic Response Spectra
Maximum Considered Earthquake
Maximum Rotated Horizontal Component
(Site Specific Soil, 5% damping)



				Attenuation Relationship			
Period (Second)	Deterministic MCE with Lower Limits (g)	Deterministic MCE w/o Lower Limits (g)	Sa (Median) (g)	Sa (BA08) (g)	Sa (CB08) (g)	Sa (CY08) (g)	Sa (AS08) (g)
Calaveras Fault, California Gridded 0.2 sec. To 0.4 sec.							
PGA	1.057	1.057	1.057	1.085	0.938	1.155	1.096
0.1	1.380	1.380	1.380	1.384	1.274	1.553	1.353
0.1	1.973	1.973	1.973	2.013	1.650	2.326	1.953
0.2	2.479	2.479	2.479	2.621	2.286	2.495	2.512
0.3	2.216	2.216	2.216	2.367	1.930	2.213	2.355
0.4	1.988	1.988	1.988	2.216	1.726	1.956	2.053
0.5	1.743	1.743	1.743	1.793	1.585	1.972	1.724
0.8	1.271	1.271	1.271	1.308	1.186	1.464	1.204
1.0	1.002	1.002	1.002	1.034	0.932	1.166	0.919
2.0	0.455	0.455	0.455	0.531	0.441	0.500	0.360
3.0	0.278	0.278	0.278	0.326	0.285	0.294	0.213
4.0	0.198	0.198	0.198	0.232	0.216	0.201	0.145
5.0	0.156	0.151	0.151	0.177	0.183	0.143	0.104
6.0	0.130	0.116	0.116	0.136	0.141	0.105	0.082
7.0	0.111	0.092	0.092	0.110	0.112	0.079	0.067
8.0	0.098	0.072	0.072	0.083	0.093	0.061	0.052
9.0	0.077	0.057	0.057	0.060	0.079	0.048	0.042
10.0	0.062	0.046	0.046	0.045	0.069	0.038	0.034
11.0	0.052	0.038	0.038	0.037	0.057	0.031	0.028
12.0	0.043	0.032	0.032	0.031	0.048	0.026	0.024

BSK ASSOCIATES

PROJECT NO.	G16-062-11L
DRAWN:	05/03/16
DRAWN BY:	M. Cline
CHECKED BY:	C. Melo
FILE NAME:	SitePlan.indd

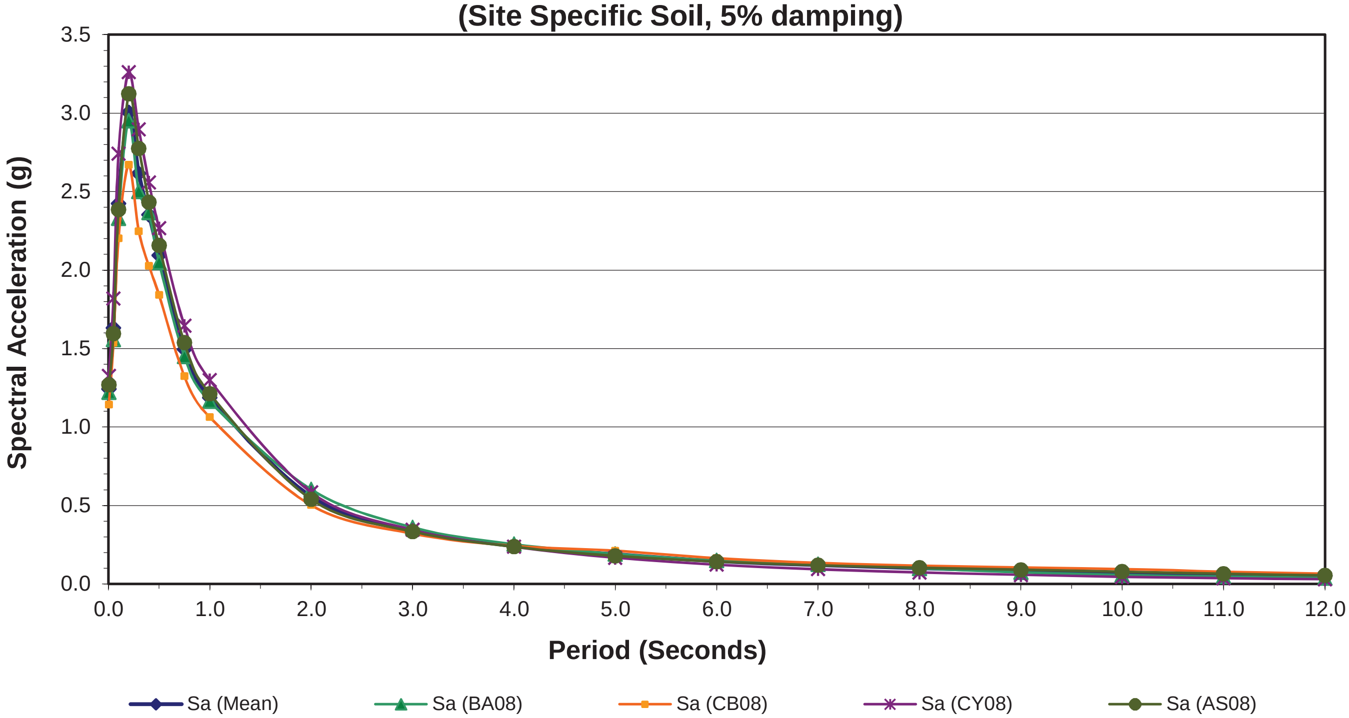
DETERMINISTIC RESPONSE SPECTRA

Castlewood Redwood Tanks Replacement
Pleasanton, California

PLATE

5

Uniform Hazard Spectra
(Maximum Considered Earthquake)
Spectral Response
Maximum Rotated Horizontal Component
(Site Specific Soil, 5% damping)



		Attenuation Relationship			
Period (Second)	Sa (Mean) (g)	(BA08) (g)	(CB08) (g)	(CY08) (g)	Sa (AS08) (g)
PGA	1.242	1.221	1.142	1.326	1.269
0.05	1.632	1.558	1.535	1.819	1.594
0.1	2.423	2.331	2.202	2.741	2.386
0.2	3.004	2.952	2.670	3.261	3.122
0.3	2.616	2.501	2.247	2.897	2.775
0.4	2.352	2.366	2.026	2.558	2.432
0.5	2.094	2.049	1.841	2.268	2.156
0.75	1.494	1.450	1.323	1.645	1.537
1.0	1.188	1.164	1.063	1.300	1.212
2.0	0.557	0.603	0.502	0.583	0.539
3.0	0.340	0.360	0.321	0.345	0.334
4.0	0.242	0.253	0.242	0.236	0.237
5	0.189	0.191	0.212	0.167	0.178
6	0.145	0.149	0.164	0.122	0.142
7	0.120	0.124	0.134	0.094	0.120
8	0.101	0.099	0.116	0.073	0.103
9	0.084	0.074	0.105	0.057	0.089
10	0.072	0.057	0.095	0.045	0.078
11	0.059	0.047	0.078	0.036	0.064
12	0.050	0.039	0.066	0.030	0.054

ASSOCIATES

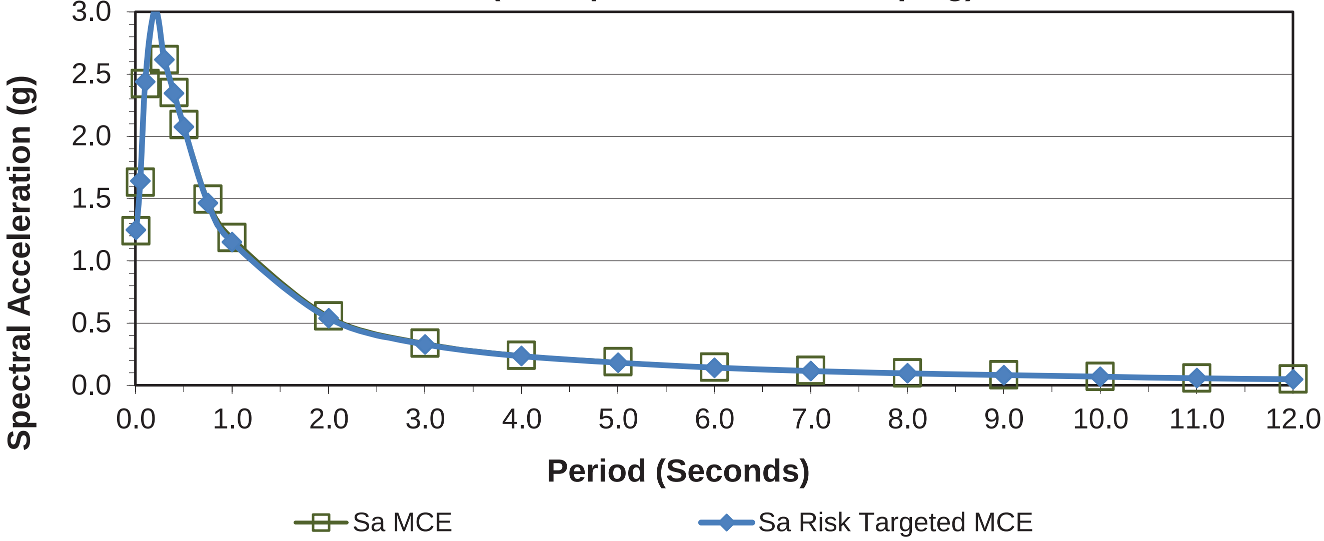
PROJECT NO. G16-062-11L
DRAWN: 05/03/16
DRAWN BY: M. Cline
CHECKED BY: C. Melo
FILE NAME: SitePlan.indd

UNIFORM HAZARD SPECTRA

Castlewood Redwood Tanks Replacement
Pleasanton, California

PLATE
6

Uniform Hazard Spectra
Risk-Targeted Maximum Considered Earthquake
Spectral Response Maximum Rotated Horizontal Component
(Site Specific Soil, 5% damping)



Period (Second)	Sa MCE (g)	Risk Coefficent C_R	Sa Risk Targeted MCE (g)
PGA	1.242	1.006	1.249
0.05	1.632	1.006	1.642
0.1	2.423	1.006	2.438
0.2	3.004	1.006	3.022
0.3	2.616	1.000	2.616
0.4	2.352	0.997	2.345
0.5	2.094	0.992	2.077
0.75	1.494	0.980	1.464
1	1.188	0.969	1.151
2	0.557	0.969	0.539
3	0.340	0.969	0.329
4	0.242	0.969	0.235
5	0.189	0.969	0.183
6	0.145	0.969	0.141
7	0.120	0.969	0.116
8	0.101	0.969	0.097
9	0.084	0.969	0.081
10	0.072	0.969	0.070
11	0.059	0.969	0.057
12	0.050	0.969	0.049

Notes: C_R From USGS Web Application

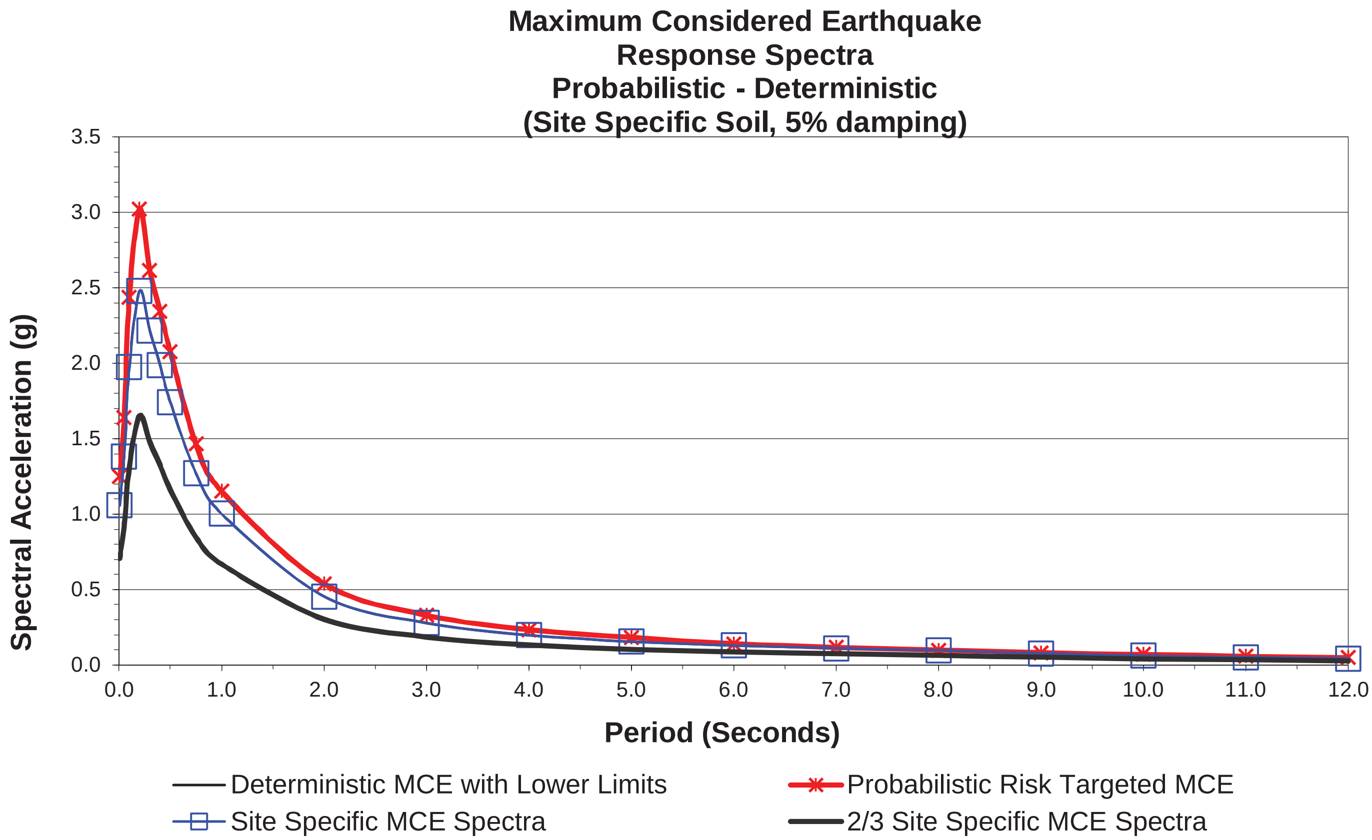
ASSOCIATES

PROJECT NO. G16-062-11L
DRAWN: 05/03/16
DRAWN BY: M. Cline
CHECKED BY: C. Melo
FILE NAME: SitePlan.indd

UNIFORM HAZARD SPECTRA

Castlewood Redwood Tanks Replacement
Pleasanton, California

PLATE
7



Period (Second)	Deterministic MCE with Lower Limits Sa (g)	Probabilistic Risk Targeted MCE Sa (g)	Site Specific MCE Spectra Sa (g)	2/3 Site Specific MCE Spectra Sa (g)
PGA	1.057	1.249	1.057	0.705
0.05	1.380	1.642	1.380	0.920
0.1	1.973	2.438	1.973	1.315
0.2	2.479	3.022	2.479	1.653
0.3	2.216	2.616	2.216	1.477
0.4	1.988	2.345	1.988	1.325
0.5	1.743	2.077	1.743	1.162
0.75	1.271	1.464	1.271	0.847
1.0	1.002	1.151	1.002	0.668
2.0	0.455	0.539	0.455	0.303
3.0	0.278	0.329	0.278	0.185
4.0	0.198	0.235	0.198	0.132
5	0.156	0.183	0.156	0.104
6	0.130	0.141	0.130	0.087
7	0.111	0.116	0.111	0.074
8	0.098	0.097	0.097	0.065
9	0.077	0.081	0.077	0.051
10	0.062	0.070	0.062	0.042
11	0.052	0.057	0.052	0.034
12	0.043	0.049	0.043	0.029

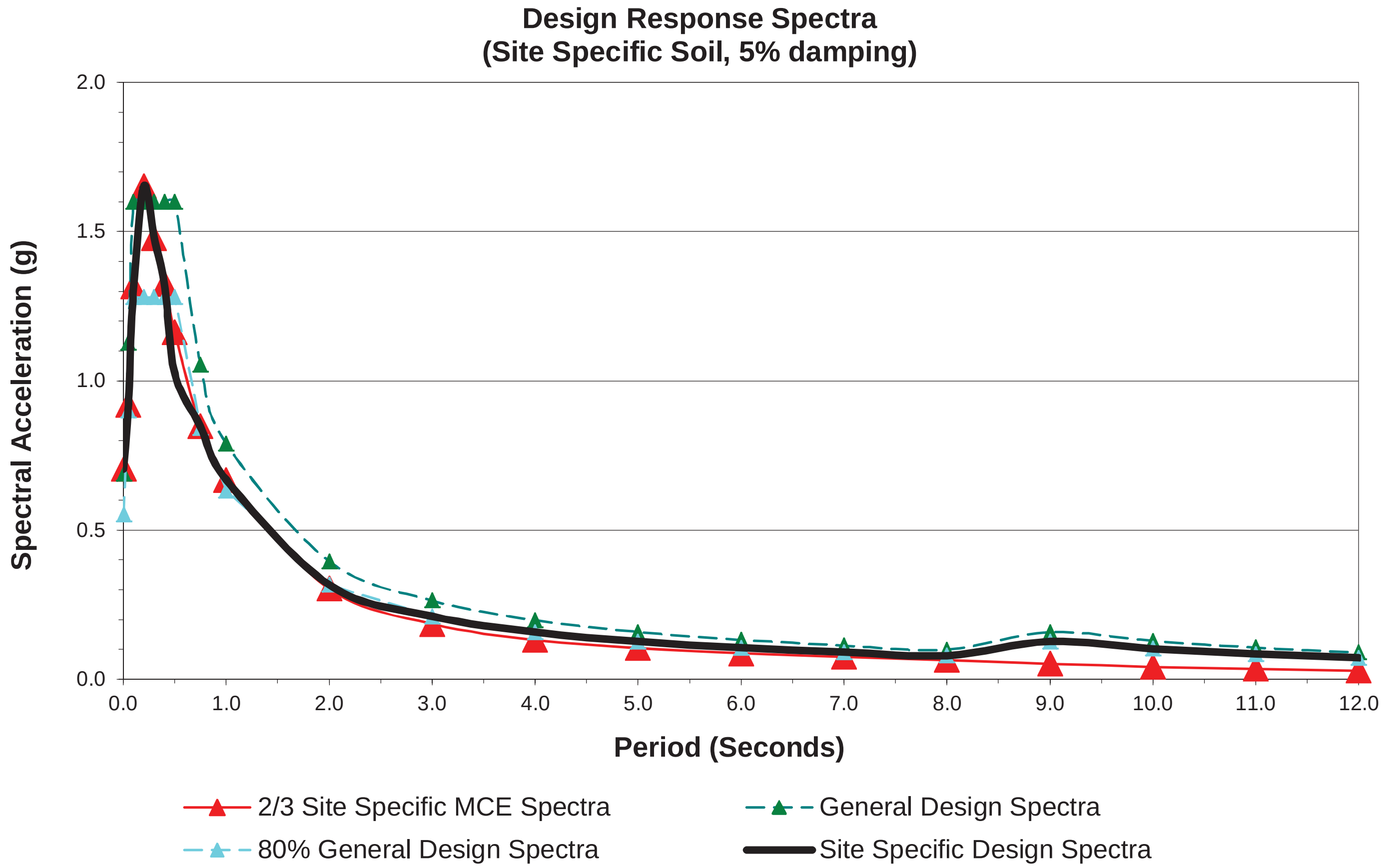
BSK ASSOCIATES

PROJECT NO. G16-062-11L
DRAWN: 05/03/16
DRAWN BY: M. Cline
CHECKED BY: C. Melo
FILE NAME: SitePlan.indd

MCE RESPONSE SPECTRA

Castlewood Redwood Tanks Replacement
Pleasanton, California

PLATE 8



Period (Second)	2/3 Site Specific MCE Spectra Sa (g)	General Design Spectra Sa (g)	80% General Design Spectra Sa (g)	Site Specific Design Spectra Sa (g)
PGA	0.705	0.689	0.551	0.705
0.05	0.920	1.126	0.901	0.920
0.1	1.315	1.600	1.280	1.315
0.2	1.653	1.600	1.280	1.653
0.3	1.477	1.600	1.280	1.477
0.4	1.325	1.600	1.280	1.325
0.5	1.162	1.600	1.280	1.024
0.8	0.847	1.053	0.842	0.847
1.0	0.668	0.790	0.632	0.668
2.0	0.303	0.395	0.316	0.316
3.0	0.185	0.263	0.211	0.211
4.0	0.132	0.197	0.158	0.158
5	0.104	0.158	0.126	0.126
6	0.087	0.132	0.105	0.105
7	0.074	0.113	0.090	0.090
8	0.065	0.099	0.079	0.079
9	0.051	0.158	0.126	0.126
10	0.042	0.128	0.102	0.102
11	0.034	0.106	0.085	0.085
12	0.029	0.089	0.071	0.071

Notes: General=General Response Spectrum based on 2013 CBC

BSK ASSOCIATES

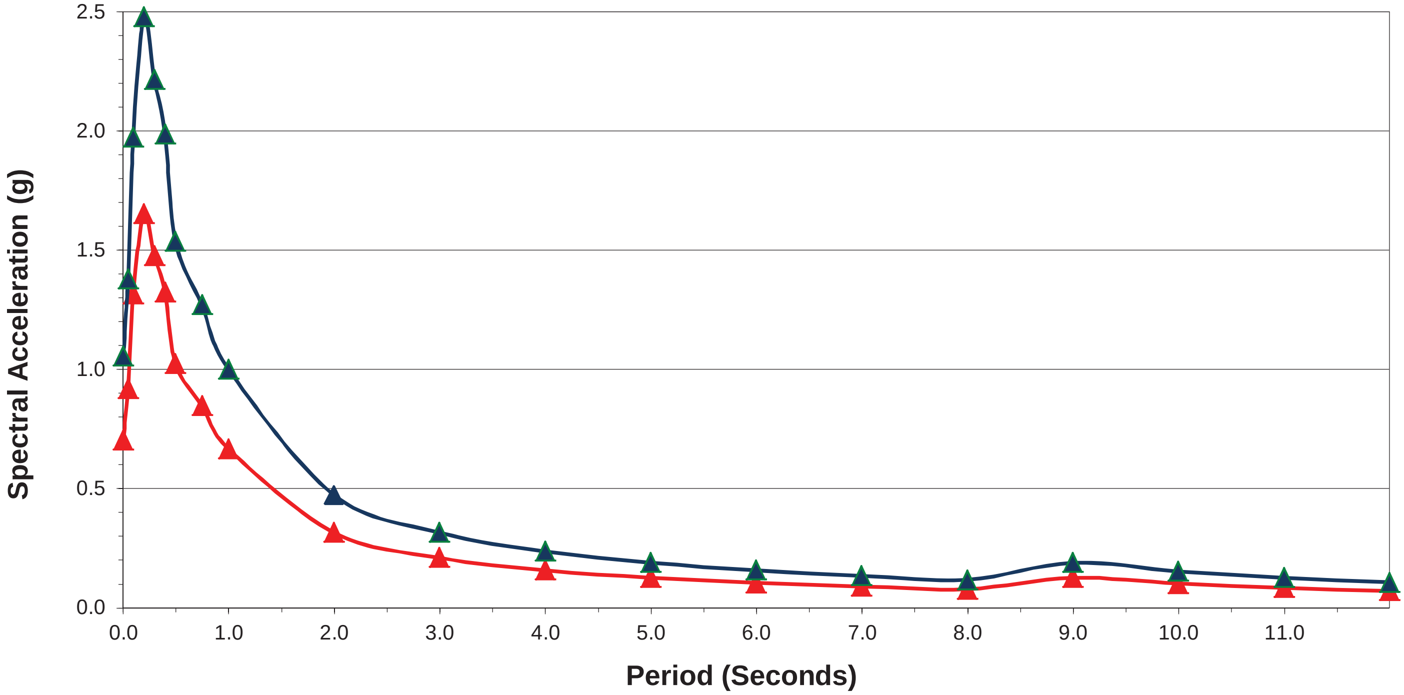
PROJECT NO. G16-062-11L
DRAWN: 05/03/16
DRAWN BY: M. Cline
CHECKED BY: C. Melo
FILE NAME: SitePlan.indd

5% DAMPING DESIGN RESPONSE SPECTRA

Castlewood Redwood Tanks Replacement
Pleasanton, California

PLATE
9

Impulsive and Convective Design Response Spectra
(Site Specific Soil/Rock)



Period (Second)	Impulsive Components 5% Damping Design Spectra Sa (g)	Component 0.5 % Damping Design Response Sa (g)
PGA	0.705	1.057
0.05	0.920	1.380
0.1	1.315	1.973
0.2	1.653	2.479
0.3	1.477	2.216
0.4	1.325	1.988
0.5	1.024	1.536
0.8	0.847	1.271
1.0	0.668	1.002
2.0	0.316	0.474
3.0	0.211	0.316
4.0	0.158	0.237
5	0.126	0.189
6	0.105	0.158
7	0.090	0.135
8	0.079	0.118
9	0.126	0.190
10	0.102	0.154
11	0.085	0.127
12	0.071	0.107

BSK ASSOCIATES

PROJECT NO. G16-062-11L
DRAWN: 05/03/16
DRAWN BY: M. Cline
CHECKED BY: C. Melo
FILE NAME: SitePlan.indd

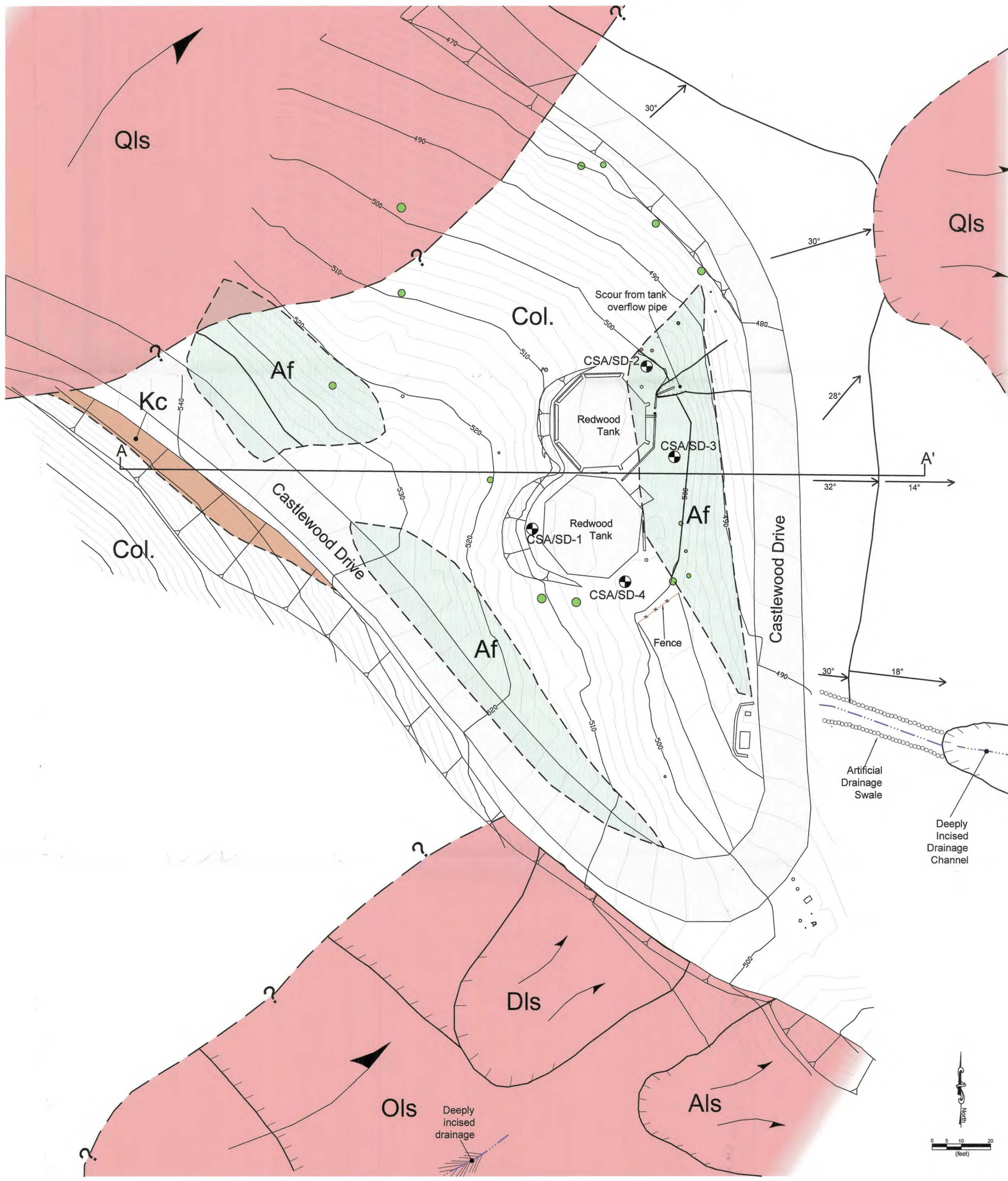
DESIGN RESPONSE SPECTRA

Castlewood Redwood Tanks Replacement
Pleasanton, California

PLATE
10

APPENDIX A

PREVIOUS STUDIES BY OTHERS



EXPLANATION

EARTH MATERIALS

Af — Fill - Sandy silt; light orange brown, medium stiff to very stiff, dry

ls — Als - Active Landslide
Dls - Dormant Landslide
Ols - Old Landslide
Qls - Quaternary Landslide

***Col.** — Colluvium - Sandy Silt with Gravels; dark yellowish brown, stiff to very stiff, moist.

Kc — Unnamed Conglomerate - well rounded cobbles and gravels in sand matrix, moderately weathered, hard

MAP SYMBOLS

? — Geologic contact, dashed where approximate, queried where unknown

Drainage

30° — Slope angle (degrees)

A — A' — Geologic Cross Section location

Cut Slope

Tree

CSA/SD-4 — Location of Small diameter exploratory boring

* Deep landsliding has been mapped by others in the area shown as colluvium, including in the vicinity of the tanks, see report text for details.

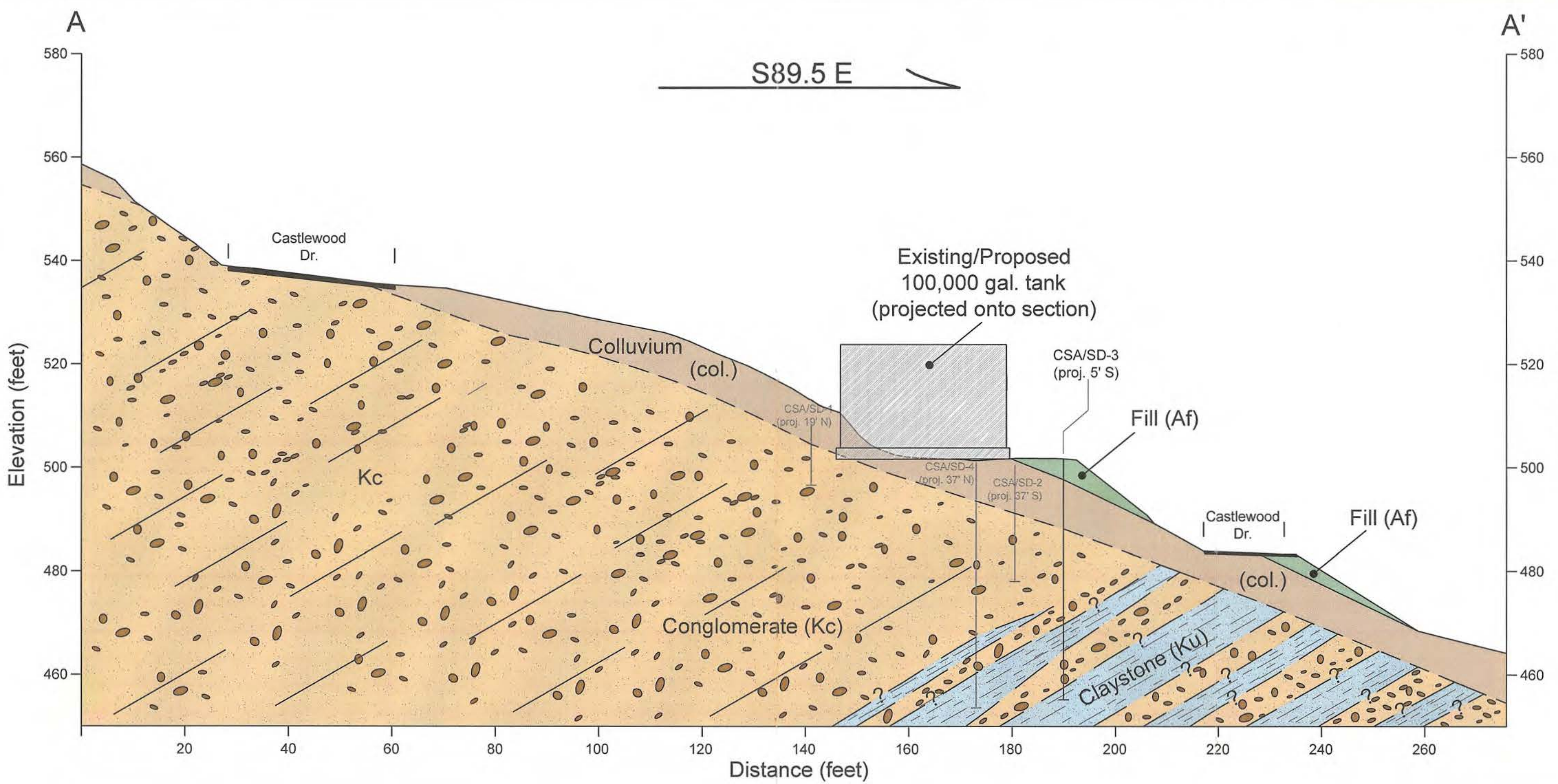
Notes: Base map compiled from detailed (2-foot contour interval) topographic survey by Cotton, Shires and Associates, Inc. on October 8, 2007. Elevation data is based on arbitrary datum set by Cotton, Shires and Associates, Inc. The elevations shown on this map are not based on established City or State elevation datum.

1. This is not a map of a boundary survey. No property corners have been set as part of this work.
2. Survey monuments found in the course of this mapping are set by others, and have been used only as a reference for the purpose of topographic mapping, without our verification of their agreement with applicable legal descriptions and seniority of deeds.
3. Relation of topographic features (i.e., fences, walls, trees, power poles, etc.) to property lines as shown on this map is subject to the adjustments that a boundary survey may require.
4. This survey was prepared without the benefit of a Title Report. Easements, if any, are not shown on this map.
5. If this map is provided in an electronic format as a courtesy to client, delivery of the electronic file does not constitute delivery of a professional work product. The signed paper print delivered with this electronic file constitutes our professional work product and, in the event the electronic file is altered, the print must be referred to for the original and correct survey information. We shall not be responsible for any modifications made to the electronic file or for any products derived from the electronic file which are not reviewed, signed and sealed by us.

COTTON, SHIRES & ASSOCIATES, INC.
CONSULTING ENGINEERS AND GEOLOGISTS

Engineering Geologic Map
Cal Water Service Company
Castlewood Tanks - Zone 2
PLEASANTON, CALIFORNIA

GEO/ENG BY	SCALE	PROJECT NO.
AM	1"=20'	E0357
APPROVED BY	DATE	PLATE NO.
DTS/TS	JANUARY 2008	1



EXPLANATION

Surficial Units

Af — Fill - Sandy silt; light orange brown, medium stiff to very stiff, dry

col. — Colluvium - Sandy Silt with Gravels; dark yellowish brown, stiff to very stiff, moist.

Bedrock Units*

Kc — Unnamed Conglomerate - well rounded cobbles and gravels in sand matrix, moderately weathered, hard

Ku — Unnamed Claystone - Dark blueish gray, highly sheared, weak, low hardness

Symbols

Geologic contact, dashed where approximate, queried where unknown

CSA/SD-3 — Small diameter exploratory boring

CSA/SD-4 — Small diameter exploratory boring projected onto section

*Deep landslideing has been mapped by others, see report text for additional explanation

COTTON, SHIRES & ASSOCIATES, INC
CONSULTING ENGINEERS AND GEOLOGISTS

Engineering Geologic Cross Section A-

California Water Service Company
Castlewood Tanks - Zone 2
PLEASANTON, CALIFORNIA

GEO/ENG BY	SCALE	PROJECT NO
AM	1"=20'	E0357
APPROVED BY	DATE	FIG NO.
DTS/TS	JANUARY 2008	4

APPENDIX A
FIELD INVESTIGATION

We explored subsurface conditions at the site of the California Water Service Company Castlewood Zone 2 facility in Livermore, California on November 12 and December 7, 2007 by means of four borings drilled to depths of 11.5 to 47.7 feet using track-mounted hollow stem, and portable solid stem auger equipment. The locations of the borings are shown on Figure 4. The engineering geologist who logged the borings visually classified the soils in accordance with ASTM D-2487. We obtained relatively undisturbed samples of the materials encountered at selected depths. These samples were obtained in brass liners that were 2.5 inches in outside diameter and 6 inches long; the liners were placed inside a 3-inch diameter modified split-barrel California Sampler for sampling. The track-mounted drill rig sampler was driven with a 140-pound hammer that was raised by an automatic hammer and allowed to freely fall about 30 inches. The portable drill rig sampler was driven with a 140-pound hammer that was raised by rope and cathead and allowed to freely fall about 30 inches. We also performed Standard Penetration Tests at selected depths. The depths of the sampling are shown on the boring logs. The bold number at the conclusion of the sampling interval represents the corrected blow count from a modified California sampler to Standard Penetration Test value accomplished by multiplying the blow count by a factor of 0.68.

Descriptive logs of the borings are presented in this appendix. These logs depict our interpretation of the subsurface conditions at the dates and locations indicated, based on representative samples collected at roughly five-foot sampling intervals. It is not warranted that they are representative of subsurface conditions at other times and locations. The contacts on the logs represent the approximate boundaries between earth materials, and the transitions between these materials may be gradual.

COTTON, SHIRES, AND ASSOCIATES, INC.
LOG OF EXPLORATORY DRILLING

Project Castlewood Tanks **Boring No.** CSA/SD-1
Location Behind South Tank (below cutslope) **Project No.** E0357
Drilling Contractor/Rig Cenozoic / Minuteman **Date of Drilling** 11/12/07
Ground Surface Elev. **Logged By** AM **Hole Diameter** 3.25" Solid Stem Auger
Surface Conditions Cut Pad / Bare Soil **Weather** Partly Cloudy

Depth (feet)	Graphic Log	USCS Class.	Geotechnical Description	Sample Desig.	Dry Density (pcf)	Moisture Content (%)	SPT Bl./ft.	Sample Type	Recov. (%)	Remarks
2		SP/ GP	FILL 0'-2.5' 0'-2.5' Sand with Gravel; moderate yellow brown, medium dense to dense, moist	T-1			17 50/3" **34/3"**	MC	7/9"	Start at 8:16 AM
4 6		ML	COLLUVIUM 2.5'-8' 2.5'-8' Sandy Silt; moderate brown, stiff to very stiff, contains gravels and cobbles of conglomerate	T-2 T-3			17 24 33 **39**	MC		LL=39, PI=21 8:38 AM
8				T-4	106	9.8	17 50/4" **34/4"**	MC		TX/UU 251 (2,000)
10		Conglomerate	CONGLOMERATE 8'-11.5' 8'-11.5' Conglomerate; hard, moderate strength, well rounded cobbles and gravel in light yellowish sand matrix	B-1			54/6" **54/6"**	SPT		9:27 AM
				B-2			100/5" **100/6"**	SPT		End at 10:00 AM
12 14 16 18 20 22 24 26 28			**Total Depth 11.5'** **No Water Encountered**							

COTTON, SHIRES & ASSOCIATES, INC.
CONSULTING ENGINEERS AND GEOLOGISTS

COTTON, SHIRES, AND ASSOCIATES, INC.
LOG OF EXPLORATORY DRILLING

Project Castlewood Tanks **Boring No.** CSA/SD-2

Location Northeast Corner of North Tank (by power pole) **Project No.** E0357

Drilling Contractor/Rig Cenozoic / Minuteman **Date of Drilling** 11/12/07

Ground Surface Elev. **Logged By** AM **Hole Diameter** 3.25" Solid Stem Auger

Surface Conditions Fill Pad / Bare Soil **Weather** Partly Cloudy

Depth (feet)	Graphic Log	USCS Class.	Geotechnical Description	Sample Desig.	Dry Density (pcf)	Moisture Content (%)	SPT Bl./ft.	Sample Type	Recov. (%)	Remarks
		ML	FILL 0'-6.5'							Start at 10:53 AM
			0'-6.5' Sandy Silt;light orange brown, medium stiff to very stiff, dry				6	MC		
				T-1			7			
2				T-2	100	9.2	13			
							14			
			@ 3.5' clasts of highly weathered conglomerate				18	MC		11:00 AM
4				T-3			47			
				T-4	113	13.3	42			
							59			
			@ 4' clast of sand matrix from conglomerate							
6			@ 5.5' drilling becomes very hard							
		ML	COLLUVIUM 6.5'-13'							
			6.5'-13' Sandy Silt/Silty Sand; dark yellowish brown, dry, very stiff				30	MC		11:40 AM
8				T-5			40			LL=42, PI=20
				T-6	123	10.3	50/5"			TX/UU 943 (4,000)
							61/11"			
			@ 8.5' clast of sand matrix from conglomerate							
10							35	MC		
				T-7	113	13.3	50/5"			12:00 PM
							34/5"			
12										
		Conglomerate	CONGLOMERATE 13'-22.5'							
14			8'-11.5' Conglomerate; hard, moderate strength, well rounded cobbles and gravel in light yellowish sand matrix							
				B-1			18	MC		
16							20			
							17			
							25			
							11	SPT		
							7			
18							4			
							11			
20				B-2			29	SPT		
							19			Stopped sample due to equipment malfunction
				B-3			40	SPT		
22							32			
							35			End at 1:20 PM
			Total Depth 22.5' **No Water Encountered**				**67**			
24										
26										
28										

COTTON, SHIRES, AND ASSOCIATES, INC.

LOG OF EXPLORATORY DRILLING

Project: Castlewood Tanks | Boring No.: CSA/SD-3

Location: Downslope side, center of two tanks | Project No.: E0357

Drilling Contractor/Rig: Britton / Crawler | Date of Drilling: 12/7/07

Ground Surface Elev.: | Logged By: AM | Hole Diameter: 8" Hollow Stem Auger

Surface Conditions: Fill Pad / Bare Soil | Weather: cool, scattered showers

Depth (feet)	USCS Class.	Geotechnical Description	Sample Desig.	Dry Density (pcf)	Moisture Content (%)	SPT Bl./ft.	Sample Type	Recov. (%)	Remarks
0	SM	FILL 0'-4' 0'-4' Silty Sand with Gravel; yellow brown to orange brown, loose to medium dense, sub angular gravels average 0.5" diameter, moderately sorted							Start at 8:12 AM
2						7 9 6 **10**	MC		
4			T-1	**112**	**9.6**	5 10 14 **16**	MC		
4	ML	COLLUVIUM 4'-13.5' 4'-13.5' Sandy Silt/Clay with Gravels; light grayish yellow green, stiff to very stiff, looks like greenstone detritus, gravel to cobble sized clasts,							
6			T-2			7 12 19 **21**	MC		8:31 AM
10			T-3 T-4	**115**	**10.9**	11 28 31 **40**	MC		8:38 AM
14	Conglomerate	CONGLOMERATE 13.5'-46.5' 13.5'-32' Conglomerate; hard, moderate strength, well rounded cobbles and gravel in light yellowish sand matrix							
16			T-5			9 12 15 **18**	MC		8:48 AM
20			B-1			8 11 11 **22**	SPT		9:00 AM
26			B-2			3 4 3 7	SPT		9:13 AM

Project Castlewood Tanks / E0357 Date 12/7/07 Boring No. CSA/SD-3

Depth (feet)	Graphic Log	USCS Class.	Geotechnical Description	Sample Desig.	Dry Density (pcf)	Moisture Content (%)	SPT Bl./ft.	Sample Type	Recov. (%)	Remarks
				B-3			5 4 4 **8**	SPT		9:32 AM
32			32′-35′ Silty sand; yellowish brown dense, moist							
34										
36			35′-37′ Silt/Clay; olive brown, stiff to very stiff	B-4 B-5			13 15 18 **33**	SPT		9:47 AM
38			37′-45.4′ Conglomerate; hard, moderate strength, well rounded cobbles and gravel in light yellowish sand matrix							
40				B-6			50/3″ **50/3″**	SPT		
42										
44										
46			45.4′-46.5′ Silt/Clay; olive brown, stiff to very stiff	B-7			11 13 17 **30**	SPT		End at 10:18 AM
			Total Depth 46.5′ **No Water Encountered**							
48										
50										
52										
54										
56										
58										
60										
62										

COTTON, SHIRES, AND ASSOCIATES, INC.
LOG OF EXPLORATORY DRILLING

Project	Castlewood Tanks	Boring No.	CSA/SD-4
Location	South side of southern tank, center of tanks	Project No.	E0357
Drilling Contractor/Rig	Britton / Crawler	Date of Drilling	12/7/07
Ground Surface Elev.	Logged By AM	Hole Diameter	8" Hollow Stem Auger
Surface Conditions	Fill Pad / Bare Soil	Weather	cool, scattered showers

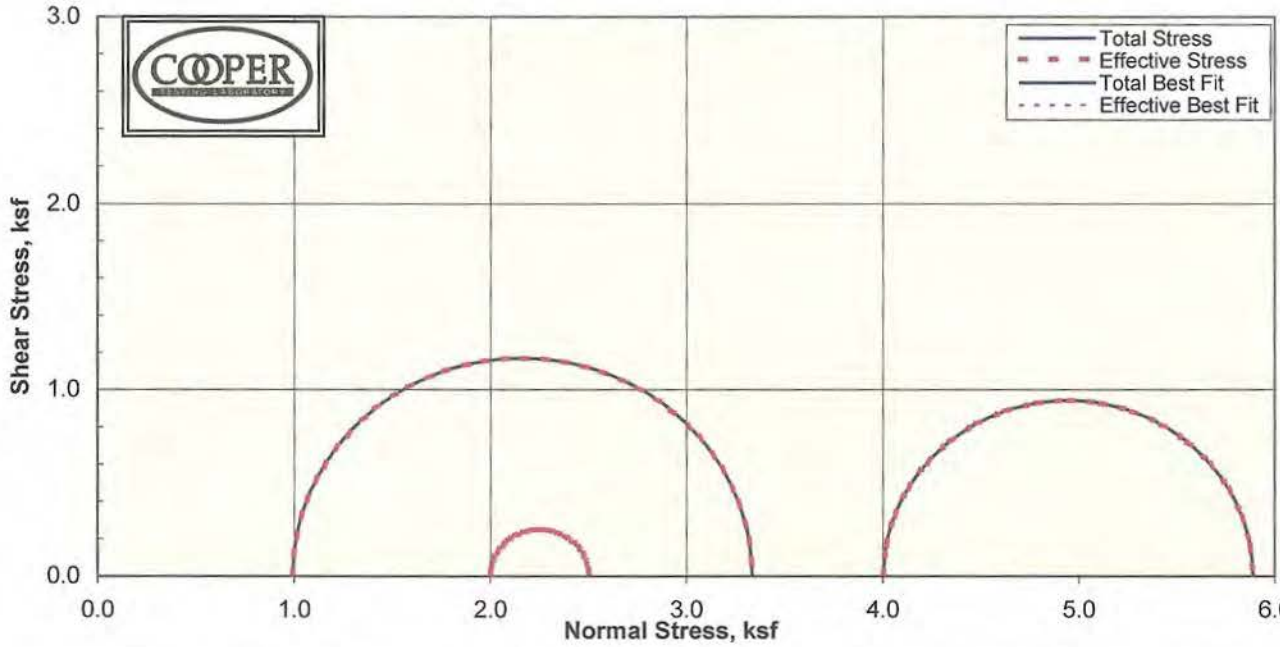
Depth (feet)	Graphic Log	USCS Class.	Geotechnical Description	Sample Desig.	Dry Density (pcf)	Moisture Content (%)	SPT Bl./ft.	Sample Type	Recov. (%)	Remarks
		GW	BASEROCK 0'-1.5'							Start at 11:34 AM
2		CL/ SC	COLLUVIUM 1.5'-13' 1.5'-13.5' Sandy Silt/Clay with Gravels; light grayish yellow green, stiff to very stiff, looks like greenstone detritus, gravel to cobble sized clasts,	T-1 T-2	105	18.8	11 7 8 **10**	MC		2 TX/UU 1,171 (1,000)
4				B-0			6 8 12 **14**	MC		4 11:42 AM
6				T-3 T-4	108	16.6	7 9 13 **15**	MC		6 11:47 AM LL=39, PI=18
8										8
10				T-5	122	13.6	5 7 11 **12**	MC		10 11:54 AM
12										12
14		Conglomerate	CONGLOMERATE 13'-34' 13'-34' Conglomerate; hard, moderate strength, well rounded cobbles and gravel in light yellowish sand matrix							14
16				B-1			5 6 10 **16**	MC		16 12:10 PM
18										18
20				B-2			13 11 10 **21**	SPT		20
22										22
24										24 12:25 PM
26				B-3			6 12 14 **26**	SPT		26
28										28

Project Castlewood Tanks / E0357 Date 12/7/07 Boring No. CSA/SD-4

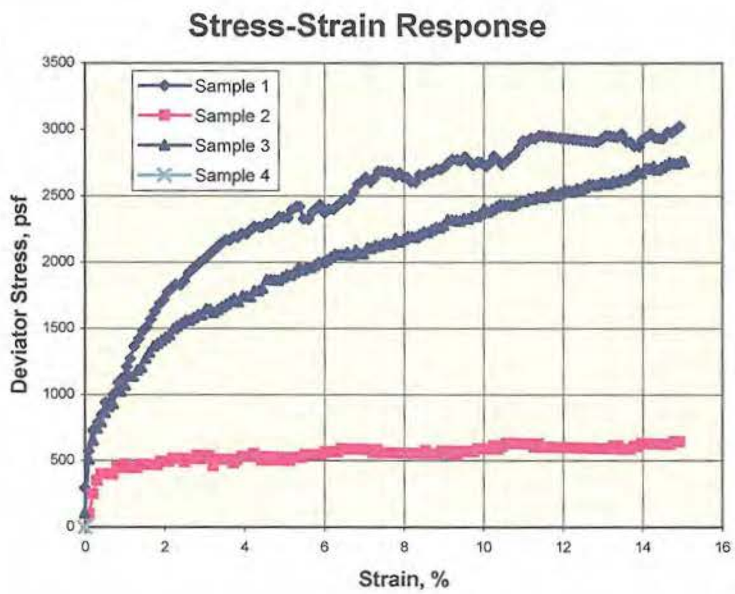
Depth (feet)	Graphic Log	USCS Class.	Geotechnical Description	Sample Desig.	Dry Density (pcf)	Moisture Content (%)	SPT Bl./ft.	Sample Type	Recov. (%)	Remarks
32		Conglomerate		B-4			9 10 13 **23**	SPT		12:43 PM
34		Claystone	CLAYSTONE 34′-47.5′ 34′-37′ Claystone; blue gray, low hardness, weak, tectonic shears throughout	B-5			18 15 18 33	SPT		
36, 38, 40		Conglomerate	37′-40.5′ Conglomerate; hard, moderate strength, well rounded cobbles and gravel in light yellowish sand matrix	B-6			29 50/5″ **50/5″**	SPT		
42, 44, 46		Claystone	40.5′-47.5′ Claystone; blue gray, low hardness, weak, tectonic shears throughout	B-7 B-8			32 37 24 **61**	SPT		1:21 PM
				B-9			12 39 37 **76**	SPT		
		▼	@ 47′ Groundwater encountered				50/2″ **50/2″**	SPT		End at 1:56 PM
48, 50, 52, 54, 56, 58, 60, 62			**Total Depth 47.5′** **Water Encountered at 47′**							

Triaxial Consolidated Undrained

(ASTM D4767 MODIFIED)



Stress-Strain Response



Sample:	1	2	3	4
MC, %	18.8	9.8	10.3	
Dry Dens, pcf	105.4	106.3	123.4	
Sat. %	84.6	45.0	76.3	
Void Ratio	0.599	0.585	0.365	
Diameter in	2.42	2.41	2.42	
Height, in	5.00	5.01	5.00	
	Final			
MC, %	22.9	23.3	17.8	
Dry Dens, pcf	104.1	103.4	113.8	
Sat. %	100.0	100.0	100.0	
Void Ratio	0.618	0.629	0.481	
Diameter, in	2.43	2.44	2.52	
Height, in	5.01	5.02	4.99	
Cell, psi	55.4	62.4	76.3	
BP, psi	48.5	48.5	48.5	
	Effective Stresses At:			
Strain, %	5.0	5.0	5.0	
Deviator ksf	2.342	0.502	1.885	
Excess PP	0.000	0.000	0.000	
Sigma 1	3.336	2.504	5.889	
Sigma 3	0.994	2.002	4.003	
P, ksf	2.165	2.253	4.946	
Q, ksf	1.171	0.251	0.943	
Stress Ratio	3.357	1.251	1.471	
Rate in/min	0.001	0.001	0.001	
Total C	N/A	ksf		
Total Phi	N/A	Degrees		
Eff. C	N/A	ksf		
Eff. Phi	N/A	Degrees		

Job No.: 026-384 **Date:** 12/21/2007

Client: Cotton, Shires & Associates BY:MD/DC

Project: Castlewood - E0357

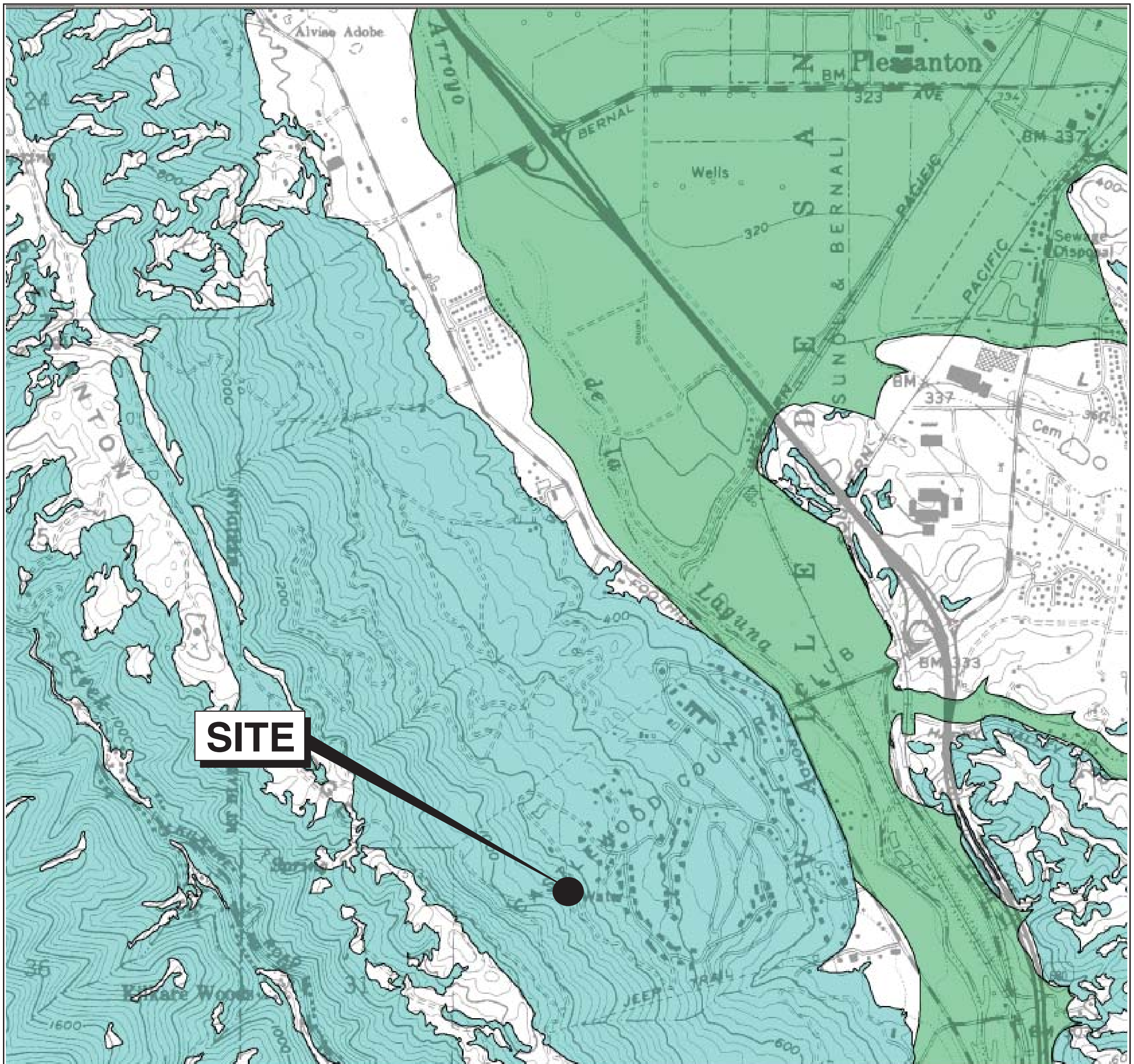
Sample 1) SD-4;T-2 @ 2' Brown Sandy CLAY w/ Gravel

Sample 2) SD-1;T4 @7.5' Brown Cl SAND/ Sa CLAY

Sample 3) SD-2;T6 @ 8' Brown Cl SAND w/ Gravel

Sample 4)

REMARKS: Strengths picked at 5% strain. Samples were back-pressure saturated. Drainage valves were closed prior to consolidation. Samples were consolidated 1.5 psi during saturation.



Reference:
State of California, California Geological Survey "Seismic Hazard Zones" Dubin Quadrangle, August 27, 2008.

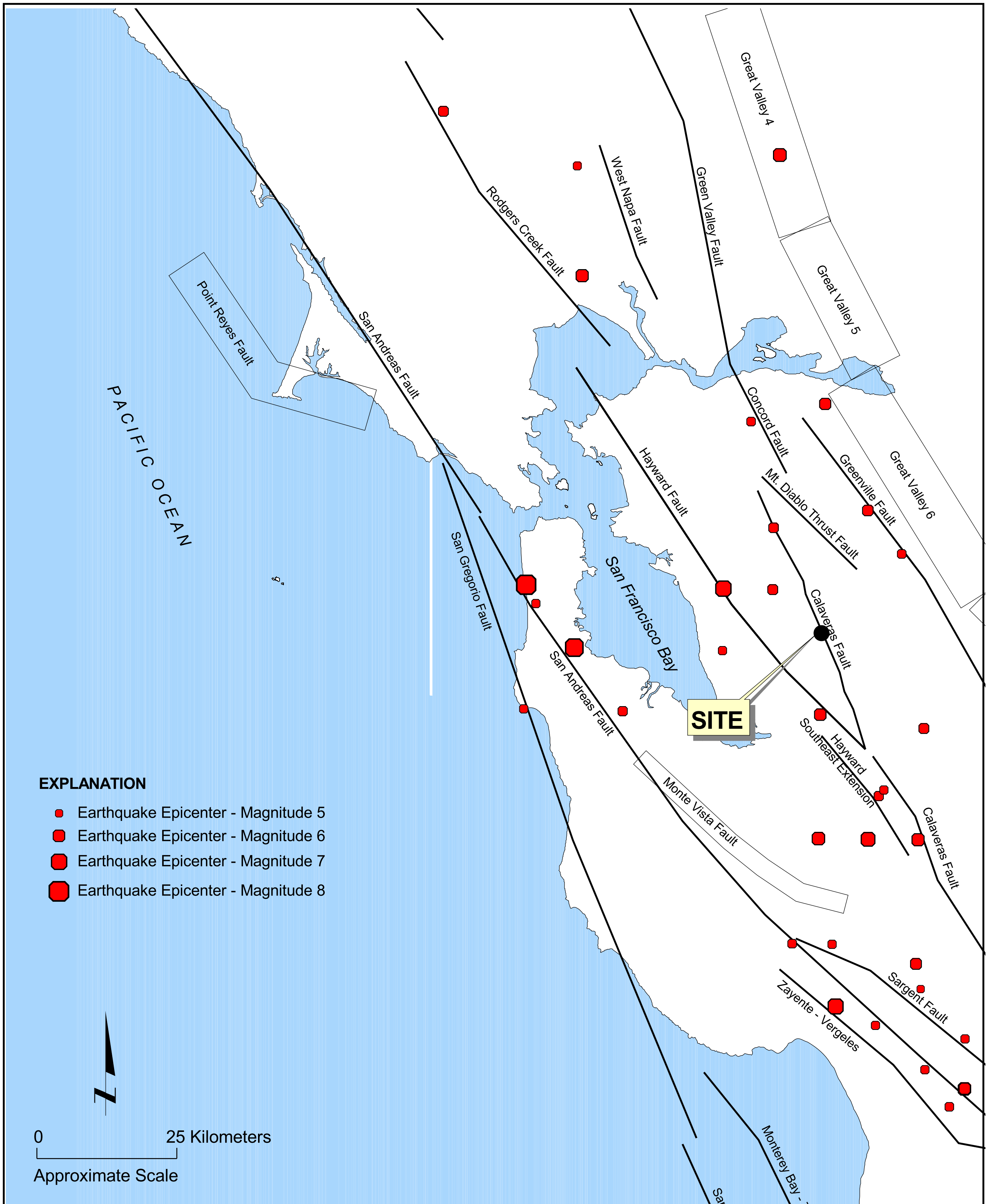
Zone of Liquefaction

Earthquake-induced Landslides

0 2000 Feet

Approximate scale

CASTLEWOOD COUNTY SERVICE AREA REDWOOD TANKS Alameda County, California	REGIONAL SEISMIC HAZARD ZONES MAP		
Treadwell&Rollo	Date 01/09/09	Project No. 4916.01	Figure 6

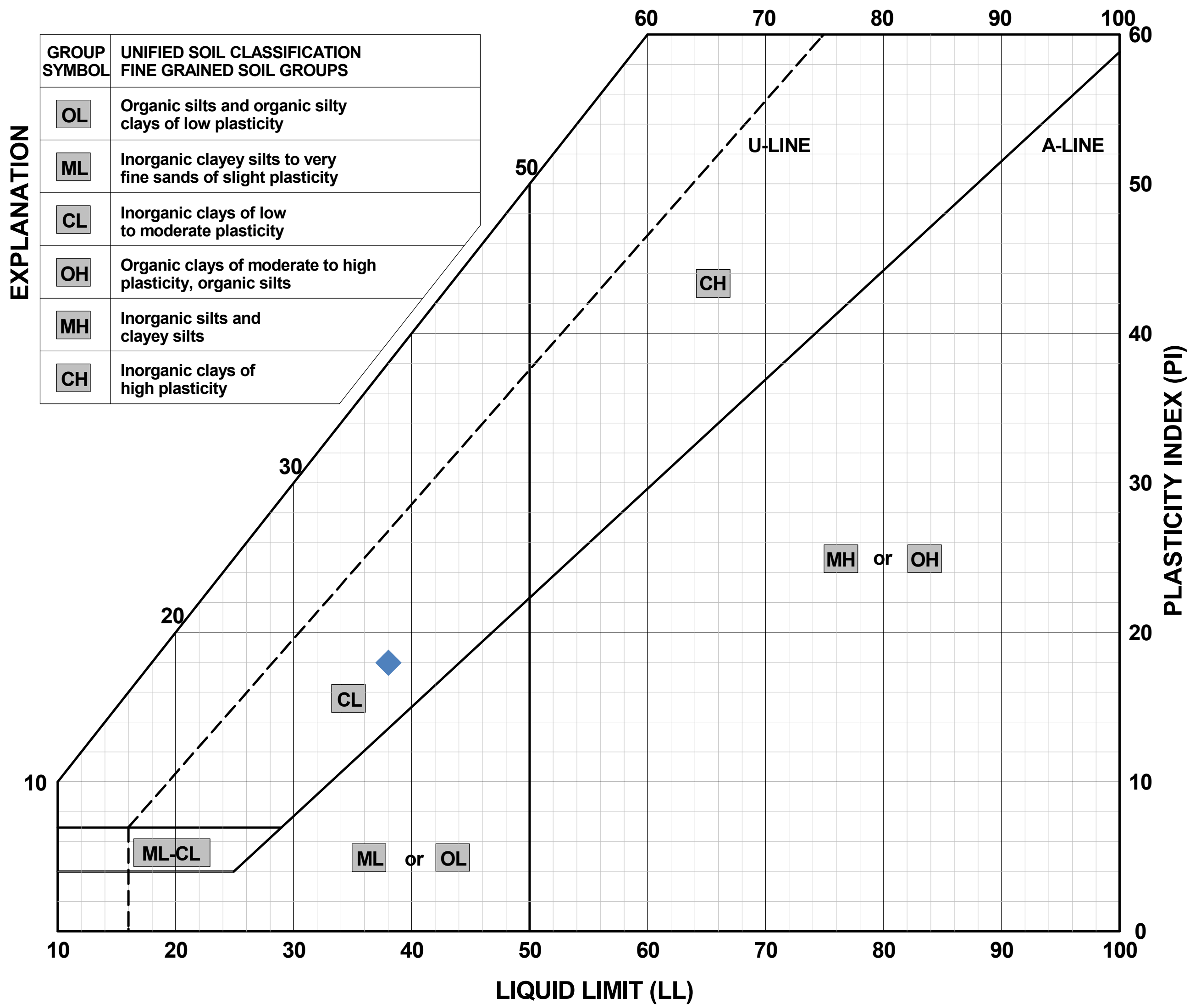


NOTES:
Digitized data for fault coordinates and earthquake catalog was developed by the California Department of Conservation Division of Mines and Geology. The historic earthquake catalog includes events from January 1800 to December 2000.

CASTLEWOOD COUNTY SERVICE AREA REDWOOD TANKS Alameda County, California	**MAP OF MAJOR FAULTS AND EARTHQUAKE EPICENTERS IN THE SAN FRANCISCO BAY AREA**
Treadwell&Rollo	Date: 01/06/09 | Project No. 4916.02 | Figure 7

APPENDIX B

LABORATORY TEST RESULTS



EXPLANATION

GROUP SYMBOL	UNIFIED SOIL CLASSIFICATION FINE GRAINED SOIL GROUPS
OL	Organic silts and organic silty clays of low plasticity
ML	Inorganic clayey silts to very fine sands of slight plasticity
CL	Inorganic clays of low to moderate plasticity
OH	Organic clays of moderate to high plasticity, organic silts
MH	Inorganic silts and clayey silts
CH	Inorganic clays of high plasticity

LEGEND:	SOURCE	DEPTH (ft)	LL	PL	PI	DESCRIPTION
◆	S-2	1.0	38	20	18	Olive Brown Clayey Sand with Gravel (SC)

BSK ASSOCIATES

PROJECT NO. G16-062-11L
DRAWN: 06/08/16
DRAWN BY: D. Tower
CHECKED BY: C. Melo
FILE NAME: SitePlan.indd

ATTERBERG LIMITS

Castlewood Redwood Tanks Replacement
Pleasanton, California

PLATE
B-1

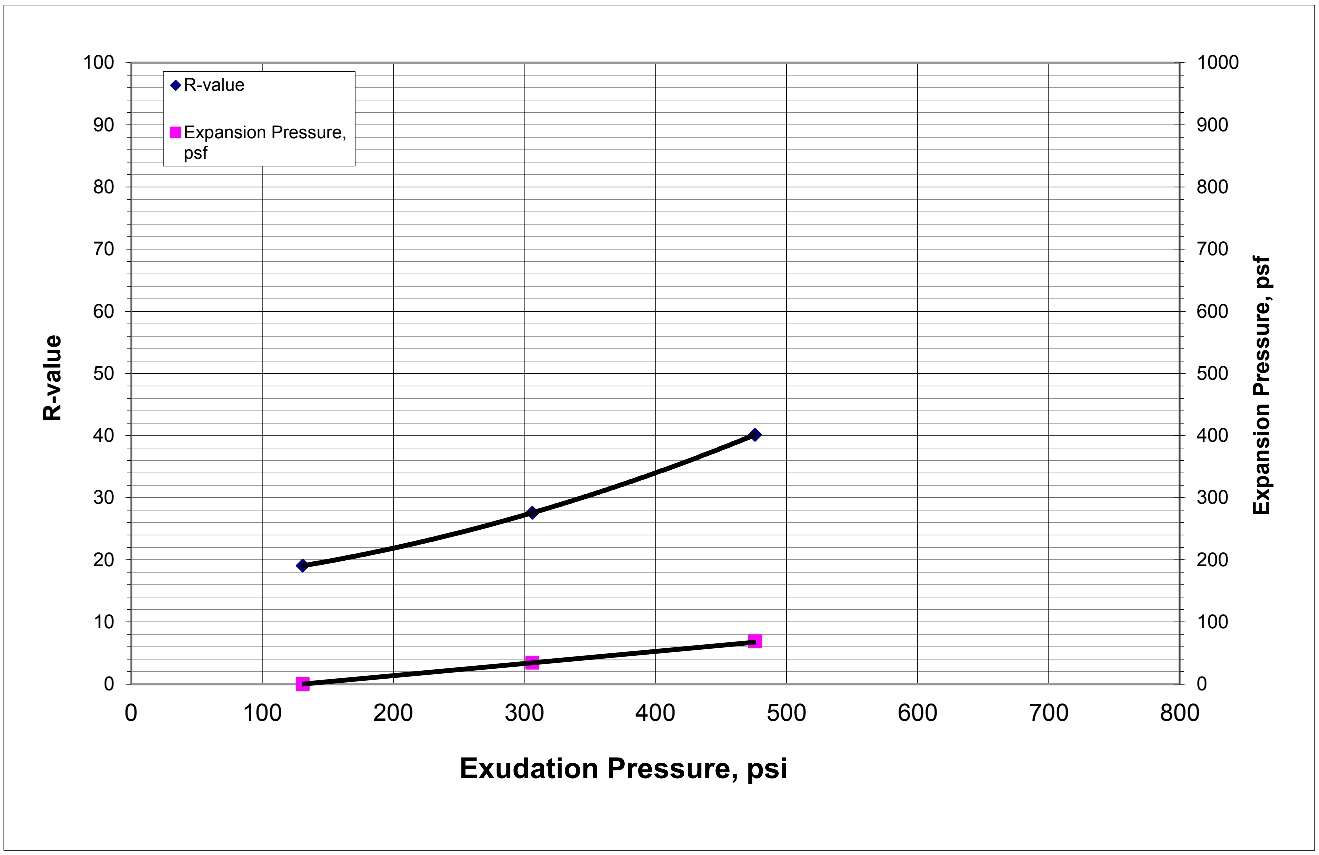
R-value Test Report (Caltrans 301)

COOPER TESTING LABORATORY

Job No.:	664-068	**Date:**	04/13/16	**Initial Moisture,**	16.5%
Client:	BSK Associates	**Tested**	MD	**R-value by Stabilometer**	**27**
Project:	G16-062-11L	**Reduced**	RU		
Sample	S-1 @ 0-1'	**Checked**	DC	**Expansion Pressure**	**35 psf**
Soil Type:	Olive Brown Clayey Sand (SC)				

Specimen Number	A	B	C	D
Exudation Pressure, psi	131	306	476	
Prepared Weight, grams	1200	1200	1200	
Final Water Added, grams/cc	70	35	10	
Weight of Soil & Mold, grams	3060	3000	3133	
Weight of Mold, grams	2106	2098	2098	
Height After Compaction, in.	2.5	2.45	2.55	
Moisture Content, %	23.3	19.9	17.5	
Dry Density, pcf	93.7	93.0	104.6	
Expansion Pressure, psf	0.0	34.4	68.8	
Stabilometer @ 1000				
Stabilometer @ 2000	120	106	90	
Turns Displacement	3.54	3.2	2.9	
R-value	19	28	40	

Remarks:



BSK ASSOCIATES

PROJECT NO. G16-062-11L	**R-VALUE**		**PLATE**
DRAWN: 06/08/16			
DRAWN BY: D. Tower			**B-2**
CHECKED BY: C. Melo	**Castlewood Redwood Tanks Replacement**		
FILE NAME: SitePlan.indd	**Pleasanton, California**		

California State Certified Laboratory No. 2153

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 **462 2771** Fax. 925 **462 2775**
www.cercoanalytical.com

6 May, 2016

Job No. 1604265
Cust. No.12667

Ms. Danaige Tower
BSK Associates Engineers & Laboratories
324 Earhart Way
Livermore, CA 94551

Subject: Project No.: G16-062-11L
Project Name: Castlewood Redwood Tank Replacement
Corrosivity Analysis – ASTM Test Methods

Dear Ms. Tower:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on April 27, 2016. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurement, the sample is classified as "moderately corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentration is none detected to 15 mg/kg.

The sulfate ion concentration is none detected to 15 mg/kg.

The pH of the soil is 7.07 which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potential is 410-mV which is indicative of aerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call *JDH Corrosion Consultants, Inc. at (925) 927-6630.*

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

California State Certified Laboratory No. 2153

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

Client: BSK Associates Engineers & Laboratories
Client's Project No.: G16-062-11L
Client's Project Name: Castlewood Redwood Tank Replacement
Date Sampled: 27-Apr-16
Date Received: 28-Apr-16
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 6-May-2016

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
1604265-001	Bulk @ 0'-1'	410	7.07	-	3,400	-	N.D.	N.D.

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
	5-May-2016	5-May-2016	-	4-May-2016	-	5-May-2016	5-May-2016

* Results Reported on "As Received" Basis

N.D. - None Detected

Cheryl McMillen
Laboratory Director

Quality Control Summary - **All laboratory quality control parameters were found to be within established limits**

APPENDIX C

EXHIBIT 1 – SUMMARY OF COMPACTION RECOMMENDATIONS

EXHIBIT 1

SUMMARY OF COMPACTION RECOMMENDATIONS

Area	Compaction Recommendations (See Notes 1, 2, 3, 4, 7)
Foundation Bottom, Subgrade Preparation and Placement of General Engineered Fill[5], Including Imported Fill	Compact upper 12 inches of foundation bottom, subgrade, and entire fill to a minimum of 90 percent compaction at near optimum moisture content for granular soils and to a minimum of 90 percent compaction at a minimum of 2 percent over optimum moisture content for clayey soils.
Trenches[6]	Compact trench backfill to a minimum of 90 percent compaction at near optimum moisture content for granular soils and to a minimum of 90 percent compaction at a minimum of 2 percent over optimum moisture content for clayey soils. Where trenches will be under flatwork or paving, the upper 12 inches should be compacted as recommended below.
Exterior Flatwork	Compact upper 12 inches of subgrade to a minimum of 90 percent compaction at near optimum moisture content for granular soils and to a minimum of 90 percent compaction at a minimum of 2 percent over optimum moisture content for clayey soils. Compact aggregate base to a minimum of 90 percent compaction at near optimum moisture content. Where exterior flatwork is exposed to vehicular traffic, compact aggregate base and upper 12 inches of subgrade to the pavement requirements below.
Pavements	Compact upper 12 inches of subgrade to a minimum of 95 percent compaction at near optimum moisture content for granular soils and to a minimum of 92 percent compaction at a minimum of 2 percent over optimum moisture content for clayey soils. Compact aggregate base to a minimum of 95 percent compaction near optimum moisture content.

Notes:

(1) Depths are below finished subgrade elevation.

(2) All compaction requirements refer to relative compaction as a percentage of the laboratory standard described by ASTM D 1557.

(3) Fill material should be compacted in lifts not exceeding 8 inches in loose thickness.

(4) All subgrades should be firm and stable.

(5) Including backfill.

(6) In landscaping areas <u>only</u>, the percent compaction in trenches may be reduced to 85 percent.

(7) Where fills are greater than 7 feet in depth below finish grade, the zone below a depth of 7 feet should be compacted to a minimum of 95 percent compaction.