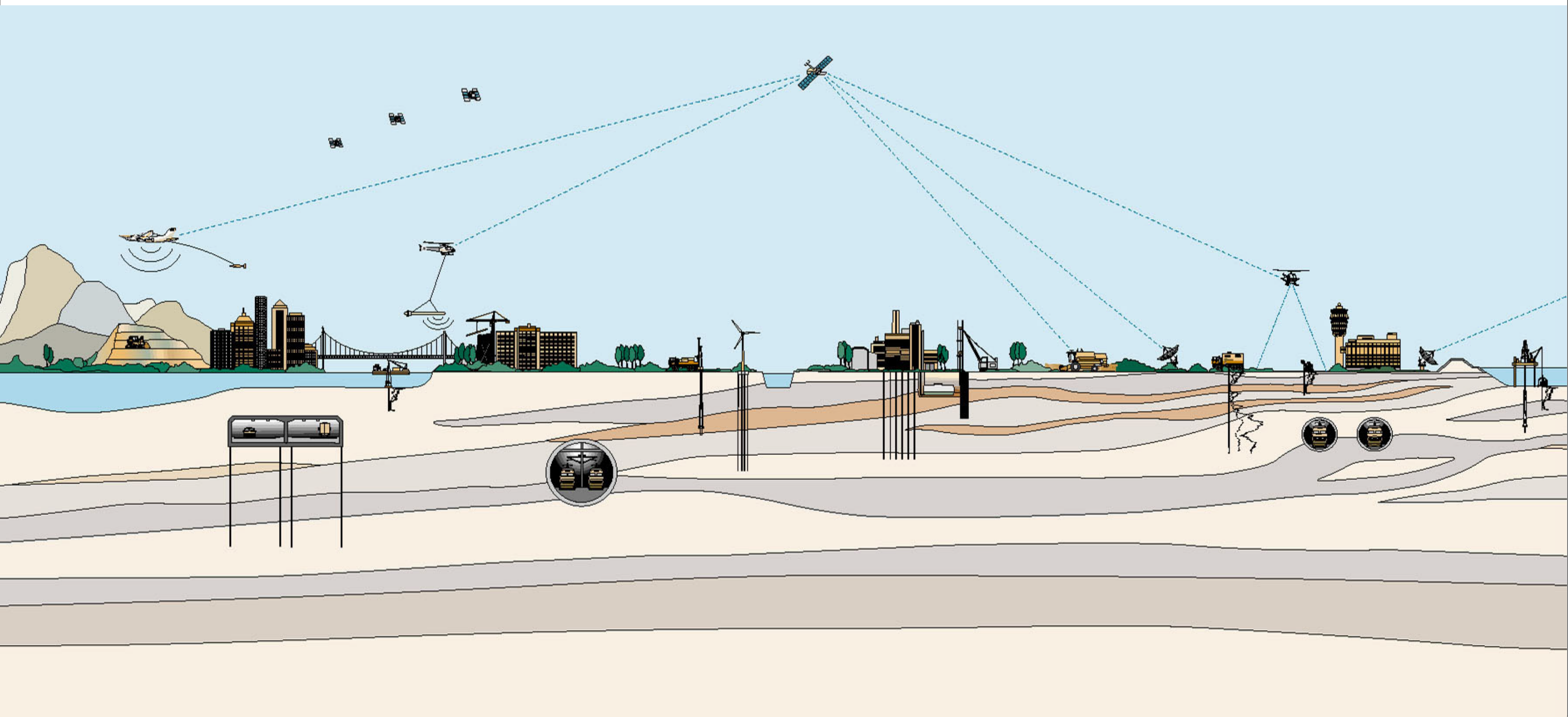
FUGRO CONSULTANTS, INC.

GEOTECHNICAL STUDY
1,235-FT BACKBONE IMPROVEMENTS PROJECT
5 MG TANK AND PIPELINE PROJECT
WESTLAKE RESERVOIR
WESTLAKE VILLAGE, CALIFORNIA

Prepared for:
AECOM

June 2013
Fugro Job No. 04.62120197



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June 3, 2013
Project No. 04.62120197

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Attention: Mr. John Coffman

Subject: Geotechnical Study, 1,235-Ft Backbone Improvements Project 5 MG Tank and Pipeline Project, Westlake Reservoir, Westlake Village, California

Dear Mr. Coffman:

Fugro is pleased to present this geotechnical report for the proposed 5 MG Tank and Pipeline project at the Westlake Reservoir in Westlake Village, California. The project involves construction of a 5 million gallon pre-stressed concrete water tank and separate 36-inch-diameter inlet and outlet pipelines. The tank site is located in a hillside area on the southern side of the west saddle dam at the Westlake Reservoir. The pipeline extends northward from the tank site, across the west dam, to the water filtration facility located on the hillside north of the west dam.

The work was performed in general accordance with our proposal dated September 6, 2012. Authorization was provided in the form of a signed Agreement dated November 13, 2012.

Thank you for the opportunity to provide geotechnical services on the tank and pipeline project. Please call if you have any questions regarding the information presented herein.

Sincerely,

FUGRO CONSULTANTS, INC.

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1.0 INTRODUCTION

1.1 GENERAL

This geotechnical report was prepared for the 1235-foot Backbone Improvements Project - 5 MG tank and pipeline project at the Westlake Reservoir, in Westlake Village, California. The project involves construction of a 5 million gallon (MG) pre-stressed concrete water tank and separate 36-inch-diameter inlet and outlet pipelines. The tank site is located in a hillside area on the southern side of the west saddle dam of the Westlake Reservoir. The pipeline extends northward from the tank site, across the west dam, to the water filtration facility located on the hillside north of the west dam. A portion of the pipeline alignment will be constructed above-grade across the eastern face of the west dam. Across the dam, the pipelines will be constructed in separate 42-inch-diameter steel carrier pipes.

The approximate location of the project is shown on Plate 1 - Vicinity Map. The tank site and pipeline alignment are shown on Plate 2 - Geologic Map.

The work was performed in general accordance with our proposal dated September 6, 2012. Authorization was provided in the form of a signed Agreement dated November 13, 2012.

1.2 PROJECT DESCRIPTION

Review of plans by AECOM (2013) indicate that the project will consist of:

- A 200-foot diameter, 25-foot-high, 5 MG, reinforced concrete water tank with an interior floor elevation (El.) of about +1,065 feet;
- A proposed 0.25h:1v backcut into the north-facing hillside on the southern side of the tank;
- A below-ground valve vault; and
- Separate 36-inch-diameter water-inlet and -outlet pipelines that will connect the new tank with the water filtration facility north of the west dam. As proposed, inlet and outlet pipelines will extend across the eastern face of the west dam, a few feet above the reservoir's high-water level. The pipelines will be supported on cast in place spread footings with pipe saddles. The pipe supports will be spaced about 30 feet on center across the west dam.

1.3 PURPOSE

The purpose of the geotechnical study was to evaluate the geologic and geotechnical conditions at the proposed tank site and along the pipeline alignment, and provide foundation and seismic design criteria for the proposed project elements.

1.4 WORK PERFORMED

Tasks performed as part of this geotechnical study consisted of data review, geologic mapping, laboratory testing, geotechnical evaluation, and reporting.

1.4.1 Data Review

We reviewed published geologic data available in our files for the site and vicinity. We also reviewed historical geologic data developed in the late 1960s during the original design of the Westlake Reservoir project. The historical geologic data included drill hole and test pit logs advanced by W.A. Wahler and Associates and a site map prepared by AECOM. We also utilized the results of our preliminary geotechnical and geophysical evaluations report (Fugro Consultants, Inc., 2010) as well as our preliminary pile capacity analysis report for the proposed pipeline support foundations (Fugro Consultants, Inc., 2013).

1.4.2 Geotechnical Exploration

Tank Site. The tank site was explored in 2010 using four backhoe-excavated trenches (Fugro Consultants, Inc., 2010). Because hard rock materials were encountered, the trenches were typically excavated to depths of less than about 4 feet. Those trenches encountered volcanic bedrock materials of the Conejo Volcanics Formation. Previous tank site exploration also included six seismic refraction surveys performed for the purpose of estimating the P-wave velocity of the rock materials in the tank site area (Fugro Consultants, Inc., 2010).

Pipeline Alignment. Engineering properties and subsurface conditions along the pipeline alignment were estimated using surface geologic mapping and historical (1970) construction/bid documents that were provided by AECOM. No subsurface exploration was performed within the west dam embankment materials or along the pipeline route near the treatment facility.

1.4.3 Laboratory Testing

Laboratory tests were performed on select earth materials sampled from surface outcrops in the tank area to help evaluate soil chemistry (pH, resistivity, sulfates, and chlorides) for purposes of estimating corrosion potential. The results of the laboratory testing program are presented in Appendix A - Laboratory Testing.

1.4.4 Geotechnical Evaluation and Reporting

Geotechnical evaluations were performed based on the results of the work described above. This report summarizes the subsurface geologic/geotechnical conditions, geotechnical design criteria, evaluations, and analyses. Supporting documentation consists of our maps and laboratory data. Additional data regarding rock excavation characteristics and blasting requirements are presented in Fugro Consultants (2010) and Revey (2013).

This geotechnical report includes the following:

- Deterministic and probabilistic seismic hazard spectra;
- 2010 California Building Code (CBC) seismic design criteria;
- Evaluation of liquefaction and seismic settlement
- Foundation design recommendations for the tank;
- Earthwork and grading recommendations;
- Anticipated excavation conditions and considerations for temporary excavations;
- Corrosion potential;
- General site grading recommendations with soil material and compaction requirements for compacted fill;
- Retaining wall recommendations;
- Geotechnical pipeline design criteria; and
- Material specifications for selected backfill and drainage materials.

2.0 FINDINGS

2.1 SITE CONDITIONS

2.1.1 Regional Setting

The Westlake Village area of Los Angeles County is located within the Transverse Ranges geologic/geomorphic province of California. That portion of the province is characterized by generally east-west-trending mountain ranges composed of sedimentary and volcanic rocks ranging in age from Cretaceous to Recent. Major east-trending folds, reverse faults, and left-lateral strike-slip faults reflect regional north-south compression and are characteristic of the Transverse Ranges. Several authors including Dibblee and Ehrenspeck (1993), Weber et al. (1973), and Weber (1984) have mapped the Westlake Village area.

2.1.2 Local Setting

The proposed tank site is located at a former borrow site adjacent to the south side of the west saddle dam for Westlake Reservoir. The pipeline alignment will be constructed across the eastern face of the west dam and along the northern side of the treatment facility. The pipeline alignment is shown on Plate 2.

2.1.3 Geologic Conditions

Published geologic mapping by Dibblee and Ehrenspeck (1993) indicates that the tank site is underlain by bedrock of the Conejo Volcanics as indicated on Plate 3 - Regional Geologic Map. The tank site area was formerly the location of a borrow site during the construction of the Westlake Reservoir. Grading during dam construction resulted in the current topography, which is now up to about 50 feet below the original ground surface. On the basis of our surficial geologic mapping and published information, the pipeline alignment is underlain by a combination of embankment fill, general fill, and volcanic bedrock (Plate 2).

Regional published mapping suggests that flow-layering within the Conejo Vocanics Formation in the area of the tank site strikes approximately north-south and dips about 25 degrees to the east. Near the western end of Trench 2 (Fugro Consultants, Inc., 2010) a possible bedding contact was observed to strike approximately north-south and dip about 10 to 12 degrees to the east. Published maps do not indicate the presence of landslides on or near the tank site or along the pipeline route.

2.2 EARTH MATERIALS

Earth materials at the site include embankment fill, general fill, and bedrock of the Conejo Volcanics Formation.

2.2.1 Embankment Fill

The west dam embankment is constructed of artificial fill (afe) materials. The embankment fill materials were placed during the construction of the west dam, and according to construction-related documents, they include five zones described by the gradation ranges presented in Boyle (1970). We understand that the embankment materials consist of a clay core surrounded by granular materials that were derived primarily from borrow sites in Conejo Volcanics bedrock.

2.2.2 General Fill

Along the southern margin of the west dam embankment are artificial fill (afg) materials that appear to have been placed to facilitate the construction of an access road. On the basis of surface exposures, those general fill materials appear to consist of granular fragments of Conejo Volcanics bedrock that were generated during dam construction.

2.2.3 Conejo Volcanics Formation

Conejo Volcanics Formation (Tcv) bedrock is exposed at the surface throughout the proposed tank site and in the vicinity of the existing facilities building. Conejo Volcanics bedrock materials also underlie the west dam embankment fill materials. Previous onsite trenching in the tank area encountered basalt, agglomerate, and diabase volcanic rocks (Fugro Consultants, Inc., 2010).

2.3 ENGINEERING PROPERTIES

Results of limited soil chemistry tests suggest that the materials are moderately corrosive to ferrous metals and negligibly corrosive to concrete. A further discussion of the limited soil chemistry testing is presented in Section 3.10.

2.4 GROUNDWATER

Groundwater was not encountered in any of the shallow trenches excavated by Fugro in 2010 within the tank pad area. However, in our opinion, groundwater may be present locally within joints and fractures in the Conejo Volcanics bedrock, particularly following periods of heavy rainfall. Groundwater is also likely to be present in bedrock and artificial fill materials located adjacent to the reservoir as a result of seepage from the reservoir pool.

We note that groundwater levels can vary as a result of changes in precipitation, irrigation, runoff, and other factors. Perched groundwater can also occur along bedding and/or along fracture surfaces.

2.5 SEISMIC DESIGN CRITERIA

The project site is located within a seismically active area and the potential exists for strong ground motion to affect the project during the design lifetime. In general, the primary effects will be associated with shaking and/or ground acceleration.

We estimated peak ground accelerations and spectral values for the site based on empirically derived attenuation relations and geological estimates of seismogenic fault source model parameters (e.g., fault geometry, maximum magnitude estimates, rupture area estimates, earthquake magnitude-occurrence distributions, etc.). Actual peak ground motions may vary from our estimated values.

Some of the significant active faults within about a 25-mile radius of the project site, their estimated maximum earthquake magnitudes, and deterministically-estimated peak ground accelerations from a maximum considered earthquake on those faults are listed in Table 1. The faults, estimated distances, and ground acceleration values presented in Table 1 are based on results obtained from the computer-based program EZ-FRISK (Risk Engineering, 2013). The nearest known active fault is the Malibu Coast fault, located approximately 9.8 miles south of the site.

Table 1. Significant Faults

Fault	Distance (miles)	Maximum Earthquake Magnitude (Mw)	Mean 50th Percentile pga (g)*	Mean 84th Percentile pga (g)*
Malibu Coast	9.8	7.0	0.298	0.509
Anacapa-Dume	13.4	7.2	0.380	0.648
Santa Monica	13.7	7.4	0.348	0.592
Simi-Santa Rosa	14.3	6.9	0.188	0.322
Oak Ridge	22.8	7.4	0.204	0.347

* Computed as the mean of four NGA attenuation relations : Boore-Atkinson (2008), Campbell-Bozorgnia (2008), Chiou-Youngs (2008), and Abrahamson-Silva (2008).

2.5.1 Ground Shaking

As summarized in Table 1, there are several active faults within a 25-mile radius of the site that have a potential to generate strong ground motion. Using the 2008 USGS Interactive Deaggregations (beta) web application, the estimated peak horizontal ground acceleration (pga) at the site, with a 10 percent probability of exceedance in 50 years (475-year-return-period), is about 0.35g. Site-specific seismic design criteria are presented below.

2.5.2 Ground Rupture

The site does not lie within an Alquist-Priolo fault rupture hazard zone; however, the site is proximal to a number of faults that are considered active or potentially active. Based on our data review, no known active faults trend toward or traverse the site. Therefore, the potential for primary ground rupture due to faulting to occur at the site is considered to be low.

2.5.3 Peak Vertical Acceleration

Studies by Boore et al. (1997) and Bozorgnia et al. (1999) have observed that for large earthquakes at close distances or sites overlying the fault rupture surface, the peak vertical acceleration can be about 1.5 to 1.6 times the peak horizontal acceleration. However, typical seismic design criteria incorporate a peak vertical acceleration that is two-thirds to three-quarters of the peak horizontal acceleration.

2.5.4 Uniform-Hazard Spectra

Uniform hazard response spectra for 5 percent damping are presented on Plate 4 - Results of Probabilistic Seismic Hazard Analysis. Plate 4 presents the 475-year-return-period spectra in terms of spectral acceleration (g) versus the undamped natural period (seconds). The site-specific uniform hazard response spectra were estimated using the 2008 Interactive Deaggregations (beta) web site developed by the United States Geological Survey (USGS, 2008). The spectra were developed using Next-Generation Attenuation (NGA) models and an average V_{s30} shear wave velocity of 790 m/s for subsurface conditions.

Table 2. Summary of 475-Year-Return-Period Site-Specific Uniform Hazard Spectra

Period (seconds)	Spectral Acceleration (g)
Pga	0.3491
0.10	0.7305
0.20	0.8367
0.30	0.6846
0.50	0.4722
1.00	0.2490
2.00	0.11324
3.00	0.06764
4.00	0.04770
5.00	0.03884

2.5.5 2010 CBC Design Criteria

The proposed water tank should be designed to resist the lateral forces generated by earthquake shaking in accordance with local design practice. The seismic design procedures outlined in Section 1613 of the California Building Code (CBC) are designed to meet the intent and requirements of ASCE 7.

The tank site is located on rock materials (average V_{s30} = 790 m/s) that meet the criteria for Site Class B. The USGS interactive web page 'Seismic Design Values for Buildings' (USGS, 2008) was also used to obtain the seismic design criteria. Based on our characterization of the subsurface conditions and the 2010 CBC, we recommend the following values be used for design.

Table 3. Summary of 2010 CBC Seismic Design Parameters

2010 California Building Code Section 1613	Seismic Parameter	Value
---	Latitude	N 34.1297°
---	Longitude	W 118.8381°
Section 1613.5.1 and Figure 1613.5(3)	Mapped Acceleration Response Parameter (S_s) Site Class B	1.768
-Section 1613.5.1 and Figure 1613.5(4)	Mapped Acceleration Response Parameter (S_1) Site Class B	0.741
Section 1613.5.2 and Table 1613.5.2	Soil Profile Type	B
Section 1613.5.3 and Table 1613.5.3(1)	Site Coefficient (Fa)	1.0
Section 1616.5.3 and Table 1613.5.3(2)	Site Coefficient (Fv)	1.0
Section 1613.5.3	Adjusted Acceleration Response Parameter for Site Class B (S_{ms})	1.768
Section 1613.5.3	Adjusted Acceleration Response Parameter for Site Class B (S_{m1})	0.741
Section 1613.5.4	Design Spectral Response Acceleration Parameter (S_{DS})	1.179
Section 1613.5.4	Design Spectral Response Acceleration Parameter (S_{D1})	0.494

2.6 SEISMICALLY INDUCED GROUND MOVEMENTS

2.6.1 Liquefaction and Liquefaction-Related Settlements

Liquefaction is the sudden loss of soil shear strength due to rapid increases in pore water pressures caused by seismic shaking. Liquefaction has generally been observed in saturated granular soils of low to medium density within about 50 feet of the ground surface and below the groundwater level, although liquefaction can occur deeper than 50 feet. The proposed tank site at foundation grade will be underlain by bedrock of the Conejo Volcanics Formation and compacted fill, materials that are not prone to liquefaction.

As noted above, a portion of the proposed pipeline will cross the west dam embankment and supporting structures for that portion of the pipeline will be founded in the dam fill materials. At the present time, we do not have enough data regarding the condition of those materials to allow for a quantitative evaluation of liquefaction for the west embankment dam materials.

2.6.2 Seismically Induced Settlement

Seismically induced settlement or collapse due to ground shaking can occur in soils that are loose, soft, or that are moderately dense but weakly cemented. Intact Conejo Volcanics bedrock materials and the thin layer of compacted fill that will underlie the tank site are not prone to seismically induced settlement. Fill materials that will form the disposal fill area and the visual-concealment berm are likely to experience seismically induced settlement, but no estimate of the magnitude of that settlement has been made.

As noted above, a portion of the proposed pipeline will cross the west dam embankment and supporting structures for that portion of the pipeline will be founded in the dam fill materials. At the present time, we do not have enough data regarding the condition of those dam materials to allow for a quantitative evaluation of seismic settlement for the west embankment dam materials.

2.7 SLOPE STABILITY EVALUATION

2.7.1 General

Unsupported 0.25h:1v bedrock cut slopes up to about 35 feet high are proposed to remain above the southern and western sides of the tank. In addition, embankment fills will be constructed on the east and west sides of the tank (Plate 2). Plans indicate that fill slopes up to about 35 feet high will be graded along the eastern side of the tank area and up to about 30 feet high along the western side of the tank area. The finished surfaces of the fill slopes are planned to be graded at 1.75h:1v to 2h:1v. We understand that the planned fill area on the eastern side of the tank is to be a disposal area for materials excavated during the site grading and that the embankment fill area on the western side of the tank is to be used as a partial visual-concealment berm.

2.7.2 Slope Stability Discussion

Most of the proposed tank cut-slopes are likely to be composed of relatively hard volcanic rock. If unfractured hard volcanic rocks are exposed in the cut-slopes, they are likely to perform satisfactorily at the proposed 0.25h:1v gradients. However, we recommend that protective wire netting be installed to help contain possible rock raveling. Because fractured rocks may result during the proposed excavation process, there is a possibility that adversely oriented fractures may form potentially unstable blocks or wedges. We recommend that the cut slopes be observed and geologically mapped during grading, so that the potential for unstable rocks can be evaluated. If unstable rocks are found to be present, additional stabilization measures beyond the recommended wire netting may be needed (e.g., rock anchors). Additionally, portions of the proposed cut (possibly in the area west of the tank) may expose weathered, friable volcanic rocks that may not perform well at 0.25h:1v. To improve the performance of those materials, we recommend that the slope gradient be reduced to 1.5h:1v along the western portion of the cut where the visual-concealment fill berm will be positioned above the cut slope. If other areas of fractured or otherwise degraded rock are exposed in the finished cut slope, additional measures may be needed to improve the slope performance.

We understand that the gross and surficial stability of the embankment fill materials to be placed in the large disposal-fill area east of the tank location is not of concern to LVMWD. Consequently, it may be acceptable to provide only minimal densification efforts during grading for those fill materials. However, we recommend that an equipment-width key be cut into bedrock at the toe of the fill slope and that the fill be placed (on horizontal benches cut into the bedrock) in lifts of 18-inches or less and densified by several passes of a heavy vibratory roller. Those efforts should help improve the stability beyond that of an uncompacted spill-fill.

Because the fill materials for the large fill area west of the tank site will be located above the adjacent reservoir access road, we recommend that additional measures be implemented to improve their stability. For fill placed in that area, we recommend that coarse grained materials be limited to about 6-inches in diameter and the materials be blended with fine-grained materials to produce a fill with about 20 percent passing the 200 sieve. We also recommend that those fill materials be placed in horizontal lifts less than 18-inch-thick and compacted using five passes of a heavy vibratory drum roller in an effort to densify the rock fill.

3.0 CONCLUSIONS AND RECOMMENDATIONS

3.1 GENERAL

The conclusions and recommendations presented in this report are based on the results of our field exploration, geologic mapping, laboratory testing, engineering evaluation, and our understanding of the project at the time the report was written. Our recommendations for site preparation and grading, fill placement, and for the design of the proposed concrete tank and pipeline are presented below.

3.2 EARTHWORK AND GRADING

Fill placement and grading operations should be performed in accordance with AECOM specifications, Standard Specifications for Public Works Construction (Greenbook, 2009), and the grading recommendations presented in this report.

3.2.1 Site Preparation

All tree root balls and structures and improvements such as wells, vaults, pipelines, abandoned utilities, etc., planned to be removed should be removed or treated in a manner prescribed by Greenbook standard specifications. Depressions or voids left from clearing and grubbing and demolition of existing structures should be filled with properly compacted fill according to the recommendations of this report.

3.2.2 Excavation Considerations

3.2.2.1 Excavation Potential

The exploratory trenches in the tank site area were excavated using a rubber-tired backhoe. In general, the onsite materials excavated using that backhoe presented significant difficulty and the trenches were only able to be excavated to depths of a few feet. At the bottoms of those trenches hard, unfractured volcanic rock was commonly encountered. The seismic refraction study at the tank site (Fugro Consultants, Inc., 2010) measured P-wave velocities in the upper 20 feet that varied from 5,000 to over 9,000 feet per second. Rocks with those properties typically require alternative excavation methods. Rock materials along the pipeline route are likely to present similarly difficult excavation characteristics. Blasting was required to excavate rock during construction of the existing dam. Contractors should make an initial site visit to observe the rock and make an evaluation of the appropriate excavation methods. Where blasting is used, the program will need to include a plan to mitigate velocities to an acceptable level and include blasting mats and other mitigations to prevent fly-rock. Fugro subcontracted with Revey Associates to develop a blasting mitigation plan for the project. The blasting mitigation plan they developed for the project is provided in Revey Associates (2013).

3.2.2.2 Temporary Excavations

Temporary slopes, excavations, and shoring should conform to federal Occupational Safety and Health Administration regulations and any other local ordinances and building codes, as required. The contractor should be responsible for the design of and all safety issues regarding temporary excavations and worker safety. The contractor should continuously monitor the temporary slopes and support and remove or stabilize any loose or unstable soil/rock masses.

For dry excavations, unshored, temporary slopes in hard, unfractured, volcanic bedrock should not be constructed steeper than 0.25h:1v. Rock slopes may need to be constructed flatter than 0.25h:1v or temporary shoring may be locally required where adverse geologic conditions are present. Stockpiled material or equipment should not be placed closer than 5 feet from a temporary slope crest. Dewatering and erosion protection, such as controlled runoff drainage, should be provided as necessary. Appropriate slope netting may be needed to protect workers from rock fragments that may originate from the planned cut-slopes.

3.2.2.3 Groundwater and Dewatering

Groundwater was not encountered in any of the shallow trenches excavated in the tank area. Groundwater levels can vary as a result of changes in precipitation, irrigation, runoff, and other factors. Perched groundwater can also occur along bedding and/or along fracture surfaces.

If groundwater seepage or perched water is encountered during grading, then measures to dewater the excavation should be implemented to maintain a dry excavation and care should be taken to minimize the disturbance of the excavation bottom. Surface water should be directed away from excavations, and should be removed from excavations prior to placement of fill materials or concrete.

Before implementing a dewatering system, we recommend that a dewatering test program be conducted to evaluate the feasibility and efficiency of the proposed dewatering system. Dewatering operations may require permitting in accordance with National Pollutant Discharge Elimination System (NPDES) regulations and possibly other local permits.

3.2.3 Excavation Bottom Preparation

We anticipate that volcanic rocks are likely to be exposed across nearly all of the structure area at the design excavation level. Prior to backfilling, the excavation bottom should be observed by Fugro. If unsuitable materials are present at that level, additional overexcavation may be required. We understand that lean concrete or concrete slurry will be placed as backfill beneath the tank and extending up from the excavated rock surface.

3.2.4 Backfilling

3.2.4.1 Fill Material Selection

Recommended fill material selection requirements for disposal fill, visual-berm fill, and tank backfill are presented below. Areas or zones where the various fill materials may be used are described above.

3.2.4.2 Disposal Fill

A disposal fill site has been designated to the east of the proposed tank site. Excavated rock materials are to be placed in that area. It is our understanding that gross and surficial stability of those disposal fill materials is not of concern to LVMWD. Consequently, only the following minimal efforts are recommended to help improve the stability of those materials. The recommendations are not expected to produce a level of slope stability and performance consistent with typical embankments constructed of compacted engineered fill.

- Cut a continuous, equipment-width key (estimated to be about 8-feet wide) into bedrock at the toe of the fill. That keyway should be constructed approximately level/horizontal with the base of the key entirely on bedrock materials.

- The disposal fill should be placed on horizontal benches cut into the bedrock as the fill is placed.
- The disposal fill should be placed in lifts of 18 inches or less and densified by several passes of a vibratory roller.

3.2.4.3 Visual-Concealment Berm Fill

A visual-concealment berm fill site has been proposed to the west of the proposed tank site. Because the fill materials for that fill area will be located above the adjacent reservoir access road, we recommend that additional measures be implemented to improve their stability. The recommendations are not expected to produce a level of slope stability and performance consistent with typical embankments constructed of compacted engineered fill.

- Cut a continuous, equipment-width key (estimated to be about 8-feet wide) into bedrock at the toe of the fill. That keyway should be constructed approximately level/horizontal with the base of the key entirely on bedrock materials.
- Coarse-grained materials should be limited to about 6-inches in maximum diameter and the materials should be blended with fine-grained materials to produce a fill with about 20 percent passing the 200 sieve.
- The disposal fill should be placed on horizontal benches cut into the bedrock as the fill is placed.
- If the gradation and composition of the fill used to construct the visual screening berm will allow for evaluating relative compaction (referenced to ASTM D1557) using traditional methods, we recommend that the fill be placed in horizontal lifts of no more than 12 inches in thickness, miosture conditioned to within 2 percent of optimum, and mechanically compacted to at least 85 percent relative compaction. However, if evaluation of relative compaction of the embankment fill is not practical using traditional methods, the disposal fill shall be placed as decribed above and densified by at least five passes of a heavy vibratory roller.

3.2.4.4 Use of Onsite Materials as Fill

Materials generated from the volcanic bedrock excavated during the grading of the tank site are likely to contain a significant number of rock fragments of various sizes, although some finer material may also be generated. Material derived from the tank excavation can be placed in the proposed visual-concealment berm west of the tank, as long as those materials contain or are mixed with sufficient fines, are free of organics, and do not contain oversize rock (that is over 6 inches in diameter), trash, debris, corrosive, or other unsuitable materials.

3.2.4.5 Fill in Tank Pad

As discussed in Section 3.2.3, we understand that fill placed in the tank pad will consist of lean concrete or concrete slurry. The maximum thickness of the concrete fill beneath the tank is anticipated to be about 2-1/2 feet. The concrete fill should extend up from the excavated rock surface to the base of the tank drainage system and should extend laterally at least 6 to 7 feet beyond the perimeter of the tank.

Fill materials used to backfill the overexcavated area of the tank pad and outside the limits of the concrete fill should consist of Class II Base material, compacted to at least 95 percent relative compaction. If alternative fill materials are proposed, they should be reviewed by the geotechnical engineer prior to being used.

3.2.4.6 Drainage System and Materials

Drainage material used for subsurface drains or behind retaining walls should conform to Section 68-1.025, "Class 2 Permeable Material" of the Caltrans (2002), Standard Specifications. Drainage materials in contact with earth materials may need to be protected with nonwoven filter fabric satisfying requirements for Underdrains per Section 88-1.03 of the Caltrans (2002), Standard Specifications.

Details regarding the drainage system, including connection details between drainage behind below-grade walls and beneath the tank bottom, placement of perforated drain pipe, clean-out points, leak detection, and discharge details (protected from soil erosion), should be provided by the civil designer.

3.2.5 Fill Placement and Compaction

3.2.5.1 Placement

Fill materials placed as backfill in the tank pad should be spread evenly, with loose lifts no thicker than 8 inches, and should be thoroughly blade-mixed during spreading to provide relative uniformity of material within each layer. Soft or yielding materials should be removed and replaced with properly compacted fill material prior to placing the next layer.

Where fill is placed on sloping ground (in excess of about 5h:1v) and at the toes of excavations, the fill should be keyed and benched into firm native materials. Keying and benching should be performed in general accordance with local agency requirements.

3.2.5.2 Compaction Requirements

All fill materials (excluding fill placed in the disposal fill and visual-concealment berm; see Sections 3.2.4.2 and 3.2.4.3 for requirements) should be moisture-conditioned to within 2 percent of optimum moisture content prior to and during compaction. Water should be added to the fill when the moisture content of the fill material is below that sufficient to achieve the recommended compaction. While water is being added, the soil should be bladed and mixed to provide uniform moisture content throughout the material. When the moisture content of the fill material is excessive, the fill should be aerated by blading or other methods.

Fine-grained materials are sensitive to changes in moisture content, and can be relatively difficult to compact. Hence, if fill materials are placed at a moisture content more than a few percent above optimum, there is a potential for the fill to pump or yield when subjected to construction traffic or compactive effort. Control of the moisture content and compaction using thin lifts will be critical in achieving the required compaction.

All tank-pad backfill materials should be compacted to a minimum of 95 percent (relative compaction) of the maximum dry density determined from ASTM D1557, latest edition. Fill materials placed in the eastern fill disposal area should be compacted using a few passes of a heavy vibratory roller and fill material placed in the visual-concealment berm fill area should be compacted using about five passes of a heavy vibratory roller.

Backfill within 5 feet of below grade walls (measured horizontally) should be compacted with lightweight, hand-operated compaction equipment to minimize the potential for large compaction stresses. If large or heavy equipment is used, compaction-induced stresses can result in increased lateral earth pressures on the tank walls.

Compaction testing should be performed during grading and fill placement of the tank-pad backfill material. Measurements of in situ or field moisture content and relative compaction should be evaluated using either ASTM D2922 (nuclear gauge) and/or ASTM D1556 (sand cone method). Grading in the eastern disposal fill area and the western visual-concealment berm area should be monitored, to confirm compliance with recommendations, but compaction tests will not be required in those areas.

3.2.6 Grading Recommendations in Pavement Areas

Where pavements will be constructed on aggregate base placed for the tank foundation or on volcanic rock, no special subgrade preparation work is anticipated. Pavement subgrade conditions along the crest of the west dam are not known. Therefore, in that portion of the roadway alignment, we recommend that the pavement subgrade soils be observed and tested by Fugro during rough grading. We anticipate that the subgrade soils on the west dam will consist of firm, compacted granular soils. Grading recommendations for those conditions can likely consist of scarifying the subgrade soils to a depth of 12 inches below the pavement section (aggregate base layer) or existing ground surface, whichever is deeper, moisture conditioning the soils to within 2 percent of optimum moisture content, and compacting the scarified subgrade soils to at least 95 percent relative compaction. The scarified and recompaction work should extend laterally at least 3 feet beyond the pavement limits.

If the subgrade soils in the west dam area are not as anticipated, additional remedial grading work may be required to provide suitable subgrade conditions for the pavement section.

3.3 TANK FOUNDATION DESIGN

Based on information provided by AECOM, the tank foundation will consist of a mat-type foundation or structural slab with thickened pads beneath interior columns. The floor slab will be founded on compacted fill and drainage material.

3.3.1 Allowable Bearing Pressure

For the new tank, shallow interior, wall or mat-type foundations bearing on compacted fill placed in accordance with recommendations herein may be designed using a net maximum allowable bearing pressure of 6,000 pounds per square foot. The maximum net allowable bearing pressures can be increased by one-third when considering short-term wind or seismic loads.

Isolated square column footings should be designed with a width of at least 3 feet. Continuous footings should have a minimum width of 2 feet.

3.3.2 Modulus of Subgrade Reaction

A modulus of subgrade reaction (Kv_1) of about 350 pounds per cubic inch may be assumed for design of mat-type foundations using a beam on elastic foundation analogy (a Winkler model). The modulus of subgrade reaction value (Kv_1) represents a presumptive value based on soil classification data and is for a 1-foot-square plate. The Kv_1 value may need to be adjusted by the designer for mat size, assuming a cohesive subgrade.

3.3.3 Resistance to Lateral Loads

3.3.3.1 Sliding Resistance

Resistance to lateral loading can be provided by sliding friction acting along the base of spread footings or slabs combined with passive pressure acting on the sides of the foundations. Ultimate sliding resistance generated through a soil/concrete or soil/soil interface can be estimated using a coefficient of friction of 0.40.

We anticipate that a membrane liner could be placed beneath the tank as part of a proposed leak detection system. We recommend that a roughened membrane liner at least 30 mils thick be used and that the liner have an ultimate soil/liner friction coefficient of at least 0.30. When evaluating the sliding potential along the membrane liner, the soil/liner friction coefficient should be used.

3.3.3.2 Passive Resistance

Ultimate passive resistance may be estimated using an equivalent fluid weight (EFW) of 350 pounds per cubic foot for walls or foundations bearing against compacted backfill or competent volcanic rock where the backfill surface or adjacent ground is horizontal. Passive resistance should not be used for the upper 1 foot of soil that is not overlain at the ground surface by a slab-on-grade or pavement.

3.3.3.3 Factors of Safety

For static conditions, a safety factor of 1.5 is recommended for sliding and overturning, but may be reduced to 1.1 for dynamic conditions.

3.3.4 Lateral Earth Pressures

3.3.4.1 Earth Pressure State

Retaining structures that are free to rotate or translate laterally are referred to as unrestrained or yielding retaining structures. Such walls can generally move enough to develop active conditions. Retaining structures that are unable to rotate or deflect laterally (e.g., restrained basement walls) are referred to as restrained or non-yielding walls. If backfill materials behind the wall consist of cohesionless soils, then unrestrained walls can usually be designed for active earth pressure conditions. For cohesionless backfills, restrained walls should be designed for at-rest earth pressure conditions. For cohesive backfills, both unrestrained and restrained walls should be designed for at-rest conditions because cohesive soils creep, undergo stress relaxation, and cannot sustain active conditions. In addition, expansive soils placed behind retaining walls can result in lateral earth pressures much greater than those originating from gravity loads.

We anticipate the walls of the proposed tank will be designed as restrained walls. For static conditions and granular wall backfill materials, EFW values provided below can be used to estimate the design lateral earth pressure. EFWs for active conditions may be used for active conditions in conjunction with dynamic force increments described below.

Table 4. Equivalent Fluid Weights for Retaining Wall Design

Backfill Slope	Active	At-Rest
Level or Descending On-site Backfill	35	65

Note: For drained conditions

Recommended EFWs for static conditions do not include hydrostatic pressures and assume that drainage measures will be incorporated into the tank design to preclude the development of hydrostatic pressures behind proposed below-grade walls.

3.3.4.2 Surcharge Loads

Uniform area surcharge pressures for the tank walls may be assumed equal to one-half of the applied surcharge pressure at the ground surface. Surcharge load from automobiles and pickup trucks (not large trucks or construction equipment) may be assumed equivalent to 2 feet of soil surcharge (area surcharge at the ground surface of 260 pounds per square foot). Lateral pressures for other surcharge loading conditions can be provided, if required.

3.3.4.3 Dynamic Pressures

For unrestrained walls, the increase in lateral earth pressure due to earthquake loading can be estimated using the Mononobe-Okabe theory, as described in Seed and Whitman (1970). That theory is based on the assumption that sufficient wall movement occurs during seismic shaking to allow active earth pressure due to earthquake loading to be estimated using the Mononobe-Okabe equations. Because the theory is based on the assumption that sufficient movement occurs so that active earth pressure conditions develop during seismic shaking, the applicability of the theory to restrained or basement walls is not direct; however, there is a supporting reference (Nadim and Whitman, 1992) that suggests the theory can be used for such walls.

In the Mononobe-Okabe approach, the total dynamic pressure can be divided into static and dynamic components. Dynamic force increments for different backfill slope conditions are presented below. To estimate the total dynamic lateral force, the dynamic lateral force increase should be added to the static earth pressure force computed using recommendations for active (not at-rest conditions) lateral earth pressures presented previously. That recommendation is based on the concept that during shaking, earth pressures recommended for permanent conditions will be reduced to those more closely approximating active conditions. The resultant dynamic lateral force increment should be applied at a distance of 0.6H above the base of the wall, while the static lateral force should be applied at a distance of one-third the wall height above the base of the wall.

For level backfill conditions, the estimated dynamic lateral force increase (based on seismic loading conditions) for either unrestrained or restrained walls may be taken as 45 x PHGA x H^2 pounds per linear foot of wall. In the above formulation, PHGA is the peak horizontal ground acceleration and H is the height of wall below the ground surface in feet. As indicated in Section 2.5, the 475-year-return-period PHGA estimate is about 0.35g. When estimating the resultant seismic force, the dynamic lateral force increment should be added to the active pressure force.

3.3.5 Foundation Settlement

For anticipated foundation loads and subsurface conditions at the tank site (including the thin layer of compacted fill to be placed beneath the tank), we estimate total settlement on the tank will likely be less than about 1 inch.

3.4 SITE DRAINAGE

Site grading and drainage swales should be provided such that positive drainage away from tank foundations and slabs is provided. Water should not be allowed to pond near the structures or pavements, or run over temporary or permanent slopes. Erosion control and maintenance of the slopes should be provided to reduce the potential for erosion, and assist in establishing vegetation on the slopes.

3.5 FLEXIBLE PAVEMENT RECOMMENDATIONS

R-value testing of the potential pavement subgrade for the access road was not performed for this work. However, we anticipate the pavement subgrade will consist of either relatively hard volcanic rock or aggregate base placed adjacent to the tank, and granular soils along the crest of the west dam. For the purposes of developing preliminary pavement sections, we have assumed that the volcanic rock subgrade will have an R-value similar to or stronger than aggregate base and that the subgrade soils along the west dam will have an R-value of at least 40. Preliminary flexible pavement design sections were estimated using the Caltrans pavement design procedures (Caltrans 2012) for assumed traffic indices ranging from 6 to 8 for R-values of 78 (near the tank and for rock subgrade) and 40 (west dam crest), respectively. The final pavement design sections should be evaluated on the basis of site specific testing or observations performed during rough grading in the pavement areas.

Table 5. Preliminary Pavement Sections -R-value =78

Traffic Index	Thickness of AC (in)	Thickness of AB (in)
6.0 through 8	3	4

Table 6. Preliminary Pavement Sections - R-value =40

Traffic Index	Thickness of AC (ft)	Thickness of AB (ft)
6.0	3.5	5
7.0	4	6
8.0	5	8

Following preparation of the overexcavation subgrade, compacted fill can be placed to the proposed finished subgrade level. Pavement materials should conform to Sections 26 and 39 of the Caltrans Standard Specifications (or equivalent) for aggregate base (AB) and asphalt concrete (AC), respectively. Subgrade and pavement materials should be compacted to at least 95 percent relative compaction.

3.6 PIPE ZONE AND TRENCH BACKFILL

3.6.1 General

As noted above, the portion of the proposed pipelines that crosses the west embankment dam will be supported above-ground by concrete footings spaced about 30 feet apart. Preliminary geotechnical design recommendations for those foundations are provided in Fugro (2013).

As previously discussed, excavations more than 4-feet deep should be excavated with sloping sidewalls, braced, shored, or shielded in accordance with DSOD, federal, and state standards and safe construction practice. Shoring and bracing of the trench sidewalls will be required in accordance with OSHA regulations. Most of the proposed trenches are likely to be excavated in firm to hard volcanic rock materials, but loose/soft fill and fractured rock may be locally encountered in some trench excavations. The contractor will be responsible for design and implementation of shoring systems and safe working conditions.

Excavation bottoms are likely to be within firm materials or bedrock and blasting or alternative excavation techniques may be required to create the trenches. However, if wet, soft, highly fractured, or yielding conditions are encountered, the bottom of the trench excavation should be stabilized prior to placement of pipe bedding so that the trench subgrade is firm and unyielding.

Compacted fill materials for the pipeline will consist of pipe-zone materials and trench backfill materials. Based on our experience, we anticipate that the native volcanic rock that will be excavated from the site should be suitable as trench backfill, as long as materials are processed to produce the trench backfill characteristics described below. The following subsections provide our suggested material types for pipe zone and trench backfill materials. Our recommendations for characteristics and placement of the pipe zone materials are largely derived from the 'Greenbook' (2009), Section 306. Actual material types considered applicable to the project could depend on input from the design team.

3.6.2 Pipe Zone Materials

Pipe zone materials are herein defined as those select earth materials used as pipeline bedding and shading, and as structure bedding. Pipe zone materials should consist of clean sand with a minimum Sand Equivalent (SE) of 30 or crushed angular gravel to facilitate placement and achieve uniform support for pipe. Gravel should conform to the gradation for 3/4-inch, crushed rock in Table 200-1.2, of the Greenbook (2009). The pipe zone materials should extend from at least 6 inches below the pipe to 12 inches above the crown.

Pipe zone materials should be properly placed and mechanically compacted in order to achieve a minimum of 90 percent relative compaction as determined by standard test method ASTM D1557. Backfill should be placed in loose lifts no greater than 6-inches thick and mechanically compacted. Gravel, if used for pipe zone backfill, should be placed in 6-inch lifts and mechanically densified/vibrated.

The trench width should be sufficient to allow compaction equipment to operate between the pipe springline and trench wall. We recommend that the trench be a least 2 feet wider than the pipe on each side to allow for shoring installation and compaction of the backfill. Jetting or flooding of pipe-zone materials should not be allowed.

3.6.3 Trench Backfill Materials

Trench backfill materials are herein defined as those materials placed above the pipe zone. We anticipate that the native volcanic rock that will be excavated from the site should be suitable as trench backfill, as long as those materials are screened to remove particles, blocky materials, or lumps, larger than 3 inches in largest dimension. In addition, the trench backfill material should contain no more than 15 percent material larger than 2 inches and the larger-size materials should not be placed in concentrated pockets.

Trench backfill should be spread in loose lifts not to exceed 6 inches in thickness, moisture-conditioned to within 2 percent of optimum moisture content for coarse-grained soils and between optimum moisture and 2 percent above optimum moisture content for fine-grained soils, and then compacted to 90 percent of the maximum dry density as determined from ASTM

D1557. The upper 1 foot of the subgrade beneath paved areas should be compacted to 95 percent of the maximum dry density ASTM D1557.

3.6.4 Filter between Pipe Zone and Gravel Backfill Materials

If gravel material is used for pipe zone backfill, there is a potential for soil particles to migrate into the interstices of the crushed rock pipe zone materials. Should that occur, settlement of the ground surface is possible. Migration of finer soil particles may occur from seeping groundwater or possibly traffic vibrations. We anticipate that a majority of the migration would result from vertical (downward) migration of trench backfill materials into the gravel pipe zone backfill.

There are several possible mitigations to reduce the amount of soil migration into the crushed gravel pipe backfill in areas where ground surface settlement would be problematic (e.g., roadways, areas sensitive to surface drainage characteristics). Where this is a concern, we recommend that the gravel pipe-zone backfill be fully encapsulated in filter fabric.

3.7 External Pipeline Loads

External loads on the pipes will consist of overlying earth materials, loads due to construction activities, and loads related to traffic or other post-construction land uses. The pipes should be designed to resist the imposed loads with a factor of safety and for an appropriate limiting deflection.

Loads on the pipe due to the overlying soil will be dependent upon the depth of placement, the type and method of backfill, the type of pipe, the configuration of the trench, and whether or not any fill will be placed above the ground surface. The earth loads on the pipe can be estimated using formulas developed by Marston (1930) and are dependent on whether 'trench conditions' or 'embankment conditions' exist along the alignment. Trench conditions are defined as those in which the pipe is installed in a relatively narrow trench, cut in undisturbed ground, and covered with earth backfill to the original ground surface. Embankment conditions are defined as those in which the pipe is covered with fill above the ground surface or when a trench in undisturbed ground is so wide that trench wall side friction does not affect the load on the pipe. It is likely that "embankment" conditions will be most appropriate for pipeline design because of the side-by-side configuration of the pipelines. Where the trenches are narrow, "trench" conditions may apply. When using Marston's formulas, the total unit weight of the backfill materials should be assumed to be 130 pcf.

The pipe may be subjected to surcharge pressures and/or point loads due to construction activities and traffic. Those other loading conditions should be considered in the design of the pipe.

3.8 Modulus of Soil Reaction

Flexible and semi-rigid pipes are typically designed to withstand a certain amount of deflection from the applied earth loads. Those deflections can be estimated with the aid of equations presented by Spangler and Handy (1982) or Howard (1996).

Based on subsurface conditions and the anticipated pipe bedding materials, we recommend a modulus of soil reaction (E') value of 2,500 pounds per square inch be used for design. The recommended design E'-value was estimated from Howard et al. (1995).

3.9 Thrust Resistance

Where the proposed pipeline changes direction abruptly, resistance to thrust forces can be provided by mobilizing frictional resistance between the pipe and the surrounding soil, by use of a thrust-block, or by a combination of the two. We anticipate that thrust resistance for the pipeline will probably be provided using restrained joints in conjunction with mobilized pipeline/soil frictional resistance.

3.9.1 Frictional Resistance

To utilize thrust resistance using frictional resistance, we recommend the following parameters to estimate the ultimate frictional resistance:

- Total unit weight, 125 pounds per cubic foot
- Buoyant unit weight, 62 pounds per cubic foot
- Coefficient of lateral pressure, at rest, k_o, 0.5
- Coefficient of friction, 0.30 between the pipe zone backfill and mortar-coated cement lined welded steel pipe

3.9.2 Thrust Blocks

Thrust blocks can be designed based on the potential applied lateral stress imparted by the pipeline. For thrust blocks bearing directly against undisturbed volcanic rock (or a relatively large zone of compacted fill), passive lateral pressures may be computed using an ultimate equivalent fluid weight for passive resistance of 350 pounds pcf assuming unsaturated soil conditions.

The passive resistance values are ultimate values and were estimated using a total unit weight of 125 pounds pcf and an assumed average friction angle or the trench fill of 30 degrees. The estimated lateral displacement needed to develop the ultimate passive pressure for a 4-foot-high thrust-block is about 1/2-inch (about 1 percent of block height). Lateral bearing should be neglected from the ground surface to a depth of 1-foot below lowest adjacent grade.

In consideration of the allowable thrust-block deflection and attendant potential for pipe-joint separation, we recommend that thrust-block design use a factor of safety of 2.

3.10 SOIL CHEMISTRY AND CORROSION

3.10.1 Test Results

Two selected composite soil samples were tested to evaluate resistivity, pH, chlorides, and sulfates. The results are presented below in Table 7. The laboratory test report is included in Appendix A.

Table 7. Summary of Chemical Test Results

Sample Number	Sample Depth (feet)	Material Description	Resistivity (ohms/cm)	pH (units)	Chloride (ppm)	Sulfate (wt %)
CR-1	Surface	Yellowish-brown silty sand w/ gravel & roots	5,874	7.0	2	0.0162
CR-2	Surface	Dark-brown silty sand, trace gravel & roots	3,313	8.3	<2	0.0004

1) ppm = parts per million
2) To convert ppm to percent of soil weight, divide ppm by 10,000.

3.10.2 Corrosion

On the basis of the Caltrans Corrosion Guidelines (2012), the chloride concentration, sulfate concentration, and pH test results are considered "not corrosive" for structural elements. However, the resistivity test results indicate that the tested materials are moderately corrosive to metal. The corrosion results should be evaluated by a corrosion engineer to assess the need for corrosion protection.

3.10.3 Cement Type

Per Table 4.3.1 in ACI 318 (2005), the results of the sulfate tests suggest that corrosivity toward concrete is negligible for the tested samples. Type II cement can be used for concrete, unless other environmental or operating conditions indicate a more severe sulfate exposure. We recommend imported backfill material (if used) be tested for corrosion potential prior to use on the project.

3.11 ADDITIONAL SERVICES

3.11.1 Plan Review

We recommend that Fugro provide a general review of the grading, improvement, and foundation plans. The purpose of that review is to assess general compliance with the earthwork and foundation recommendations of this report, and to confirm that the recommendations given in this report are incorporated in the project design plans and specifications.

3.11.2 Observation and Testing

The construction process is an integral part of the geotechnical design. We recommend that the geotechnical professional be retained during the grading, excavation, and foundation phases of the work. The temporary cut slopes, foundation excavations, and subgrade conditions should be observed at the time of construction. The purpose of these services is to observe compliance with the initial development design concepts, the specifications, and the geotechnical recommendations. The observation and testing services will allow for changes in the recommendations in the event that subsurface conditions differ from those anticipated prior to construction.

4.0 CLOSURE

Fugro prepared the conclusions and professional opinions presented in this report according to generally accepted geotechnical engineering practices at the time and in the region that this report was prepared. This statement is in lieu of all warranties, express or implied.

The conclusions and recommendations presented in this report are based on the results of our field exploration and laboratory testing programs, engineering evaluation, and our understanding of the project. Our recommendations for site preparation and grading, fill placement, and for the design of the proposed concrete tank are presented herein.

This report has been prepared for the exclusive use of AECOM and the Las Virgenes Municipal Water District for design of the proposed 5-MG tank and pipeline project at the Westlake Reservoir, Westlake Village, California. The report may not contain sufficient information for other parties or other uses. If any changes are made in the project described in this report, the conclusions and recommendations contained in this report should not be considered valid. Fugro should review any changes in the project, and modify and approve in writing the conclusions and recommendations of this report for those changes. This report and the figures contained in this report are intended for input to the design of the proposed tank; they are not intended to act as construction drawings or specifications.

Soil and rock deposits vary in type, strength, and other geotechnical properties between points of observations and explorations. Additionally, groundwater and soil moisture conditions vary seasonally or for other man-induced and natural reasons. Therefore, we do not and cannot have complete knowledge of subsurface conditions underlying the site. The criteria presented in this report are based upon findings at the points of exploration and on interpolation and extrapolation of information obtained at the points of observation.

The scope of our services did not include the assessment of the presence or absence of hazardous/toxic substances in the soil, groundwater, surface water, or atmosphere. Statements in this report regarding odors or conditions observed are strictly for descriptive purposes and are not intended to convey engineering judgment regarding potential hazardous/toxic substances.

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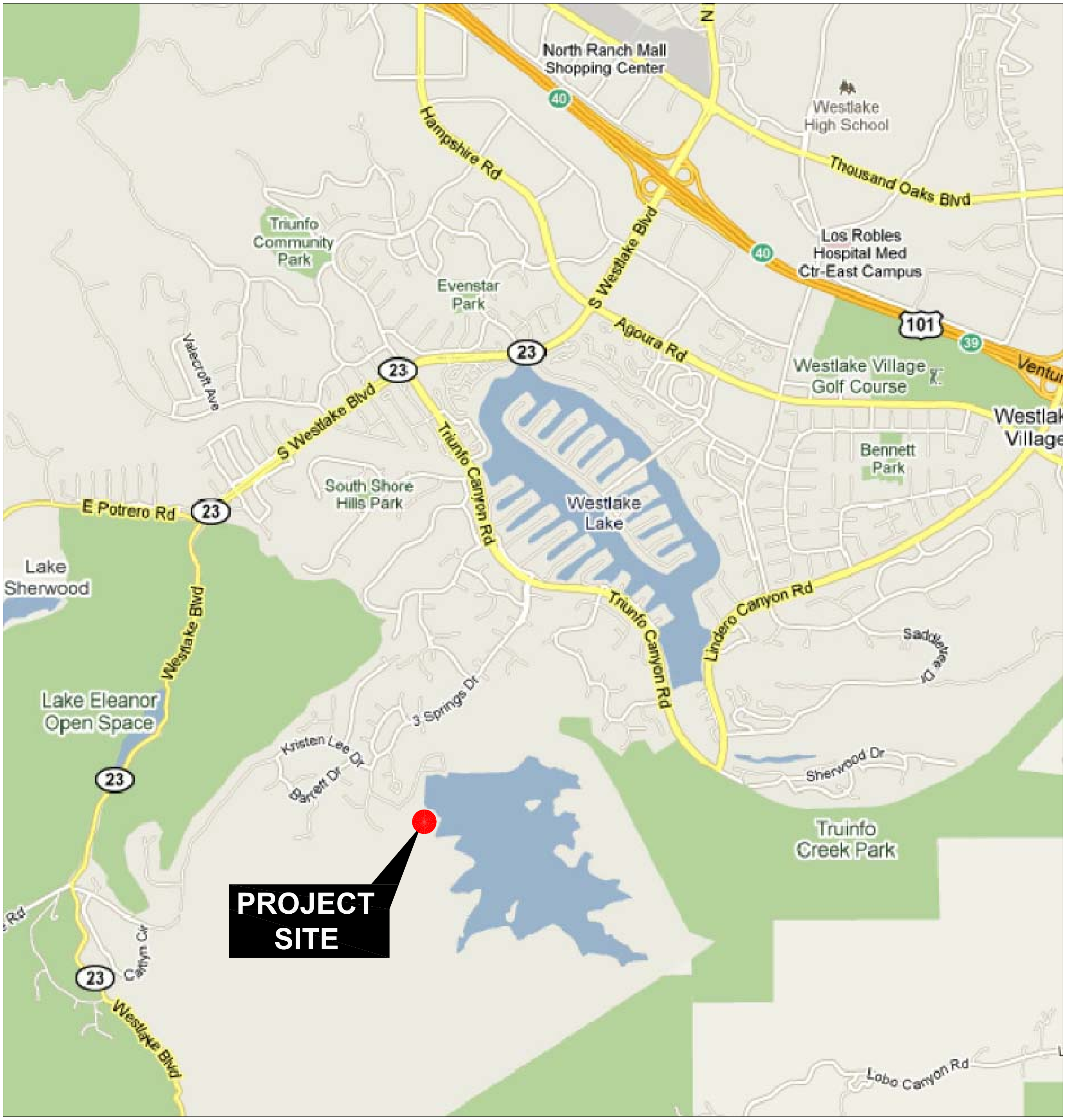
PLATES



BASE MAP SOURCE: Google Maps 2010.

NORTH

0 2400

FEET

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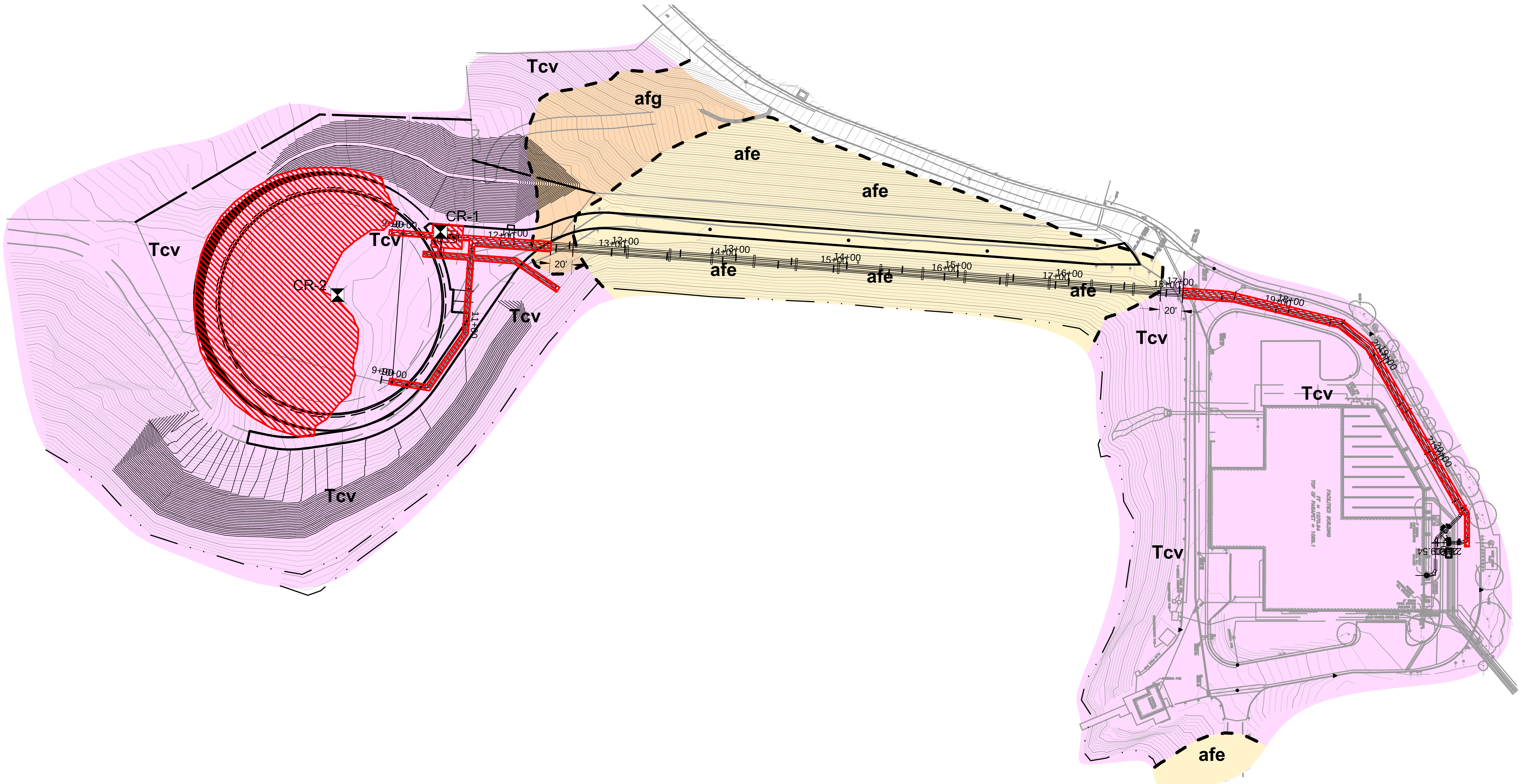
VICINITY MAP

LVMWD 5MG Water Storage Tank

Westlake Village, California

PLATE 1



BASE MAP SOURCE: CAD data from AECOM.

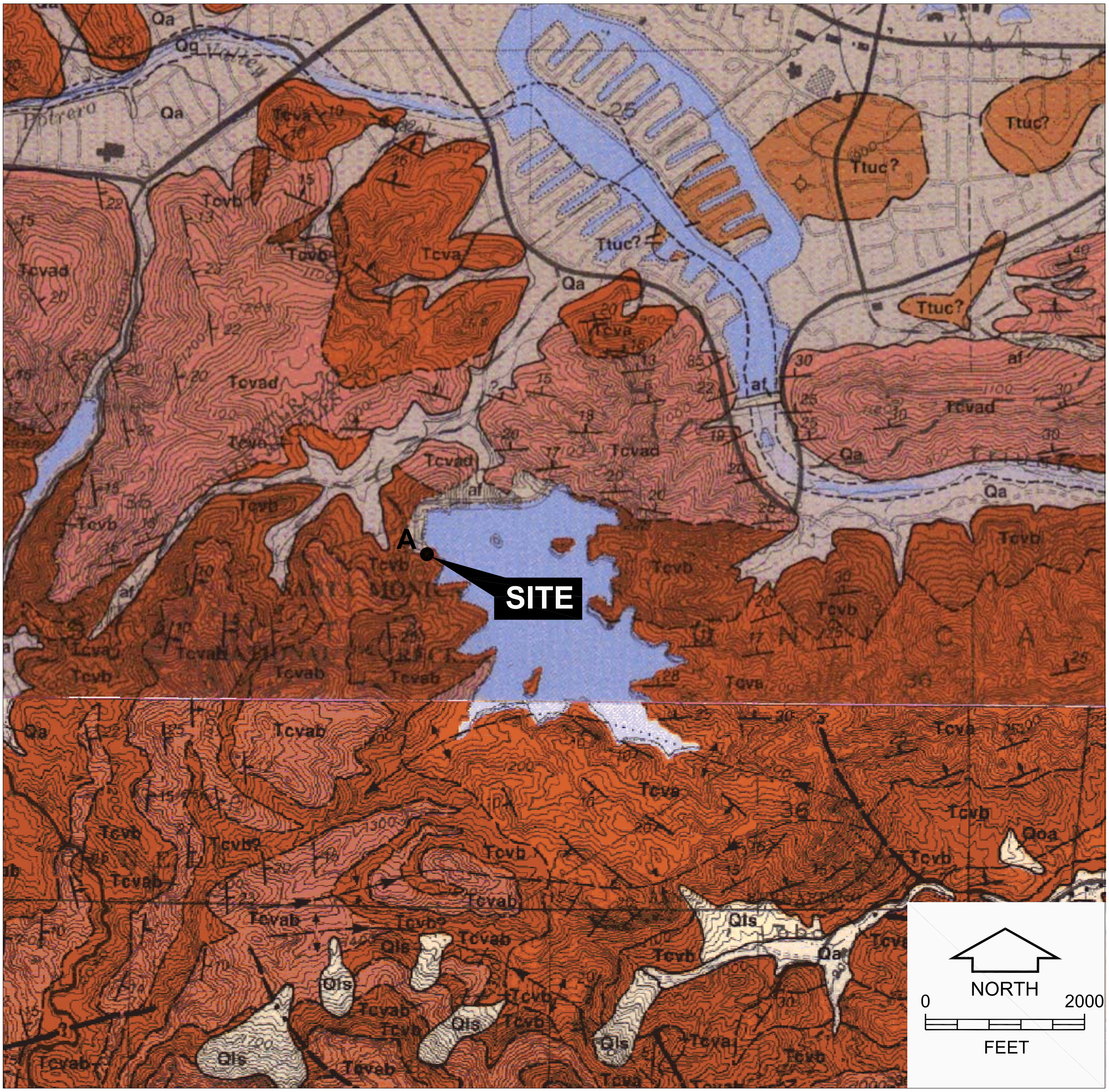
LEGEND

(dashed line)	Geologic contact
afe	Dam Embankment Fill
afg	General Fill
Tcv	Conejo Volcanics
(red hatching)	Areas where blasting may be performed
CR-2	Location of Corrosion Sample

NORTH

0 100 200
FEET

GEOLOGIC MAP
LVMWD 5MG Water Storage Tank
Westlake Village, California



BASE MAP SOURCE: Geologic Map of the Thousand Oaks and Point Dume Quadrangles (Dibblee & Ehrenspeck, 1993).

LEGEND

Qa	Alluvium
Qls	Landslide debris
Qoa	Older surficial sediments
Tm	Monterey Formation
Tmss	Monterey Formation, light gray to tan, semi-friable bedded sandstone
Tml	Monterey Formation, lower part
Tcvab	Conejo Volcanics, andesitic beccia with cobble to boulder-size inclusions
Tcva	Conejo Volcanics, andesitic flows, fine grained
Tcvb	Conejo Volcanics, basalt flows and breccia massive to vaguely bedded
Tcvad	Conejo Volcanics, andesite-dacite breccia with cobble to boulder-size angular inclusions
Tls	Detrital sediments of Lindero Canyon, sandstone
Tlsc	Detrital sediments of Lindero Cyn., conglomerate of granitic detritus
Tlvc	Detrital sediments of Lindero Cyn., basal epiclastic conglomerate
Ttuc	Upper Topanga Formation, clay shale and siltstone
Kcs	Chatsworth Formation, light gray to light brown sandstone

Formation contact

Member contact

Contact between surficial sediments

Anticline, dashed where approximate, dotted where concealed, arrow on axis indicates direction of plunge

Syncline, dashed where approximate, dotted where concealed, arrow on axis indicates direction of plunge

Strike and Dip of Beds:

20 inclined

inclined (approx.)

cv White calcite or travertine vein

Sandstone marker bed

A Approximate location of tank site

REGIONAL GEOLOGIC MAP

LVMWD 5MG Water Storage Tank
Westlake Village, California

PLATE 3

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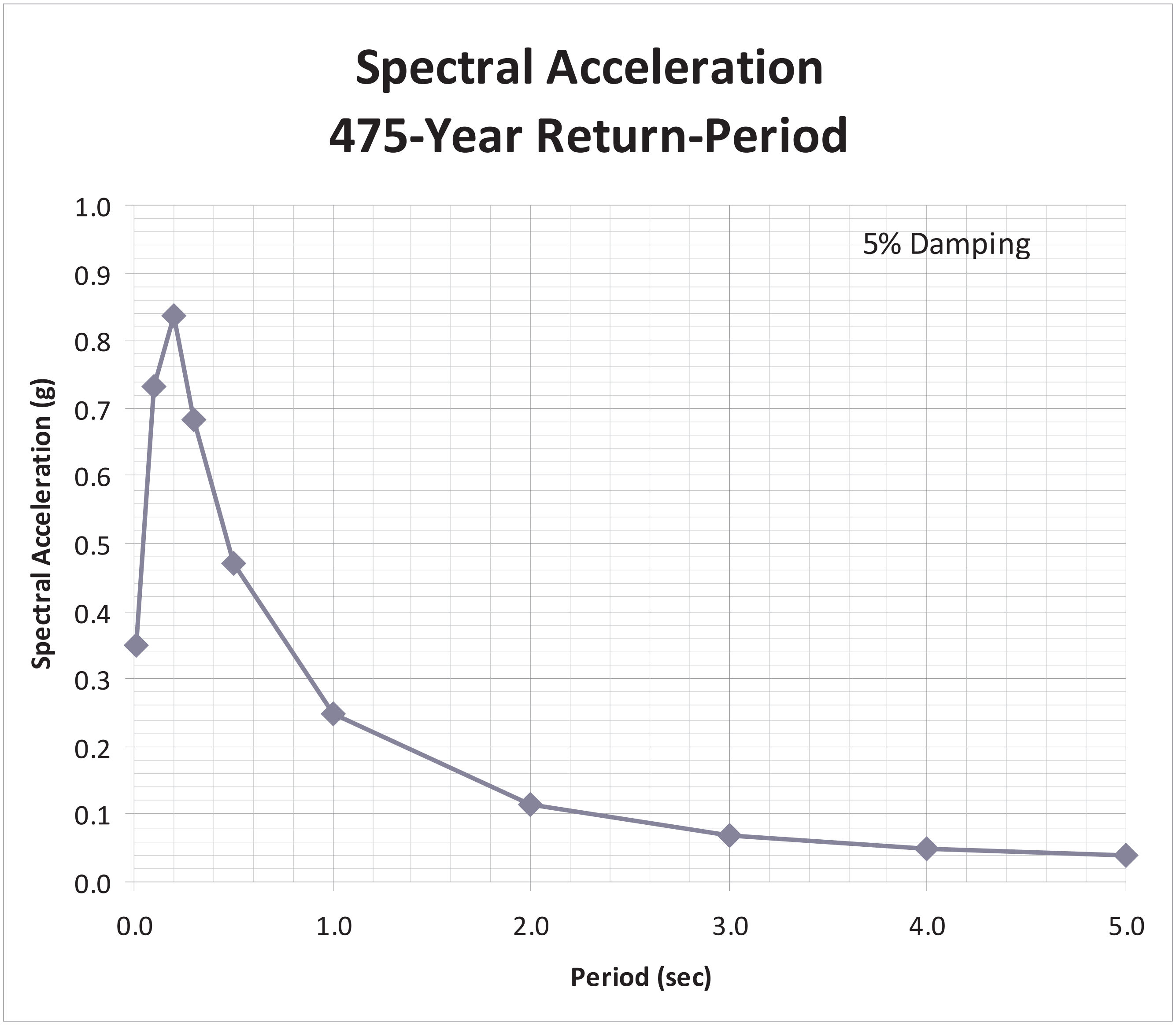

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475-YEAR RETURN PERIOD
RESPONSE SPECTRA
LVMWD 5MG Water Storage Tank
Westlake Village, California

APPENDIX A
LABORATORY TESTING

APPENDIX A
LABORATORY TESTING

INTRODUCTION

The contents of this appendix shall be integrated with the geotechnical engineering study of which it is a part. The data contained in this appendix shall not be used in whole or in part as a sole source for information or recommendations regarding the subject site.

LABORATORY ANALYSES

Laboratory tests were performed on bulk soil samples to estimate soil chemistry and corrosion characteristics of the earth materials selected. Testing was performed in general accordance with ASTM Standards for Soil Testing, latest revision.

Soil Chemistry Tests/Corrosion Tests

Soil chemistry tests were performed on two samples to evaluate resistivity, pH, sulfate, and chloride. The testing was performed by Cooper Testing of Palo Alto, California. The results of the testing and an analysis of the corrosivity to pipe and concrete materials are summarized in Section 3.10 of this report, and are shown on Plate A-1.

Corrosivity Test Summary

CTL # 446-168 **Date:** 2/20/2013 **Tested By:** PJ **Checked:** PJ

Client: Fugro **Project:** Westlake Res. 5MG Tank **Proj. No:** 04.62120197

Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate-(water soluble)		pH	ORP		Sulfide	Moisture	Soil Visual Description
Boring	Sample, No.	Depth, ft.	As Rec.	Minimum	100% Saturated	mg/kg Dry Wt.	mg/kg Dry Wt.	% Dry Wt.		(Redox) E_H (mv)	At Test Temp °C	Qualitative by Lead	% At Test	
			ASTM G57	Cal 643	ASTM G57	ASTM D4327 / Cal 422-mod.	ASTM D4327 / Cal 417-mod.		ASTM G51	ASTM G200		Acetate Paper	ASTM D2216	
-	CR1	-	-	-	5,874	2	162	0.0162	7.0	-	-	-	11.7	Yellowish Brown Silty SAND w/ Gravel & Roots
-	CR2	-	-	-	3,313	<2	4	0.0004	8.3	-	-	-	12.5	Dark Brown Silty SAND, trace Gravel & Roots