APPENDIX 4

GEOTECHNICAL EXPLORATION
MSWD VISTA RESERVOIR TANK SITE
VALENCIA DRIVE, DESERT HOT SPRINGS
COUNTY OF RIVERSIDE, CALIFORNIA

Prepared for

TKE ENGINEERING, INC.
2305 Chicago Avenue
Riverside, California 92563

Project No. 12761.001

September 18, 2020

Leighton Consulting, Inc.
A LEIGHTON GROUP COMPANY

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TKE Engineering, Inc.
2305 Chicago Avenue
Riverside, California 92563

Attention: Mr. Steven Ledbetter, PE and/or Ms. Yesenia Diaz

Subject: Geotechnical Exploration
MSWD Vista Reservoir Tank Site
Valencia Drive, Desert Hot Springs, County of Riverside, California

In accordance with your authorization and our proposal dated April 29, 2020, we performed a geotechnical exploration for the subject Site. This report presents our findings and provides our geotechnical recommendations for design and construction of the proposed improvements. Based on the results of our exploration, the proposed tank site is generally underlain by dense silty sand soils with varying amounts of gravel and cobbles (Fanglomerate). Based on published geologic maps, the site is located within currently designated County Fault Hazard Zone. From a geotechnical perspective, the constructability of proposed improvements is considered feasible provided the recommendations included in this report are implemented during design and construction phases.

The opportunity to be of service is sincerely appreciated. If you should have any questions, please do not hesitate to call our office.

Respectfully submitted,
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1.0 INTRODUCTION

1.1 Site Description

The Vista Reservoir Tank Site is located at the northern terminus of Valencia Drive in the City of Desert Hot Springs (see Figure 1). The site is currently occupied by 30,000-gallon circular steel tank. This overall site is currently a vacant hillside parcel that slopes moderately down to the north and west. Site access is through a locked gate off Valencia Drive, through the driveway to the existing tank. As can be seen on Figure 1, this parcel is bordered by desert hills to the north, east and south, and existing single-family residences to the west.

1.2 Project Description

Based on information provided, we understand that a new 30,000-gallon steel welded tank with 40-foot diameter, will be located approximately 30 feet northeast of the existing tank, along with related auxiliary structures. The pad for the new tank will require a cut into an existing slope along the east side that may require a retaining wall up to 15 feet in height. The design pad grades (Elevation ~1609 feet) will also require up to 4 feet of fill along the west side of the pad (see Figure 2). The proposed tank/reservoir will have a similar diameter as the existing tank (~40 feet) and hydrostatic pressure is expected to be less-than (≤) 2,000 pounds-per-square-foot.

1.3 Purpose and Scope of Exploration

The purpose of our exploration is to (1) evaluate geotechnical engineering characteristics of the earth materials for the project site, and (2) provide geotechnical recommendations for design and construction of the proposed improvements. As described in our proposal, the scope of our evaluation included the following tasks:

- Desktop Review: Reviewed available in-house and published geologic reports/maps (USGS, CGS, etc.).
- Geotechnical Borings: Drilled, logged and sampled three (3) hollow-stem auger borings within an accessible area of the site to a maximum depth of 25 feet using a truck-mounted drill rig equipped with an 8-inch hollow stem auger. The borings were backfilled with the excavated soils.
- Geotechnical Laboratory testing: Driven "California" ring-lined samples and bulk soil samples will be collected from our borings and transported to our in-house geotechnical laboratory for testing. Tests may include insitu moisture/density, sieve analysis, sand equivalent, expansion potential, maximum dry density/

optimum moisture content, and one corrosivity test (pH and resistivity, chloride and soluble sulfate content).

- Report Preparation: Results of this evaluation have been summarized in this report, presenting our findings, conclusions and geotechnical recommendations for the proposed tank and site improvements.

This report does not address the potential for encountering hazardous materials or fault displacement along this site. Important information about limitations of geotechnical reports is presented in Appendix D.

1.4 Field Exploration

Our field exploration consisted of the excavation of three (3) geotechnical borings (LB-1 through LB-3) in accessible areas within project site. Prior to drilling, we located and marked boring locations for coordination with Underground Service Alert (USA) and MSWD personnel. Our field exploration was performed on May 19, 2020. Approximate locations of the borings are depicted on the Boring Location Plan (Figure 2). The exploratory borings were excavated utilizing a truck-mounted, CME 75 drill rig using an 8-inch hollow-stem flight auger. During the drilling operation, bulk and relatively undisturbed samples were obtained from the borings for laboratory testing and evaluation. Sampling of the borings was conducted by an engineer from our office. The collected samples were transported to our laboratory for testing. Borings were backfilled with native soils. The logs of borings are presented in Appendix A.

1.5 Laboratory Testing

Laboratory tests were performed on representative samples to provide a basis for development of geotechnical design parameters. Selected samples were tested to determine the following parameters: insitu moisture/density, sieve analysis, sand equivalent, maximum dry density/ optimum moisture content, and one corrosivity test (pH and resistivity, chloride and soluble sulfate content). The results of our laboratory testing are presented in Appendix B.

2.0 SUMMARY OF GEOTECHNICAL FINDINGS

A summary of our findings from research of pertinent literature, site-specific field exploration, geotechnical laboratory testing and engineering analysis, is discussed in this section.

2.1 Subsurface Conditions

Our field exploration indicates that the subsurface conditions at the tank facility are primarily underlain by minor amounts of artificial fill underlain by dense Fanglomerate which is turn underlain (unconformably) by gneissic and mafic igneous rocks. Detailed descriptions of the earth materials encountered in each boring are provided on the logs of borings in Appendix A.

2.1.1 Artificial Fill

Artificial fill is expected locally in existing utility trenches and pads of existing equipment and previous grading. The fill appears to be generated from onsite sources and generally consist of silty sand (SM) with varying amounts of gravel. The fill is not expected to exceed 5 feet in depth.

2.1.2 Fanglomerate

These dense materials were found in all of our borings and extends to the explored depth of 26 feet. The Fanglomerate (possibly Whitehouse Canyon Fanglomerate) generally consisted of silty sand (SM) with varying amounts of gravel and cobbles (GP-GM). The N-value ranges from 17 to greater than 50 blows per foot. Based on the results of our laboratory testing on representative samples, the Sand Equivalent (SE) for the majority of onsite materials is expected to be greater than 30 and the Expansion Index (EI) is expected to be less than 21.

2.1.3 Gneissic Bedrock

These dense bedrock materials were found in Boring LB-2 and extends to the explored depth of 15.5 feet. The granitic/gneissic bedrock generally recovered as highly weathered silty sand (SM) with varying amounts of gravel and cobbles (GP-GM). The N-value averaged to greater than 50 blows per foot.

2.2 Surface and Groundwater

Groundwater was not encountered in any of our borings to the maximum explored depth of 26 feet BGS. Based on historic data from existing wells in the vicinity of this site, groundwater is not expected to be shallower than 200 feet below existing site grades, and it indicates groundwater to exist at an approximate elevation 1050 msl according to California Water Data Library Well 339628N1165004W001 located approximately 4,000 feet southwest (recorded on December 18, 2019).

2.3 Faulting and Seismicity

The subject site, like the rest of Southern California, is located within a seismically active region as a result of being located near the active margin between the North American and Pacific tectonic plates. The principal source of seismic activity on this site is movement along the northwest-trending San Andreas Fault. Historically, the San Andreas Fault zone has produced earthquakes in the magnitude range of 7.1Mw to 7.4Mw ('Mw' is the Moment Magnitude as defined by the U.S.G.S). Of all the fault systems in California, the San Andreas Fault is among the most active. Since the recording of seismic events in the mid-19th century, at least 3 major earthquakes have occurred along the San Andreas Fault Zone. Each of these major quakes have produced moderate to severe damage to buildings and roads, and have resulted in several fatalities over this time-period. Hundreds of minor earthquakes (magnitude < 2.9) occur annually in the Coachella Valley. The majority of these earthquakes occur in the bedrock underlying the alluvium unit typically at depths of 3 to 5 miles (5-8 km).

Based on our review of published geologic map (Hart, 2007), the subject site is not included within an Earthquake Fault Zone per the Alquist-Priolo Earthquake Fault Zoning Act. The Mission Creek branch of the San Andreas Fault Zone is located approximately 7,000 feet southwest of the project site. However, the site is located within the Riverside County Zoned Blind Canyon Fault (see Figure 4). A fault or ground rupture can presumably occur anywhere within the mapped zones unless proven otherwise. This geologic hazard exists for similar Water storage facilities in this region and as well as the existing onsite tank.

For the purpose of structural design, seismic coefficients based on the 2019 California Building Code (CBC) are provided in Table 1 below. These seismic coefficients were calculated based on a software program, available on the United States Geological Survey website, which follows the procedures, included in American Society of Civil Engineers (ASCE) Publication ASCE 7-16.

Table 1. 2019 CBC Site Categorization and Seismic Coefficients

Parameters	Proposed Tank Site
Site Longitude (decimal degrees)	-116.4932°
Site Latitude (decimal degrees)	33.9828°
Site Class Definition	**C**
Mapped Spectral Response Acceleration at 0.2s Period, S_s	2.07
Mapped Spectral Response Acceleration at 1s Period, S_1	0.78
Short Period Site Coefficient at 0.2s Period, F_a	1.2
Long Period Site Coefficient at 1s Period, F_v	1.4
Adjusted Spectral Response Acceleration at 0.2s Period, S_{MS}	2.48
Adjusted Spectral Response Acceleration at 1s Period, S_{M1}	1.09
Design Spectral Response Acceleration at 0.2s Period, S_{DS}	**1.65**
Design Spectral Response Acceleration at 1s Period, S_{D1}	**0.72**

The site modified peak ground acceleration PGAM is 1.03g (see Appendix C for further details). Additionally, we performed a probabilistic seismic hazard analysis utilizing the Unified Hazard Map application provided through the USGS website (USGS, 2020). Probabilistic design level events are defined in Table 2 below, along with calculated PHGA for each design-level.

Table 2. Probabilistic Seismic Hazard Analyses

Design Level	Return Period (years)	Definition	Peak Horizontal Ground Acceleration (g)
DBE	475	10% probability of exceedance in 50 years	0.57
UBE	975	5% probability of exceedance in 50 years	0.75
MCE	2475	2% probability of exceedance in 50 years	1.04

2.4 Secondary Seismic Hazards

The potential for secondary hazards such as seiches and tsunamis, landslide, rockfall, and lateral spreading, are considered very low for the project site. Additional, secondary seismic hazards such as ground rupture and liquefaction are discussed below.

2.4.1 Ground Rupture

As indicated in Section 2.3 above, the site is located within a mapped County Earthquake Fault Zone (see Figure 4), with the proposed tank being located approximately 150 feet northeast of a mapped fault. According to County and State guidelines, a fault or ground rupture can presumably occur anywhere within the mapped zones unless proven otherwise. Although no evidence of site faulting was observed during our field exploration, the evaluation of onsite faulting is beyond the scope of this report.

2.4.2 Dynamic Settlement / Liquefaction

Liquefaction of saturated cohesionless soils can be caused by strong ground motion resulting from earthquakes. Soil liquefaction is a phenomenon in which saturated, cohesionless soils lose their strength due to the build-up of excess pore water pressure during cyclic loading such as that induced by earthquakes. Due to the absence of shallow groundwater, potential for liquefaction is considered non-existent. Furthermore, dynamic settlement can also exist if loose sandy soils are subjected to ground shaking. However, due to the dense nature of underlying materials dynamic dry settlement within the project site is expected to be negligible and not a significant design concern.

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3.0 CONCLUSIONS AND RECOMMENDATIONS

3.1 General

The proposed improvements appear feasible from a geotechnical viewpoint provided that the following recommendations are incorporated into the design and construction phases of development. The soils encountered may be considered CalOSHA Type C soils, and sloped excavations will be required to protect workers, if shoring and/or shields are not used. The site is located within a mapped County Earthquake Fault Zone and risk of ground rupture at this site exists according to County guidelines. However, this risk poses similar risk to the existing tank and the evaluation of onsite faulting is beyond the scope of this report.

3.2 Earthwork Considerations

Earthwork associated with the proposed improvements should be performed in accordance with any applicable MSWD specifications, "Standard Specifications for Public Works Construction" (GreenBook, latest edition) and the recommendations included herein.

3.2.1 Excavation Characteristics

Based on the results of our exploratory borings, the onsite soils should generally be relatively easy to moderately difficult to excavate (boulders & gneissic bedrock) with conventional earthmoving excavation equipment. Excavation should be performed in accordance with the project plans, specifications, and all applicable OSHA requirements. The contractor should be responsible for providing the "competent person" required by OSHA standards. Contractors should be advised that sandy soils (such as the existing, onsite soils) could make excavations particularly unsafe, and hence necessary safety precautions should be taken at all times.

3.2.2 Subgrade Preparation/Remedial Grading

The subgrade preparation for the proposed improvements should be as follows:

- ***Proposed Tank***– The upper 5 feet (minimum) of existing soils, or 3 feet below bottom of ring foundations (whichever deeper) should be removed and recompacted prior to foundation construction or placement of new fill. This remedial grading is not required if bottom of footings is founded a minimum of 6 feet below existing ground surface or bearing solely on undisturbed Fanglomerate pending verification by the geotechnical consultant. This remedial grading should be performed a minimum of 5 feet beyond the limits of improvements/foundations.

- ***Retaining Walls / Auxiliary Structures***– The upper 3 feet (minimum) of existing soils, or 2 feet below bottom of foundations (whichever deeper) should be removed and recompacted prior to foundation construction or placement of new fill. This remedial grading is not required if bottom of footings is founded a minimum of 3 feet below existing ground surface or bearing solely on undisturbed Fanglomerate pending verification by the geotechnical consultant. This remedial grading should be performed a minimum of 3 feet beyond the limits of improvements/foundations.

- ***Pavement / Flatwork*** – Similarly, for any site pavement or hardscape, the upper 1.5 feet of soils should be removed or scarified and recompacted. Localized of over-excavation may be needed depending on the actual conditions encountered during construction.

After completion of the recommended removal of unsuitable soils and prior to fill placement or foundation construction, the exposed bottom/surface should be scarified to a minimum depth of 8-inches and recompacted to unyielding condition or minimum 90 percent relative compaction per ASTM D1557. Subsequently, all structural fill should be compacted minimum of 90 percent relative compaction.

3.2.3 Pipe Subgrade Preparation

Pipe subgrade soils are expected to consist of relatively medium dense to dense silty sand with varying amounts of gravel. These materials should provide adequate seating and support for any proposed pipelines placed on compacted bedding material. Any oversize particles larger than 3-inches in largest dimension, if any within the subgrade, should be removed from the trench bottom and replaced with compacted uniform bedding materials. Where the subgrade becomes disturbed due to localized seepage or surface water, the contractor should excavate the disturbed soils to a maximum depth of 2 feet and replace with suitable materials to provide a stable bottom. Crushed rock (1/2-inch maximum size) may be used if found necessary to stabilize bottom of trench prior to placing bedding materials.

3.2.4 Trench Backfill

Prior to backfilling trenches, pipes should be bedded in and covered with a uniform, granular material that has a Sand Equivalent (SE) of 30 or greater, and a gradation meeting requirement of the pipe manufacturer and District Standards. A minimum cover of 12 inches of bedding material should be provided above the top of the pipe. Pipe bedding should be water-densified in-place. Some onsite soils (SM) may be too silty to be considered for bedding material.

Native soils are generally considered suitable as backfill materials over the pipe bedding zone provided any cobbles are removed prior to backfilling. These materials should be placed in thin lifts moisture conditioned, as necessary, and mechanically compacted to a minimum of 90 percent relative compaction per ASTM

D 1557 or as required per District standard specifications. The actual lift thickness should depend on the compaction equipment used. If rolling equipment including sheepsfoot, smooth-wheel, segmented wheels, etc., the lift should be a maximum of 8 inches in thickness prior to compaction. For hand-directed mechanical equipment as vibratory plates or tamper, the maximum lift thickness should not exceed 4 inches.

3.2.5 Shrinkage and Subsidence

Change in volume of excavated and recompacted soil varies according to initial density, which is a function of soil type and location. This volume change is represented as a percentage increase (bulking) or decrease (shrinkage) in volume of fill after removal and recompaction. Subsidence occurs as natural ground is moisture-conditioned and densified to receive fill. Field and laboratory data used in our calculations included laboratory-measured maximum dry densities for soil types encountered at this site relative to measured, in-place densities of soils sampled. We estimate that shrinkage due to recompaction of onsite soils will vary from one location to another and with depth. We suggest an estimated shrinkage ranging from 5 to 15 percent be considered for the upper 5 feet below ground surface.

3.3 Bearing Capacity and Earth Pressures

3.3.1 Bearing Capacity

A net allowable bearing capacity of 2,500 psf, or a modulus of subgrade reaction of 200 pci may be used for design of footings of appurtenant structures founded into a minimum of 12-inches of compacted fill. A minimum base width of 18 inches for continuous footings and a minimum bearing area of 3 square feet (1.75 ft by 1.75 ft) for pad foundations should be used. Additionally, an increase of one-third may be applied when considering short-term live loads (e.g. seismic and wind). No minimum embedment is required for shallow mat/slab foundations. A minimum of 12-inch embedment should be considered for all isolated shallow spread and continuous footings.

If applicable, lateral loads on thrust blocks and other appurtenant structures may be resisted by passive soil pressure and friction, in combination. An allowable passive pressure based on an equivalent fluid pressure of 350 pounds-per-cubic-foot (pcf), not to exceed 3,500 pounds per square foot (psf) can be used if the pipe is embedded in the alluvium or compacted fill (minimum 2 feet embedment). This equivalent fluid pressure may be doubled for isolated thrust blocks. We have not applied a factor-of-safety to these values. A soil-pipeline surface friction of 0.20 for PVC pipes. A maximum allowable frictional resistance of 0.40 may be used for estimating lateral loads caused by friction between the footings/concrete and the supporting subgrade. Additionally, an increase of one-third may be applied when considering short-term live loads (e.g. seismic and wind).

A modulus of soil reaction (E') of 1,200 psi can be used to estimate the stiffness of the soil bedding backfill at the sides and below buried flexible pipelines for the purpose of evaluating deflection caused by weight of the backfill over the pipe. This value assumes that the proposed pipelines in embedded at 5 feet below exiting grades and a granular bedding material with an average relative compaction of 90 percent or more (per ASTM D1557) is placed.

3.3.2 Soils Parameters for Pipeline Design

Structural design of pipes requires proper evaluation of possible loads acting on the pipe, including dead and live or transient loads. Stresses and strains induced in a buried pipe depend on many factors, including the type of pipe, depth and width of trench, bedding and embedment conditions, soil density, angle of internal friction, coefficient of passive earth pressure, and coefficient of friction at the interface between the backfill and in-situ soils. We recommend the following soil parameters for the proposed pipe design:

Table 3. Soil Parameters for Pipe Design

Soil Parameters	Recommended Values
Average compacted fill moist unit weight, (pcf)	125
Angle of internal friction of soils (degrees)	34
Soil cohesion, c (psf)	0
Sliding friction between pipe and native soils	0.20
Coefficient of friction between backfill and native soils	0.45

3.3.3 External Loads on Pipe by Soil

Structural design of pipes requires proper evaluation of possible loads acting on the pipe, including dead and live or transient loads. Stresses and strains induced. The magnitude of the load supported depends on the amount of backfill, type of soil, and pipe stiffness. For flexible pipes, the approximate dead load per unit length can be calculated from the following formula:

$$W = C\gamma\ BD$$

Where,

W External soil load on pipe: (pounds per foot of pipe)

C Unit less load coefficient (C = 1.4 for 5 feet deep trench, and 1.8 for 10 feet deep trench, assuming a trench width of 3 feet just above the pipe)

γ Total unit weight of soil above pipe (pounds-per-cubic-foot)

B Width of the trench (width just above top of the pipe, in feet)

D Pipe diameter (feet)

In addition to the load from backfill (above equation), loads due to embankments (if applicable) and other loads (live loads) should be considered.

3.4 Temporary Cut Slopes

The contractor is responsible for all temporary slopes and trenches excavated at the site and the design of any required temporary shoring. Shoring, bracing and benching should be performed by the contractor in accordance with the current edition of the *California Construction Safety Orders*, see:

http://www.dir.ca.gov/title8/sb4a6.html

During construction, exposed earth material conditions should be regularly evaluated to verify that conditions are as anticipated. The contractor is responsible for providing the "competent person" required by OSHA standards to evaluate soil conditions. Close coordination between the competent person and geotechnical consultant should be maintained to facilitate construction while providing safe excavations. Existing alluvial soils encountered are classified as OSHA soil Type C. Therefore, unshored temporary cut slopes should be no steeper than 1½:1 (horizontal:vertical), for a height no-greater-than (≤) 20 feet (*California Construction Safety Orders*, Appendix B to Section 1541.1, Table B-1). These recommended temporary cut slopes assume a level ground surface for a distance equal to one-and-a-half (x1.5) the depth of excavation. For steeper temporary slopes, deeper excavations, and/or where slopes terrain exists within close proximity to excavation (<1.5xdepth), appropriate shoring methods or flatter slopes may be required to protect the workers in the excavation and adjacent improvements. Such methods should be implemented by the contractor and approved by the consultant.

3.5 Retaining Walls

The pad for the new tank will require a cut into an existing slope along the east side that may require a retaining wall up to 15 feet in height. Retaining wall earth pressures are a function of the amount of wall yielding horizontally under load. If the wall can yield enough to mobilize full shear strength of backfill soils, then the wall can be designed for "active" pressure. If the wall cannot yield under the applied load, the shear strength of the soil cannot be mobilized and the earth pressure will be higher. Such walls should be designed for "at rest" conditions. If a structure moves toward the soils, the resulting resistance developed by the soil is the "passive" resistance. Retaining walls backfilled with non-expansive soils should be designed using the following equivalent fluid pressures:

Table 4. Retaining Wall Design Earth Pressures (Static, Drained)

Loading Conditions	Equivalent Fluid Density (pcf)	
	Level Backfill	2:1 Backfill
Active	36	50
At-Rest	55	85
Passive*	300	150 (2:1, sloping down)

* This assumes level condition in front of the wall will remain for the duration of the project, not to exceed 4,500 psf at depth.

Unrestrained (yielding) cantilever walls should be designed for the active equivalent-fluid weight value provided above for very low expansive soils that are free draining. In the design of walls restrained from movement at the top (non-yielding) such as basement or elevator pit/utility vaults, the at-rest equivalent fluid weight value should be used. Total depth of retained earth for design of cantilever walls should be measured as the vertical distance below the ground surface measured at the wall face for stem design, or measured at the heel of the footing for overturning and sliding calculations. Should a sloping backfill other than a 2:1 (horizontal:vertical) be constructed above the wall (or a backfill is loaded by an adjacent surcharge load), the equivalent fluid weight values provided above should be re-evaluated on an individual case basis by us. Non-standard wall designs should also be reviewed by us prior to construction to check that the proper soil parameters have been incorporated into the wall design.

The above equivalent fluid pressures do not include the effect of earthquake loading. Based on recent studies (Sitar, et. al., 2013), a uniform pressure distribution of 16H (psf) or incremental earth pressures of 26 pounds-per-cubic-foot (pcf) may be considered to estimate seismic lateral pressures acting against retaining walls for level backfill. An incremental earth pressures of 40 pounds-per-cubic-foot (pcf) shoudl be considered to estimate seismic lateral pressures acting against retaining walls with 2:1 sloped backfill. These pressures need only to be applied to walls supporting more than 6 feet of level backfill per the 2019 California Building Code.

All retaining walls should be provided with appropriate drainage. The outlet pipe should be sloped to drain to a suitable outlet. Wall backfill should be non-expansive (EI $\leq$ 21) sands compacted by mechanical methods to a minimum of 90 percent relative compaction (ASTM D 1557). Clayey site soils should not be used as wall backfill. Walls should not be backfilled until wall concrete attains the 28-day compressive strength and/or as determined by the Structural Engineer that the wall is structurally capable of supporting backfill. Lightweight compaction equipment should be used, unless otherwise approved by the Engineer.

3.6 Dewatering

Based on the results of our exploration, no groundwater was encountered within the borings performed. If encountered during excavations, groundwater control, such as dewatering, will be required to limit instability of the excavation bottom, side and face, and aid foundation construction and soil backfill. Groundwater due to perched saturated conditions can be dewatered utilizing sump-pumps. Dewatering or any other suitable method for stabilizing excavation bottom may be selected by the contractor based on actual groundwater conditions encountered and based on the contractor's chosen means-and-methods of construction. The selected method by the contractor should be able to effectively mitigate for bottom heave or stabilize subgrade soils during construction/ backfilling.

3.7 Corrosivity Testing

Sulfate ions in the soil can lower soil resistivity and can be highly aggressive to portland cement concrete by combining chemically with certain constituents of the concrete, principally tricalcium aluminate. This reaction is accompanied by expansion and eventual disruption of the concrete matrix. Potentially high sulfate content could also cause corrosion of the reinforcing steel in concrete. Table below summarizes current standards for concrete exposed to sulfate-containing solutions.

Table 5. Sulfate Concentration and Sulfate Exposure

Sulfate In Water (parts-per-million)	Water-Soluble Sulfate (SO4) in soil (percentage by weight)	Sulfate Exposure
0-150	0.00 - 0.10	Negligible
150-1,500	0.10 - 0.20	Moderate (Seawater)
1,500-10,000	0.20 - 2.00	Severe
>10,000	Over 2.00	Very Severe

The sulfate content was determined in the laboratory for representative onsite soil sample. The results indicate that the water soluble sulfate range is less than 0.1 percent by weight for this site, which is considered negligible as per Table above. Based on the test results, Type II cement or equivalent may be used.

Many factors can affect corrosion potential of soil including soil moisture content, resistivity, permeability and pH, as well as chloride and sulfate concentration. In general, soil resistivity, which is a measure of how easily electrical current flows through soils, is the most influential factor. Based on the findings of studies presented in ASTM STP 1013 titled "Effects of Soil Characteristics on Corrosion" (February, 1989), the approximate relationship between soil resistivity and soil corrosiveness was developed as shown in Table below.

Table 6. Relationship between Soil Resistivity and Soil Corrosivity

Soil Resistivity (ohm-cm)	Classification of Soil Corrosiveness
0 to 900	Very Severely Corrosive
900 to 2,300	Severely Corrosive
2,300 to 5,000	Moderately Corrosive
5,000 to 10,000	Mildly Corrosive
10,000 to >100,000	Very Mildly Corrosive

Acidity is an important factor of soil corrosivity. The lower the pH (the more acidic the environment), the higher the soil corrosivity will be with respect to buried metallic structures and utilities. As soil pH increases above 7 (the neutral value), the soil is increasingly more alkaline and less corrosive to buried steel structures, due to protective surface films, which form on steel in high pH environments. The pH of representative soils sample from the site is 8.50 which is generally considered less corrosive. Chloride and sulfate ion concentrations, and pH appear to play secondary roles in affecting corrosion potential. High chloride levels tend to reduce soil resistivity and break down otherwise protective surface deposits, which can result in corrosion of buried steel or reinforced concrete structures.

Based on laboratory testing results of soil resistivity, the onsite soil is considered **Moderately Corrosive**. Ferrous pipe can be protected by polyethylene bags, tape or coatings, di-electric fittings, concrete encasement or other means to separate the pipe from wet onsite soils. Further testing of import and possibly site soil corrosivity could be performed and specific recommendations for corrosion protection may need to be provided by a qualified corrosion engineer.

Table 7. Corrosion Sample Results

Boring	Sample Depth (ft)	Sulfate Content (ppm)	Chloride Content (ppm)	pH	Minimum Resistivity (ohm-cm)
LB-1	0-5	160	80	8.50	3,600
LB-2	0-5	230	-	-	-

3.8 Preliminary Pavement Design

Our preliminary pavement design is based on an assumed R-value of 45 and the guidelines included in Caltrans Highway Design Manual. For planning and estimating purposes, the pavement sections are calculated based on Traffic Indexes (TI) as indicated in Table below:

Table 8. Asphalt Pavement Sections

General Traffic Condition	Design Traffic Index (TI)	Asphalt Concrete (inches)	Aggregate Base* (inches)
Automobile Parking Lanes	4.5	3.0	4.0
	5.0	3.0	4.0
Truck Access & Driveways	6.0	3.5	4.0
	6.5	4.0	5.0

Appropriate Traffic Index (TI) should be selected or verified by the project civil engineer and appropriate R-value of the subgrade soils will need to be verified after completion of site grading to finalize the pavement design. Pavement design and construction should also conform to applicable local, county and industry standards. The Caltrans pavement section design calculations were based on a pavement life of approximately 20 years with a normal amount of flexible pavement maintenance.

For preliminary planning purposes, fire lanes and truck loading areas may be constructed of Portland Cement Concrete (PCC) with a minimum thickness of 6.0 inches assuming light axle loads and an average daily truck traffic (ADTT) of less than 500. For medium/heavy axle loads and an ADT of 500 or more, a minimum PCC thickness of 8 inches should be used, such as for trash corrals and trash truck aprons, loading docks, etc. All PCC pavement should have a minimum 28-day concrete compressive strength of 3,250 psi and have appropriate joints and saw cuts in accordance with either Portland Cement Association (PCA) or American Concrete Institute (ACI) guidelines. PCC subgrade should be compacted to 95 percent relative compaction in the upper 6 inches. For truck lanes and ramps, a 4-inch (minimum) layer of Class 2 aggregate base at 95 percent relative compaction should be considered beneath the PCC paving. This 4-inch layer of Class 2 aggregate may be used beneath other areas of PCC pavement to improve performance. The upper 6 inches of the underlying subgrade soils should also be compacted to at least 95 percent relative compaction (ASTM D1557). Minimum relative compaction requirements for aggregate base should be 95 percent of the maximum laboratory density as determined by ASTM D1557. If applicable, aggregate base should conform to the "Standard Specifications for Public Works Construction" (green book) current edition or Caltrans Class 2 aggregate base.

If pavement areas are adjacent to heavily watered landscape areas, some deterioration of the subgrade load bearing capacity may result. Moisture control measures such as deepened curbs or other moisture barrier materials may be used to prevent the subgrade soils from becoming saturated. The use of concrete cutoff or edge barriers should be considered when pavement is planned adjacent to either open (unfinished) or irrigated landscaped areas.

3.9 Additional Geotechnical Services

Recommendations are based on information available at the time our report was prepared and may change as plans are developed, or if supplemental subsurface exploration is authorized. Leighton Consulting, Inc. should review site, grading and foundation plans, when available, and comment further on geotechnical aspects of the project. Geotechnical observation and testing should be conducted during excavation and all phases of grading. Geotechnical conclusions and preliminary recommendations should be reviewed and verified by us (Leighton Consulting, Inc.) during construction, and revised accordingly if geotechnical conditions encountered vary from our findings and interpretations. Geotechnical observation and testing should be provided:

- Perform fault trenching to determine fault setback zone, if any,
- To observe trench excavation for indications of faulting,
- To approve subgrade soils prior to placing bedding materials,
- During compaction of trench backfill,
- After excavation of all footings and prior to placement of concrete,
- During pavement subgrade and base and/or sub-base preparation, and
- When any unusual conditions are encountered.

4.0 LIMITATIONS

This report was necessarily based in part upon data obtained from a limited number of observances, site visits, soil samples, tests, analyses, histories of occurrences, spaced subsurface explorations and limited information on historical events and observations. Such information is necessarily incomplete. The nature of many sites is such that differing characteristics can be experienced within small distances and under various climatic conditions. Changes in subsurface conditions can and do occur over time. This exploration was performed with the understanding that the project as described in Section 1.2 of this report.

This report was prepared for TKE Engineering, Inc. based on TKE Engineering, Inc. needs, directions, and requirements at the time of our investigation. This report is not authorized for use by, and is not to be relied upon by any party except TKE Engineering, Inc., and its successors and assigns as owner of the property, with whom Leighton Consulting, Inc. has contracted for the work. Use of or reliance on this report by any other party is at that party's risk. Unauthorized use of or reliance on this report constitutes an agreement to defend and indemnify Leighton Consulting, Inc. from and against any liability which may arise as a result of such use or reliance, regardless of any fault, negligence, or strict liability of Leighton Consulting, Inc.

The client is referred to Appendix D regarding important information provided by the Geoprofessional Business Association (GBA) on geotechnical engineering studies and report and their applicability.

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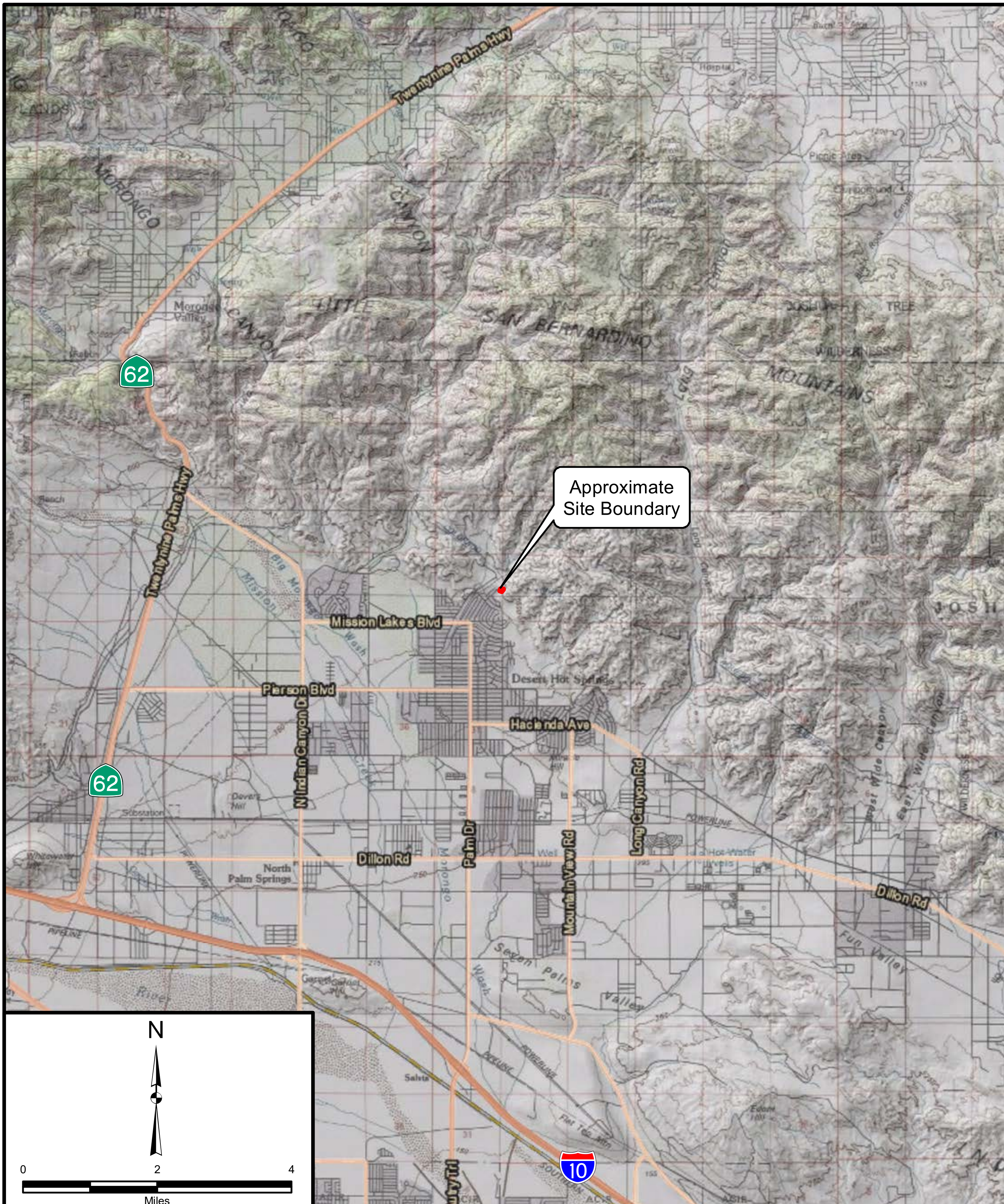


Project: 12761.001	Eng/Geol: SIS/RFR
Scale: 1 " = 2 miles	Date: July 2020

Base Map: Bing Maps 2020
Thematic Information: Leighton
Author: Leighton Geomatics (mmurphy)

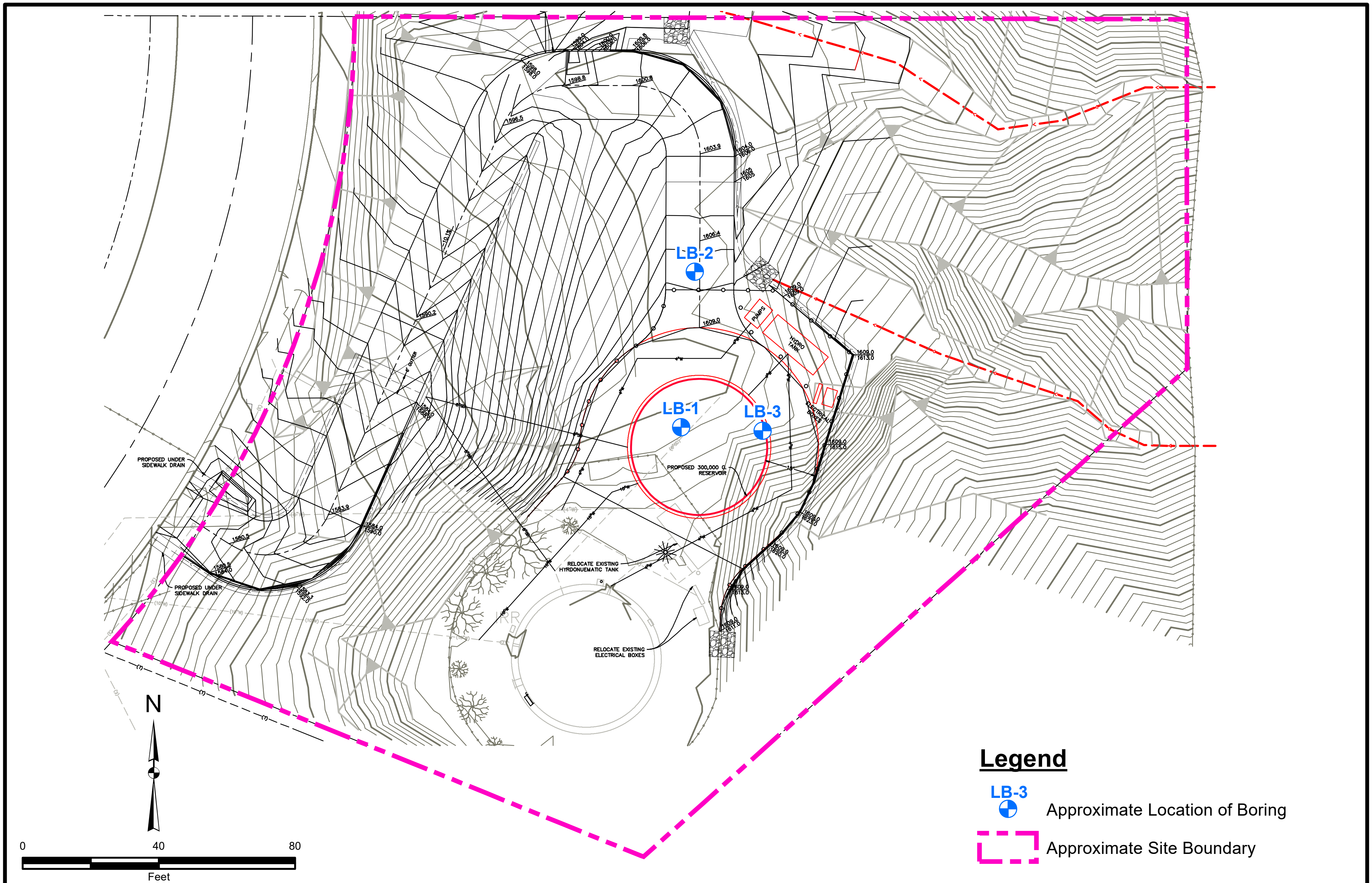
SITE LOCATION MAP

Vista Reservoir Proposed Tank
Mission Springs Water District
Desert Hot Springs, California

Figure 1

Leighton

Map Saved as P:\Drafting\12761\001\Maps\12761-001_F01_SLM_2020-05-27.mxd on 5/27/2020 3:31:24 PM



Project: 12761.001	Eng/Geol: SIS/RFR
Scale:1 " = 40 '	Date: July 2020

Base Map: Vista Reservoirs Proposed Tank Alternative 1 by TKE Engineering.

Author: Leighton Geomatics (mmurphy)

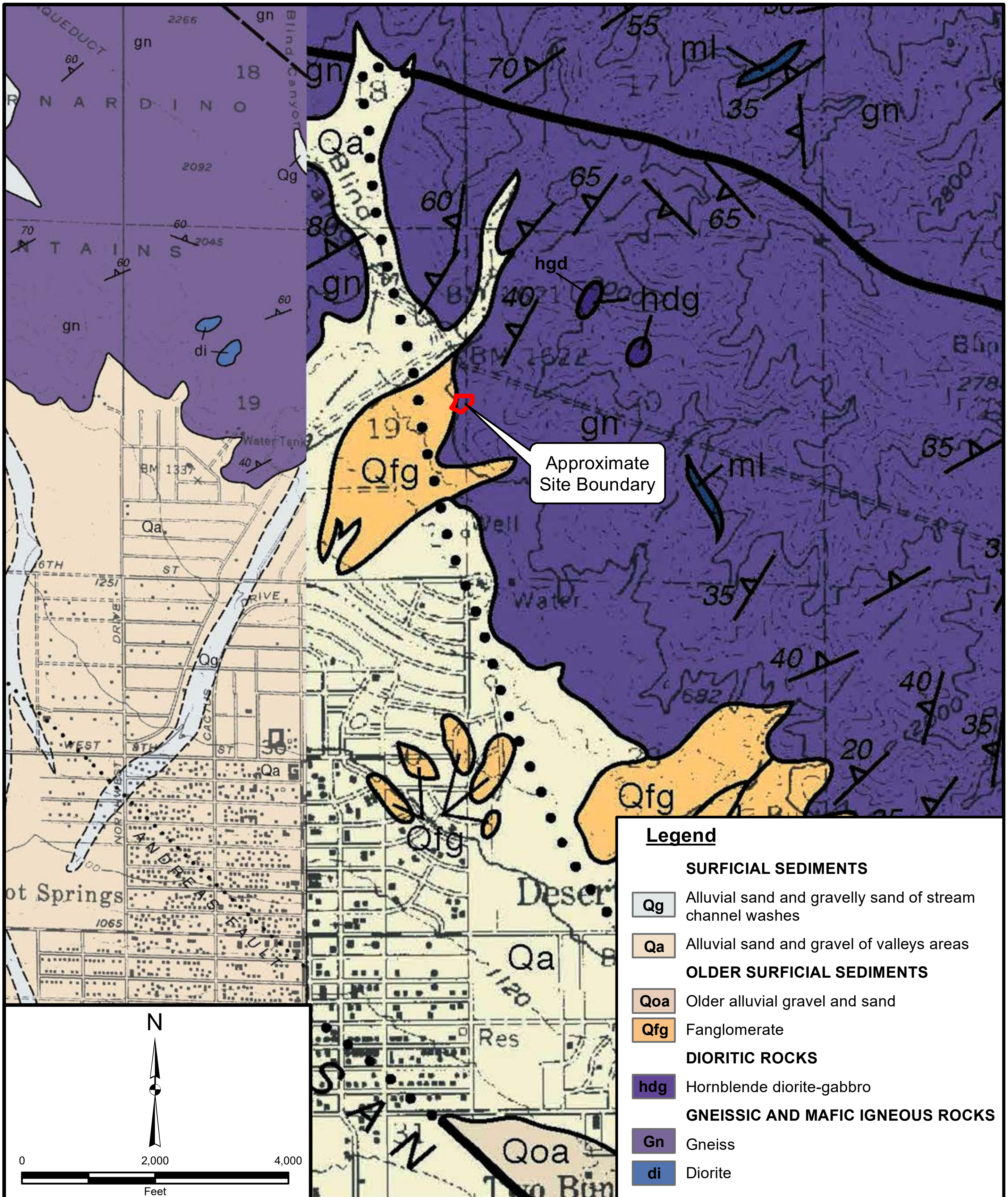
BORING LOCATION MAP

Vista Reservoir Proposed Tank
Mission Springs Water District
Desert Hot Springs, California

Figure 2

Leighton

Map Saved as P:\Drafting\12761\001\Maps\12761-001_F02_BLM_2020-05-27.mxd on 5/27/2020 3:24:00 PM



Project: 12761.001	Eng/Geol: SIS/MSB
Scale: 1 " = 2,000 '	Date: July 2020

Base Maps Geologic Map of the Thousand Palms & Lost Horse Mountain and Desert Hot Springs Quadrangle, by Thomas W. Dibblee Jr., 2004.
Author: Leighton Geomatics (mmurphy)

REGIONAL GEOLOGY MAP

Vista Reservoir Proposed Tank
Mission Springs Water District
Desert Hot Springs, California

Figure 1

Leighton

Map Saved as P:\Drafting\12761\001\Maps\12761-001_F03_RGM_2020-05-27.mxd on 5/27/2020 3:37:48 PM

Project: 12761.001	Eng/Geol: SIS/RFR
Scale: 1 " = 1,000 '	Date: July 2020

Base Map: Bing Maps 2020
Thematic Information: Leighton
Author: Leighton Geomatics (mmurphy)

REGIONAL FAULT MAP

Vista Reservoir Proposed Tank
Mission Springs Water District
Desert Hot Springs, California

Figure 4

Leighton

Map Saved as P:\Drafting\12761\001\Maps\12761-001_F04_RFM_2020-05-27.mxd on 5/27/2020 3:44:03 PM

APPENDIX A

Field Exploration / Logs of Exploratory Borings

Our field exploration consisted of a site reconnaissance and a subsurface exploration program consisting of hollow-stem auger soil borings. Approximate locations of the borings are depicted on the Boring Location Plan (*Figure 2*). Encountered soils were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D 2488). Logs of these subsurface explorations, as well as a key to the classification of the soil, are included as part of this appendix.

Relatively undisturbed soil samples were obtained at selected intervals within the borings using a California ring sampler, with 2.42-inch inside diameter brass rings, driven into the soil with a 140-pound hammer free falling 30-inches in general accordance with ASTM Test Method D3550. The numbers of blows required for each 6 inches of drive penetration were noted in the field and are recorded on the boring logs. Unless otherwise indicated, the blows per foot recorded on the boring logs represent the number of blows required to drive 18 inches in 6 inch increments. In addition, disturbed bag (or bulk) samples were also obtained from soil cuttings. Types of samples obtained from each location are shown on the boring logs at corresponding depths. Our borings were backfilled with soil cuttings obtained during the drilling, and with bentonite grout in some cases. Representative earth-material samples obtained from these subsurface explorations were transported to our Temecula geotechnical laboratory for evaluation and appropriate testing.

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

GEOTECHNICAL BORING LOG LB-1

Project No.	12761.001	**Date Drilled**	5-19-20
Project	TKE MSWD Vista Reservoir	**Logged By**	BSS
Drilling Co.	2R Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	NA'
Location	See Boring Location Map	**Sampled By**	BSS

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			B-1				SM	**Fanglomerate (Qfg):** SILTY SAND with GRAVEL, medium dense, light brown, slightly moist, fine to medium sand, (SA: 11% fines, 37% gravel, MD: 134.0 @ 8.0%, SE = 42)	SA, MD, SE, CR
	5			R-1	5 5 5	110	7		loose, olive brown, moist, fine to coarse sand, more gravel	
	10			R-2	17 50-5"				very dense, dark brown, moist, fine to coarse sand, some gravel (sample disturbed)	
	15			R-3	50-2"			SP-SM	Poorly graded SAND with SILT and GRAVEL, very dense, dark olive brown, moist, fine to medium sand, few coarse sand	
	20			R-4	50-3"			GP-GM	Poorly graded GRAVEL with SILT and SAND, hard, grayish brown, slightly moist, fine to medium sand	
	25			R-5	50-3"				(no recovery)	
									Drilled to 25' Sampled to 20' Groundwater not encountered Backfilled with soil cuttings (5/19/20)	
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG LB-2

Project No.	12761.001	**Date Drilled**	5-19-20
Project	TKE MSWD Vista Reservoir	**Logged By**	BSS
Drilling Co.	2R Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	NA'
Location	See Boring Location Map	**Sampled By**	BSS

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0			B-1				SM	**Fanglomerate (Qfg):** SILTY SAND with GRAVEL, dense, dark brown, moist, fine to coarse sand	
				R-1	15 50-5"	116	4		very dense, brown, moist, fine to coarse sand, subangular, some gravel (quartz)	
	5			R-2	50-6"				**Granitic/Gneiss Bedrock (gn):** Highly weathered bedrock, recovered as: SILTY SAND with GRAVEL, very dense, light gray, slightly moist, very fine to coarse sand	
	10			R-3	40 50-2"				no recovery	
	15			R-4	50-3"				no recovery	
									Drilled to 15' Groundwater not encountered Backfilled with soil cuttings (5/19/20)	
	20									
	25									
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

* * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * *

GEOTECHNICAL BORING LOG LB-3

Project No.	12761.001	**Date Drilled**	5-19-20
Project	TKE MSWD Vista Reservoir	**Logged By**	BSS
Drilling Co.	2R Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	NA'
Location	See Boring Location Map	**Sampled By**	BSS

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0							SM	**Fanglomerate (Qfg):** SILTY SAND, medium dense, light brown, slightly moist, fine to medium sand, few gravel	
				R-1	7 8 8	119	2		medium dense, light brown, slightly moist, fine to coarse sand, some gravel	
	5			R-2	5 8 8	121	2		medium dense, light brown, slightly moist, fine to coarse sand with fine gravel	
	10			R-3	50-5"			SP-SM	Poorly graded SAND with SILT and GRAVEL, very dense, dark gray, dry, fine to medium sand	
	15			R-4	50-4"				very dense, light brownish gray, dry, fine to coarse sand, to Sandy GRAVEL	
	20			R-5	20-3"				no recovery	
									Drilled to 20' Sampled to 15' Groundwater not encountered Backfilled with soil cuttings (5/19/20)	
	25									
	30									

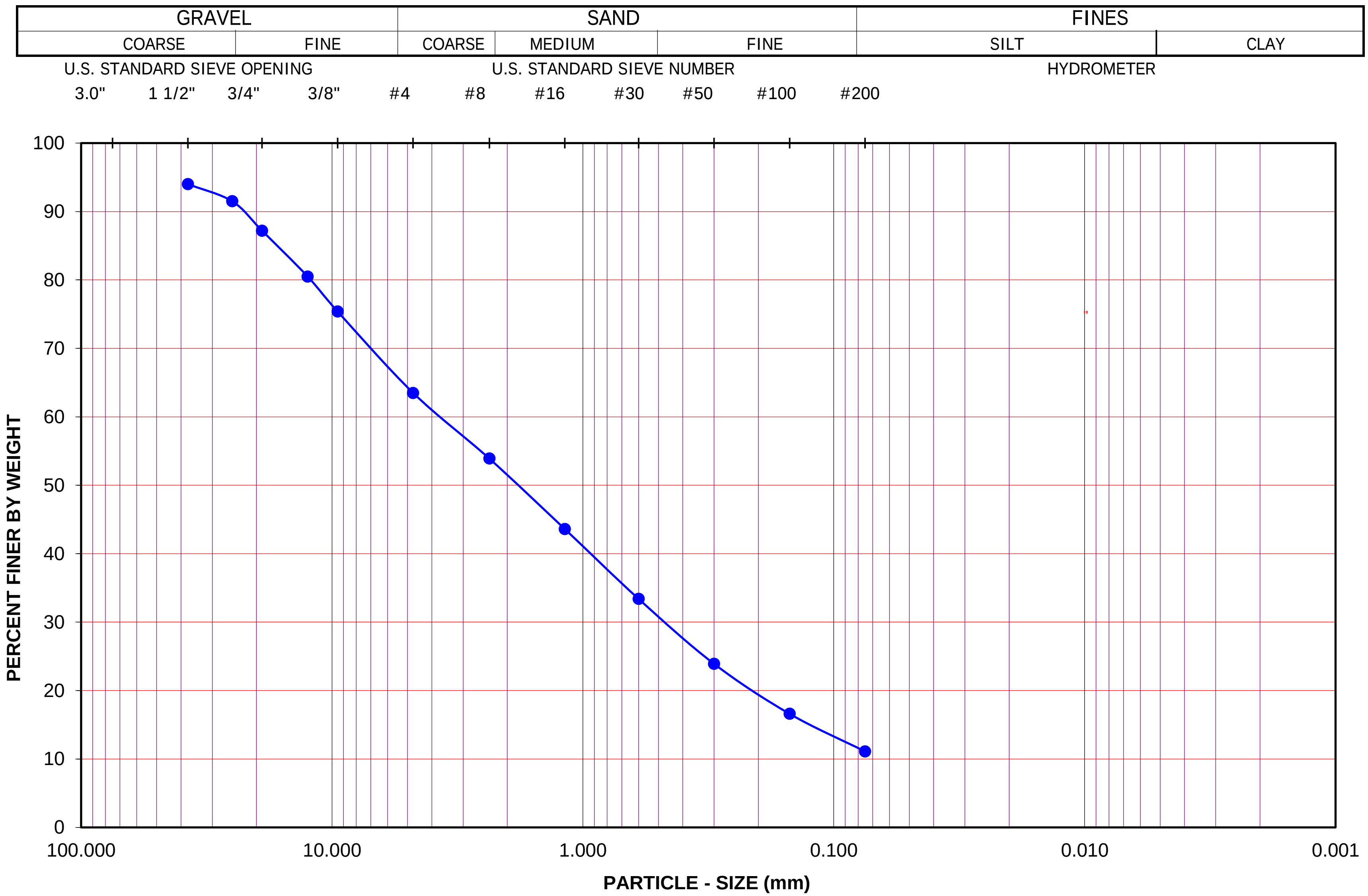
SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

* * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * *

APPENDIX B

Results of Geotechnical Laboratory Testing



Project Name: TKE MSWD Vista Reservoir

Project No.: 12761.001

Boring No.: LB-1 Sample No.: B-1

Depth (feet): 0 - 5.0 Soil Type : (SW-SM)g

Soil Identification: Well-Graded Sand with Silt and Gravel (SW-SM)g, Dark Yellowish Brown.

GR:SA:FI : (%) 37 : 52 : 11

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PARTICLE - SIZE DISTRIBUTION ASTM D 6913

Jun-20

Sieve; LB-1, B-1 (05-19-20)

MODIFIED PROCTOR COMPACTION TEST
ASTM D 1557

Leighton

Project Name:	TKE MSWD Vista Reservoir	Tested By:	F. Mina	Date:	06/05/20
Project No.:	12761.001	Input By:	M. Vinet	Date:	06/17/20
Boring No.:	LB-1	Depth (ft.):	0 - 5.0		
Sample No.:	B-1				
Soil Identification:	Well-Graded Sand with Silt and Gravel (SW-SM)g, Dark Yellowish B				

Note: Corrected dry density calculation assumes specific gravity of 2.70 and moisture content of 1.0% for oversize material

Preparation Method:	X	Moist
		Dry
Compaction Method	X	Mechanical Ram
		Manual Ram

Scalp Fraction (%)	
#3/4	12.8
#3/8	
#4	

Rammer Weight (lb.) = 10.0
Height of Drop (in.) = 18.0
Mold Volume (ft³) 0.07500

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	10270	10397	10216			
Weight of Mold (g)	5559	5559	5559			
Net Weight of Soil (g)	4711	4838	4657			
Wet Weight of Soil + Cont. (g)	3383.1	3366.8	2978.6			
Dry Weight of Soil + Cont. (g)	3156.6	3083.6	2688.9			
Weight of Container (g)	280.8	279.1	278.3			
Moisture Content (%)	7.9	10.1	12.0			
Wet Density (pcf)	138.5	142.2	136.9			
Dry Density (pcf)	128.4	129.2	122.2			

Maximum Dry Density (pcf)	**130.0**	**Optimum Moisture Content (%)**	**9.0**
Corrected Dry Density (pcf)	**134.0**	**Corrected Moisture Content (%)**	**8.0**

☐ **Procedure A**
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
May be used if +#4 is 20% or less

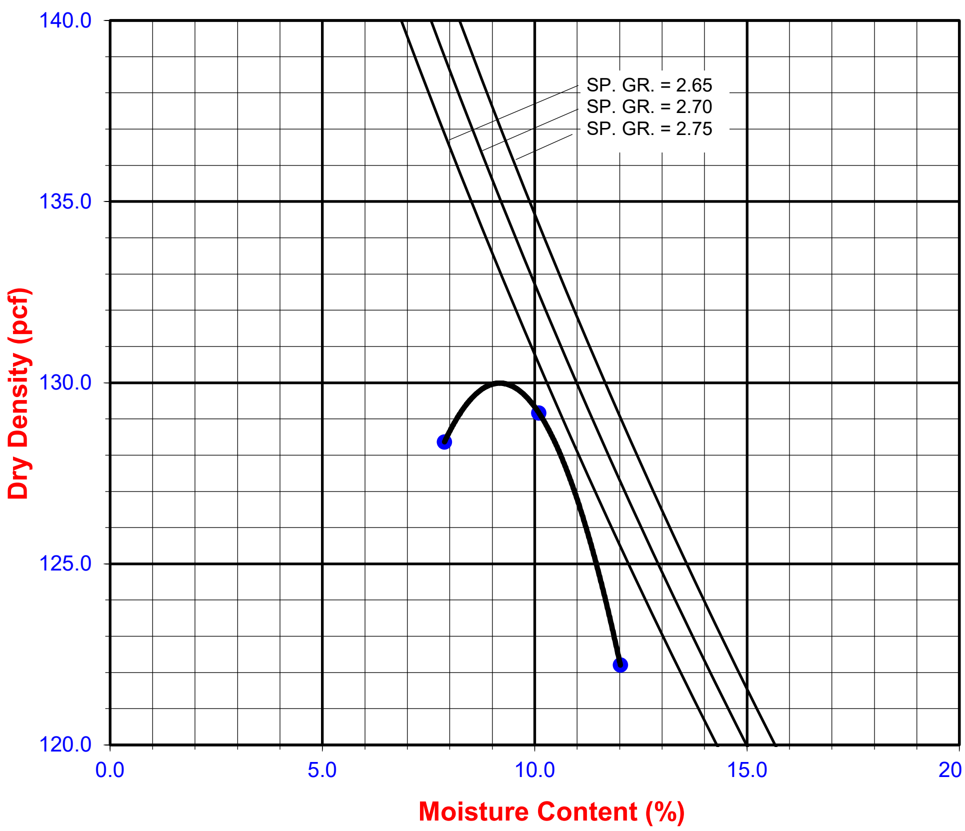
☐ **Procedure B**
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
Use if +#4 is >20% and +3/8 in. is 20% or less

X **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:
37:52:11
GR:SA:FI

Atterberg Limits:

LL,PL,PI



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SAND EQUIVALENT TEST

ASTM D 2419 / DOT CA Test 217

Project Name:	TKE MSWD Vista Reservoir	Tested By:	F. Mina	Date:	6/6/20
Project No. :	12761.001	Computed By:	F. Mina	Date:	6/6/20
Client:	TKE Engineering, Inc	Checked By:	M. Vinet	Date:	6/17/20

Boring No.	Sample No.	Depth (ft.)	Soil Description	T1	T2	T3	T4	R1	R2	SE	Average SE
LB-1	B-1	0 - 5.0	(SW-SM)g	09:00	09:10	09:12	09:32	7.7	3.1	41	**42**
				09:02	09:12	09:14	09:34	8.0	3.4	43	

T1 = Starting Time

T2 = (T1 + 10 min) Begin Agitation

T3 = Settlement Starting Time

T4 = (T3 + 20 min) Take Clay Reading (R1)

Sand Equivalent = R2 / R1 * 100

Record SE as Next Higher Integer

TESTS for SULFATE CONTENT
CHLORIDE CONTENT and pH of SOILS

Leighton

Project Name:	TKE MSWD Vista Reservoir	Tested By :	F. Mina	Date:	06/06/20
Project No. :	12761.001	Data Input By:	M. Vinet	Date:	06/17/20

Boring No.	LB-1	LB-2		
Sample No.	B-1	B-1		
Sample Depth (ft)	0 - 5.0	0 - 5.0		
Soil Identification:	(SW-SM)g	(SM)		
Wet Weight of Soil + Container (g)	100.00	100.00		
Dry Weight of Soil + Container (g)	100.00	100.00		
Weight of Container (g)	0.00	0.00		
Moisture Content (%)	0.00	0.00		
Weight of Soaked Soil (g)	100.00	100.00		

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	1	2		
Crucible No.	1	2		
Furnace Temperature (°C)	850	850		
Time In / Time Out	Timer	Timer		
Duration of Combustion (min)	45	45		
Wt. of Crucible + Residue (g)	25.0269	24.5236		
Wt. of Crucible (g)	25.0230	24.5180		
Wt. of Residue (g) (A)	0.0039	0.0056		
PPM of Sulfate (A) x 41150	160.49	230.44		
PPM of Sulfate, Dry Weight Basis	**160**	**230**		

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	30	--		
ml of AgNO3 Soln. Used in Titration (C)	1.0	--		
PPM of Chloride (C -0.2) * 100 * 30 / B	80	--		
PPM of Chloride, Dry Wt. Basis	**80**	**--**		

pH TEST, DOT California Test 643

pH Value	**8.50**	**--**		
Temperature °C	21.0	--		

SOIL RESISTIVITY TEST
DOT CA TEST 643

Leighton

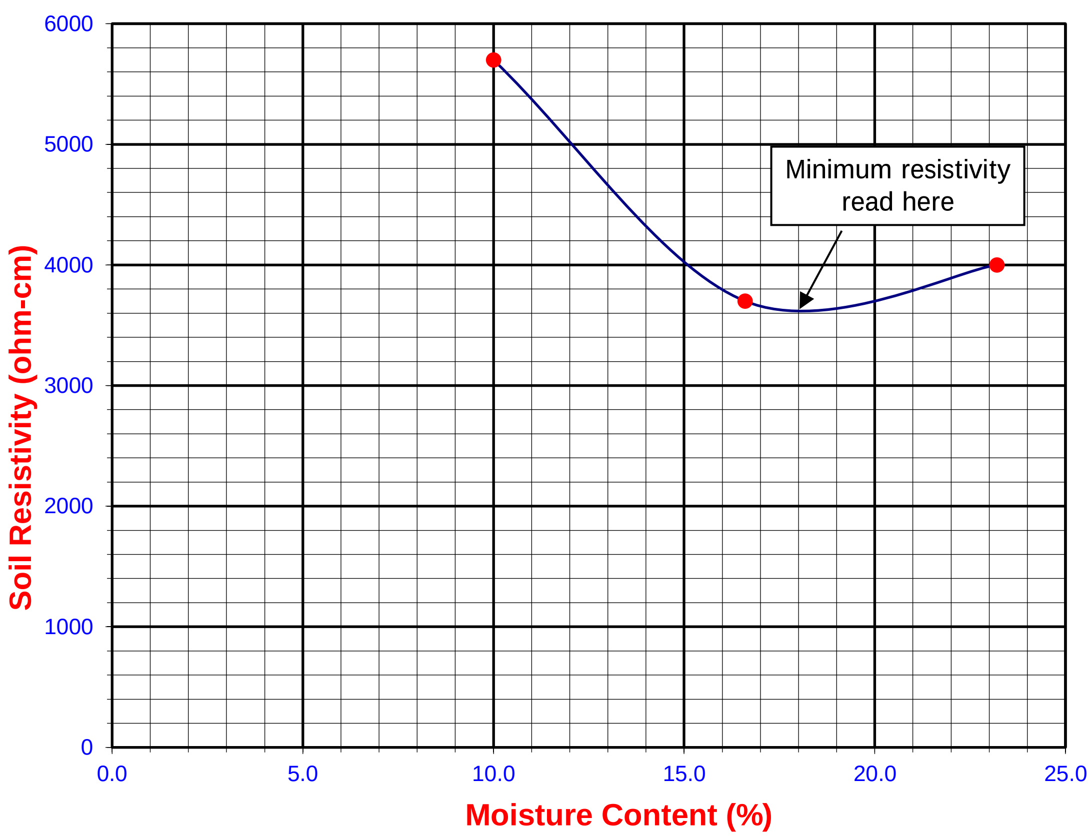
Project Name:	TKE MSWD Vista Reservoir	Tested By :	F. Mina	Date:	06/06/20
Project No. :	12761.001	Data Input By:	M. Vinet	Date:	06/17/20
Boring No.:	LB-1	Depth (ft.) :	0 - 5.0		
Sample No. :	B-1				
Soil Identification:*	(SW-SM)g				

*California Test 643 requires soil specimens to consist only of portions of samples passing through the No. 8 US Standard Sieve before resistivity testing. Therefore, this test method may not be representative for coarser materials.

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	50	10.00	5700	5700
2	83	16.60	3700	3700
3	116	23.20	4000	4000
4				
5				

Moisture Content (%) (MCi)	0.00
Wet Wt. of Soil + Cont. (g)	100.00
Dry Wt. of Soil + Cont. (g)	100.00
Wt. of Container (g)	0.00
Container No.	A
Initial Soil Wt. (g) (Wt)	500.00
Box Constant	1.000
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH: pH	Soil pH: Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
3600	**18.0**	**160**	**80**	**8.50**	**21.0**

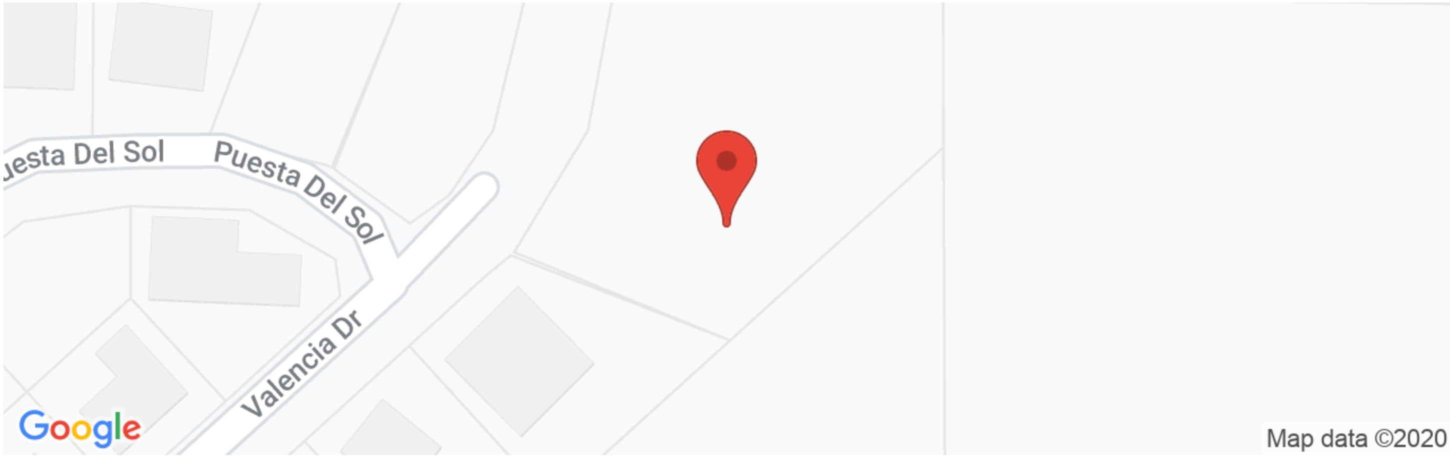


APPENDIX C

Seismic Parameters Output

OSHPD

Latitude, Longitude: 33.9828, -116.4932



Date	5/8/2020, 2:18:09 PM
Design Code Reference Document	ASCE7-16
Risk Category	IV
Site Class	C - Very Dense Soil and Soft Rock

Type	Value	Description
S_S	2.068	MCE_R ground motion. (for 0.2 second period)
S_1	0.776	MCE_R ground motion. (for 1.0s period)
S_{MS}	2.481	Site-modified spectral acceleration value
S_{M1}	1.086	Site-modified spectral acceleration value
S_{DS}	1.654	Numeric seismic design value at 0.2 second SA
S_{D1}	0.724	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	F	Seismic design category
F_a	1.2	Site amplification factor at 0.2 second
F_v	1.4	Site amplification factor at 1.0 second
PGA	0.858	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	1.03	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	2.214	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.441	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.068	Factored deterministic acceleration value. (0.2 second)
S1RT	0.856	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.963	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.776	Factored deterministic acceleration value. (1.0 second)
PGAd	0.858	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.907	Mapped value of the risk coefficient at short periods
C_{R1}	0.889	Mapped value of the risk coefficient at a period of 1 s

U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

Input

Edition

Dynamic: Conterminous U.S. 2014 (u…

Latitude

Decimal degrees

33.9828

Longitude

Decimal degrees, negative values for western longitudes

-116.4932

Site Class

537 m/s (Site class C)

Spectral Period

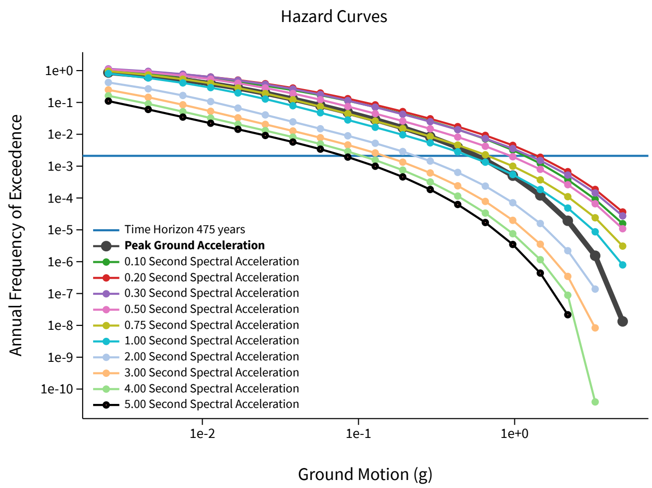
Peak Ground Acceleration

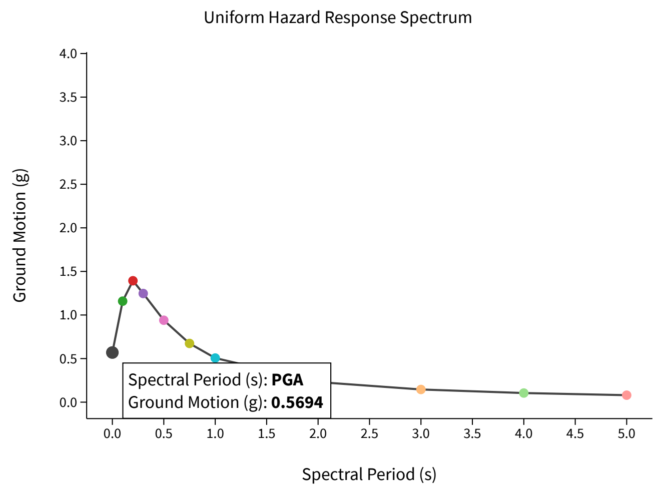
Time Horizon

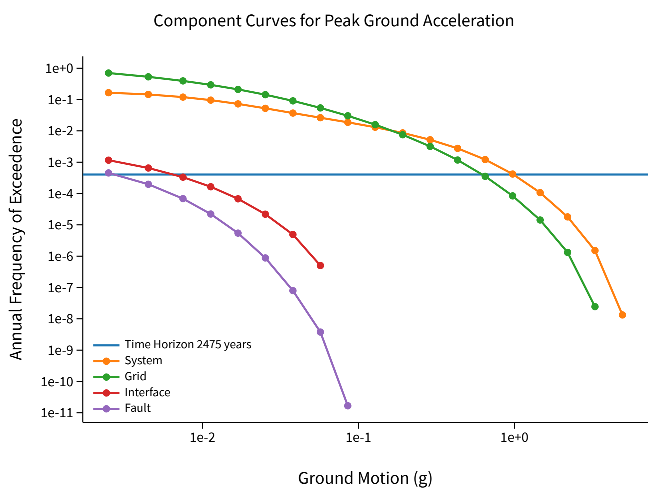
Return period in years

475

Hazard Curve







View Raw Data

U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

Input

Edition

Dynamic: Conterminous U.S. 2014 (u…

Latitude

Decimal degrees

33.9828

Longitude

Decimal degrees, negative values for western longitudes

-116.4932

Site Class

537 m/s (Site class C)

Spectral Period

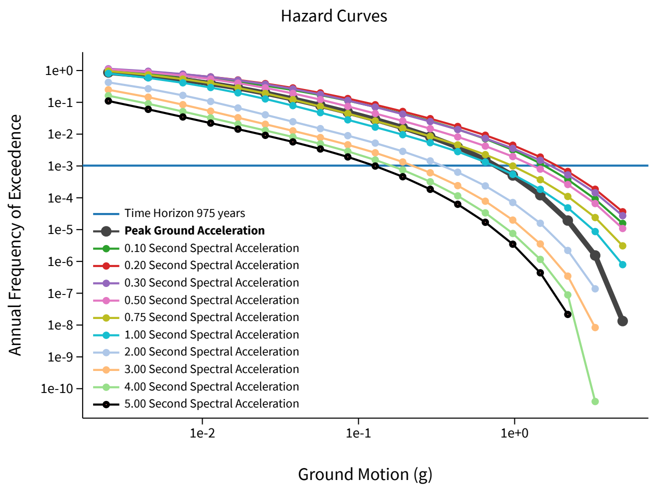
Peak Ground Acceleration

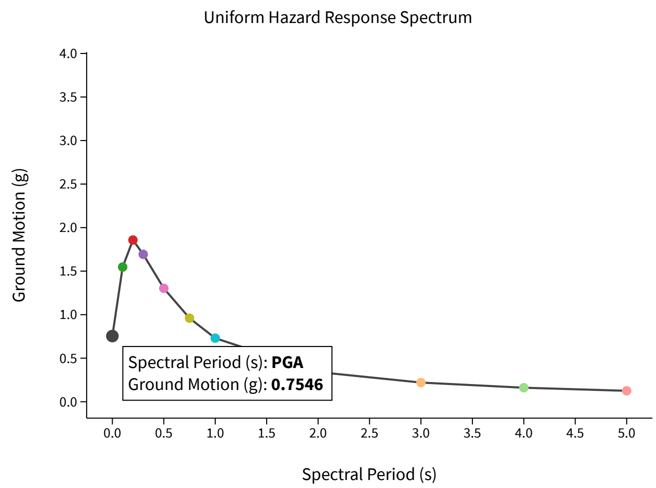
Time Horizon

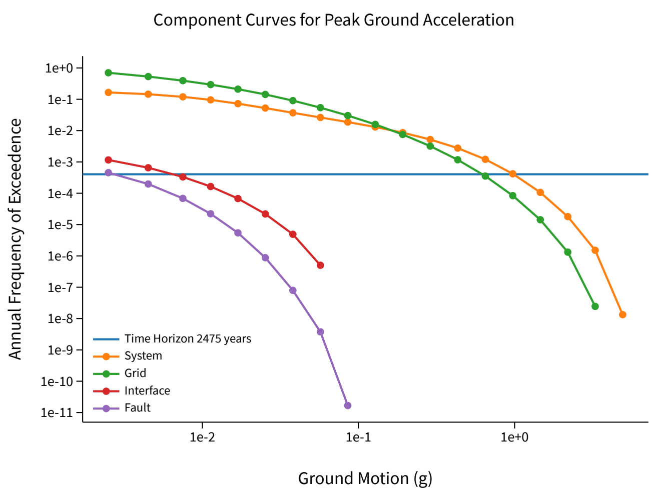
Return period in years

975

Hazard Curve







View Raw Data

U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

Input

Edition

Dynamic: Conterminous U.S. 2014 (u…

Spectral Period

Peak Ground Acceleration

Latitude

Decimal degrees

33.9828

Time Horizon

Return period in years

2475

Longitude

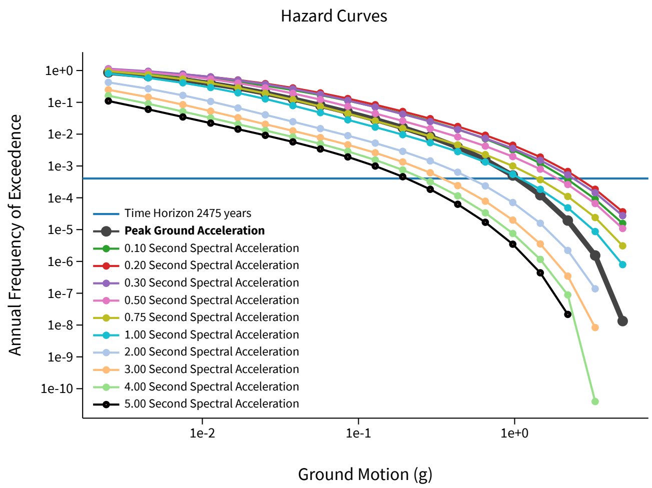
Decimal degrees, negative values for western longitudes

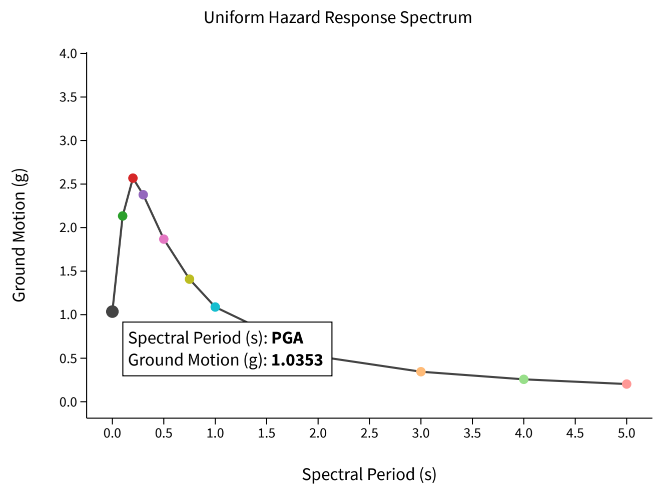
-116.4932

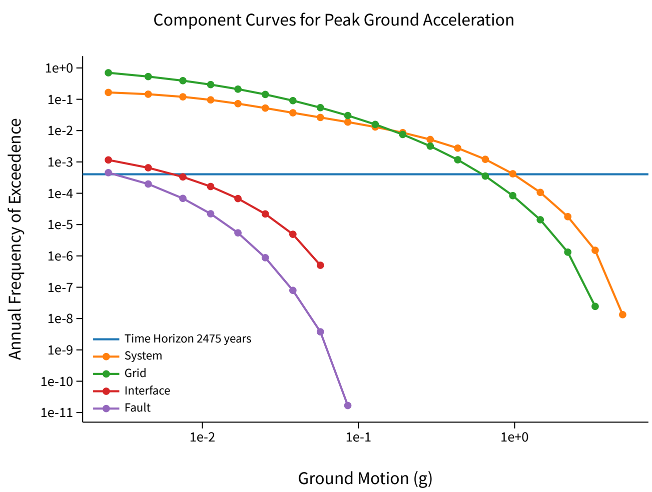
Site Class

537 m/s (Site class C)

Hazard Curve







View Raw Data

Deaggregation

Component

Please select “Edition”, “Location” “Site Class”, “Spectral Period” & “Time Horizon“ above to compute a deaggregation.

Compute Deaggregation

APPENDIX D

GBA Important Information About This Geotechnical Report

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way
exposure to problems associated with subsurface conditions at project sites and development of them that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation

everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed

and imes

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will not be adequate to develop geotechnical design recommendations for the project.

Do not rely on this report if your geotechnical engineer prepared it:
- for a different client;
- for a different project or purpose;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, the reliability of a geotechnical-engineering report can be affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do not rely on an executive summary. Do not read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:
- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept* *responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team through project completion to obtain informed guidance quickly, whenever needed.

This Report's Recommendations Are

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, they are <u>not</u> final, because the geotechnical engineer who developed them relied heavily on judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a continuing member of the design team, to:

- confer with other design-team members;
- help develop specifications;
- review pertinent elements of other design professionals' plans and specifications; and
- be available whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction-phase observations.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for information purposes only.* To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are <u>Not</u> Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

Obtain Professional Assistance to Deal with

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer's services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will <u>not</u> of itself be sufficient to prevent moisture infiltration. Confront the risk of moisture infiltration*** by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are <u>not</u> building-envelope or mold specialists.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org