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August 11, 2016

Don Barraza, P.E.
Principal Engineer
Kennedy/Jenks Consultants
303 Second Street, Suite 300 South
San Francisco, CA, 94107

Subject: **Geotechnical Investigation Report**
East Dunne Hillside Water Reservoir Project
Morgan Hill, California

Dear Mr. Barraza:

Cal Engineering & Geology, Inc. (CE&G) is pleased to submit this geotechnical investigation report to support the design for the East Dunne Hillside Water Reservoir Project in Morgan Hill, California. Our investigation included compiling and reviewing existing data; performing a field exploration program, geotechnical laboratory testing, engineering evaluations and analyses; and preparing this report.

CE&G appreciates the opportunity to submit this geotechnical investigation report. If there are questions concerning the information provided herein, please do not hesitate to contact us.

Sincerely,

CAL ENGINEERING & GEOLOGY

Dan Peluso, P.E., G.E.
Associate Engineer

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1. Introduction

1.1 General

Design of the City of Morgan Hill's (City) East Dunne Hillside Water Reservoir Project is led by Kennedy/Jenks Consultants. Cal Engineering & Geology, Inc. (CE&G) has provided geotechnical engineering services for the Project, which is located in the Jackson Oaks area of eastern Morgan Hill, California. In support of the Kennedy/Jenks, Cal Engineering & Geology's work included compiling and reviewing available pertinent geotechnical and geologic data; performing field reconnaissance, a field exploration and laboratory testing program, and geotechnical engineering analyses; developing geotechnical design recommendations for the proposed improvements; and preparing this report. The work has been completed to collect geotechnical data and provide engineering analyses and geotechnical design recommendations for the design team to design a water tank, pump station pad, access road, and associated retaining walls to be constructed at the site. The location of the Project is shown in Figure 1, Site Location Map.

1.2 Project Description

As currently conceived, the project includes: an approximately 850,000-gallon steel water tank approximately 80 feet in diameter; a 15-foot-wide perimeter access strip immediately encircling the tank; tiered retaining walls along the upslope approximately half of the tank pad; a pump station and slab-on-grade pad along the downslope side of the tank pad; an access road stemming northeastward from the NE-bound lane of East Dunne Avenue; retaining walls along portions of the access road; and connective piping between the tank/pump station and East Dunne Avenue.

1.3 Purpose and Scope of Services

The investigation completed by CE&G was undertaken to assess the existing surface and subsurface conditions in the immediate vicinity of the proposed project, and to develop geotechnical design recommendations for the proposed improvements.

The scope of work completed for the geotechnical investigation and report included:

1. Meetings with the City and Kennedy/Jenks and management of geotechnical explorations.

2. Completion of an office study to identify and evaluate relevant geologic and geotechnical information available for the site, including published geologic maps, and previously prepared reports regarding the site and vicinity.

3. Geologic reconnaissance to observe current site conditions.

4. A subsurface exploration and laboratory testing program to develop information needed to complete geotechnical analyses and prepare this geotechnical report.

5. Completion of engineering analyses to develop geotechnical parameters for the design of the water tank foundations, retaining walls, access road, and pump station pad.

6. Preparation of a draft and final geotechnical investigation report.

1.4 Information Provided and Previous Site Investigations

CE&G previously evaluated the preliminary engineering geologic suitability of the site, and prepared a report entitled *Preliminary Engineering Geologic Feasibility Evaluation, Proposed East Dunne Tank Site, Morgan Hill, California*, dated 27 July 27, 2015. Information from this previous study was used in developing the scope for the geotechnical investigation and for refining the siting of the water reservoir. Pertinent background information is carried forward in this report.

The following information was provided by the Kennedy/Jenks and/or Mark Thomas & Co., the project surveyors:

- A composite topographic and orthophoto base map of the project area, and stationing data for the access road alignment in the form of electronic AutoCAD files.

- Preliminary access road alignment and profiles.

- Technical Memorandum #1, Design Alternative Evaluation No. 1 - Retaining Wall Alternatives

2. Site Conditions

2.1 Site Description

The proposed tank site is currently an undeveloped, generally open, grass-covered hillslope with sparse oak trees. To the west and downslope lies a sweeping switchback turn in East Dunne Avenue, with a cut slope bordering the roadway on the west side of the site. To the north and upslope are residential properties. Downslope (to the south) of the site, the grassy slopes yield to scattered oak trees clustered along the axis of a northeast-southwest-trending topographic swale. Land use in the vicinity is residential. Based on available information, the site has not been previously developed.

The hilly terrain encompassing the site is located on the western flank of the Diablo Range, one of the component ranges of the Coast Ranges geomorphic province of California. The slopes of the tank site descend westward to the floor of Coyote Valley, within which the City of Morgan Hill is centered.

The tank site is located on a southwest-facing slope with overall gradients ranging from approximately 16 – 19 degrees in the upper portion of the site and tank vicinity, to 22 - 28 degrees in the lowermost portion of the site, downslope and southwest of the proposed access road. An unnamed drainage course defined by the topographic swale drops from northeast to southwest, passing downslope of the tank and access road. Slope gradients within approximately 150 feet of this swale are steeper than the overall slopes farther uphill.

The overall surface water flow pattern in the site vicinity is westward toward East Dunne Avenue, and southwestward toward the unnamed topographic swale that ultimately drains into Upper Llagas Creek.

Elevations across the property range from approximately 675 feet above mean sea level (msl) in the unnamed topographic swale near the downslope property boundary, to approximately 870 feet msl near the existing residences upslope of the upper property boundary. The tank pad would be constructed at elevation 780 ft msl.

2.2 Topographic and Survey Information

Topography of the site was provided by Kennedy/Jenks. The topographic data are in LiDAR (Light Detection and Ranging) format derived from the San Jose Phase 3 LIDAR project. Latitude and longitude coordinates are based on the California Coordinate System Zone 3 and the 1983 North American Datum (NAD83). Elevation references are based on 1988 North American Vertical Datum (NAVD88).

3. Geology

The regional geologic setting and observations regarding surface outcrops and site geomorphology are contained in our preliminary engineering geologic feasibility report (CE&G, 2015), and are not reproduced fully herein. The reader is referred to that report for additional detail pertaining to the site geology.

3.1 Geologic Setting

The East Dunne tank site lies within the Coast Ranges geomorphic province of California. This province is characterized by northwest-southeast trending mountain ranges and intervening valleys such as that occupied by San Francisco Bay and the Santa Clara Valley. The geologic setting is shown on our Regional Geologic and Index Map (Figure 2).

3.1.1 Bedrock Geology

Regional geologic mapping by Wentworth and others (1999), shows the upslope (eastern) part of the site as being underlain by the Pliocene-age Basalt of Anderson and Coyote Reservoirs. The western part of the site vicinity is mapped as being underlain by the Silver Creek Gravels of similar age. Slightly younger deposits known as the Packwood Gravels lie just upslope and east of the site. The Silver Creek Gravels are described as consisting of interbedded conglomerate, sandstone, siltstone, tuffaceous sediment, tuff, and basalt. The Basalt of Anderson and Coyote Reservoirs is described as pyroclastic andesite and basalt flows. The Packwood Gravels consist typically of gravel, cobbles, sandy conglomerate, silty sandstone, sandy siltstone and minor claystone. Regionally, all of these units overlie ophiolitic (ocean floor) and Franciscan Complex metamorphic rocks; the nearest exposures of these rocks is to the north, along the spine of the ridge crest west of Anderson Lake. Wentworth's mapping considered and incorporated earlier more detailed mapping by PGE (1991) described below.

Detailed geologic mapping performed for the City of Morgan Hill (PGE, 1991) shows similar rock types, although the names and ages assigned to the map units differs from those used by Wentworth and others. As shown on PGE (1991), the site is underlain by rocks of the Santa Clara Formation (map unit QTs on Plate 1 below). In general, this formation consists of "poorly to well-consolidated" non-marine sediments largely reflective of an alluvial fan setting. Within this formation are intervals of basalt lava flows and flow breccia (map unit QTsb); at least two of these intervals are shown on the City Geologic Map, although this mapping is somewhat interpretive.

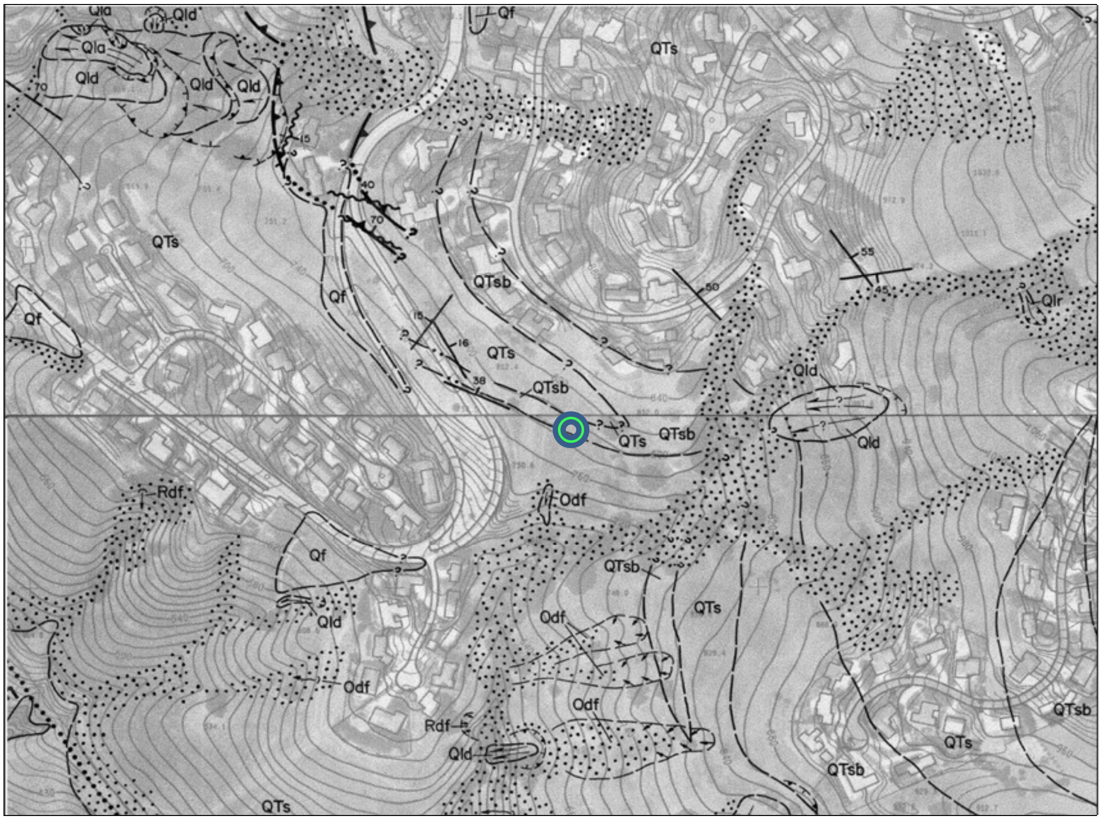

Plate 1 - Excerpt of PGE (1991), with site location at green circle.

Geologic interpretation and analysis performed for the Anderson Dam Seismic Retrofit Project highlighted extensive folding and possible broken folds within the Santa Clara Formation; the implication of this for the tank site is that belts of rock shown as continuous on maps such as PGE (1991) may in fact not be nearly as continuous.

3.1.2 Landslide Geology

Regional landslide mapping (Nilsen, 1975; excerpt provided in CE&G, 2015) does not show any landslides at the site, although earthflow-style landslide deposits are shown in the general vicinity of the site.

The mapping of PGE (1991) found the extent of landslide deposits to be considerably less than was interpreted by Nilsen (1975). As shown on the City of Morgan Hill Geologic Map (see excerpt above), colluvium occupies the topographic swale areas. Localized landslide deposits are mapped within the general vicinity (within hundreds of feet), and are generally shown as confined to topographic swale areas.

A regional landslide inventory map by Delattre and others (2006; excerpt provided in CE&G, 2015) largely supports the mapping of PGE (1991) insofar as is pertinent to the site vicinity. The nearest mapped landslide has an overall direction of movement that is westward, away from the slopes encompassing the site. A substantial spur ridge divides the portion of the regional slope affected by landsliding from the portion of the slope encompassing the site.

3.2 Faulting

No active faults are mapped as passing through the site in the general project vicinity. Several fault strands are mapped west of the Calaveras fault and east of the toe of the Diablo Range. Collectively, these faults are referred to as the Coyote Creek-Range Front fault zone, which consists of an anastomosing zone of variable width that juxtaposes different rock types. The closest mapped fault strand is shown by PGE (1991) as passing near the valley floor/toe-of-slope hinge, approximately 1,400 feet west of the site (see the dotted line at the extreme lower left corner of the excerpt from PGE (1991) shown above). This fault – the Range Front Fault of PGE (1991) -- was evaluated together with the Coyote Creek fault in depth as part of investigations for the Anderson Dam Seismic Retrofit Project (HDR, 2013). Summarizing, work by several investigators concluded that the fault is not seismically capable if it is even present as mapped. Seismicity is discussed further, below. Figure 3, Regional Fault Map, shows known active faults in the region.

3.3 Geohazard Mapping

The site is not mapped within a California Geological Survey (CGS) Earthquake Fault Rupture Hazard Zone (Bryant and Hart, 2007).

The site is not located within a fault rupture hazard zone established by the local jurisdiction (Morgan Hill General Plan 2035 Update, Draft Housing and Safety Element, accessed May 2016).

The site is shown on the City of Morgan Hill Ground Movement Potential Map (PGE, 1991) as lying within map unit "Ps," which is defined as "relatively unstable surficial deposits or bedrock materials including landside debris, colluvium, and weak bedrock, commonly less than about 10 feet thick on moderate to steep slopes. Subject to shallow, slow-moving landsliding and soil creep."

The site is not located within a California Geological Survey (CGS) Seismic Hazard Zone (CGS, 2006). These zones were established to trigger further evaluation (for certain projects) of the potential for seismically induced landsliding in hillside areas, and liquefaction potential in valley floor areas.

The site is mapped within a County of Santa Clara Landslide Hazard Zone; these zones are established in most hillside areas in order to help confirm that slope stability considerations are addressed in certain project classes (Santa Clara County Planning Dept. online GIS database at https://sccplanning.maps.arcgis.com, accessed May 2016).

The site is not mapped within a County of Santa Clara Fault Rupture Hazard Zone, Liquefaction Hazard Zone, Collapsible Soil or Dam Inundation hazard zone (see link above).

3.4 Regional Groundwater

Groundwater within the hillslope areas encompassing the site is commonly at tens of feet in depth below ground surface, though variable. We are not aware of regional groundwater contouring of sufficient detail to apply to this project. Widely scattered springs and seeps in the general vicinity are interpreted to represent the intersection of the local water table with the ground surface.

3.5 Seismicity

3.5.1 Active Faults

The East Dunne tank site is located within the greater San Francisco Bay Area, which is recognized as one of the more seismically active regions of California. The right-lateral strike-slip San Andreas fault system controls the northwest-southeast structural grain of the Coast Ranges and the Bay Area. The fault system marks the major boundary between two of earth's tectonic plates, the Pacific Plate on the west and the North American Plate on the east. The Pacific Plate is moving north relative to the North American plate at approximately 40 mm/yr in the Bay Area (WGCEP, 2003).

The transform boundary between these two plates has resulted in a broad zone of multiple, subparallel faults within the North American Plate, along which right-lateral strike-slip faulting predominates. In this broad transform boundary, the San Andreas Fault accommodates less than half of the average total relative plate motion. Much of the remainder in the greater South Bay Area is distributed across faults such as the San Gregorio-Hosgri, Monte Vista-Shannon, Sargent, Berrocal, Hayward (southern segment), Calaveras, Zayante-Vergeles, and Greenville fault zones.

Since the East Dunne tank site is in the seismically active San Francisco Bay Area, it will likely experience significant ground shaking (moment magnitude greater than 7.0) from one or more of the nearby active faults during the design lifetime of the project. Major seismic sources in the San Francisco Bay area include those summarized in Table 1. For major active faults within 50 km of the site, the distance from the site and the estimated maximum moment magnitude are listed.

Distances are estimated with respect to an approximate project center at latitude 37.13774°, longitude -121.59512°.

Two seismogenic (capable of generating significant earthquakes) earthquake faults near the site are the Calaveras fault (approximately 1.9 km [1.2 mi] east of the site, essentially coincident with the axis of Anderson Lake); and the San Andreas fault (approximately 19.6 km [12.2 mi] west of the site).

Table 1 - Distances to Selected Major Active Faults

Fault Name	Distance and Direction From Site to Fault
Calaveras (central segment)	1.9 km northeast
San Andreas	19.6 km southwest
Berrocal	15.4 km southwest
Sargent	16.5 km southeast
Zayante-Vergeles	25.0 km southeast
Monte Vista-Shannon	30.6 km northwest
Ortigalita	31.0 km northeast
Greenville	32.2 km northeast
Hayward (southern segment)	42.0 km northwest
San Gregorio	57.9 km southwest

3.5.2 Liquefaction and Seismic Densification

Soil liquefaction is a phenomenon in which saturated, cohesionless soils (generally sands) lose their strength due to the build-up of excess pore water pressure during cyclic loading, such as that induced by earthquakes. Soils most susceptible to liquefaction are saturated, clean, loose, fine-grained sands and silts. The primary factors affecting soil liquefaction include: 1) intensity and duration of seismic shaking; 2) soil type and relative density; 3) overburden pressure; and 4) depth to ground water.

The soil and groundwater conditions needed for soil liquefaction do not appear to be present in the site vicinity, and none of the onsite earth materials are considered susceptible to liquefaction. The soils encountered at the site are relatively thin (combined thickness of colluvium and uppermost severely weathered rock on the order of up to 10 feet in thickness), contain significant proportions of clay and silt and are relatively stiff in consistency. Additionally, shallow (within 50 ft bgs) groundwater conditions are not present in the site soils. Based on subsurface information collected during this investigation, we judge the potential for liquefaction at this site to be very low because the groundwater level is generally low, the granular soils locally present at the site are generally too dense to liquefy, and the clayey soils locally present at the site are sufficiently plastic and stiff to preclude liquefaction.

Seismic densification is the densification of unsaturated, loose to medium dense granular soils due to strong vibration such as that resulting from earthquake shaking. Materials considered susceptible to seismic densification were not encountered in our borings.

4. Site Investigation

4.1 Previous Investigations

As noted above, CE&G previously prepared a Preliminary Engineering Geologic Feasibility Evaluation report (CE&G, 2015). Geologic mapping included in that report was largely carried forward for this report, and refined on the basis of findings gathered during this investigation. Additional information regarding surface exposures is presented in CE&G (2015).

CE&G (2015) described degraded surface exposures of generally pebbly sandstone with variable fines content; and intervals of common cobble- to boulder-size rubble composed of basalt. The best exposures in the vicinity are provided by the road cut along East Dunne Avenue. Areas where basaltic cobbles and boulders were concentrated in the surficial colluvium were inferred to approximately mark the location of discontinuous basalt flows and/or breccia in the subsurface. These observations are consistent with regional exposures in the area.

4.2 Site Reconnaissance

CE&G performed field reconnaissance of the site on several dates, in coordination with representatives of the City of Morgan Hill, Kennedy/Jenks, and Mark Thomas & Co. No evidence of significant settlement, structural distress, erosion, stability problems, or maintenance problems were observed.

4.3 Subsurface Exploration

4.3.1 Scope of Explorations

CE&G prepared a preliminary subsurface exploration plan that showed planned boring locations. The preliminary plan was submitted to the City for review prior to execution of subsurface exploration.

Seven geotechnical borings and an additional probe were completed for the investigation of East Dunne Tank site to characterize the soil/bedrock conditions in the area of the tank and to evaluate anticipated excavation conditions near the upslope limit of the tank footprint. All borings were drilled using a track-mounted drilling rig. The locations of the borings were selected based on review of published geologic mapping; our own site geologic reconnaissance mapping (performed for CE&G [2015] and this investigation); evaluation of the locations of existing improvements (sanitary sewer) and the proposed improvements; access; environmental constraints; and public/pedestrian safety.

Prior to drilling, CE&G coordinated with the City regarding selection of the final locations of the borings. CE&G marked, and coordinated a USA (Underground Service Alert); obtained an encroachment permit through the City of Morgan Hill; obtained an exploratory boring permit from the Santa Clara Valley Water District; and obtained a hydrant water meter (through City of Morgan Hill DPW). The locations of the completed borings were marked in the field and recorded by measuring with a tape from established points of reference and by using a handheld GPS device. Following drilling, the completed borings were surveyed by the Mark Thomas & Co. surveying team, for plotting as shown on Figure 4, Vicinity Geologic Map.

The geotechnical borings were drilled by Britton Exploration on April 11-13, 2016, utilizing a track-mounted CME-55 drill rig. Surface conditions at all of the borings were similar, consisting of grassy hillslope terrain with surface gradients on the order of 17 to 20 degrees. The drill rig utilized a 6-inch solid stem auger, with tooling on hand to permit switchover to hollow stem or rotary wash tri-cone bit drilling depending on conditions encountered. The borings were drilled to depths ranging between approximately 20 and 52 feet below existing grade (B-1: 51.5 feet; B-2: 50.0 feet; B-3: 51.5 feet; B-4: 20.0 feet; B-5: 25.0 feet; B-6: 25.0 feet; B-7: 25.0 feet), with the additional probe (P-1) drilled to 40.0 feet below existing grade. Sampling protocol and boring depths were determined based upon geologic conditions; expected elevation of the tank and pump station pad; configuration of the planned retaining walls; and by materials encountered during the drilling operation.

Upon completion, the borings were backfilled with neat cement grout in accordance with the Santa Clara Valley Water District's permit criteria. Drilling spoils were distributed unobtrusively on site.

4.3.2 Logging and Sampling

The materials encountered in the borings were logged in the field by a CE&G geologist. The soils were visually classified in the field, office, and laboratory according to the Unified Soil Classification System (USCS) in general accordance with ASTM D2487 and D2488.

During the drilling operations, soil samples were obtained using one of the following sampling methods:

- California Modified (CM) Sampler; 3.0 inch outer diameter (O.D.), 2.5 inch inner diameter (I.D.) (ASTM D1586)

- Standard Penetration Test (SPT) Split Spoon Sampler; 2.0 inch O.D., 1.375 inch I.D. (ASTM D1586)

The samplers were driven 18 inches (unless otherwise noted on the boring logs) with a 140-pound automatic trip-hammer dropping 30 inches in general conformance with ASTM D6066 procedures. The number of blows required to drive the SPT or CM sampler 6 inches was recorded for each sample. The results are included on the boring logs in Appendix A. The blow counts included on the boring logs are uncorrected and represent the field values.

Soil samples obtained from the borings were packaged and sealed in the field to reduce the potential for moisture loss and disturbance. The samples were taken to CE&G's Oakland office for laboratory testing and storage.

4.3.3 Soil Conditions Encountered

Relatively uniform soil conditions were encountered in the borings. Subsurface soil conditions encountered in our borings were generally consistent with geologic mapping performed for CE&G (2015), except that the extent of hard basaltic flow and/or flow breccia encountered was less than anticipated within the area of improvements based on surface exposures.

None of the borings encountered existing fill on the undeveloped site, although previous disking for fire prevention, and faint remnant ranch/fire roads suggest that the upper approximately 1 foot of soil has locally been disturbed.

Colluvium – All the borings encountered colluvium. Texturally, the colluvium was field classified as generally lean clay to sandy lean clay (CL), with sandy fat clay (CH) described in B-4 and B-6. These deposits are firm and moist. The colluvium is inferred to be derived from the underlying Santa Clara Formation bedrock, and the transition between colluvium and the underlying severely weathered rock is gradual.

Santa Clara Formation bedrock – All of the borings bottomed in Santa Clara Formation bedrock. The dominant rock types encountered are sandstone, clayey sandstone, claystone, and sandy claystone. Scattered concentrations of gravel were noted either through behavior of the drill rig or visible in the samples. Boring B-1 encountered near refusal at a depth of approximately 41 feet. A switchover in drilling technique allowed the boring to penetrate to a depth of 51.5 feet at a very slow rate. The refusal was at first suspected to be due to a basaltic interval, however the few fragments retrieved indicated that hard, cemented sandstone had been encountered. Clayey sandstone with gravel was also encountered in B-5 and B-6. B-6 encountered an apparent interval of basalt within a thicker interval of sandstone with gravel that presented hard drilling.

B-1 was the only boring that encountered near-refusal. None of the borings (except B-6) recovered any basalt flow and/or breccia. Surface exposures indicate a greater proportion of basalt and breccia than was encountered by our borings. The geotechnical probe boring (P-1) location was selected to investigate whether the hard material encountered in B-1 extended into the proposed tank backcut. That material was not encountered in P-1.

Based on the relative lack of basaltic material suggested by B-1, B-2, and B-3 in contrast to surface concentrations of cobbles and boulders, we have adjusted the inferred limits of basaltic intervals on our Vicinity Geologic Map.

Slope gradients are distinctly steeper downslope of a topographic bench at approximately the location of B-6. We infer that the clayey sandstone with gravel encountered in B-6 corresponds to a slightly harder, stronger interval that "daylights" in the slope at approximately that elevation. Observed landsliding appears to be limited to the steeper slopes below this location.

For a more detailed description of the soils encountered in the borings, the logs of the borings and laboratory test results are included in Appendices A and B.

4.3.4 Groundwater Conditions Encountered

Groundwater was not found in any of the borings. Soil and bedrock colors observed in samples indicate consistently oxidized conditions, which suggests that the water table does not tend to fluctuate through the intervals drilled. Conversely, a fluctuating water table is likely to result in mottled coloration, and presence of green, gray, and blue hues that indicate reducing conditions.

4.4 Geotechnical Laboratory Testing

Laboratory testing was performed to obtain information regarding the physical and index properties of selected samples recovered from the exploratory borings. Tests performed included natural moisture content, dry unit weight, Atterberg Limits, grain size distribution, Caltrans corrosion testing, and triaxial unconsolidated undrained testing. Tests were completed in general conformance with applicable ASTM standards. The laboratory testing indicates that the Plasticity Index of the clay soil layers ranges between 11 and 41 percent for the samples tested. The results of the laboratory tests are summarized on the boring logs in Appendix A and in Appendix B.

4.5 Slope Stability Assessment

CE&G performed global stability analyses to develop an opinion regarding the stability of proposed bedrock cuts upslope of the proposed water tank and to develop recommendations for earth retention structures.

CE&G used stability software GSLOPE with search routines to evaluate the stability of the proposed cuts. CE&G then varied the depth of the failure surfaces to get insight into the stability of the proposed cut. Our evaluation indicates that shallow failure surfaces do not have adequate factor of safety. As a result engineered earth retention is required. Our analyses suggest that removal and reconstruction of the slope with geogrid reinforcement would increase the grading by 50 to 100% over that required to construct the tank pad. In addition, our analysis indicates that cantilever retaining walls do not provide an adequate factor of safety against global slope failures.

Based on our experience, the most economical solutions for large bedrock cuts like that proposed are tieback retaining walls or soil nail retaining walls. These wall types are generally used since the construction sequence results in a continuously stabilized excavation. Support of the slope is provided in a top-down manner as the excavation is being made so that when the pad elevation is reached, the walls are already installed. This expedites the construction schedule. Additionally, the construction sequence and methods are conducive to variable height permanent walls. In general, soil nail walls are more economical than tieback retaining walls.

The stability analyses are included in Appendix D.

5. Conclusions and Discussion

5.1 General Summary

Based on the results of our investigation, it is our opinion the site is geologically and geotechnically suitable for the proposed improvements shown on our Vicinity Geologic Map (Figure 4), provided the recommendations presented in this report are followed.

A review of our conclusions with respect to various geologic and geotechnical issues is presented below, beginning with landsliding/slope stability, since this is arguably the most important geologic hazard with respect to site suitability. Geotechnical recommendations for design and construction of the proposed improvements are presented in the "Recommendations" section of this report.

5.2 Landsliding

As described above, no evidence of deep-seated landsliding was detected at the site. Relatively restricted shallow sloughing (landsliding) has affected the colluvium in portions of the slopes south (downslope) of the site. Such shallow instability appears to have been associated with concentration of surface runoff in topographic swales.

In our judgment, the potential for deep-seated landsliding (involving bedrock) to adversely affect the site improvements is low under both static and seismic conditions. We base this on several lines of evidence, including: the presence of interlayered basaltic rocks in an overall favorable orientation within the rock sequence observed; the lack of evidence for previous deep-seated landsliding with areas of interlayered basaltic rocks in the general region; and the site's location outside of a topographic swale, with minimal contributing watershed upslope.

We also judge the potential for shallow-seated landsliding (under static and seismic conditions) to adversely affect the site improvements to be low, provided site improvements are appropriately designed and constructed and surface runoff is appropriately managed. There is a moderate potential for the mapped past shallow landsliding on the steeper slopes below (south of) the access road to reactivate under current site conditions. However, if surface drainage in this vicinity is appropriately controlled, the area will not receive the concentrated runoff that we judge to be a primary factor in the formation of this landsliding, which will lessen the potential for reactivation. Additionally, the proposed access road we understand will be supported along this interval with an outboard retaining wall deriving support from the relatively strong bedrock beneath the slide.

5.3 Seismic Hazards

Large magnitude earthquakes and strong ground shaking are likely to affect the project area within the design lifetime of the proposed improvements. Peak ground shaking parameters are presented below in Section 6.2 and should be considered in the design of the proposed improvements. Local ground-modifying effects of high intensity ground shaking are considered secondary seismic effects. Our review of these processes is presented below.

- We confirm our judgment that the potential for fault ground rupture or coseismic faulting to significantly affect the proposed improvements is low.
- We confirm our judgment that the potential for ridgetop fissuring, ridgetop shattering, ridgetop spreading or other seismically induced ground deformation to significantly affect the proposed improvements is low.
- We confirm our judgment that the potential for soil liquefaction to significantly affect the proposed project is low.

5.4 Soil Permeability

We understand the design team requires an estimate of the on-site soil permeability that will be used in the site drainage assessment. The permeability of the on-site soil was not tested. However, based on the type and consistency of the soils encountered at the site during the subsurface exploration, the following permeability estimates are provided for use in estimating the amount of rainfall that will infiltrate into the site soils.

The types of soils encountered at the site in the upper colluvial soil included primarily Sandy Lean Clay (CL) and Sandy Fat Clay (CH), for which a typical permeability value of 7 x 10-5 in/hr is representative. Below the colluvium, some of the weathered bedrock that consists of sandstone has a higher permeability. The sandstone typically has been weathered to the consistency of Silty Sand (SM) and Clayey Sand (SC), for which a value of between 0.04 and 4 x 10-4 in/hr may be used. Where the bedrock consists of claystone, weathered to Sandy Clay (CL), the value above for the colluvial soil may be used.

5.5 Geotechnical Considerations

Significant geotechnical issues that will affect the design and construction of the proposed water tank, retaining walls, and access road are as follows:

- **Water Tank Foundation** – In order to reduce the potential for differential settlement of water tank foundations, we recommend that tank foundations be extended into bedrock materials. This is conceptually shown in Figure 5, Geologic Cross-Section A-A' and detailed recommendations are provided in Section 6.3.

- **Retaining Walls** –We recommend that the proposed retaining walls around the uphill side of the tank pad be designed as tieback or soil nail retaining walls. As an alternative, cantilever retaining walls utilizing spread footings that bear in competent bedrock materials may be considered. Detailed recommendations are provided in Section 6.6, Retaining Wall Design.

- **Surface Water Drainage** – Localized shallow landsliding, gullying and erosion have occurred within the central parts of the swale areas immediately south of the tank access road. Surface drainage improvements should be designed to adequately collect and accommodate the volumes of water that reach these drainages.

- **Rippability** – Subsurface exploration was completed using primarily hollow stem augers and only encountered drilling refusal in Boring B-1 below a depth of 40 feet, which is below the planned tank excavation. Based on the subsurface exploration, the majority of soil and bedrock underlying the project site is anticipated to be excavated with conventional heavy earthwork and excavation equipment. The need for jack hammers, hoe rams or blasting is not currently anticipated for the majority of the planned excavations. However, such equipment may be necessary in isolated locations.

6. Recommendations

6.1 Earthwork

6.1.1 Clearing and Stripping

Prior to grading, areas that will support foundations, concrete slabs-on-grade, pavements or engineered fill should be cleared of all deleterious material that may be present at the site. The root systems of trees designated for removal should be completely grubbed and removed. All deleterious material generated during the clearing operation should be removed from the construction areas.

After clearing, soil surfaces should be stripped of all vegetation and organic material. Organic laden soils are defined as soils with more than 3 percent by weight of organic content. The required stripping depth should be determined in the field by the geotechnical engineer at the time of construction. For planning purposes, an average stripping depth of 3 inches may be assumed. Organic laden topsoil can be stockpiled for reuse in the upper 12 inches of landscape areas or removed from the construction areas.

6.1.2 Excavations

Excavations for this site will include cuts for the water tank pad, cuts along the access road alignment, excavation of water tank and retaining wall foundations; excavations for keying and benching of fills; and trenching for and utility lines. The excavation for the water tank pad is expected to be up to approximately 34 feet below the existing grade. The tank pad retaining walls will likely be required prior to the construction of the tank foundation. Excavation for the access road is anticipated to be up to 12 feet below the existing grade.

The stability of temporary excavations, braced or unbraced, is the responsibility of the contractor. All excavations and shoring systems should meet the minimum requirements given in the State of California Occupational Safety and Health Standards, latest edition.

6.1.3 Cut and Fill Slopes

Permanent cut slopes in colluvial soil should be constructed at inclinations no steeper than 2-½:1 (horizontal:vertical). Final cut slopes in bedrock should be constructed at inclinations no steeper than 2:1 (horizontal:vertical). All permanent cut slopes should be less than 10 feet in height. Cuts slopes over 10 feet high should be reduced in height by designing retained walls. Final fill slopes should be constructed at inclinations no steeper than 2:1 (horizontal:vertical) and should be limited to a maximum vertical fill depth of 10 feet. Fill slopes should be overbuilt and trimmed` back to their final configurations.

Pavements should be separated at least 2 feet horizontally from the crests of all cut slopes and fill slopes.

6.1.4 Dewatering

Perched and shallow ground water will not likely be encountered in the excavations. Therefore, the need for temporary dewatering systems, such as sloping excavations to a sump pump location, trenching from the base of excavations to discharge water by gravity flow, or other means are not currently anticipated. If the need arises, design of construction dewatering should be determined by the contractor in consultation with our field representative at the time of construction.

6.1.5 Subgrade Preparation

Subgrade preparation should be performed after stripping and any necessary excavations have been performed. Subgrade soil in areas to receive engineered fill, foundations, or pavements should be scarified to a minimum depth of 8 inches, moisture conditioned and compacted to the recommendations presented in Section 6.1.7. Prepared soil subgrades should be non-yielding when proof-rolled by a fully loaded water truck or equipment of similar weight.

Subgrade preparation should extend a minimum of 5 feet beyond the outermost limits of the proposed improvements. After the subgrades have been prepared, the areas may be raised to design grades by placement of engineered fill.

If unstable, wet or soft soil is encountered, the soil will require processing before compaction can be achieved. When construction schedule does not allow for air-drying, other means such as lime treatment, over-excavation and replacement, geotextile fabrics, etc. may be considered to help stabilize the subgrade. The method to be used should be determined at the time of construction based on the actual site conditions. We recommend obtaining unit prices for subgrade stabilization during the construction bid process.

6.1.6 Material for Engineered Fill

On-site soils with an organic content of less than 3 percent by weight, free of any hazardous or deleterious materials, and meeting the gradation requirements below may be used as general engineered fill to achieve project grades, except when special materials (such as drainage material) are required.

Engineered fill material should not contain rocks or lumps larger than 6 inches in greatest dimension, should not contain more than 15 percent of the material larger than 2½ inches, and should contain at least 20 percent passing the No. 200 sieve.

All import fills should be approved by the project geotechnical engineer prior to delivery to the site. At least five (5) working days prior to importing to the site, a representative sample of the proposed import fill should be delivered to our laboratory for evaluation.

Possible sources for import fill include the Aromas Quarry located south of Gilroy, California and Stevens Creek Quarry located near Cupertino, California.

6.1.7 Engineered Fill Placement and Compaction

Engineered fill should be placed in horizontal lifts each not exceeding 8 inches in uncompacted thickness, moisture conditioned to the required moisture content, and mechanically compacted to the recommendations below. Relative compaction or compaction is defined as the in-place dry density of the compacted soil divided by the laboratory maximum dry density as determined by ASTM Test Method D1557, latest edition, expressed as a percentage. Moisture conditioning of soils should consist of adding water to the soils if they are too dry and allowing the soils to dry if they are too wet.

Engineered fills consisting of on-site soils or imported soils of low expansion potential should be compacted to no less than 90 percent relative compaction with moisture content between about 1 and 3 percent above the laboratory optimum value. In pavement areas, the upper 6 inches of subgrade soil should be compacted to no less than 95 percent relative compaction with moisture content between 1 and 3 percent above the optimum value. Aggregate base in vehicle pavement areas should be compacted at slightly above the optimum moisture content to no less than 95 percent relative compaction.

For fill to be placed on an existing slope with an inclination of 5:1 (horizontal:vertical) or steeper, the fill should be keyed and benched into the existing slope. Toe keys should extend a minimum of 2 feet into the bedrock material and have a width of 8 feet or 1½ times the width of the compaction equipment, whichever provides a wider excavation. Toe keys should slope toward their backs with a slope of at least 2 percent. Benches should be created by cutting a minimum of 6 feet into the existing slopes as the new fill is being placed. Vertical spacing of benches should not be more than about 6 feet. The materials excavated from the benches can be mixed with the slope fill and the fill should be compacted to the requirements in this section.

6.1.8 Utility Trench Excavation and Backfill

Utility trenches will likely extend through recompacted engineered fill in some cases, or native soil or bedrock. Utility trenches in bedrock material should be able to stand near vertical with minimal bracing.

Excavations should be constructed in accordance with the current CAL-OSHA safety standards and local jurisdiction. The stability and safety of excavations, braced or unbraced, is the responsibility of the contractor.

Bedding material, extending from the bottom of the trench to about 1 foot above the top of pipe, may consist of free-draining sand (less than 5% passing a No. 200 sieve), lean concrete or sand cement slurry. Sand if used as bedding should be compacted to no less than 90 percent relative compaction. Jetting of trench backfill shall not be allowed. If sand is used as bedding in utility lines located on slopes, soil plugs should be provided at about 30 feet intervals to reduce the potential for the utility trenches to serve as a conduit for water.

6.1.9 Wet Weather Construction

We recommend that earthwork not be performed during wet weather seasons. If site grading and construction is to be performed during the rainy periods, the owner and contractors should be fully aware of the potential impact of wet weather. Rainstorms could cause unstable excavations, delay to construction and damage to previously completed work by saturating compacted pads or subgrades, or flooding excavations.

Earthwork during rainy months will require extra effort and caution by the contractors. The grading contractor should be responsible to protect his work to avoid damage by rainwater. Standing pools of water should be pumped out immediately. Construction during wet weather conditions should be addressed in the project construction bid documents and/or specifications. We recommend the grading contractor submit a wet weather construction plan outlining procedures they will employ to protect their work and to minimize damage to their work by rainstorms.

6.1.10 Erosion Control

Disturbing areas around the project site should be minimized as much as possible. Areas disturbed by construction activities should be protected from erosion by hydroseeding and/or installing erosion control mats.

The tops of fill or cut slopes should be graded in such a way as to prevent water from flowing freely across the face of the slopes. A positive gradient away from the tops of slopes should be provided to direct surface water runoff away from the slopes to suitable drainage points. Completed slopes should be provided with erosion control measures prior to the winter season following grading.

Because the existing bedrock is relatively nutrient-poor, it will be difficult for vegetation to become properly established, resulting in a higher potential for slope erosion. Revegetation of graded slopes can be aided by retaining the organic-rich strippings within the upper few inches of on-site soil during the site stripping operations and spreading these materials in a thin layer (approximately 6 inches thick) on the graded slopes prior to the winter rains and following rough grading. When utilizing this method, it may be possible to reduce the amount of hydroseeding. All landscaped slopes should be maintained in a vegetated state after project completion. The use of native drought-tolerant vegetation is recommended. No pressurized irrigation lines should be placed on or near the tops of graded slopes.

6.2 Seismic Design Parameters

Because of the uncertainty of when and where earthquakes will occur, the extent of potential seismic damage to the water tank facility over their expected design life is difficult to predict. Seismic design parameters were determined based on soil type, design earthquake magnitude, and peak ground acceleration. The soil type was determined using an interactive map on the United States Geologic Survey (USGS) Earthquake Hazards Program website (USGS, 2015). The use of the National Earthquake Hazards Reduction Program (NEHRP) deaggregation, provided by the USGS, determined that a design earthquake with magnitude 6.5 and peak ground acceleration (PGA) of 0.73g should be used for seismic design. This value of PGA is based on a 475-year return period. The following seismic design parameters are from Chapter 16 of the 2013 California Building Code for Site Class C type soils (California Building Code, 2013).

Table 2 - Seismic Design Parameters

Item	Factor or Coefficient	Value	CBC 2013* Table/Figure
Site Class Definition	Site Class	C	Table 1613.5.2
0.2 Second Spectral Response Acceleration	S_S	2.165g	Figure 1613.5(3)
1.0 Second Spectral Response Acceleration	S_1	0.827g	Figure 1613.5(4)
Values of Site Coefficient	F_a	1.0	Table 1613.5.3(1)
Value of Site Coefficient	F_v	1.3	Table 1613.5.3(2)
Designed Spectral Response Acceleration for Short Periods	S_{DS}	1.443	Equation 16-38 (S_{DS}=2/3(F_a S_S))

Item	Factor or Coefficient	Value	CBC 2013* Table/Figure
Designed Spectral Response Acceleration for 1-Second Periods	S_{D1}	0.717g	Equation 16-40 (SDS=2/3(Fv S1)

6.3 Water Tank Foundation

We recommend the proposed tank be supported by a reinforced concrete ring foundation bearing in competent bedrock. The ring foundation may be designed to impose an allowable soil bearing pressure of 6,000 pounds per square foot. The ring footings should be embedded at least 24 inches below pad grade or lowest adjacent grade, whichever provides a deeper embedment. Where the ring is less than 5 feet horizontally from a slope it should be deepened to extend at least 24 inches into competent bedrock, as verified in the field by an engineer or geologist from our office (See Figure 5).

Ring walls should be reinforced to resist hoop stresses within the foundations. Hoop stresses may be calculated by assuming an outward lateral pressure equal to one-half the vertical pressure acting on the adjacent subgrade inside the ring wall.

Concrete should be placed only in excavations that are clean and free of loose soil and debris. All foundation excavations should be observed by a member of our staff to verify that adequate foundation bearing soils have been reached.

Soil resistance to lateral loads for the foundation will be provided by a combination of frictional resistance between the bottom of the footing and underlying soils and by passive pressures acting against the embedded sides of the footing. For frictional resistance, an ultimate coefficient of friction of 0.44 may be used for design. In addition, an ultimate passive lateral bearing pressure equal to an equivalent fluid pressure of 425 psf/ft may be used, provided the footings are poured tight against undisturbed competent bedrock. These values may be used in combination without reduction. The passive pressure can be assumed to act from the top of the lowest adjacent grade if the ring foundation is surrounded by pavements or concrete or at a depth of 1 foot below grade in unpaved areas. Total post-construction settlement of the tank foundation is expected to be less than 1 inch.

Ring foundations should be constructed and backfilled in consideration of the tank manufacturer's specifications. Our firm should be commissioned to review the foundation plans to determine if our recommendations are incorporated in the design. Our representative should observe the foundation excavations to determine if the excavations extend into suitable bearing material.

6.4 Pump Station Foundations

The proposed pump station structure is anticipated to be constructed over an engineered fill pad (see Figures 4 and 5) and may be supported on conventional shallow foundations founded on compacted engineered fill or undisturbed native soils. The footings should be embedded at least 18 inches below rough pad grade or lowest adjacent finish grade, whichever provides a deeper embedment. Footings may be designed using a net allowable soil bearing pressure of 2,500 pounds per square foot (psf) for dead plus live loads. This value may be increased by one-third when considering short-term loads such as wind and seismic forces. Reinforcement for the foundations should be determined by the project structural engineer.

Lateral loads may be resisted by a combination of friction between the bottom of foundations and the supporting subgrade in engineered fill, and by passive resistance acting against the vertical sides of the foundations. An ultimate friction coefficient of 0.35 may be used for friction between the foundations and supporting subgrade. Ultimate passive resistance equal to an equivalent fluid weight of 350 pounds per cubic foot (pcf) acting against the embedded sides of the foundations may be used for design purposes. The passive pressure can be assumed to act starting at the top of the lowest adjacent grade in paved areas. In unpaved areas, the passive pressure can be assumed to act starting at a depth of 1 foot below grade. It should be noted that the passive resistance value discussed above is only applicable where the concrete is placed directly against undisturbed soil or engineered fills. Voids created by the use of forms should be backfilled with soil compacted to the requirements given in this report or with concrete.

Total post-construction settlement of the structure is anticipated to be less than 1/2 inch.

To maintain foundation support, footings located near utility trenches oriented parallel to the structure should be deepened so that the bearing surfaces are below an imaginary plane having an inclination of 1½:1 (horizontal to vertical). This imaginary plane should be drawn extending upward from the bottom edge of the adjacent utility trench.

Our firm should be commissioned to review the foundation and utility plans to determine if our recommendations are incorporated in the design. Our representative should observe the foundation excavations to determine if the excavations extend into suitable bearing material.

6.5 Concrete Slabs-On-Grade

Concrete slabs-on-grade are anticipated for the interior floor within the pump station structure. Preparation of subgrade soil and placement and compaction of engineered fill should be as outlined in the "Earthwork" section of this report. Soil subgrade should be maintained in a moist condition prior to pouring the concrete slab.

Interior concrete slabs-on-grade where vapor transmission through the slabs is undesirable, should be underlain by at least 4 inches of capillary break material such as free draining, clean drain rock or 3/8 inch pea gravel. A visqueen should be placed over the capillary break material. The visqueen should be a high quality polymer at least 10 mils thick that is resistant to puncture during slab construction. Typically, the membrane and the slab are separated by 2 inches of sand. For interior or exterior slabs where moisture transmission through the slabs is not an issue, the above recommended capillary break section is optional.

A lower water-cement ratio (0.45 to 0.50) will also help reduce the permeability of the concrete slab.

For on-site exterior flatwork where moisture transmission through the slabs is not an issue, concrete slabs may be constructed directly on the compacted soil subgrade. If a concrete slab is used for the driveway, we recommend the slab be underlain by a minimum of 6 inches of Class 2 aggregate base compacted to no less than 95 percent relative compaction.

Exterior concrete slabs-on-grade should be cast free from adjacent footings or other non-heaving edge restraints. This may be accomplished by using a strip of 1/2-inch asphalt-impregnated felt divider material between the slab edges and the adjacent structure. Construction and/or control joints should be provided in concrete slabs. Continuous reinforcing or dowels at the construction and control joints will help reduce differential slab movements.

6.6 Retaining Wall Design

Retaining walls are currently proposed at the site and will include: a) retaining walls around the upslope side of the water tank pad; and b) retaining wall along the downslope edge of the access road.

Based on our topographic profiling and topography provided by Kennedy/Jenks, we understand that the upslope side of the water tank pad will be supported by tiered retaining walls between 12 and 15 feet tall. The height of the retaining walls will depend largely on the height of cuts in the slope above the upper wall and the gradient of the slope between the walls. We understand that the access road retaining walls will be less than about 6 feet tall. We request the opportunity to review the locations of proposed walls to verify that the following design parameters apply to the wall locations.

Retaining walls must be designed to resist static earth pressures due to the supported soil and bedrock, surcharge pressures induced by loads close to the walls, and seismic loads. For this project, we recommend the walls be designed using the lateral pressures presented below.

The effects of surcharge loads close to the walls should be included in the wall design. While the surcharge loads on the tank pad retaining walls will likely be minimal, the surcharge loads on the access road retaining wall will include heavy equipment used during construction and on occasion for repair or maintenance at the tank site. For uniform vertical surcharge loading behind the walls, the additional lateral surcharge pressure should be 1/3 of the vertical surcharge load. For other surcharge loads, please contact our office.

6.6.1 Active Soil Pressures

Active soil pressures may be used for the design of unrestrained walls where the top of the wall is allowed to deflect and minor settlement of wall backfill is tolerable. These may include the access road retaining walls and conventional cantilever retaining walls supporting the upslope side of the water tank pad. Unrestrained walls with drained backfill conditions may be designed using the following active soil pressures:

Table 3 – Active Equivalent Fluid Pressures

Backfill Slope	Equivalent Fluid Pressure for Soil	Equivalent Fluid Pressure for Weathered Bedrock
Horizontal	40 pcf	37 pcf
2:1 (hor:vert)	60 pcf	53 pcf

6.6.2 Seismic Design Increment

As a result of earthquake shaking, the soil or bedrock behind the retaining walls will exert an additional horizontal force on the walls. We recommend using an additional equivalent fluid pressure of 40 pcf to model the earthquake-induced force on the walls, applied at 1/3xH (H= design wall height) up from the base of the wall.

6.6.3 Soil Nail Retaining Wall

Soil nail retaining walls are to be used above the water tank pad. The following recommendations should be incorporated in the design. We understand the project structural engineer will design the soil nail retaining wall based on design values provided herein, which are intended for low-pressure grouted soil nails.

Table 4 – Soil Nail Design Recommendations

Soil Nail Reinforcement	
Minimum Reinforcement Bar Size	#8 for bar anchors
Minimum Grout Hole Diameter	6 inch
Corrosion Protection	Double corrosion protection
Soil Nails	
Minimum Length	30 feet beyond unbounded zone
Inclination	15 - 20 degrees
Unbonded Length	Determine graphically assuming a minimum unbounded zone taken as a 2H:1V from the base of the lowest retaining wall. This added unbonded length is intended to address global stability of the retaining walls.
Soil Nail Spacing	4 to 5 feet in both the vertical and horizontal directions

The following points should be incorporated into the design and construction of soil nail retaining walls:

- The design should be based upon the methods described in the latest Federal Highway Administration manual titled, Geotechnical Engineering Circular No. 7, Soil Nail Walls-Reference Manual" (FHWA-NHI-14-007).
- The design of the soil nails should use the computer program SNAP-2 referenced in the FHWA manual or using a comparable software program that can be shown to conform to the recommended design procedure.
- As noted in Chapter 5 of the FHWA manual, the design needs to address the failure modes shown on Figure 5.8 of the manual. The failure modes include: internal stability, global stability, the presence of weak layers, pullout, tensile overstress of the soil nails, and facing failures.
- The following soil and bedrock parameters should be used for design of the soil nail retaining wall(s).
- All aspect of design, construction, and testing and inspections shall be in general conformance with the FHWA manual.

Table 5 –Soil Nail Design Parameters

Ultimate Soil-Grout Bond Strength (Assuming augered soil nail installation)	15 psi
Minimum diameter	6 inches
Effective Cohesion Values (Colluvium – Sandy Clay)	1500 psf
Effective Friction Angles (Colluvium – Sandy Clay)	27 degrees
Effective Cohesion Value (Weathered Sandstone	300 psf
Effective Friction Angle (Weathered Sandstone Bedrock)	36 degrees
Wall / Soil Interface Friction Coefficient	0.50
Soil Nail Inclination	15 - 20 degrees

6.6.4 Soldier Pile Retaining Walls

If soldier cast-in-drilled-hole (CIDH) piles and lagging are to be used to retain slopes, the retaining walls may be supported by a drilled foundation system designed according to the criteria outlined below. The proposed retaining walls may be supported on a CIDH pile system that penetrates into bedrock.

CIDH piles should be designed to derive their vertical supporting capacity from skin friction between the pile shafts and the surrounding earth material. Piles should have a minimum diameter of 18 inches, and should extend to a minimum depth of 10 feet and a minimum of 6 feet into bedrock, whichever provides a deeper embedment. Center to center spacing of the piles should be a minimum of three pile diameters.

Piles should be reinforced throughout their entire length and designed by the structural engineer. As a minimum, we recommend four No. 5 reinforcing bars.

Resistance to lateral loads may be calculated based on passive soil pressure acting against the piles. For dead plus live loads, the ultimate passive soil resistance may be calculated using an equivalent fluid weight of 275 pounds per square foot acting over a width of 1-½ pile diameters on the portion of the piles in bedrock. This passive soil resistance assumes a 3:1 (horizontal:vertical) slope below the wall. The top of the passive pressure zone should be assumed to begin at the top of the bedrock or at the bottom of the active pressure zone, whichever is deeper. The top of the bedrock is estimated to be 6 feet below the ground surface in the area of the access road.

Prior to the placement of steel and concrete, the bottom of pile excavations should be cleaned of loose soil. If groundwater is encountered during drilling, it should either be sumped from the holes or the concrete should be placed by the tremie method. Our field representative should be present during foundation drilling to verify that the piles extend sufficiently into the recommended earth materials.

We should be commissioned to review the retaining wall design plans to determine if our recommendations are incorporated in the design. We should observe the foundation excavations to determine if the excavations extend into suitable bearing material. This will involve intermittent to full time observation during pile drilling, and intermittent observation of the grade beam and footing excavations prior to placement of reinforcing steel and concrete.

We anticipate that wood lagging will be incorporated in the retaining wall design. The base of the lagging should extend at least 2 feet below the lowest adjacent final grade. If this is not attainable, a slurry trench should be constructed at the base of the lagging. At least 3 inches of the edge of the lagging should be in contact with the wide flange beam in the piles.

The top of the lagging should extend between 6 and 12 inches above the final grade above the retaining wall in order to prevent surface water runoff from discharging over the slope.

6.6.5 Cantilever Retaining Walls

In areas where shallow bedrock is present below the retaining wall, a conventional cantilever retaining wall may be used. For this case, an allowable bearing pressure of 3,500 psf DL + LL may be used. For resistance to lateral loads, an ultimate passive equivalent fluid pressure of 425 psf may be used. An ultimate friction value of 0.40 may also be used to resist lateral loads.

6.6.6 MSE Walls

We understand Mechanically Stabilized Earth (MSE) retaining walls are being considered for support along the access road. The following parameters are recommended for use in the design of MSE walls:

The following parameters can be used in the design of MSE walls.

- Effective friction angle, φ' = 32 degrees
- Effective Unit Weight, γ' = 125 pcf

- Effective cohesion, c' = 0 psf

We anticipate the MSE walls will be reinforced with geosynthetic reinforcement. The native soils may be used in the construction of the MSE walls.

If geosynthetic reinforcement is to be used, the backfill material should meet the following gradation requirements:

Table 6 – MSE Wall Backfill Gradation Requirements

Sieve Size	Percent Passing
6-inch	100%
3-inch	75% to 100%
No. 4	50% to 80%
No. 40	0% to 60%
No. 200	0% to 20%

6.6.7 Retaining Wall Drainage

A subdrain should be constructed on the backfill side of the retaining walls. The drain should consist of Class 2 Permeable drainage material complying with Section 68 Caltrans Standard Specification, latest edition. The permeable material should be at least 12 inches wide and should extend up the back of the wall to within 12 inches of the top of the wall. Native clayey soil or aggregate base and asphalt pavement should be used for the upper foot of wall backfill and should cap the drainage material. As an alternative to the Class 2 Permeable drainage material, a clean coarse gravel or drain rock may be used. If coarse gravel or drain rock is selected as a drainage material it should be separated from all adjacent soil by an engineering filter fabric such as Mirafi 140N, or a similar geotextile. Enough space should be provided between the laggings to allow seepage through the face of the wall.

In lieu of the above mentioned drain rock, a prefabricated drainage composite such as "CCW MiraDRAIN 6000XL" or equivalent may be used for drainage behind the retaining walls. This drainage composite should be installed on the back of the tieback wall at least 1 foot below the ground surface and should be wrapped around a drainage pipe at the base of the wall.

Backfill against retaining walls should be compacted as discussed in the "Earthwork" Section of this report. Over-compaction should be avoided because increased compaction effort can result in lateral pressures significantly higher than those recommended above. Backfill placed within 5 feet of the walls should be compacted with hand-operated equipment.

6.6.8 Construction Considerations

It is anticipated that difficult drilling conditions could be encountered during the tieback installation operation and the contractor should provide suitable equipment to install tiebacks to the depths indicated on the plans. It is recommended that considerations such as the use of additional specialized equipment be fully evaluated by the contractor during the bidding process.

Free groundwater was not encountered during the exploratory drilling at the site and based on our review of available groundwater data for the area, it is not anticipated to be encountered during construction.

For the soil nail wall construction, localized sloughing of the retaining wall cut slope may occur before the shotcrete has been applied to the slope. While there is a low likelihood for this to occur, the contract may consider using Stay Forms to provide a surface against which the shotcrete may be applied. Following the curing of the shotcrete, the void behind the form should be backfilled with low strength concrete flowable fill to within 12 inches below the final grade. The upper 12 inches should be backfilled with compacted native soil.

6.7 Surface Drainage

Engineering design of grading and drainage at the site is the responsibility of the project Civil Engineer. We recommend the following be considered by the project Civil Engineer and incorporated into the project plans where appropriate. Collected surface water within the swales crossed by the access road should be conveyed by a pipe to a discharge point below any active sliding or gullying, and appropriate energy dissipaters should be constructed at the outlet points to reduce the potential for future slope instability or erosion/gullying.

Generally, surface drainage should be directed away from structure foundations, concrete slabs-on-grade, fill slopes and pavements and directed towards suitable discharge locations below the graded pad areas. Ponding of surface water should be avoided by establishing positive drainage away from all improvements. Collected surface water should be discharged into a pipe or towards drainage structures and the water carried to a suitable discharge point. Collected surface water runoff should not be discharged directly on slopes.

6.8 Soil or Bedrock Corrosion Potential

Two samples from the borings were tested to provide general information regarding corrosion potential of site materials. Test results from Cooper Testing Lab are included in Appendix C of this report and summarized in Table 7 below. Project designers should review the report and incorporate into the design as appropriate. Additional testing may be necessary to address specific project needs.

Table 7 – Corrosion Test Results

Chemical Analysis	Test Method	Sample Boring Number and Depth		Corrosion Classification
		B-1 15.5 & 20.5 feet	B-2 24 & 29 feet	
pH	Cal 643	8.3	8.3	not corrosive
Chloride (ppm)	Cal 422 Mod.	N.D.	N.D.	not corrosive
Sulfate (ppm)	Cal 417 Mod.	N.D.	N.D.	not corrosive
Minimum Resistivity (ohm-cm)	Cal 643	790	1,000	corrosive

According to Corrosion Guidelines Version 2.1, dated January 2015, prepared by Corrosion and Structural Concrete Field Investigation Branch, Materials Engineering and Testing Services, Division of Engineering Services, California Department of Transportation, a site is considered to be corrosive to structural elements if one or more of the following conditions exist for the representative soil samples taken at the site.

> *Chloride concentration is 500 ppm or greater, sulfate concentration is 2,000 ppm or greater, or the pH is 5.5 or less.*

Based on the above guidelines and laboratory test results, the samples tested are “not corrosive.”

7. Limitations

The findings and conclusions of this report are based upon information provided to us regarding the existing improvements, our geologic reconnaissance, subsurface conditions described on the boring logs, the results of the laboratory testing program, interpretation and analysis of the collected data, and professional judgment.

It is the client's responsibility to ensure that recommendations contained in this report are carried out during the design and construction phases of the project.

Site conditions described in the text of this report are those existing at the time of our last field reconnaissance and are not necessarily representative of the site conditions at other times or locations.

The findings of this report should be considered valid for a period of five years unless the conditions of the site change. After a period of three years, CE&G should be contacted to review the site conditions and prepare a letter regarding the applicability of this report.

The evaluation or identification of the potential presence of hazardous materials at the site was not requested and was beyond the scope of this investigation and report.

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FIGURES

BASEMAP REFERENCE

1. ORTHOIMAGERY FROM ESRI.
2. STREET CENTERLINES FROM CALTRANS GIS DATA, 2016

0 0.25 0.5 1
MILES

N

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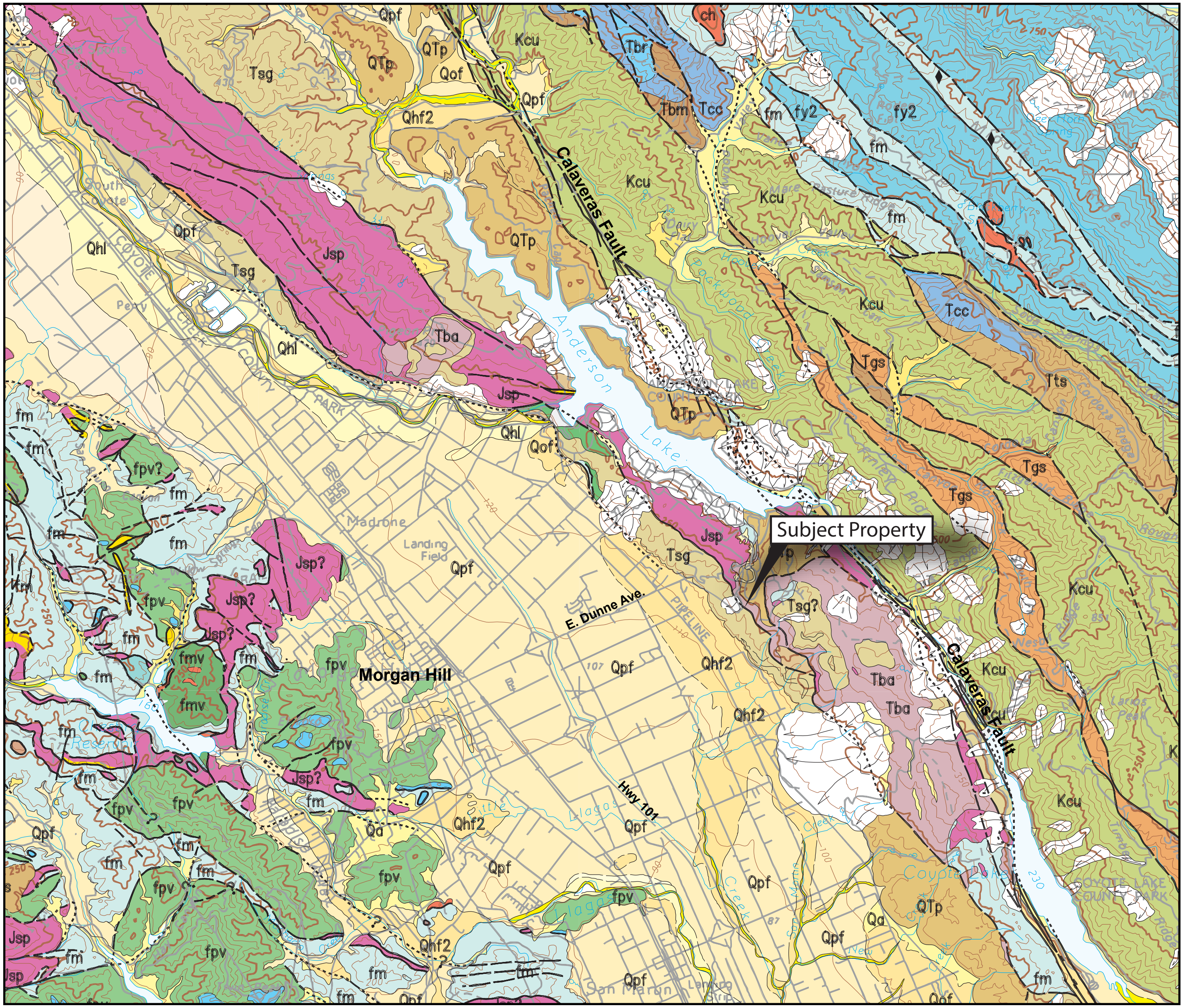
SITE LOCATION MAP

EAST DUNNE TANK SITE
MORGAN HILL

160200	AUGUST 2016	FIGURE 1

M:\2016\160200 Kennedy Jenks-East Dunne Tank\GIS\160200_Fig1_Site Location Map.mxd; 6/6/2016; kdrozynska



EXPLANATION

EARTH MATERIALS

- Landslide deposit
- **Qhl** Levee deposit
- **Qhf2** Older alluvial fan
- **Qpf** Alluvial fan deposit
- **Qof** Older alluvial fan deposits
- **QTp** Packwood Gravels
- **Tsg** Silver Creek Gravels
- **Tba** Basalt
- **Tgs** Glauconitic sandstone and mudstone
- **fm** Melange
- **fpv** Basaltic volcanic rocks
- **Kcu** Sandstone, mudstone, conglomerate
- **Jsp** Serpentinized harzburgite and dunite

SYMBOLS

- Contact, dashed where approximately located, dotted where concealed
- Fault, dashed where approximately located, dotted where concealed

N

Base Map: Modified from Wentworth, et.al, 1999

0 2 4

Approximate Scale in Miles

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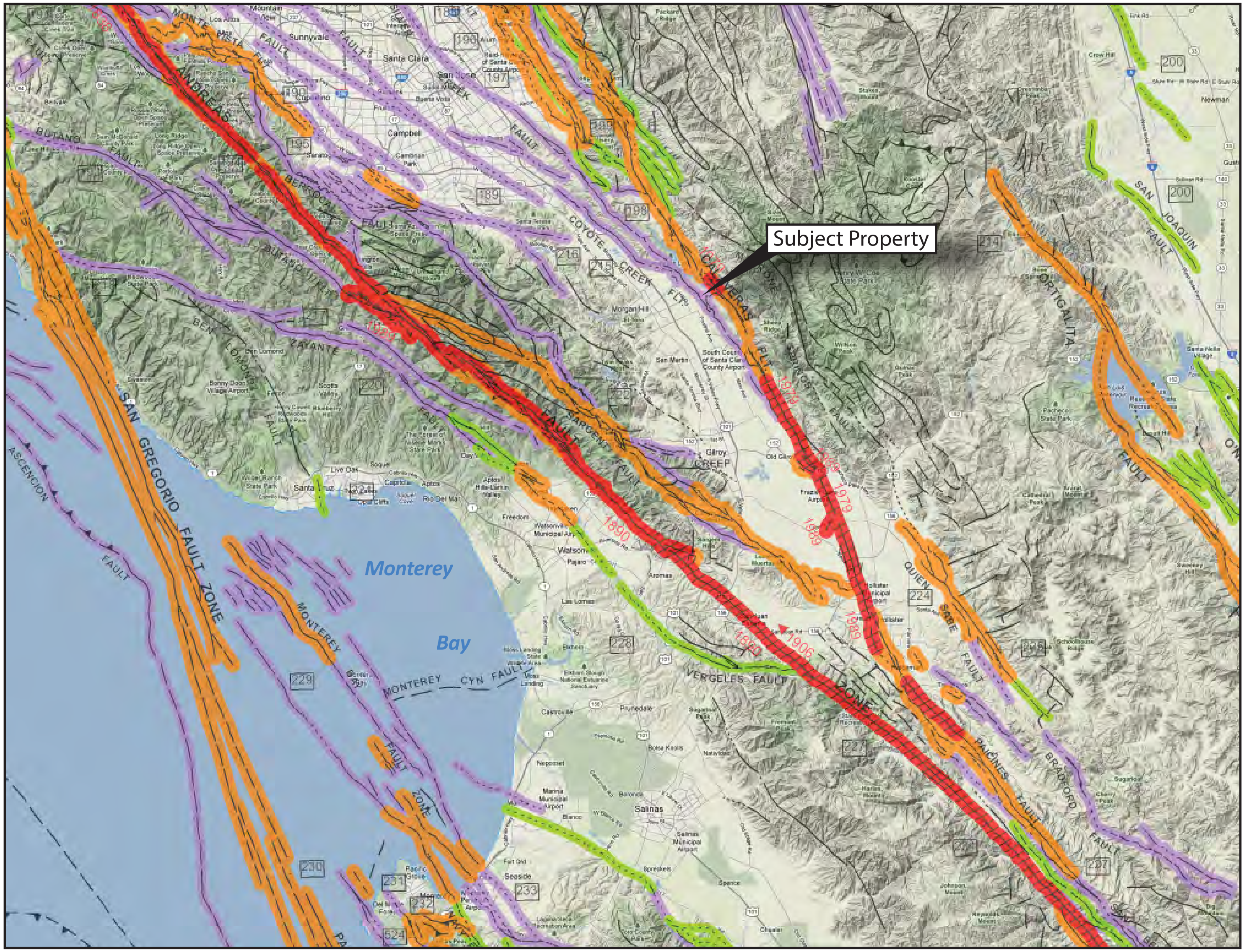
REGIONAL GEOLOGIC & INDEX MAP

EAST DUNNE TANK SITE
MORGAN HILL, CALIFORNIA

160200	AUGUST 2016	FIGURE 2



Base Map: Modified from Jennings and Bryant, 2010

EXPLANATION

Historic Fault

Holocene Fault

Quaternary Fault

Late Quaternary Fault

Pre-Quaternary Fault

N

0 10 20

Approximate Scale in Miles

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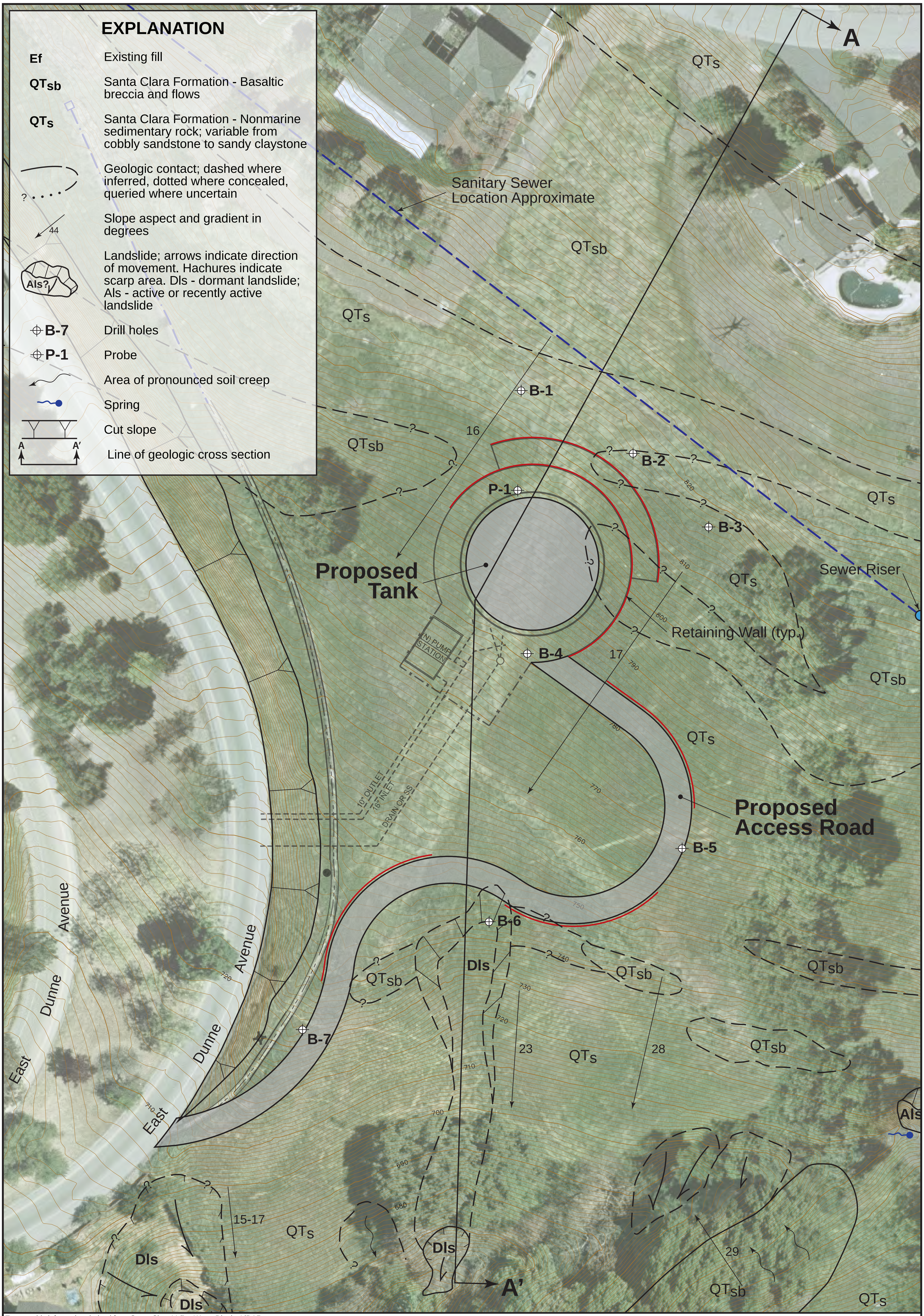
REGIONAL FAULT MAP

EAST DUNNE TANK SITE
MORGAN HILL, CALIFORNIA

160200	AUGUST 2016	FIGURE 3



Topographic information derived from Santa Clara Valley Water District LiDAR dataset (flown 2006). Aerial image from Triathlon, dated April 28, 2006.

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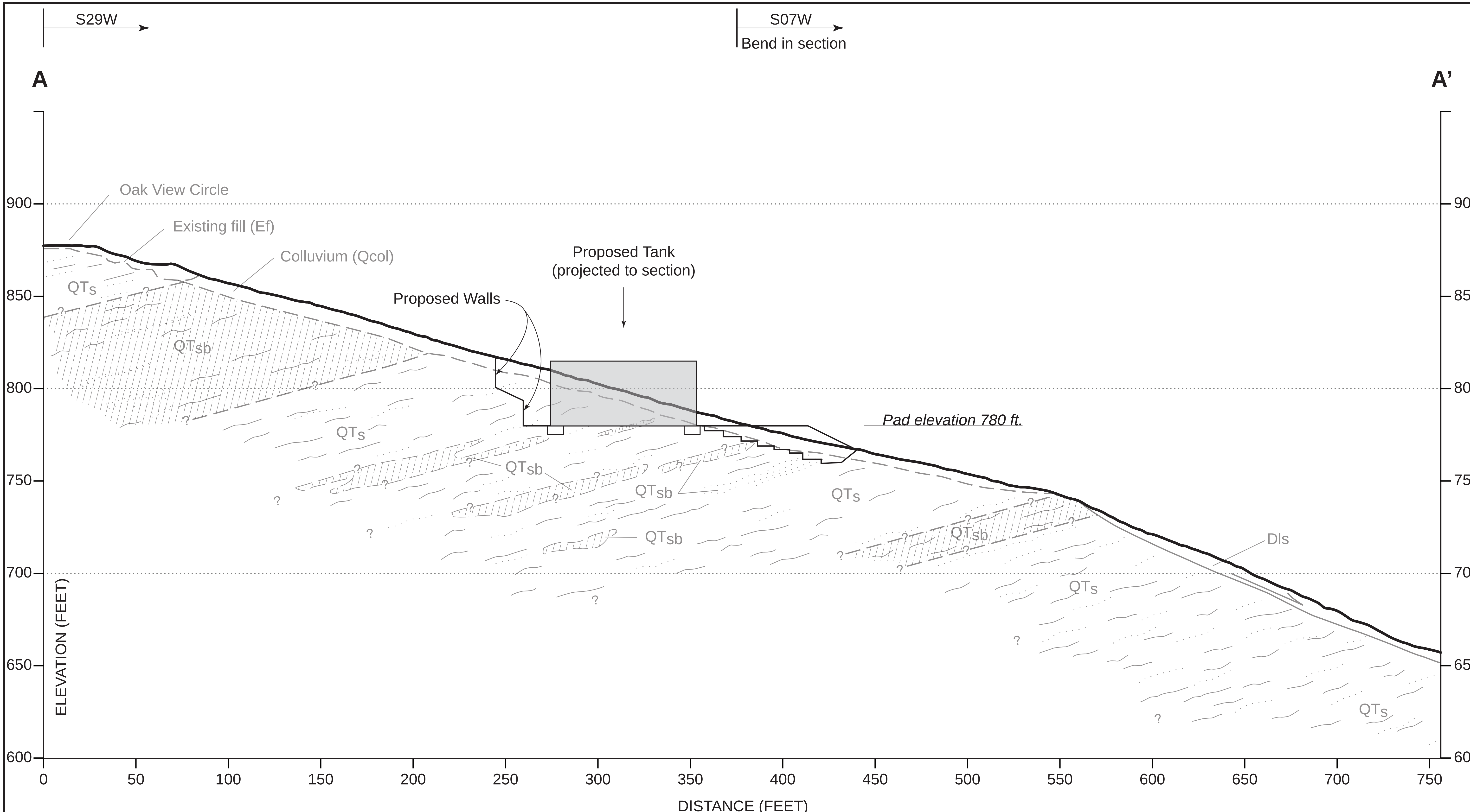
VICINITY GEOLOGIC MAP

EAST DUNNE TANK SITE
MORGAN HILL, CALIFORNIA

160200	AUGUST 2016	FIGURE 4



Notes:

1. Colluvium mantle not shown on Vicinity Geologic Map (Figure 4).
2. Subsurface extent and nature of QTsb is speculative.
3. See text for discussion of conceptual tank, wall and pad footprints shown.
4. Improvements shown schematically; location adjusted laterally to preserve elevation, and provide cut pad in rock.

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GEOLOGIC CROSS SECTION A - A'
EAST DUNNE TANK
MORGAN HILL, CALIFORNIA

PROJECT 160200	AUGUST 2016	FIGURE 5

APPENDIX A

- Boring Logs

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BORING NUMBER B-1

CLIENT Kennedy-Jenks
PROJECT NAME E. Dunne Tank
PROJECT NUMBER 160200
PROJECT LOCATION Morgan Hill, California
DATE STARTED 4/11/2016 **COMPLETED** 4/13/2016
GROUND ELEVATION ______ **DATUM** Site Specific **HOLE SIZE** 6 in.
DRILLING CONTRACTOR Britton Exploration
COORDINATES: LATITUDE 37.13802 **LONGITUDE** -121.59519
DRILLING RIG/METHOD 6-in. Solid Flight Auger, Rotary Wash
GROUNDWATER AT TIME OF DRILLING --- N/A
LOGGED BY R. Fisher **CHECKED BY** ______
GROUNDWATER AT END OF DRILLING --- N/A
HAMMER TYPE 140 lb hammer with 30 in. autotrip
GROUNDWATER AFTER DRILLING --- N/A

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	FINES CONTENT (%)
0		LEAN CLAY (CL), dark brown, moist, firm (COLLUVIUM)									
5		SANDSTONE, brown, weak to medium strength, massive, moist, severely weathered, trace 1/8 in. gravel, otherwise medium gravel (BEDROCK WEATHERED to SILTY SAND)	CM	9-13-16		96	18	39	28	11	27
10		dark yellow brown	CM	10-13-14		100	15				15
15		CLAYEY SANDSTONE with GRAVEL, olive brown, weak to medium strong, medium hardness, massive, severely weathered, estimated 30 % subrounded gravel up to 1/2 in., CaCO3 in matrix. (WEATHERED to CLAYEY SAND)	CM	10-14-17		111	16				16
20		SANDSTONE, pale olive, moist, medium to weak, massive, (WEATHERED to SANDY CLAY)	CM	11-25-40		85	35				54
25			SPT	9-14-24			33				
30		Clay content increasing CLAYSTONE, mottled olive brown and gray, weak, massive, moist, severely weathered	CM	19-17-30		102	22				
35											

(Continued Next Page)

CLIENT Kennedy-Jenks
PROJECT NAME E. Dunne Tank
PROJECT NUMBER 160200
PROJECT LOCATION Morgan Hill, California

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	FINES CONTENT (%)
35		light yellowish brown to olive brown,weak to medium strength, massive, moist, severely weathered, sand is very fine to fine CLAYSTONE, mottled olive brown and gray, weak, massive, moist, severely weathered *(continued)*	CM	14-30-50/6"		97	21				
40		SANDSTONE, olive brown and gray, medium strength, medium hard, massive, dry, intensely fractured, caliche, severely weathered Very hard drilling, switched over to Rotary Wash	SPT	28-50/3"							
45			SPT	50/0"							
50		No recovery, sandstone in wash	SPT	50/0"							

Bottom of borehole at 51.5 ft. Borehole backfilled with grout.

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BORING NUMBER B-2

CLIENT Kennedy-Jenks

PROJECT NAME E. Dunne Tank

PROJECT NUMBER 160200

PROJECT LOCATION Morgan Hill, California

DATE STARTED 4/13/2016 **COMPLETED** 4/13/2016

GROUND ELEVATION **DATUM** Site Specific **HOLE SIZE** 6 in.

DRILLING CONTRACTOR Britton Exploration

COORDINATES: LATITUDE 37.13799 **LONGITUDE** -121.59494

DRILLING RIG/METHOD 6-in. Solid Flight Auger

GROUNDWATER AT TIME OF DRILLING --- N/A

LOGGED BY R. Briseno **CHECKED BY**

GROUNDWATER AT END OF DRILLING --- N/A

HAMMER TYPE 140 lb hammer with 30 in. autotrip

GROUNDWATER AFTER DRILLING --- N/A

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	FINES CONTENT (%)
0											
		SANDY LEAN CLAY (CL), olive brown, moist, firm (COLLUVIUM)									
5											
10		SANDSTONE, olive brown, friable, loosely consolidated, moist, severely weathered, fine sand up to small subrounded gravel, weak (WEATHERED to WELL GRADED SAND with SILT)	CM	7-8-10		100	12				10
15		SILTY SANDSTONE, olive gray, weak, moist, severely weathered, some isolated 1 in. gravel, some iron staining along fractures, severe caliche at 15 ft. (WEATHERED to SILTY SAND)	CM	7-11-14		100	17				26
20		CLAYSTONE, gray, hard, moist, some iron stains sandy lens at 20 ft. very fine to fine sand	CM	15-35-50/4"		100	22				
25		SANDY CLAYSTONE, olive, weak, thumbnail can penetrate, moist, very fine sand, iron stained mottled with gray at 24.5 ft. sandy lens at 25 ft.	CM	9-20-50		104	22				
30		SANDSTONE, olive mottled with gray, friable, moist, severy weathered, few iron stains	CM	21-50/5"		104	19				
35			CM	27-50/3"		99	20				

(Continued Next Page)

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CLIENT Kennedy-Jenks **PROJECT NAME** E. Dunne Tank
PROJECT NUMBER 160200 **PROJECT LOCATION** Morgan Hill, California

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	FINES CONTENT (%)
35		Increase in iron staining at 34.5 ft.									
40		SANDSTONE interbedded with CLAY STONE, olive and gray respectively, CACO3 vein between beds. beds are at least 1 ft. thick	SPT	17-28-50							
45		SANDSTONE, olive, hard, moist	SPT	18-30-39							
50		CLAYSTONE, dark gray, weak, thumbnail can penetrate, moist, caliche lens between yellowish brown SANDSTONE, mottle with gray, heavily iron stained. bedded	SPT	15-23-34							

Bottom of borehole at 50.0 ft. Borehole backfilled with grout.

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BORING NUMBER B-3

CLIENT Kennedy-Jenks
PROJECT NAME E. Dunne Tank
PROJECT NUMBER 160200
PROJECT LOCATION Morgan Hill, California
DATE STARTED 4/11/2013 **COMPLETED** 4/11/2016
GROUND ELEVATION **DATUM** Site Specific **HOLE SIZE** 6 in.
DRILLING CONTRACTOR Britton Exploration
COORDINATES: LATITUDE 37.13781 **LONGITUDE** -121.59481
DRILLING RIG/METHOD 6-in. Solid Flight Auger
GROUNDWATER AT TIME OF DRILLING --- N/A
LOGGED BY R. Fisher **CHECKED BY**
GROUNDWATER AT END OF DRILLING --- N/A
HAMMER TYPE 140 lb hammer with 30 in. autotrip
GROUNDWATER AFTER DRILLING --- N/A

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	FINES CONTENT (%)
0		SANDY LEAN CLAY (CL), light yellowish brown, moist, firm (COLLUVIUM)									
5		CLAYEY SANDSTONE, light olive brown, low hardness,weak, easily carved with knife, possible 1 in. clay interbeds, fracture indeterminate, moist, severely weathered, CACO3 distributed throughout rock mass (BEDROCK WEATHERED to SANDY CLAY)	CM	9-16-21		94	27				52
10		SANDSTONE, light yellowish brown, weak, low hardness, massive, fracture indeterminate, moist to dry, severely weathered, very fine sand to silt	CM	21-50		102	21				
15			CM	15-29-50/5"		95	24				
		SANDSTONE interbedded with CLAYEY SANDSTONE, light yellowish brown to gray, weak to medium strength, low hardness, possible 3/4 in. beds, fracture indeterminate, dry, severely weathered, sand fine to medium									
20		SANDSTONE with pebbly interbeds, grayish brown, friable to weak, low hardness, 3 in. beds, fracture indeterminate, dry to slightly moist, severely weathered	SPT	7-8-11			11				24
25		CLAYEY SANDSTONE with GRAVEL, grayish brown, weak, medium hardness, 3 in. pebbly beds, fracture indeterminate,fracture indeterminate, dry to slightly moist, severely weathered with CACO3 in rock matrix, some angular gravel	CM	12-19-30		108	11				
30			CM	25-30-36		112	11				
35											

(Continued Next Page)

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CLIENT Kennedy-Jenks **PROJECT NAME** E. Dunne Tank
PROJECT NUMBER 160200 **PROJECT LOCATION** Morgan Hill, California

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTICITY INDEX (%)	FINES CONTENT (%)
35		CLAYEY SANDSTONE interbedded with SANDY CLAYSTONE, light yellowish brown, weak, low hardness, 4 in. beds at 50 degrees dip, fracture indeterminate, moist, severely weathered	SPT	11-13-23			22				
40		SANDY CLAYSTONE, yellowish brown, weak, low hardness, possibly massive, moist, severely weathered	CM	13-32-45		100	25				
45		mottled with olive brown and dark gray	CM	14-15-21		97	25				
50		CLAYEY SANDSTONE interbedded with SANDY CLAYSTONE, dark yellowish brown, weak, low hardness, 2-6 in. beds, fracture indeterminate, moist, severely weathered	CM	15-24-47		101	24				

Bottom of borehole at 51.5 ft. Borehole backfilled with grout.

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BORING NUMBER B-4

CLIENT Kennedy-Jenks
PROJECT NAME E. Dunne Tank
PROJECT NUMBER 160200
PROJECT LOCATION Morgan Hill, California
DATE STARTED 4/12/2016 **COMPLETED** 4/12/2016
GROUND ELEVATION **DATUM** Site Specific **HOLE SIZE** 6 in.
DRILLING CONTRACTOR Britton Exploration
COORDINATES: LATITUDE 37.13762 **LONGITUDE** -121.59516
DRILLING RIG/METHOD 6-in. Solid Flight Auger
GROUNDWATER AT TIME OF DRILLING --- N/A
LOGGED BY R. Briseno **CHECKED BY**
GROUNDWATER AT END OF DRILLING --- N/A
HAMMER TYPE 140 lb hammer with 30 in. autotrip
GROUNDWATER AFTER DRILLING --- N/A

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	FINES CONTENT (%)
0	SANDY FAT CLAY (CH), very dark gray brown, moist, firm, sparse rootlets (COLLUVIUM)	CM	4-5-7		95	24	66	25	41	64
		CM	5-8-14		105	21				75
5	SANDY CLAY (CL), brown, moist, firm, caliche, iron stains caliche increases at 5 ft. (HIGHLY WEATHERED BEDROCK)									
		CM	11-13-31		92	18				
10	SILTY SANDSTONE, light yellowish brown, hard, dry to moist, very severely weathered, caliche in matrix (WEATHERED BEDROCK)									
	CLAYSTONE, gray, weak to medium strength, dry to moist, severely weathered, caliche in matrix, isolated fine gravel	CM	11-30-42		109	19				
15										
20	mottled with brown, iron stains along fractures, sparse caliche, some very fine sand	CM	15-35-43		106	21				

Bottom of borehole at 20.0 ft. Borehole backfilled with grout.

BORING NUMBER B-5

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CLIENT Kennedy-Jenks	**PROJECT NAME** E. Dunne Tank
PROJECT NUMBER 160200	**PROJECT LOCATION** Morgan Hill, California
DATE STARTED 4/12/2016 **COMPLETED** 4/12/2016	**GROUND ELEVATION** **DATUM** Site Specific **HOLE SIZE** 6 in.
DRILLING CONTRACTOR Britton Exploration	**COORDINATES: LATITUDE** 37.13733 **LONGITUDE** -121.59485
DRILLING RIG/METHOD 6-in. Solid Flight Auger	**GROUNDWATER AT TIME OF DRILLING** --- N/A
LOGGED BY R. Briseno **CHECKED BY**	**GROUNDWATER AT END OF DRILLING** --- N/A
HAMMER TYPE 140 lb hammer with 30 in. autotrip	**GROUNDWATER AFTER DRILLING** --- N/A

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	FINES CONTENT (%)
0		LEAN CLAY (CL), dark brown, moist, firm, rootlets (COLLUVIUM)	CM	4-8-9		77	32				
5		CLAYEY SANDSTONE with GRAVEL, gray, friable, loosely consolidated, dry to moist, very severely weathered, silt to fine sand, subangular gravel (HIGHLY WEATHERED BEDROCK) color change to light olive brown	CM	4-5-6		93	15				
10		CLAYSTONE, gray, hard, dry to moist, severely weathered, some very fine sand	CM	8-18-25		104	20				
15		isolated grains of coarse sand	CM	15-18-32		110	9				
		SANDSTONE, olive yellow, hard, dry, coarse grained, iron bands									
20		color change to brown, fine sand	CM	15-24-40		104	18				
		CLAYSTONE interbedded with SANDSTONE, gray and brown respectively, hard, dry, severely weathered, caliche stains									
25		greater than or equal to 6 in. interbeds	SPT	11-14-15			20				

Bottom of borehole at 25.0 ft. Borehole backfilled with grout.

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BORING NUMBER B-6

CLIENT Kennedy-Jenks
PROJECT NAME E. Dunne Tank
PROJECT NUMBER 160200
PROJECT LOCATION Morgan Hill, California
DATE STARTED 4/12/2016 **COMPLETED** 4/12/2016
GROUND ELEVATION **DATUM** Site Specific **HOLE SIZE** 6 in.
DRILLING CONTRACTOR Britton Exploration
COORDINATES: LATITUDE 37.1372 **LONGITUDE** -121.59523
DRILLING RIG/METHOD 6-in. Solid Flight Auger
GROUNDWATER AT TIME OF DRILLING --- N/A
LOGGED BY R. Briseno **CHECKED BY**
GROUNDWATER AT END OF DRILLING --- N/A
HAMMER TYPE 140 lb hammer with 30 in. autotrip
GROUNDWATER AFTER DRILLING --- N/A

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	FINES CONTENT (%)
0		SANDY FAT CLAY (CH), dark gray brown, moist, firm (COLLUVIUM) rootlets at 1.5 ft.	CM	3-5-6		99	18	57	21	36	62
		caliche, iron staining and sparse isolated pebbles at 2.5 ft.									
5		SANDY CLAY (CL), very dark gray brown, moist, firm, caliche, iron staining (HIGHLY WEATHERED BEDROCK)	CM	3-5-9		93	26				79
10		CLAYEY SANDSTONE with GRAVEL, light yellowish brown, friable, dry, fine sand to coarse subrounded to rounded gravel, clay nodules	CM	20-23-30		106	14				
15		some chert observed	SPT	13-17-23			11				
20			CM	27-50			14				
25			SPT	15-30-31			10				

Bottom of borehole at 25.0 ft. Borehole backfilled with grout.

BORING NUMBER B-7

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CLIENT Kennedy-Jenks
PROJECT NAME E. Dunne Tank
PROJECT NUMBER 160200
PROJECT LOCATION Morgan Hill, California
DATE STARTED 4/12/2016 **COMPLETED** 4/12/2016
GROUND ELEVATION ____ **DATUM** Site Specific **HOLE SIZE** 6 in.
DRILLING CONTRACTOR Britton Exploration
COORDINATES: LATITUDE 37.13705 **LONGITUDE** -121.59554
DRILLING RIG/METHOD 6-in. Solid Flight Auger
GROUNDWATER AT TIME OF DRILLING --- N/A
LOGGED BY R. Briseno **CHECKED BY** ____
GROUNDWATER AT END OF DRILLING --- N/A
HAMMER TYPE 140 lb hammer with 30 in. autotrip
GROUNDWATER AFTER DRILLING --- N/A

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	FINES CONTENT (%)
0		LEAN CLAY (CL), dark brown, moist, firm									
		CLAYEY SANDSTONE with GRAVEL, light yellowish brown, dry, friable, fine sand to coarse subrounded gravel, chert	CM	11-17-18		99	10				
5		clay nodules	CM	19-21-25		111	12				
10		SANDSTONE interbedded with CLAYEY SANDSTONE, olive yellow and gray respectively, medium strength, dry, severely weathered	CM	23-34-47		104	19				
15		CLAYEY SANDSTONE with GRAVEL, light yellowish brown, dry, friable, fine sand to coarse subrounded gravel, chert, clay nodules, iron stained, caliche	CM	50							
20		SANDSTONE, light olive brown, friable, weak, dry	SPT	17-23-32			11				
25		CLAYEY SANDSTONE with GRAVEL, light yellowish brown, dry, friable, coarse sand to subangular gravel, chert, clay nodules, iron stained, caliche	SPT	14-19-35							

Bottom of borehole at 25.0 ft. Borehole backfilled with grout.

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BORING NUMBER Probe

CLIENT Kennedy-Jenks
PROJECT NAME E. Dunne Tank
PROJECT NUMBER 160200
PROJECT LOCATION Morgan Hill, California
DATE STARTED 4/13/2016 **COMPLETED** 4/13/2016
GROUND ELEVATION ______ **DATUM** Site Specific **HOLE SIZE** 6 in.
DRILLING CONTRACTOR Britton Exploration
COORDINATES: LATITUDE 37.13786 **LONGITUDE** -121.59513
DRILLING RIG/METHOD 6-in. Solid Flight Auger
GROUNDWATER AT TIME OF DRILLING --- N/A
LOGGED BY R. Briseno **CHECKED BY** ______
GROUNDWATER AT END OF DRILLING --- N/A
HAMMER TYPE 140 lb hammer with 30 in. autotrip
GROUNDWATER AFTER DRILLING --- N/A

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	FINES CONTENT (%)
0		LEAN CLAY (CL), dark brown, moist (COLLUVIUM)									
		CLAYEY SANDSTONE, olive. Driller indicates that drilling is very consistent all the way, no gravel, feels like claystone									
5											
10											
15											
20											
25											
30											
35											

(Continued Next Page)

CAL ENGINEERING & GEOLOGY

BORING NUMBER Probe

CLIENT Kennedy-Jenks
PROJECT NAME E. Dunne Tank
PROJECT NUMBER 160200
PROJECT LOCATION Morgan Hill, California

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT (%)	ATTERBERG LIMITS PLASTIC LIMIT (%)	ATTERBERG LIMITS PLASTICITY INDEX (%)	FINES CONTENT (%)
35											
40		CLAYEY SANDSTONE, olive. Driller indicates that drilling is very consistent all the way, no gravel, feels like claystone *(continued)*									

Bottom of borehole at 40.0 ft. Borehole backfilled with grout.

APPENDIX B

- Laboratory Test Results

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CAL ENGINEERING & GEOLOGY

SUMMARY OF LABORATORY RESULTS

CLIENT Kennedy-Jenks

PROJECT NAME E. Dunne Tank

PROJECT NUMBER 160200

PROJECT LOCATION Morgan Hill, California

Borehole	Depth	Date Tested	Liquid Limit	Plastic Limit	Plasticity Index	Maximum Screen Size (mm)	%<#200 Sieve	Class-ification	Water Content (%)	Dry Density (pcf)	Satur-ation (%)	Void Ratio
B-1	5.5	4/22/2016	39	28	11	19	27	SM	18.0	95.6		
B-1	11.0	4/22/2016				25	15		14.6	100.4		
B-1	16.0	4/22/2016				37.5	16		16.3	111.0		
B-1	21.0	4/22/2016				4.75	54		35.2	85.1		
B-1	25.0	4/20/2016							32.9			
B-1	31.0	4/20/2016							22.4	102.1		
B-1	35.5	4/20/2016							21.2	97.4		
B-2	9.5	4/22/2016				19	10		11.9	99.7		
B-2	14.5	4/22/2016				19	26		17.5	99.9		
B-2	19.5	4/20/2016							22.0	100.4		
B-2	24.5	4/21/2016							22.4	103.9		
B-2	28.5	4/21/2016							19.4	103.6		
B-2	33.5	4/21/2016							19.6	99.3		
B-3	6.0	4/25/2016				19	52		26.5	93.8		
B-3	10.5	4/22/2016							20.5	102.1		
B-3	15.5	4/22/2016							24.1	95.1		
B-3	20.0	4/22/2016				25	24		10.9			
B-3	26.0	4/22/2016							11.2	107.9		
B-3	31.0	4/22/2016							10.8	112.1		
B-3	35.0	4/20/2016							22.2			
B-3	40.5	4/22/2016							24.5	100.3		
B-3	46.0	4/22/2016							25.3	97.3		
B-3	51.0	4/22/2016							23.6	101.0		
B-4	2.0	4/22/2016	66	25	41	25	64	CH	24.4	94.6		
B-4	4.5	4/25/2016				19	75		20.8	105.4		
B-4	9.0	4/22/2016							17.7	92.3		
B-4	14.5	4/22/2016							19.4	109.1		
B-4	18.5	4/22/2016							20.8	106.1		

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CAL ENGINEERING & GEOLOGY

SUMMARY OF LABORATORY RESULTS

CLIENT Kennedy-Jenks

PROJECT NAME E. Dunne Tank

PROJECT NUMBER 160200

PROJECT LOCATION Morgan Hill, California

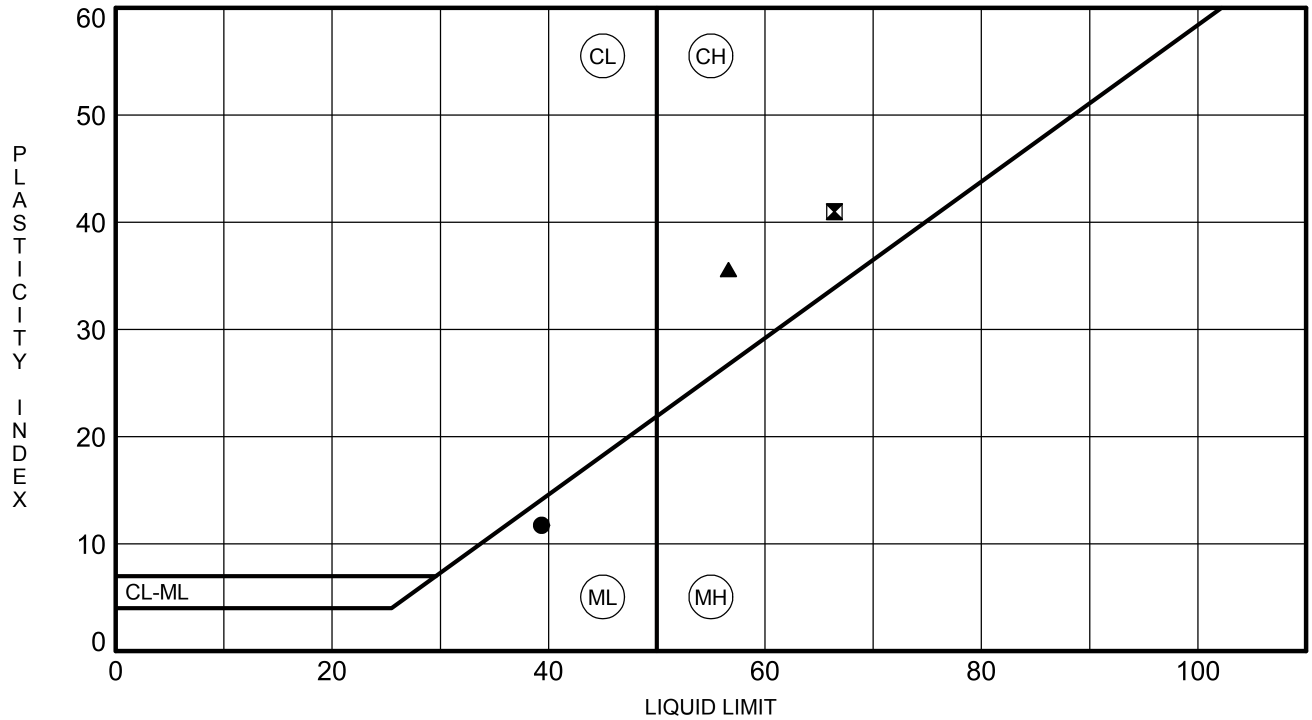
Borehole	Depth	Date Tested	Liquid Limit	Plastic Limit	Plasticity Index	Maximum Screen Size (mm)	%<#200 Sieve	Class-ification	Water Content (%)	Dry Density (pcf)	Satur-ation (%)	Void Ratio
B-5	2.0	4/25/2016							32.3	77.0		
B-5	4.5	4/25/2016							14.6	93.2		
B-5	9.5	4/25/2016							19.6	104.1		
B-5	14.5	4/25/2016							8.8	110.0		
B-5	19.5	4/25/2016							18.4	104.0		
B-5	23.5	4/25/2016							20.3			
B-6	2.0	4/26/2016	57	21	36	37.5	62	CH	18.2	98.8		
B-6	4.5	4/27/2016				19	79		26.2	93.2		
B-6	9.5	4/25/2016							14.1	105.8		
B-6	13.5	4/25/2017							10.9			
B-6	19.0	4/25/2016							13.9			
B-6	23.5	4/25/2016							10.1			
B-7	2.0	4/25/2016							9.6	98.6		
B-7	4.5	4/25/2016							12.0	110.9		
B-7	9.5	4/25/2016							19.1	103.9		
B-7	18.5	4/20/2016							10.6			

ATTERBERG LIMITS RESULTS

CAL ENGINEERING & GEOLOGY

CLIENT Kennedy-Jenks | **PROJECT NAME** E. Dunne Tank

PROJECT NUMBER 160200 | **PROJECT LOCATION** Morgan Hill, California



	BOREHOLE	DEPTH	LL	PL	PI	Fines	Classification	Date Tested
●	**B-1**	**5.5**	**39**	**28**	**11**	**27**	**Brown Silty Sand**	**4/26/2016**
⊠	**B-4**	**2.0**	**66**	**25**	**41**	**64**	**Very Dark Gray Brown Sandy Fat Clay**	**4/26/2016**
▲	**B-6**	**2.0**	**57**	**21**	**36**	**62**	**Dark Gray Brown Sandy Fat Clay**	**4/26/2016**

CAL ENGINEERING & GEOLOGY

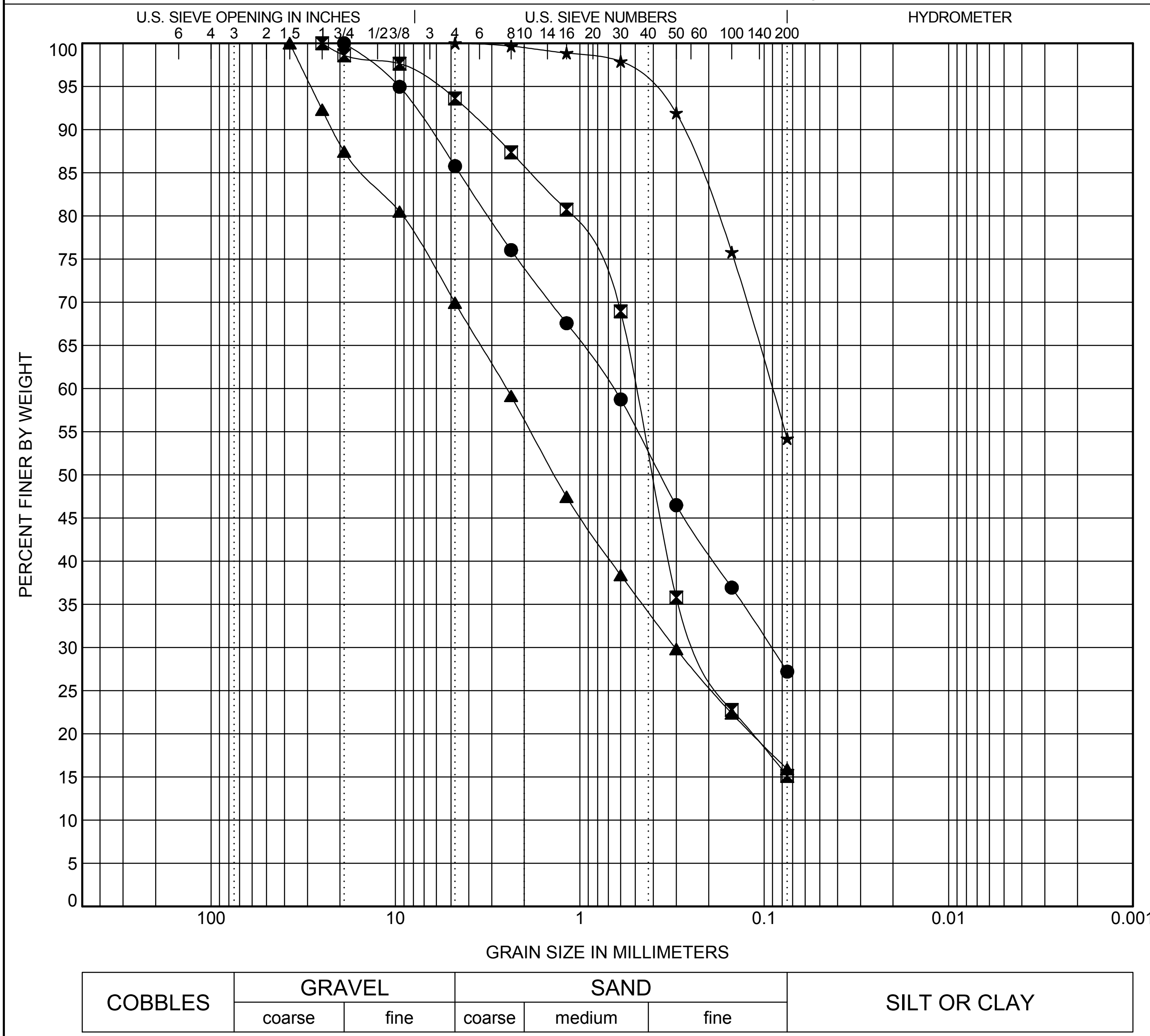
GRAIN SIZE DISTRIBUTION

CLIENT Kennedy-Jenks

PROJECT NAME E. Dunne Tank

PROJECT NUMBER 160200

PROJECT LOCATION Morgan Hill, California



BOREHOLE		DEPTH	DATE TESTED	Classification	LL	PL	PI	Cc	Cu
●	**B-1**	**5.5**	**4/22/2016**	**Brown Silty Sand**	**39**	**28**	**11**		
⊠	**B-1**	**11.0**	**4/22/2016**	**Dark Yellow Brown Silty Sand**					
▲	**B-1**	**16.0**	**4/22/2016**	**Olive Brown Clayey Sand**					
★	**B-1**	**21.0**	**4/22/2016**	**Pale Olive Sandy Clay**					

BOREHOLE		DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-1**	**5.5**	**19**	**0.661**	**0.092**		**14.2**	**58.6**	**27.2**	
⊠	**B-1**	**11.0**	**25**	**0.498**	**0.221**		**6.4**	**78.5**	**15.1**	
▲	**B-1**	**16.0**	**37.5**	**2.497**	**0.305**		**30.1**	**54.0**	**15.9**	
★	**B-1**	**21.0**	**4.75**	**0.09**			**0.0**	**45.8**	**54.2**	

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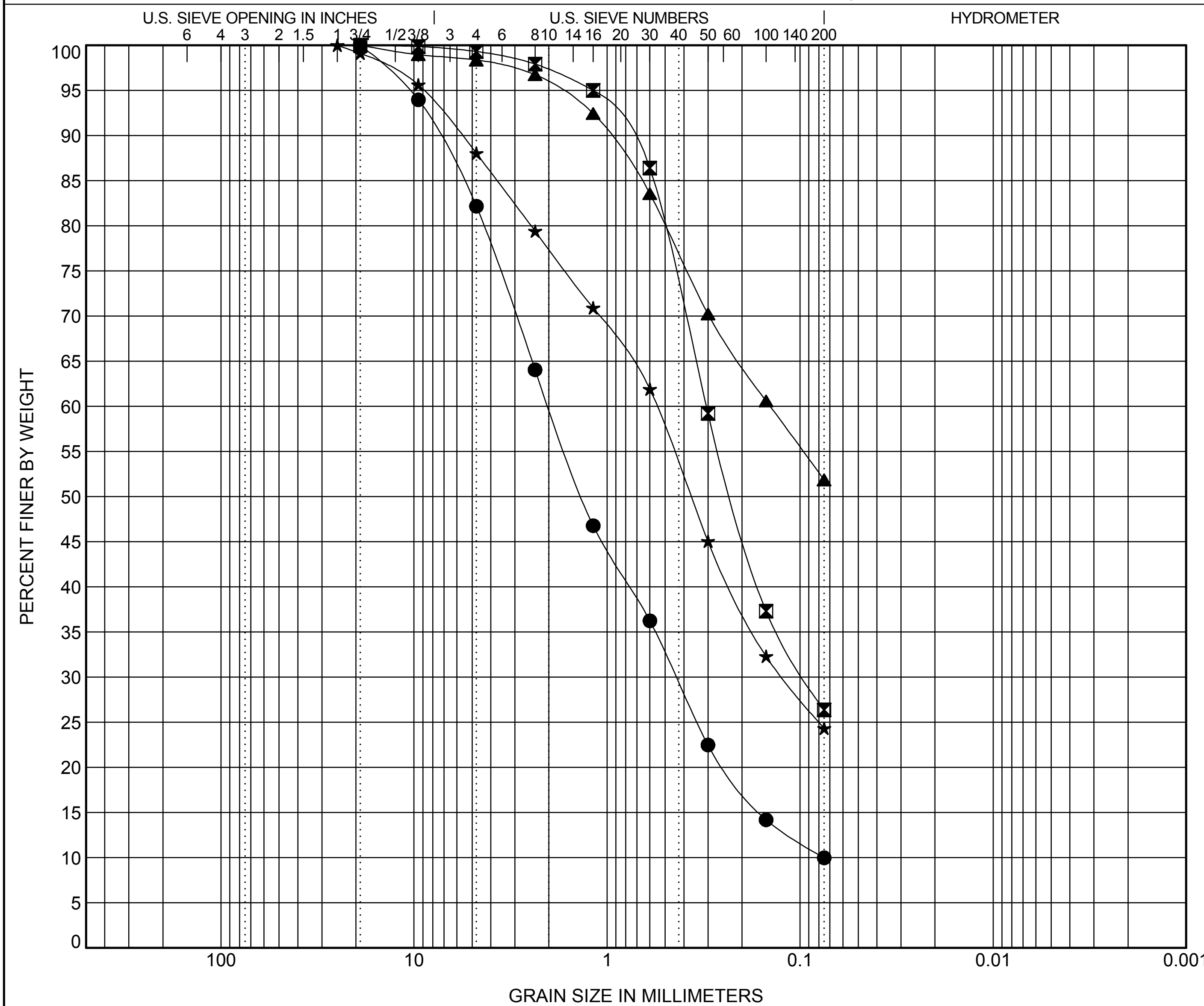
CAL ENGINEERING & GEOLOGY

GRAIN SIZE DISTRIBUTION

CLIENT Kennedy-Jenks **PROJECT NAME** E. Dunne Tank

PROJECT NUMBER 160200 **PROJECT LOCATION** Morgan Hill, California



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BOREHOLE	DEPTH	DATE TESTED	Classification	LL	PL	PI	Cc	Cu
●	**B-2**	**9.5**	**4/22/2016**	**Olive Brown Well graded Sand with Silt**				**1.27**	**26.63**
⧗	**B-2**	**14.5**	**4/22/2016**	**Olive Gray Silty Sand**					
▲	**B-3**	**6.0**	**4/22/2016**	**Light Olive Brown Sandy Clay**					
★	**B-3**	**20.0**	**4/22/2016**	**Dark Olive Silty Sand**					

	BOREHOLE	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-2**	**9.5**	**19**	**2.006**	**0.438**	**0.075**	**17.8**	**72.2**	**10.0**	
⧗	**B-2**	**14.5**	**19**	**0.306**	**0.094**		**0.7**	**73.0**	**26.4**	
▲	**B-3**	**6.0**	**19**	**0.143**			**1.6**	**46.5**	**51.9**	
★	**B-3**	**20.0**	**25**	**0.555**	**0.123**		**12.0**	**63.7**	**24.3**	

GRAIN SIZE DISTRIBUTION

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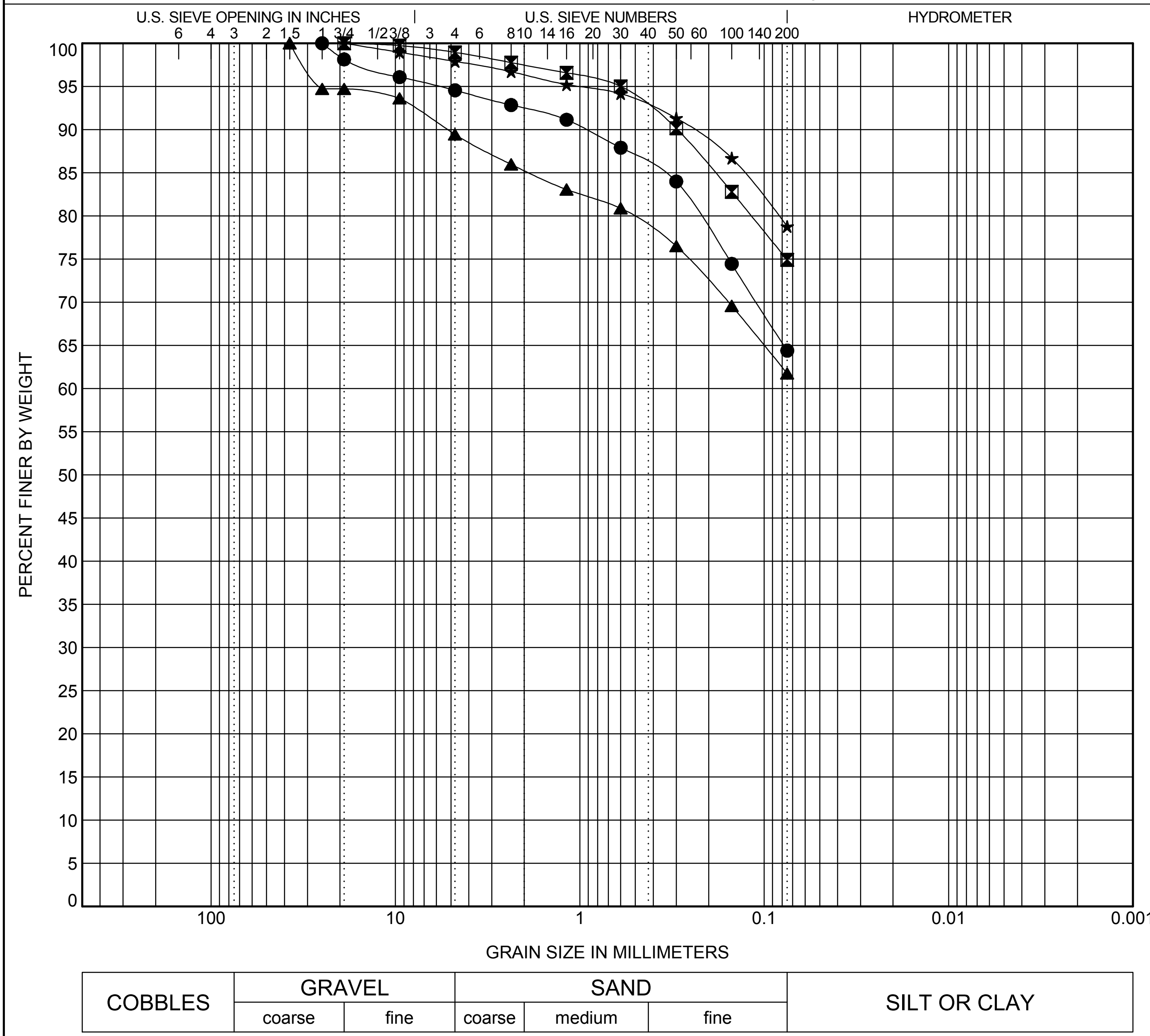
CAL ENGINEERING & GEOLOGY

CLIENT Kennedy-Jenks
PROJECT NAME E. Dunne Tank
PROJECT NUMBER 160200
PROJECT LOCATION Morgan Hill, California



	BOREHOLE	DEPTH	DATE TESTED	Classification	LL	PL	PI	Cc	Cu
●	**B-4**	**2.0**	**4/27/2016**	**Very Dark Gray Brown Sandy Fat Clay**	**66**	**25**	**41**		
⊠	**B-4**	**4.5**	**4/27/2016**	**Brown Sandy Clay**					
▲	**B-6**	**2.0**	**4/27/2016**	**Dark Gray Brown Sandy Fat Clay**	**57**	**21**	**36**		
★	**B-6**	**4.5**	**4/27/2016**	**Very Dark Gray Brown Sandy Clay**					

	BOREHOLE	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-4**	**2.0**	**25**				**5.5**	**30.2**	**64.4**	
⊠	**B-4**	**4.5**	**19**				**1.1**	**24.0**	**74.9**	
▲	**B-6**	**2.0**	**37.5**				**10.6**	**27.7**	**61.7**	
★	**B-6**	**4.5**	**19**				**2.1**	**19.1**	**78.8**	

APPENDIX C

- Corrosion Test Results

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

20 May 2016

Job No. 1605100
Cust. No. 11770

Mr. Dan Peluso
Cal. Engineering & Geology
1870 Olympic Blvd. #100
Walnut Creek, CA 94596

Subject: Project No.: 160200
Project Name: East Dunne Tank
Corrosivity Analysis – CalTrans Test Methods with Brief Evaluation

Dear Mr.Peluso:

Pursuant to your request, CERCO Analytical has analyzed the soil sample submitted on May 12, 2016. Based on the analytical results, a brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, both samples are classified as "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 concentrations are none detected to 15 mg/kg

The sulfate ion concentrations are none detected to 15 mg/kg.

The pH of the soils are both 8.30 which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

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:	Cal Engineering and Geology
Client's Project No.:	160200
Client's Project Name:	East Dunne Tank
Date Sampled:	Not Indicated
Date Received:	12-May-16
Matrix:	Soil
Authorization:	Signed Chain of Custody

Date of Report: 20-May-2016

Job/Sample No.	Sample I.D.	Moisture (%)	pH	Min.Resistivity (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
1605100-001	B-1 @ 15.5' & 20.5'	-	8.30	790	-	N.D.	N.D.
1605100-002	B-2 @ 24' & 29'	-	8.30	1,000	-	N.D.	N.D.

	Moisture	pH	Min.Resistivity	Sulfide	Chloride	Sulfate
Method:	CT 226 [a]	CT 643 [b]	CT 643 [b]	-	CT 422 [c]	CT 417 [c]
Reporting Limit:	-	-	-	50	15	15
Date Analyzed:	-	18-May-2016	18-May-2016	-	18-May-2016	18-May-2016

* Results Reported on an "As Received" Basis

N.D. - None Detected

[a] Rev. July 2010 [b] Rev. June 2007 [c] Rev. November 2006

Cheryl McMillen
Laboratory Director

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

Chain of Custody

Page of

1100 Willow Pass Court
Concord, CA 94520-1006
925 462 2771
Fax: 925 462 2775

CERCO analytica

Job No.	CU#	Client Project I.D.	Schedule / Analyte	Date Sampled	Date Due
1605100	11770	160200			

Full Name Dan Peluso w/ Cal Engineering + Geology

Fax

Phone 925-935-9771

Company

Cell ☒

Sample Source East Dunne Tank

CalTrans w/Brief Evalation — ANALYSIS

Lab No.	Sample I.D.	Date	Time	Matrix	Contain.	Size	Preserv.	Qty.	pH	Sulfate	Chloride	Resistivity-Minimum	Brief Evaluation
	B-1@15.5'+20.5'								X	X	X	X	X
	B-2@24'+29'								X	X	X	X	X
	DPeluso@CalEng.Com												

MATRIX
DW - Drinking Water
GW - Ground Water
SW - Surface Water
WW - Waste Water
Water
SL - Sludge
S - Soil
Product

ABBREVIATIONS
HB - Hosebib
PV - Petcock Valve
PT - Pressure Tank
PH - Pump House
RR - Restroom
GL - Glass
PL - Plastic
ST - Sterile

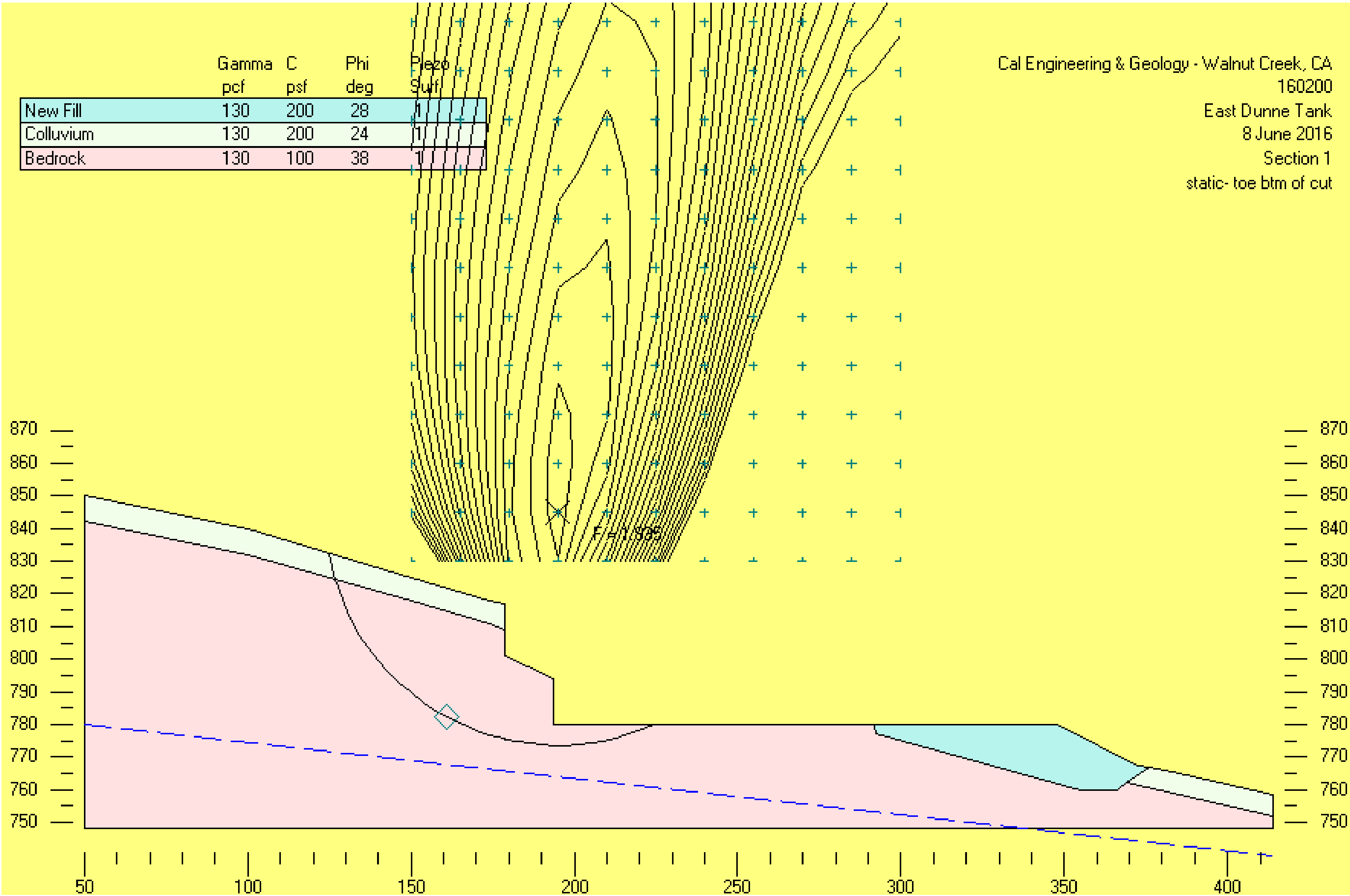
SAMPLE RECEIPT
Total No. of Containers
Rec'd Good Cond/Cold
Conforms to Record
Temp. a t Lab -°C
Sampler

	Date	Time
Relinquished By: [signature]	5/12/16	9:20 AM
Received By: [signature]	5/12/16	9:20
Relinquished By:		
Received By:		
Relinquished By:		
Received By:		

Comments:
THERE IS AN ADDITIONAL CHARGE FOR METALS/POLY TUBES

APPENDIX D

- Slope Stability Analysis



GSLOPE Static Slope Stability Calculation Output