SAN BERNARDINO COUNTY

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SECTION H

GEOTECHNICAL EXPLORATION REPORT

COUNTY SERVICE AREA (CSA) 70 F MORONGO VALLEY – TANK REPLACEMENT PROJECT

FOR

COUNTY SERVICE AREA (CSA) 70 F
SAN BERNARDINO, CALIFORNIA

PROJECT NO.: 30.30.0157

GEOTECHNICAL EXPLORATION
PROPOSED MORONGO VALLEY WATER TANK REPLACEMENT PROJECT, COUNTY SERVICE AREA (CSA) 70F
TERRACE DRIVE, NORTH OF EL DORADO DRIVE
ASSESOR PARCEL NUMBER (APN) 0580-241-16-0000
MORONGO VALLEY, SAN BERNARDINO COUNTY, CALIFORNIA
PROJECT SERVICE REQUEST #SD003, PROJECT NO. 30.30.00157

Prepared For

SAN BERNARDINO COUNTY
DEPARTMENT OF PUBLIC WORKS
SPECIAL DISTRICTS
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Project No. 038.0000020646

January 10, 2024

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Project No. 038.0000020646

San Bernardino County
Department of Public Works – Special Districts
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San Bernardino, California 92451-0450

Attention: Mr. Russel Viloria
Project Manager

Subject: Geotechnical Exploration
Morongo Valley Water Tank Replacement Project
County Service Area (CSA) 70F
Terrace Drive, North of El Dorado Drive
Assessor's Parcel Number (APN) 0580-241-16-0000
Morongo Valley, San Bernardino County, California
Project Service Request #SD003
Project No. 30.30.00157

In accordance with our December 6, 2023 proposal, Leighton Consulting, Inc. (Leighton) is pleased to present this *Geotechnical Exploration* report for use in designing the San Bernardino County, Department of Public Works – Special District's proposed Morongo Valley Water Tank Replacement (Project No. 30.30.00157) within Assessor' Parcel Number (APN) 0580-241-16-0000 located along Terrace Drive just northeast of El Dorado Drive, Morongo Valley, San Bernardino County, California.

The site is not located within a State or County designated Earthquake Fault Zone. As is the case for most of southern California, strong ground shaking is expected to occur at this site. Groundwater was not encountered during drilling, so damaging liquefaction at this site is unlikely. As regionally mapped by Dibblee (2008), the site is underlain by Quaternary-age alluvial silt, sand, and gravel of valley areas (Qa). Based on our exploration, we estimate that approximately 5 feet of artificial fill exists on the existing tank pad.

This report presents our findings and conclusions regarding this project. Based upon our geotechnical investigation, the proposed improvements are feasible from a geotechnical viewpoint, provided our recommendations are incorporated into the design and construction of the project. The most significant geotechnical issues at the site are strong seismic shaking and potentially compressible near-surface soils. These and other geotechnical issues are discussed in this report.

We appreciate the opportunity to be of service. If you have questions or if we can be of further service, please contact us at your convenience at ***866-LEIGHTON***, specifically at the phone extensions and/or e-mail addresses listed below.

Respectfully submitted,

LEIGHTON CONSULTING, INC.

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1.0 INTRODUCTION

1.1 Site Description

As regionally mapped on Figure 1, "*Site Location Map,*" this site is located along Terrace Drive, north of El Dorado Drive, APN 0580-241-16-0000, in the Morongo Valley area, San Bernardino County, California. This site is at the northeastern corner of Section 12, Township 1 South and Range 4 East. Topographically, this is at the base of the foothills of the San Bernardino Mountains, on an alluvial fan that slopes gently down to the southwest. The site contains an existing water tank. Based on our review of historical aerial photographs, the existing tank was constructed some time between 1996 and 2002. The surrounding area is largely undeveloped and contains native desert vegetation.

1.2 Proposed Potable Water Tank

Based on the provided Exhibit "C", *Preliminary 50% Draft Submittal Plans* prepared by Kimley Horn (reproduced on Figure 2), we understand the proposed bolted-steel tank will be constructed slightly northeast of the existing 260,000-gallon tank and consist of the following preliminary dimensions:

Table 1. Proposed Water Tank

Tank	Capacity (gallons)	Diameter (feet)	Tank Height (feet)	Water Height (feet)
Proposed Tank	155,000	55	16	8.73

Back-calculated hydrostatic pressure at the bottom of the proposed tank has been estimated to be on the order of 545 pounds-per-square-foot (psf) when filled. We understand the new tank pad will be at an elevation of 3,033.75 feet above mean sea level (msl). The depths of cuts and fills are unknown, but generally expected to be minor (on the order of 5 feet or less).

The project also calls for the installation of three temporary storage tanks, each with a proposed diameter of 12 feet and a capacity of 7,510 gallons, while the main tank is under construction to maintain a water supply for nearby residents.

1.3 Purpose and Scope of Exploration

The purpose of this exploration was to (1) explore site geotechnical conditions, (2) identify significant geotechnical issues at the site, and (3) provide preliminary geotechnical recommendations for design and construction of the tank foundation. Our services were provided in accordance with our December 6, 2023 proposal. The scope of our current exploration specific to the tanks project included the following:

- **Subsurface Exploration:** Prior to subsurface exploration, we marked our proposed boring locations for coordination with DigAlert (811) for utility clearance at our proposed boring locations, we also retained the services of a private utility locator in order to further identify any unmarked utilities not shown on provided plans. On December 15, 2023, three borings were drilled with a truck-mounted hollow-stem-auger rig at this site to depths of 20½ to 51½ feet, at locations shown on Figure 2, *Exploration Location Map*. A more detailed description of our subsurface exploration program and boring logs are presented in Appendix A, *Subsurface Exploration*.

- **Geotechnical Laboratory Testing:** Geotechnical laboratory tests were performed on selected recovered earth material samples obtained during our subsurface exploration, at our in-house geotechnical laboratory. This laboratory-testing program was designed to provide information about physical and engineering characteristics of sampled earth materials. Test procedures and results are presented in Appendix B, *Geotechnical Laboratory Testing*.

- **Geotechnical Analysis:** Data obtained from our background review, field reconnaissance and geotechnical laboratory testing was evaluated to develop geotechnical conclusions and recommendations presented in this report. Seismic and liquefaction analyses were performed with results included in Appendix C, *Seismic Analysis*.

- **Report Preparation:** Results of this geotechnical exploration have been summarized in this *Geotechnical Exploration* report, presenting our findings, conclusions and geotechnical recommendations regarding suitability of this proposed tank site. This report includes, Appendix D, *Earthwork and Grading Guide Specifications*, as current standard specification for grading and earthwork.

This report does **not** address the potential for hazardous materials in soil and/or groundwater. Important information about limitations of geotechnical reports in general is presented in Appendix E, *GBA's Important Information About This Geotechnical-Engineering Report*.

2.0 FINDINGS

2.1 Regional Geologic Setting

This site is located at the base of the eastern San Bernardino Mountains in San Bernardino County, California, and is part of the Transverse Ranges Geomorphic Province. Mesozoic granitic batholithic rock with Precambrian gneiss roof pendants makes up San Bernardino Mountain's basement. These basement rocks have been uplifted sometime after the Miocene as a result of transpression from the San Andreas transform system. Uplift of the San Bernardino Mountains has resulted in erosion of the basement rocks and transport and deposition of sediment into the lowlands, which includes the Morongo Valley where the proposed tank will be located. The margins of the San Bernardino Mountains are defined to the southwest by the faults relating to the San Andreas transform system (located approximately 7.0 miles to the southwest of the project) and to the southeast by faults relating to the Pinto Mountain fault zone (located approximately 0.4 mile southeast of the project). The location of the project relative to mapped Quaternary fault traces is depicted in Figure 3, *Regional Fault and Historic Seismicity Map*.

This project has been regionally mapped to be underlain by Quaternary alluvial deposits of valley areas and floodplains of canyons (Dibblee and Minch, 2008). These deposits have been described to consist of silt, sand, and gravel. The location of the project relative to published geologic mapping (Dibblee and Minch, 2008) is depicted in Figure 2, *Regional Geology Map*.

2.2 Site Geologic Units

Based on our review of pertinent geotechnical literature and our subsurface exploration, the site is underlain by Quaternary alluvial deposits (Qa). Encountered near-surface soils generally consisted of well-graded sands (SW), well-graded sand with silt (SW-SM), and silty sands (SM). Intensely weathered bedrock was encountered beneath alluvial deposits and was comprised of granitic, coarse-grained intrusive igneous rock.

The geologic units encountered within our explorations at this site are described below:

- **Undocumented Artificial Fill (Afu)**: Artificial fill was encountered in our borings to a depth of approximately 5 feet below the existing ground surface (bgs). Encountered fill was predominantly well-graded sand with minor amounts of silt and gravel; discernable as fill based only on observed structure and texture. We are unaware of documentation of engineered placement of this fill, including any geotechnical observation and testing documentation.

- **Quaternary-Age Alluvial Deposits (Qa)**: Figure 3 shows that this site is regionally mapped as being underlain by alluvial deposits (Qa); presumably derived from the transport of local weathered basement rock from the surrounding mountains. Well-graded sand with minor amounts of silt and gravel were encountered in all three of our borings below undocumented fill soils. Also encountered were layers of silty sands with varying amounts of gravels. These soils were generally found to be medium dense to very dense based on standard penetration testing.

- **Cretaceous-Age Igneous Rock (grd)**: Bedrock was encountered within one of our borings (LB-1) at a depth of approximately 40 feet bgs. The encountered material was consistent with the regionally mapped geology shown on Figure 3, which is predominately medium to coarse-grained granitic rock. Samples collected during our investigation were intensely weathered and slightly oxidized down to the maximum depth explored (51½ feet bgs).

2.3 Subsurface Soil Conditions

2.3.1. Compressible and Collapsible Soil

Soil compressibility refers to a soil's potential for settlement when subjected to increased loads as from fill surcharge. Collapse potential refers to the potential settlement of soil under existing stresses upon being wetted. Based on the results of our investigation along with the implementation of our overexcavation recommendations during grading, compressibility and collapse potential of site soils are considered low.

2.3.2. Expansive Soils

Expansive soils contain significant amounts of clay particles that swell considerably when wetted and shrink when dried. Foundations constructed on these soils are subjected to large uplifting forces caused by the swelling. Without proper measures taken, heaving and cracking of foundations and slabs-on-grade could result.

Based on our laboratory testing, and observations made in the field, onsite soils are anticipated to exhibit very low expansion potential.

2.3.3. Sulfate Content

Water-soluble sulfates in soil can react adversely with concrete. However, concrete in contact with soil containing sulfate concentrations of less than 0.1 percent by weight is considered to have negligible sulfate exposure based on

American Concrete Institute (ACI) provisions, adopted by the 2022 CBC (CBC, 2022, Chapter 19, and ACI 318, 2014).

A near-surface soil sample collected during subsurface exploration for this project was tested for soluble sulfate content. Based on the results of this testing, the sulfate exposure from onsite soils is expected to be negligible (Exposure Class S0).

2.3.4. Resistivity, Chloride, and pH

Soil corrosivity to ferrous metals can be estimated by the soil's electrical resistivity, chloride content, and pH. In general, soil having a minimum resistivity less than 1,000 ohm-cm is considered severely corrosive. Soil with a chloride content of 500 parts-per-million (ppm) or more is considered corrosive to buried ferrous metals.

As a screening for potentially corrosive soil, a representative soil sample was tested for minimum resistivity, chloride content, and pH. Our laboratory test result for a representative near surface sample resulted in a minimum resistivity of 17,500 ohm-cm, a chloride content of 40 ppm and a pH of 7.7. Based on our laboratory testing, onsite soils are considered "mildly" corrosive to buried ferrous metals.

2.4 Groundwater

Groundwater was not encountered in our three borings drilled on December 15, 2023, to a maximum depth of 51½ feet. Encountered alluvium was dense with low moisture contents ranging from approximately 1 to 3 percent. Based on review of well data from the California Department of Water Resources' SGMA Data Viewer, State Well Nos. 01S05E05A001S and 01S04E13B006S dating back to 1969, historic high groundwater levels in the site vicinity are estimated to be deeper than 100 feet.

Local groundwater conditions can fluctuate due to adjacent leaking water tanks, heavy irrigation, precipitation, or other factors not initially observed at the time of exploration. Groundwater levels can be expected to fluctuate seasonally. Fluctuations of groundwater level, localized zones of perched groundwater and an increase in soil moisture should be anticipated during and following the rainy seasons, periods of locally intense rainfall and/or storm water runoff.

2.5 Faulting

Active surface fault classification criteria adopted by the California Geological Survey (CGS), formerly the California Division of Mines and Geology (CDMG), defines an active fault as one that has ruptured during Holocene time (the last 11,000 years). A fault that has ruptured during the last 1.8 million years (Quaternary time) but has not been proven by direct evidence to have not moved within Holocene time, is considered to be potentially active. A fault that has not moved during both Pleistocene and Holocene time (that is no movement within the last 1.8 million years) is considered to be inactive.

The site has been mapped to be outside of any State or County designated Earthquake Fault Zones. Published geologic mapping of the region (Dibblee and Minch, 2008) has indicated no faults transecting through or trending towards the site. Based on our understanding of the current geologic framework, the potential for future surface rupture of active faults onsite is considered low.

2.6 Secondary Seismic Hazards

2.6.1. Liquefaction Potential: Liquefaction is the loss of soil strength due to a buildup of pore-water pressure during severe ground shaking. Liquefaction is associated primarily with loose (low density), saturated, fine- to medium-grained, clean cohesionless soil. As the shaking action of an earthquake progresses, soil granules are rearranged, and the soil densifies within a short period of time. This rapid densification of soil results in a buildup of pore-water pressure. When the pore-water pressure approaches the total overburden pressure, the soil reduces greatly in shear strength and temporarily behaves similarly to a fluid. For liquefaction to occur there must be:

- loose granular soil,
- shallow groundwater, and
- strong ground shaking

occurring or existing simultaneously. If one component is missing, then liquefaction will not occur. Effects of liquefaction can include sand boils, settlement and bearing capacity failures below structural foundations.

Groundwater was not encountered in any of our borings drilled onsite on December 15, 2023. The onsite alluvium is dense and shown to be resistant to liquefaction. Soils encountered during drilling were medium dense to very dense. Historically highest groundwater levels have been estimated to be on the order of 100 feet below the current ground surface. Due to the relatively dense nature of the underlying alluvial soils and lack of shallow groundwater, the potential for liquefaction is considered very low at this site.

2.6.2. **Lateral Spreading**: Lateral spreading is highly unlikely to occur at this site due to the very low liquefaction potential the site.

2.6.3. **Seismically Induced Settlement:** During a strong seismic event, seismically induced settlement can occur within loose to moderately dense, unsaturated granular soil. Settlement caused by ground shaking is often non-uniformly distributed, which can result in differential settlement. We have performed analyses to estimate the potential for seismically induced settlement, using the method of Tokimatsu and Seed, and based on Martin and Lew (1999). Based on our analysis, and after our recommendations for the tank overexcavation and recompaction is accomplished, less than 1 inch of cumulative seismic settlement is estimated. Potential differential settlement is estimated as half of the total seismic settlement over a horizontal distance of 30 feet, which is a maximum of less than 0.5 inch in 30 feet, or angular distortion of 0.0014L. The structural engineer should determine Structure Type and Risk Category and evaluate whether the differential settlement estimates described above are tolerable.

3.0 CONCLUSIONS AND RECOMMENDATIONS

3.1 Conclusions

3.1.1. Seismic Considerations: As is the case for most of southern California, strong ground shaking is expected to occur at this site. Groundwater was not encountered during drilling, so damaging liquefaction at this site is unlikely. Dynamic differential settlement of thin pockets of underlying loose undocumented artificial fill soils as observed within our borings onsite may occur between opposite end tank edges from a local large-magnitude earthquake. Overexcavation and recompaction of underlying near surface soils can aid in reducing this differential settlement.

3.1.2. Foundation Considerations: We understand that the footprint of the new tank will be approximately 2,375 square feet on this previously graded pad, with the new tank pad at an approximate elevation of 3,034 feet. Due to the presence of undocumented fill soils within the upper approximately 5 feet, we recommend the entire tank footprint, extending 3 feet out radially (horizontally) from the tank foundation exterior perimeter, be overexcavated 3 feet below bottom of the ring wall foundation, or at least 5 feet below existing grade, whichever is deeper. After overexcavation is completed, the exposed subgrade should be scarified, moisture-conditioned and compacted.

This overexcavation should then be backfilled with onsite soils, compacted to at least 95% of the ASTM D1557 (modified Proctor) laboratory maximum density. A conventionally reinforced concrete ring wall to support this new tank can then bear on this properly recompacted zone.

Earthwork outside of the permanent tank footprint can be summarized as follows:

- **Ancillary Permanent Structures**: Ancillary permanent structures may be supported on shallow foundations after overexcavation of underlying existing fill (to the depths locally encountered) and compaction as described above.
- **Temporary Tanks (<18 months)**: Temporary tank overexcavation may be limited to 18 inches, but the exposed subgrade should be confirmed to be firm and unyielding.

Geotechnical recommendations are presented in the following sections for the earthwork and foundations associated with the proposed replacement tank and associated development.

3.2 Earthwork

All earthwork should be performed in accordance with the *Earthwork and Grading Guide Specifications* presented in Appendix E, unless specifically revised or amended below or by future review of project plans.

3.2.1. Tank and Ancillary Structure Footprint Preparation: Any remaining underground obstructions under proposed structure footprints should be removed. Existing buried conduits and other substructures within new structure footprint areas should be removed or rerouted.

Before backfilling, all foundation remnants, piping and loose soils encountered at the bottom of the excavation during demolition should be excavated to expose dense undisturbed alluvium. To reduce differential settlement potential for the tank on this pad, the entire tank footprint, extending 3 feet out radially (horizontally) from the tank foundation exterior perimeter should be overexcavated 3 feet below bottom of ring wall foundation, or at least 5 feet below existing grade, whichever is deeper.

If very low expansive onsite soils with Expansion Indices of 20 or less can be used to backfill the proposed demolished reservoir area in thin lifts, compacted to at least 95% of the ASTM D1557 (modified Proctor) laboratory maximum density, at or slightly above optimum moisture, then this proposed steel water tank may be supported on a ring-wall foundation system bearing uniformly on compacted fill.

Earthwork outside of the permanent tank footprint can be summarized as follows:

- **Ancillary Permanent Structures**: Ancillary permanent structures may be supported on shallow foundations after overexcavation of all underlying existing fill (to the depths locally encountered) and compaction as described above.
- **Temporary Tanks (<18 months)**: Temporary tank overexcavation can be limited to 18 inches, but the exposed subgrade must be firm and unyielding.
- **Pavements**: In areas outside the structure limits planned for new asphalt or concrete pavement, and/or new flatwork, the upper 18 inches of exposed subgrade soil should be overexcavated, moisture conditioned, and compacted. This is not a requirement or recommendation to remove existing pavements, although a 5-foot-wide radial area around the permanent tank should be paved with at least a 2% slope away from the tank, to reduce stormwater infiltration into underlying soils; with pavement sealed abutting against the ring-wall footing.

Resulting removal excavation bottom surfaces should be observed by Leighton Consulting, Inc., prior to placement of any backfill or new construction.

3.2.2. Backfill Placement and Compaction: The onsite soil, free of organic material, cobbles, boulders, rubble, and rock less than 6 inches in largest dimension, is suitable to be used as structural fill. All fill soil should be placed in loose lifts no greater than 8 inches in thickness, moisture-conditioned as necessary to at least optimum moisture content and compacted using proper equipment and then mechanically compacted as follows:

- **Tank Pads**: All fill under proposed new potable-water tanks should be compacted to a minimum of 95% relative compaction as determined by ASTM D1557 modified Proctor test method within the backfill zone under the new tank.
- **Outside Tank Pad**: Except for pavement aggregate base, fill placed outside of the tank footprint can be compacted to 90% relative compaction as determined by ASTM D1557 modified Proctor test method.

Material imported to the site for use as fill should be reviewed and approved by the geotechnical engineer prior to import to the site and placement as fill. Imported soils should be very low expansive (EI ≤ 20); non-corrosive to metals and concrete; and be free of hazardous substances.

3.2.3. Utility Trench Backfill: Utility trenches should be backfilled in accordance with Section 306-12.2 (for narrow trenches) or Section 306-12.3 (for mechanically compacted backfill) of the *Standard Specifications for Public Works Construction* ("Greenbook"), 2021 Edition. Utility trenches can be backfilled with on-site soils free of debris, organic and oversized material up to 3 inches in largest dimension. Prior to backfilling trenches, pipes should be bedded in and covered with either:

(1) **Sand:** A uniform, granular material that has a Sand Equivalent (SE) of 30 or greater and a maximum particle size of ¾ inch (or as specified by the pipe manufacturer), water densified in place, or

(2) **CLSM:** Controlled Low Strength Material (CLSM) conforming to Section 201-6 of the *Standard Specifications for Public Works Construction*, ("Greenbook"), 2021 Edition.

Open-graded gravel or crushed rock should not be used for trench backfill due to potential for subterranean piping erosion (filling open voids with fines washed in by stormwater infiltration). If gravel or open-graded rock is approved and used as bedding or shading, then such open-graded material should be wrapped in Mirafi® 140N non-woven filter fabric, or equivalent, to prevent surrounding fine soil from washing into pore spaces within open-graded gravel or rock.

Pipe bedding should extend at least 4 inches below any pipeline invert and at least 12 inches over the top of the pipeline. Onsite soil is predominantly unsuitable for the pipe bedding zone.

Native soils (free of large cobbles and boulders) can be used as backfill over the pipe-bedding zone, and should be placed in thin lifts, moisture conditioned above optimum, and mechanically compacted to at least 90 percent relative compaction, relative to the ASTM D1557 modified Proctor laboratory maximum density outside of the proposed tanks footprint, or 95 percent within the tanks footprint. Backfill above the pipe zone should not be jetted. Backfill above the pipe zone (bedding) should be observed and tested by Leighton Consulting, Inc.

3.2.6. **Surface Drainage:** Surface drainage should be designed to direct water away from slopes and foundations, toward approved drainage devices. Irrigation of landscaping (if any) should be controlled to maintain, as much as possible, a consistent moisture content sufficient to provide healthy plant growth without over-watering. Water should not be allowed to pond on site or flow uncontrolled over slopes.

3.2.7. **Oversize Rocks**: No oversized material was encountered during our exploration. However, if any oversized material (>6 inches in largest dimension) is encountered during excavation should be removed prior to placement as fill. Oversize rocks produced during excavation may be disposed off-site, stockpiled at remaining flat and stable undeveloped ground surface away from the tank or reduced in size by pulverizing or other approved means and methods.

3.3 Seismic Design Parameters

To accommodate ground shaking produced by regional seismic events, seismic design can, at the discretion of the designing Structural Engineer, be performed in accordance with the 2022 Edition of the *California Building Code* (CBC). Table 2, *2022 CBC Site-Specific Seismic Parameters*, lists (below) site-specific seismic design parameters based on the 2022 CBC method, which is based on the *2021 International Building Code* and ASCE/SEI 7-16:

Table 2. 2022 CBC Site-Specific Seismic Parameters

2022 CBC Seismic Design Parameters	Value
Site Longitude (decimal degrees)	-116.5206
Site Latitude (decimal degrees)	34.1005
Site Class Definition (ASCE 7 Table 20.3-1)	C
Mapped Spectral Response Acceleration at 0.2s Period, S_s (Figure 1613.3.1(1))	2.042g
Mapped Spectral Response Acceleration at 1s Period, S_1 (Figure 1613.3.1(2))	0.752g
Short Period Site Coefficient at 0.2s Period, F_a (Table 1613.3.3(1))	1.200
Long Period Site Coefficient at 1s Period, F_v (Table 1613.3.3(2))	1.400
Adjusted Spectral Response Acceleration at 0.2s Period, S_{MS} (Eq. 16-37)	2.451g
Adjusted Spectral Response Acceleration at 1s Period, S_{M1} (Eq. 16-38)	1.052g
Design Spectral Response Acceleration at 0.2s Period, S_{DS} (Eq. 16-39)	1.634g
Design Spectral Response Acceleration at 1s Period, S_{D1} (Eq. 16-40)	0.702 g
Peak Ground Acceleration, PGA_M (ASCE 7-10, Eq. 11.8-1)	1.04g
Long-Period (T_L, seconds)	8

3.4 Shallow Footing Foundations

Conventional shallow footing foundations may be used to support the tank (ring wall) and ancillary structures, as follows:

3.4.1. Minimum Embedment and Width: Conventional ring wall and interior column spread footings may be used to support the proposed water tank, bearing solely on newly placed properly compacted fill. Footings should have a minimum width of 12 inches and should extend at least 18 inches below lowest adjacent finished grade. Lowest adjacent finished grade may be taken as either (1) the lowest adjacent interior slab-on-grade surface, or (2) finished exterior grade, whichever is lower.

3.4.2. Allowable Bearing Pressure: An allowable bearing capacity of 3,000 pounds-per-square-foot (psf) may be used for the proposed tank, based on the above minimum embedment depth and width, for footings bearing solely on properly compacted fill. This allowable bearing value may be increased by 500 psf per foot increase in depth and width to a maximum allowable bearing pressure of 5,000 psf. These allowable bearing pressures are for total dead load and frequently applied live loads; so, can be increased by one-third for short duration wind and seismic loads. Footing reinforcement should be designed by the structural engineer.

3.4.3. Lateral Load Resistance: Lateral loads due to seismic shaking can be resisted by sliding friction at the base of foundations and passive pressure against the side of foundations. Although both may be used in combination, passive resistance requires some lateral deflection to be fully developed. Frictional resistance for concrete foundations cast directly on the subgrade soil may be computed using a coefficient of friction of 0.35. Passive resistance in newly placed properly compacted fill may be computed using an equivalent fluid pressure of 250 pounds-per-square-foot per foot embedment (pcf), assuming there is constant contact between the footing and undisturbed soil. In no case, should passive resistance exceed 3,000 psf. These friction and passive values have already been reduced by a factor-of-safety of 1.5. These design passive pressure and coefficient of friction values may be increased by one-third when considering short duration seismic loads.

3.4.4. **Settlement Estimates**: Our recommended allowable bearing pressure is generally based on a total allowable, post-construction static settlement of one-inch. Differential settlement due to static loading is estimated at ½ inch over a horizontal distance of 30 feet, based on our overexcavation and recompaction recommendations presented in Section 3.2. Since settlement is a function of footing size and contact bearing pressure, larger differential settlements can be expected between adjacent foundation components where a large differential loading condition exists. These settlement estimates should be reevaluated by Leighton Consulting, Inc. for unusual loading condition exceeding what was stated in the project description for this report, when foundation plans and loads for the proposed structures become available.

3.5 Retaining Wall Design

Retaining walls are not currently anticipated but could be introduced as the grading plan is refined. Specific preliminary design recommendations for earth retaining structures are presented in the following subsections.

3.5.1. **Lateral Earth Pressures**: We recommend that retaining walls be backfilled with non-expansive (EI≤30) soil, and constructed with a backdrain as described in Section 3.5.5, below. Based on these recommendations, the following parameters may be used for retaining wall design:

Table 3. Retaining Wall Design Earth Pressures

Static Equivalent Fluid Weight (pcf)	
Condition	Level Backfill
Active	40 pcf
At-Rest	60 pcf
Passive	240 pcf (allowable) (Maximum of 3,500 psf)

The above values do not contain an appreciable factor of safety unless noted, so the structural engineer should apply the applicable factors of safety and/or load factors during design, as specified by the California Building Code.

Cantilever walls that are designed to yield at least 0.001H, where H is equal to the wall height, may be designed using the active condition. Rigid walls and walls braced at the top (e.g. valve vaults and manholes) should be designed using the at-rest condition. Passive pressure is used to compute soil resistance to lateral structural movement.

3.5.2. <u>Retaining Wall Surcharges</u>: In addition to the above lateral forces due to retained earth, surcharge due to above grade loads on the wall backfill, such as an adjacent structure or traffic, should be considered in design of the retaining wall. Vertical surcharge loads behind the retaining wall on or in the backfill within a 1:1 plane projection up and out from the retaining wall toe, should be considered as lateral and vertical surcharge. Unrestrained (cantilever) retaining walls should be designed to resist one-third of these surcharge loads applied as a uniform horizontal pressure on the wall. Braced walls should also be designed to resist an additional uniform horizontal-pressure equivalent to one-half of uniform vertical surcharge loads.

3.5.3. <u>Incremental Seismic Loads on Retaining Walls</u>: For retaining walls less than 6 feet in height, incremental seismic loads need not be considered. However, for walls with a retained height over 6 feet, or where otherwise required by Code or deemed appropriate by the structural engineer, we recommend that the wall designs be checked seismically using an additive seismic Equivalent Fluid Pressure (EFP) of 23 pcf, which is added to the active EFP. Such walls that are to be designed in the static case assuming the at-rest condition should be checked seismically using this additive seismic EFP added to the active condition (i.e., the additive seismic EFP is not added to the at-rest EFP value shown in Table 3 above). The additive seismic EFP should be applied with a standard EFP pressure distribution (i.e., it is not an inverted triangle).

3.5.4. <u>Sliding and Overturning</u>: Total depth of retained earth for design of walls should be measured as the vertical height of the stem below the ground surface at the wall face for stem design, or measured at the heel of the footing for overturning and sliding. A soil unit weight of 120 pcf may be assumed for calculating the actual weight of the soil over the wall footing, assuming drained conditions, for properly compacted backfill.

3.5.5. <u>Retaining Wall Drainage</u>: Adequate drainage may be provided by a subdrain system positioned behind the walls. This system should consist of a 4-inch minimum diameter perforated pipe placed near the base of the wall (perforations placed downward). The pipe should be bedded and backfilled with pervious backfill material described in Section 300-3.5.2 of the *Standard Specifications for Public Works Construction* (Greenbook), 2021 Edition. This pervious backfill should extend at least 2 feet out from the wall and to within 2 feet of the outside finished grade. This pervious backfill and pipe should be wrapped in filter fabric, such as Mirafi 140N or equivalent, placed as described in Section 300-8 of the *Standard Specifications for Public Works Construction* (Greenbook), 2018 Edition. The subdrain outlet should be connected to a free-draining outlet or sump.

Miradrain or Enkadrain drainage geocomposites, or similar, may be used for wall drainage as an alternative to pervious backfill or drain rock backfill, particularly where horizontal space is limited adjacent to shoring or near vertical cuts. Drainage geocomposites should be connected to a perforated drainpipe at the base of the wall.

3.6 <u>Preliminary Pavement Recommendations</u>

Based on design procedures outlined in the 2017 Caltrans Highway Design Manual and using a design R-value of 50, which is based on the laboratory test of sampled site soils, preliminary flexible pavement sections may consist of the following for the Traffic Indices (TI) indicated:

Table 4 Preliminary Pavement Sections

Traffic Index (TI)	Asphalt Concrete (AC) Thickness (inches)	Class 2 Aggregate Base Thickness (inches)
5.0 or less	3	4
6.0	3½	4
7.0	4	4½

Final pavement design should be based on a Traffic Index (TI) determined by the owner or Civil Engineer (based on traffic counts and/or traffic projections). However, for very light traffic (driveways behind gates without significant truck traffic), a TI=4.0 can be used.

Pavement construction should be performed in accordance with the current (2021 Edition) *Standard Specifications for Public Works Construction* or *Caltrans Standard Specifications*. Geotechnical field observations and periodic testing, as needed during placement of base course, should be undertaken to check that the requirements of the standard specifications are fulfilled.

Prior to placement of aggregated base, the subgrade soil should be processed to a minimum depth of 6 inches, moisture-conditioned, as necessary, and recompacted to at least **90%** relative compaction, relative to the ASTM D1557 modified Proctor laboratory maximum density. Aggregate base should be moisture conditioned, as necessary, and compacted to a minimum of **95%** relative compaction.

4.0 CONSTRUCTION CONSIDERATIONS

4.1 Utility Potholing/Locating

Before excavation begins, utility locating should be performed to seek-out and identify existing utilities in areas of proposed foundation construction and over excavation. If utilities are identified, then potholing may be required to further identify the existing utilities.

4.2 Geotechnical Construction Observation

Leighton Consulting, Inc. should observe and test grading and earthwork, to check that the site is properly prepared, sufficient overexcavation is performed, selected fill soil is satisfactory, and that placement and compaction of fill has been performed in accordance with our recommendations and the project specifications. Sufficient notification to us prior to earthwork is essential. Project plans and specifications should incorporate the recommendations contained in the text of this report.

Variations in site conditions are possible and may be encountered during construction. To confirm correlation between subsurface data obtained during our subsurface explorations and actual subsurface conditions encountered during construction, and to observe conformance with the plans and specifications, it is essential that we be retained to perform continuous or intermittent review during earthwork, excavation and foundation construction phases.

4.3 Rippability and Oversize Materials

We were able to drill borings to depths ranging from 20 to 51½ feet. Existing fill and alluvium is expected to be conventionally excavated using conventional excavation equipment in good condition.

4.4 Temporary Excavations

The contractor is responsible for all temporary excavations and trenches excavated at the site and is responsible for design of 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).

All temporary excavations should be performed in accordance with project plans, specifications, and all OSHA requirements. Excavations 5 feet or deeper should be laid back or shored in accordance with OSHA requirements before personnel are allowed to enter. Fill soils (of any kind) are OSHA soil Type C. Therefore, un-shored temporary cut slopes should be cut 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).

Surcharge loads should not be permitted within a horizontal distance equal to the height of excavation or 5 feet, whichever is greater, measured from the top of the excavation. An equivalent fluid pressure of 25 pcf may be used for level backfill. 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 Leighton Consulting, Inc. field representative should be maintained to facilitate construction while providing safe excavations.

4.5 Additional Geotechnical Services

Geotechnical recommendations presented in this report are based on anticipated subsurface conditions, as interpreted from limited subsurface explorations and limited laboratory testing. Leighton Consulting, Inc. should review the grading and foundation plans, and specifications, when available, to comment on geotechnical aspects. Final design versions should be reviewed. Our conclusions and recommendations presented in this report should be reviewed and verified by Leighton Consulting, Inc. during construction, and revised accordingly, if exposed geotechnical conditions vary from our preliminary findings and interpretations. Recommendations presented in this report are only valid if Leighton Consulting, Inc. verifies site conditions during construction. Geotechnical observation and testing should be provided by Leighton Consulting, Inc. during earthwork, and/or when unusual geotechnical conditions are encountered.

5.0 LIMITATIONS

This report was based on three soil borings excavated on December 15, 2023. Such information is, by necessity, incomplete. The nature of many sites is such that differing soil or geologic conditions can be present within small distances and under varying climatic conditions. Changes in subsurface conditions can and do occur over time. Therefore, findings, conclusions and recommendations presented in this report are based on the assumption that Leighton Consulting, Inc. will provide geotechnical observation and testing during construction.

This report was prepared for the sole use of San Bernardino County Department of Public Works (SBCDPW) – Special Districts and their design team, to design these replacement tanks in accordance with generally accepted geotechnical engineering practices at this time in San Bernardino County, California. This report is not authorized for use by, and is not to be relied upon by, any party except SBCDPW – Special Districts (and their design and construction management team), with whom Leighton Consulting, Inc. has contracted for this 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 that may arise as a result of such use or reliance, regardless of any fault, negligence, and/or strict liability of Leighton Consulting, Inc.

Environmental services were not included as part of this study. See Appendix E, *GBA's Important Information About This Geotechnical-Engineering Report*, for information about limitations common to all geotechnical reports.

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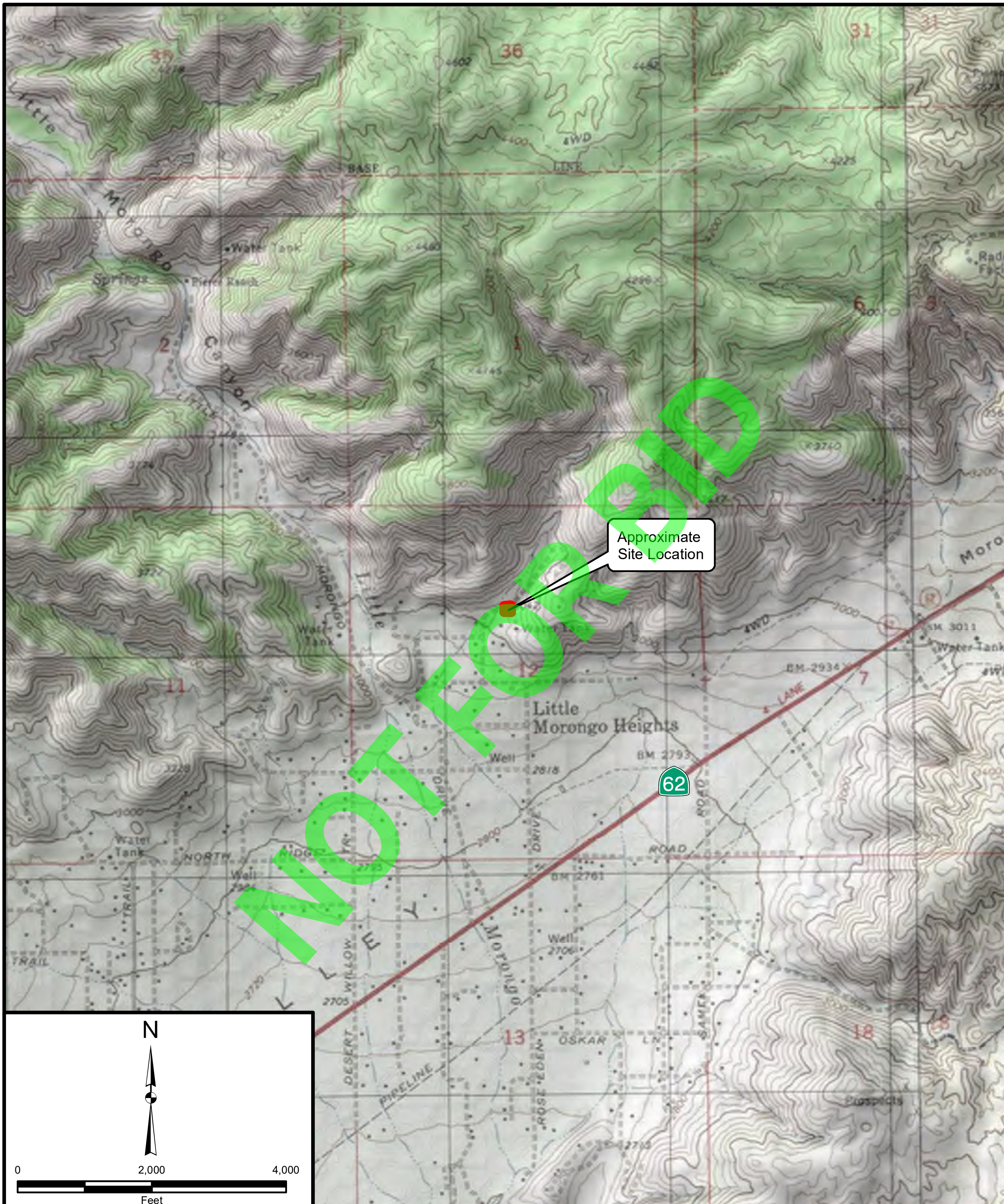


Project: 20072	Eng/Geol: SGO/JDH
Scale: 1 " = 2,000 '	Date: January 2024

SITE LOCATION MAP

Proposed 155,000 Gallon Tank
San Bernardino County Dept of Public Works
Special Districts, County Service Area (CSA) 70F
Terrace Drive, APN: 0580-241-16-0000
Morongo Valley, San Bernardino County, California

FIGURE 1

Leighton

Map Saved as J:\Drafting\20646\Maps\20646_F01_SLM_2024-01-02.mxd on 1/2/2024 1:12:45 PM Author: KVM (kmanchikanti)

Project: 20646	Eng/Geol: JDH/SGO
Scale: 1 " = 15 '	Date: January 2024

Reference: © 2023 Microsoft Corporation © 2023 Maxar ©CNES (2023) Distribution Airbus DS © 2023 TomTom

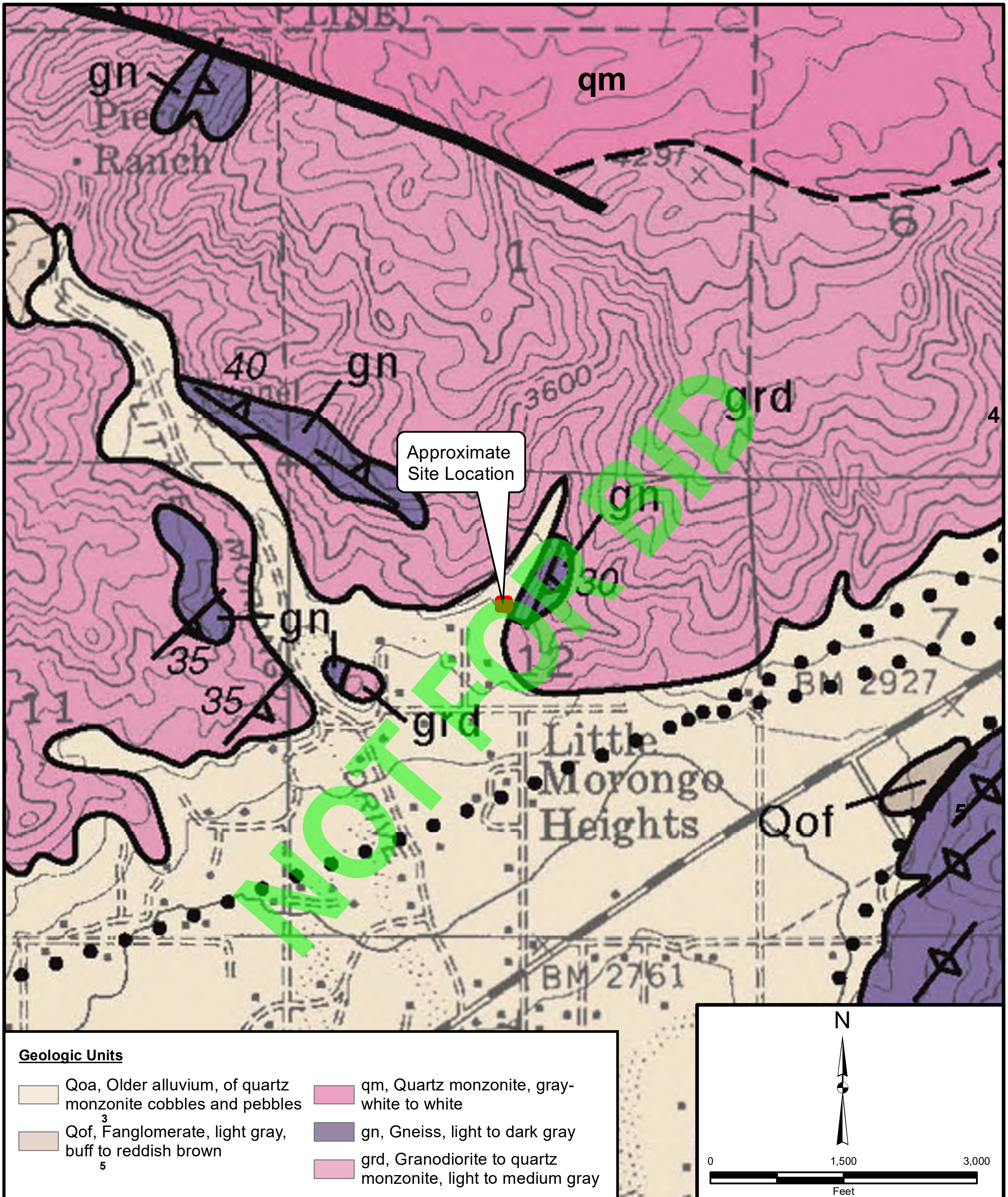
EXPLORATION LOCATION MAP

Proposed 155,000 Gallon Tank
San Bernardino County Dept of Public Works, Special Districts, County Service Area (CSA) 70F
Terrace Drive, APN: 0580-241-16-0000, Morongo Valley, San Bernardino County, California

FIGURE 2

Leighton

Map Saved as J:\Drafting\20646\Maps\20646_F02_ELM_2023-12-28.mxd on 1/5/2024 2:26:39 PM Author: KVM (kmanchikanti)



Project: 20646	Eng/Geol: JDH/SGO
Scale: 1 " = 1,500 '	Date: January 2024

Reference: Geologic map of the San Gorgonio Mountain & Morongo Valley 15' Quadrangles, San Bernardino and Riverside Counties by Thomas W. Dibblee Jr., 2008

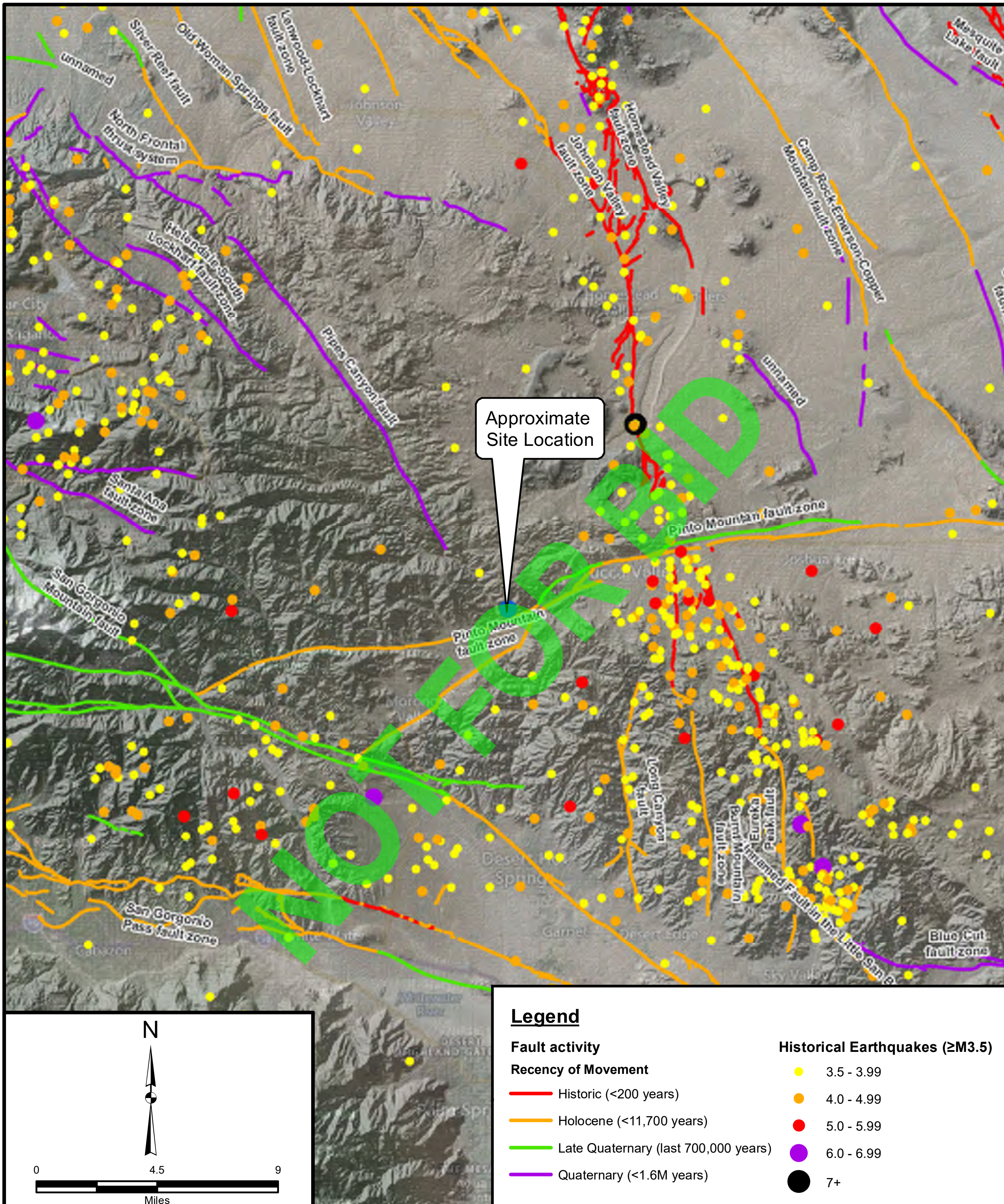
REGIONAL GEOLOGY MAP

Proposed 155,000 Gallon Tank,
San Bernardino County Dept of Public Works
Special Districts, County Service Area (CSA) 70F
APN: 0580-241-16-0000, Terrace Drive
Morongo Valley, San Bernardino County, California

FIGURE 2

Leighton

Map Saved as J:\Drafting\20646\Maps\20646_F03_RGM_2023-12-28.mxd on 1/5/2024 7:21:21 AM Author: KVM (kmanchikanti)



Project: 20646	Eng/Geol: JDH/SGO
Scale: 1 " = 5 miles	Date: January 2024

Basemap Reference: © 2023 Microsoft Corporation Earthstar Geographics SIO © 2023 TomTom
Seismicity Data Reference: maps.conservation.ca.gov

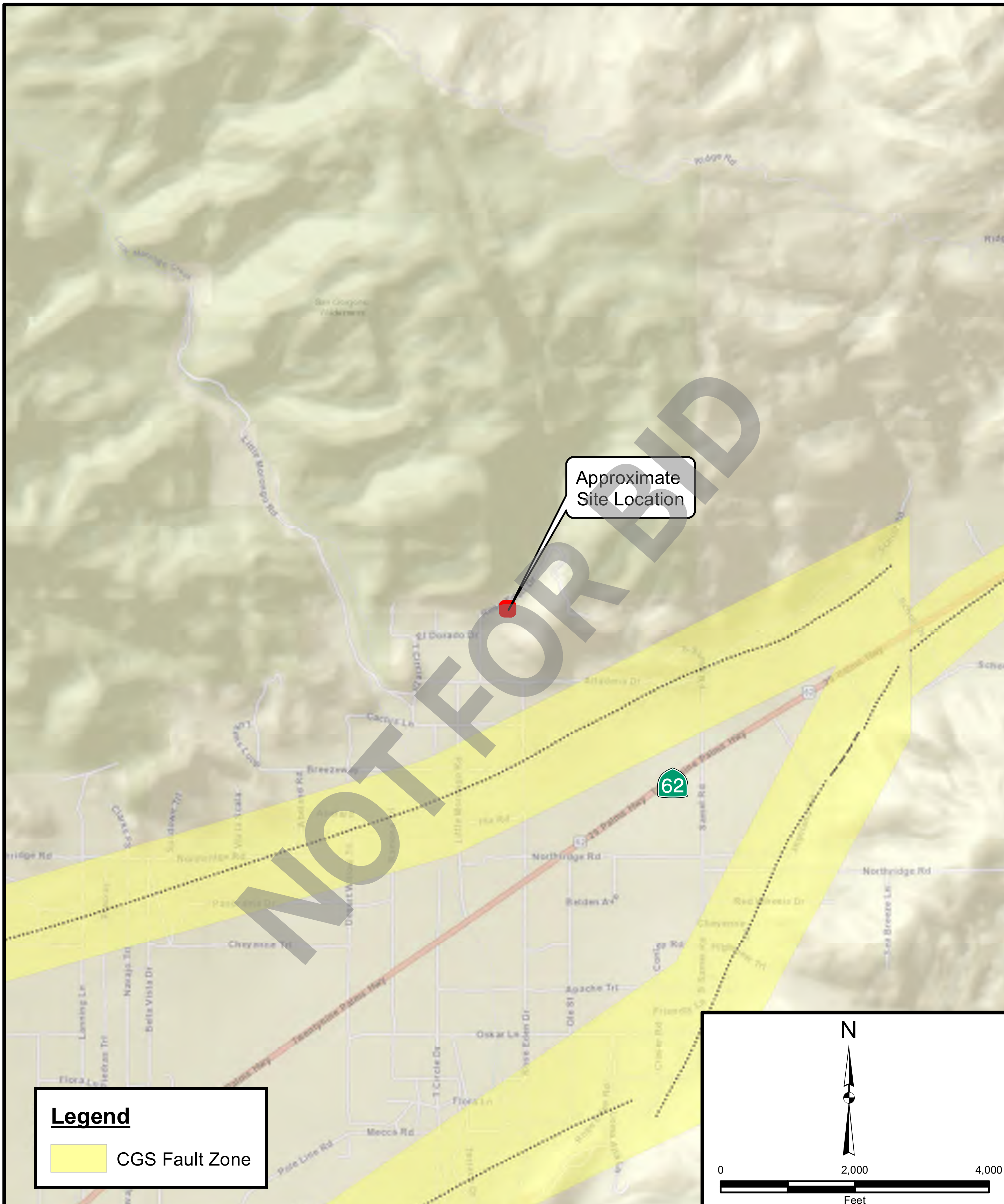
REGIONAL FAULT AND HISTORIC SEISMICITIY MAP

Proposed 155,000 Gallon Tank,
San Bernardino County Dept of Public Works, Special Districts
County Service Area (CSA) 70F, APN: 0580-241-16-0000
Terrace Drive, Morongo Valley, San Bernardino County, California

FIGURE 3

Leighton

Map Saved as J:\Drafting\20646\Maps\20646_F04_RFHSM_2023-12-21.mxd on 1/5/2024 6:21:03 AM Author: KVM (kmanchikanti)



Project: 20646

Eng/Geol: JDH/SGO

Scale: 1 " = 2,000 '

Date: January 2024

Service Layer Credits: Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community
Seismic Hazards Program, California Geological Survey, California

SEISMIC HAZARD MAP

Proposed 155,000 Gallon Tank,
San Bernardino County Dept of Public Works,
Special Districts, County Service Area (CSA) 70F,
APN: 0580-241-16-0000, Terrace Drive,
Morongo Valley, San Bernardino County, California

FIGURE 4

Leighton

Map Saved as J:\Drafting\20646\Maps\20646_F05_SHM_2023-12-21.mxd on 1/5/2024 6:32:16 AM Author: KVM (kmanchikanti)

APPENDIX A

SUBSURFACE EXPLORATION

Prior to subsurface exploration, we marked our proposed boring locations for coordination with DigAlert (811). On December 15, 2023, three borings were drilled with a truck-mounted hollow-stem-auger rig at this site to depths of 20 to 51½ feet. Encountered soils were logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). Relatively undisturbed California ring-lined soil drive-samples were obtained at selected depth intervals within these borings in accordance with ASTM Test Method D3550. Also, Standard Penetration Tests (SPTs) were driven at selected depths within these borings. Both of these drive samplers in the hollow-stem borings were driven with a 140-pound hammer falling 30 inches. Number of blows per 6 inches of penetration was recorded on our boring logs. Near surface bulk soil samples were also collected from these borings. Soil samples from our borings were transported to our in-house geotechnical laboratory for evaluation and appropriate testing. Our borings were backfilled with soil cuttings immediately after sampling and logging. Boring logs are included in this appendix.

Attached boring logs and related exploration information depict subsurface conditions only at the location indicated and at the particular date designated on the log. Subsurface conditions at other locations may differ from conditions occurring at this location. Passage of time may result in altered subsurface conditions due to environmental changes. In addition, stratification lines on these logs represent an approximate boundary between soil and rock types and these transitions may be gradual.

GEOTECHNICAL BORING LOG LB-1

Project No.	038.0000020646	**Date Drilled**	12-15-23
Project	SBCDPWSDD CSA 70F Tank Replacement	**Logged By**	BTM
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	3036'
Location	See Figure 2 - *Exploration Location Map*	**Sampled By**	BTM

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				SW	**Artificial Fill, undocumented (Afu)** @Surface: Gravel over well-graded SAND, yellowish brown, slightly moist, fine to coarse sand, few fine gravel	RV, CR
3035				S-1	6 3 6		2	SW	@2.5': Well-graded SAND, loose, orange brown, moist, fine to coarse sand, few fine granitic gravel, trace fines, nonplastic	
	5			S-2	12 50/5"		2	SW	**Quaternary-Age Alluvial Deposits (Qa)** @5': Well-graded SAND, very dense, orange brown, moist, fine to coarse sand, few fine gravel, trace fines, nonplastic, crushed rock in bottom of sample, 4% fines (lab)	-200
3030				S-3	10 19 34		2	SW-SM	@7.5': Well-graded SAND with SILT, dense, yellowish brown, slightly moist, fine to coarse sand, few to little fine gravel, trace to few fines, nonplastic	
3025	10			S-4	10 13 16		2	SW-SM	@10': Well-graded SAND with SILT, medium dense, yellowish brown, slightly moist, fine to coarse sand, few to little fine gravel, trace to few fines, nonplastic	
3020	15			S-5	14 21 18		2	SW-SM	@15': Well-graded SAND with SILT, dense, yellowish brown, slightly moist, fine to coarse sand, little fine gravel, trace to few fines, nonplastic	
3015	20			S-6	15 50/6"			SM	@20': SILTY SAND with GRAVEL, very dense, yellowish brown, slightly moist, fine to coarse sand, some fine gravel and broken rock bits, nonplastic	
3010	25			S-7	15 20 18			SW-SM	@25': Well-graded SAND with SILT and GRAVEL, dense, orangish brown, slightly moist, fine to coarse sand, some fine gravel, trace fines, nonplastic	
	30									

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.

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

Leighton

GEOTECHNICAL BORING LOG LB-1

Project No.	038.0000020646	**Date Drilled**	12-15-23
Project	SBCDPWSDD CSA 70F Tank Replacement	**Logged By**	BTM
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	3036'
Location	See Figure 2 - *Exploration Location Map*	**Sampled By**	BTM

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	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
3005	30			S-8	13 28 24			SW-SM	@30': Well-graded SAND with SILT, very dense, orangish brown, slightly moist, fine to coarse sand, few fine gravel, trace fines, nonplastic, 5% fines (lab)	-200
3000	35			S-9	19 30 34			SW-SM	@35': Well-graded SAND with SILT, very dense, dark yellowish brown, slightly moist, fine to coarse sand, some fine gravel, trace fines, nonplastic	
2995	40			S-10	18 30 47			IGNEOUS	**Grandodiorite to Quartz Monzonite (grd)** @40': IGNEOUS ROCK, granitic, intensely weathered, friable, white and black, slightly moist, fine to medium sand sized grains, some chemical weathering, some zones of more intact rock	
2990	45			S-11	37 50/5"			IGNEOUS	@45': IGNEOUS ROCK, granitic, intensely weathered, friable, white and black with some pink and orange oxidized zones, slightly moist, fine to medium sand sized grains, some chemical weathering, some zones of more intact rock	
2985	50			S-12	21 46 50/5"			IGNEOUS	@50': IGNEOUS ROCK, granitic, intensely weathered, friable, zones of some clay development, white and black with some pink and orange oxidized zones, slightly moist, fine to medium sand sized grains, some chemical weathering, some zones of more intact rock	
2980	55									
	60									

Total Depth: 51.5 feet bgs
No groundwater encountered during drilling
Boring backfilled with soil cuttings to surface.

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

Leighton

GEOTECHNICAL BORING LOG LB-2

Project No.	038.0000020646	**Date Drilled**	12-15-23
Project	SBCDPWSDD CSA 70F Tank Replacement	**Logged By**	BTM
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	3037'
Location	See Figure 2 - *Exploration Location Map*	**Sampled By**	BTM

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				SW-SM	**Artificial Fill, undocumented (Afu)** @Surface: Gravel over Poorly-graded SAND with SILT and GRAVEL, yellowish brown, slightly moist, fine to coarse sand, trace fine gravel, nonplastic, 5% fines (lab)	SA, EI, MD
3035				S-1	7 3 4			SW-SM	@2.5': Poorly-graded SAND with SILT, loose, yellowish brown, slightly moist, fine to coarse sand, trace fine granitic gravel, trace fines, nonplastic	
	5			S-2	23 23 24			SW-SM	**Quaternary-Age Alluvial Deposits (Qa)** @5': Well-graded SAND with SILT to SILTY SAND with GRAVEL, dense, yellowish brown, slightly moist, fine to coarse sand, few fine gravel, nonplastic	
3030				S-3	11 14 23			SW-SM	@7.5': Well-graded SAND with SILT and GRAVEL, dense, orange to yellowish brown, slightly moist, fine to coarse sand, some fine gravel, nonplastic	
	10			S-4	10 14 15			SW-SM	@10': Well-graded SAND with SILT and GRAVEL, medium dense, orange to yellowish brown, slightly moist, fine to coarse sand, some fine gravel, nonplastic	
3025										
	15			S-5	15 19 25			SM	@15': SILTY SAND with GRAVEL, dense, light brown, fine to coarse sand, slightly moist, some fine gravel, nonplastic	
3020										
	20			S-6	10 13 11			SM	@20': SILTY SAND with GRAVEL, medium dense, light brown, fine to coarse sand, slightly moist, some fine gravel, nonplastic	
3015										
	25			S-7	40 40 42			SM	@25': SILTY SAND with GRAVEL, very dense, light brown, fine to coarse sand, slightly moist, some fine gravel, nonplastic, some chunks of intact granitic rock (cobbles)	
3010										
	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

Leighton

GEOTECHNICAL BORING LOG LB-2

Project No.	038.0000020646	**Date Drilled**	12-15-23
Project	SBCDPWSDD CSA 70F Tank Replacement	**Logged By**	BTM
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	3037'
Location	See Figure 2 - *Exploration Location Map*	**Sampled By**	BTM

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
	30			S-8	14 20 23			SM	@30': SILTY SAND with GRAVEL, dense, light brown, slightly moist, fine to coarse sand, trace to few fine gravel, nonplastic	
3005										
3002	35			S-9	13 18 25			SW-SM	@35': Well-graded SAND with SILT, dense, yellowish brown, slightly moist, fine to coarse sand, trace to few fine gravel, nonplastic	
3000										
	40			S-10	50/5"			SW-SM	@40': Well-graded SAND with SILT, very dense, dark yellowish brown, moist, fine to medium sand, few coarse sand and fine gravel, nonplastic	
2995										
	45			S-11	16 17 14			SM	@45': SILTY SAND, dense, dark yellowish brown, slightly moist, fine to coarse sand, few fine gravel, nonplastic	
2990										
	50			S-12	6 9 18			SM	@50': SILTY SAND, medium dense, dark grayish brown, mostly fine sand, few medium to coarse sand, trace fine gravel, micaceous, very slightly cohesive	
2985									**Total Depth: 51.5 feet bgs** **No groundwater encountered during drilling** **Boring backfilled with soil cuttings to surface.**	
	55									
2980										
	60									

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

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GEOTECHNICAL BORING LOG LB-3

Project No.	038.0000020646	**Date Drilled**	12-15-23
Project	SBCDPWSDD CSA 70F Tank Replacement	**Logged By**	BTM
Drilling Co.	2R Drilling, Inc.	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	3037'
Location	See Figure 2 - *Exploration Location Map*	**Sampled By**	BTM

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
									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.	
	0			B-1				SW-SM	**Artificial Fill, undocumented (Afu)** @Surface: Gravel over well-graded SAND, yellowish brown, slightly moist, fine to coarse sand, few fine gravel	
3035				R-1	9 13 17		2	SW-SM	@2.5': Well-graded SAND, medium dense, yellowish brown, slightly moist, fine to coarse sand, trace to few fine granitic gravel, nonplastic	
	5			R-2	10 20 24		1	SW-SM	**Quaternary-Age Alluvial Deposits (Qa)** @5': Well-graded SAND with SILT, medium dense, yellowish brown, slightly moist, fine to coarse sand, few to little fine gravel, nonplastic, 3% fines (lab)	-200
3030				R-3	26 29 34			SW-SM	@7.5': Well-graded SAND with SILT, dense, yellowish brown, slightly moist, fine to coarse sand, few to little fine gravel, trace coarse gravel, nonplastic	
	10			R-4	50/2"			SW-SM	@10': Well-graded SAND with SILT, dense, yellowish brown, slightly moist, fine to coarse sand, few to little fine gravel, trace coarse gravel, nonplastic	
3025										
	15			S-5	50/5"			SW-SM	@15': Well-graded SAND with SILT, dense, yellowish brown, slightly moist, fine to coarse sand, few to little fine gravel, trace coarse gravel, nonplastic	
3020										
	20			R-6	50/4"			SM	@20': POOR RECOVERY; SILTY SAND with GRAVEL, very dense, fine to coarse sand, fine to coarse granitic gravel, nonplastic	
3015									**Total Depth: 20.5 feet bgs** **No groundwater encountered during drilling** **Boring backfilled with soil cuttings to surface.**	
	25									
3010										
	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

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

APPENDIX B

GEOTECHNICAL LABORATORY TESTING

This geotechnical laboratory testing program was directed toward quantitative and qualitative evaluation of physical and mechanical properties of soil underlying the site and to aid in soil classification.

In Situ Moisture: As-received sampled moisture content was evaluated (ASTM D2216) for samples of soil recovered from our subsurface explorations. Results of these tests are shown on boring logs at the appropriate sample depths, in Appendix A.

Percent Fines (Percentage Passing No. 200 Sieve, -200): bulk soil samples were wet-washed through a No. 200 U.S. Standard brass sieve in accordance with ASTM Test Methods D1140 to measure percent fines (silts and clays). This data was used to refine the Unified Soil Classification for tested soil. Test results are tabulated in this appendix and listed on our boring log in Appendix A.

Particle Size (Sieve) Analysis: Particle size analysis of bulk soil samples by passing sieves was evaluated using the ASTM D6913 Standard Test Method. Results of these analyses are presented on the Particle-Size Distribution ASTM D6913 sheets in this appendix.

Modified Proctor Compaction Curve (MD): A laboratory maximum-density compaction curve (ASTM D1557) was established for a bulk soil-sample to evaluate the modified-Proctor laboratory maximum dry density and optimum moisture content. Results of this test are presented on the following *Modified Proctor Compaction Test* sheet in this appendix.

Corrosivity Tests: To evaluate corrosion potential of subsurface soils at the site, we tested a bulk soil sample collected during our subsurface exploration for pH, electrical resistivity (CTM 532/643), soluble sulfate content (CTM 417 Part II) and soluble chloride content (CTM 422) testing. Results of these tests are enclosed within this appendix.

Expansion Index (EI): An Expansion Index (EI) test was performed on a representative earth material bulk sample from the site, in general accordance with the ASTM D4829 Standard Test Method. Results of this test are presented on the following “Expansion Index of Soils” in this appendix.

Resistance Value (R-Value): R-Value for a shallow bulk soil sample was established by California Test Method 301 to assist in preliminary pavement design recommendations. R-Value results are presented in this appendix on the R-Value Test Results sheets.

Boring No.	LB-1	LB-1	LB-3					
Sample No.	S-2	S-8	R-2					
Depth (ft.)	5.0	30.0	5.0					
Sample Type	SPT	SPT	RING					
Soil Classification	(SW)g	(SW-SM)g	(SW)g					
Soak Time (min)	10	10	10					
Moisture Correction								
Wet Weight of Soil + Container (gm.)	579.1	653.6	680.3					
Dry Weight of Soil + Container (gm.)	571.1	642.9	674.9					
Weight of Container (gm)	279.9	280.3	278.2					
Moisture Content (%)	2.7	3.0	1.4					
Container No.:	20	MA	LB					
Sample Dry Weight Determination								
Weight of Sample + Container (gm.)	571.1	642.9	674.9					
Weight of Container (gm.)	279.9	280.3	278.2					
Weight of Dry Sample (gm.)	291.2	362.6	396.7					
Container No.:	20	MA	LB					
After Wash								
Dry Weight of Sample + Container (gm)	558.2	625.1	662.1					
Weight of Container (gm)	279.9	280.3	278.2					
Dry Weight of Sample (gm)	278.3	344.8	383.9					
% Passing No. 200 Sieve	**4**	**5**	**3**					
% Retained No. 200 Sieve	96	95	97					

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PERCENT PASSING
No. 200 SIEVE
ASTM D 1140

Project Name: Morongo Valley Water Tank Replac.
Project No.: 038.0000020646
Client Name: SBCSDD
Tested By: M. Vinet Date: 12/19/23

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PARTICLE-SIZE DISTRIBUTION (GRADATION) of SOILS USING SIEVE ANALYSIS
ASTM D 6913

Project Name:	Morongo Valley Water Tank Replacement	Tested By:	MRV	Date:	12/20/23
Project No.:	038.0000020646	Checked By:	MRV	Date:	12/21/23
Boring No.:	LB-2	Depth (feet):	0 - 5.0		
Sample No.:	B-1				
Soil Identification:	Well-Graded Sand with Silt and Gravel (SW-SM)g, Yellowish Brown.				

Calculation of Dry Weights	Whole Sample	Sample Passing #4	Moisture Contents	Whole Sample	Sample passing #4
Container No.:	R2	R2	Wt. of Air-Dry Soil + Cont.(g)	3055.4	607.6
Wt. Air-Dried Soil + Cont.(g)	3055.4	607.6	Wt. of Dry Soil + Cont. (g)	3017.7	607.6
Wt. of Container (g)	276.5	276.5	Wt. of Container No._____(g)	276.5	276.5
Dry Wt. of Soil (g)	2740.5	331.1	Moisture Content (%)	1.4	0.0

Passing #4 Material After Wet Sieve		
	Container No.	R2
	Wt. of Dry Soil + Container (g)	587.7
	Wt. of Container (g)	276.5
	Dry Wt. of Soil Retained on # 200 Sieve (g)	311.2

U. S. Sieve Size		Cumulative Weight of Dry Soil Retained (g)		Percent Passing (%)
	(mm.)	Whole Sample	Sample Passing #4	
1 1/2"	37.500	0.0		100.0
1"	25.000	266.0		90.3
3/4"	19.000	366.6		86.6
1/2"	12.500	509.0		81.4
3/8"	9.500	609.6		77.8
#4	4.750	906.1		66.9
#8	2.360		62.4	54.3
#16	1.180		135.8	39.5
#30	0.600		200.0	26.5
#50	0.300		254.4	15.5
#100	0.150		289.0	8.5
#200	0.075		307.7	4.7
PAN				

GRAVEL: **33 %**

SAND: **62 %**

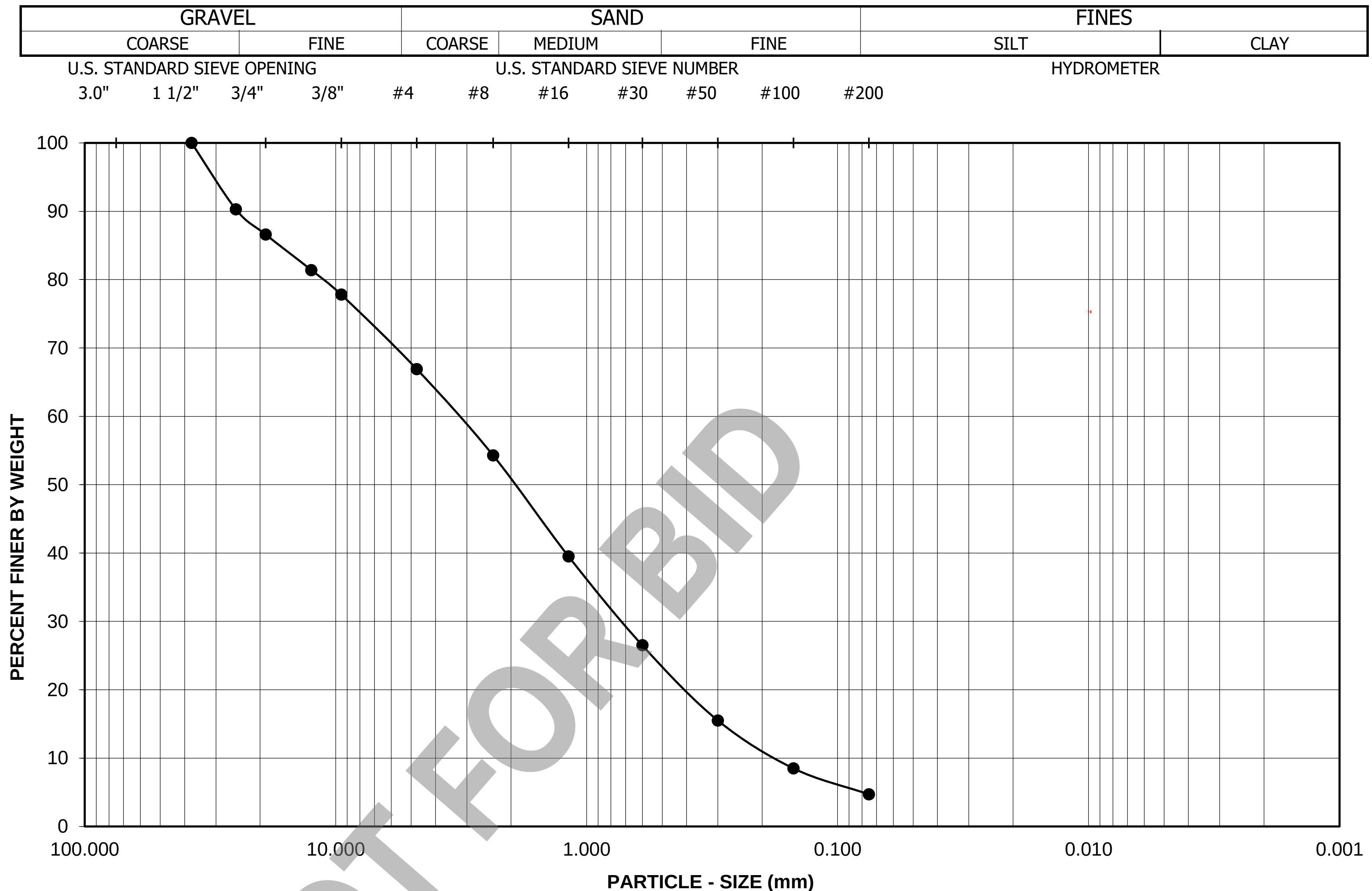
FINES: **5 %**

GROUP SYMBOL: **(SW-SM)g**

$Cu = D60/D10 =$ 18.24

$Cc = (D30)^2/(D60*D10) =$ 1.01

Remarks:



Project Name: Morongo Valley Water Tank Replacement

Project No.: 038.0000020646

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

Boring No.: LB-2 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, Yellowish Brown.

GR:SA:FI : (%) 33 : 62 : 5

Dec-23

MODIFIED PROCTOR COMPACTION TEST
ASTM D 1557

Leighton

Project Name:	Morongo Valley Water Tank Replacement	Tested By:	G. Stearns	Date:	12/20/23
Project No.:	038.0000020646	Input By:	M. Vinet	Date:	12/21/23
Boring No.:	LB-2	Depth (ft.):	0 - 5.0		
Sample No.:	B-1				
Soil Identification:	Well-Graded Sand with Silt and Gravel (SW-SM)g, Yellowish Brown				

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

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

Scalp Fraction (%)	
#3/4	13.4
#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)	9647	9836	9952	9933		
Weight of Mold (g)	5468	5468	5468	5468		
Net Weight of Soil (g)	4179	4368	4484	4465		
Wet Weight of Soil + Cont. (g)	1150.2	1325.6	1009.2	1009.2		
Dry Weight of Soil + Cont. (g)	1120.0	1270.0	958.4	946.5		
Weight of Container (g)	277.8	276.8	277.5	277.5		
Moisture Content (%)	3.6	5.6	7.5	9.4		
Wet Density (pcf)	122.8	128.4	131.8	131.2		
Dry Density (pcf)	118.6	121.6	122.7	120.0		

Maximum Dry Density (pcf) 122.8
Optimum Moisture Content (%) 7.0
Corrected Dry Density (pcf) 127.5
Corrected Moisture Content (%) 6.0

☐ **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

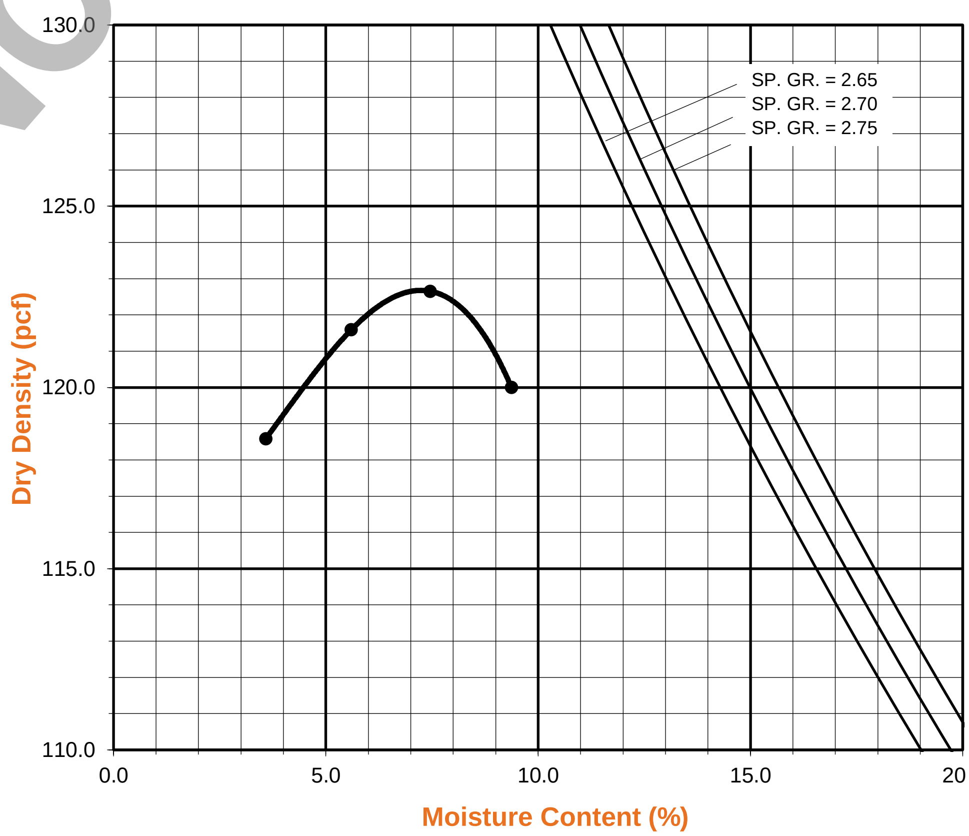
☐ **Procedure B**
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
Use if +#4 is >20% and +3/8 in. is 20% or less

☒ **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:
33:62:5
GR:SA:FI

Atterberg Limits:

LL,PL,PI



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EXPANSION INDEX of SOILS

ASTM D 4829

Project Name:	Morongo Valley Water Tank Replacement	Tested By:	M. Vinet	Date:	12/20/23
Project No. :	038.0000020646	Checked By:	M. Vinet	Date:	12/21/23
Boring No.:	LB-2	Depth:	0 - 5.0		
Sample No. :	B-1	Location:	N/A		
Sample Description:	Well-Graded Sand with Silt and Gravel (SW-SM)g, Yellowish Brown.				

Dry Wt. of Soil + Cont. (gm.)	2740.5
Wt. of Container No. (gm.)	0.0
Dry Wt. of Soil (gm.)	2740.5
Weight Soil Retained on #4 Sieve	906.1
Percent Passing # 4	66.9

MOLDED SPECIMEN	Before Test	After Test
Specimen Diameter (in.)	4.01	4.01
Specimen Height (in.)	1.0000	0.9920
Wt. Comp. Soil + Mold (gm.)	609.2	626.2
Wt. of Mold (gm.)	199.5	199.5
Specific Gravity (Assumed)	2.70	2.70
Container No.	9	9
Wet Wt. of Soil + Cont. (gm.)	576.5	626.2
Dry Wt. of Soil + Cont. (gm.)	551.7	375.9
Wt. of Container (gm.)	276.5	199.5
Moisture Content (%)	9.0	13.5
Wet Density (pcf)	123.6	129.8
Dry Density (pcf)	113.4	114.3
Void Ratio	0.487	0.475
Total Porosity	0.327	0.322
Pore Volume (cc)	67.8	66.1
Degree of Saturation (%) [S meas]	**49.9**	**76.9**

SPECIMEN INUNDATION in distilled water for the period of 24 h or expansion rate < 0.0002 in./h.

Date	Time	Pressure (psi)	Elapsed Time (min.)	Dial Readings (in.)
12/20/23	10:00	1.0	0	0.5000
12/20/23	10:10	1.0	10	0.5000
	Add Distilled Water to the Specimen			
12/21/23	9:30	1.0	1400	0.4920
12/21/23	10:30	1.0	1460	0.4920

Expansion Index (EI meas) =	((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	-8.0
Expansion Index (Report) =	Nearest Whole Number or Zero (0) if Initial Height is > than Final Height	**0**

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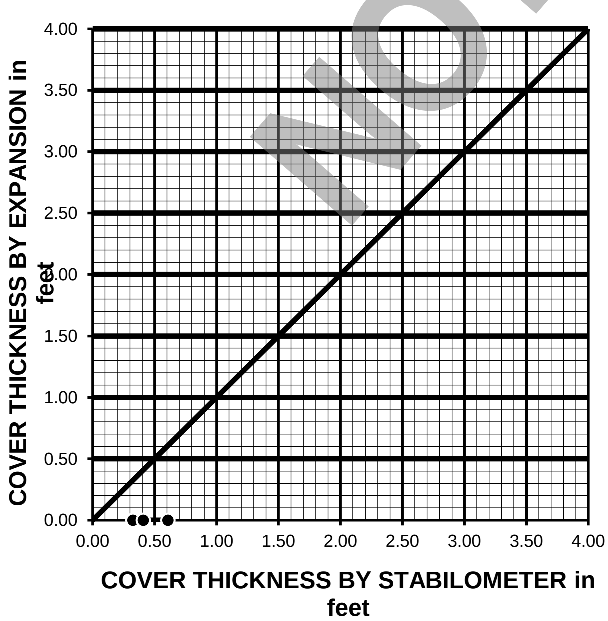
R-VALUE TEST RESULTS
ASTM D 2844

Project Name:	Morongo Valley Water Tank Replacement	Date:	12/21/23
Project Number:	038.0000020646	Technician:	M. Vinet
Boring Number:	LB-1	Depth (ft.):	0 - 7.0
Sample Number:	B-1	Sample Location:	N/A
Sample Description:	Well-Graded Sand with Silt and Gravel (SW-SM)g, Yellowish Brown.		

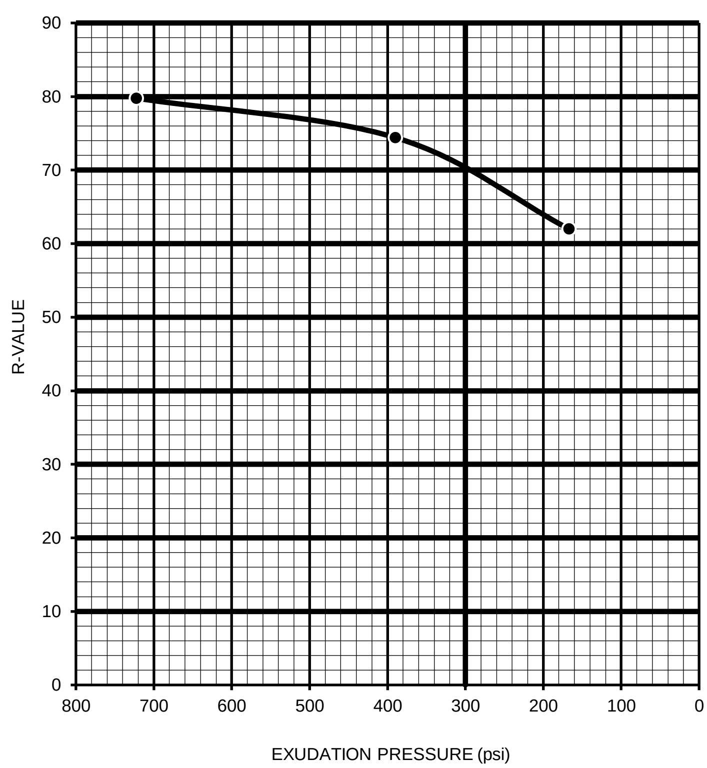
TEST SPECIMEN	A	B	C
MOISTURE AT COMPACTION %	7.3	8.4	9.4
HEIGHT OF SAMPLE, Inches	2.48	2.52	2.55
DRY DENSITY, pcf	117.1	118.2	114.5
COMPACTOR AIR PRESSURE, psi	350	350	350
EXUDATION PRESSURE, psi	722	390	167
EXPANSION, Inches x 10exp-4	0	0	0
STABILITY Ph 2,000 lbs (160 psi)	18	23	36
TURNS DISPLACEMENT	5.00	5.12	5.28
R-VALUE UNCORRECTED	80	74	62
R-VALUE CORRECTED	80	74	62

DESIGN CALCULATION DATA	a	b	c
GRAVEL EQUIVALENT FACTOR	1.0	1.0	1.0
TRAFFIC INDEX	5.0	5.0	5.0
STABILOMETER THICKNESS, ft.	0.32	0.41	0.61
EXPANSION PRESSURE THICKNESS, ft.	0.00	0.00	0.00

EXPANSION PRESSURE CHART



EXUDATION PRESSURE CHART



R-VALUE BY EXPANSION:	N/A
R-VALUE BY EXUDATION:	70
EQUILIBRIUM R-VALUE:	70

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TESTS for SULFATE CONTENT CHLORIDE CONTENT and pH of SOILS

Project Name: Morongo Valley Water Tank Replacement | Tested By : M. Vinet | Date: 12/21/23

Project No. : 038.0000020646 | Data Input By: M. Vinet | Date: 12/21/23

Boring No.	LB-1			
Sample No.	B-1			
Sample Depth (ft)	0 - 7.0			
Soil Identification:	(SW-SM)g			
Wet Weight of Soil + Container (g)	100.0			
Dry Weight of Soil + Container (g)	100.0			
Weight of Container (g)	0.0			
Moisture Content (%)	0.0			
Weight of Soaked Soil (g)	100.0			

SULFATE CONTENT, Hach Kit Method

Dilution : 1	3			
Water Fraction (ml)	25			
Tube Reading	<50			
PPM Sulfate	**<150**			
% Sulfate	<0.0150			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	30			
ml of AgNO3 Soln. Used in Titration (C)	0.6			
PPM of Chloride (C -0.2) * 100 * 30 / B	40			
PPM of Chloride, Dry Wt. Basis	**40**			

pH TEST, DOT California Test 643

pH Value	**7.70**			
Temperature °C	21.0			

SOIL RESISTIVITY TEST
DOT CA TEST 643

Project Name: Morongo Valley Water Tank Replacement

Project No. : 038.0000020646

Boring No.: LB-1

Sample No. : B-1

Soil Identification:* (SW-SM)g

Tested By : M. Vinet Date: 12/21/23

Data Input By: M. Vinet Date: 12/21/23

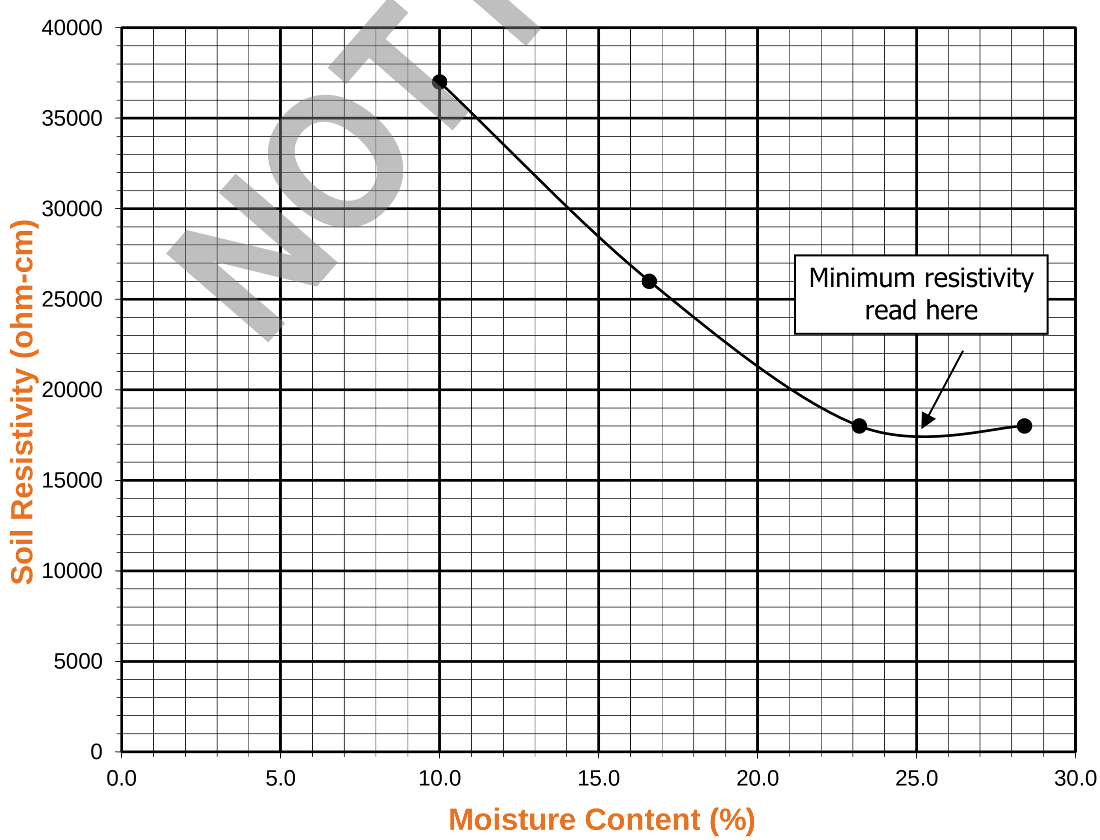
Depth (ft.) : 0 - 7.0

*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	37000	37000
2	83	16.60	26000	26000
3	116	23.20	18000	18000
4	142	28.40	18000	18000
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		Hach Kit	DOT CA Test 422	DOT CA Test 643	
17500	**25.0**	**<150**	**40**	**7.70**	**21.0**



APPENDIX C

SEISMIC ANALYSIS

Determination of Site Class and Estimation of Shear Wave Velocity

Project: 20646 CSA 70F Tank Replacement

Depth (ft)	di, Layer Thick (ft)	Field Blow Counts, Ni Corrected for Cs and sampler type Blows per foot (bpf)				Average Ni (bpf)	Ni Hammer Corr:	di / Ni
		LB-1	LB-2	LB-3			1.3	
5	7.5	100	47	26		58	75	0.10
10	5	29	29	60		39	51	0.10
15	5	39	44	100		61	79	0.06
20	5	100	24	60		61	80	0.06
25	5	38	82			60	78	0.06
30	5	52	43			48	62	0.08
35	5	64	43			54	70	0.07
40	5	77	100			89	100	0.05
45	5	100	31			66	85	0.06
50	7.5	100	27			64	83	0.09
60	10	100	27	*Assumed based on blowcount at 50'		64	83	0.12
70	10	100	27			64	83	0.12
80	10	100	27			64	83	0.12
90	10	100	27			64	83	0.12
100	5	100	27			64	83	0.06
Summatioı	100							1.29
							Navg = Sum(di) / Sum(di / Ni) =	**78**

Extract of ASCE 7-16 Table 20.3-1 Site Classification (2019 CBC 1613A.2.2):

Site Class	Soil Profile Name	Avg. N upper 100' from	to	Vs30 (ft/sec) from	to	Vs30 (m/s) from	to	Site Avg N	Interpolated vs30 (ft/s)
A	Hard Rock	-		5000	10000	1524	3048		
B	Rock	-		2500	5000	762	1524		
C	VD soil & soft rock	**50.001**	**100**	1200	2500	366	762	78	1923
D	Stiff Soil	15	50	600	1200	183	366		
E	Soft Soil	0	14.999	0	600	0	183		
F		-	-			0	0		

SITE CLASS, Table 20.3-1: **C**

Estimation of Average Shear Wave Velocity in upper 100 ft (Vs30):

	ft/s	m/s
Approx. Vs30 (interpolation of Table 20.3-1) =	0	0
Approx. Vs30 sands (Imai and Tonouchi, 1982) =	1374	419
Approx. Vs30 sands (Sykora and Stokoe, 1983) =	1134	346
Approx. Vs30 (Maheswari, Boominathan, Dodagoudar, 2009) =	1119	341

Liquefaction Susceptibility Analysis: SPT Method

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Youd and Idriss (2001), Martin and Lew (1999)

Description: CSA 70F Tank Replacement; Case 1; PGAm 1.04; design GW 100; No overex 0
Project No.: 20646
Dec 2023

General Boring Information:

Boring No.	Existing GW Depth (ft)	Design GW Depth (ft)	Design Fill Height (ft)	Overex. depth bgs (ft)	Ground Surface Elev (ft)	design gw elve
LB-1	430	100		0	3036	2936
LB-2	430	100		0	3037	2937
LB-3	430	100		0	3037	2937
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0

Boring Location Coordinates X (ft)	Y (ft)

General Parameters:
a_{max} = 1.04g
M_W = 7.3
MSF eq: 1
MSF = 1.07
Hammer Efficiency = 84
C_E = 1.40
C_B = 1
C_S for SPT? TRUE
Unlined, but room for liner
Rod Stickup (feet) = 3
Ring sample correction = 0.65

Summary of Liquefaction Susceptibility Analysis: SPT Method

Leighton

Liquefaction Method: Youd and Idriss (2001). Seismic Settlement Method: Tokimatsu and Seed (1987) and Martin and Lew (1999).

Project: CSA 70F Tank Replacement; Case 1; PGAm 1.04; design GW 100; No overex 0

Project No.: 20646

Boring No.	Approx. Layer Depth (ft)	SPT Depth (ft)	Approx Layer Thick-ness (ft)	Plasticity ("n"=non susc. to liq.)	Estimated Fines Cont (%)	γ_t (pcf)	N_m or B (blows/ft)	Sampler Type (enter 2 if mod CA Ring)	Cs	N_m (corrected for Cs and ring->SPT) (blows/ft)	Exist σ_{vo}' (psf)	$(N_1)_{60}$	$(N_1)_{60CS}$	**$CRR_{7.5}$**	Design σ_{vo}' (psf)	$CSR_{7.5}$	CSR_M	**Liquefaction Factor of Safety**	$(N_1)_{60CS}$ (for Settle-ment) (blows/ft)	Dry Sand Strain (%) (Tok/ Seed 87) (%)	Sat Sand Strain (%) (Tok/ Seed 87) (%)	Seismic Sett. of Layer (in.)	**Cummulative Seismic Settlement** (in.)
LB-1	0 to 3.8	2.5	3.8		5	120	9	1	1.19	10.7	300	19.1	19.1	0.205	300	0.67	0.63	**NonLiq**	19.1	1.07		0.48	0.8
LB-1	3.8 to 6.3	5	2.5		4	120	100	1	1.3	130.0	600	232.1	232.1	>Range	600	0.67	0.62	**NonLiq**	232.1	0.02		0.01	0.4
LB-1	6.3 to 8.8	7.5	2.5		10	120	53	1	1.3	68.9	900	117.5	120.9	>Range	900	0.66	0.62	**NonLiq**	120.9	0.03		0.01	0.4
LB-1	8.8 to 12.5	10	3.8		10	120	29	1	1.3	37.7	1200	59.2	61.3	>Range	1200	0.66	0.62	**NonLiq**	61.3	0.10		0.05	0.3
LB-1	12.5 to 17.5	15	5.0		10	120	39	1	1.3	50.7	1800	65.0	67.3	>Range	1800	0.65	0.61	**NonLiq**	67.3	0.06		0.03	0.3
LB-1	17.5 to 22.5	20	5.0		20	120	100	1	1.3	130.0	2400	161.3	177.7	>Range	2400	0.64	0.60	**NonLiq**	177.7	0.03		0.02	0.3
LB-1	22.5 to 27.5	25	5.0		10	120	38	1	1.3	49.4	3000	54.8	56.9	>Range	3000	0.64	0.59	**NonLiq**	56.9	0.16		0.10	0.2
LB-1	27.5 to 32.5	30	5.0		5	120	52	1	1.3	67.6	3600	72.1	72.1	>Range	3600	0.63	0.59	**NonLiq**	72.1	0.06		0.04	0.1
LB-1	32.5 to 37.5	35	5.0		5	120	64	1	1.3	83.2	4200	82.1	82.1	>Range	4200	0.60	0.56	**NonLiq**	82.1	0.06		0.04	0.1
LB-1	37.5 to 42.5	40	5.0		45	120	77	1	1.3	100.1	4800	92.4	115.9	>Range	4800	0.57	0.54	**NonLiq**	115.9	0.04		0.03	0.1
LB-1	42.5 to 47.5	45	5.0		45	120	100	1	1.3	130.0	5400	113.2	140.8	>Range	5400	0.55	0.51	**NonLiq**	140.8	0.04		0.02	0.0
LB-1	47.5 to 52.0	50	4.5		45	120	100	1	1.3	130.0	6000	107.4	133.8	>Range	6000	0.52	0.48	**NonLiq**	133.8	0.04		0.02	0.0
LB-2	0 to 3.8	2.5	3.8		5	120	7	1	1.14	8.0	300	14.3	14.3	0.153	300	0.67	0.63	**NonLiq**	14.3	1.93		0.87	1.6
LB-2	3.8 to 6.3	5	2.5		5	120	47	1	1.3	61.1	600	109.1	109.1	>Range	600	0.67	0.62	**NonLiq**	109.1	0.06		0.02	0.7
LB-2	6.3 to 8.8	7.5	2.5		10	120	37	1	1.3	48.1	900	82.1	84.7	>Range	900	0.66	0.62	**NonLiq**	84.7	0.05		0.01	0.7
LB-2	8.8 to 12.5	10	3.8		10	120	29	1	1.3	37.7	1200	59.2	61.3	>Range	1200	0.66	0.62	**NonLiq**	61.3	0.10		0.05	0.7
LB-2	12.5 to 17.5	15	5.0		20	120	44	1	1.3	57.2	1800	73.3	82.8	>Range	1800	0.65	0.61	**NonLiq**	82.8	0.04		0.03	0.7
LB-2	17.5 to 22.5	20	5.0		20	120	24	1	1.3	31.2	2400	38.7	45.4	>Range	2400	0.64	0.60	**NonLiq**	45.4	0.15		0.09	0.6
LB-2	22.5 to 27.5	25	5.0		20	120	82	1	1.3	106.6	3000	118.3	131.3	>Range	3000	0.64	0.59	**NonLiq**	131.3	0.06		0.04	0.5
LB-2	27.5 to 32.5	30	5.0		20	120	43	1	1.3	55.9	3600	59.6	68.0	>Range	3600	0.63	0.59	**NonLiq**	68.0	0.07		0.04	0.5
LB-2	32.5 to 37.5	35	5.0		10	120	43	1	1.3	55.9	4200	55.2	57.2	>Range	4200	0.60	0.56	**NonLiq**	57.2	0.09		0.06	0.5
LB-2	37.5 to 42.5	40	5.0		10	120	100	1	1.3	130.0	4800	120.0	123.5	>Range	4800	0.57	0.54	**NonLiq**	123.5	0.04		0.02	0.4
LB-2	42.5 to 47.5	45	5.0		20	120	31	1	1.3	40.3	5400	35.1	41.5	>Range	5400	0.55	0.51	**NonLiq**	41.5	0.16		0.10	0.4
LB-2	47.5 to 52.0	50	4.5		20	120	27	1	1.29	34.7	6000	28.7	34.6	>Range	6000	0.52	0.48	**NonLiq**	34.6	0.53		0.29	0.3
LB-3	0 to 3.8	2.5	3.8		10	120	30	2	1	19.5	300	34.8	36.4	>Range	300	0.67	0.63	**NonLiq**	36.4	0.21		0.10	0.2
LB-3	3.8 to 6.3	5	2.5		3	120	44	2	1	28.6	600	51.1	51.1	>Range	600	0.67	0.62	**NonLiq**	51.1	0.14		0.04	0.1
LB-3	6.3 to 8.8	7.5	2.5		10	120	63	2	1	41.0	900	69.9	72.2	>Range	900	0.66	0.62	**NonLiq**	72.2	0.06		0.02	0.1
LB-3	8.8 to 12.5	10	3.8		10	120	100	2	1	65.0	1200	102.0	105.1	>Range	1200	0.66	0.62	**NonLiq**	105.1	0.06		0.03	0.1
LB-3	12.5 to 17.5	15	5.0		10	120	100	1	1.3	130.0	1800	166.6	171.1	>Range	1800	0.65	0.61	**NonLiq**	171.1	0.02		0.01	0.1
LB-3	17.5 to 22.0	20	4.5		20	120	100	2	1	65.0	2400	80.6	90.7	>Range	2400	0.64	0.60	**NonLiq**	90.7	0.07		0.04	0.0

Liquefaction Susceptibility Analysis: SPT Method

Leighton

Youd and Idriss (2001), Martin and Lew (1999)

Description: CSA 70F Tank Replacement; Case 3; PGAm 1.04; design GW 100; Overex./scarify 5
Project No.: 20646
Dec 2023

General Boring Information:

Boring No.	Existing GW Depth (ft)	Design GW Depth (ft)	Design Fill Height (ft)	Overex. depth bgs (ft)	Ground Surface Elev (ft)	design gw elve
LB-1	430	100		5	3036	2936
LB-2	430	100		5	3037	2937
LB-3	430	100		5	3037	2937
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0

Boring Location Coordinates X (ft)	Boring Location Coordinates Y (ft)

General Parameters:

- a_{max} = 1.04g
- M_W = 7.3
- MSF eq: 1
- MSF = 1.07
- Hammer Efficiency = 84
- C_E = 1.40
- C_B = 1
- C_S for SPT? TRUE
- Unlined, but room for liner
- Rod Stickup (feet) = 3
- Ring sample correction = 0.65

Summary of Liquefaction Susceptibility Analysis: SPT Method

Liquefaction Method: Youd and Idriss (2001). Seismic Settlement Method: Tokimatsu and Seed (1987) and Martin and Lew (1999).

Project: CSA 70F Tank Replacement; Case 3; PGAm 1.04; design GW 100; Overex./scarify 5

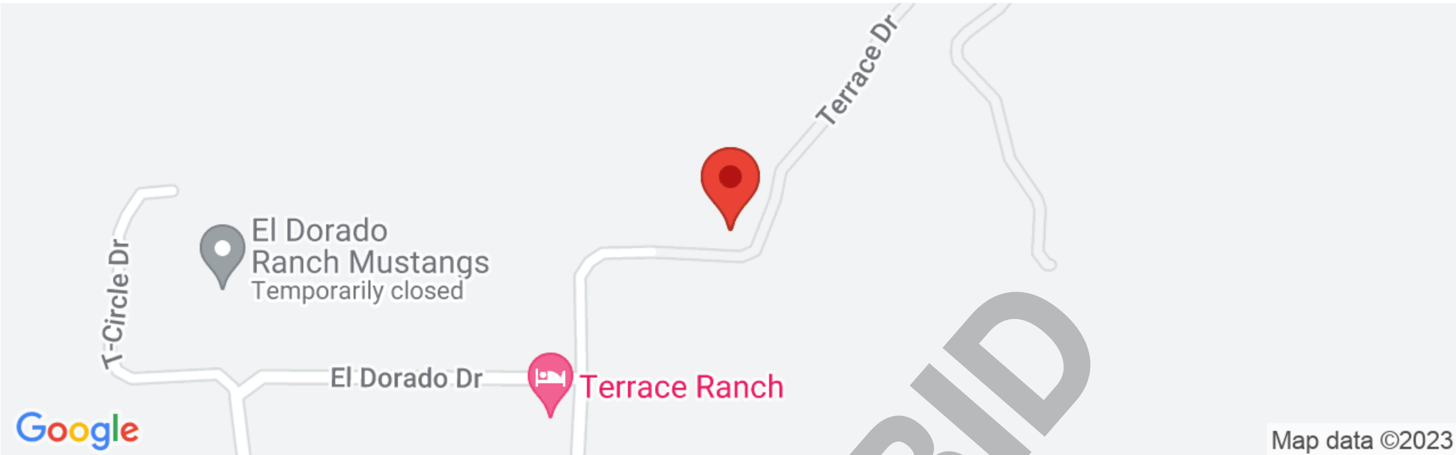
Project No.: 20646

Boring No.	Approx. Layer Depth (ft)	SPT Depth (ft)	Approx Layer Thick-ness (ft)	Plasticity ("n"=non susc. to liq.)	Estimated Fines Cont (%)	γ_t (pcf)	N_m or B (blows/ft)	Sampler Type (enter 2 if mod CA Ring)	Cs	N_m (corrected for Cs and ring->SPT) (blows/ft)	Exist σ_{vo}' (psf)	$(N_1)_{60}$	$(N_1)_{60CS}$	**$CRR_{7.5}$**	Design σ_{vo}' (psf)	$CSR_{7.5}$	CSR_M	**Liquefaction Factor of Safety**	$(N_1)_{60CS}$ (for Settle-ment) (blows/ft)	Dry Sand Strain (%) (Tok/ Seed 87) (%)	Sat Sand Strain (%) (Tok/ Seed 87) (%)	Seismic Sett. of Layer (in.)	**Cummulative Seismic Settlement** (in.)
LB-1	0 to 3.8	2.5	3.8	OX	5	120	50	1	1.3	65.0	300	116.0	116.0	>Range	300	0.67	0.63	**NonLiq**	116.0	0.00		0.00	0.4
LB-1	3.8 to 5.0	5	1.3	OX	**4**	120	50	1	1.3	65.0	600	116.0	116.0	>Range	600	0.67	0.62	**NonLiq**	116.0	0.00		0.00	0.4
LB-1	5.0 to 6.3	5	1.3		**4**	120	100	1	1.3	130.0	600	232.1	232.1	>Range	600	0.67	0.62	**NonLiq**	232.1	0.02		0.00	0.4
LB-1	6.3 to 8.8	7.5	2.5		10	120	53	1	1.3	68.9	900	117.5	120.9	>Range	900	0.66	0.62	**NonLiq**	120.9	0.03		0.01	0.4
LB-1	8.8 to 12.5	10	3.8		10	120	29	1	1.3	37.7	1200	59.2	61.3	>Range	1200	0.66	0.62	**NonLiq**	61.3	0.10		0.05	0.3
LB-1	12.5 to 17.5	15	5.0		10	120	39	1	1.3	50.7	1800	65.0	67.3	>Range	1800	0.65	0.61	**NonLiq**	67.3	0.06		0.03	0.3
LB-1	17.5 to 22.5	20	5.0		20	120	100	1	1.3	130.0	2400	161.3	177.7	>Range	2400	0.64	0.60	**NonLiq**	177.7	0.03		0.02	0.3
LB-1	22.5 to 27.5	25	5.0		10	120	38	1	1.3	49.4	3000	54.8	56.9	>Range	3000	0.64	0.59	**NonLiq**	56.9	0.16		0.10	0.2
LB-1	27.5 to 32.5	30	5.0		**5**	120	52	1	1.3	67.6	3600	72.1	72.1	>Range	3600	0.63	0.59	**NonLiq**	72.1	0.06		0.04	0.1
LB-1	32.5 to 37.5	35	5.0		5	120	64	1	1.3	83.2	4200	82.1	82.1	>Range	4200	0.60	0.56	**NonLiq**	82.1	0.06		0.04	0.1
LB-1	37.5 to 42.5	40	5.0		45	120	77	1	1.3	100.1	4800	92.4	115.9	>Range	4800	0.57	0.54	**NonLiq**	115.9	0.04		0.03	0.1
LB-1	42.5 to 47.5	45	5.0		45	120	100	1	1.3	130.0	5400	113.2	140.8	>Range	5400	0.55	0.51	**NonLiq**	140.8	0.04		0.02	0.0
LB-1	47.5 to 52.0	50	4.5		45	120	100	1	1.3	130.0	6000	107.4	133.8	>Range	6000	0.52	0.48	**NonLiq**	133.8	0.04		0.02	0.0
LB-2	0 to 3.8	2.5	3.8	OX	**5**	120	50	1	1.3	65.0	300	116.0	116.0	>Range	300	0.67	0.63	**NonLiq**	116.0	0.00		0.00	0.7
LB-2	3.8 to 5.0	5	1.3	OX	5	120	50	1	1.3	65.0	600	116.0	116.0	>Range	600	0.67	0.62	**NonLiq**	116.0	0.00		0.00	0.7
LB-2	5.0 to 6.3	5	1.3		5	120	47	1	1.3	61.1	600	109.1	109.1	>Range	600	0.67	0.62	**NonLiq**	109.1	0.06		0.01	0.7
LB-2	6.3 to 8.8	7.5	2.5		10	120	37	1	1.3	48.1	900	82.1	84.7	>Range	900	0.66	0.62	**NonLiq**	84.7	0.05		0.01	0.7
LB-2	8.8 to 12.5	10	3.8		10	120	29	1	1.3	37.7	1200	59.2	61.3	>Range	1200	0.66	0.62	**NonLiq**	61.3	0.10		0.05	0.7
LB-2	12.5 to 17.5	15	5.0		20	120	44	1	1.3	57.2	1800	73.3	82.8	>Range	1800	0.65	0.61	**NonLiq**	82.8	0.04		0.03	0.7
LB-2	17.5 to 22.5	20	5.0		20	120	24	1	1.3	31.2	2400	38.7	45.4	>Range	2400	0.64	0.60	**NonLiq**	45.4	0.15		0.09	0.6
LB-2	22.5 to 27.5	25	5.0		20	120	82	1	1.3	106.6	3000	118.3	131.3	>Range	3000	0.64	0.59	**NonLiq**	131.3	0.06		0.04	0.5
LB-2	27.5 to 32.5	30	5.0		20	120	43	1	1.3	55.9	3600	59.6	68.0	>Range	3600	0.63	0.59	**NonLiq**	68.0	0.07		0.04	0.5
LB-2	32.5 to 37.5	35	5.0		10	120	43	1	1.3	55.9	4200	55.2	57.2	>Range	4200	0.60	0.56	**NonLiq**	57.2	0.09		0.06	0.5
LB-2	37.5 to 42.5	40	5.0		10	120	100	1	1.3	130.0	4800	120.0	123.5	>Range	4800	0.57	0.54	**NonLiq**	123.5	0.04		0.02	0.4
LB-2	42.5 to 47.5	45	5.0		20	120	31	1	1.3	40.3	5400	35.1	41.5	>Range	5400	0.55	0.51	**NonLiq**	41.5	0.16		0.10	0.4
LB-2	47.5 to 52.0	50	4.5		20	120	27	1	1.29	34.7	6000	28.7	34.6	>Range	6000	0.52	0.48	**NonLiq**	34.6	0.53		0.29	0.3
LB-3	0 to 3.8	2.5	3.8	OX	10	120	50	1	1.3	65.0	300	116.0	119.4	>Range	300	0.67	0.63	**NonLiq**	119.4	0.00		0.00	0.1
LB-3	3.8 to 5.0	5	1.3	OX	**3**	120	50	1	1.3	65.0	600	116.0	116.0	>Range	600	0.67	0.62	**NonLiq**	116.0	0.00		0.00	0.1
LB-3	5.0 to 6.3	5	1.3		**3**	120	44	2	1	28.6	600	51.1	51.1	>Range	600	0.67	0.62	**NonLiq**	51.1	0.14		0.02	0.1
LB-3	6.3 to 8.8	7.5	2.5		10	120	63	2	1	41.0	900	69.9	72.2	>Range	900	0.66	0.62	**NonLiq**	72.2	0.06		0.02	0.1
LB-3	8.8 to 12.5	10	3.8		10	120	100	2	1	65.0	1200	102.0	105.1	>Range	1200	0.66	0.62	**NonLiq**	105.1	0.06		0.03	0.1
LB-3	12.5 to 17.5	15	5.0		10	120	100	1	1.3	130.0	1800	166.6	171.1	>Range	1800	0.65	0.61	**NonLiq**	171.1	0.02		0.01	0.1
LB-3	17.5 to 22.0	20	4.5		20	120	100	2	1	65.0	2400	80.6	90.7	>Range	2400	0.64	0.60	**NonLiq**	90.7	0.07		0.04	0.0

USGS web services were down for some period of time and as a result this tool wasn't operational, resulting in *timeout* error.
USGS web services are now operational so this tool should work as expected.

OSHPD

Latitude, Longitude: 34.1005, -116.5206



Date	12/18/2023, 9:40:30 AM
Design Code Reference Document	ASCE7-16
Risk Category	IV
Site Class	C - Very Dense Soil and Soft Rock

Type	Value	Description
S_S	2.042	MCE_R ground motion. (for 0.2 second period)
S_1	0.752	MCE_R ground motion. (for 1.0s period)
S_{MS}	2.451	Site-modified spectral acceleration value
S_{M1}	1.052	Site-modified spectral acceleration value
S_{DS}	1.634	Numeric seismic design value at 0.2 second SA
S_{D1}	0.702	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	0.866	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	1.04	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	2.042	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.229	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.31	Factored deterministic acceleration value. (0.2 second)
S1RT	0.752	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.832	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.851	Factored deterministic acceleration value. (1.0 second)
PGAd	0.959	Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA_{UH}	0.866	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.916	Mapped value of the risk coefficient at short periods
C_{R1}	0.903	Mapped value of the risk coefficient at a period of 1 s
C_V	1.3	Vertical coefficient

DISCLAIMER

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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.

Please also see the new USGS Earthquake Hazard Toolbox for access to the most recent NSHMs for the conterminous U.S. and Hawaii.

Input

Edition

Dynamic: Conterminous U.S. 2014 (update) (4.2.0)

Latitude

Decimal degrees

34.1005

Longitude

Decimal degrees, negative values for western longitudes

-116.5206

Site Class

537 m/s (Site class C)

Spectral Period

Peak Ground Acceleration

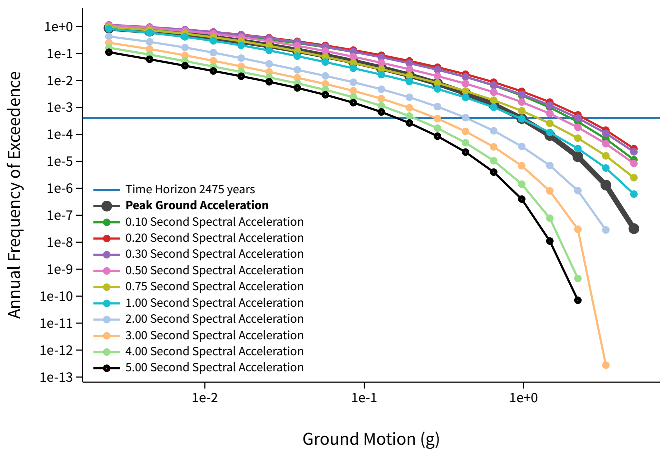
Time Horizon

Return period in years

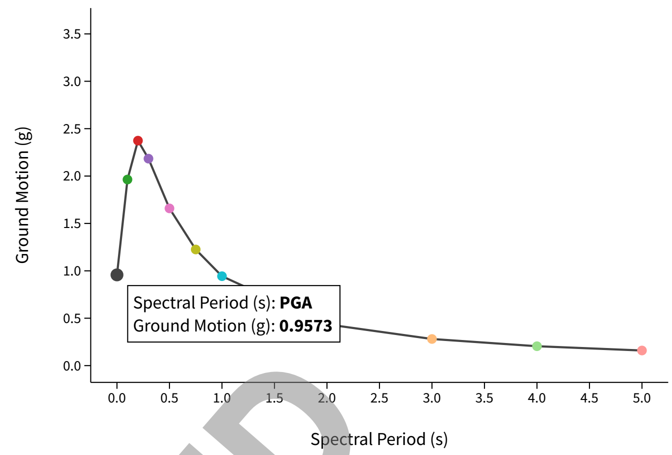
2475

Hazard Curve

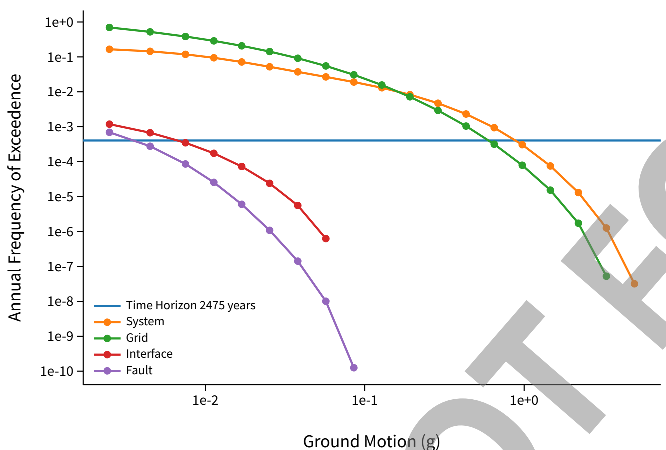
Hazard Curves



Uniform Hazard Response Spectrum



Component Curves for Peak Ground Acceleration

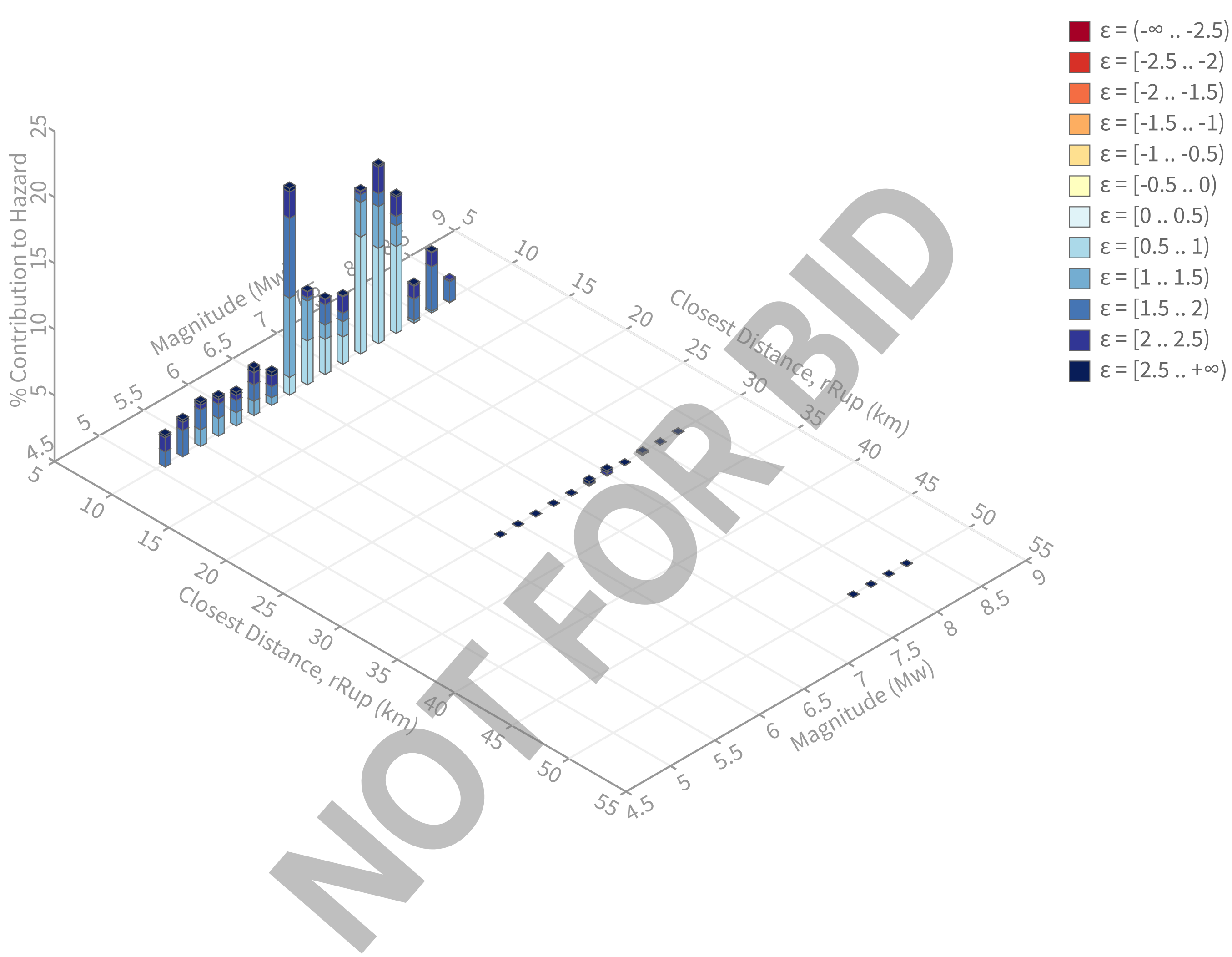


View Raw Data

^ Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.95728454 g

Recovered targets

Return period: 2942.3325 yrs
Exceedance rate: 0.00033986642 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.05 %

Mean (over all sources)

m: 6.94
r: 6.98 km
ε_0: 1.43 σ

Mode (largest m-r bin)

m: 6.56
r: 8.44 km
ε_0: 1.61 σ
Contribution: 15.64 %

Mode (largest m-r-ε_0 bin)

m: 7.29
r: 1.64 km
ε_0: 0.84 σ
Contribution: 8.85 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM32	System							40.59
Pinto Mtn [2]		1.64	7.22	0.91	116.516°W	34.098°N	118.67	20.57
Burnt Mtn [0]		9.32	6.58	1.57	116.418°W	34.106°N	86.44	5.20
San Andreas (San Gorgonio Pass-Garnet HIll) [5]		18.91	7.88	2.02	116.588°W	33.915°N	196.81	4.75
Pinto Mtn [3]		3.55	7.38	1.10	116.487°W	34.108°N	74.37	2.26
North Frontal (East) [2]		16.00	7.03	1.77	116.530°W	34.294°N	357.64	1.44
San Andreas (North Branch Mill Creek) [7]		12.82	8.05	1.70	116.571°W	33.998°N	202.21	1.29
Eureka Peak [2]		11.95	6.47	2.46	116.395°W	34.122°N	78.08	1.02
UC33brAvg_FM31	System							39.84
Pinto Mtn [2]		1.64	7.28	0.90	116.516°W	34.098°N	118.67	19.84
Burnt Mtn [0]		9.32	6.58	1.57	116.418°W	34.106°N	86.44	5.23
San Andreas (San Gorgonio Pass-Garnet HIll) [5]		18.91	7.88	2.03	116.588°W	33.915°N	196.81	4.75
Pinto Mtn [3]		3.55	7.40	1.09	116.487°W	34.108°N	74.37	2.15
North Frontal (East) [2]		16.00	7.03	1.77	116.530°W	34.294°N	357.64	1.43
Mission Creek [0]		10.81	7.34	1.53	116.572°W	34.011°N	205.31	1.20
San Andreas (North Branch Mill Creek) [7]		12.82	8.07	1.69	116.571°W	33.998°N	202.21	1.14
Johnson Valley (No) 2011 rev [0]		11.97	6.92	2.21	116.421°W	34.168°N	50.58	1.05
UC33brAvg_FM31 (opt)	Grid							9.78
PointSourceFinite: -116.521, 34.114		5.37	5.60	1.68	116.521°W	34.114°N	0.00	3.70
PointSourceFinite: -116.521, 34.114		5.37	5.60	1.68	116.521°W	34.114°N	0.00	3.70
UC33brAvg_FM32 (opt)	Grid							9.78
PointSourceFinite: -116.521, 34.114		5.37	5.60	1.68	116.521°W	34.114°N	0.00	3.70
PointSourceFinite: -116.521, 34.114		5.37	5.60	1.68	116.521°W	34.114°N	0.00	3.70

APPENDIX D

EARTHWORK AND GRADING GUIDE SPECIFICATIONS

GENERAL EARTHWORK AND GRADING SPECIFICATIONS FOR ROUGH GRADING

Table of Contents

1.0 General

1.1 Intent: These General Earthwork and Grading Specifications are for the grading and earthwork shown on the approved grading plan(s) and/or indicated in the geotechnical report(s). These Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the specific recommendations in the geotechnical report shall supersede these more general Specifications. Observations of the earthwork by the project Geotechnical Consultant during the course of grading may result in new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).

1.2 The Geotechnical Consultant of Record: Prior to commencement of work, the owner shall employ the Geotechnical Consultant of Record (Geotechnical Consultant). The Geotechnical Consultants shall be responsible for reviewing the approved geotechnical report(s) and accepting the adequacy of the preliminary geotechnical findings, conclusions, and recommendations prior to the commencement of the grading.

Prior to commencement of grading, the Geotechnical Consultant shall review the "work plan" prepared by the Earthwork Contractor (Contractor) and schedule sufficient personnel to perform the appropriate level of observation, mapping, and compaction testing.

During the grading and earthwork operations, the Geotechnical Consultant shall observe, map, and document the subsurface exposures to verify the geotechnical design assumptions. If the observed conditions are found to be significantly different than the interpreted assumptions during the design phase, the Geotechnical Consultant shall inform the owner, recommend appropriate changes in design to accommodate the observed conditions, and notify the review agency where required. Subsurface areas to be geotechnically observed, mapped, elevations recorded, and/or tested include natural ground after it has been cleared for receiving fill but before fill is placed, bottoms of all "remedial removal" areas, all key bottoms, and benches made on sloping ground to receive fill.

The Geotechnical Consultant shall observe the moisture-conditioning and processing of the subgrade and fill materials and perform relative compaction testing of fill to determine the attained level of compaction. The Geotechnical Consultant shall provide the test results to the owner and the Contractor on a routine and frequent basis.

1.3 The Earthwork Contractor: The Earthwork Contractor (Contractor) shall be qualified, experienced, and knowledgeable in earthwork logistics, preparation and processing of ground to receive fill, moisture-conditioning and processing of fill, and compacting fill. The Contractor shall review and accept the plans, geotechnical report(s), and these Specifications prior to commencement of grading. The

Contractor shall be solely responsible for performing the grading in accordance with the plans and specifications.

The Contractor shall prepare and submit to the owner and the Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of "spreads" of work and the estimated quantities of daily earthwork contemplated for the site prior to commencement of grading. The Contractor shall inform the owner and the Geotechnical Consultant of changes in work schedules and updates to the work plan at least 24 hours in advance of such changes so that appropriate observations and tests can be planned and accomplished. The Contractor shall not assume that the Geotechnical Consultant is aware of all grading operations.

The Contractor shall have the sole responsibility to provide adequate equipment and methods to accomplish the earthwork in accordance with the applicable grading codes and agency ordinances, these Specifications, and the recommendations in the approved geotechnical report(s) and grading plan(s). If, in the opinion of the Geotechnical Consultant, unsatisfactory conditions, such as unsuitable soil, improper moisture condition, inadequate compaction, insufficient buttress key size, adverse weather, etc., are resulting in a quality of work less than required in these specifications, the Geotechnical Consultant shall reject the work and may recommend to the owner that construction be stopped until the conditions are rectified.

2.0 Preparation of Areas to be Filled

2.1 Clearing and Grubbing: Vegetation, such as brush, grass, roots, and other deleterious material shall be sufficiently removed and properly disposed of in a method acceptable to the owner, governing agencies, and the Geotechnical Consultant.

The Geotechnical Consultant shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 1 percent of organic materials (by volume). No fill lift shall contain more than 5 percent of organic matter. Nesting of the organic materials shall not be allowed.

If potentially hazardous materials are encountered, the Contractor shall stop work in the affected area, and a hazardous material specialist shall be informed immediately for proper evaluation and handling of these materials prior to continuing to work in that area.

As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant, etc.) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed.

2.2 Processing: Existing ground that has been declared satisfactory for support of fill by the Geotechnical Consultant shall be scarified to a minimum depth of 6 inches. Existing ground that is not satisfactory shall be overexcavated as specified in the following section. Scarification shall continue until soils are broken down and free of large clay lumps or clods and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.

2.3 Overexcavation: In addition to removals and overexcavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be overexcavated to competent ground as evaluated by the Geotechnical Consultant during grading.

2.4 Benching: Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), the ground shall be stepped or benched. Please see the Standard Details for a graphic illustration. The lowest bench or key shall be a minimum of 15 feet wide and at least 2 feet deep, into competent material as evaluated by the Geotechnical Consultant. Other benches shall be excavated a minimum height of 4 feet into competent material or as otherwise recommended by the Geotechnical Consultant. Fill placed on ground sloping flatter than 5:1 shall also be benched or otherwise overexcavated to provide a flat subgrade for the fill.

2.5 Evaluation/Acceptance of Fill Areas: All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by the Geotechnical Consultant as suitable to receive fill. The Contractor shall obtain a written acceptance from the Geotechnical Consultant prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys, and benches.

3.0 Fill Material

3.1 General: Material to be used as fill shall be essentially free of organic matter and other deleterious substances evaluated and accepted by the Geotechnical Consultant prior to placement. Soils of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to the Geotechnical Consultant or mixed with other soils to achieve satisfactory fill material.

3.2 Oversize: Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 8 inches, shall not be buried or placed in fill unless location, materials, and placement methods are specifically accepted by the Geotechnical Consultant. Placement operations shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 vertical feet of finish grade or within 2 feet of future utilities or underground construction.

3.3 Import: If importing of fill material is required for grading, proposed import material shall meet the requirements of Section 3.1. The potential import source shall be given to the Geotechnical Consultant at least 48 hours (2 working days) before importing begins so that its suitability can be determined and appropriate tests performed.

4.0 Fill Placement and Compaction

4.1 Fill Layers: Approved fill material shall be placed in areas prepared to receive fill (per Section 3.0) in near-horizontal layers not exceeding 8 inches in loose thickness. The Geotechnical Consultant may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.

4.2 Fill Moisture Conditioning: Fill soils shall be watered, dried back, blended, and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly over optimum. Maximum density and optimum soil moisture content tests shall be performed in accordance with the American Society of Testing and Materials (ASTM Test Method D1557-91).

4.3 Compaction of Fill: After each layer has been moisture-conditioned, mixed, and evenly spread, it shall be uniformly compacted to not less than 90 percent of maximum dry density (ASTM Test Method D1557-91). Compaction equipment shall be adequately sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.

4.4 Compaction of Fill Slopes: In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by backrolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet in fill elevation, or by other methods producing satisfactory results acceptable to the Geotechnical Consultant. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of maximum density per ASTM Test Method D1557-91.

4.5 Compaction Testing: Field tests for moisture content and relative compaction of the fill soils shall be performed by the Geotechnical Consultant. Location and frequency of tests shall be at the Consultant's discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations shall be selected to verify adequacy of compaction levels in areas that are judged to be prone to inadequate compaction (such as close to slope faces and at the fill/bedrock benches).

4.6 Frequency of Compaction Testing: Tests shall be taken at intervals not exceeding 2 feet in vertical rise and/or 1,000 cubic yards of compacted fill soils embankment. In addition, as a guideline, at least one test shall be taken on slope faces for each 5,000 square feet of slope face and/or each 10 feet of vertical height of slope. The Contractor shall assure that fill construction is such that the testing schedule can be accomplished by the Geotechnical Consultant. The Contractor shall stop or slow down the earthwork construction if these minimum standards are not met.

4.7 Compaction Test Locations: The Geotechnical Consultant shall document the approximate elevation and horizontal coordinates of each test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that the Geotechnical Consultant can determine the test locations with sufficient accuracy. At a minimum, two grade stakes within a horizontal distance of 100 feet and vertically less than 5 feet apart from potential test locations shall be provided.

5.0 Subdrain Installation

Subdrain systems shall be installed in accordance with the approved geotechnical report(s), the grading plan, and the Standard Details. The Geotechnical Consultant may recommend additional subdrains and/or changes in subdrain extent, location, grade, or material depending on conditions encountered during grading. All subdrains shall be surveyed by a land surveyor/civil engineer for line and grade after installation and prior to burial. Sufficient time should be allowed by the Contractor for these surveys.

6.0 Excavation

Excavations, as well as over-excavation for remedial purposes, shall be evaluated by the Geotechnical Consultant during grading. Remedial removal depths shown on geotechnical plans are estimates only. The actual extent of removal shall be determined by the Geotechnical Consultant based on the field evaluation of exposed conditions during grading. Where fill-over-cut slopes are to be graded, the cut portion of the slope shall be made, evaluated, and accepted by the Geotechnical Consultant prior to placement of materials for construction of the fill portion of the slope, unless otherwise recommended by the Geotechnical Consultant.

7.0 Trench Backfills

7.1 Safety: The Contractor shall follow all OHSA and Cal/OSHA requirements for safety of trench excavations.

7.2 Bedding and Backfill: All bedding and backfill of utility trenches shall be done in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 (SE>30). The bedding shall be placed to 1 foot over the top of the conduit and densified by jetting. Backfill shall be placed and densified to a minimum of 90 percent of maximum from 1 foot above the top of the conduit to the surface.

The Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill.

7.3 Lift Thickness: Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to the Geotechnical Consultant that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method.

7.4 Observation and Testing: The jetting of the bedding around the conduits shall be observed by the Geotechnical Consultant.

APPENDIX E

GBA'S IMPORTANT INFORMATION ABOUT THIS GEOTECHNICAL-ENGINEERING 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, clients can benefit from a lowered exposure to the subsurface problems 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 below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs 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. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project 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.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- 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, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been 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 your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

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

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment 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 from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed 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 full-time 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 on hand quickly 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 observation.

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 informational 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, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. 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. 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 Not 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 relate any 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 yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled 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 not 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 not building-envelope or mold specialists.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

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